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Seawater intake riser and brine linear outfall diffuser pile design and impacts from tunnel construction

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ABSTRACT: Stage 1 of the Alkimos Seawater Desalination Plant (ASDP) is currently being designed and constructed by the Alkimos SeaWater Alliance (ASWA). This paper provides an overview of the geotechnical interpretation for analysis and design of the piled foundations for the Seawater Intake Riser (SIR) and Brine Linear Outfall Diffuser (BLOD) structures, and the potential impact from Tunnel Boring Machine (TBM) excavations from tunnels. Piled foundations comprise driven steel tubes through calcareous soil deposits, which pose known issues and challenges for geotechnical pile design and tunnelling works. The potential impact of the TBM tunnel excavation works on the SIR and BLOD structures was assessed using 2D finite element analyses to assess soil movements and structural effects. Risk mitigation measures such as monitoring the tunnelling excavation volume during construction, grouting works, groundwater control and other measures have been considered to prevent the risk of excessive movements.

1 INTRODUCTION

Stage 1 of the Alkimos Seawater Desalination Plant (ASDP) is currently being designed and constructed by the Alkimos SeaWater Alliance (ASWA). The ASDP is located within the high growth area of Alkimos about 40 km to the north of Perth, is less than about 1 km from the coast and located to the west of the existing Alkimos Wastewater Treatment Plant (AWWTP). When commissioned, Stage 1 of the ASDP will deliver 50 GL per annum of drinking water to the northern suburbs of Perth for commercial and residential use. Stage 2 of the ASDP (planned for future expansion) will provide an additional 50 GL per year.

Nearshore components of the ASDP include the Seawater Intake Tunnel (SIT), Brine Outfall Tunnel (BOT), two Seawater Intake Riser (SIR) structures and a Brine Linear Outfall Diffuser (BLOD) structure.

2 NEARSHORE STRUCTURES

The SIT and BOT have a total length of about 2.6 km and 4.1 km respectively and will be excavated separately using slurry TBMs with a 4.185 m excavation diameter. Excavation will begin on land from a temporary access shaft in the north-western corner of the onshore ASDP site, progressing outward to the ocean. The two TBM shields will be sacrificed at the end of the drives. Both tunnels will have reinforced concrete segmental linings with a design internal diameter (ID) of 3.505 m and a design outer diameter (OD) of 3.905 m.

Two SIR structures will convey seawater into the SIT. The proposed foundation system for each SIR structure comprises of four driven steel circular tube piles with an OD of 1050 mm and wall thickness (WT) of 40 mm (1050 OD x 40 WT) with a pile spacing of 8.95 m. The piles extend above the seabed level and are structurally connected to the head of the SIR structure. A driven steel circular liner (casing) with an OD of 3074 mm and WT of 37 mm (3074 OD x 37 WT) shall be placed to provide ground support for construction of an 1800 mm internal diameter (ID) Glass Fibre Reinforced Plastic (GRP) riser that connects into the SIT. Figure 1 illustrates the proposed foundation system for a SIR structure.

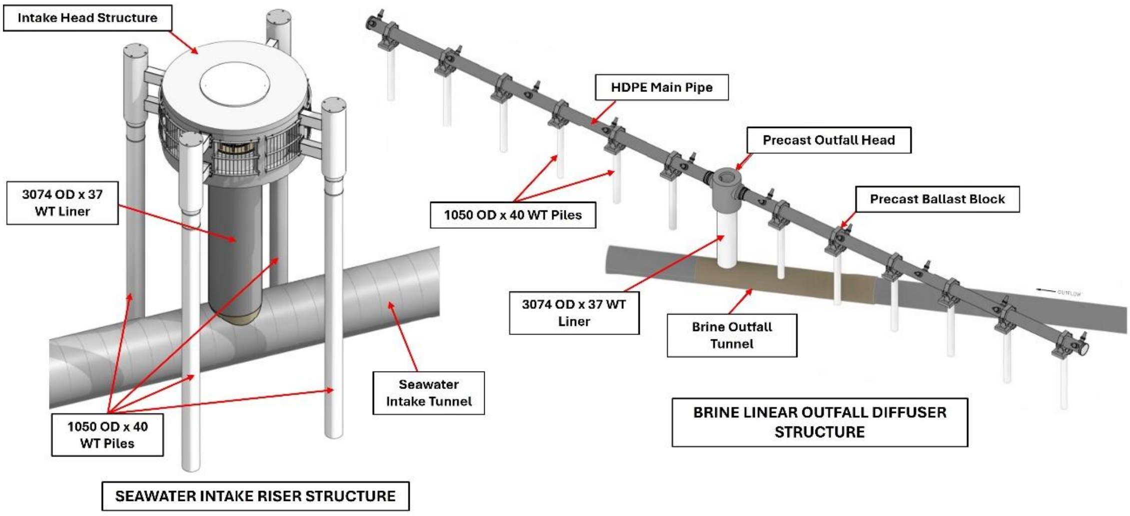

Figure 1. Proposed foundation system for the Seawater Intake Riser (left image) and Brine Linear Outfall Diffuser (right image) structures.

The BLOD diffuser will expel brine coming from the BOT into the ocean. The BLOD comprises of a horizontal High-Density Polyethylene (HDPE) pipe supported by pile foundations and has a total length of about 135 m. Precast reinforced concrete ballast blocks are placed along the BLOD and are supported by twelve driven steel circular tube piles with an OD of 1050 mm and WT of 40 mm (1050 OD x 40 WT). Centre to centre pile spacings are 10.0 m to 10.5 m. A driven steel circular liner (casing) with an OD of 3074 mm and WT of 37 mm (3074 OD x 37 WT) shall be placed to provide ground support for construction of an 1800 mm ID GRP riser that connects into the BOT and to support the precast outfall head which connects the two HDPE pipe sections together. Figure 1 illustrates the proposed foundation system for the BLOD structure.

3 GEOLOGICAL MODEL

A comprehensive assessment of site investigation information led to the development of an Engineering Geological Model (EGM) for the ASDP. The primary engineering geological units encountered at nearshore structures included the Marine Sediments (Unit 2), Tamala Limestone (Unit 5), Ascot Formation (Unit 6), Osborne Formation (Unit 8) and the TQ Sands (Unit 9).

The Marine Sediments are the youngest unit being of recent to Quaternary age deposits. They primarily comprise of coarse grained (i.e. non-cohesive) soils, and ascribed sub-unit 2c. They are present along the ocean seabed at both the SIR and BLOD structures.

The Tamala Limestone is of Quaternary geological age and only encountered at the SIR structures, underlying the Marine Sediments. Three sub-units relating to the degree of leaching and engineering properties of the Tamala Limestone have been ascribed: Unit 5a Fresh to Moderately Leached; Unit 5b Moderately to Highly Leached; Unit 5c Extremely Leached to Residual Soil.

The Ascot Formation is of Tertiary geological age, underlying the Tamala Limestone at the SIR structures and underlying the Marine Sediments at the BLOD structure. Four sub-units have been determined: Unit 6a Mostly rock strength and cemented materials with soil, rock strengths varying from very low to medium, and typically varying from type D1 to type D3 duricrust according to AS1726:2017; Unit 6b Mostly soils but with some cemented layers; Unit 6c Variable sand layers; Unit 6d Silt / Siltstone materials.

The Osborne Formation is of Cretaceous geological age, underlying the Ascot Formation at the SIR structures and underlying the TQ Sands at the BLOD structure. Three sub-units have been determined: Unit 8a Variably cemented fine-grained material and weathered sandstones, claystones; Unit 8b Fine grained (i.e. cohesive) materials; Unit 8c Coarse grained (i.e. non-cohesive) materials.

The TQ Sands are of Tertiary geological age and only encountered at the BLOD structure, underlying the Ascot Formation and above the Osborne Formation. The unit comprises of predominantly coarse grained (i.e. non-cohesive) materials, and ascribed sub-unit 9a.

Calcareous deposits typically consist of carbonate materials from marine life debris. Unlike silica soils, calcareous soils are primarily composed of calcium carbonate, which has a lower hardness compared to quartz. This makes them more susceptible to crushing under relatively low stresses. Calcareous soils typically have high porosity (and void ratio), resulting in low density and high compressibility. They are also prone to post-deposition alterations by biological and physiochemical processes under normal pressure and temperature conditions, forming irregular cemented layers that significantly impact their mechanical behavior.

The Marine Sediments (Unit 2), Tamala Limestone (Unit 5) and Ascot Formation (Unit 6) projects units have been treated as calcareous soil deposits for the purposes of pile foundation design at both the SIR and BLOD structures.

4 GEOTECHNICAL INTERPRETATION

The interpreted geotechnical design ground models for the SIR and BLOD structures are illustrated in Figure 2.

Table 1 presents characteristic geotechnical design parameters for project units at nearshore structures. Whilst these values represent each unit, variability within units is expected. Upper and lower bound parameters were also assessed using available site data and considered in design where necessary, though not reported here.

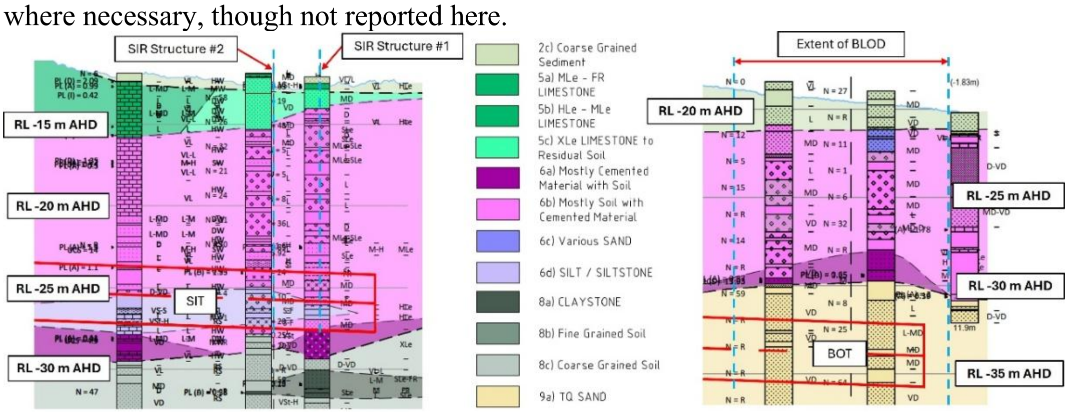

Figure 2. Interpreted ground models for the nearshore structures.

Table 1. Characteristic geotechnical design parameters.

Project Unit	Unit Weight, γ (kN/m^3)	Effective Cohesion, c' (kPa)	Effective Friction Angle, φ' (degrees)	Poisson's Ratio	Drained Elastic Modulus (MPa)	OCR	At-rest Earth Pressure Coefficient, K0
2c	18	0	35	0.3	30	1	0.52
5c	19	0	36	0.3	50	2	0.62
6a	20	0	38	0.3	60	3	0.68
6b	19	0	36	0.3	50	2	0.71
6d	19	0	30	0.3	35	2	0.79
8c	20	0	38	0.3	90	3	0.82
9a	20	0	39	0.3	90	3	0.82

5 PILE DESIGN

5.1 *Design standards and codes*

Pile design was primarily undertaken in accordance with the AS/NZS 1170 series and AS2159:2009 – Piling Design and Installation. Several other international design standards and codes were referred to and used to inform design.

5.2 *Design loading*

The predominant loading type on the structures is wave and current induced. A combination of dead loads and wave induced loads was assessed as the critical load combination with appropriate partial factors applied for Serviceability Limit State (SLS) and Ultimate Limit State (ULS) design combinations. SLS wave loads considered a 1:1 Average Recurrence Interval (ARI) event whilst ULS was loads considered a 1:2000 ARI event. Pile and liner design loads were provided at the design seabed and scour levels for geotechnical assessment. The design scour depths (below design seabed levels) at the SIR and BLOD structures was 4.0m and 4.5 m respectively.

5.3 *Ultimate limit state design*

A geotechnical strength reduction factor (φ_g) of 0.75 has been adopted for axial compression loading in accordance with Clause 4.3 of AS 2159:2009. High-strain dynamic pile load testing has been nominated at pile locations at both the SIR and BLOD structures.

The ultimate skin friction (f_s) for piles in non-cohesive (i.e. sand and gravel) materials has been calculated in terms of effective stress using $f_s = K \tan \delta \sigma'_v$, where K = coefficient of lateral pressure; δ = pile-soil interface friction angle; σ'_v = effective vertical stress. A K/K0 value of 1.0 and a δ/φ' value of 0.6 have been used for geotechnical design.

The ultimate end bearing (f_b) pressure under compression for non-cohesive materials has been calculated using $f_b = \sigma'_t$ Nq, where σ'_t = effective vertical stress at the pile tip (toe); Nq = bearing capacity factor. The bearing capacity factor (Nq) has been assessed based on the effective friction angle (φ') with reference to the works by Berezantzev et al. (1961). The effective friction angles at the pile tip were adjusted according to Poulos and Davis (1980) to account for pile installation.

Geotechnical pile design has incorporated findings from previous offshore investigations and studies, accounting for the calcareous nature of certain project units. This approach included selecting appropriate limiting skin friction values, end bearing pressures, a low pile-soil interface friction angle (δ) for driven piles in sand, and a suitable K/K0 value.

Lateral pressure-displacement (p-y) curves were developed to account for the nonlinear behaviour of soil. Three different methods of calculating p-y curves have been considered to investigate the behaviour of piled foundations under lateral loading. These methods are documented in Poulos and Ameratunga (2022), the American Petroleum Institute (API) RP 2A (2010), and Dyson and Randolph (2001).

A geotechnical strength reduction factor of 0.4 was considered when assessing pile lateral stability under ULS loading with and without consideration of scour. The lateral deflection and structural responses in piles (i.e. induced bending moment and shear force) due to ULS design loads have been assessed using LPILE.

5.4 *Serviceability limit state design*

Pile vertical settlements under SLS design loads were assessed using PLAXIS3D. Laterally loaded piles under SLS design loads have been assessed using LPILE. The assessed SLS vertical settlements and lateral displacements were provided to the ASWA Marine Structural Design Team for consideration of serviceability performance requirements.

The PLAXIS3D models, using the Mohr-Coulomb soil material model, were also used to compare the pile behaviour (i.e. the calculated shape of shear force and bending moment diagrams along the pile) and results under SLS loads with those from the three p-y curve methods used in LPILE. Pile design loads were applied at the design seabed and / or scour level.

5.5 *Other considerations*

Other notable considerations included:

- Cyclic Effects: the predominant loading on the SIR and BLOD structures is due to wave loading which can be considered as cyclic loading. It was shown that cyclic loading would have an insignificant impact on pile axial design requirements. The lateral pressure-displacement relationship from API RP 2A (2010) can cater for cyclic loading, by further reducing the soil lateral resistance for cyclic load effects. This has been considered in pile designs.
- Liquefaction: Borehole and laboratory test data was used to inform liquefaction assessments at nearshore structure locations. The outcome of the liquefaction assessment indicated that the risk of soils liquefying below the ground water table at the SIR and BLOD structures is low and therefore unlikely.
- Earthquake Loading: AS2159:2009 states that the effects of earthquake loading on both the design axial and lateral ultimate geotechnical capacities (strengths) are to be considered. The assessment of both the inertial and kinematic effects was undertaken but noted that earthquake loading was not a governing load case, and that wave loading on the structures provides the most adverse loading conditions for design.
- Pile Driveability: Thorough pile drivability assessments have been undertaken considering the proposed hammer and driving setup, drop heights, and plugged versus unplugged base soil conditions. There is also the potential for driven piles and liners to refuse on dense to very dense sand layers (within the Ascot Formation, Osborne Formations and TQ-Sand) and / or weathered bedrock (e.g. Tamala Limestone) and / or cemented zones (e.g. within Unit 6a from the Ascot Formation). A 'drill and drive' method may be required to overcome this to reach design pile toe founding levels.

6 IMPACT FROM TUNNELS

PLAXIS2D finite element analyses assessed the potential impact of TBM tunnel excavation on SIR and BLOD pile foundations. The assessment aimed to determine additional pile movements, shear forces, and bending moments resulting from tunneling.

The PLAXIS2D models incorporated construction tolerances for TBM alignment, excavation, grout support, and pile positioning. A single steel tube pile was modeled as an embedded beam element, with half of the 4.185 m circular excavation diameter modeled for both tunnels. SLS design loads were considered without scour, assuming unlikely ULS loading during the brief tunneling period.

The TBM and segmental lining were modeled as plate elements, with a 2% volume loss applied as a line contraction. Annulus grouting was simulated using radial pressures of 280 kPa (SIT) and 490 kPa (BOT) as advised by the ASWA Tunnel Design Team. The Hardening Soil material model was used for all project units. A typical 2D finite element model is shown in Figure 3.

An additional assessment of ground movement was assessed without pile loads to develop a lateral soil movement profile due to TBM tunneling for use in LPILE with SLS and ULS loads.

Figure 3 shows horizontal pile movements from TBM tunnel excavation at SIR structure #2, with similar results at SIR structure #1 and slightly lower displacements at the BLOD. The lateral soil (free field) movement on the piles resulting from TBM tunnel excavation and without SLS design loads was similar. The additional vertical settlement of the piles due to TBM tunnel excavation was also assessed.

The lateral soil movement profiles were applied in LPILE with SLS and ULS design loads in both the same and opposite directions. The results from the API RP 2A (2010) method with consideration of cyclic load effects were used to compare findings of the effects on piles with and without TBM tunnel excavation. For the SIR structures there was a notable increase in the calculated shear force and bending moment values in the piles adjacent to the SIT whilst for the BLOD there was an insignificant increase. Maximum calculated shear force and bending moment values at all structure locations were less than the design ultimate structural capacities on the piles.

TBM tunnel excavation induces additional down drag axial force on piles due to negative skin friction from downward soil movement, as illustrated in Figure 4. These forces were assessed for each structure and provided to the ASWA Marine Structural Design Team. Per AS2159:2009

Clause 3.3.2 (b), this down drag force must be multiplied by 1.2 and added to design axial actions for structural design.

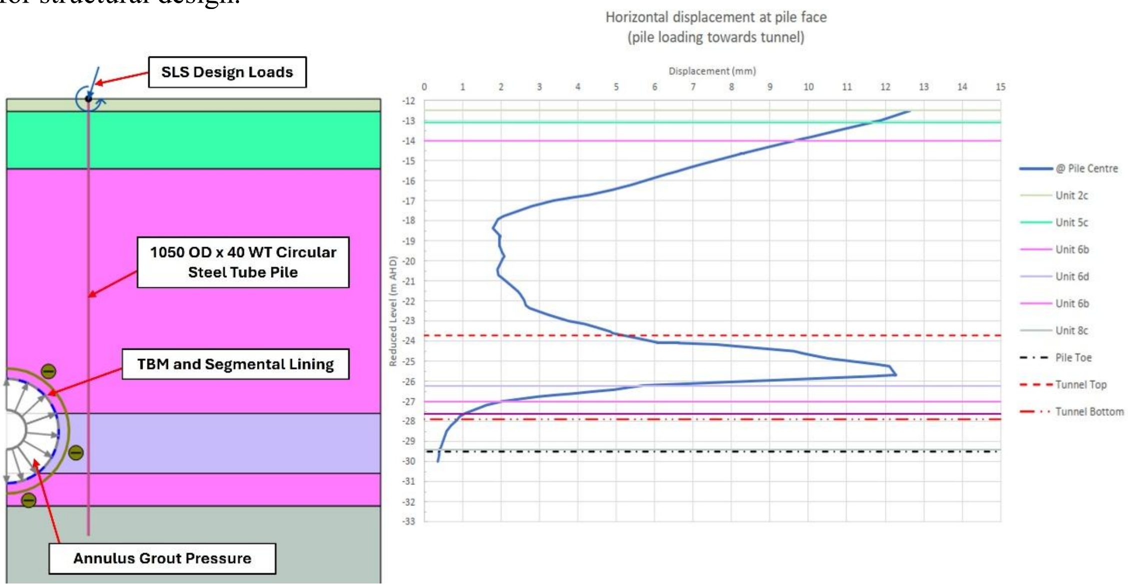

Figure 3. Example of 2D finite element model in PLAXIS2D (left image). Example of calculated horizontal movement on pile due to tunnel excavation (right image)

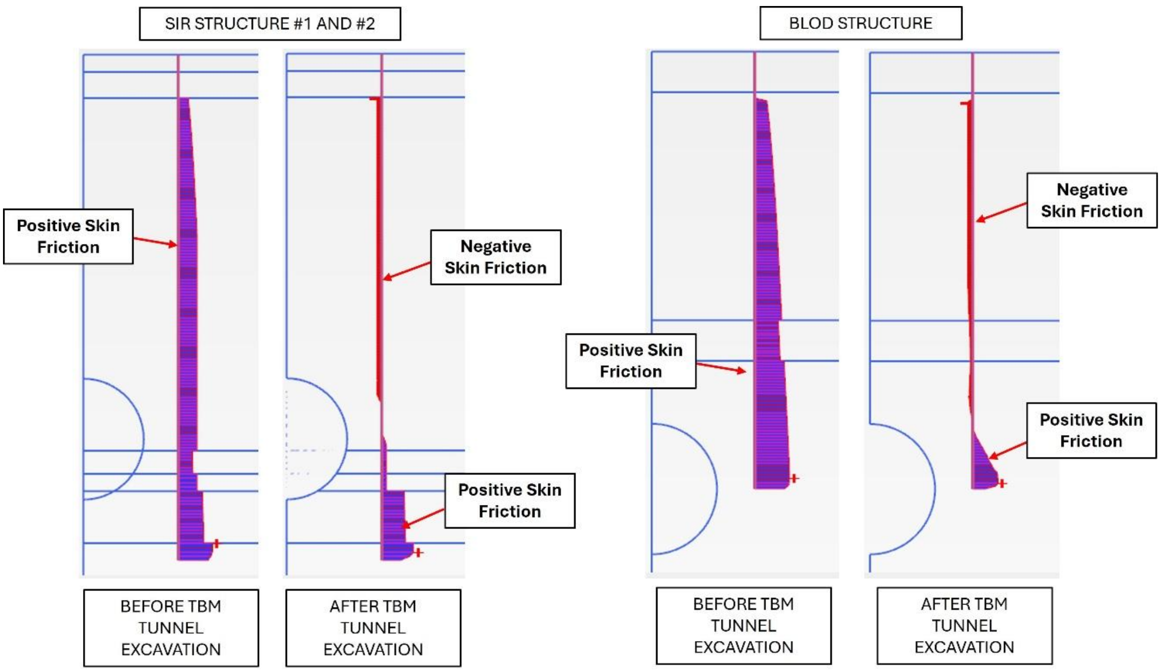

Figure 4. Development of down drag axial force on pile resulting from TBM tunnel excavation.

7 RISK MITIGATION MEASURES

Due to the highly variable geological, geotechnical and hydrogeological properties of the encountered project units there is the potential for complications associated with controlling face pressures, stability of the formations, limiting ground / volume loss and minimising groundwater ingress during TBM tunnelling. Grouting in front of the TBM face may be required to help alleviate some of these concerns. Grouting will also likely be required to infill the annulus between the tunnel concrete lining segments and the surrounding soil / rock behind the TBM shield. Grouting conducted in these zones must consider the possible presence of high permeability material (Tamala Limestone, Ascot Formation, TQ Sands) which may result in grout loss or loss of grout pressure during the grouting process. In this case it may be necessary to carry out additional grouting during construction.

The volume loss is a key factor in assessing ground movements resulting from tunnelling. Grout pressures may need to be adjusted (that is increased) to minimise the potential for significant ground / volume loss to occur which could adversely impact pile foundations. Furthermore, appropriate TBM operating procedures including consideration given to speed of excavation, type of cutting tool, slurry support mix and maintenance intervals should be allowed for. Convergence monitoring of installed segmental lining segments is also a useful way to investigate ground movements following tunnel excavation.

Sophisticated and state of the art monitoring and global position systems are available and are understood to have been included with the TBM's for the ASDP Project that will allow for meticulous excavation control along the tunnel alignments, limiting deviations and over-excavation to in the order of 100 mm. A bigger challenge is the location and positioning of the piles and liners from a jack-up barge. A temporary pile driving steel template, that can be lowered and placed on the seabed, has been recommended to assist with locating the plan position of the piles and liners.

8 CONCLUSIONS

The susceptibility to crushing of the grains and the random cementation extent have generally been understood to be key features which distinguish calcareous sediments apart other sediments. Given their unique and intricate composition, they deserve special consideration when assessing them for design and construction purposes. Sufficient, targeted and specialised geotechnical in-situ and laboratory testing should be undertaken to assist with determining critical design parameters and to better inform their challenging engineering behaviour. Field tests and trials are encouraged to be considered in complex and difficult geological environments where anisotropic conditions and a high degree of material variability are anticipated.

Appropriate risk mitigation measures should also be developed to prevent or minimse the risk of excessive ground movements resulting from TBM tunnelling and to not adversely impact pile foundations. Some of these measures include grouting ahead of the TBM, maintaining slurry and grout pressures during segmental lining installation, convergence monitoring and onboard real time monitoring of TBM excavations.

9 ACKNOWLEDGEMENTS

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