MOTT MACDONALD

Technical Note

Project:	Port Salford		
Our reference:	392905_TN_Geo_002	**Your reference:**	
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Approved by:	A.S. O'Brien	**Checked by:**	N. Haynes
Subject:	Pile Capacity & Settlement Analysis for Bridging Slabs		

1 Introduction

This Technical Note (TN) provides an assessment of pile and pile group behaviour within the vicinity of the proposed development area where a build-over agreement will require a future piled transfer deck over the tunnel. Ground conditions are summarised for the development site within Technical Note 392905-nh-002.

Pile design and constructability have been determined accounting for the following factors:

i. Relative phasing of piling works to tunnel works – at this stage it is unknown whether piling will precede tunnelling or vice versa, hence both cases have been analysed. In the instance of piles being in place in advance of tunnelling, sufficient lateral clearance from the proposed outer TBM diameter will be required. In this case pile design must be capable of tolerating any lateral or vertical displacements induced by the tunnel drive. In the alternative case of tunnel construction in advance of piling, pile design will require sufficient lateral clearance from the completed tunnel inner diameter. For this case, pile design must minimise vertical and lateral displacements such that the tunnel lining is not impacted.

ii. Variation in ground conditions – rockhead deepens significantly over the eastern 2/3^{rds} of the development site meaning that whilst piles over the western development area will found within Sherwood Sandstone, piles within the eastern development area will require to found within the Glacial Till.

Loads required to be carried by transfer decks have been taken from the MM Technical Note: Foundation Design for Build Over. This TN then summarises assessed individual pile capacities to determine the likely size of piles and the number required for pile groups. Pile group analyses have been undertaken to allow an assessment of pile group settlement under a column load for all design and load cases. Finally, for the design case where tunnelling precedes piling, this TN assesses the impact pile group settlement may have on the tunnel, derived from published research and empirical correlations.

For the design case where piling precedes tunnelling, a separate technical note assessing lateral displacement induced by tunnelling and hence bending moments and shear forces required to be accommodated by the piles, is provided within Technical Note: 392905_TN_Geo_003.

Figure 1: Site Plan

2 Ground and Groundwater Conditions

Geotechnical data available for the project has been digitised into ags4 format and modelled within Leapfrog Works 2.0. The source data included is detailed in MM Technical Note: 392905_TN_Geo_001, and includes project specific data sets and wider area data sets for both the Port Salford Development and the Barton Bridge M60 widening.

A long section through the tunnel alignment particular to the proposed retail development area, derived from the global data set, is provided in Figure 2 below; cross-sections through the proposed development plot are provided in Figures 3a and 3b. The proposed tunnel outline is shown on both figures.

Figure 2 clearly shows the rockhead declination across the eastern half of the development area, representing the western edge of a buried glacial valley as defined within geological mapping discussed within MM Technical Note: 392905_TN_Geo_001. Rockhead is not proven however the Barton Bridge M60 widening boreholes do record Glacial Till to depths in excess of 30m.

Figure 2: Geological long section

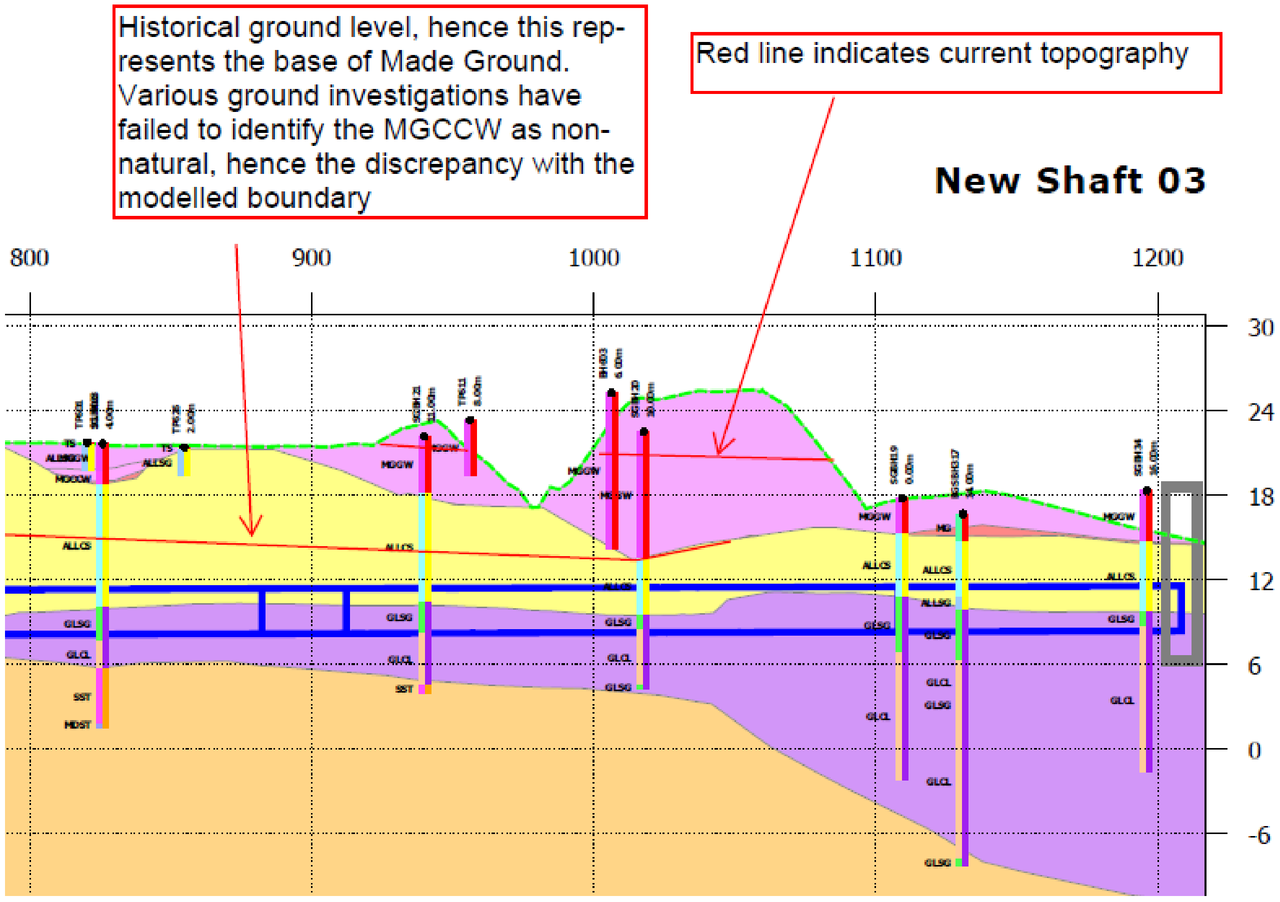


Figure 3a: Geological cross section 1

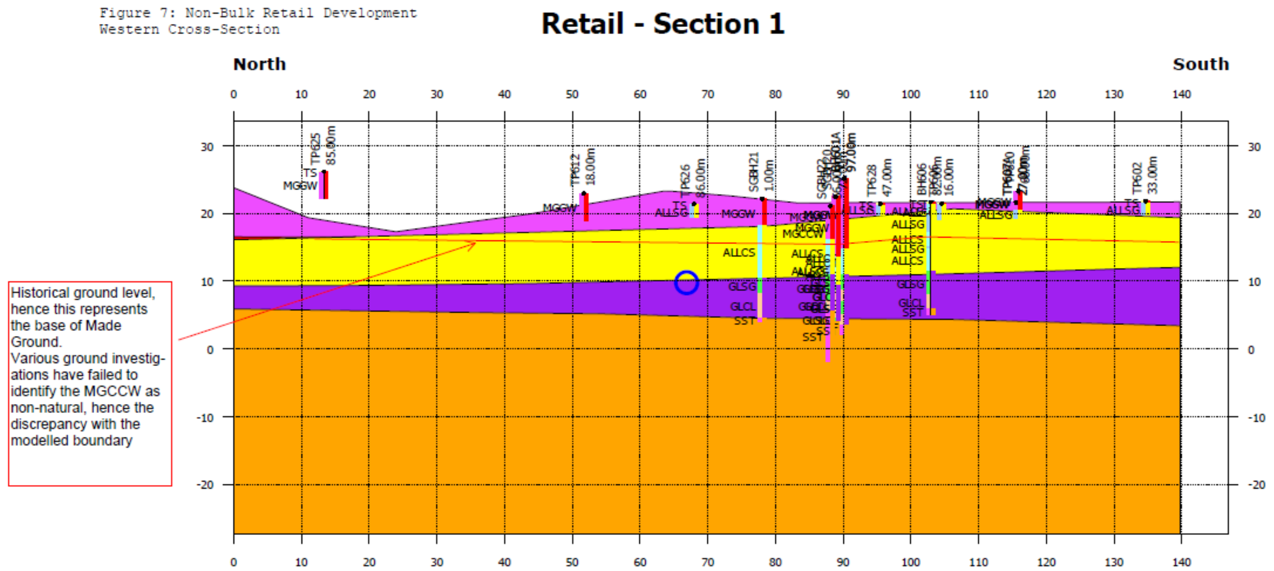
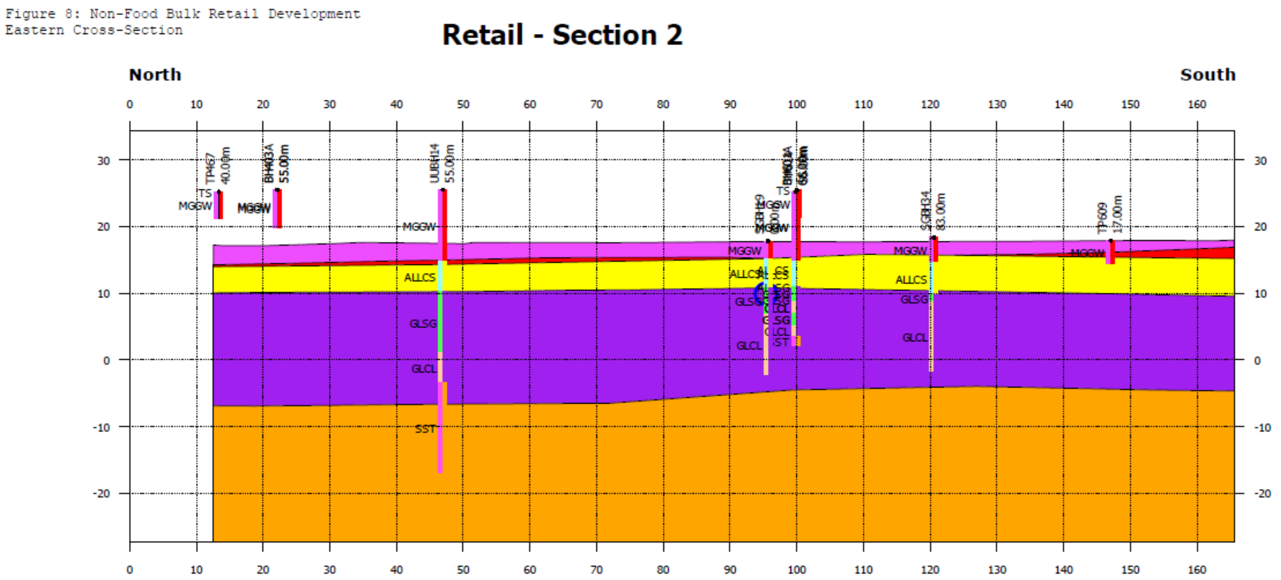


Figure 3b: Geological cross section 2



To account for potential uncertainty, particularly with regard to rockhead elevation as the development traverses the buried glacial valley margin, two ground models for design analysis situated at either end of the development traverse in line with Sections 1 and 2 were developed. These are summarised in figure 4 and figure 5.

2.1 Piling Techniques

From a piling perspective the optimal piling technique for adoption would be Continuous Flight Auger (CFA) as this offers the most cost-effective piling methodology. Within the UK standard CFA rigs can typically extend to maximum depths of the order of 25m through superficial deposits. Within Sherwood Sandstone, which is a very weak to moderately weak rock, within the Manchester area, standard CFA piling rigs can typically penetrate to 3-5m below rockhead. Successful reinforcement cage installation is a function of pile size – for small and medium diameter piles (300-750mm), cage installation is considered reliable only to depths of the order of 10m. For larger diameter piles (900mm +), deeper reinforcement is achievable, however as pile size increases, the economies of CFA piling reduce and bored piling may offer improved economies. Adoption of bored piling would likely require casing and slurry support through the water-bearing Alluvial and Glacio-fluvial Deposits.

Considering the above, the following piling techniques are considered appropriate for the varying design cases:

- Outside the Zone of Tunnel Influence (ZTI) – CFA 600mm diameter (Pile Type B1/B2)
- Inside the ZTI, Tunnel precedes piles – CFA 600mm diameter (Pile Type B1/B2)
  Note: in this instance piles accommodate structural development load only and deep reinforcement cages are not considered to be required.
- Inside the ZTI, Piles precede the tunnel – Bored with cased slurry support 1050mm diameter (Pile Type A1/A2)

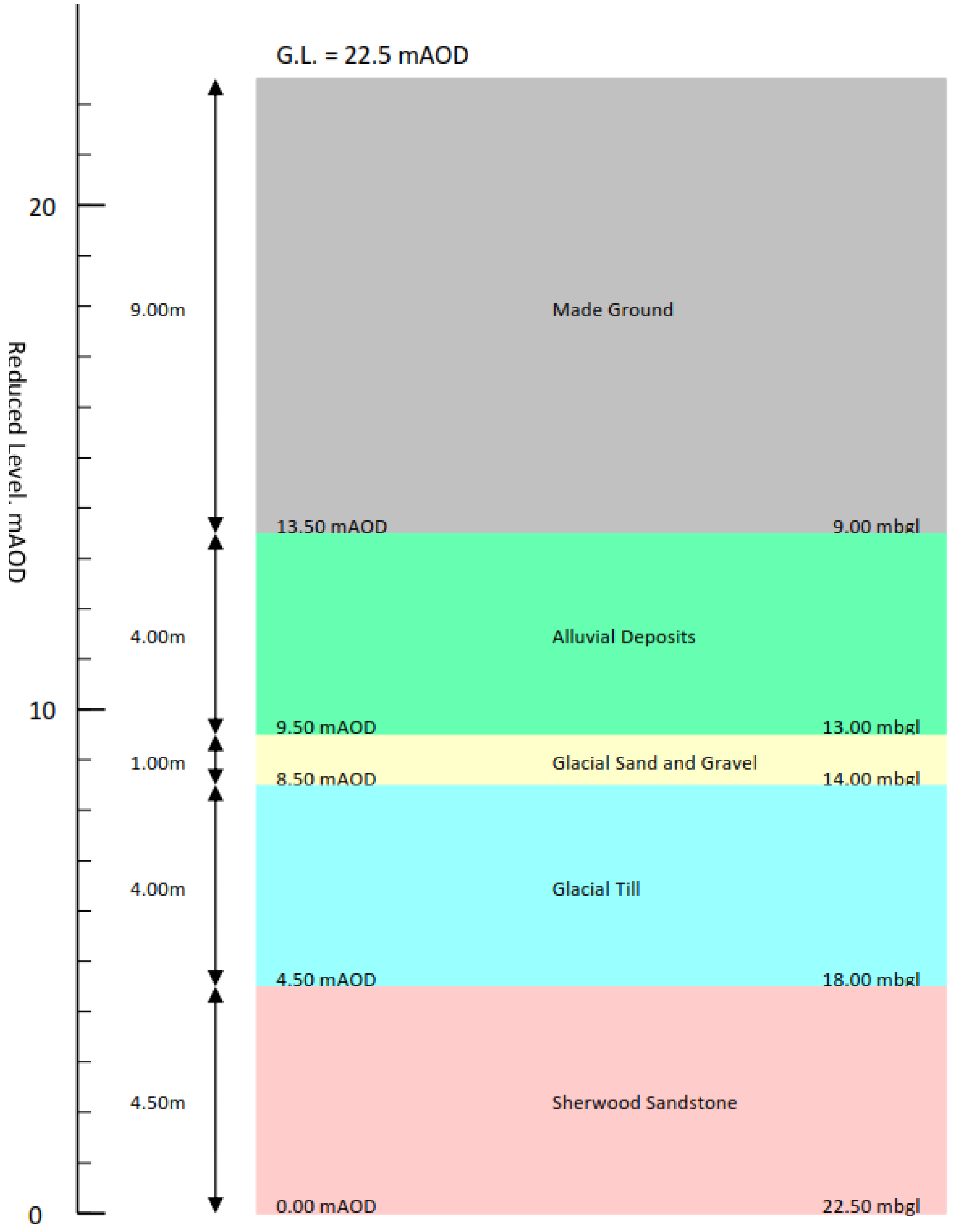
Note: in this instance piles accommodate structural development load plus lateral deformation imposed by the tunnel, hence deep reinforcement cage required to approximately 15m depth.

Table 1 identifies the different pile types proposed. Variants of pile types B and A exist to account for the differing founding conditions which are anticipated across the development and specifically the tunnel alignment due to the buried glacial valley. Design ground models are provided within Figures 4 and 5.

In order to minimise vertical settlements, piles have been designed as friction piles only. Skin friction has been assumed through the Glaciofluvial Deposits (drained) and Sandstone socket only; negative skin friction through the Made Ground and Alluvial Deposits has been discounted given the safety factor allowance provided.

<u>Non-food Bulk Retail Development – Design Ground Model (Pile founded in Sherwood Sandstone)</u>

Relevant exploratory holes: BH20, BH21, BH603, BH601A and BH 601



Note: Groundwater level is taken as 16.5mAOD

Figure 4: Design Ground Model (Pile founded in Sandstone layer)

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Non-food Bulk Retail Development – Design Ground Model (Pile founded in Glacial Till layer)

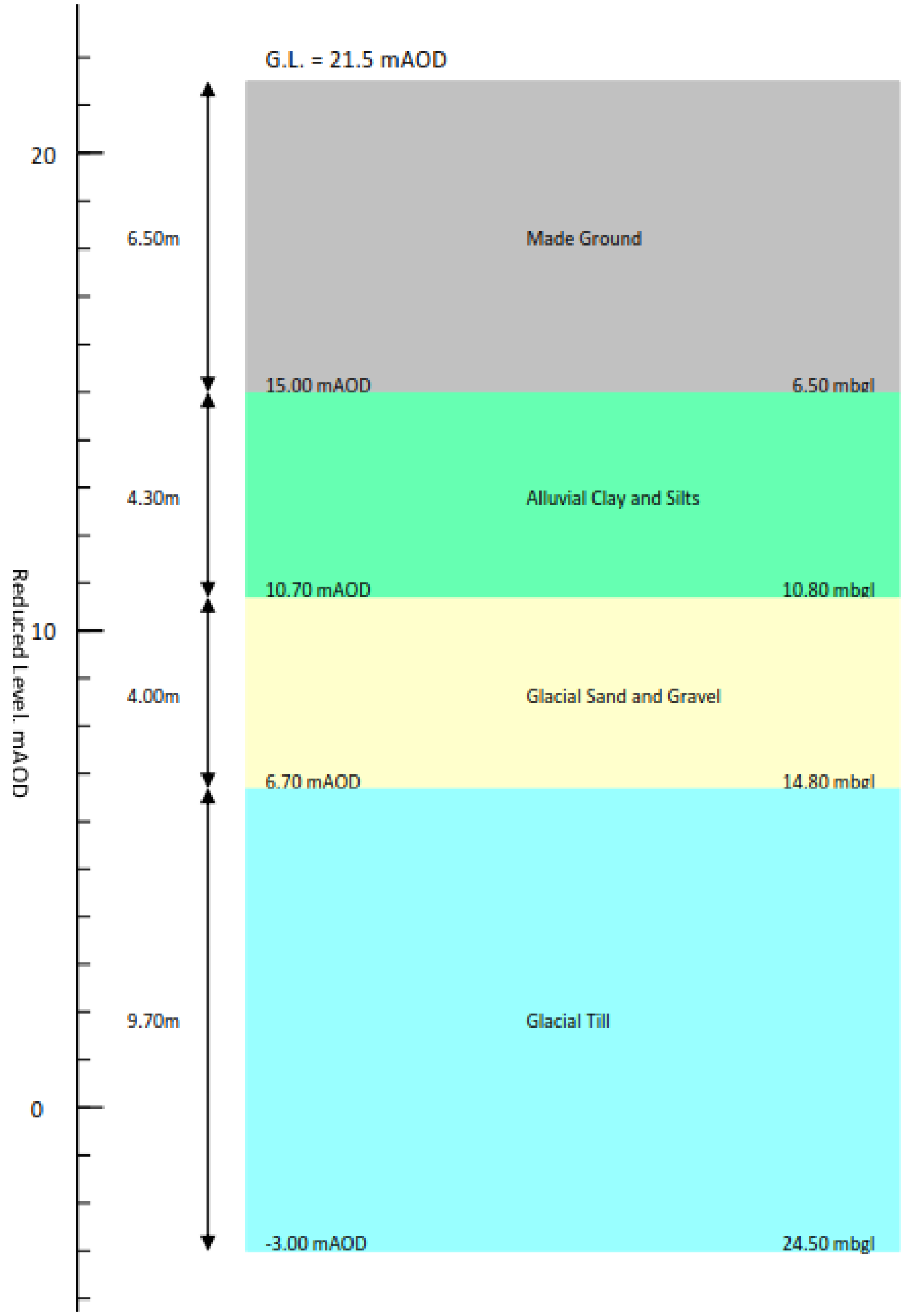


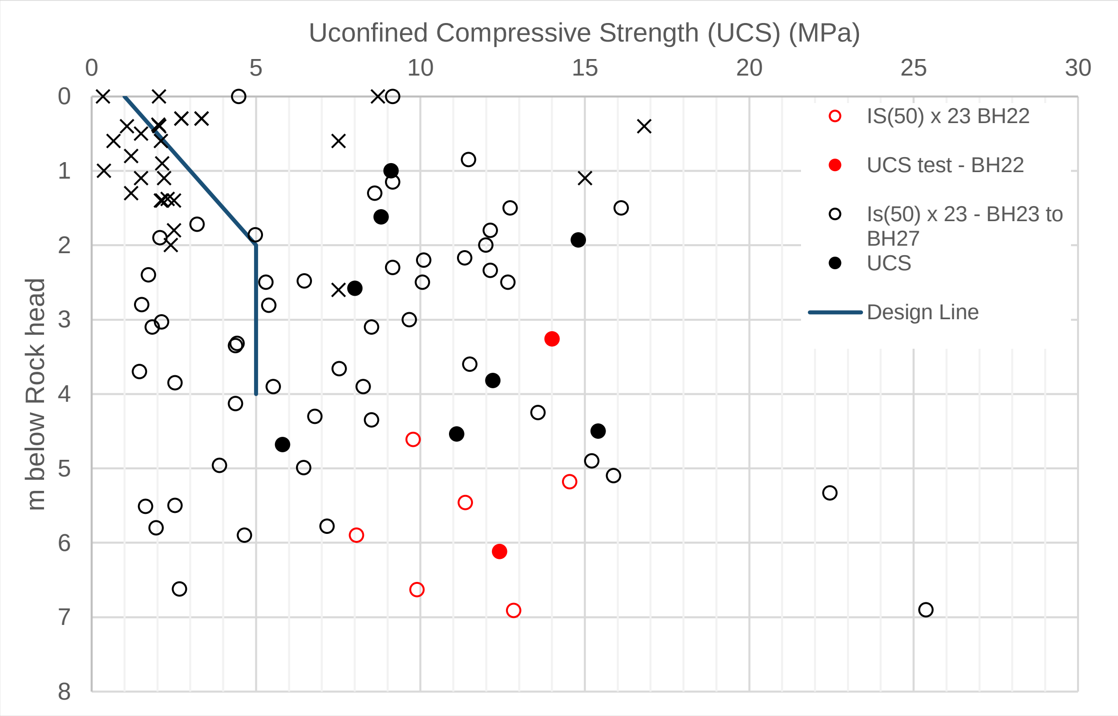
Figure 5: Design Ground Model (Pile founded in Glacial Till layer)

2.2 Geotechnical Parameters

Geotechnical parameters adopted for preliminary pile capacity assessments are as detailed in Technical Note 392905-TN-Geo-001 Table 4, save for two amendments, as outlined below:

1. Sherwood Sandstone (SST) Strength

   Figure 6 below shows the rock strength data as derived from consideration of SPT 'N' values, Unconfined Compressive Strength (UCS) and point load index. Correlations used are provided within the figure footnotes.



Note: UCS = Is(50) x 23 (MPa)
UCS = SPT 'N' /100 (MPa)

Figure 6: Unconfined Compressive Strength (UCS) (MPa)

2. Glacial Clay Strength
   Glacial Clay strength derivation for skin friction has utilised the drained strength approach as defined by

   $$q_s = \sigma_v' K \tan\delta.$$

   But has utilised the undrained shear strength for determination of end-bearing.

   As discussed within the Ground Conditions Technical Note 392905-nh-002, Glacial Clay comprises two distinct lithologies, Flow Till and Lodgement Till. The following strength parameters have been adopted for design which represent a weighting towards the Flow Till:

   c' = 0kPa; phi' = 28^0; delta =26^0 , s_u= 130kPa, OCR = 4, K = 0.8

3 Single Pile Capacity

3.1 Design Parameters and Assumption

- Pile Type: CFA or Bored
- Pile diameter: 600mm or 1050mm
- Pile length: 21m, 25m or 30m
- For geotechnical ULS design, an overall Factor of Safety of 2.5 was adopted to determine pile safe working load. (however, as outlined in this Note, for CFA piles founded in Sandstone Geotechnical ULS capacity will not govern design).
- For commonly available CFA piling rigs, an embedment of up to 3m into Sandstone is likely to be the maximum achievable, before the auger torque demands become excessive.

The ultimate shaft Resistance were calculated by:

$Q_s = \alpha \, . \, c_u \, . \, A_s$	-	for fine grained strata (Glacial Clay analysed as drained strata with maximum average skin friction capped at 110KPa).
$Q_s = K_s \, . \, \sigma'_v \, . \tan\delta \, . \, A_s$	-	for coarse grained strata
$Qs = 0.25.q_{uc}^{0.5} \, .A_s$	-	for rock

Where

- A_s is the embedded surface area of pile.
- α is adhesion factors
- Ks is earth pressure coefficient after pile installation
- $\sigma'v$ is the effective overburden pressure
- δ is angle of skin friction of pile
- q_{uc} is unconfined compression strength

The theoretical ultimate base resistance was calculated by:

$Qb=9 \, . \, C_u \, .A_b$	-	for fine grained strata
$Q_b = q_{uc} \, .A_b$	-	for rock

Where

- A_b is the area of the base of the pile.
- Cu is undrained shear strength
- q_{uc} is unconfined compression strength

The EC7 Code compliant base capacity (defined as resistance mobilised at a settlement equivalent to 10% of pile diameter), is based on a CEMSET analysis, which is equivalent to approximately 90% of UCS at 3m embedment (in practice this is largely controlled by base stiffness).

Allowable concrete stress for CFA pile concrete is assumed to be 0.25 (concrete strength, assumed as 30N/mm2).

The pile axial capacity details are summarised in Table 1.

Table 1: Pile Details

Pile Type	B1	B2	A1	A2
Pile diameter	600mm	600mm	1050mm	1050mm
Pile Type	CFA	CFA	Bored	Bored
Pile length	25m	21m	30m	21m
Founding Stratum	Glacial Till	Sandstone	Glacial Till	Sandstone
Skin friction	2427	3476	6114	6084
Base resistance	254	1414	779	4330
Ultimate pile capacity (kN)	2681	4890	6893	6500 (structural capacity)
Safe pile capacity with F.O.S of 2.5 (kN)	1072	1956	2757	4165
F.O.S on Shaft capacity	-	1.85 >1.3	-	Min 1.46 >1.3

4 Single Pile and Pile Group Settlement

4.1 Single Pile Settlement

The CEMSET pile analysis was used to determine the single pile settlement. The details and assumptions are summarised below.

Concrete

- Concrete Strength =30N/mm^2
- Ec=30 x 10^6 kN/m^2

4.1.1 Base Stiffness (Sherwood Sandstone)

An assessment of the intrinsic sandstone stiffness of the pile base has been made based on the empirical correlations outlined below:.

- Method 1 - Whiteworth and Turner: Em = 275 $q_u^{0.5}$ (Ref: Rock Socket piles in Sherwood Sandstone of Central Birmingham)
- Method 2 – Tomlinson: Em=j.Mr.q_u (Ref: Foundation Design and Construction 5th edition, P144-146)

Table 2: Modulus ratio

		Values for M_r
Group 1	Pure limestones and dolomites Carbonate sandstones of low porosity	600
Group 2	Igneous Oolitic and marly limestones Well-cemented sandstones Indurated carbonate mudstones Metamorphic rocks including slates and schists (flat cleavage/foliation)	300
Group 3	Very marly limestones Poorly cemented sandstones Cemented mudstones and shales Slates and schists (steep cleavage/foliation)	150
Group 4	Uncemented mudstones and shales	75

Source: M.J. Tomlinson, Foundation Design and Construction 5th edition, P145 - 146

Table 3: Mass factor values

Table 2.7 Mass factor values

Quality classification[1]	*RQD* (%)	*Fracture frequency per metre*	*Velocity index*[2] $(V_f/V_L)^2$	*Mass factor* (*j*)
Very poor	0–25	15	0–0.2	0.2
Poor	25–50	15–8	0.2–0.4	0.2
Fair	50–75	8–5	0.4–0.6	0.2–0.5
Good	75–90	5–1	0.6–0.8	0.5–0.8
Excellent	90–100	1	0.8–1.0	0.8–1.0

Source: M.J. Tomlinson, Foundation Design and Construction 5th edition, Table 2.6 Mass factor values

- Method 3 – Rowe and Armitage 1984: Em = 215 $q_u^{0.5}$ (Ref: Wei Dong Guo, Theory and practice of pile design)

The CEMSET analytical method for single pile settlement has been developed from back analysis of several thousand pile tests in a wide range of ground conditions. The mobilised base stiffness for use in CEMSET is a function of both the intrinsic ground stiffness and the condition of the pile base (which is mainly a function of piling method and technique). Experience indicates that the mobilised base stiffness, in Sandstone, is of the order of between 150 and 250 MN /m2. The selected design line for base stiffness for use in CEMSET is shown in Figure 6.

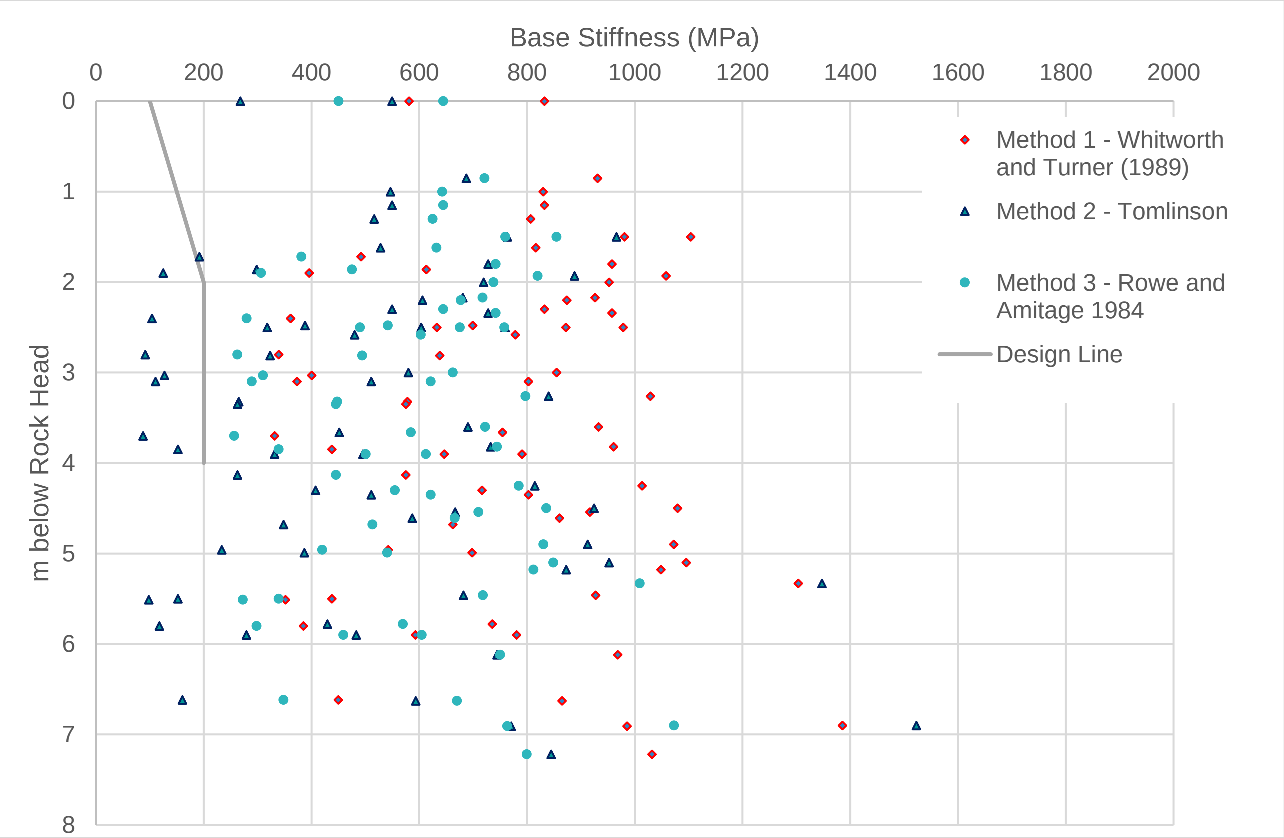


Figure 7: Base stiffness plot

To ensure single pile settlement is small (< 1% of pile diameter) the pile working load should be kept to about three-quarters of the shaft capacity (ie a partial factor of 1.3 applied on shaft capacity, ICE Manual of Geotechnical Engineering, Chapter 56, Figure 56.18). For a 600mm CFA pile embedded 3m into sandstone the factored shaft resistance is estimated to be about 2600 kN. Note -the proposed working load of 1860 kN (refer to Table 5) represents only about 70 % of this resistance, hence single pile settlement of less than 6mm should be anticipated.

4.1.2 CEMSET analysis

For the CEMSET analysis, the base sttiffness of 100+50z and maximum of 200 MPa was adopted. (Note: z= below rock head)

4.1.2.1 Single Pile settlement - Non-food Bulk Retail Development (Pile Type: B2)

The following parameters was used for the CEMSET analysis, and the force-displacement curve is presented in Figure 8 below.

Shaft Capacity	3.48MN
Base Capacity	1.41MN
Base Stiffness	200MN/m^2
L_o	9
L_f	12
K_e	0.45
E_c	30000MN/m^2
Ms	0.004

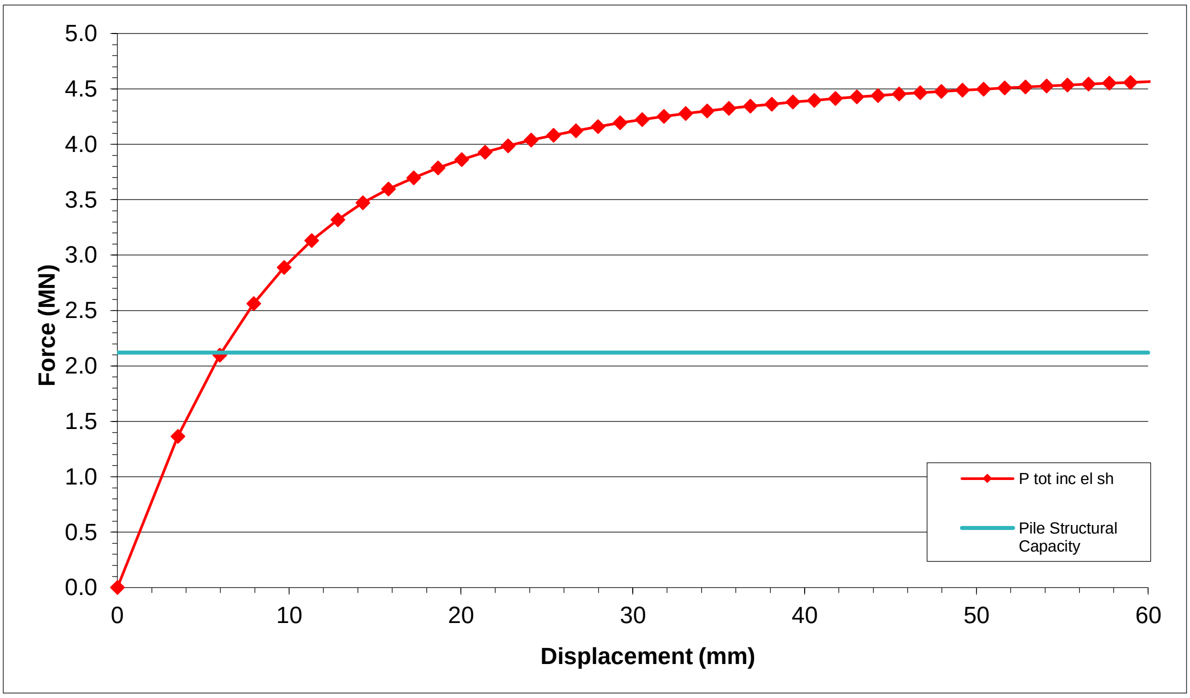


Figure 8: CEMSET analysis

<u>**The single pile settlement from the CEMSET analysis at a working load of 1860 kN per pile is about 5mm , consistent with using less than 75% of the available shaft resistance.**</u>

4.1.2.2 Single Pile settlement - Non-food Bulk Retail Development (Pile Type: B1)

The following parameters was used for the CEMSET analysis, and the force-displacement curve is presented in Figure 9 below.

Shaft Capacity	2.43MN
Base Capacity	0.254MN
Base Stiffness	15MN/m^2
L_o	6.5
L_f	18.5
K_e	0.45
E_c	30000MN/m^2
Ms	0.002

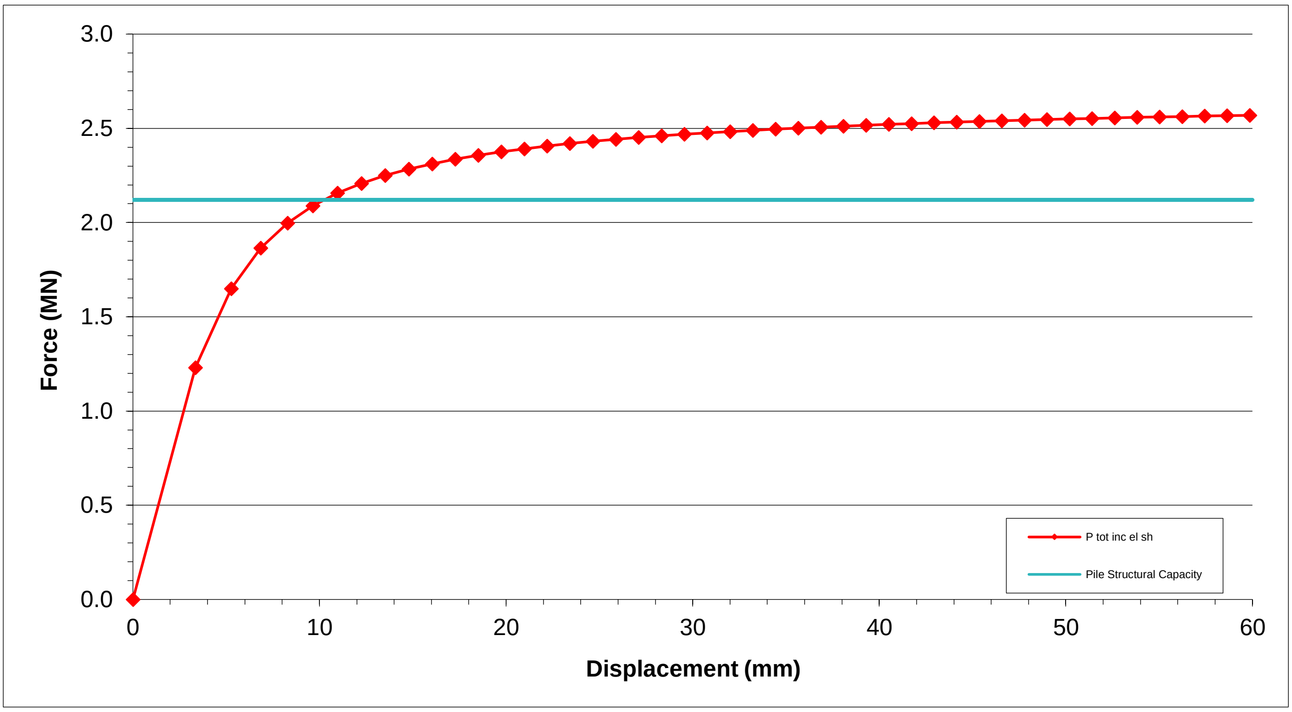


Figure 9: CEMSET analysis

4.1.2.3 Single Pile settlement - Non-food Bulk Retail Development (Pile Type: A1)

The following parameters was used for the CEMSET analysis, and the force-displacement curve is presented in Figure 10 below.

Shaft Capacity	6.11MN
Base Capacity	0.779MN
Base Stiffness	40MN/m^2
L_o	6.5
L_f	23.5
K_e	0.45
E_c	30000MN/m^2
Ms	0.002

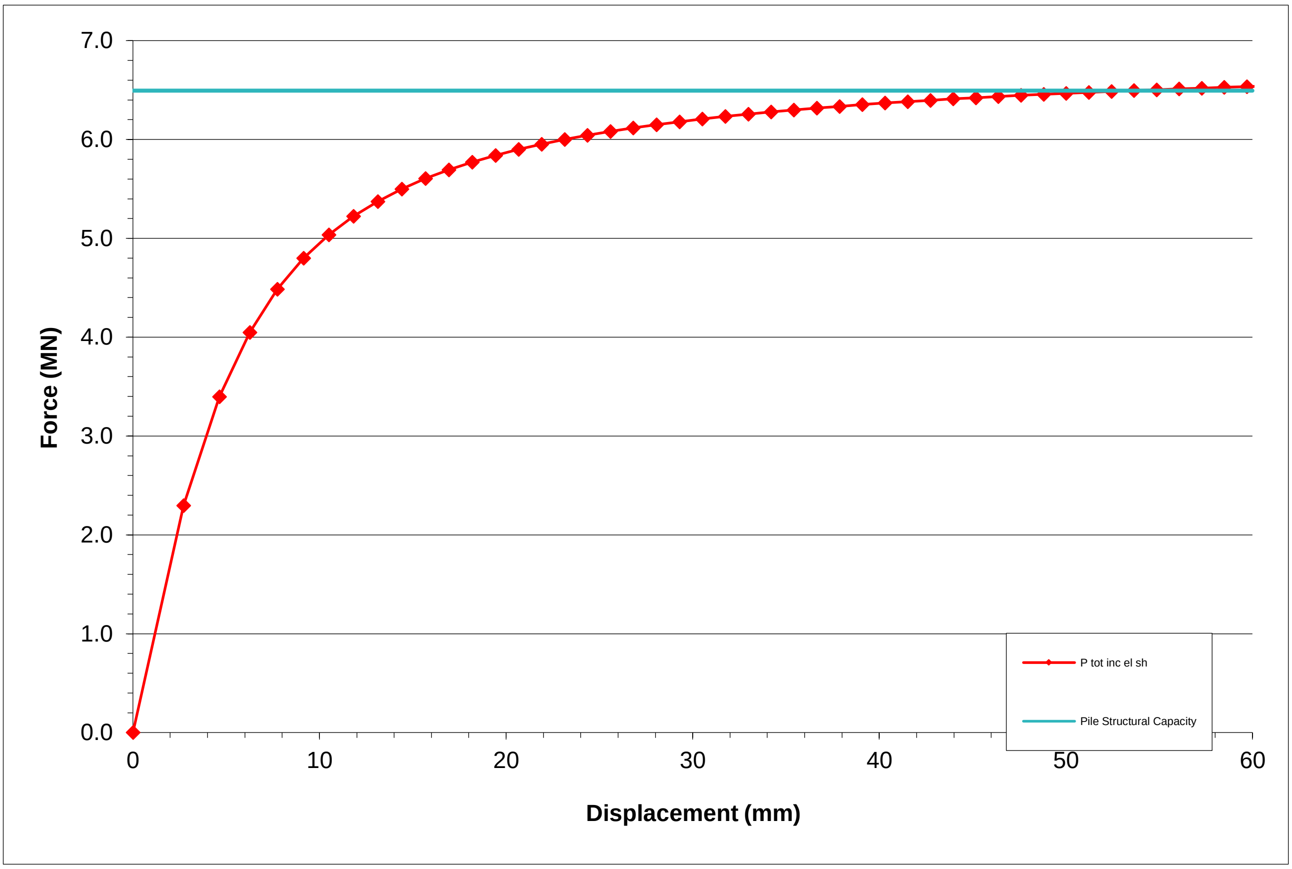


Figure 10: CEMSET analysis

4.1.2.4 Single Pile settlement - Non-food Bulk Retail Development (Pile Type: A2)

The following parameters was used for the CEMSET analysis, and the force-displacement curve is presented in Figure 11 below.

Shaft Capacity	6.084MN
Base Capacity	4.33MN
Base Stiffness	200MN/m^2
L_o	9
L_f	12
K_e	0.45
E_c	30000MN/m^2
Ms	0.004

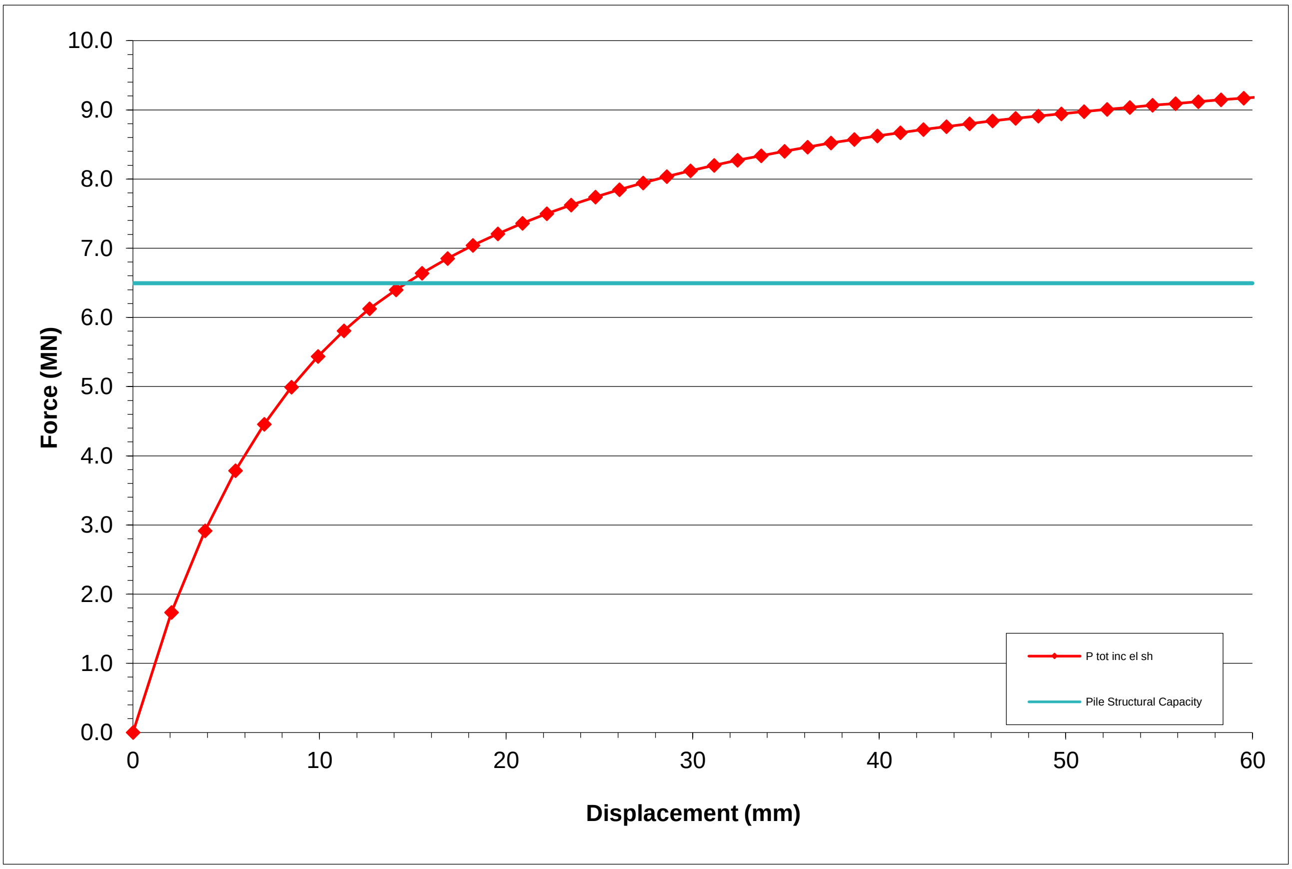


Figure 11: CEMSET analysis

4.1.3 Repute analysis - Single Pile

Pile group behaviour was analysed within Repute. The Repute Model was undertaken using a full non-linear hyperbolic analysis adopting a shaft non-linearity (R_{fs}) of 0.75 and a base non-linearity (R_{fb}) of 0.999. Other input parameters were as detailed in Table 4.

Table 4: Ground parameters

Strata	Bulk Density (kN/m³)	Cu (kPa)	c'	ϕ' (°)	Eo (MPa)	v
Made Ground	17	-	-	-	10	0.3
Alluvial Deposits	17.5	30	-	26	7.5	0.4
Glacial Sand and Gravel	20	-	0	32 to 36	25	0.3
Glacial Till	20	100	0	28	200 +10z	0.25
Sherwood Sandstone	21	UCS – 1+2zMPa (max 5MPa)	20	40	300 + 100z (max 500)	0.2

Note: z – depth below top of strata

4.1.3.1 Pile Type B2 – Founding stratum at Sherwood Sandstone

Figure 12 below compares the Repute analysis with the CEMSET analysis, which indicates an excellent match across the working load range ,ie pile working load will be less than 2000 kN

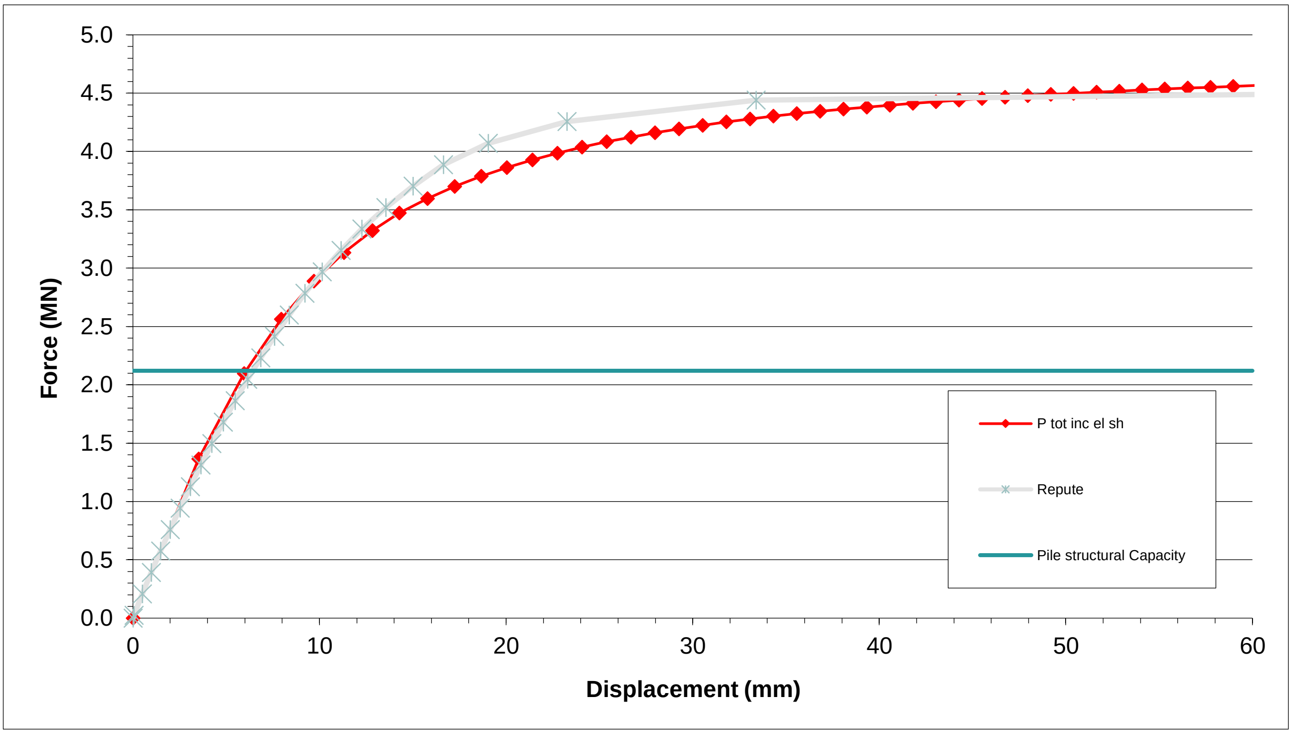


Figure 12: Repute analysis compared to CEMSET analysis (Pile Type B2)

4.1.3.2 Pile Type A2 – Founding stratum at Sherwood Sandstone

Figure 13 below compares the Repute analysis with the CEMSET analysis, which indicates an excellent match across the working load range ,ie pile working load will be less than 4200 kN

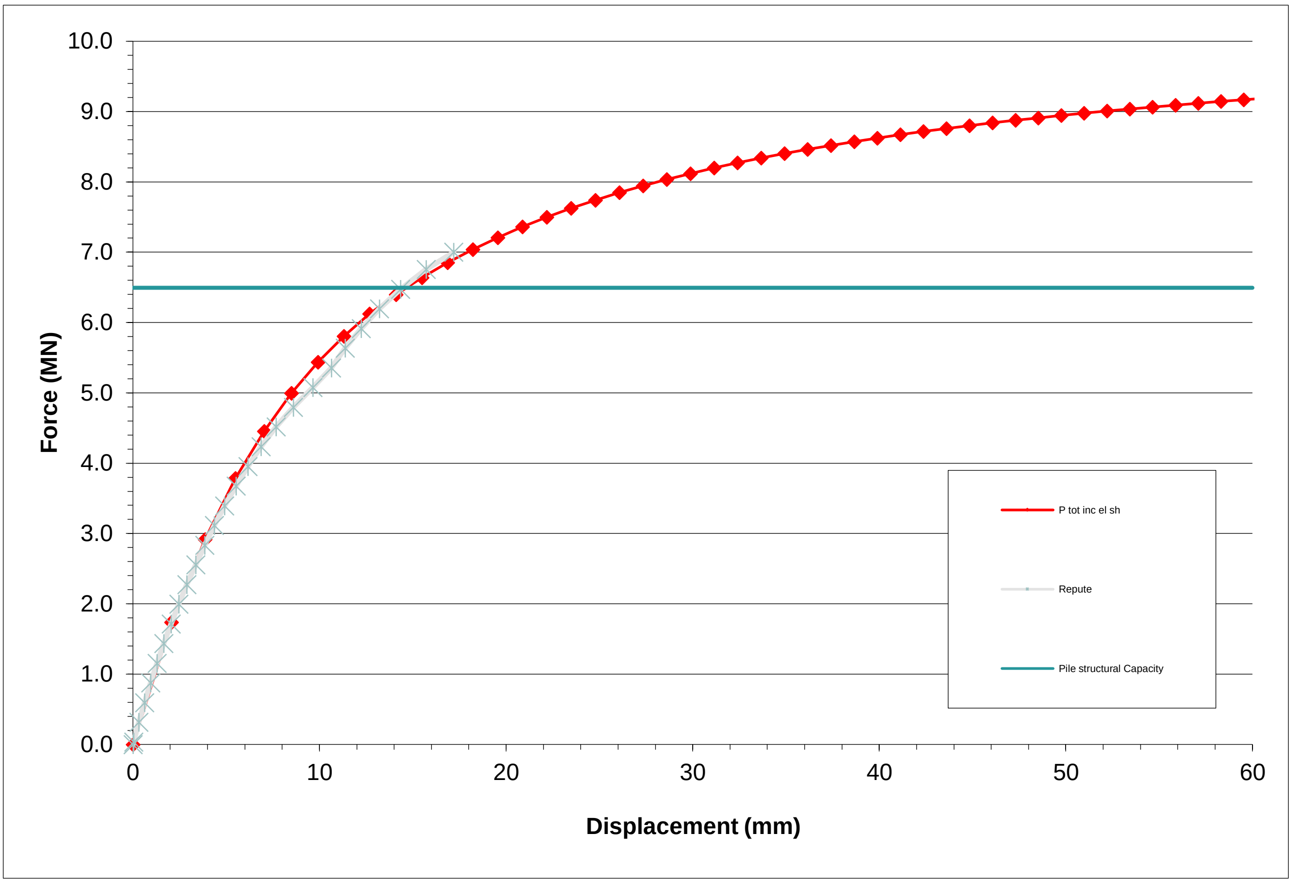


Figure 13: Repute analysis compared to CEMSET analysis (Pile Type A2)

4.1.3.3 Pile Type A1 – Founding stratum at Glacial Till

Figure 14 below compares the Repute analysis with the CEMSET analysis, which indicates an excellent match across the working load range ,ie pile working load will be less than 2800 kN

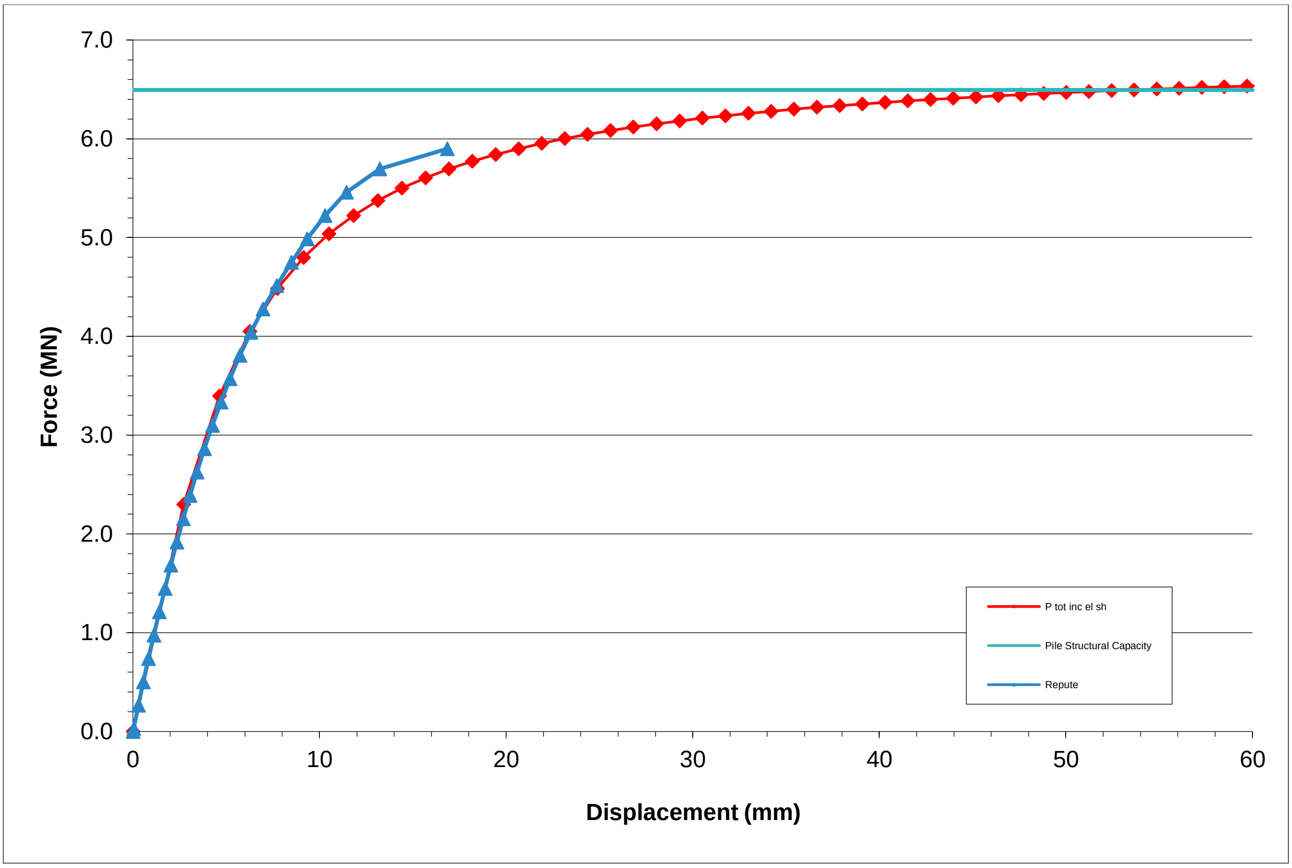


Figure 14: Repute analysis compared to CEMSET analysis – Pile Type A1 (1050mm dia. Bored pile)

4.1.3.4 Pile Type B1 – Founding stratum at Glacial Till

Figure 15 below compares the Repute analysis with the CEMSET analysis, which indicates an excellent match across the working load range ,ie pile working load will be less than 1100 kN

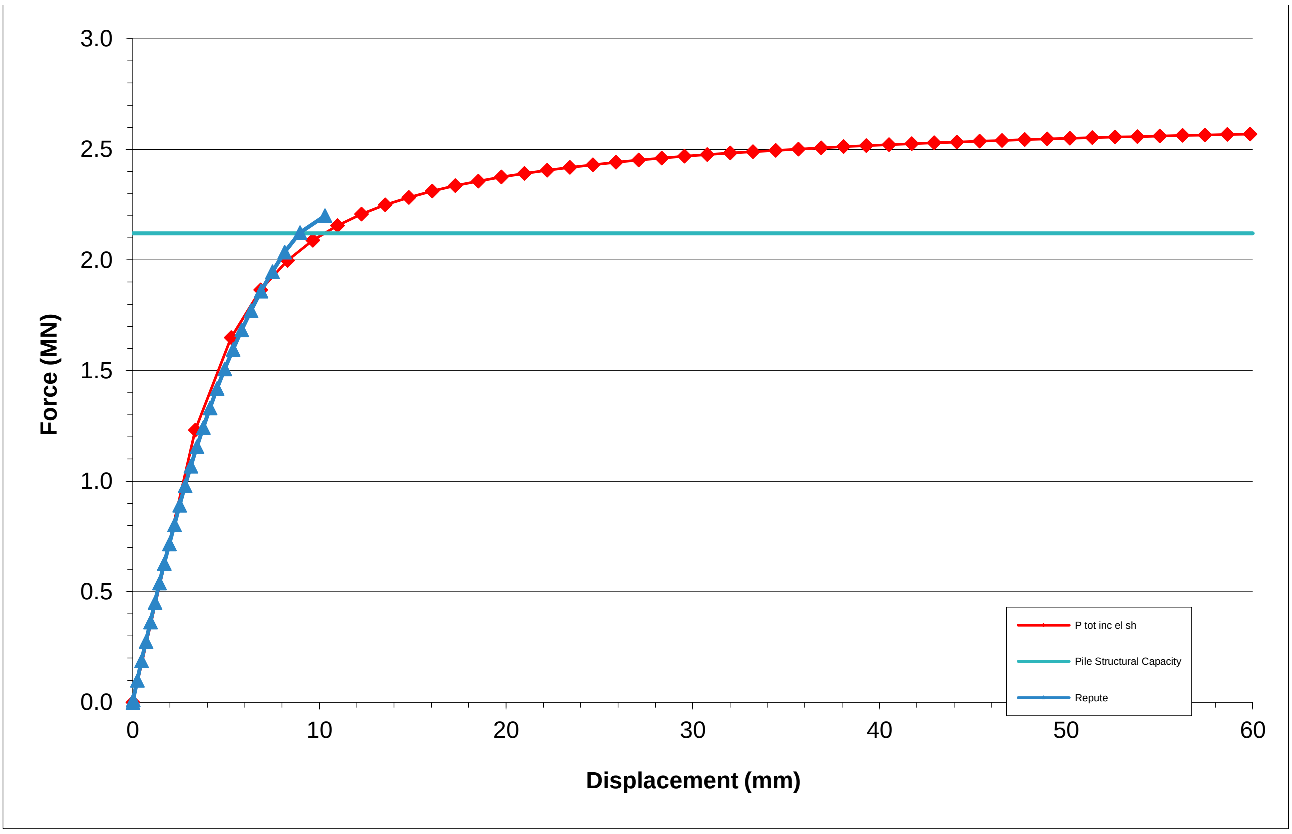


Figure 15: Repute analysis compared to CEMSET analysis – Pile Type B1 (600mm dia. CFA pile)

4.2 Pile Group Settlement

The proposed pile layout and details are listed below.

Pile Type: B1

- Pile diameter: 600mm
- Pile length: 25m
- Pile Type: CFA
- Pile Spacing: 3D (1.8m)
- Allowable Concrete Stress = $1072/(0.3^2.\pi)$ = 3791kPa < 0.25fcu (-0.25 x 30000)=7500kPa
- Maximum capacity based on allowable concrete stress = $7500 \times 0.3 \times 0.3 .\pi$ = 2120kN

Figure 16: Pile Cap Type B1 - 1

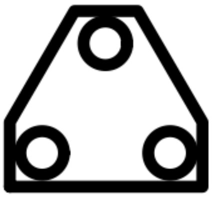


Figure 17: Pile Cap Type B1 - 2

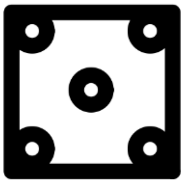


Figure 18: Pile Cap Type B1 - 3

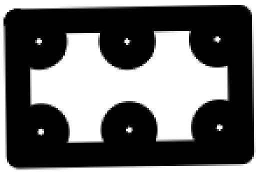


Figure 19: Pile Cap Type B1 - 4

Table 5: Pile Group details

Pile Cap Type	Max. Column load	No. of Pile	Nominal Pile Load within Group
Pile Cap Type B1 - 1	2144kN	2	1072kN
Pile Cap Type B1 - 2	3216kN	3	1072kN
Pile Cap Type B1 - 3	5360kN	5	1072kN
Pile Cap Type B1 – 4	6432kN	6	1072kN

Pile Type: B2

- Pile diameter: 600mm
- Pile length: 21m
- Pile Type: CFA
- Pile Spacing: 3D (1.8m)
- Allowable Concrete Stress = $1875/(0.3^2.\pi)$ = 6631kPa < 0.25fcu (-0.25 x 30000)=7500kPa
- Maximum capacity based on allowable concrete stress = 7500 x 0.3 x 0.3 .π = 2120kN

Table 5 provides vertical column loads as advised within MM Technical Note: 392905-sdp-001; apportioned pile numbers and resultant nominal pile load within the group. Figures 9-12 display typical pile cap configurations adopting a minimum 3D pile spacing.

Table 6: Pile Group details

Pile Cap Type	Max. Column load	No. of Pile	Nominal Pile Load within Group
Pile Cap Type B2 -1	3912kN	2	1956kN
Pile Cap Type B2 -2	5868kN	3	1956kN
Pile Cap Type B2 -3	7824kN	4	1956kN

Figure 20: Pile Cap Type B2 - 1

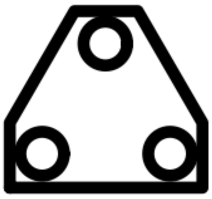


Figure 21: Pile Cap Type B2- 2

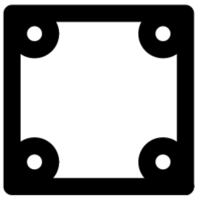


Figure 22: Pile Cap Type B2 - 3

<u>Pile Type: A1</u>

- Pile diameter: 1050mm
- Pile length: 30m
- Pile Type: Bored
- Pile Spacing: 3D (1.8m)
- Allowable Concrete Stress = $2757/(1.05^2.\pi/4)$ = 3184kPa < 0.25fcu (0.25 x 30000)=7500kPa
- Maximum capacity based on allowable concrete stress = 7500 x 0.525 x 0.525 .π = 6494kN

Figure 23: Pile Cap Type A1 - 1

Figure 24: Pile Cap Type A1 - 2

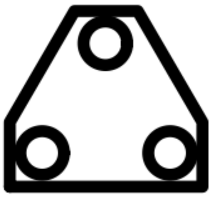


Figure 25: Pile Cap Type A1 - 3

Table 7: Pile Group details

Pile Cap Type	Max. Column load	No. of Pile	Nominal Pile Load within Group
Pile Cap Type A1 -1	2757kN	1	2757kN
Pile Cap Type A1 -2	5500kN	2	2750kN
Pile Cap Type A1 -3	7000kN	3	2334kN

Pile Type: A2

- Pile diameter: 1050mm
- Pile length: 21m
- Pile Type: Bored
- Pile Spacing: 3D (3.15m)
- Allowable Concrete Stress = $4165/(1.05^2.\pi/4)$ = 4810kPa < 0.25fcu (0.25 x 30000)=7500kPa
- Maximum capacity based on allowable concrete stress = 7500 x 0.525 x 0.525 $.\pi$ = 6494kN

Figure 26: Pile Cap Type A2 - 1

Figure 27: Pile Cap Type A2 - 2

Table 8: Pile Group details

Pile Cap Type	Max. Column load	No. of Pile	Nominal Pile Load within Group
Pile Cap Type A2 -1	4165kN	1	4165kN
Pile Cap Type A2 -2	7000kN	2	3500kN

4.2.1 Empirical method

To check the Repute analysis, an empirical method, as outlined below, was used to determine the pile group settlement.

Pile-group settlement, $W=R_{se}.W_s$ (Ref: ICE Manual Of Geotechnical Engineering (2012) Equation 55.10)

Where

- W_s – Single pile settlement
  - Pile Type B2 - approx. 5.5mm at nominal load of 1956kN
  - Pile Type B1 – approx. 2.8mm at nominal load of 1072kN
  - Pile Type A1 – approx. 2.6mm at nominal load of 2757kN
  - Pile Type A2 – approx.. 6.5mm at nominal load of 4165kN
- n is the number of piles in the group,
- The pile group aspect ratio $R=(ns/L)^{0.5}$ (Ref: ICE Manual Of Geotechnical Engineering (2012) Section 55.5.2)
- Lower bound $R_{se} = \frac{0.17}{R^{1.35}}(n)$ (Ref: ICE Manual Of Geotechnical Engineering (2012) Equation 55.12)

Table 9: Empirical Method calculation

Pile Type	Pile Cap Type	Pile Group aspect ratio R=(ns/L)^0.5	Lower bound Rse Rse=0.17· n /R1.35	Pile Group Settlement (mm)
600mm diameter CFA pile , Founding Stratum: Glacial Till, , Pile length: 25m				
B1	Pile Cap Type B1 -1 2 piles	-	-	-
B1	Pile Cap Type B1 - 2 3 piles	0.47	1.44	4.01
B1	Pile Cap Type B1 - 3 5 piles	0.60	1.70	4.74
B1	Pile Cap Type B1 - 4 6 piles	0.66	1.80	5.03
600mm diameter CFA pile , Founding Stratum: Sandstone, Pile length: 21m				
B2	Pile Cap Type B2 - 1 2 piles	-	-	-
B2	Pile Cap Type B2 - 2 3 piles	0.51	1.28	7.01
B2	Pile Cap Type B2 - 3 4 piles	0.59	1.40	7.70
1050mm diameter Bored pile , Founding Stratum: Glacial Till, Pile length: 30m				
A1	Pile Cap Type A1 - 1 2 piles	-	-	-
A1	Pile Cap Type A1- 2 3 piles	0.56	1.11	2.89
1050mm diameter Bored pile , Founding Stratum: Sandstone, Pile length: 25m				
A2	Pile Cap Type A2 - 1 1 pile	-	-	-
A2	Pile Cap Type A2 - 2 2 piles	-	-	-

4.2.2 Repute

The software 'Repute' was used to determine the pile group settlement. The results are shown in Figure 27 and 28 below.

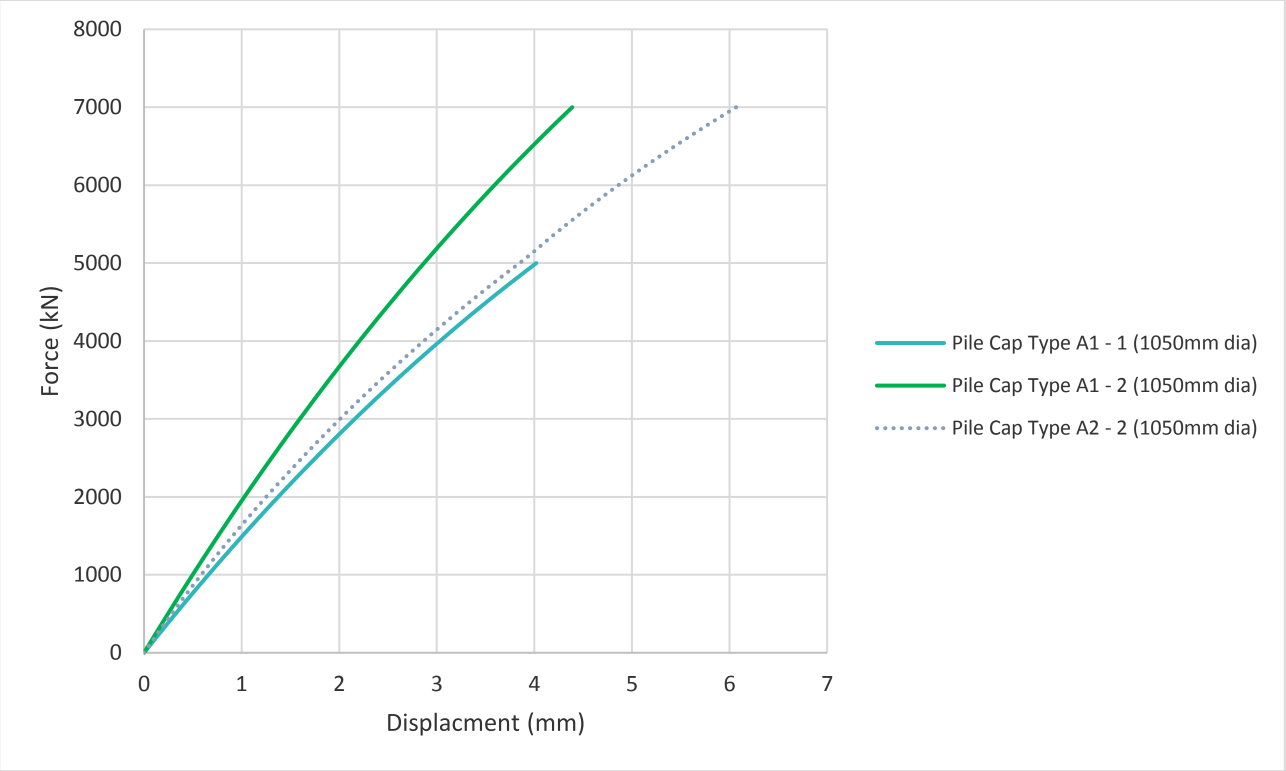


Figure 28: Repute Pile Group Settlement (Pile Type A1 and A2, 1050mm dia. Pile)

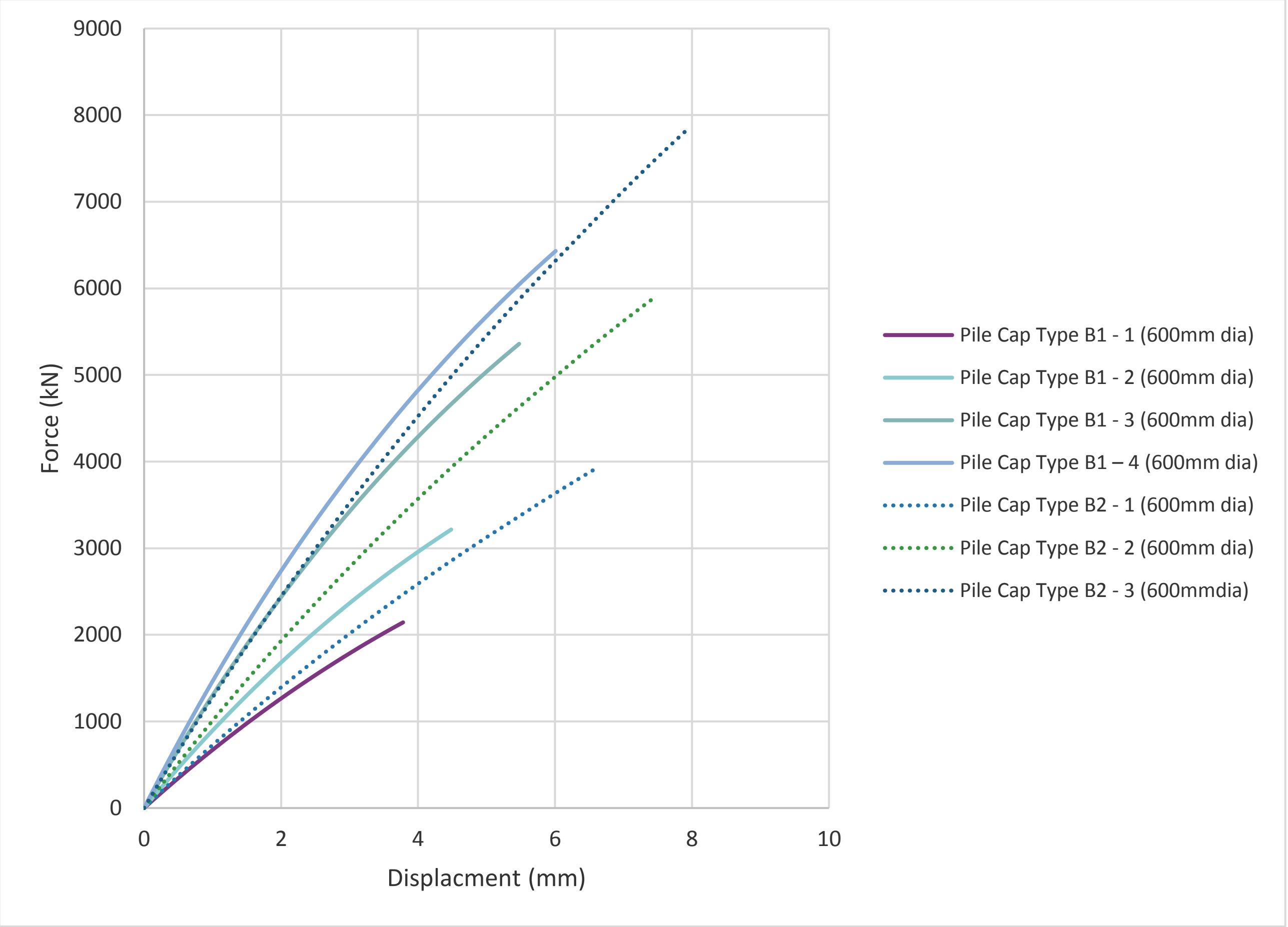


Figure 29: Repute Pile Group Settlement (Pile Type B1 and B2, 600mm dia. Pile)

The results are summarised in Table 10.

Table 10: Results – Pile Group settlement

Pile Type	Pile Cap Type	Pile Group settlement (Empirical method)	Pile Group Settlement (Repute)
600mm diameter CFA pile , Founding Stratum: Glacial Till, , Pile length: 25m			
B1	Pile Cap Type B1 -1 2 piles	-	3.78mm
B1	Pile Cap Type B1 - 2 3 piles	4.01mm	4.48mm
B1	Pile Cap Type B1 - 3 5 piles	4.74mm	5.48mm
B1	Pile Cap Type B1 - 4 6 piles	5.03mm	6.01mm
600mm diameter CFA pile , Founding Stratum: Sandstone, Pile length: 21m			
B2	Pile Cap Type B2 - 1 2 piles	-	6.16mm
B2	Pile Cap Type B2 - 2 3 piles	7.01mm	6.87mm
B2	Pile Cap Type B2 - 3 4 piles	7.70mm	7.35mm
1050mm diameter Bored pile , Founding Stratum: Glacial Till, Pile length: 30m			
A1	Pile Cap Type A1 - 1 2 piles	-	4.0mm
A1	Pile Cap Type A1- 2 3 piles	2.89mm	4.3mm
1050mm diameter Bored pile , Founding Stratum: Sandstone, Pile length: 21m			
A2	Pile Cap Type A2 - 1 1 pile	6.5mm (Single Pile)	-
A2	Pile Cap Type A2 - 2 2 piles (Max column =7000kN)	-	6.32mm

Note: The above predicted settlements are for comparison purposes. They should be rounded and are indicative of an order of magnitude of settlement (i.e. <10mm)

Based upon Table 11, both the empirical method and REPUTE give consistent predictions of pile group settlements.

5 Pile / Tunnel interaction

5.1 Displacements due to piling post tunnel Construction

The empirical methods (Ref: ICE Manual Of Geotechnical Engineering (2012) (MOGE) Figure 55.12 and Figure 56.28, reproduced here as Figure 14) was used to determine the ground movement due to the pile settlement. Details are summarised below within Table 8. Based on the Section 4 results, the pile group settlement under the maximum column load is likely to be of the order of 10mm.

Parameters

- Pile diameter: 600mm or 1050mm
- Pile radius (r_0) = 300mm or 525mm
- Soil Type: non-linear soil
- Offset distance between pile and tunnel = r (see table below)
- Assume pile group settlement of 10mm.

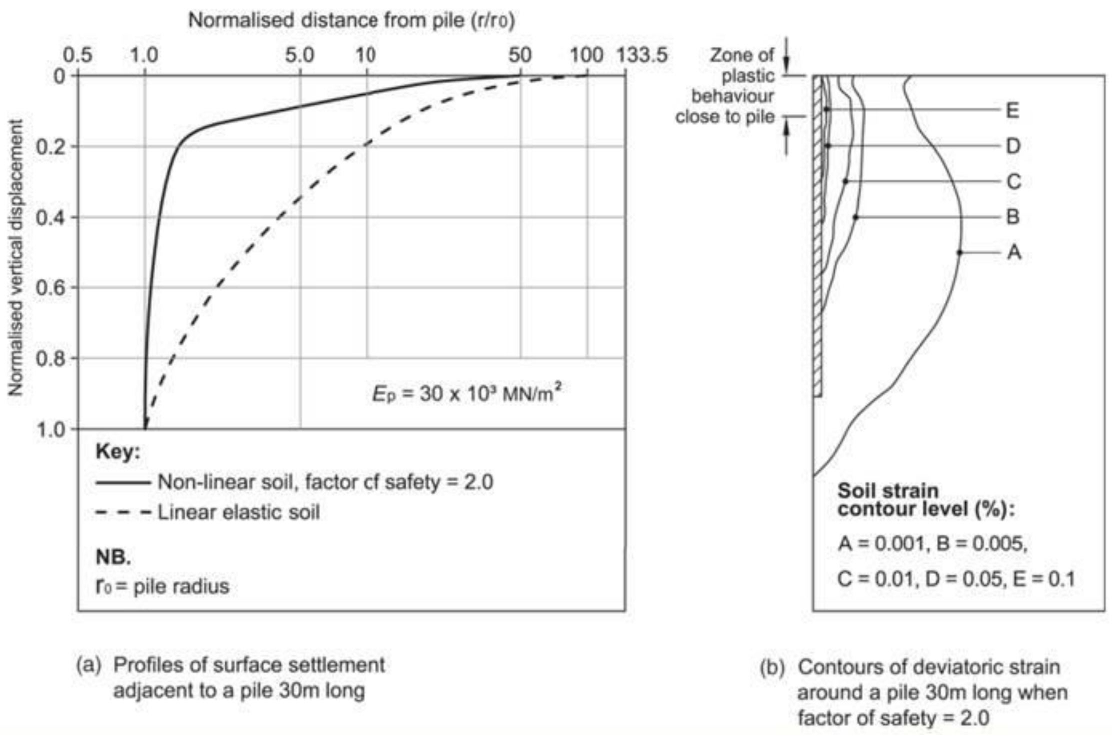


Figure 30: Influence of soil nonlinearity on pile-to-pile interaction

Source: ICE Manual Of Geotechnical Engineering (2012) Figure 55.12

Table 11: Settlement Calculation

r/r_0	r (m)	δr/δc	Settlement (mm)
600mm dia. CFA pile			
5	1.5	0.1	1
10	3	0.05	0.5
20	6	0.03	0.03
1050mm dia. bored pile			
5	2.625	0.1	1
10	5.25	0.05	0.5
20	10.5	0.03	0.03

Note: The above predicted settlements are for comparison purposes. They should be rounded and are indicative of an order of magnitude of settlement (ie. <10mm)

As a secondary check, Figure 56.28 within the MOGE, reproduced here as Figure 15, provides vertical shear stress contours within a linear elastic perfectly plastic soil, corresponding to vertical settlement around a central axis of 50mm. At 5D vertical shear stress increase has degraded to 10%, whereas at 10D vertical shear stress increase has degraded to 3.5%. These values are of similar magnitudes to those determined within Table 6, hence settlement at the proposed tunnel may be anticipated to be <1mm for both the 3D and 6D span arrangement.

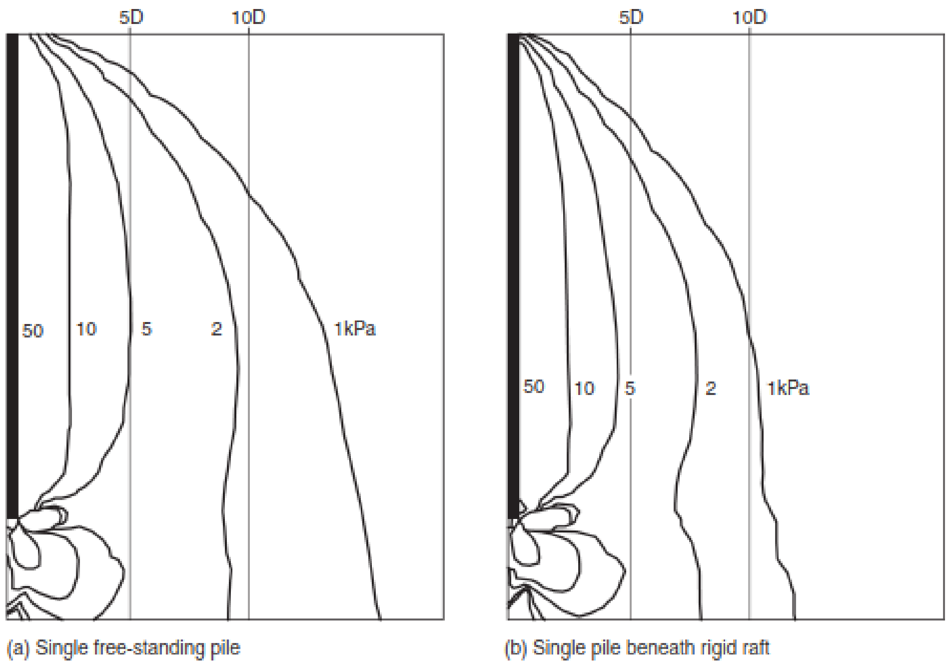


Figure 56.28 Contours of vertical shear stress corresponding to 50 mm settlement for a linear elastic-perfectly plastic soil with stiffness increasing with depth
Modified from Burland (1995), all rights reserved

Figure 31: Contours of vertical shear stress corresponding to 50mm settlement for linear elastic-perfectly plastic soil with stiffness increasing with depth

Source: ICE Manual Of Geotechnical Engineering (2012 Figure 56.28