Proc. of the 8th International Symposium on Geotechnical Safety and Risk (ISGSR)
Edited by Jinsong Huang, D.V. Griffiths, Shui-Hua Jiang, Anna Giacomini and Richard Kelly
 Published by Research Publishing, Singapore.
doi: 10.3850/978-981-18-5182-7_05-001-cd

ISGSR 2022

Design of Rock Socketed Piles in Complex Geological Environments: Two Case Studies from Central and North Queensland

Katherine Farry[1], Natasha Joynson[2] and Sharath Kudumula[3]

[1]Jacobs Engineering Group, Brisbane, Australia.
E-mail: katherine.farry@jacobs.com
[2]Jacobs Engineering Group, Townsville, Australia.
E-mail: natasha.joynson@jacobs.com
[3]Jacobs Engineering Group, Hyderabad, India.
E-mail: sharath.chandra@jacobs.com

Abstract: It is necessary to review usual practices and design approaches for bored pile foundations in complex geological environments. This is warranted to be able to produce a robust, economical design which provides a value for money solution. There are many factors and constraints influencing the chosen project site and often there is insufficient information in the planning stage of the project to determine the extent of the geological site complexity. This paper will describe some of the approaches that may be adopted for bored pile design socketed into rock where geological complexities and uncertainties are encountered. Case studies from Central and North Queensland will be discussed and an approach that was adopted in rock socket design to address challenging ground conditions. Influence of various rock parameters such as unconfined compressive strength, geological strength index and rock mass modulus will be discussed in this paper.

Keywords: Rock socket; complex geology; uncertainty; design methodology; bored pile foundations; risk.

1 Introduction

Complex geological environments can present challenges to design of bored piles socketed into rock. It is important to understand and address the various geotechnical risk factors when developing the ground model. This paper will discuss some of the risks associated with piling in complex geological environments and the approaches that may be adopted for rock socket pile design.

Pile design challenges in complex geological environments will be discussed with reference to two case studies; the first in Central Queensland where karstic limestone was encountered; and the second in North Queensland where microdiorite and granodiorite have been intruded by secondary mafic dykes of variable strength and weathering. Ground model parameters for rock socket design, such as unconfined compressive strength, geological strength index and rock mass modulus, which are inputs for the development of the ground model, will be described, along with strategies to reduce the level of risk due to geological uncertainties. Finally, the rock socket design approach and the methodologies adopted for the case studies will then be described.

2 Geology

The subsurface geology within which piles are to be founded can influence pile design, construction, and performance. The level of understanding of the various geological properties that inform the ground model are essential for a robust design. Two different complex geological environments which were encountered in the Central and North Queensland case studies, are described in the following sections.

2.1 Karstic limestone (Central Queensland case study)

In the Central Queensland case study, the geology comprised of the Rockhampton Group and Mount Alma Formation (Cr, DCa), from the late Devonian to early Carboniferous age. This formation contains complex folding and faulting repetitions of the two groups, with the Mount Alma Formation/l (DCa/l), comprising fossiliferous limestone and marble and the Rockhampton Group/l (Cr/l) containing oolitic limestone and calcareous sandstone.

Limestone is a sedimentary rock composed wholly or predominantly of calcium carbonate. As a foundation material, limestone differs from other rocks in that voids (karsts) may be found at almost any depth within the rock mass. Karst features are common in limestone, which are characterised by sinkholes, caves, voids formed from dissolution of the limestone, weathered zones, and/or large underground drainage systems. As such, limestone can be problematic to design, if identified as karstic, due to difficulties in identifying these features in the subsurface. The application of geophysical methods such as seismic refraction can be used to supplement borehole information as they provide a continuous subsurface record of the extents of karstic features.

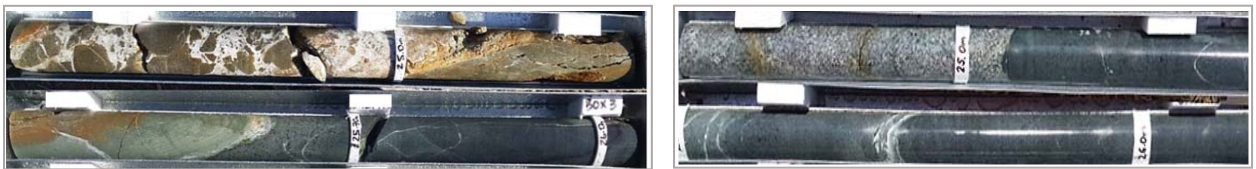
During the planning phase of the case study from Central Queensland limestone at several locations were identifed. As part of the design process additional site investigations were undertaken which identifed that it was unlikely that the geotechnical conditions included an advanced karstic environment, however there were potential solution features identified (Refer Figure 1). This consisted of material weathered to such an extent that it displays soil properties, evidence of a smooth surface at the soil/rock interface and lack of gradual weathering profile.

Figure 1. Example of solution weathering in limestone: slightly weathered limestone interbedded with extremely weathered seams, recovered as a soil, red-brown in colour (Central Queensland case study).

2.2 Intrusive igneous rocks (North Queensland case study)

In the North Queensland case study, the geology encountered within the project corridor consisted of intrusive rock types microdiorite and granodiorite belonging to the Cretaceous aged Wundaru Granodiorite (Kgwu). Microdiorite and granodiorite are both medium grained igneous rock types which form when magma intrudes through the earth's crust and lithifies upon cooling. Both rock types have a massive crystalline texture; and from a geotechnical design perspective in relation to the North Queensland case study, were considered to have the same rock mass characteristics. Plutonic rock masses are often subjected to later intrusive events which occur after the original intrusive body has cooled and lithified. This was evident in the North Queensland case study and occurred in the form of dolerite dyke and hydrothermal fluid intrusions which passed through existing joints and other weak seams which had formed in the microdiorite and granodiorite rock mass following lithification.

Dolerite is a fine grained, generally grey mafic intrusive rock composed of plagioclase feldspar, pyroxene, hornblende and minor quartz. The dykes were found to have a high variability in thickness, from as small as 10cm to several metres. Weathering and strength were also highly variable and ranged from extremely weathered rock with soil strength (UCS $\leq$ 200 kPa) properties to fresh, extremely high strength (UCS > 200 MPa) rock. Another geotechnical consideration identified in the boreholes, was the contact margin between the dykes and the main rock mass, were observed to occasionally be heat affected. Coupled with hydrothermal alteration of calcite rich fluids which also targeted these contact zones, the result was localised decreases in weathering and strength combined with increased fracturing when compared to the surrounding rock mass. The presence of these dyke and hydrothermal intrusions was found to be a risk to pile design, which were either presented as localised weak contact zones, or as hard fresh rock with healed joints (Refer Figure 2). Correlating the dykes between boreholes was challenging, as the borehole data indicated a high variability in dyke orientation.



Example 1 - Dolerite dyke and hydrothermal intrusion into microdiorite. The microdiorite has become brecciated and altered due to the secondary intrusions. Calcite seams have also weathered to clay in some places.

Example 2 – Dolerite dyke and microdiorite contact. Both rock types show little to no change from fresh rock.

Figure 2. Examples of variability of weathering, strength and fracture spacing at locations of secondary dolerite dyke and hydrothermal intrusions into microdiorite (North Queensland case study).

3 The Ground Model

The ground model is the fundamental basis of rock socketed pile design and provides information on the engineering characteristics of the rock mass. The availability of geotechnical data and how this is interpreted constitutes a major contributing factors to the robustness of the ground model. The ability to obtain highly reliable site-specific data is often influenced by factors such as project budget, borehole spacing and depth, type of borehole information, the degree of variability of geological features over short distances and the presence of localised geological anomalies, which may not be intersected by the boreholes or other field investigation techniques. Where direct test data is unavailable, empirical relationships based on published relationships are often used, along with local experience and engineering judgement. Ground model parameters for rock socket design typically include 1) Unconfined Compressive Strength (σ_c); 2) Geological Strength Index (GSI) incorporating Rock Quality Designation (RQD); 3) Joint Condition ($JCond_{89}$); and 4) Rock Mass Modulus (E_m).

3.1 Unconfined compressive strength (σ_c)

Unconfined compressive strength of rock (σ_c) is usually based on Uniaxial Compressive Strength (UCS) testing of selected borehole core samples. UCS testing is often accompanied by Point Load (PL) testing which is an effective option as it can provide similar data at lower cost. The point load strength index ($I_{s(50)}$) can then be used to establish the linear relationship $UCS = K \, . \, I_{s(50)}$ using conversion factor *K*.

Although there are typical published values of *K* that can be referred to in the absence of available or limited data, it is also noted that *K* varies considerably between different geological environments and ideally should be estimated from site-specific data through calibration with UCS test results on the same rocks from which the point load tests are performed. It is noted that AS1726 uses a *K* of 20 for a range of UCS and point load values from very low to extremely high strength rock.

Figures 3 and 4 show site-specific correlations between UCS and $I_{s(50)}$ for the limestone rock (Central Queensland case study) and microdiorite/granodiorite rock (North Queensland case study). For the Central Queensland case study, thirty (30) test results for slightly weathered, high to very high strength limestone were used to establish a zero-intercept regression equation of $\mathbf{UCS = 18.I_{s(50)}}$ (Refer Figure 3). Figure 4 shows the UCS and $I_{s(50)}$ plot of eighty-five (85) test results for highly weathered to slightly weathered microdiorite and granodiorite core samples from the North Queensland case study. It should be noted that for geotechnical design purposes, the intact strength properties of microdiorite and granodiorite were considered to be the same and therefore were used interchangeably for establishing rock strength parameters. The zero-intercept regression equation was estimated to be $\mathbf{UCS = 15.I_{s(50)}}$.

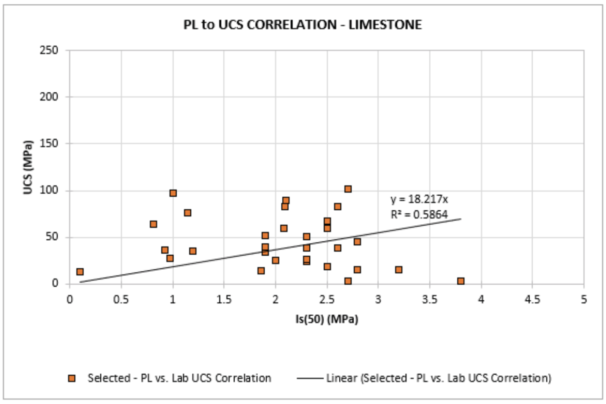


Figure 3. Point load strength index – UCS relationships. (Central Queensland case study).

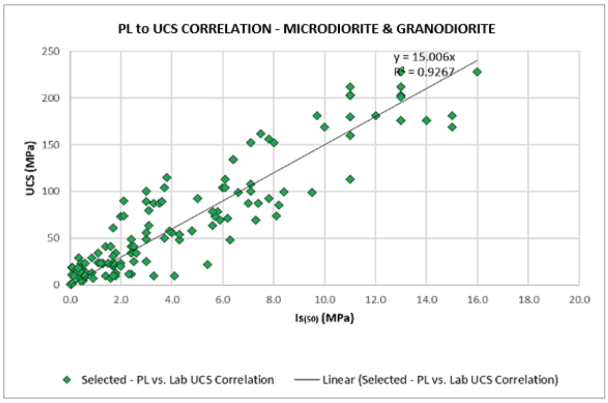


Figure 4. Point load strength index – UCS relationships. (North Queensland case study).

3.1.1 Design unconfined compressive strength

Establishing design unconfined compressive strength (design σ_c) for use in assessing pile capacity requires inputs from both quantitative and qualitative data sources. Quantitative data generally consist of rock strength test results (UCS and Point Load Strength), while qualitative data includes interpretations of strengths which form part of the borehole log descriptions.

One of the limitations to rock strength testing is the difficulty in testing very low strength rock as these rocks are often associated with high degrees of weathering and fracturing making sampling difficult. For a valid UCS test result, the sample specimen is required to be intact and without structural defects to yield a value that adequately represents the intact strength of the in-situ rock. This consequently requires more reliance on the field descriptions in boreholes logs and core photographs where it is not possible to recover suitable samples.

To account for the presence of weaker rock zones within the geological profile, typical values may be used in consultation with borehole log field descriptions. AS1726: 2017 – Geotechnical Site Investigations provides a range of typical UCS values and corresponding rock strength field descriptions. Very low strength rock ranges from 0.6 MPa to 2 MPa. For the North Queensland case study, the lower bound value of 0.6 MPa was used to represent zones of very low strength rock that were not able to be tested.

For the Central Queensland case study, in order to deal with karstic limestone and the possibility of encountering weathered seams or voids within the socket, a reduction was applied to the UCS value established. This was simplified to a percentage reduction to the UCS, depending on the thickness of seam likely to be encountered e.g., if a 600mm of weathered material was likely to be encountered in a 1.2 m long socket, then a 50% reduction to the UCS was applied. For cases where the strength of rock was greater than the concrete design compressive strength, the design UCS value was typically limited to the concrete strength, to limit stresses in the pile. Inputs into the design σ_c, include: 1) Direct UCS test results, 2) PL-UCS correlation values, 3) Typical values from published literature for untested weak zones, and 4) Account for weathered seams.

Figure 5 shows an example of a design UCS plot for the North Queensland case study. The design UCS line has been interpreted by assessing the UCS and point test data along with review of borehole log field descriptions.

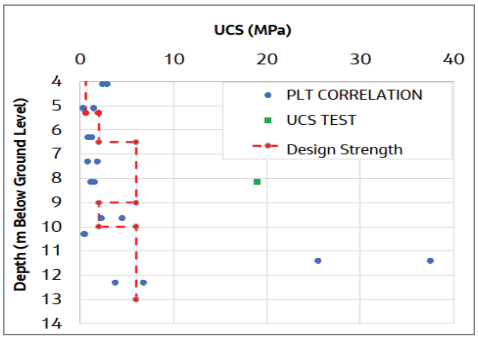


Figure 5. Example of design UCS line (North Queensland case study).

3.2 Geological strength index (GSI)

Geological Strength Index (GSI) is an estimation of rock mass strength properties. In rock socket design, one of its applications is to inform deformation parameters such as rock mass modulus (E_m), as discussed further in Section 3.3 below. Any rock mass characteristics to be derived at pile foundation level are interpreted or measured from cored borehole data. The two parameters of Rock Quality Designation (RQD) and Joint Condition ($JCond_{89}$) can be easily obtained in cored boreholes and can be used to estimate GSI using the following expression:

$$GSI = 1.5.JCond_{89} + \frac{RQD}{2} \tag{1}$$

3.2.1 Rock quality designation (RQD)

The Rock Quality Designation (RQD) was developed by Deere (1963) as a quantitative measurement of the quality of a rock mass. It is a standard parameter in drill core logging and forms a basic element of several rock mass classification systems. In cored boreholes, RQD is measured using the following equation:

$$RQD = \frac{\Sigma Length\ of\ sound\ core\ pieces > 100mm\ in\ length}{Length\ of\ core\ run} \text{ x } 100\ \% \tag{2}$$

3.2.2 Joint condition ($JCond_{89}$)

Bieniawski's 1989 joint condition ($JCond_{89}$) rating is estimated as the sum of individual ratings of key joint parameters (persistence, separation, roughness, infill and weathering) and ranges from 0 (soft gouge > 5mm thick or separation > 5mm, continuous) to 30 (very rough surface, not continuous, no separation, unweathered). Those individual parameters form part of the defect descriptions in cored borehole logs. The $JCond_{89}$ rating, once estimated, can then be input into the equation for GSI for a specific rock interval.

3.3 Rock mass modulus (E_m)

Rock mass modulus (E_m) is a key parameter used in bored pile design to represent the deformation characteristics of the foundation rock. Obtaining insitu rock mass moduli from direct testing can be cost prohibitive with designers often having to use published empirical relationships to assess E_m. Where empirical relationships are applied, inputs derived from site-specific data such as unconfined compressive strength (σ_c) and GSI should be used. Two common relationships for estimating E_m are provided in Sections 3.3.1 and 3.3.2.

3.3.1 Rowe and Armitage relationship

The relationship proposed by Rowe and Armitage (1987) is as follows:

$$E_m = 215(\sigma_c)^{0.5} \tag{3}$$

The design rock mass modulus can be reduced by a factor of 0.7 to allow for spatial variability in rock properties, as recommended in Rowe and Armitage (1987).

3.3.2 Hoek and Brown relationship

The Hoek and Brown (1998) relationship allows for GSI to be incorporated, which takes into consideration RQD and the joint condition as described in Section 3.2:

$$E_m = \sqrt{\frac{\sigma_c}{100}}\, 10^{\left(\frac{GSI-10}{40}\right)} \tag{4}$$

3.3.3 Modulus reduction ratio

RQD from borehole data is usually the only measurement available for representing rock discontinuities at depth. Using data from several case histories, Bieniawski (1984) established empirical correlations between RQD ranges and modulus reduction ratios E_m/E_i where E_m is the insitu rock mass modulus and E_i is the laboratory tested intact modulus. Typical ranges of reduction ratios are between 0.15 (for RQD 0-50%) and 0.7 (RQD >90%) of equivalent laboratory tested values for rock with the same rock mass characteristics. Modulus reduction ratios were applied to the calculation of design E_m for both case studies using laboratory modulus values from site-specific boreholes.

4 Rock Socket Design Approach

The design of rock socketed piles requires evaluation of axial and lateral pile capacities and corresponding settlement and deflection. Both case studies refer to transport infrastructure projects managed by the Queensland Department of Transport and Main Roads (DTMR) and therefore require compliance to DTMR's Geotechnical Standard – Minimum Requirements and relevant Technical Specification MRTS63 Cast-In-Place-Piles, with the pile design methodology described in the following sections.

4.1 Axial analysis

Calculation of the pile socket length was estimated using the design method of Pells (1999) incorporating the work done by Rowe and Armitage (1984). The main design outputs were determination of the shaft adhesion and end bearing resistance.

4.1.1 Shaft adhesion

For shaft adhesion, the following relationship was adopted for the socket design, in both case studies:

$$f_s = \alpha\, \beta\, \sigma_c \tag{5}$$

Where α = Reduction Factor, from Figure 2 in Pells (1999); β = Modulus Reduction Factor, from Figure 4 in Pells (1999).

The reduction factor, α, is a factor in relation to side shear resistance, which looks at a correlation between socket roughness and unconfined compressive strength. The modulus reduction factor, β, is essentially a reduction factor that accounts for the difference in rock mass stiffness (refer Section 3.3.3). Further to this, a minimum 30% reduction to shaft friction was adopted, for piles not able to be cast in dry conditions, as the design assumptions may not be fully verified by inspection, during construction.

4.1.2 End bearing resistance

Two methods were considered for pile end bearing; Rowe and Armitage (1987) and Zhang and Einstein (1998).

Rowe and Armitage (1987):

$$f_b = 2.5 \text{ x } \sigma_c \tag{6}$$

Zhang and Einstein (1998):

$$f_b = a_b\, x\, \sqrt{\sigma_c} \tag{7}$$

(a_b = 3.0 adopted for rock with very low to low strength or fractured rock and a_b = 4.8 was adopted for rock with medium or higher strength).

For the Central Queensland case study, generally the lower bound of the two equations was adopted in the design. However, for karstic limestone, end bearing was ignored altogether in the design, due to the potential for encountering solution features beneath the pile toe. The North Queensland case study only considered the lower bound equation from Zhang and Einstein (1998), due to the fractured nature and variability in strength of the rock.

4.1.3 Settlement

Settlement of the pile was calculated using the method of Rowe and Armitage (1984), as referenced in Pells' 1999 publication. For serviceability limit state the settlement of the pile was limited to 1% of the pile diameter.

4.2 Lateral analysis

Lateral pile analysis was carried out using the nonlinear p-y curves approach for both case studies. The p-y curves for rock were generated by considering the rock as massive rock model (Liang, Yang, and Nusairat (2009)). The critical parameters to generate p-y curves for massive rock were design σ_c, material index m_i, rock mass modulus and GSI. The lateral response of pile due to external structural loading was evaluated by considering a series of lateral springs generated by hyperbolic p-y curves of massive rock model given by Eq. (8). The lateral deflection did not exceed the performance requirements of the structure.

$$p = \frac{y}{\frac{1}{K_i}+\frac{y}{P_u}} \tag{8}$$

Where: p = Lateral reaction from rock, y = lateral deflection, Ki = Initial slope of p-y curve, Pu = Ultimate lateral resistance calculated from rock mass properties.

5 Risk Mitigation Strategies

For the Central Queensland case study, on the identification of limestone a risk assessment was undertaken which highlighted a potential for different ground conditions being encountered during construction. The main risks identified with limestone were cavities or seams being encountered in the rock socket shaft or at the toe of the pile. The design methodology adopted allowed for risk mitigation, by applying a reduction to the UCS value (Section 3.1.1), effectively reducing shaft capacity to account for potential seams and also ignoring end bearing altogether (Section 4.2.1). It was not possible to identify all seam locations from the site investigations alone but noted design assumptions should be verified during the construction. For the North Queensland case study, locations where there were strength inversions, the pile rock socket was designed to embed at least one times pile diameter into high strength rock. The designer highlights the importance of the socket base to be free of loose material and debris to mobilise required end bearing resistance. Also, where very high strength rock was found, special piling equipment was recommended to drill into high strength dykes. For both case studies it is emphasised that close supervision from the Geotechnical Assessor is needed to address risks identified.

6 Conclusions

Outlined in this paper is an approach to address design of rock sockets in complex geological environments. It is acknowledged there may be other approaches suited or more applicable to different geological conditions or site-specific challenges, that may dictate a different design method. It is recommended that the designer should carefully review rock parameters and select a design method and approach to account for the specific conditions encountered. In complex geological settings, the importance of a highly qualified Geotechnical Assessor, present during construction for the assessment of ground conditions and rock socket verification is acknowledged.

Acknowledgments

The authors would like to acknowledge DTMR for their endorsment in writing this paper and extend their gratitude to Jacobs Engineering management for their encouragement and support in preparing this paper.

References

AS 1726-2017. (2017). Geotechnical Site Investigation. *Standards Australia*.
AS 2159-2009. (2019). Piling – Design and Installation. *Standards Australia*.
Barton, N.R., Lien, R. and Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. *Rock Mech*. 6(4), 189-239.
Bieniawski, Z.T. (1984). Rock mechanics design in mining and tunneling. *A.A Balkema, Boston*.
Bieniawski, Z.T. (1989). Engineering rock mass classifications*, Wiley, New York*.
Deere, D.U. (1963). Technical description of rock cores for engineering purposes. *Felsmechanik und Ingenieurgeologie (Rock Mechanics and Engineering Geology)*, 1 (1), 16-22.
Fookes, P.G. and Hawkins, A.B. (1988). Limestone weathering: its engineering significance and a proposed classification. *Quarterly Journal of Engineering Geology, London*, Vol 21, pp.7-31.
Hoek, E. and Brown, E.T. (1998). Practical estimates of rock mass strength. *Int. J. Rock Mech, Min. Sci.* Vol.34 No.8, pp.1165-1186.
Hoek, E., Carter, T.G., Diederichs, M.S. (2013). Quantification of the Geological Strength Index Chart*. 47th US Rock Mechanics / Geomechanics Symposium held in San Francisco, CA, USA*, June 23-26.
Hoek, E, Marinos, P.G. (2000). Predicting tunnel squeezing problems in weak heterogeneous rock masses. *Tunnels and Tunnelling International 2000*; 132(11):45e51.
Liang, R., Yang, K., and Nusairat, J. (2009). p-y criterion for rock mass. *Journal of Geotechncial and Geoenvironmental Engineering*, Vol. 135, No.1, pp. 26-36.
Pells, P.J.N. (1999). State of Practice for the Design of Socketed Piles in Rock, *Proceedings 8th Australia New Zealand Conference on Geomechanics, Hobart*, pp.-307 to 1-327.

Rowe, R.K. and Armitage, H.H. (1987). A design method for drilled piers in soft rock. Geotechnical Research Centre, Faculty of Engineering Science, The University of Western Ontario, *Canadian Geotech Journal* Vol. 24., pp 126-142.

Rowe, R.K. and Armitage, H.H. (1984). The design of piles socketed into weak rock. *Geotechnical Research Report GEDT-11-84, University of Western Ontario, Canada*.

Transport and Main Roads (Dec. 2020). Geotechnical Design Standard – Minimum Requirements.

Transport and Main Roads (Nov. 2020). Technical Specification MRTS63 Cast-In-Place-Piles.

Waltham, A.C. and Fookes, P.G. (2003). Engineering classification of karst ground conditions, *Quarterly Journal of Engineering Geology and Hydrogeology*, vol.36, pp. 101-118.

Zhang, L and Einstein, H.H. (1998). End bearing capacity of drilled shafts in rock. *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 12, 574-585.