Geotechnical design of large diameter monopiles for New Bridgewater Bridge

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| **Abstract**  The New Bridgewater Bridge (NBB) Project is Tasmania’s largest ever transport infrastructure project. The project is a critical link in Tasmania’s transport network and forms part of the Australian Government’s National Land Transport Network. The NBB project involves construction of a four-lane, 1.28-kilometre-long bridge over the River Derwent with new interchanges at Granton and Bridgewater, a shared path for cyclists and pedestrians, and a navigation clearance consistent with the Bowen Bridge, all aimed at improving traffic flow and connectivity. The NBB replaces the existing steel-truss vertical lift bridge, which was completed in 1946. The new bridge comprises a total of 21 bridge piers and 2 abutment piers with each span varying from 44 m to 64 m in length. The total width of the bridge deck ranges from 25m to 30m and will accommodate four traffic lanes and a shared use path. A precast single-cell box girder structure supporting each carriageway is supported on a 2.8m diameter single pier connected to a 2.5m diameter concrete bored pile socketed into very variable weathered rock except for the abutment piers where 2.1 m diameter piles are adopted. The centre to centre spacing of the piles within each individual pier is approximately 14 m. The length of foundation piles below the top of the pile varies from 12.5 m to 86.5 m with the latter likely to be the deepest mono bored pile in the Southern Hemisphere. This paper commences with a brief description of geological conditions along new bridge alignment and then discusses the basis of design. The paper moves further to provide an overview of the foundation systems adopted for the bridge, and to discuss design approaches, key geotechnical issues and challenges encountered in design and construction of production piles; subsequently presents an overview of the use of Osterberg cell testing to verify axial capacity; and finally presents some technical insights for future design and construction of similar deep foundations as conclusions.  **Keywords:** monopile, geotechnical design, deep bridge foundation, design validation, |

# Introduction

The New Bridgewater Bridge (NBB) Project is Tasmania’s largest ever transport infrastructure project. The project is a critical link in Tasmania’s transport network and forms part of the Australian Government’s National Land Transport Network. The NBB project involves construction of a four-lane, 1.28-kilometre-long bridge over the River Derwent with new interchanges at Granton and Bridgewater, a shared path for cyclists and pedestrians, and a navigation clearance consistent with the Bowen Bridge, all aimed at improving traffic flow and connectivity. The NBB replaces the existing steel-truss vertical lift bridge, which was completed in 1946.

Figure 1 presents the alignment with locations of bridge pile foundations and test pile locations. At time of preparing this paper, all bridge foundation piles have been installed successfully.

* This paper commences with a brief description of geological conditions along new bridge alignment and then provides an overview of the foundation systems adopted for the bridge.
* The paper moves further to discuss the basis of design, design approaches, key geotechnical issues and challenges encountered in design and construction of production piles; subsequently presents an overview of the use of Osterberg cell testing to verify axial capacity; and finally presents some technical insights for future design and construction of similar deep foundations as conclusions.

***Figure 1: NBB alignment***

A map of a highway

Description automatically generated

# Geological settings and overview of ground conditions

Based on the site investigation data available during tender design and local geological map, the ground conditions along the bridge alignment were found to be very variable in terms of 1) depth of estuarine sediments and ice age (Quaternary) sediments, 2) types of rock and 3) weathering degree and strength of rocks etc. As part of design and construction risk management of such highly variable ground conditions, the authors of this paper as project geotechnical designers and representative of the contractor allowed a minimum of one borehole drilled at the centre of each pile with necessary in-situ and laboratory tests to inform geological settings and ground conditions that were needed for design and construction. Based on all site investigation data available in detailed design phase, a simplified illustration of the subsurface geology along left pier alignment and the location of each individual bridge pier foundation is shown on Figure 2 below. Due to limited space, the similar subsurface geological long section along right pier alignment is not presented in this paper. The following section summarises some key geological findings that were inferred and interpreted based on all site investigation data available and local experience.

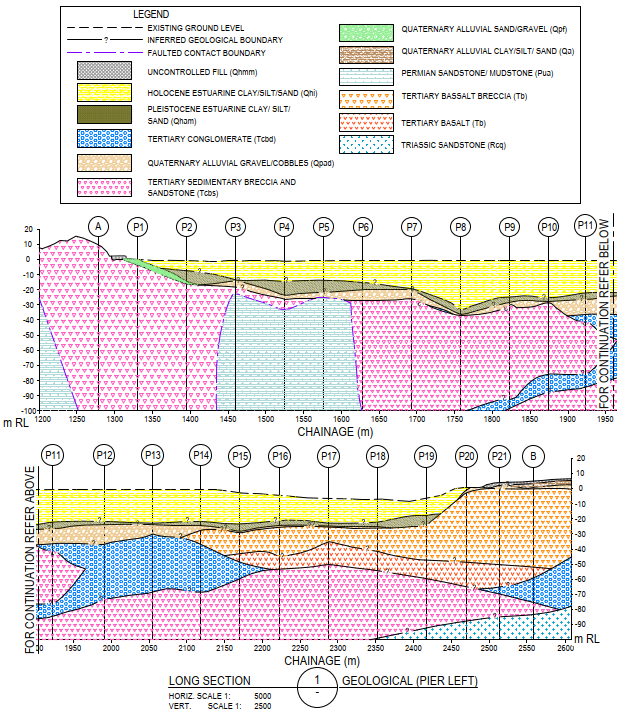
The River Derwent channel bed, across which the new Bridgewater Bridge spans, is predominantly comprised of recently deposited (Holocene) estuarine muds, of silt/clay layers with potential thin layers of sandy horizons (in the channel and/or on tidal mudflats), extending up to 30 m below the ground level, especially near the southern abutment. Below the estuarine sediments, ice age (Quaternary) sediments overlie (only partially as suggested by historical site investigation data) the existing bedrock and predominantly comprise dense to very dense coarse-grained materials (gravels and cobbles). These deposits can extend up to 10 m or more below the overlying soft silts/clays.

Tertiary Age bedrock deposits comprise four main units including, basalt breccia and basalt flows on the north riverbank and extending under the navigation channel, highly weathered conglomerates ('cemented gravels') in buried channels in the riverbed and on the southern approaches in the area of Black Snake Rivulet.

Low strength landslide breccia is identified on the south riverbank within the Granton Fault zone. Sedimentary bedrock is expected at varying depths, between 5 m and 30 m, with Triassic rocks expected to be competent for rock sockets and end-bearing piles.

Triassic sedimentary rocks consist of sandstones, mudstones, and breccias predominantly varying from very low to low rock strength and varying degrees of weathering and jointing. A fault zone (the Granton Fault) runs oblique to the bridge alignment and is likely to underlie the southern approach, southern abutment and extend into the river channel. The Tertiary landslide breccia is currently interpreted to be sitting within this zone.

***Figure 2: Geological long section along left pier alignment***



Based on geological interpretations and borehole logs available, the ground conditions at every pier location are found to be significantly different between neighbouring pier locations and even between left pile and right pile at the same pier location particularly for those piers from southern abutment pier to river Pier 14 / Pier 15.

This meant that every foundation pile would have to be investigated and designed independently in order to maximize its potential capacity and to reduce its potential risks as well from design and construction perspective. For example, highly to slightly weathered with low to high strength siltstone and sandstone were encountered at relative shallower depths of approximately 25 m below the existing seabed level at locations of Pier 3 Left, Pier 3 Right, Pier 4 Left, Pier 5 Right, Pier 5 Left; but not at Pier 4 Right where a very thick layer of weak breccia with very low to low strength was encountered down to a deep depth of approximately 69 m below the existing seabed level. The top surface levels of competent rocks varied significantly between the left and right piers at some locations, particularly at Pier 4. Another example is at Pier 8 Left where the presence of weak breccia / siltstone / sandstone with very low to low strength, that were often mixed up with clay and sand like matrices, could not be confirmed by the borehole drilled and terminated at a depth of approximately 89 m below the existing seabed level. This demonstrates that there were significant risks associated with design and construction of the bridge foundation piles along the bridge alignment that needed to be managed properly.

# Overview of foundation systems

As part of tender optioning, two different options, i.e. single-cell box girder structure supporting each carriageway versus I girder beam, were investigated by McConnell Dowell in detail during the tender design phase. The single-cell box girder structure with associated foundation was finally developed with details and submitted as the preferred option (see Figure 3 below). As for the pier foundation, different options such as a large diameter monopile without pile cap and multiple small diameter piles with pile cap as well as driven steel tubes, were investigated during tender design phase. The large diameter monopile was finally adopted and submitted as part of preferred solution with consideration of a number of factors (see Figure 3 below). Those factors cover: 1) preferring no pile caps; 2) preferring better connection to pier using monopile; 3) no scouring protection required; 4) optimum for piling program. Figure 3 below presents a typical arrangement of the abutments and piers. H.A.T and L.A.T. presented in Figure 3 stand for Highest Astronomical Tide and Lowest Astronomical Tide, respectively. MSL stands for mean sea level. It is worthwhile to mention adoption of a precast shell in the splash zone addressed durability requirements.

***Figure 3: Typical abutment arrangement (left hand side (LHS)) and pier arrangement (right hand side (RHS))***

A blueprint of a building

Description automatically generated

The new bridge comprises a total of 21 bridge piers and 2 abutment piers with each span varying from 44 m to 64 m in length.

The total width of the bridge deck ranges from 25m to 30m and will accommodate four traffic lanes and a shared use path. The precast single-cell box girder structure is supported on a 2.8m diameter single pier connected to a 2.4m diameter concrete bored pile socketed into very variable weathered rock except for the abutment piers where 2.1 m diameter piles are adopted. The centre to centre spacing of the piles within each individual pier is approximately 14 m. The length of foundation piles below the top of the pile varies from 12.5 m to 86.5 m with the latter likely to be the deepest mono bored pile in the Southern Hemisphere.

# Design basis of foundation piles

As part of project requirements, bridge foundation piles were designed based on Appendix 25 of the Contracted Project Scope and Technical Requirements (PSTR) (i.e. Requirements for Structures), Part 2 of Australia Standard of AS5100-2017 (+A1) (i.e., Design Loads), Part 3 of AS5100-2017 (+A1) (i.e., Foundation and Soil-Supporting Structures of Bridge Design)1, Part 4 of AS 1170-2007 (R2018) incorporating Amendments Nos 1 and 2 (i.e., Earthquake actions in Australia)2, Part 8 of Austroads (2018) (i.e., Hydraulic Design of Waterway Structures)3, Chapter 5 of Part 8 of Austroads Guide to Bridge Technology (2019) (i.e., Bridge Scour)4 and AS2159-2009 (i.e., Piling - Design and Installation)5. Table 1 below presents a summary of a few key technical requirements for design of River Derwent foundation piles which were discussed and agreed with the DSG.

***Table 1: Key technical requirements***

|  |  |
| --- | --- |
| **Item** | **Key technical requirements** |
| 1 | The design life of the main bridge structure is 100 years and of its foundation piles is 130 years, respectively. |
| 2 | Limit state design for both strength and settlement must be considered. |
| 3 | Bridge design must be in accordance with AS 5100 and provide for a 0.05% annual exceedance probability (AEP) flood loading on bridge piers as an ultimate limit state. Design must also provide for a 1% AEP flood loading on bridge piers as a serviceability limit state. |
| 4 | Any risks of scour to the bridge foundations must be mitigated for the Design Life of the bridge. |
| 5 | Seismic loading and the potential for liquefaction in the foundation materials resulting from a seismic event must be considered and addressed as part of the design. |
| 6 | The design methodology must relate to the chosen construction techniques and must directly address uncertainty in design parameters. |
| 7 | No other drilling fluid, other than water and bio-degradable polymer that is approved by the State, i.e., by DSG, may be used in the construction of piles. |
| 8 | A minimum of three (3) bored test piles with a minimum diameter of 1.5 m, strategically placed in the vicinity of the piers for the New Bridgewater Bridge to the approval of the State’s Representative and Independent Verifier, are to be tested using Osterberg Type Cells or as otherwise agreed with the State. |
| 9 | A nominal differential settlement allowance of 10 mm between adjacent piers under the permanent loads was specified by project bridge structural designer. This 10 mm differential settlement allowance is in addition to the settlement of the overall structure considered in the structural model based on the vertical spring stiffness provided by the project geotechnical designer. |
| 10 | The Contractor must, by remote inspection of sockets, verify that the side walls and base of the socket is consistent with design assumptions. Inspection of the side walls and the base of the socket must be carried out using a remote sensing technique capable of distinguishing between the actual base material and the detritus on the base. The remote sensing technique used by the Contractor must be capable of identifying debris deposited at the rock socket base and estimating their thickness in low to zero visibility underwater condition. The Contractor shall nominate the cleaning tool and provide evidence of effectiveness in cleaning out debris in similar foundation types to the Principal, i.e., DSG, before applying the technique. |
| 11 | Pile integrity testing must be undertaken for all bored-in-situ piles. |

# Design of foundation piles

1. **Design assumptions**

**Table 2** below presents few key design assumptions.

***Table 2: Key design assumptions***

|  |  |
| --- | --- |
| Item | Key design assumptions |
| 1 | The contribution from the estuarine clay layer on pile side friction is negligible hence ignored in the axial capacity estimation. |
| 2 | Pile steel casings were driven and drilled to prevent top unstable shafts from potential collapses with casing penetrations up to 1.5m into the upper weathered sedimentary conglomerate/sandstone and 0.5m into basalt breccia, respectively. |
| 3 | A utilization factor of 0.5 applied to the ultimate base resistance, i.e., at least 50% base cleanliness is achievable (subject to onsite verification). |
| 4 | A geotechnical strength reduction factor, i.e., fg, adopted to obtain design geotechnical strength of pile, i.e., Rd,g, in accordance with AS2159 considering the level of importance and the high-risk characteristic of the project. |
| 5 | Although not required by the Project Specification, a traditional global factor of safety of 2.5 on working loads was also adopted and applied to obtain both allowable pile shaft and end bearing resistances. |

1. **Geotechnical strength reduction factor, fg**

Risk assessment to obtain the geotechnical strength reduction factor, i.e., fg, in accordance with AS2159 was performed with consideration of 1) site risk factors; 2) design risk factors; 3) installation risk factors; 4) completion of one borehole per pile; and 5) three (3) test piles conducted successfully. In accordance with AS2159, a geotechnical strength reduction factor of 0.81 could have been adopted, but to maintain a conservative design, a lower value of 0.7 was adopted in the design considering the level of importance and the high-risk characteristic of the project.

1. **Structural loading**

The pile loads adopted for the design of the bridge foundation were provided by the project bridge structural designer, Tony Gee and Partners (TGP), Hong Kong, with loading acting at 'pile head' reduced level. The pile loads were assessed with iterative process between both the project bridge structural designer and the project geotechnical designer and with further details to be discussed below.

Based on the vertical pile loads provided by the structural designers (TGP), the loads under serviceability limit state (SLS), i.e., Eds, varied from 21.20 MN to 32.50 MN for the main piers and from 16.05 MN to 17.90 MN for the abutment piers, respectively. The loads under ultimate limit state (ULS) varied from 28.50 MN to 44.20 MN for the main piers and from 21.50 MN to 24.90 MN for the abutment piers, respectively.

Apart from vertical loads, typical horizontal loads (2 different directions) and moments (3 different components) were also provided by TGP for geotechnical assessment. The greatest moments of 40.75 MN.m along transverse direction and 40.48 MN.m longitudinal direction under ULS condition were found to occur at Pier 17 where the channel is at its deepest. The maximum design action moments were assessed to take place under dynamic condition with scouring effect.

1. **Geotechnical models and design parameters**

As discussed above, the ground conditions along the bridge alignment were found to be very variable in many ways. For such reason, a geotechnical model and associated design parameters were developed for each individual foundation pile. Both historical and additional site investigation data were utilised including borehole logs, laboratory test data such as point load test (PLT) results, unconfined compressive strength (UCS) with and without measurement of deformation, direct shear test (DS) results, unconsolidated undrained (UU) test results for extremely / highly weathered soil like weak rocks with very low strength, as well as in-situ pressuremeter test results etc. Due to limited size of rock core samples to be tested, disturbance caused by drilling process and presence of defects and clay matrices within rock core samples, often the test results from laboratory would not truly represent the in-situ rock mass. Hence, the pressuremeter test results were used to set as benchmarks to calibrate findings from borehole logs and laboratory tests. From the pressuremeter test results, both equivalent UCS and Young’s modulus for the rock mass were back calculated. In addition, an approach with overall view to consider global mass behaviour for rock strength assessment was taken in the derivation of strength profile with engineering judgement applied by very experienced designers rather than blindly using factual logs and test data without properly examining and filtering some of unreliable data.

For the design of foundation piles, geological strata with reduced levels of each geological unit and strength profile were assessed and derived at each foundation pile location. From that, ultimate shaft skin friction, end bearing resistance, and Young’s modulus were assessed and derived using the most adequate industrially available methods. During tender design, Rowe and Armitage method6&7 was adopted as part of project requirements to develop the bridge foundation design. In detailed design phase, Rowe and Armitage method was removed from contracted project requirements. There are many different methods / approaches available for assessing 1) shaft skin friction, 2) end bearing resistance; and 3) Young’s modulus of soils and rock mass. Each of them has pros and cons. The following section describes relevant methods / approaches adopted and associated key assumptions made for the design of the bridge foundation piles.

* The a method with a reduction factor of a as per Figure 16-14 of Joseph E. Bowles (1997)8 was adopted for deriving ultimate shaft skin friction of “fine grained soils”.
* The b method with reduction factor of b as per Section of 16-9.3 of Joseph E. Bowles (1997) was adopted for deriving ultimate shaft skin friction of “granular soils”.
* The equation of as UCS bs (MPa) as per Zhang L. (1999)9 was adopted for deriving ultimate shaft skin friction of “rocks”, where, as = 0.20 to 0.30; bs = 0.50. Note that the value of as is dependent upon socket roughness and may be higher (i.e., 0.22 to 0.67) than the conservative range proposed above (i.e., 0.20 to 0.30). A conservative assumption of 0.20 - 0.30 was adopted for NBB project to take into consideration of potential softening effects attributed to stress relaxation over a relative long period of time where the support to the shaft was significantly reduced due to shaft excavation prior to concrete pouring. As part of design strategies, the conservative as was proposed and adopted to allow a departure from physical validation / checking on shaft roughness during construction because it was discovered that there was no current technology available to validate / check shaft roughness during construction when a polymer drilling fluid is also adopted.
* A conservative equation of 4.8 UCS 0.5 was adopted to assess the ultimate end bearing pressure for highly weathered or better, rocks. This equation was simplified based on Zhang L. and Einstein H.H. (1998)10 to account for potential softening effects to occur below the pile toe. The equation of j Mr UCS (MPa) as per Michael J. Tomlinson (2001)11 was adopted to assess vertical mass Young’s modulus of rocks; where j is mass factor related to discontinuity spacing in the rock mass; Mr is ratio between deformation modulus and UCS of rocks which will depend on the rock type and origin.
* A combined factor of j Mr was evaluated for different rocks based on UCS test results with deformation, pressuremeter test results and presence of clay matrices & core losses with reference made to the rock mass properties assessed using a commercial software RocLab.
* An empirical relationship of 200 su (kPa) was adopted to assess vertical mass Young’s modulus of fine-grained soils, where su is undrained shear strength and this formula should not be used for soils loaded beyond the pre-consolidation pressure.
* Empirical relationships of 1.0 N60 (MPa) and 1.6 N60 (MPa) were adopted to assess vertical mass Young’s modulus of coarse-grained soils for loose to medium dense soils and dense to very dense soils, respectively, where N60 is corrected SPT-N value.

1. **Scouring effects**

The bridge was designed for the effect of scour, which in the design flood event will remove materials from around the base of the piers as there is no specific scour protection material provided around the piers. The scour modes considered comprise: 1) long-term bed degradation; 2) general (or morphological) scour; 3) contraction scour; 4) local pier scour (including potential debris raft exacerbation), and 5) thalweg migration and lowering. The scour assessment used the geological model to inform the reasonable and credible limits of bed scour (typically, the top of the breccia layer). The scour assessment used bed material grain size to compute general (morphological) scour and to ascertain if there will be "live bed" or "clear water" scour conditions in flood. There was sufficient information already from the ground investigations and material sampling and testing to determine the general scour and to confirm that live bed scour conditions are expected at the bridge site. Based on scouring assessment, the predicted scour depth under 1 in 2000 (0.05%) AEP event varies from approximately 4 m along south alignment to approximately 15 m within the deepest channel if coincident with thalweg migration and lowering. The predicted scour depth under 1 in 100 (1%) AEP event varies from approximately 1.4 m along south alignment to approximately 14 m within the deepest channel if coincident with thalweg migration and lowering.

In bridge structural assessment, according to the project bridge structural designer, the full scour depth associated with 1 in 2000-year flood event was considered in combination with all other relevant loadings and combinations, except for the extreme events of seismic and vessel collisions. For seismic and vessel collision load combinations, in the absence of any specific requirement within the PSTR, the requirements of AASHTO LRFD 202012 were followed by the project bridge structural designer and one half of the 1-in-2000-year scour depth was considered in accordance with these extreme events (Refer to article 3.4.1 and 3.14.1 of AASHTO LRFD 202012 and commentary). In the global structural analysis of the bridge with lower bound soil stiffness provided by the project geotechnical designer, the stiffness and passive limit of horizontal soil springs supporting the piles took account of the removal of top layers of soil due to the scouring.

1. **Seismic and liquefaction effects**

It is critical to have assessed potential impact of seismic and liquefaction hazards to the new Derwent River bridge foundations in order to enable design of the foundation piles according to the PSTR adequately. The assessments were undertaken based on peak ground accelerations (PGAs) for a 1/1000 AEP (Annual Exceedance Probability), which represents the seismic ULS event. Although not a requirement, PGAs for a 1/200 AEP event were also assessed to compare against the ULS case, and to understand the level of shaking required to trigger liquefaction and cyclic softening. Subsoil class, soil behaviour / susceptibility, liquefaction potential, cyclic softening / displacement, lateral spread, and 1D site response were assessed based on laboratory test data (Atterberg limit test results), cone penetration test results (CPTs) at pier locations and inferred in-between piers based on ground model, shear wave velocity from cone penetration tests with in-built seismic module (sCPTs) and inferred correlations from CPTs, undrained shear strengths from field vane shear tests and shear wave velocity from CPTs. Subsoil classes were assessed to be Class C for onshore piers and Class D for offshore piers. Such classifications were communicated timely to the project bridge structural designer for superstructural design. Peak ground accelerations were assessed to be 0.13g and 0.15g for Class C and Class D, respectively under ULS conditions.

Table 3 below presents assessed potential of liquefaction/cyclic softening, cyclic displacement and lateral spreading. The seismic induced deformation results shown in Table 3 below are the estimated maximum deformations; and the displacement profiles would vary from pier location to pier location as the magnitude of lateral spreading depends on the distance from the free face.

For the seismic assessment of the foundation piles, the loading cases presented in Table 4 below and associated soil conditions / parameters based on guidance provided within Section 6.3.5 (i, ii & iii) of the New Zealand Bridge Manual13 were adopted in lieu of relevant Australian Standards.

Reductions in shear strength attributed to liquefaction / cyclic softening were reflected in Young’s modulus and p-y curves for lateral support to the foundation piles. The impact to the foundation piles due to the seismic induced lateral displacements were assessed using design software L-Pile.

***Table 3: Summary of key assessment results from seismic analysis***

|  |  |
| --- | --- |
| Item | Key assessment results |
| Potential of liquefaction / cyclic softening | * The likelihood of liquefaction was assessed to be very low during the 1 in 200-year AEP event. * The likelihood of liquefaction was assessed to be low under ULS shaking as a widespread phenomenon due to the predominately clay like materials present. Sand layers were identified within the CPT traces, but largely at depth. * Cyclic softening of very soft clay layers could be expected under ULS shaking (for cone tip resistance qc ≤ 0.3 MPa). * Under ULS seismic loading, post-earthquake, free field consolidation settlements of up to 120 mm have been calculated because of liquefaction. * Surface manifestation liquefaction, such as ejecta or sand boils, is not generally expected. An exception could be at CPT-02 location near existing causeway which is over 100 m away from the new bridge alignment, where there are loose sand lenses close to the ground surface, but this is likely to have limited impact on the proposed structure. |
| Cyclic displacement | * There is predicted to be negligible cyclic displacement under a 1 in 200 years seismic event. * Due to the very soft nature of the soft estuarine materials, large cyclic displacement (>1 m) was assessed for some locations. In general, the displacements range from 400 mm to 900 mm across all piers. |
| Lateral spreading. | * During the ULS return interval of 1 in 1000 years, a lateral spreading of up to 250 mm near the riverbanks based on the seismic displacement assessment. * Similarly, at the deep estuary area based on the seismic displacement assessment, a lateral spread of up to 400 mm was assessed. |

***Table 4: Summary of pile loading cases to consider potential seismic / liquefaction impacts***

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Case | Condition | Pile loading | Cyclic displacement | Lateral spread |
| A | Dynamic – During earthquake prior to liquefaction | 100% of inertial loading | No | No |
| B | Dynamic – During earthquake post liquefaction | 80% of dynamic loading | Yes | No |
| C | Dynamic – Immediately after earthquake | 25% of dynamic loading | No | Yes |

1. **Design assessments and outcomes**

As mentioned above, an iterative process was implemented between both the project bridge structural designer and the project geotechnical designer to obtain structural loading acting on the cut-off level of the bridge foundation piles. Ground spring stiffnesses in both vertical and lateral directions under static and dynamic conditions as well as p-y curves were assessed and provided to the project bridge structural designer for their three-dimensional assessment for each individual bridge foundation. Both lower bound and upper bound soil and rock spring stiffnesses were considered for the normal static load combinations. The lower and upper bound values of spring stiffness were based on 50% and 200% of the mean stiffness from the geotechnical assessment. The effect of scouring described above was included in the lower bound spring stiffness set but excluded in the upper bound set to cover the full range of ground restraint to the piles according to the project structural designer. Design rock socket length of foundation piles were iterated and finalized with detailed recommended founding toe level of each foundation pile presented in Table 5 below.

From Table 5, the pile toe levels vary from RL-8.0 m AHD / RL-9.0 m AHD at the northern abutment to RL-33.5 m AHD / RL-43.0m AHD at the southern abutment to RL-87.5m AHD for Pier 8 left pile in the middle of the River Derwent. The variations in pile toe levels presented in Table 5 were attributed to many different factors such as type of bridge superstructure (i.e., single large box girder for each carriageway), type of bridge foundation (i.e., monopile), bridge spans, loadings, founding materials, scouring effects and seismic effects etc.

The key contributing factors were founding materials, type of bridge superstructure and foundation selected. Both scouring and seismic effects were found to be not governing contributing factors. Rock socket lengths for most foundation piles were assessed to be greater than three times pile diameter with few piles that has rock socket lengths as short as 2.5 times pile diameter in slightly weathered basalt breccia with high strength. The longest estimated rock socket length was socketed within the Tertiary landslide breccia layer with extremely low to very low strength at Pier 7 right side with a ratio of the rock socket length over the pile diameter to be approximately 21.

Pile load-settlement was assessed using a few different methods as a trial, such as Rowe and Armitage method, Williams, Johnston and Donald method, a commercial software of SHAFT, PLAXIS 2D (axisymmetric) and PLAXIS 3D. It was found to be quite time consuming to assess pile load-settlement using PLAXIS 3D for each individual pile. Pre-assessments were carried out using both PLAXIS 2D (axisymmetric) and PLAXIS 3D for selected piles in order to compare analysis results obtained from both software packages. A trial-and-error approach was adopted to calibrate PLAXIS 2D (axisymmetric) in terms of inputs and interface factors to be adopted. PLAXIS 2D (axisymmetric) was finally adopted with a great confidence as design tool to assess pile load-settlement with maximum pile head settlements reported in Table 5 below for each foundation pile under SLS loading condition. Based on Table 5, 38 foundation piles out of 46 have estimated pile head settlements reported to be less than or equal to 20 mm under SLS loading condition. The maximum estimated pile head settlement was reported to be only 30 mm under SLS loading condition with approximately 60% of that to take place during the construction phase prior to final stitching of both carriageways together. A portion of that maximum settlement was attributed to elastic shortening of the pile with a long pile length. Most importantly, the design of bridge superstructure had taken into consideration of anticipated variable pile settlement at each pile location.

The pile response to lateral loading was assessed in both structural and geotechnical non-linear models. Lateral springs constant and limiting p-y curves developed for each soil and rock layers were supplied to TGP as inputs to structural global model. Piles were designed to cater for the lateral responses under ultimate design load cases, i.e., bending moments and shear forces. Similarly, the superstructure and substructures account for the lateral movements under the most adverse serviceability load cases. Given that the abutment/pier bored piles are spaced at least five times the pile diameter, group interaction effects are considered to be insignificant. The response of the individual pile subject to lateral loads was thus assessed using the commercial software L-PILE, which is suitable for single pile analysis. Nonlinear soil-resistance (p-y) curves for static, during earthquake ('dynamic'), post-earthquake ('softened') and collision ('dynamic') cases were developed for each soil layer at each representative geological zone.

PLAXIS 3D modelling had also been carried out to assess the forces in pile under selected representative critical load cases for the purpose of validating the results from the structural model. The second order effect from P-Delta has been accounted for in the PLAXIS 3D calculation phase by selecting the updated mesh option.

Results from non-linear geotechnical analysis (i.e. forces in pile and deflections) using PLAXIS 3D appear to be comparable to the results from the structural model, confirming the compatibility of the vertical and horizontal springs and limiting p-y curves supplied to TGP for structural modelling.

***Table 5: Summary of pile head settlement and pile toe level for each foundation pile***

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Pile # | Settlement (mm) | RL (m AHD) | Pile # | Settlement (mm) | RL (m AHD) | Pile # | Settlement (mm) | RL (m AHD) |
| Abut A-L | 12 | -43.0 | P8L | 30 | -87.5 | P16L | 8 | -35.0 |
| Abut A-R | 14 | -33.5 | P8R | 18 | -63.5 | P16R | 8 | -34.0 |
| P1L | 16 | -57.0 | P9L | 20 | -70.0 | P17L | 8 | -36.8 |
| P1R | 12 | -55.0 | P9R | 18 | -73.5 | P17R | 8 | -31.5 |
| P2L | 20 | -63.0 | P10L | 19 | -58.8 | P18L | 8 | -31.6 |
| P2R | 21 | -70.0 | P10R | 16 | -76.0 | P18R | 7 | -30.6 |
| P3L | 9 | -36.2 | P11L | 22 | -75.0 | P19L | 7 | -29.8 |
| P3R | 11 | -42.6 | P11R | 15 | -61.0 | P19R | 6 | -25.5 |
| P4L | 11 | 46.5 | P12L | 21 | -60.5 | P20L | 4 | -19.1 |
| P4R | 17 | -72.0 | P12R | 21 | -61.7 | P20R | 3 | -13.3 |
| P5L | 6 | -31.3 | P13L | 13 | -52.5 | P21L | 3 | -12.7 |
| P5R | 7 | -33.2 | P13R | 13 | -54.5 | P21R | 3 | -9.5 |
| P6L | 19 | -69.0 | P14L | 13 | -40.0 | Abut B-L | 3 | -8.0 |
| P6R | 21 | -80.5 | P14R | 12 | -40.0 | Abut B-R | 3 | -9.0 |
| P7L | 21 | -75.0 | P15L | 7 | -33.2 |  |  |  |
| P7R | 25 | -82.0 | P15R | 7 | -33.5 |  |  |  |

Notes: RL = Reduced level; AHD = Australian Height Datum; Abut A = Southern Abutment; Abut B = Northern Abutment; L = Left; R = Right; P5L = Pier 5 Left Pile; P5R = Pier 5 Right Pile; Pile head settlement is the estimated settlement at pile cut-off-level under SLS loading condition.

Figure 4 below presents plots of induced bending moments obtained from geotechnical assessment and structural modelling for Pier 9 left pile as a comparison under 4 different critical loadings (i.e., maximum horizontal force, Fy in longitudinal direction, maximum horizontal force, Fz in transverse direction, maximum moment, My in-transverse direction and maximum moment, Mz in longitudinal direction).

Figure 5 below presents pile load-settlement curve for Pier 9 left pile. Geotechnical models and associated design parameters for Pier 9 lift pile are presented in Table 6 below.

***Figure 4: Plots of induced bending moments versus depth for Pier 9 Left Pile***



***Figure 5: Plot of pile load versus pile head settlement for Pier 9 Left Pile***



***Table 6 - Summary of geotechnical model and assessed design parameters for Pier 9 Left Pile***

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| Reduced Level (RL, m AHD) | | Materials | UCS\* or su (MPa) | fs# (kPa) | py## (MPa) | Ev### (MPa) |
| -0.60^^ | -25.00 | Clayey SILT / Silty CLAY / Sandy CLAY (very soft to soft / loose) | - | - | - | - |
| -25.00 | -33.70 | SAND / GRAVEL with clay and 2 sections of core loss (very loose to loose to dense) | - | 27 | 0.75 | 15.8 |
| -33.70 | -36.70 | Gravelly CLAY / Clayey SILT / Clayey SAND (dense / hard) | 0.12\*\* | 43 | 1.00 | 15.9 |
| -36.70 | -37.20^ | SILTSTONE / SANDSTONE / BRECCIA (very low to low strength) | 1.32 | 130 | 2.73 | 125.0 |
| -39.00 | -46.90 | SILTSTONE / SANDSTONE / BRECCIA (very low to low strength) | 1.32 | 260 | 2.73 | 125.0 |
| -46.90 | -59.50 | SANDSTONE / BRECCIA with 3 sections of soil like material (low to medium strength / hard) | 1.78 | 315 | 3.13 | 155.1 |
| -59.50 | -61.30 | SANDSTONE / BRECCIA (very low to low to medium strength) | 2.54 | 310 | 3.50 | 152.8 |
| -61.30 | -65.80 | BRECCIA with 1 section of soil like material and 1 section of core loss (low to medium strength / hard) | 2.53 | 345 | 3.50 | 102.3 |
| -65.80 | -68.80 | Sandy CLAY with gravel (very hard) | 0.20\*\* | 100 | 1.80 | 40.0 |
| -68.80 | -73.80 | SANDSTONE / BRECCIA with 1 section of soil like material (low to medium strength / hard) | 2.93 | 310 | 3.50 | 153.7 |
| -73.80 | -76.80 | Gravelly CLAY with sand trace cobbles with 1 section of soil like material and core loss (very hard) | 0.25\*\* | 125 | 2.15 | 38.1 |
| -76.80 | -91.80 | BRECCIA (low to medium strength) | 2.13 | 350 | 3.45 | 203.3 |

Notes: \* - UCS = unconfined compressive strength for rocks; \*\* - undrained shear strength value of su for fine grained materials; # - shaft skin friction; ## - limiting passive resistance; ### - vertical Young’s modulus; ^ - nominated steel case toe level; ^^ - the existing seabed level.

From Figure 4, the induced bending moments of P9 left pile were assessed using PLAXIS 3D with consideration of non-linear characteristic behaviour to be less than the estimated bending moments obtained structural model at depth within sandy / gravelly layer. The load-settlement curve was assessed to be in linear behaviour with loading up to ULS load as presented in Figure 5. This implies that both shaft and base of Pier 9 left pile will most likely behave within elastic range even under ULS loading condition.

# Design validation using Osterberg cell (O-Cell) testing

Three (3) bored test piles with a minimum diameter of 1.5 m were designed, installed strategically in the vicinity of the representative pier groups (see Figure 1) and performed successfully in terms of meeting PSTR’s requirements and validation of design approaches, assumptions, construction methodologies adopted. The design approaches and assumptions presented above in this paper were proved to be appropriate through test piles. This is because all three successful test piles mobilized greater ultimate shaft skin frictions and end bearing resistances for various targeted soils and rocks than anticipated in general. The assumed Young’s modulus for various soils and rocks were tested to be higher than expected however still within 50% to 200% sensitivity range adopted by bridge structural designer (see Figure 6 below). In addition, all three successful tests were loaded to more than 2 x highest ULS (i.e., Ultimate Design Action Effect, S\* in AS2159 and AS5100.3 terminology) or 3 x working loads of production piles, whichever is greater, with half of this load upwards and half load downwards.

As part of construction methodologies, using bio-degradable polymer as drilling fluid to support open shaft within weak rocks; cleaning low part of shaft and shaft base using cleaning bucket and agitator pump at the shaft base, departing from validating shaft roughness “pile by pile” through applying conservative shaft skin friction resistance coefficient to assess shaft skin friction for rocks had been proven to be effective. Due to limited space, more details related to design and testing of test piles using O-Cell method will be presented in a separate paper to be published in future.

***Figure 6: Plots of O-Cell movements versus test loads for Test Pile 1***



# Discussions and conclusions

A key element in the delivery of the foundation systems for the New Bridgewater Bridge was a high degree of co-operation between the design and construction teams, including a continued involvement of the designers during construction. This allowed adoption of observational techniques to modify designs if needed in response to observed ground conditions and test results. The ultimate result is a series of design solutions to varying ground conditions which achieved a robust foundation system for the bridge while facilitating constructability.

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