Advanced engineering methods to extend and expand the use of reclaim tunnels at Pilbara mine sites

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ABSTRACT: Rio Tinto has operated iron ore mines in Western Australia's Pilbara region since the 1960s. Recent changes in risk management practices have driven a re-evaluation of the structural capacity of key infrastructure, particularly tunnels. These tunnels are now subject to increased external loads from heavier mobile equipment operating above them, and additional internal loads from processing equipment upgrades mounted to the tunnel directly. Subsequently, a region-wide structural assessment program has been initiated to evaluate tunnel performance under current and emerging load conditions. This paper presents a case study of an operational concrete arch tunnel, built in the late 1970s, which supports a 30 m high stockpile over a rail corridor. Initial assessments using modern design codes indicated overutilisation which resulted in stockpile height restrictions. However, by integrating tunnel monitoring data with finite element analysis, a more accurate structural rating was achieved, therefore demonstrating the importance of advanced methods in managing ageing assets.

1 INTRODUCTION AND TUNNEL BACKGROUND

Rio Tinto has been operating iron ore mines in the Pilbara region of Western Australia since the mid-1960s. To support mining activities and material processing plants, various overpass and underpass structures have been constructed. These structures provide grade separation between ore stockpiles, conveyor systems, railways, heavy mobile equipment roads, light mobile equipment roads, personnel access routes, and utility corridors that are part of the process circuit.
Across the Pilbara operations, Rio Tinto operates approximately 90 overpass and underpass structures. These structures were designed and constructed over the last 60 years, each reflecting the engineering standards, material strength and design philosophies common of their respective eras.

In recent years, evolving approaches to risk management have prompted a reassessment of the structural capacity of these tunnels. This is due to increased loading both externally (i.e., surface loading from mobile equipment above the tunnel) and internally (e.g., upgrades and/or additional mechanical equipment to ancillary systems such as instrumentation cabinets, ventilation units, and operational services) within the tunnel structure. In response, a Pilbara-wide review of the structural capacity of these tunnels has been initiated to better understand the utilization under current loading conditions. These assessments are being conducted against current design methodologies to ensure the continued safety and performance of these critical assets.

It is a general practice to use current design standards for assessing existing structures. Current assessment methodologies, which were largely developed for newer structures[1], generally yield conservative evaluations of older structures. As a result, a significant number of existing assets are identified as structurally deficient despite exhibiting no outward signs of distress. This discrepancy highlights the necessity for more sophisticated and accurate methods of structural assessment[2]. The AS ISO 13822 standard recommends that the structural assessment shall withstand a plausibility check[3].

This paper presents the reassessment of a reinforced concrete (RC) arch tunnel (Figure 1) constructed in the mid-1970s, which was designed to accommodate rail traffic beneath a large iron ore stockpile. The tunnel structure is approximately 400m long and features a roof with integrated discharge gates. These discharge gates allow for ore to be loaded into rail wagons positioned inside the tunnel. The reassessment adopts a multidisciplinary strategy, integrating a review of legacy design reports, geotechnical investigations and analysis to the current methodology, advanced nonlinear structural modelling, and targeted field instrumentation.

Figure 1. RC arch tunnel.

2 ORGINAL DESIGN

The original design of the concrete arch tunnel, developed in the mid-1970s, was driven by the intent to minimise internal bending and shear through geometry rather than material strength. A continuously curved section approximating an elliptical profile (refer to Figure 2), was adopted to promote arching action and carry loads primarily in axial compression under typical stockpile and earth pressure load combinations. While this shape was idealised for a specific vertical-to-horizontal load ratio, it remained effective across a range of load conditions, offering significantly lower internal forces than flat-sided alternatives. To accommodate the operational requirement for a flat floor, the base of the arch was concrete infilled and the final cross-section was formed from compound circular arcs.

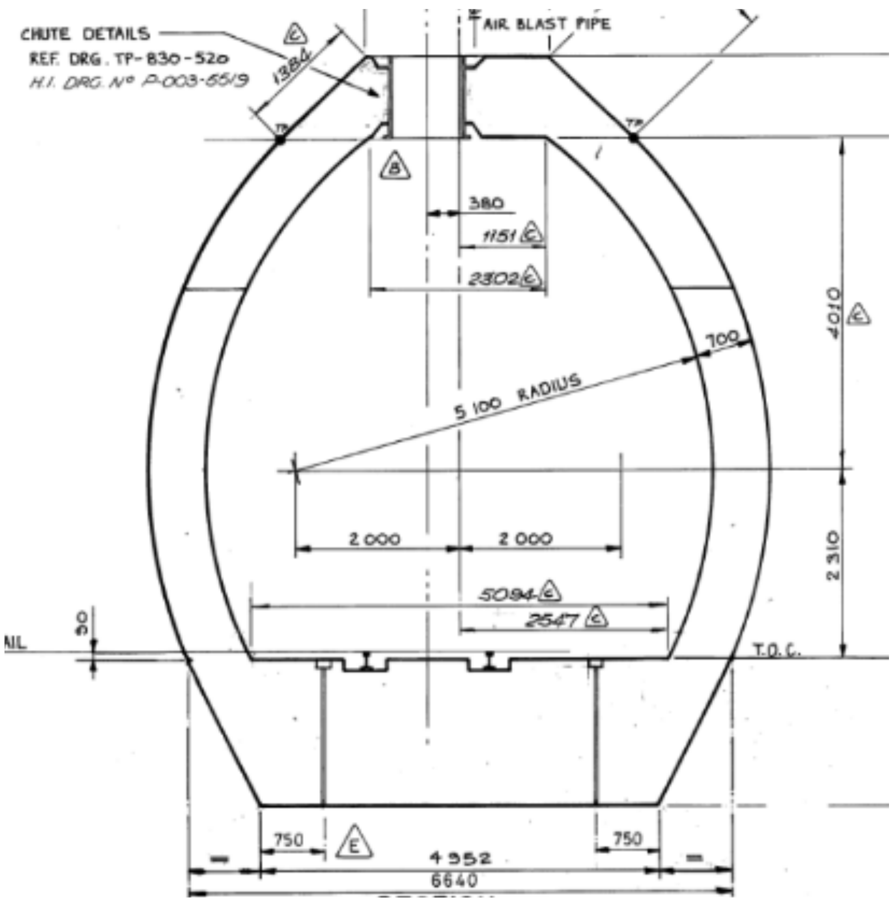

Figure 2. Original design of the arch tunnel.

Structural analysis at the time predicted minimal deformation under full loading (approximately 0.4 mm) and outward wall movement under purely vertical loads (approximately 12 mm), therefore confirming compatibility with the earth pressure assumptions. Longitudinal stresses from shrinkage and settlement were addressed through the inclusion of 12-metre construction modules with 6-metre control joints. These were aligned with discharge gate spacing, to allow the tunnel to accommodate ground movement without excessive internal stress.

3 FINITE ELEMENT MODELLING

A pseudo 3D Finite Element Analysis (FEA) model of the overall structural system was developed in ANSYS (refer to Figure 3). The model comprised of a 360 mm slice of the tunnel, the surrounding stockpile, embankment, foundation and select fill soil material.

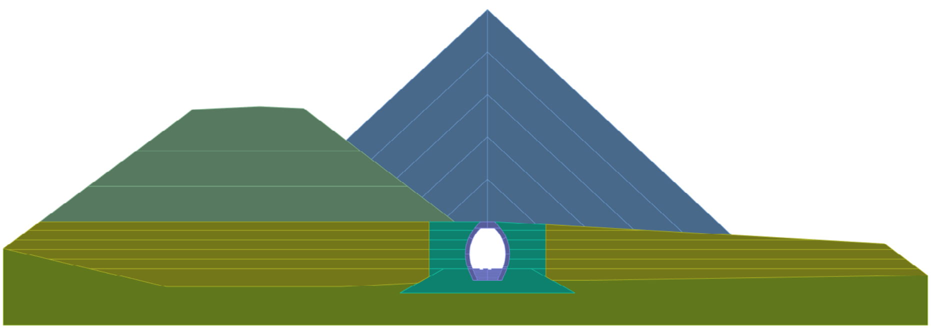

Figure 3. Pseudo 3D Finite Element Analysis (FEA) model.

The objective of the FEA assessment was to address the shortfalls of the conventional assessments which indicated significant overutilisation despite limited visible structural distress. The goals of the FEA assessment were as follows:

1. Determine realistic values for the design capacity of the tunnel by accounting for non-linear effects and non-linear material properties.
2. Determine the maximum allowable stockpile height above the tunnel.
3. Identify areas of high stress which may not have been captured in previous assessments.
4. Conduct a sensitivity analysis to understand the relationship between the tunnel's design capacity with respect to variability in the materials properties used in the FEA model.

The Menetrey-Willam model was used as an advanced constitutive model for concrete. It is based on the flow plasticity theory for modelling concrete behaviour. The baseline parameters used to define this material model were selected from FIB – Model Code[4] for Concrete Structures 2010. The parameters were then calibrated and verified by performing a simulation on a simply supported RC beam. The simulation results were compared against the expected behaviour of the RC beam to validate the use of some of the parameters. The reinforcing bars were represented with a bilinear stress-strain relationship to capture yielding and post-yield behaviour.

For the geotechnical parameters, upper and lower bound values were adopted for conducting a sensitivity analysis. Duncan & Selig's[5] method was used to interpret the soil properties including Young's Modulus, Poisson's Ratio, Cohesion, Friction Angle and Dilatancy Angle for both the upper and lower bound cases. The foundation was assumed to be a banded iron formation material (BIF). To enable comparison of our assessment with previous assessments, the same geotechnical properties were adopted for the foundation material.

4 METHODOLOGY

The state of principal stress was assessed using the FEA model. Both maximum and minimum principal stress states of the concrete, as well as the normal stress along the length of the reinforcing steel bars were investigated to understand the response of the tunnel at varying levels of stockpile load acting on top of it. The maximum principal stress results were used to understand the level of tensile stress being experienced whereas the minimum principal stress was used to understand the level of compressive stress being experienced. The principal stress results were used in conjunction with the total strain being experienced by the tunnel to get a perspective on the formation and propagation of cracks in the tunnel. A sensitivity analysis was performed and therefore the response of the tunnel in a soft and stiff select fill soil medium was investigated.

For both the upper and lower bound cases, the model was analysed with incrementally increased stockpile heights. After reaching the maximum geometrical height, the stockpile load was "virtually raised" by incrementally increasing the density of the stockpile material. The following steps were then performed:

1. The load step at which the model ceased to converge was identified.
2. The corresponding virtual stockpile density at this load step was identified.
3. The stockpile base pressure resulting from this virtual stockpile density and the maximum stockpile height was calculated.
4. An equivalent stockpile height was then determined. This is the height that would generate the same base pressure using the original (unmodified) stockpile density.
5. This equivalent height was then selected as the maximum unfactored system capacity.

The design capacity was determined by applying the system strength reduction factor in accordance with AS 5100.5:2017 into the equivalent height. FEA results indicated that at failure, the concrete at the tunnel wall reached its limit state before yielding occurred in the reinforcement steel.

Selected results at the ultimate failure point are shown in Figure 4.

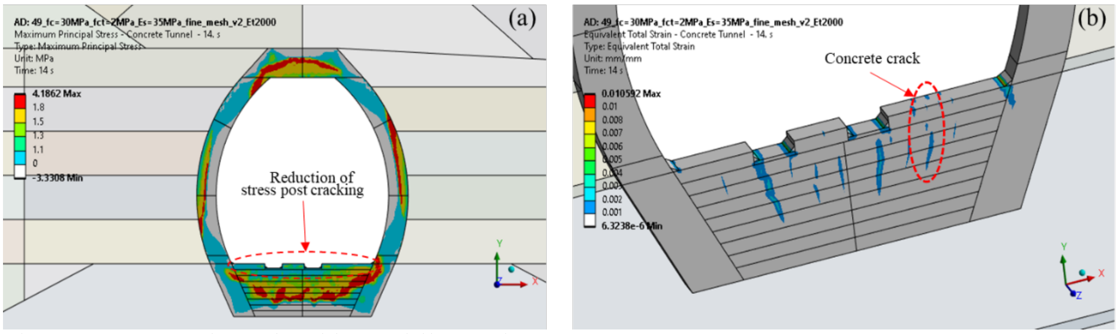

Figure 4. FEA results at the ultimate failure point.

5 INSTRUMENTATION

At the outset of the instrumentation strategy, the project team undertook a structured review of available monitoring methods and technologies to determine their suitability for assessing tunnel performance in a manner that could reliably support the verification of structural capacity. A number of technologies, including vibration-based as well as distance-based measurement systems, were explored. Strain gauges were ultimately selected as the most appropriate and accurate method for this application. These sensors can be installed on both the surface of the concrete and, where accessible, directly onto the steel reinforcement. This approach enables a direct measurement of strain, which can be confidently correlated with stress outputs from structural analysis models. As such, strain gauges provide a higher level of fidelity and reliability in understanding material behaviour and verifying structural utilisation under operational loads.

Strain gauges were installed to monitor the strain response across nine (9) cross-sections of the tunnel as shown in Figure 5. At each cross-section five (5) strain gauges were installed and the locations were selected based on the high-stress locations from the FEA results.

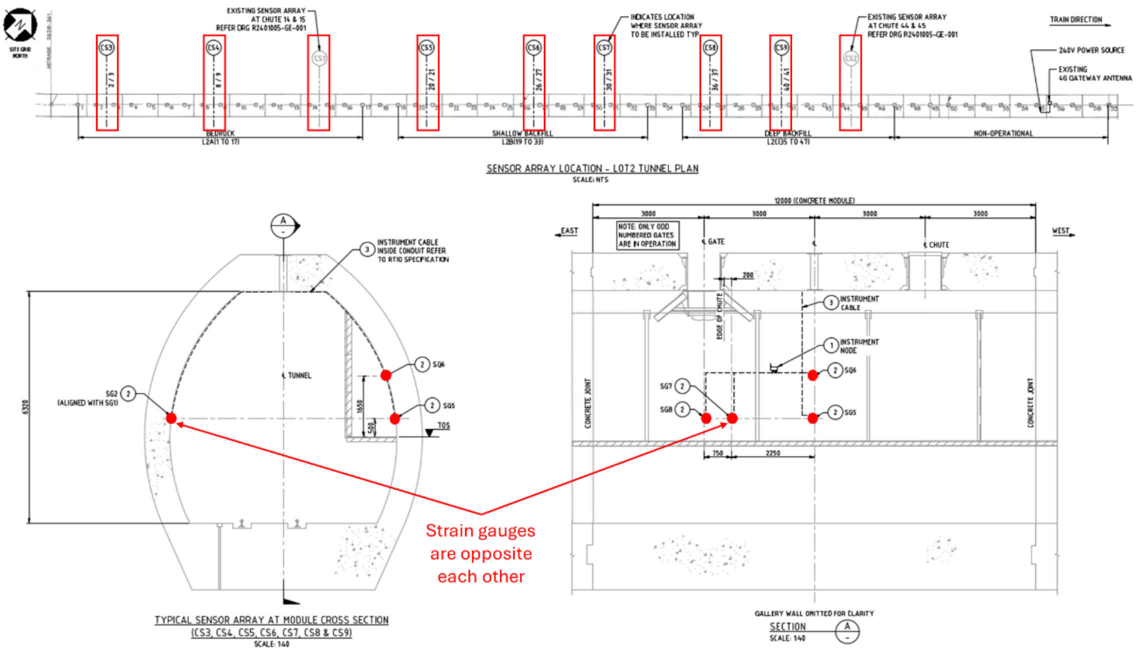

Figure 5. Strain gauge locations (red dots) repeated along 9 cross-sections (red rectangles).

Analysis of the strain data revealed clear spatial trends along the tunnel length. At the tunnel entrance, gauges consistently recorded higher compressive strain than other sections. This observation aligned with historical design documentation that indicated the presence of weaker founding material in this area, possibly associated with an infilled natural drainage line. Although recent borehole data did not explicitly confirm this zone, the measured strain response provided empirical evidence supporting the legacy assumption of non-uniform subgrade stiffness.

Toward the tunnel midsection and exit, strain magnitudes were generally lower and more consistent with stiffer backfill and more uniform overburden geometry. Importantly, the strain trends qualitatively matched the predicted strain envelopes generated by the ANSYS nonlinear model, especially when adjusted for soil modulus and stockpile geometry variability.

Figure 6 shows the cross-sections 1 and 9 (CS1 and CS9) strain readings within a selected month in September 2024.

Figure 6. Strain gauge readings.

The field data collected from the strain gauges was compared with nonlinear FEA model predictions. Results showed a correlation match between the observed strain and local ground condition showing higher strain was detected at the tunnel entrance, in agreement with historical documentation indicating weak backfill in that area, even though recent geotechnical investigations did not explicitly identify this feature.

The strain response also aligned with predicted ground stiffness profiles along the tunnel length. However, uncertainty in the actual stockpile load, due to fluctuating ore density and a radar-based stockpile height measurement system with ±0.5 m accuracy, required careful interpretation of strain magnitudes.

Despite these limitations, the instrumentation confirmed bidirectional deformation patterns during stockpile loading/unloading, reinforcing the model's assumptions. The team verified that gauges captured the full strain range expected, and that even with imperfect loading data, relative strain behaviour provided valuable validation for the structural model.

Additionally, the use of non-linear FEA to identify critical failure points allowed a new pathway to define the failure points based on strain limit. This is a closer correlation to the actual behaviour of the tunnel, which considers the soil structure interaction. This has offered a different method to the normal load and reduction factor approach for green field design. Based on extensive ANSYS modelling and sensitivity analyses, a set of allowable strain limits were developed and provided to Site Operations to monitor the tunnel performance under varying stockpile load.

6 USING MONITORING DATA TO SUPPORT STOCKPILE HEIGHT MANAGEMENT

The strain data was post-processed to compensate the temperature effects and account for the self-weight and deadload stockpile of 2 m. The post-processed strain data of CS1 is illustrated in Figure 7. Loading and unloading events are marked on the figures as red and green dots, respectively. The negative sign convention denotes a compressive strain, where more negative values indicate higher levels of compression, corresponding to increased stockpile height.

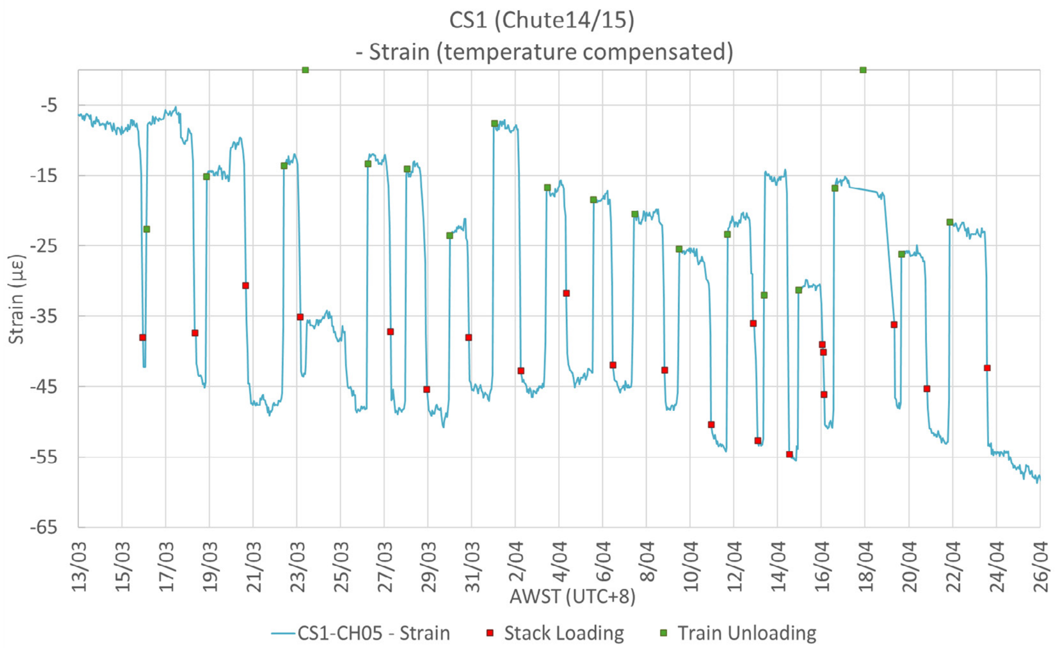

Figure 7. Stockpile loading and unloading events.

As shown in the figure above the data clearly captured the loading and unloading cycles. Utilising this information, the strain gauge monitoring has played a central role in validating the structural performance of the tunnel under operational loading and informing strategic decisions around stockpile management.

Beyond height control, the monitoring system also provided new opportunities to refine the stockpiling method employed above the tunnel. Operationally, stockpiles may be constructed using two main techniques:

– Chevron stacking, where material is deposited in longitudinal layers along the length of the tunnel until the desired height is reached; and
– Cone shell stacking, where the stacker discharges material at a fixed location to form a cone-shaped pile, then moves to the next location once the target height is achieved.

Traditionally, each of these methods require separate loading scenarios to be modelled in the design phase to confirm structural adequacy, often necessitating multiple iterations to evaluate the effects of varying load distributions. The use of strain gauge data eliminates this complexity. Because strain is a direct and reliable indicator of structural response, the effect of different stacking patterns can be assessed empirically without the need to re-analyse each case through time-consuming simulations. This approach also removes the need to explicitly model complex soil-structure interaction phenomena, such as soil arching, or account for stress-dependent backfill behaviour in the structural model.

Moreover, the strain gauge system has enabled the development of a real-time performance envelope that supports responsive stockpile management. For example, in the event of backfill saturation due to extreme weather or infrastructure leakage, the effective stiffness of the supporting ground may decrease, leading to higher strains in the tunnel structure. With continuous monitoring, excessive strain readings can serve as an early warning, prompting temporary restrictions on stockpile height while the source of the issue is investigated and addressed. This ensures that the tunnel remains within its elastic performance range and avoids the onset of damage or irreversible deformation.

The ability to monitor strain trends during both loading and unloading cycles also provides insight into the reversibility and resilience of the tunnel under varying operational conditions. No significant residual strain accumulation was observed over multiple stacking cycles, supporting the conclusion that the tunnel remains in an elastic response regime under current loading practices.

7 CONCLUSIONS

The tunnel was initially assessed as overutilised using liner-elastic based finite element models with simplified soil-structure interaction, resulting in stockpile height restrictions to manage structural risk. However, these models did not fully capture actual tunnel behaviour under operational conditions. Through the integration of advanced nonlinear finite element analysis validated with strain gauge instrumentation, it was demonstrated that the tunnel performed within acceptable limits, even under increased loads. This evidence supported the safe relaxation of stockpile restrictions.

This shift from assumption-based to performance-based management led to key outcomes:

- Justification for higher stockpile zones,
- Reduced need for multiple model scenarios,
- Early warning of structural or ground condition changes, and
- Improved coordination between engineering and operations.

The study highlights how legacy infrastructure can be safely re-rated using real-time data and advanced modelling. It provides a scalable framework for similar buried assets, demonstrating the growing importance of instrumentation-informed strategies in modern asset management.

8 REFERENCES

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