Case study: Geotechnical design of bored piles in rock for the Kawarau Falls Bridge

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Keywords: Piles, Queenstown, bridge, schist, elastic continuum method

ABSTRACT

The two-lane bridge on State Highway 6 (SH6) at Kawarau Falls near Queenstown is scheduled to open in the second half of 2017. Designed to withstand a 1/ 2500 hazard level earthquake, the new bridge will provide better access into the Queenstown area and better account for closures and traffic congestion than the existing iconic bridge.

The Structural Designers adopted a two-group pile system for the bridge abutments and five monopile piers to support the deck spanning across the Kawarau River. Given the large loading demands and the anisotropic schist bedrock, the lateral pile performance governed the geotechnical design of the rock socket conditions.

The Elastic Continuum Method (ECM) was utilised for the pile-rock analysis. In this paper, the authors outline their design approach and compare their results with those derived from the Finite Element Method (FEM). The authors discuss the merits and limitations of each method and provide a framework for lateral pile analysis work.

1 PROJECT INTRODUCTION

Queenstown is important to the New Zealand tourism industry and its local population also continues to grow. Over 8,000 vehicles per day cross the existing iconic single-lane Kawarau Falls Bridge which was originally built in 1926 as a dam structure to help with the recovering gold in river below the lake. To alleviate traffic congestion, the New Zealand Transport Agency (NZTA) proposed a new 250 m long and 14 m wide two-lane bridge to replace the existing bridge as the main carriageway.

A two-group pile system was adopted for the north and south abutments and five monopile piers were adopted to support the deck spanning across the Kawarau River. The piers were embedded directly into rock whereas the abutment piles were embedded in lake sediments and glacial till and socketed in rock. The minimum rock socket depths were 5 m for the abutment piles and 7 m for the piers. For Pier 2 (P2) a minimum 8 m rock socket was recommended as this pier had the longest free head length (approximately 17 m). For brevity, the analysis presented in this paper is for P2 only.

2 GEOLOGICAL AND SEISMIC SETTING

Queenstown and the surrounding Wakatipu Valley are underlain by schist of the Caples and Rakaia Terranes. The schist is actively being folded, faulted and eroded in response to regional compression and strain distributed across the mid to lower South Island. Geologists suggest that much of the fault activity and uplift in the area has occurred over the past five million years, and that the ongoing high-stress folding is reflected in the ruggedness of the local mountain ranges.

Repeated glaciations have carved deep troughs into the landscape, with Lake Wakatipu occupying the largest trough in the area. Lake Wakatipu covers an area of approximately 290 km^2 and is over 370 m deep. Furthermore, during the last glacial (Otira) period, the glacial lake covered a much larger area and was significantly deeper.

There is a schist rock outcrop in the middle of the Kawarau River. However, the steeply dipping folded rock meant that that the top of rock elevation varied at each pile location.

The nearby mapped active faults – the Alpine Fault, Nevis Fault and Cardrona Fault – are all capable of producing earthquakes with moment magnitudes (M_w) greater than 7. According to a recent seismic hazard study undertaken by the Otago Regional Council for the Queenstown Lakes District, the primary seismic hazard facing the region is an Alpine Fault earthquake; which is predicted to have a 30% probability of rupturing in the next 50 years.

A 1/ 2500-year return period design peak ground acceleration (α_{max}) of 0.83 g was adopted for the site Class B (Rock) conditions.

3 GEOTECHNICAL PROPERTIES OF SCHIST BEDROCK

The schist bedrock is technically referred to as a 'quartzofeldspathic grey schist and green schist'. It is moderately weathered to unweathered along the pile socket depths. By observing the retrieved core specimen, the platy structure of the rock allows for it to break more readily in the lateral direction (parallel with the schistosity) than in the vertical direction.

The index properties of the schist are outlined below in Table 1 and a photo of the retrieved specimen is shown in Figure 1.

Table 1: Schist Index Properties

Property	Value
Unit weight, γ	27.5 kN/m^3
Intact unconfined uniaxial compressive strength, σ_{ci}	20 – 40 MPa
Rock quality designation (RQD)	10 – 95 %
Intact rock modulus, E_i	7 – 14 GPa
Rock mass modulus, E_m	2.9 GPa
Rock mass modulus parallel with schistosity, E_{m1}	0.6 – 3.7 GPa
Rock mass modulus perpendicular to schistosity, E_{m2}	1.9 – 4.7 GPa
Rock mass rating (RMR)	26 - 50
Geological strength index (GSI)	21 – 45

The rock mass shear strength was estimated using the Hoek and Brown failure criterion (edition 2002).

Figure 1: Photo of typical retrieved core specimen showing foliation patterns

4 PILE DESIGN METHODOLOGY & RESULTS

4.1 Choice of ECM over p-y curves method

The geotechnical analysis of the bridge foundations was carried out using the ECM (by means of in-house Coffey software), as presented in Poulos & Davis (1980). The pile is modelled as a thin elastic strip and the geotechnical layers are modelled as a continuous medium and are assumed to have an elastic stress-strain behaviour. Therefore, the load-deformation pile response output is for small-strain ground behaviour. This method adopts a simplified boundary element approach by limiting the axial and lateral ground pressures to ultimate values specified by the Designer.

The ground reaction-pile displacement (p-y curves) analysis method is the most widely used method for laterally loaded pile design. The ground reaction–pile displacement response is represented by a series of independent bi-linear springs so that at each depth the ground reaction is a non-linear function of lateral pile displacement. This non-linear relationship is represented by the differential equation proposed by Hetenyi (1946):

$$EI.\ d^4y/dx^4 + P_x.d^2y/dx^2 - p - W = 0 \qquad (1)$$

Where: EI = pile flexural stiffness, P = axial load on pile, p = lateral ground reaction (per unit length), w = distributed load along length of pile. The integration of this original formula then produces profiles for shear (V), bending moment (M), bending slope (S) and lateral displacement (y).

Over the years, full-scale lateral load tests on various types of soils have led to established p-y curves being adopted for soil but only a limited number of load tests have been carried out for rock strata. The most commonly used p-y curve rock is the one proposed by Reese (1997) for bored piles in ‘weak rock’ (σ_{ci} = 0.5 – 5 MPa). Although this approach is commonly used within the industry, it was not adopted as the primary lateral pile design methodology for the Kawarau Falls Bridge due to its limitations and sources of uncertainty.

These sources of uncertainty are:

- It is based on intact rock strength (σ_{ci}) and does not account for rock mass strength, which is important given the schist anisotropy.
- It is based on a limited number of full-scale load tests – one carried out on a brittle coral limestone in Florida and another carried out on sandstone in California. And further, these load tests were for 1.2 m and 2.25 m diameter piles. In reality, the lateral ground reaction depends on the size (diameter) of the foundation.
- It represents an elastoplastic deformation behaviour. The schist rock at Queenstown, being a harder material, is expected to have a more brittle and bi-linear deformation behaviour.

The theoretical advantage of the ECM over the p-y curves method for rock is that the p-y curves method is cruder due to the spacing between the lateral springs acting on the pile.

4.2 Geotechnical strength reduction factor

The geotechnical strength reduction factor (Φg) for pile foundations was derived using the risk-based methodology set out in the Australian piling standard (AS2159-2009) and was calculated to have a value of 0.52.

4.3 Modelling of the pile free length

The free length of the monopiles was modelled as a “virtual layer” (i.e. minimal stiffness values input [E = 0.001 MPa]) because the ECM software program requires input parameters for every layer along the pile length.

4.4 Modelling of pile head and pile toe conditions

As there is no true free head or fixed head condition, both head conditions were modelled to provide a displacement range for the structural modelling to account for. To adopt a reasonable rock socket depth, the toe condition was not fixed against rotation but was fixed against translation (i.e. pinned). Poulos (1972) suggests the pinned toe condition be adopted to model a pile bearing on (but not socketed into) rock. Poulos (1972) initially proposed modelling a rock socketed pile by completely fixing the toe against rotation and displacement. However, for this particular design case, the pinned toe model was adopted to ensure that the stiffness of the surrounding rock was suppressing the pile rotations and bringing the system to equilibrium.

4.5 Analysis of P2

The input parameters for P2 are outlined in Table 2.

Table 2: P2 Analysis Input Parameters

Property	Value
Vertical load at pile head	22.03 MN
Horizontal load at pile head	0.6 MN
Moment at pile head	10.3 MN.m
Concrete pile modulus, E_{pile}	28 GPa
Overall rock mass modulus, E_m	1.9 GPa
Schist skin friction pressure, f_s	0.9 MPa
Schist end bearing pressure, f_b	21.5 MPa
Schist lateral yield pressure, P_y	5.4 MPa (for upper 1.5 m) 10.7 MPa (for the rest)

To account for the anisotropic behavior in relation to lateral pile performance;

- A lower bound E_m value was adopted for design.
- A reduced lateral yield pressure value within the top 1.5 m of rock was adopted due to the increased likelihood of failure along pre-existing fractures at low overburden pressures.

The results of the maximum pile actions for both the fixed head and free head conditions is presented in Table 3.

Table 3: P2 Maximum Pile Actions

Pile Action	Fixed Head	Free Head
Bending moment (MN.m)	6	20
Lateral displacement (mm)	12	120
Shear force (MN)	1	5

5 EVALUATION – COMPARING ECM WITH FEM

To further evaluate the use of the ECM, the authors comparatively assessed the P2 pile response using a Mohr-Coulomb constitutive model with 3D FEM software. In addition to the elastic properties of the pile and rock, using the FEM software required the rock shear strength properties and unit weights of the rock and the pile be input into the model. A concrete unit weight of 25 kN/m3 was assumed for the pile.

The meshed 3D model showing a free head pile embedded into the ground is shown in Figure 3 and the comparative assessment of the results is shown in Figure 4.

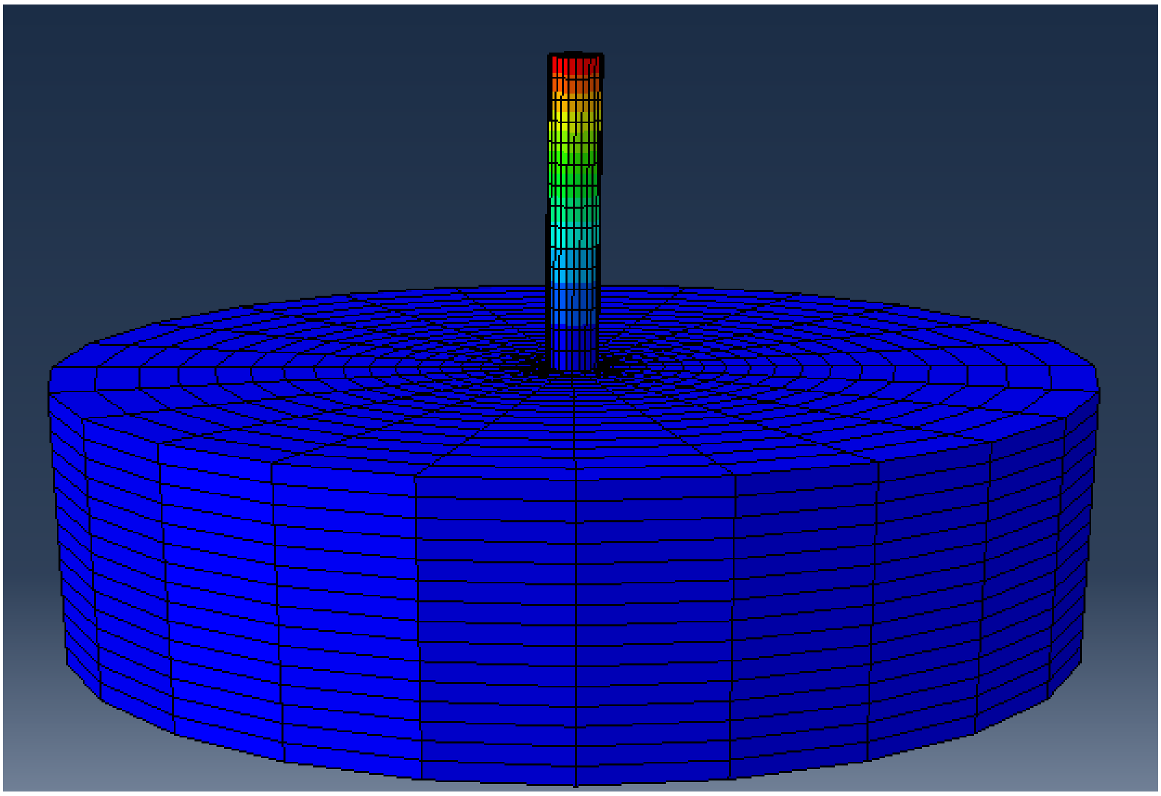

Figure 3: P2 meshed model and for free head pile socketed in rock

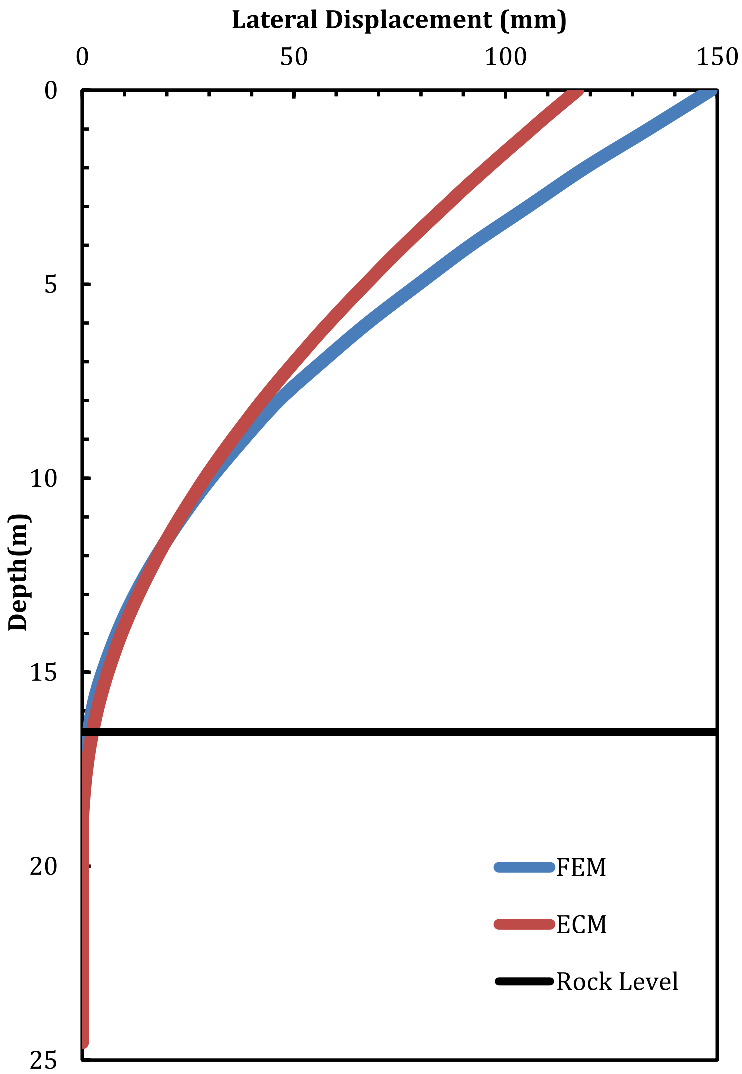

Figure 4: Comparison of ECM with FEM for P2

The results in Figure 4 show that the pile-rock interaction along the rock socket length are almost identical and that the differences between the 1D and 3D analyses along the free head length lie within reasonable limits.

The ECM and FEM are similar in that both methods consider the geotechnical and structural failure of the piles. While they do not account for all the structural details (e.g. the presence and configuration of pile reinforcement), they do provide a reasonable assessment of how the structural element interacts with the surrounding ground. The limitation of FEM, in this particular case, is that it could only model a free head condition because no other superstructure element was constructed in the model to restrain the head movement. However, this limitation also demonstrates an advantage of FEM in that if the ground-structure interaction was modelled for the entire superstructure rather (as opposed to being limited to a single pile element), this would better indicate the actual pile actions at each location. However, the use of the ECM to model a single pile element with both head conditions is a quicker process.

6 DESIGN PROCESS RECOMMENDATIONS

Given the complex pile-rock interaction, an iterative collaborative process between Geotechnical and Structural Engineers is needed throughout the design process for laterally loaded piles. It is recommend that preliminary geotechnical modelling be carried out on trial design lengths, pile diameters, rock socket lengths, etc. Based on the subgrade reaction profile, the Geotechnical Engineers then provide lateral spring values for the Structural Engineers to model the superstructure (and revise loads where necessary). The iterative process is repeated until both the geotechnical and structural models provide comparable pile displacement behaviour (e.g. values within a 10% range) where the pile structural capacity and the geotechnical capacity of the ground have not been exceeded.

7 CONCLUSIONS

Given the complex pile-rock interaction, an iterative collaborative process between Geotechnical and Structural Engineers is needed throughout the design process for laterally loaded piles. Based on this case study, the authors consider that utilising the ECM for piles socketed in rock provides reasonable results when the applied lateral ground pressures are not exceeding the given lateral yield pressure. Further, the results of the ECM are comparable with those of the FEM. Alternatively, more full-scale field load tests on various rock types are recommended and the subsequent p -y curve parameters should be related to the rock mass properties, not only intact rock properties.

8 ACKNOWLEGEMENTS

The authors wish to acknowledge the NZTA and McConnell Dowell Contractors.

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