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Evaluation of rockbolt performance using field testing

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ABSTRACT: Permanent rockbolts are essential components of tunnel ground support systems, providing structural stability and controlling ground movement. Their design must account for axial and shear loading capacity, effective load transfer along the bolt length, and compatibility with expected rock mass conditions and ground behaviour. The stiffness characteristics of rockbolts are critical in determining how effectively load will be distributed and transferred along the bolt length. Pull-out testing is routinely conducted to validate design parameters and assess installation quality. This study presents a detailed analysis of production pull-out testing data from 1306 tests recently undertaken on Sydney Road tunnel projects, supplemented with published research, to assess the influence of rockbolt type, length, and grout bond strength on rockbolt performance. Findings show that rockbolt tests provide concordant stiffness and displacement control in most conditions, while cable bolts deliver higher stiffness but display greater variability in weak or fractured rock masses. The study proposes improved testing strategies to improve design validation, predictive modelling, and tunnel safety.

1 INTRODUCTION

Rockbolts are essential structural elements used extensively in tunnel ground support systems to stabilise excavations and mitigate ground movement in challenging geological environments. The stiffness characteristics (load-displacement relationship) of rockbolts play a crucial role in determining their load-bearing capacity and effectiveness in transferring forces along the bolt length. Additionally, stiffness is a key input parameter for all finite element (FE) modelling, influencing the accuracy and reliability of simulations used to assess tunnel support performance. The most common types used and investigated for this study are cement grouted double corrosion protected (DCP) combination bolts commonly referred to as DCP CT bolts and DCP cable bolts. This paper presents an evaluation of the performance of these rockbolts using extensive field-testing data collected from multiple Sydney tunnel projects in different ground conditions. Findings from this study contribute to the improvement of existing validation and design practices for rockbolts and provide practical recommendations for performance-based ground support design.

2 BACKGROUND – ROCK REINFORCEMENT SYSTEM

2.1 *Rock Reinforcement System*

The rock reinforcement system consists of a steel bar (CT bolt) or steel cable (cable bolt) pre-fitted with an expansion shell and a corrugated polyethylene sheath. This arrangement allows the bolt to provide immediate mechanical anchorage via the expansion shell while facilitating complete grout encapsulation for long-term bond integrity and corrosion protection. Typical mechanical characteristics of CT bolts and cable bolts, including tensile strength, stiffness, and elongation capacity, are summarised in Table 1 (Bluey, 2016; Bluey, 2025; DSI, 2025). Cement grout used is a standard formulation with low shrinkage characteristics, specific gravity greater than 1.95, compressive strength greater than 40 MPa, and high early strength exceeding 10 MPa at 24 hours.

Table 1. Typical characteristics of steel elements within bolt types (Bluey 2016, Bluey, 2023, DSI,2025)

Property	Unit	CT Bolt (DSI)	Cable bolt (BlueGeo CF 22)
Diameter	mm	21.7 / 23.2	21.8 (19 Wires)
Area	mm^2	370	312.9
At Yield (@ proof 0.2% strain)	MPa	600 MPa (220kN)	525 kN
Tensile Strength	MPa	840 MPa (310kN)	590 kN
Young Modulus	MPa	200 GPa	189.5 GPa
Elongation	%	13%	Min 3.5% / Typical 6-7%

2.2 *Design Considerations*

Axial performance of rockbolts needs to be assessed in both elastic and plastic domains. The elastic limit is typically defined by the yield strength of the bar or cable, and design loads are usually limited to less than 80% of the yield strength to avoid permanent deformation under expected service loads. However, when rockbolts are provided as a system (e.g. pattern), yielding of discrete rockbolts may be acceptable as loads are distributed to adjacent rockbolts. If this occurs however, adjacent bolts may also yield, leading to a potential for progressive collapse occurring. For bolts designed to behave plastically, the design must demonstrate stable post-yield behaviour without sudden rupture or progressive failure.

Shear loading on rockbolts arises from rock mass movements in stratified or jointed rock because of stress relief or creep movement, as well as from ground movement due to tunnel excavation. These shear forces may act independently or concurrently with axial loads. Shear performance is particularly critical for sheathed bolts, where protecting the integrity of the sheath is essential to maintain long-term durability. According to Pellet & Egger (1996), combined axial (P_u) and shear (V_u) loading can be assessed using an elliptical failure criterion:

$$\left(\frac{P}{P_u}\right)^2 + \left(\frac{V}{V_u}\right)^2 \leq 1$$

2.3 *Stiffness and Bond Characteristics*

Stiffness characteristics of fully grouted rockbolts are predominantly governed by the bond behaviour at the bolt-grout interface. There are a number of design approaches that model stiffness behaviour, including Clarke (2012) (bilinear stiffness behaviour), Blanco-Martín et al. (2013) (trilinear bond-slip model), Nemcik et al. (2014) (non-linear bond-slip constitutive law) and Aghchai et al. (2020) (modelling the bolt as an elastic-plastic continuum). The assessment by Aghchai et al. (2020) demonstrates that upon yielding of the bolt, shear resistance at the bolt-grout interface diminishes rapidly, leading to progressive debonding and ineffective load transfer. This approach also identifies that bond strength is underestimated if plastic deformation is ignored. Plastic behaviour studied by Yu et al. (2019) observed that even with increased embedment length and grout strength, the bond interface cannot sustain load transfer once bolts yield, resulting in crack propagation in grout as well as interface and bond failure.

2.4 *Rockbolt Verification and Validation Testing*

Typical rock reinforcement specifications require performance-based validation of ground support elements. Verification and validation procedures include pre-production acceptance testing and quality assurance production testing. Pre-production pull out tests are conducted to validate the load transfer mechanism, bond strength and identify any installation-induced issues. Assessments of pre-production testing exist, e.g. Salcher & Bertuzzi (2018), but production testing assessments such as this paper are rare.

Production testing validates bolt performance consistency during construction. Pull-out tests are performed incrementally, with load steps in the expected elastic state (~70% of yield) only held for specified durations and movements recorded. Typical acceptance criteria require bolts to sustain designated loads with minimal displacement and stability for at least 10 minutes. Typical initial testing frequencies are 5% of installed bolts in each rock mass type, reducing to 1% following successful validation. Should failures occur, testing may revert to a higher frequency, and adjacent bolts are subjected to additional verification to ensure local stability.

Full-scale pre-production shear testing of sheathed bolts is also specified, evaluating sheath integrity and bolt response under imposed displacements of 10 mm to 25 mm, with shearing limits for durability typically set at 10mm and higher limits permitted for safety.

3 PERFORMANCE ASSESSMENT AND DATA ANALYSIS

3.1 *Overview and Rock Mass Conditions*

This study analysed a comprehensive dataset comprising 1,306 production pull-out tests conducted across multiple tunnels in the Sydney region. The unfiltered dataset includes a total of 1,306 pull-out production tests, of which 32 tests (2.4%) resulted in failure. Out of the total tests, 251 were conducted on cable bolts and 1,055 on CT bolts (Figure 1).

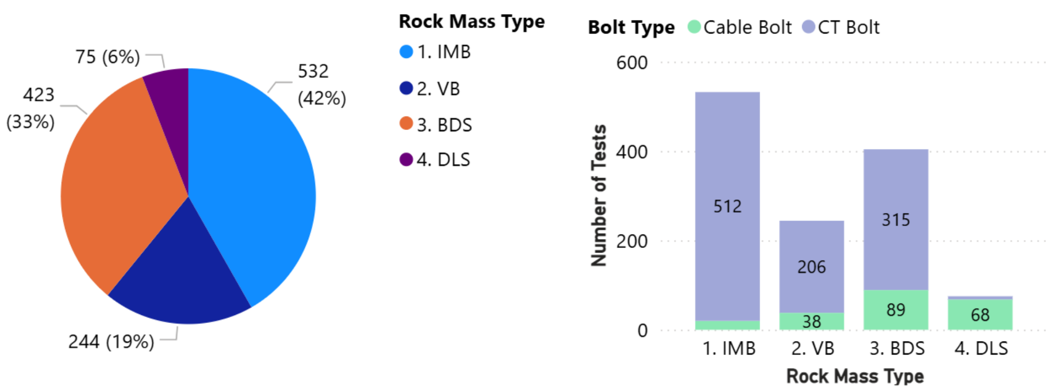

Figure 1. Production pull-out test data summary.

For analytical purposes, rock mass conditions encountered during tunnel excavation were categorised into four distinct classes, broadly equivalent to rock mass classes described in the GSI system (Marinos and Hoek, 2000) and the Sydney Rock Mass Classification (SRMC) system (Pells et al. 1998), and broadly corelated as described in Bertuzzi (2014) and Bertuzzi et al. (2016):

1) Intact or Massive to Blocky (IMB)- Intact, massive or undisturbed rock masses with minimal defects (broadly equivalent to SRMC Class I/ II Sandstone).
2) Very Blocky (VB) - Rock masses exhibiting multiple defect sets, weak bedding planes, or occasional minor clay seams, where defects regularly impact local stability (broadly equivalent to SRMC Class III-V Sandstone and Class I/II Shale).
3) Blocky/Disturbed/Seamy (BDS) - Rock masses surrounding major geological structures (e.g., faults and dykes) or a disturbed rock fabric and sheared defect surfaces broadly equivalent to Class III/IV shale and damage zones that surround fault as described in Kim, et al. (2004).
4) Disintegrated, Laminated or Sheared (DLS) - Zones of significantly fractured, crushed or sheared rock masses, often with clay fault gouge. Typically associated with fault zones or high to extreme weathering (broadly equivalent to SRMC Class V Shale and fault zones as described in Childs, et al. (2009).

Raw data from the pull-out test results were assessed for potential errors. Since most production tests were conducted on hardened shotcrete, a stable and consistent base was assumed, minimising the risk of inaccuracies in baseline displacement readings. As a result, errors related to initial readings were limited to only a few tests. In cases where anomalies were identified, base corrections were applied during the assessment of net cumulative displacement at each load increment. This ensured that the derived load–displacement curves shown below accurately reflects the true performance of the rock bolts under in-situ conditions.

3.2 *Impact of Rock Reinforcement Length*

Analysis of the pull-out test data revealed significant variations in bolt deformation patterns depending on bolt type and rock mass type. Figure 2 presents cumulative displacement (loading stage) trends for CT bolts (less than 7.5m long) and cable bolts (greater than 6.5m long), emphasising the differing responses of these systems to applied loads.

Box and whisker plots are used as an effective graphical method to summarise data distribution. The boxes in the following plots represent the interquartile range (IQR), capturing the central 50% of the data. The whiskers extend to the 5th and 95th percentiles, providing a representation of characteristic lower and upper bounds, while identifying potential outliers beyond these limits.

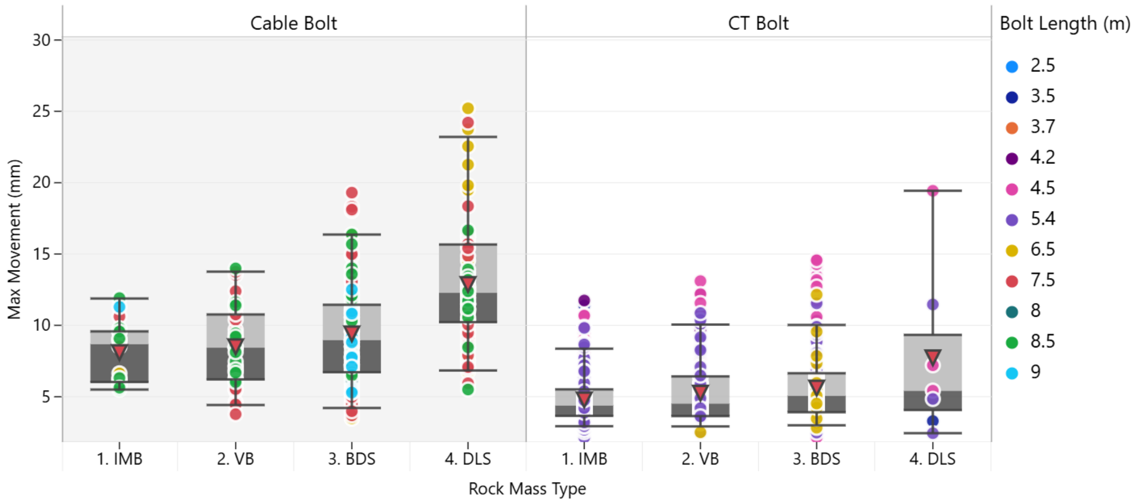

Figure 2. Distribution of cumulative displacement for CT and cable bolts

Key observations in Figure 2 include cable bolts show greater variability in poorer quality rock masses, CT bolts have consistent performance across all rock mass types, no clear correlation between bolt length and movement for either bolt type. These observations highlight CT bolts superior capability to limit ground movement, particularly under challenging conditions and suggests that initial bonded segments govern the bolt load–displacement response, with total installed length having a limited influence on the measured displacement range.

3.3 Evaluation of bolt stiffness

To estimate stiffness at various stress levels for CT and cable bolts the following formula is used:

$$Stiffness, k\ (\ kN/mm) = \frac{\Delta Load\ (kN)}{\Delta Displacement\ (mm)}$$

Bolt stiffness was evaluated using two complementary approaches:

1. Individual Test Stiffness Estimation:
   The Stiffness (secant) value was calculated for each individual pull test by determining the slope of the load–displacement curve. These values were then subjected to statistical assessment for various rock mass types to identify trends and consistency in bolt behaviour.

2. Displacement-based Stiffness Estimation:
   Rockbolt displacement values at key load levels were statistically analysed to assess deformation patterns. Stiffness (secant) was then estimated from the incremental changes in load and displacement, providing a load specific understanding of bolt response.

3.3.1 *Individual Test Stiffness Estimation*

The box and whisker plot comparison of cable bolts and CT bolts for the four rock mass types is shown in Figure 3.

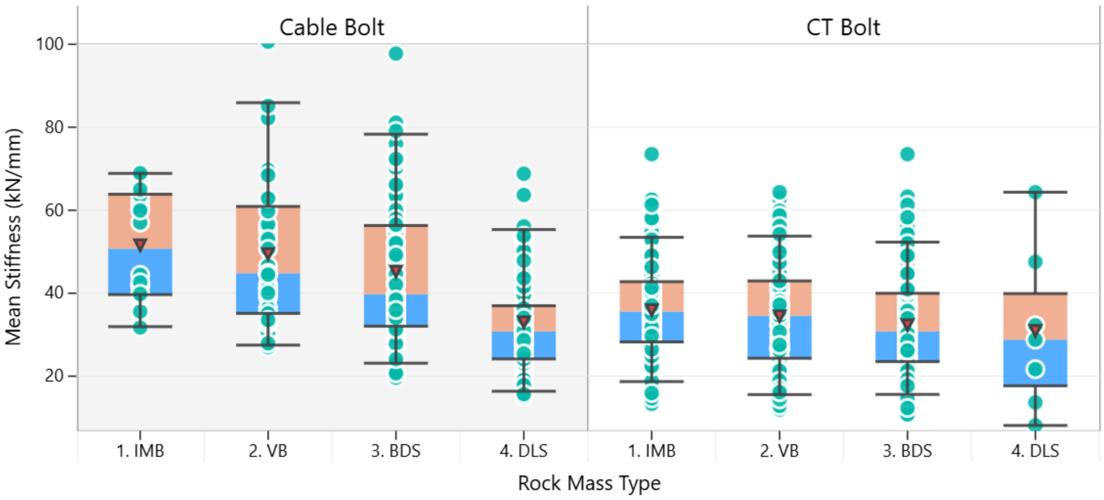

Figure 3. Statistical distribution of rockbolt stiffness value from individual test stiffness estimation

For cable bolts, median stiffness remains relatively high for all rock mass types. The highest median stiffness is observed in IMB rock masses, with a relatively narrow interquartile range, indicating consistent performance in relatively good quality rock masses. In VB and BDS rock masses, cable bolts continue to perform well but with greater variability. Notably, BDS and DLS rock masses show significant data dispersion. This high variability in poor quality rock masses suggests that while cable bolts can offer high stiffness, their performance becomes more dependent on installation quality and local anchorage conditions.

In contrast, CT bolts exhibit lower mean stiffness across all rock mass types, with stiffness values tightly clustered (narrow IQR), indicating greater consistency and predictability even in poor geological conditions. CT bolts exhibit lower stiffness variability across rock mass types, justifying their suitability for applications requiring uniform response and controlled deformation.

3.3.2 Displacement-based Stiffness Estimation

Rockbolt displacement distribution at various loading levels are statistically analysed for the four types of rock masses and bolt types. Figure 4 shows the actual load displacement profiles from the field production pull out tests. Bolt stiffness is then estimated by averaging values for each rock mass type. The distribution of displacement data is shown in Figure 4 and stiffness values for various loads and rock mass types are assessed in Figure 4 using the formula provided in Section 3.3.

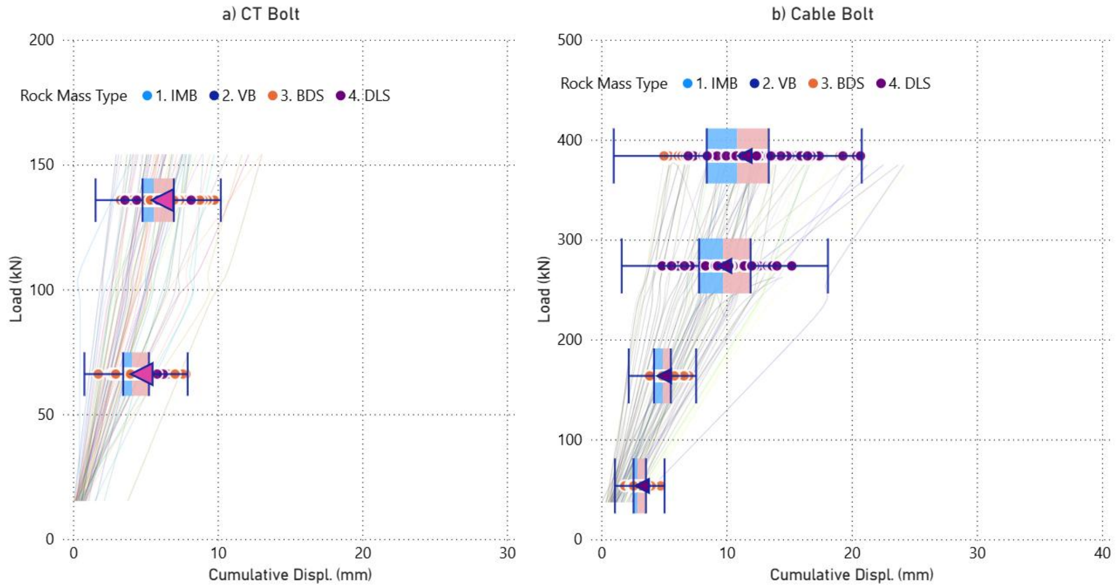

Figure 4. Statistical distribution of rock bolt displacement used in displacement-based stiffness estimation.

The predicted load-displacement behaviour of CT and cable bolts (Fig. 5), demonstrates clear trends between the four rock mass types. IMB rock masses exhibit comparable stiffness value of for both bolt types, reflecting stable and predictable performance. However, as rock mass conditions deteriorate, disparities in behaviour become evident. In VB and BDS rock masses, cable bolts maintain higher stiffness values, whereas CT bolts show a significant reduction, particularly in VB rock masses. Overall, this assessment suggests that cable bolts provide enhanced resistance to displacement and maintain better load transfer in weak or fractured rock masses.

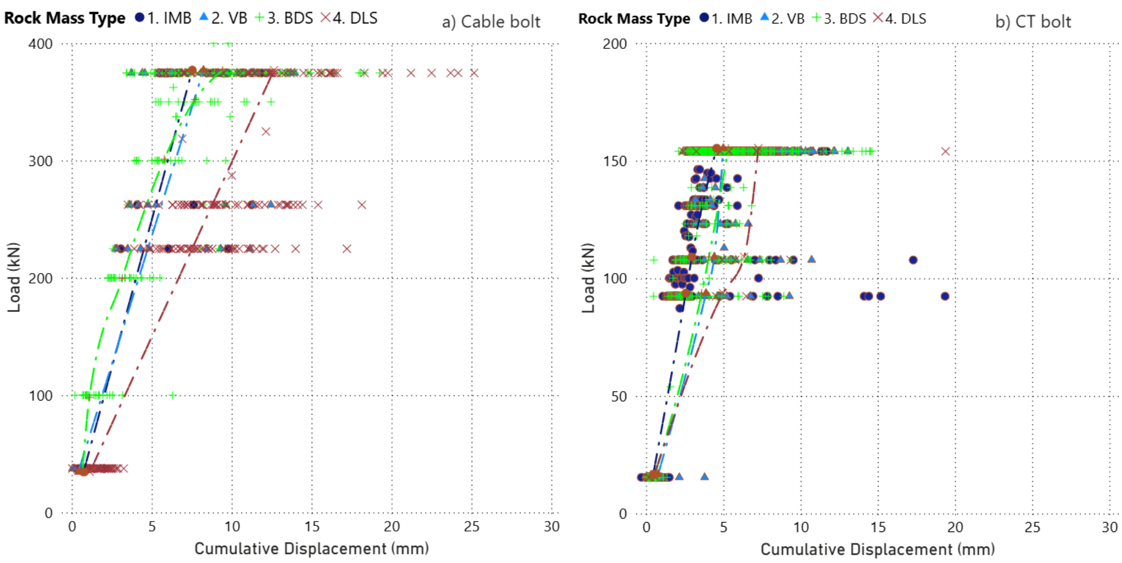

Figure 5. Predicted stiffness using displacement-based estimation, Lines indicate mean value at load.

3.3.3 Comparison of Stiffness

The stiffness comparison of CT and cable bolts for the four rock mass types are summarised in Table 2 for both individual test estimation and displacement methods. This table provides mean values as well as upper and lower bound characteristic values for use in design.

Table 2. Summary of estimated stiffness from individual test estimation and displacement method estimation. Mean value is provided along with 5% and 95% characteristic values in brackets

Rock Mass Type	Individual Test Stiffness (kN/mm)		Displacement-based Stiffness (kN/mm)	
	CT Bolt	Cable Bolt	CT Bolt	Cable Bolt
1. IMB	35 [18 - 53]	~50 [31 - 69]	36 [21 - >100]	36 [25 - >100]
2. VB	35 [15 - 54]	~50 [27 - 86]	26 [13 - 79]	43 [24 - >100]
3. BDS	32 [15 - 52]	~45 [23 - 78]	27 [15 - 106]	47 [32 - 78]
4. DLS	~30 [8 - 64]	33 [16 - 55]	25 [15 - 50]	28 [17 - 65]

Individual test estimation, where stiffness is calculated from each bolt's load–displacement values tend to produce a tighter and more consistent distribution. This is because each test captures the full load history and progressive deformation of a single bolt, reflecting gradual ground-bolt interaction. In contrast, displacement-based stiffness methods rely on averaging ground movement at fixed load levels across multiple bolts. This introduces variability due to differences in installation quality, ground heterogeneity, and local conditions, but consequently, is more representative of actual bolt performance.

The comparison of both individual test estimation and displacement-based stiffness estimation methods shows similar trends for all four rock mass types, with displacement-based values generally being slightly lower. This assessment is consistent with Clarke (2012) that showed a bilinear stiffness pattern for short bolts and noting that stiffness in the early phase is mainly controlled by the initial bonded segment.

The comparison of stiffness values for CT and cable bolts across different rock mass types reveals clear performance trends. In IMB rock masses, both bolts perform well, with cable bolts showing slightly higher stiffness. However, as rock mass conditions deteriorate from VB rock masses through to DLS rock masses, cable bolts consistently maintain higher stiffness and more stable performance than CT bolts. This indicates that cable bolts are better able to adapt to variable rock masses, potentially because of their flexible, multi-strand structure. In contrast, CT bolts show a noticeable decline in stiffness in weaker rock masses, reflecting their more rigid response and greater sensitivity to bond degradation. Overall, cable bolts demonstrate superior stiffness retention in challenging conditions, while CT bolts offer more predictable performance in stable rock masses.

4 DISCUSSION AND RECOMMENDATIONS

The analysis of production testing shows distinct performance differences between cable and CT bolts. CT bolts exhibit more concordant stiffness and displacement control across all rock masses. In contrast, cable bolts showed higher stiffness in IMB rock masses and VB rock masses, but their performance is more variable in BDS and DLS rock masses, albeit still maintaining higher stiffness than CT bolts. These trends align with the bilinear stiffness model presented in Clarke (2012).

The analysis also revealed that although the overall failure rate of rock bolts was 2.4%, only around 1% of installed bolts are typically subjected to pull-out testing. This suggests that current testing frequencies may be insufficient to detect and address performance variability, especially in critical or complex geological settings. Given the essential role of rock bolts in maintaining tunnel stability, the current production testing frequency warrants revising.

A robust evaluation of rockbolt performance must distinctly address two critical mechanisms, namely, bond interface behaviour and steel yield capacity. It is important to note that pull-out testing only assesses behaviour within the elastic limit of the bolt, meaning the steel component in a rockbolt remained un-yielded during testing. The current practice of using elastic-state assumptions for bond failure testing results in a limitation of how bond degradation interacts with plastic deformation of the steel element in a rockbolt. This represents a significant limitation, as real tunnel loading scenarios, particularly in highly deformable or faulted rock masses, may induce strains that exceed the yield point of the reinforcement. Without understanding the bond behaviour post yielding, the interaction between steel and grout, and between the grout and rock, remains uncertain for design cases that predict yielding and current production testing will not validate post yielding performance.

To improve current practice, the following recommendations are suggested:

1) Where plastic deformations are permitted, establish performance-based criteria for plastic deformation for rockbolts and pre-production suitability test methods to verify the criteria.
2) Undertake suitable design assessments and ensure parametric or sensitivity assessments are undertaken for the range of likely performance of rockbolts. Appropriate characteristic ranges for bolt stiffness values are provided in this paper.
3) Ensure production testing protocols include post-yield performance for all rock mass types.
4) Adopt combined axial-shear interaction envelopes (e.g., Pellet & Egger method) as standard practice for rockbolt design evaluation and validation.
5) Production testing frequencies be increased, especially in poorer ground conditions where installation quality and bond performance are more variable. For example, in BDS and DLS, a testing rate of at least 10% may be more appropriate to confirm system reliability.

5 CONCLUSIONS

This study has provided a comprehensive evaluation of rockbolt performance in Sydney tunnels through the analysis of an extensive dataset of pull-out test data. The results confirm that CT bolts deliver consistent stiffness and displacement control across most conditions, making them suitable for most rock mass conditions. In contrast, cable bolts demonstrated higher stiffness and greater adaptability in poor quality rock masses, although with increased variability in poorer quality rock masses.

A critical limitation of the current testing practices is the focus on elastic-state behaviour only. The lack of testing beyond yield strain means that the interaction between the bolt, grout, and rock under post-yield conditions remains poorly understood. This is especially critical in rock masses affected by major geological structures such as faults and dykes and other highly deformable ground where plastic deformation is possible.

To enhance ground support reliability, it is recommended that design and design validation methods be expanded to include post-yield bolt performance and combined axial-shear interaction assessments. This approach will ensure greater reliability for performance-based design approaches in relatively complex geological settings.

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6 REFERENCES

Aghchai, M.H., Maarefvand, P. & Salari Rad, H. 2020. Analytically determining bond shear strength of fully grouted rockbolt based on pullout test results. In Periodica Polytechnica Civil Engineering, 64(2): 371–378. Budapest: Budapest University of Technology and Economics.

Bertuzzi, R., 2014. Sydney sandstone and shale parameters for tunnel design. Australian Geomechanics Journal 49, 95–104.

Bertuzzi, R, Douglas, K, Mostyn, G. 2016. Comparison of quantified and chart GSI for four rock masses. In Engineering Geology 202, 24-35.

Blanco-Martín, L., Tijani, M., Hadj-Hassen, F. & Noiret, A. 2013. Assessment of the bolt-grout interface behaviour of fully grouted rockbolts from laboratory experiments under axial loads. In International Journal of Rock Mechanics and Mining Sciences, 63: 50–61. Amsterdam: Elsevier.

Bluey. 2016. BluGeo CF Rockbolt – Technical Report rv3. Sydney: Bluey Technologies Pty Ltd.
Bluey. 2023. BluGeo CF Rockbolt – Technical Data Sheet R01/2023. Available at: https://www.bluey.com.au/wp-content/uploads/2016/07/BluGeo-CF-Rock-Bolt-TDS-R012023.pdf (accessed 16 May 2025).
Childs, C., Manzocchi, T, Walsh, J.J, Bonson, C.G., Nicol, A., Schopfer, M.P.J. 2009. A geometric model of fault zone and fault rock thickness variations. In Journal of Structural Geology 31 117-127.
Clarke, S.J. 2012. Interpretation of pull test results of grouted rockbolts. In Proceedings of the 11th Australia & New Zealand Conference on Geomechanics, Melbourne, 15-18 July 2012: 89–95. Melbourne: Institution of Engineers, Australia.
DSI. 2025. CT Corrosion Protected Mechanical Bolt. Available at: https://www.dsiunderground.com.au/fileadmin/downloads/dsi-underground-com.au/products/CT_-_CT_Corosion_Protected_Mechanical_Bolt.pdf (accessed 16 May 2025).
Kim, Y.S, Peacock, D.C.P, Sanderson, D.J. 2004. Fault Damage Zones. In Journal of Structural Geology 26, 503-517
Marinos, P, Hoek, E. 2000. GSI – A geologically friendly tool for rock mass strength estimation. Proceedings of GeoEng 2000, International conference on geotechnical and geological engineering, Melbourne Australia, P 1422-40
Nemcik, J., Aziz, N., Ren, T. & Geng, X.Y. 2014. Numerical modelling of failure propagation in fully grouted rockbolts subjected to tensile load. In International Journal of Rock Mechanics and Mining Sciences, 71: 237–245. Amsterdam: Elsevier.
Pellet, F. & Egger, P. 1996. Analytical model for the mechanical behaviour of bolted rock joints subjected to shearing. In Rock Mechanics and Rock Engineering, 29(2): 73–97. Vienna: Springer.
Pells, P.J.N., Mostyn, G.R., Walker, B.F., 1998. Foundations on sandstone and shale in the Sydney region. Australian. Geomechanics Journal. 33, 17–29.
Salchar, M. & Bertuzzi, R. 2018. Results of pull tests of rockbolts and cable bolts in Sydney sandstone and shale. In Tunnelling and Underground Space Technology, 74: 60–70. Amsterdam: Elsevier.
Yu, S., Zhu, W., Niu, L., Zhou, S. & Kang, P. 2019. Experimental and numerical analysis of fully grouted long rockbolt load-transfer behavior. In Tunnelling and Underground Space Technology, 85: 56–66. Amsterdam: Elsevier.