STRAIN SOFTENING AND NAVIGATING KARSTIC RISKS IN THE DESIGN AND CONSTRUCTION OF LARGE RETAINING WALLS

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ABSTRACT

The METRONET Yanchep Rail Extension (YRE) is one section of Perth's most ambitious public transport program of works. The passenger rail infrastructure runs 15 km north from the existing Butler Station to the Yanchep stowage yard and includes three new station precincts and over a dozen bridge structures. The rail alignment passes through regions known for karstic limestone conditions and is primarily formed within a large cut setting requiring bulk excavation in the order of up to 10 m to 15 m below the existing ground level.

Due to project and site constraints, contiguous piled walls without propping, anchoring or tiebacks have been constructed to support the ground within these large cuts. Fully cantilevering piled retaining walls of this height and within these geological settings are unconventional. Importantly, they require consideration of post-peak strength loss associated with the predicted strains and lateral wall movements, coupled with effective management of karstic risk.

This paper discusses the development, application, advantages and limitations of an innovative "strain-softening" numerical analysis approach developed for the wall design. An efficient and pragmatic approach used to mitigate karstic risks during design and construction is presented, along with the instrumentation and monitoring that was adopted to monitor the wall performance.

1 INTRODUCTION

The METRONET Yanchep Rail Extension (YRE) is one section of Perth's most ambitious public transport program of works. YRE is the foundation for an integral part of WA's future growth, with the Yanchep – Two Rocks area alone anticipated to accommodate 2% to 3% of Australia's population growth over the next 40 years.

The scope of the YRE is to design and deliver the northern section of the Joondalup Line from Butler to Yanchep as shown in Figure 1. The new passenger rail infrastructure runs 15 km from the existing Butler crossovers to the Yanchep stowage yard north of Yanchep Station and comprises the construction of the following key elements:

- Three new station precincts built at Alkimos, Eglinton and Yanchep;
- 11 new over-rail bridge structures;
- Bus and pedestrian bridge structures;
- Fauna overpass and underpass structures;
- Retaining walls, noise walls and Overline Rail Equipment (OLE) along the alignment; and
- Associated civil components such as track, pavement and drainage.

The alignment is primarily within a large cut setting requiring bulk excavation in the order of up to 10 m to 15 m below the project boundary which interfaces with existing and proposed residential subdivisions and "Bush Forever" regions (Department of Planning, 2021). To meet project and site spatial constraints through narrow sections of the alignment, top-down contiguous piled walls without propping, anchoring or tiebacks, were required to support the large cuts in lieu of more conventional solutions (battered slopes and gravity retaining walls) which would have otherwise extended beyond the project boundary or required extensive temporary works to permit construction. Fully cantilevering piled retaining walls of this height are unconventional and requires consideration of post-peak strength loss, associated with the predicted strains and lateral wall movements.

Additionally, the alignment passes through regions known for karstic limestone conditions. Subsurface conditions are characterized by frequent voids, vugs and pockets of loose sands below the rockhead, limestone pinnacles as well as the formation of sinkholes and caves in nearby areas. Collectively, these ground conditions present a significant engineering challenge to the design and construction.

The scope of the contiguous wall package was significant, with a total of 3282 bored piles distributed over 31 contiguous walls, and with a cumulative pile length of about 50 km. An additional 636 contiguous bored piles were installed at top-down bridge structures along the alignment. This paper discusses the development, application, advantages and limitations of an innovative strain-softening numerical analysis approach developed for the wall design. An efficient and pragmatic approach used to mitigate karstic risks during design and construction is presented, along with the instrumentation and monitoring that was adopted to validate the wall performance.

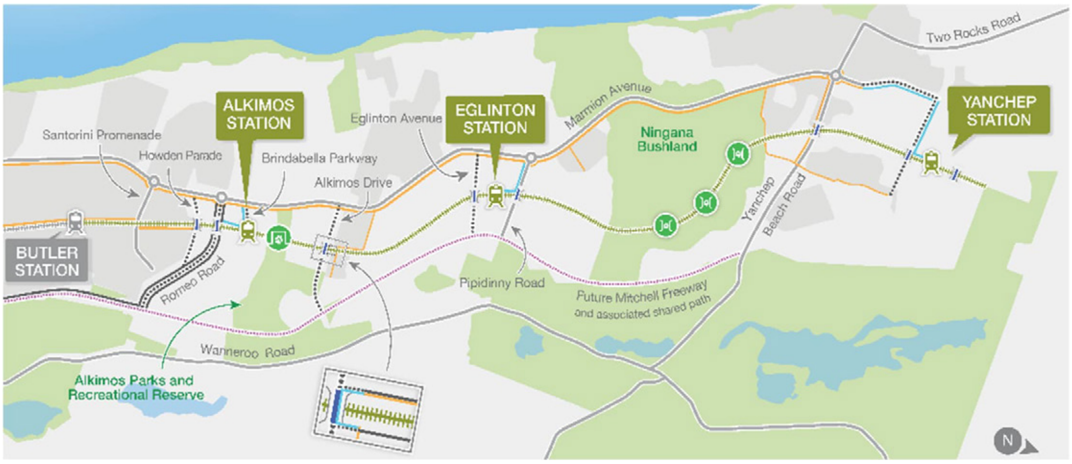


Figure 1: Proposed YRE alignment and new station precincts (METRONET, 2024)

2 GEOLOGICAL SETTING

The ground surface along the YRE alignment is predominantly undulating due to the site being located within the coastal dune system. The existing surface elevation along the alignment is between approximately RL 10 m AHD to RL 55 m AHD. Localised areas of steeper relief are present at dune ridgelines throughout the alignment and are more common in the northern portion, where the adjacent land is predominantly undeveloped.

A summary of the geological units and potential geotechnical risks along the YRE alignment is provided in Table 1. Subsurface conditions along the YRE alignment predominately comprise Safety Bay Sand and/or Tamala Sand, overlying Tamala Limestone. The historical maximum groundwater level along the alignment is well below rail level, and ranges between RL 2 and 4 m AHD. Site investigation included in the order of 420 No. Cone Penetrometer Tests, 150 No. boreholes, 15 No. test pits, geophysics survey and laboratory testing along the rail alignment.

Table 1: Summary of geological units along the YRE alignment

Unit Name	Description / Origin	Geotechnical Risks / Notes
Topsoil	Thin, generally organic rich, surficial layer	• Unsuitable for structural applications
Safety Bay Sand	Aeolian – very loose to medium dense, calcareous sand	• Erosion and instability of batter slopes • Differential settlement
Cemented Safety Bay Sand	Duricrust / Aeolianite – very weakly to moderately cemented siliceous calcarenite	• Generally very loose
Tamala Sand	Residual – loose to dense silica sand derived from weathering of the Tamala Limestone	• Erosion and instability of batter slopes • Can result in low bearing capacity and instability of slopes when very loose
Tamala Limestone	Aeolianite – predominantly siliceous calcarenite, but also as a duricrust ‘caprock’ calcreted calcarenite layer and as siliceous carbonate sand	• Highly variability in elevation and cementation • Differential settlement of footings • Variable stability relating to degree of cementation • Karst features (e.g., cavities, caves, sinkholes) • Caprock – hard ripping, localised difficult excavations • Potential for leached zones (Tamala Sand) within unit

Investigations identified that the Safety Bay Sand stratum typically occurs in localised deposits along the alignment and is generally limited in thickness to about 3 m depth below the existing ground surface. In contrast, deeper and thicker Tamala Sand was observed at most of the investigation locations with a tested thickness of up to 25 m. At some locations, the in-situ sand units are overlain by variable thicknesses of granular fill material.

The in-situ density of the sand units is generally very loose to loose near the surface, becoming medium dense to dense with increasing depth. The Tamala Limestone weathering profile is inherently variable, with varying degrees of cementation and strength occurring within short lateral and vertical distances, as highlighted in Figure 2, below. Zones of uncemented sands beneath 'Top of Rock' level are not uncommon, which influenced the approach and philosophy adopted in the design and construction of YRE.

A primary risk associated with limestone is the potential presence of karst features which are formed within the unit due to the dissolution of carbonate minerals by groundwater over geological time. Well-developed karstic features such as cave systems, sinkholes, dolines, and solution pipes are present in the limestone north of Perth.

Figure 2: Inherently variable top of limestone rock profile along the YRE alignment

3 CONTIGUOUS PILE WALL OVERVIEW

Given the linear nature of the project and significant scope of the contiguous wall package, the wall geometry became standardised, where practical, to streamline the design, procurement and construction process, which required up to seven piling rigs working in parallel. The design of bridge structures adopted 900 mm diameter piles at 1100 mm c/c spacing, whilst corridor walls adopted 1050 mm diameter piles at 1300 mm c/c spacing. Piles were constructed using dry rotary boring techniques with temporary steel casing to maintain the sidewall stability through uncemented zones.

Reinforcement cages adopted between 10 and 20 No. N40 diameter steel bars arranged symmetrically, with 80 mm of Class S40 concrete cover. Consideration was given to limiting pile lengths to 15 m and 12 m to reduce the amount of reinforcement cage splicing and to alleviate transportation requirements, respectively. A monolithic concrete capping beam provided connectivity between the piles within each wall (no waler beams were used). The wall was then finished with a layer of shotcrete to prevent soil spalling through the pile-to-pile gap. Battered slopes with a permanent gradient of 1V:2H or flatter were also introduced above the walls to reduce the vertical wall height. The height of the slope was typically less than 6 m and governed by the horizontal offset between the project boundary and the wall alignment (i.e. a taller wall with a smaller upper slope would be required in areas where the wall was next to the project boundary.

Design efficiency was also achieved by developing a database of representative plane strain analysis sections using PLAXIS 2D which could then be allocated along the alignment based on wall and upper slope height. The design ground models were generalised by adopting sandier profiles to account for the inherent variability in ground conditions and widely spaced investigation information at the wall locations. The primary wall design criteria included:

- 120-year design life.
- Ultimate Limit State Factor of Safety (FoS) criteria:
  - Long term static load case: FoS ≥ 1.5;
  - Short term static load case: FoS ≥ 1.3 (such as in the planned over-dig scenario);
  - Short term transient and accidental design cases: FoS ≥ 1.2 (such as unplanned over-dig); and
  - Pseudo-static earthquake load case: FoS ≥ 1.1.
- Serviceability Limit State design criteria, where H_{exc} is the depth of excavation in front of the wall:
  - Short term deflection limit – $H_{exc}/125$ or $0.8\%H_{exc}$;
  - Long term deflection limit – $H_{exc}/100$ or $1\%H_{exc}$; and
  - Deflection not to cause damage to adjacent property beyond acceptable limits.

- Over-excavation criteria: 1.05 m planned allowance for service installation within the rail formation and an additional 0.5 m accidental allowance (i.e. 1.55 m total), which may occur at any point in the design life.
- 20 kPa surcharge requirement behind the permanent works boundary.

The wall designs were primarily serviceability driven, requiring control of deflections for the taller retained heights (typically >6 m). This aspect of design required consideration of soil structure interaction at relatively high soil strain conditions and the related challenge of meeting deflection performance criteria. The use of propping (temporary or permanent) was not practical given spatial constraints within the alignment, and anchoring or tiebacks would have extended beyond the project boundary which was not permitted. An alternative solution was developed which increased the wall flexural stiffness by introducing a rear row of piles and, whilst narrow in section, provided additional restraint due to the portal frame action via the capping beam. The rear piles were typically positioned behind alternating front piles at varying spacing depending on deflection control requirements and were integrated into a widened capping beam. The solution was adequately space-proofed and significantly reduced wall deflections to acceptable magnitudes. An indicative schematic showing the wall arrangements is shown in Figure 3.

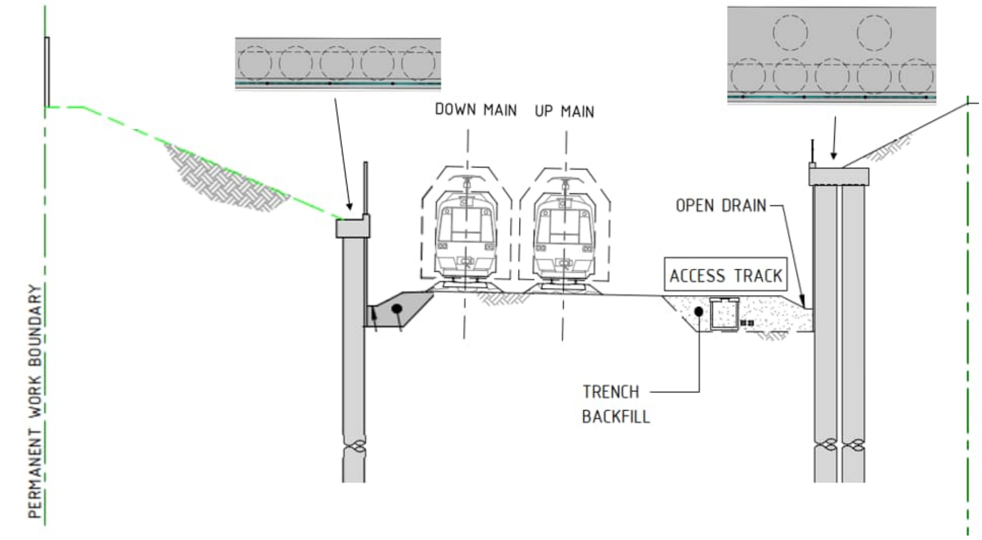


Figure 3. Typical section through piled walls showing single pile row (left) and double row arrangement (right).

4 STRAIN SOFTENING

4.1 WHAT IS STRAIN SOFTENING?

Dilatancy describes the change in a soil's volume as shearing is applied and can be used to explain the "strain softening" phenomenon. As shown in Figure 4 below, an initially loose, granular soil will typically contract in response to increased shearing and is characterised by the progressive decrease in volume and increase in shear strength. In contrast, a denser soil will undergo an initial contraction, followed by an increase in volume (dilative behaviour) as more shearing is induced. The shear strength reaches a peak strength but at sufficiently large strains, will exhibit a reduction in strength as it approaches the critical state (defined as the point where ongoing shearing will occur at a constant volume). It is prudent to ensure that the strength parameters adopted in design and analyses are appropriate for and reflect the level of strain and deformation that is likely to occur.

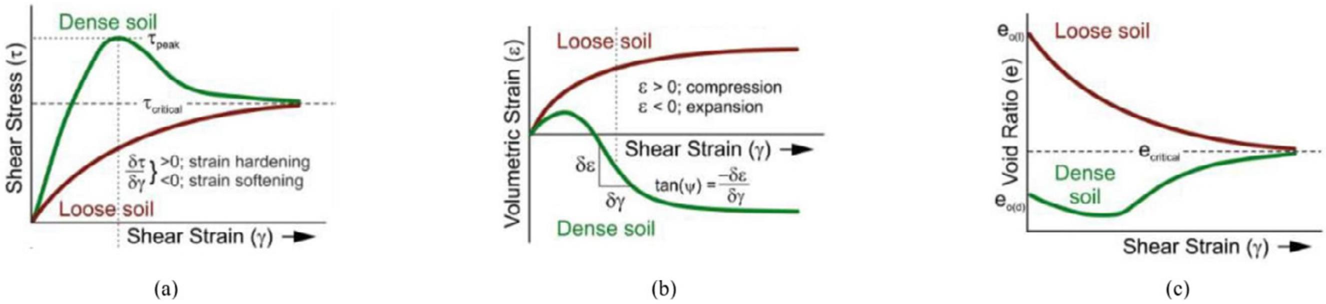


Figure 4. Behaviour of loose and dense soils with increasing shear strains (Keaton, 2018)

4.2 THE DEVELOPED "STRAIN SOFTENING METHOD"

Geotechnical analyses to support the structural design of contiguous piled walls and bridges were undertaken using PLAXIS 2D, a two-dimensional finite element analysis package developed by Bentley. For some structures, where the geometry or construction staging was increasingly complex, a three-dimensional model using PLAXIS 3D was adopted. The estimated pile behaviour, namely pile bending moment, shear force and lateral movements from PLAXIS were observed to be a reasonably close match with other software packages, such as WALLAP and RS2.

Due to the height of the proposed unsupported, cantilevering and relatively slender walls, in conjunction with the over-excavation requirements and challenging ground conditions discussed in Sections 2 and 3, localised regions of relatively high strains within the soil were identified in the analyses indicating relatively extensive zones of Mohr-Coulomb yielding. Corresponding lateral wall movements were due to a combination of active strain within retained soils, passive soil displacement in front of the piles and pile wall flexure. In some instances, the lateral movement at the pile crest was estimated to be in the order of 1% of the pile retained height, which exceeds the typical guidance and commentary provided in various design guidelines such as CIRIA C760 (CIRIA, 2017). A literature review also indicated limited case histories of fully cantilevered contiguous piled structures of this height in similar ground conditions. Consequently, it was prudent that the analytical tools and methods adopted were able to account for the post-peak strength loss and behaviour likely to occur at relatively large soil strains.

At the time of writing, PLAXIS does not offer a standardised "off-the-shelf" constitutive soil model which allows the strain softening phenomenon to be suitably modelled. It is noted that a GSI-softening model is available though this is generally not compatible with the soil materials observed along the YRE alignment. Other numerical analyses along the alignment were previously undertaken in PLAXIS 2D and therefore remained the preferred analytical tool for consistency and familiarity, given the multi-disciplinary nature of the project. Therefore, a manual softening process was developed for use with PLAXIS 2D software and was subsequently adopted in the design of YRE contiguous piled structures. This approach is herein referred to as the "*Strain Softening Method*" and its key components and steps are summarised as follows:

1. The active and passive regions of soil around the pile were split into four zones as shown in Figure 5a, below.
2. The Mohr-Coulomb and Hardening Soil constitutive soil models were adopted and the design strength parameters (i.e. peak) were initially assigned to all soil zones within the model.
3. A screening analysis was undertaken which involved modelling different pile arrangements (i.e. pile height, retained slope heights) and depths of excavation. The corresponding lateral movements were assessed at different depths of excavation to inform when softening was required.
4. For soils where screening analysis showed that strain softening was expected to occur at sufficiently large strains in medium dense or denser soils, two additional sets of strength parameters were developed to reflect the post-peak soil behaviour as follows:
    a. "50% Softened" strength parameters represented the mean of the peak and critical state strength parameters. This is graphically illustrated in Figure 5b below.
    b. "100% Softened" strength parameters represented the critical state or residual strength.
5. As increasing depths of excavation in front of the pile were modelled in PLAXIS, a larger U_x/H was observed, where U_x was the lateral pile crest movement and H reflected the permanent pile retained height.
    a. Depending on the U_x/H achieved at each stage of construction, the soil strength of each zone was progressively reduced in accordance with Table 2 below.

Figure 5a depicts the typical strain distribution in the soil around the piled wall. The development of active and passive wedges are clearly shown and it is acknowledged that different regions of soil would undergo varying degrees of strain softening in response to the amount of strain induced. However, modelling the degree of strain softening in all soil regions is not practical or efficient. Therefore, the primary intent of developing and using this *Strain Softening Method* was to capture the overall behaviour and impact of strain softening in the context of pile movement and actions, through an intuitive, simplified and repeatable approach. An example of this process is graphically represented in Figure 6, below.

RS2, a separate geotechnical finite element software package developed by RocScience, offers an embedded constitutive model to simulate the strain softening behaviour in soils. A benchmarking exercise was undertaken to compare results from the PLAXIS 2D *Strain Softening Method* models with RS2 embedded strain softening model, which resulted in similar pile actions and movements.

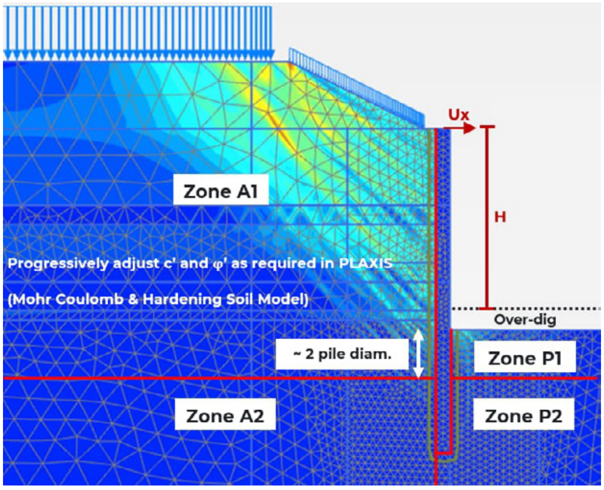


Figure 5(a): Schematic showing key soil zones used in the *Strain Softening Method*

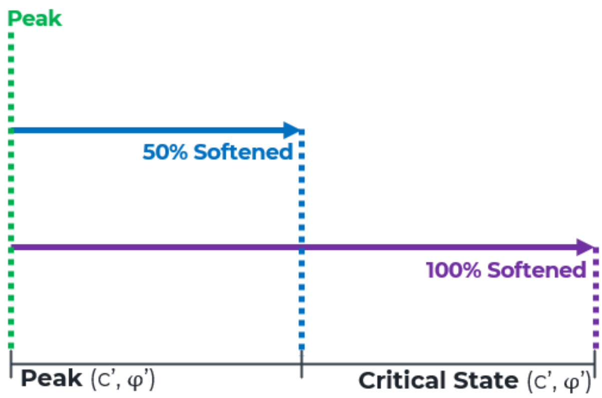
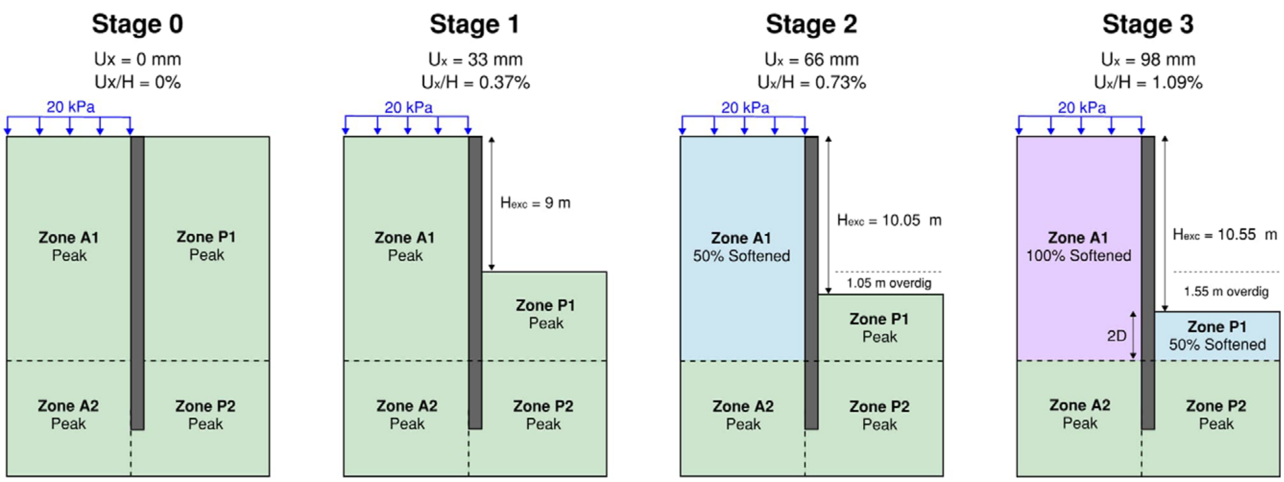


Figure 5(b): Development and relationship between peak and softened strength parameters

Table 2 – Criteria for assigning strain softened strength parameters to soil zones

U_x/H ratio	Zone A1	Zone P1	Zone A2	Zone P2
$U_x/H < 0.5\%$	Peak	Peak	Peak	Peak
$0.5\% \leq U_x/H < 0.75\%$	50% Softened	Peak	Peak	Peak
$U_x/H \geq 0.75\%$	100% Softened	50% Softened	Peak	Peak



Where:

Peak : φ' = 35 degrees, c' = 0 kPa

50% Softened: φ' = 33.5 degrees, c' = 0 kPa

100% Softened: φ' = 32 degrees, c' = 0 kPa

D = 1.05 m

H = 9 m

Figure 6: Worked example illustrating the *Strain Softening Method*

4.3 DISCUSSION

The effects of strain softening were shown to significantly impact the structural and civil design in the scenario of high free-standing cantilevered pile walls. For example, Figure 7 below compares the estimated bending moment and lateral pile movement across three stages of excavation. The solid lines represent results obtained by following the *Strain Softening Method* whilst the dashed lines were obtained by considering the design strength parameters only. The key observations are as follows:

- Bending moment and lateral wall displacements increased exponentially once the depth of excavation in front of the wall progressed beyond a certain depth, triggering the need to consider strain softening.
- The initial design strength parameters were appropriate for Stage 1 excavation, given that U_x/H was less than 0.5%.
- As excavation progressed deeper to Stage 2, the U_x/H was between 0.5% and 0.75%. Therefore, the 50% Softened Parameters were adopted.
- Once the Stage 3 excavation depth was reached, the U_x/H exceeded 0.75% and the 100% Softened Parameters were subsequently assigned.
- The *Strain Softening Method* resulted in about a 10% and 20% increase in estimated bending moment and lateral movement in Stages 2 and 3, respectively (as compared to when the initial design strength parameters were adopted instead). This is a notable difference, which can influence the design and specification of pile reinforcement cages.

Despite its usefulness, there are some limitations of this manual *Strain Softening Method*, which include:

- An abrupt change in strength parameters is imposed when the limiting values of U_x/H are reached. Careful and detailed assessment of the strain fields should be examined in these instances to test the impact upon resulting pile movements and behaviour. Manual adjustments may be required where U_x/H are close to the bounding limits.
- At least two runs of the PLAXIS 2D model is required to check the U_x/H ratios achieved and to amend the strength parameters accordingly. This results in increased analysis time and requires manual inspection and iteration.
- Some soil regions located outside of the active and passive failure wedges may be "softened" through this process despite limited strains being developed in the area. For the purpose of wall design, this impact is negligible but may require additional adjustments in cases where the model is used to assess ground movements or interaction with other structures located away from the wall.

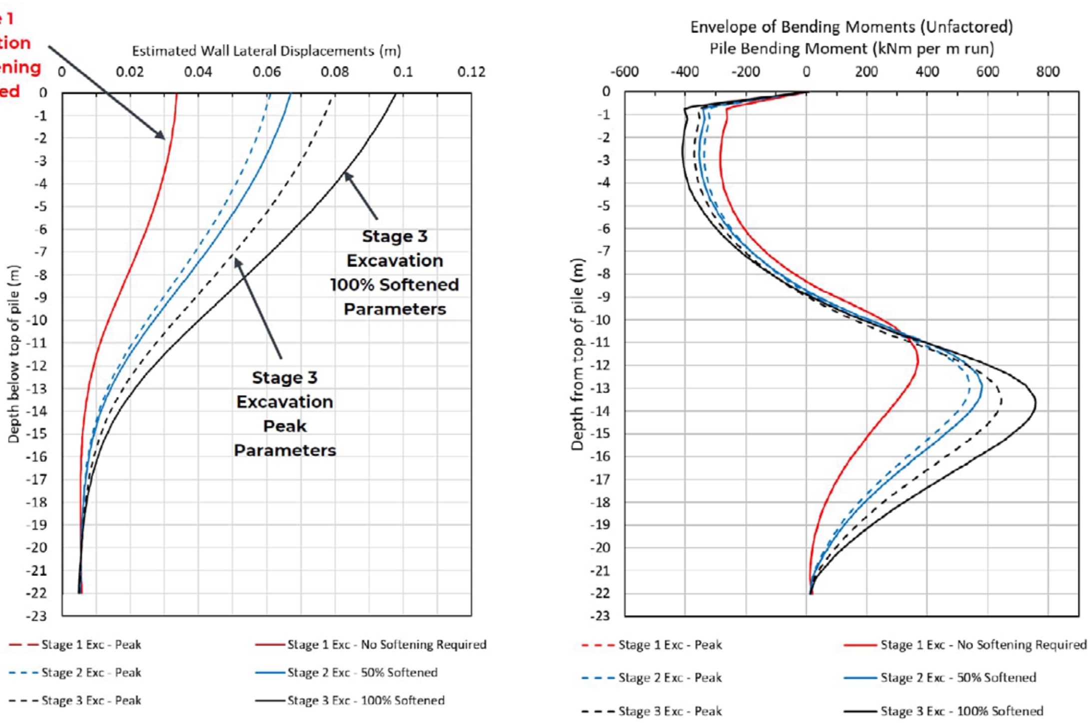


Figure 7: (a) left: Impact of the *Strain Softening Method* upon estimated pile lateral movement and (b) right: pile bending moment

5 KARSTIC RISK

5.1 OVERVIEW OF KARSTIC RISK

Desk studies and early stage investigations identified the potential for karstic conditions along the project corridor. According to the City of Wanneroo Karstic Features Zones Map (2018), the YRE rail corridor alignment predominantly transects zones of mapped ‘low’ karst risk, with about 30% of the alignment located within zones of mapped ‘medium’ karst risk.

As outlined in Bastian (2003), several hundred limestone caves exist in the Yanchep area, and mainly within the boundary of the Yanchep National Park. In this part of the Perth Basin, the Tamala Limestone abuts and in places overlies an extensive formation of water-bearing Bassendean Sand, and the contact between the two slopes westwards towards the coast. As surface and groundwater migrates coastward in sands along the interdune corridor in the Yanchep area, shallow groundwater and lake systems develop.

Where groundwater intersects the base of the overlying limestone, groundwater is virtually free of dissolved carbonate and can dissolve the limestone vigorously. Due to seasonal fluctuations of the water table, where it comes into contact with limestone there is elevated karst risk, resulting in a belt of karstic features including dolines, fissures, and numerous caves, some of which can be entered and others which have collapse potential. Risk can also be elevated where the natural groundwater conditions are altered, such as through the construction of drainage basins or soak wells.

Figure 8 below, shows an example of a large solution feature at Crystal Cave, located in the Yanchep National Park, about 4.2 km NNE of Pipidinny Road Bridge. Other karstic features were also encountered during bulk earthworks and foundation preparation for gravity retaining walls and structures along the YRE alignment, which is also shown below.

As the current regional groundwater water table along the YRE rail corridor is well below the proposed cut levels and retaining wall excavation depth (in the order of 10 m to 25 m lower), the corresponding risk of ongoing formation of large, problematic karstic features was generally reduced. However, water tables have fluctuated over time and were known to be significantly higher in past geological time. Consequently, several karst features were identified during investigation and bulk earthworks which required active management through diligent additional investigations, construction monitoring and contingency planning.

Figure 8: Example of a karstic features: (a) left: interior of Crystal Cave (Morey, 2025)**, a solution cave located 4 km from Pipidinny Road Bridge and (b) right: karstic features encountered during earthworks along YRE**

5.2 KARST TREATMENT AT PIPIDINNY ROAD BRIDGE

Geotechnical investigations within the YRE rail corridor identified an increased frequency of relatively small karst features, most notably in the vicinity of Pipidinny Road Bridge, as summarised in Figure 9 below.

Here, a potential cavity was identified in BH47+533(E), where a 0.5 m thick zone of core loss was observed at 6.0 m depth and it was noted that the rods dropped about 0.2 m. Additionally, at 11.0 m depth, the SPT rod was observed to drop 0.95 m after the first blow and only recovered limestone gravel fragments. These drops occurred adjacent to zones of uncemented sand or core loss which may be indicative of subsurface voids or cavities. Investigation concerns were confirmed based on early works piling and construction observations as summarised in Table 3.

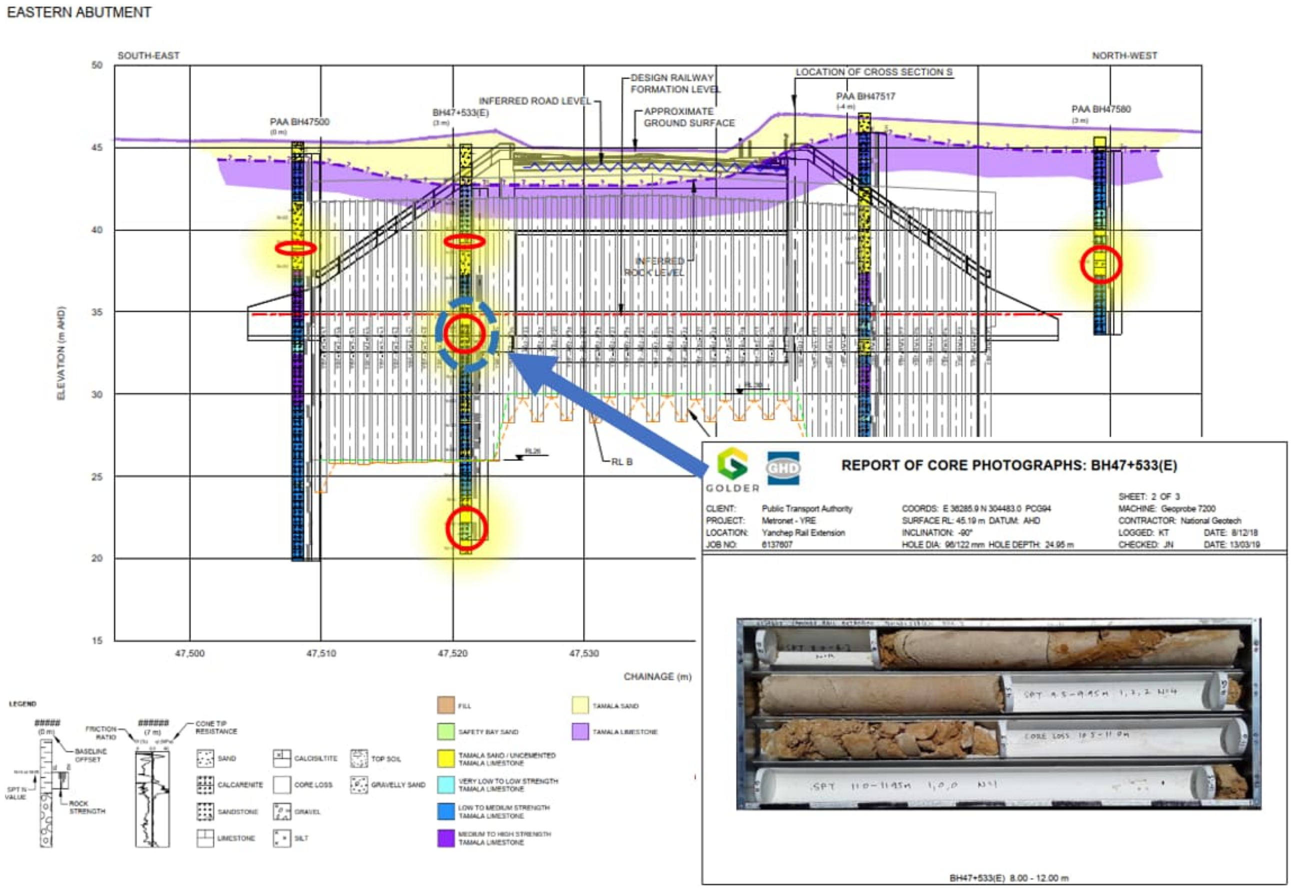


Figure 9: Example of karstic feature identified during geotechnical investigations along YRE

Table 3 – Karst features during early construction

Chainage	Structure	Description
Ch44,442	Road overbridge, piled	Solution feature observed by downhole camera in pile socket
Ch45290 – Ch45125	Alignment retaining wall, gravity structure	Void observed in batter slope. Over-excavation of adjacent footing revealed a void beneath the footing. Over-excavation carried out with Ground Penetrating Radar (GPR) on final surface. Geogrid placed on prior to backfilling
Ch47500 – Ch47570	Piled road over bridge	Karstic voids encountered during piling, with resulting concrete loss. Grouting plan + GPR proposed
Ch50230 – Ch50320	Fauna overbridge, gravity structure	Crack observed in footing excavation. GPR carried out to assess for larger void. No void observed, no further mitigation measures specified

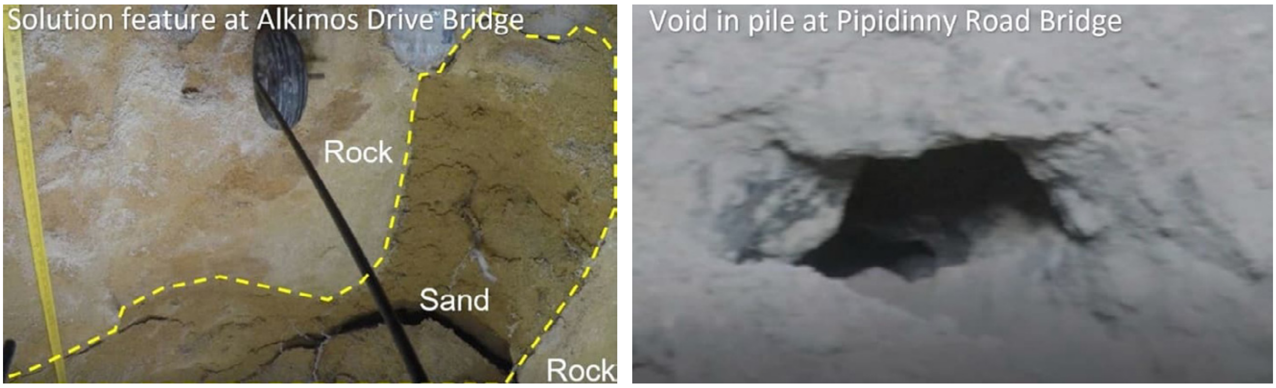


Figure 10: Examples of karstic features observed by during piling with a downhole camera

Pile construction commenced at Pipidinny Road Overbridge in late 2021. Prior to commencement of production piling, two high-strain dynamic pile load tests were undertaken adjacent to the southern wing walls. The results indicated that all tested piles met the required ultimate geotechnical design loads in compression. No voids were encountered during the test piling. However, cavities were encountered during later construction of some 900 mm diameter contiguous production piles resulting in concrete losses at three piles of 8.5, 12.6 and 21 m^3. The following measures were initially adopted to address the presence of voids:

Method 1: Where voids were encountered in the pile shaft prior to the concrete pour:

a. Concrete pour carried out to allow void to be filled and stabilise the pile. A constant head of concrete was maintained prior to stopping the pour.

b. Where an excess of concrete was consumed, the pour was stopped, and the pile backfilled with cuttings.

c. The pile was re-drilled the next day. Provided the socket was consistent (no voids), concrete pour was carried out as per the standard methodology.

Method 2: Where concrete was lost during the pile pour, as indicated by concrete height not increasing to design level:

a. The cage was removed from the pile, and the void attempted to be filled with available concrete on site.

b. The concrete head was monitored during the pour, and where it did not increase the pour was stopped after trucks on site discharged.

c. The following day, the pile was re-drilled and inspected. Where the socket was sound (no voids), the cage was re-inserted and concreting undertaken, otherwise Method 1 above was repeated.

Backfill of one pile was initially attempted with a flowable sand-grout mix (low strength mix with high slump), however this mix was assessed to be overly viscous and unlikely to penetrate an aperture to adequately fill the encountered void. Due to the elevated karst risk, it was decided after void encounters in early piles to deepen remaining pile toe levels at both abutments by 1.5m.

Cured concrete overbreak from ‘leaking’ piles installed in voided ground was encountered during the drilling of 16 later ‘gap-filling’ contiguous piles, which confirmed that that concrete in karstic ground had flowed laterally into the surrounding rock mass. To mitigate residual risk of remaining cavities on the cantilever retaining wall performance, post-construction grouting was implemented.

The grouting program was developed as follows:

- The target zone included potentially critical voids in the passive zone extending laterally within about 3 pile diameters (i.e. approximately 3 m) in front of the piled wall and over the full pile embedment depth.
- A two-stage grout hole pattern was adopted using a bottom-up grouting approach, and a single packer system for both primary and secondary grout holes as shown conceptually in Figure 11 below.
- Primary grout holes were drilled in a single row offset 1.5 m from the eastern face of the piles with a row of gap-filling secondary holes between initial primary holes.
- Casing was drilled into the top of limestone (typically 8 m depth below partial cut level) and an HQ-diameter hole extended to the base of the grout hole before lifting in stages.
- A cementitious grout comprising Ordinary Portland Cement (OPC) was used to grout the rock mass with a water/cement ratio in the range of 0.6 to 1.1:1 (with plasticizer).

- At completion of each grouting stage, the packer was deflated and pulled up about 2.0 m above the next stage elevation and the process repeated. QA procedures included grout testing and post-grouting coring.
- Grout stage refusal occurred when the maximum well-head pressure was reached, and take was negligible. Maximum well-head pressure was assessed as a function of the depth of the grout stage and the mix.
- Where holes were taking well, grout flow was allowed to continue. In the case of large voids or runaway takes, the mix required thickening incrementally to reduce bleed and induce refusal.
- Hydrostatic well head pressures of between 125 kPa (1.25 Bar) and 270 kPa (3 Bar) were adopted for staging. The use of high grout injection pressures was considered inappropriate since the ground does not contain small aperture discontinuities typical of a jointed rock mass. The use of low grout injection pressures also mitigated the risk of jacking/heave and damaging the rock mass.
- Overall, after the initial concrete volume loss during piling (in the order of 42 m^3), at the western bridge abutment, an additional 13 m^3 of grout was injected as part of the remedial grouting program, indicating a potential cavity network of about 55 m^3 in the vicinity of the bridge structure.

A plan and elevation of Pipidinny Road Bridge Western abutment showing the layout of grout holes and grout take along and below the abutment is shown in Figure 11.

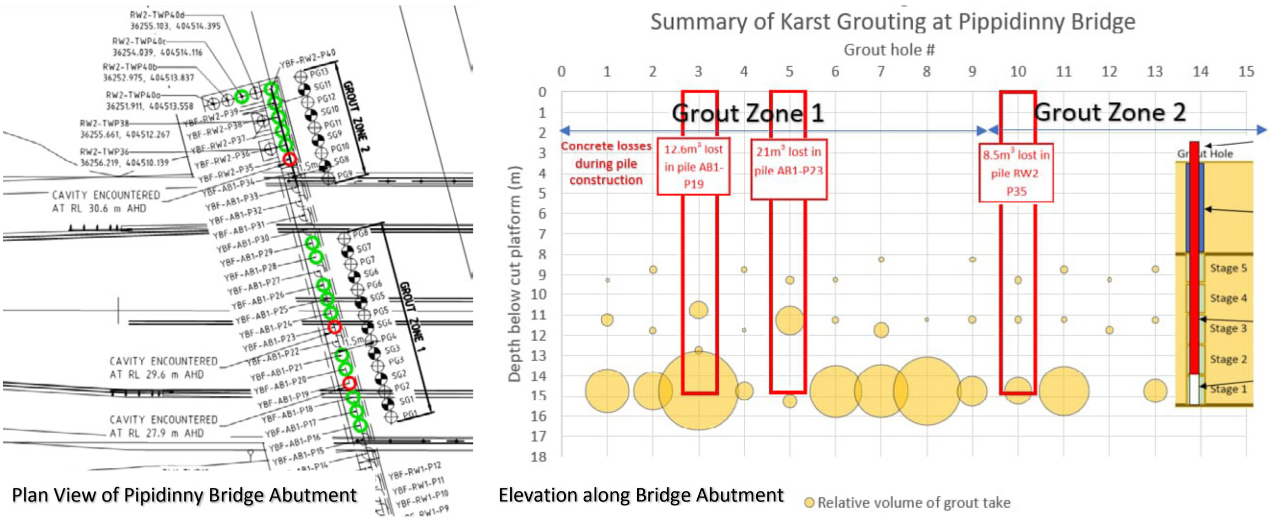


Figure 11: Plan (left) and elevation (right) of grouting operations at Pipidinny Road Bridge

Lessons learned when dealing with karstic ground conditions on this project were as follows:

- Investigation, design, risk management and treatment resulted in an evolving design, specification and testing process; the risk position was sharpened by experience at Pipidinny Road Bridge.
- Spatial uncertainty and variability made complete karst risk identification and mitigation by conventional investigation difficult and inefficient; residual risk required management during construction.
- Except for high-risk areas where karstic ground was identified through design stage boreholes, construction risks during piling were managed through a combination of downhole/pile camera inspections, close monitoring of concrete loss, diligent logging of concrete return in gap filling piles, GPR and grouting where needed.
- GPR provided a useful quick 'yes/no' screening tool for potential large near-surface voids. Where high risk areas were identified through GPR screening or other construction indicators, additional probing (DCP, air-track / conventional targeted drilling and optionally grouting) was sometimes needed depending on relative proximity and sensitivity of nearby structures.
- Geotechnical and structural design teams worked efficiently to collaborate with each other and the contractor to agree on a quick and efficient decision tree-process to reduce collective risk, as discussed in Section 5.3, below.
- Subsequent quality assurance post-grouting coring and Instrumentation and Monitoring (I&M, predominantly comprising in-place inclinometers and wall survey targets/prisms) assisted with performance verification during subsequent excavation stages and validated the adopted process.

A photograph of Pipidinny Road Bridge during construction is shown in Figure 12, below.

Figure 12: Contiguous Piles exposed after bulk excavation during construction of Pipidinny Road Bridge

5.3 PILING DECISION TREE, SUPERVISION REQUIREMENTS

Full time supervision of piling works was implemented both for piled corridor retaining walls and road over bridges such as Pipidinny Road Bridge. Pile design and socket requirements were based on the critical ground conditions encountered at each structure from site investigation and initial pile testing works, however it was understood that due to ground variability, poorer conditions could be encountered during production piling at some locations. For example, Figure 13 below shows the as-logged soil/rock conditions along the length of one of the corridor walls.

Piling decision trees were developed to allow rapid risk screening and site-based decisions or alternatively trigger escalation through the design team. For example, in the instance where thick sand zones or karstic features were encountered within the pile socket or rock level was deeper than anticipated within a given pile. The proposed solutions were tailored to the sensitivity of the relevant structure. An example decision tree for management of karstic risk is shown in the Figure 14 below.

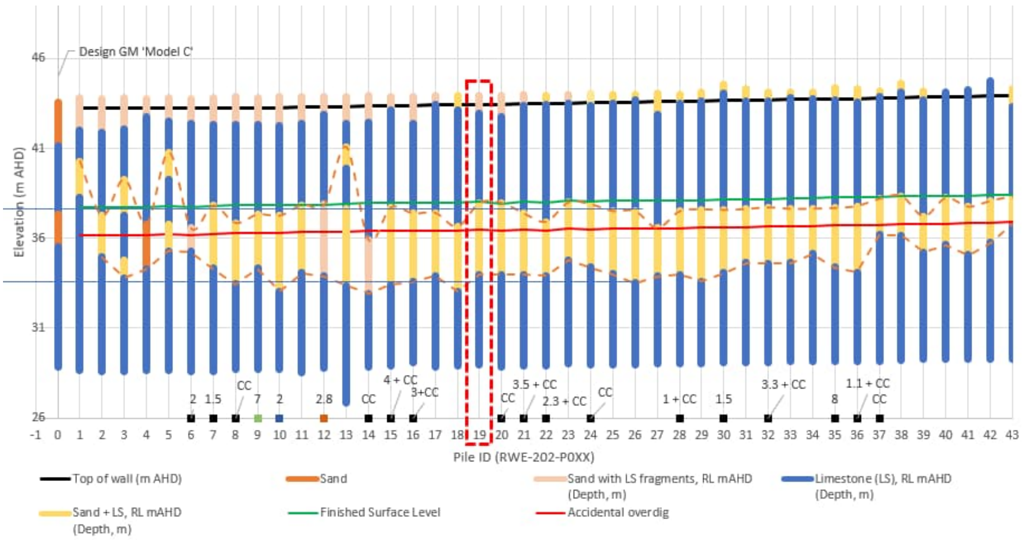


Figure 13: Long section to highlight the variable ground conditions encountered during piling

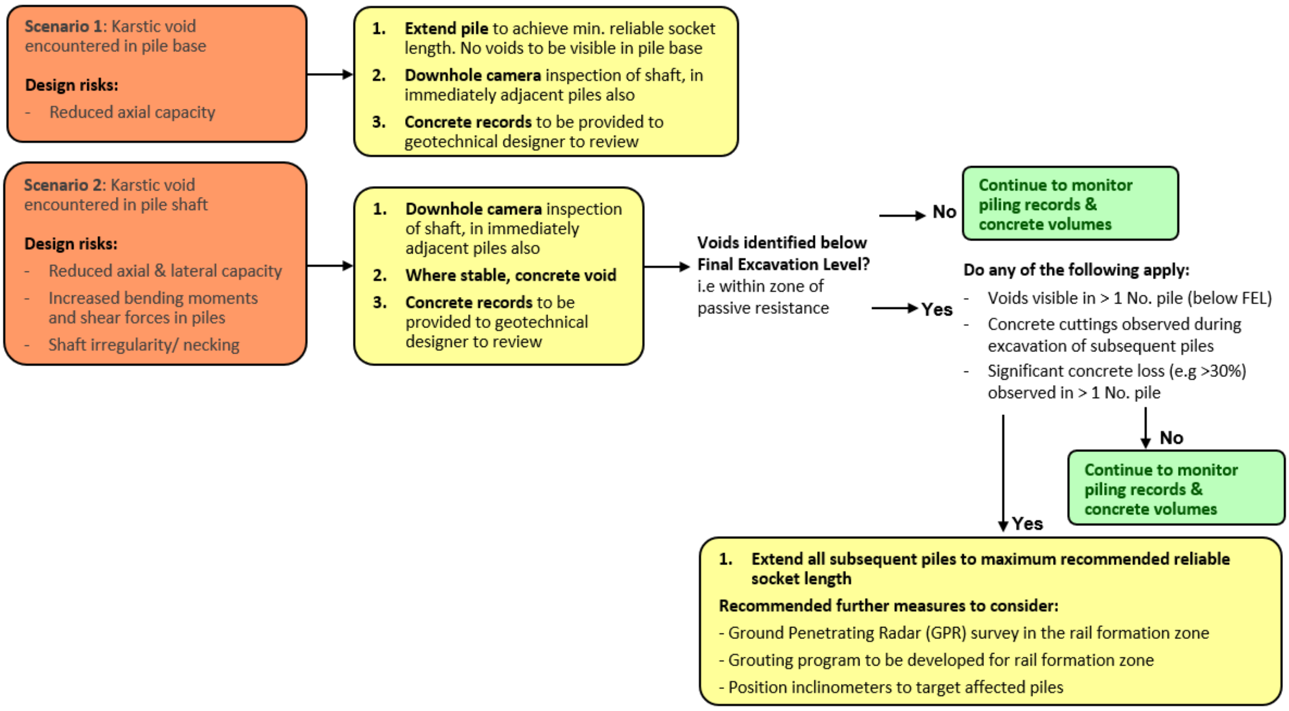


Figure 14: Example Decision Tree used during YRE Piling Works

6 PILE INTEGRITY, INSTRUMENTATION AND MONITORING

6.1 PILE INTEGRITY TESTING

Thermal Integrity Profiling (TIP) was adopted as the primary method for assessing the integrity of the constructed piles, with testing completed by CMR Technical Services. TIP is a non-destructive test method for evaluating the post construction quality of cast-in place foundations. Temperature measurements (hydration energy) from the curing cement are used to assess shaft shape and integrity, concrete quality, and the relative alignment of the reinforcing cage. The testing frequency guideline was considered on a per-wall-basis as a minimum of 6 piles or 3.5% of the overall number of piles in the wall (whichever was greater). Tests were allocated with consideration to the pile arrangement (single piles, front and rear piles), ground conditions, retained height and wall length. Testing was carried out in accordance with ASTM D7949-14 Method B and the manufacturer's recommendations, using the Thermal Integrity Profiler and the thermal wire system developed by Pile Dynamics Inc.

Necking and bulging characteristics were relatively common in the TIP profiles and considered, in-part, to reflect the influence of variably cemented ground conditions on bored pile construction. Shaft enlargement typically occurred through uncemented zones/layers which was likely caused from "flighting", where the auger had advanced ahead of the casing. Necking characteristics were observed over the cased length, however the casing thickness (and additional concrete) assisted to balance out the impact on the net diameter.

Bulging profiles were not considered adverse for geotechnical design but had potential to cause constructability challenges for adjacent pile installation, wall drainage installation and shotcrete installation. Necking profiles required further review by the structural and geotechnical teams to assess the influence on the wall design. Figure 15, below provides an example of TIP test profiles for two of the more adverse tests results across the alignment.

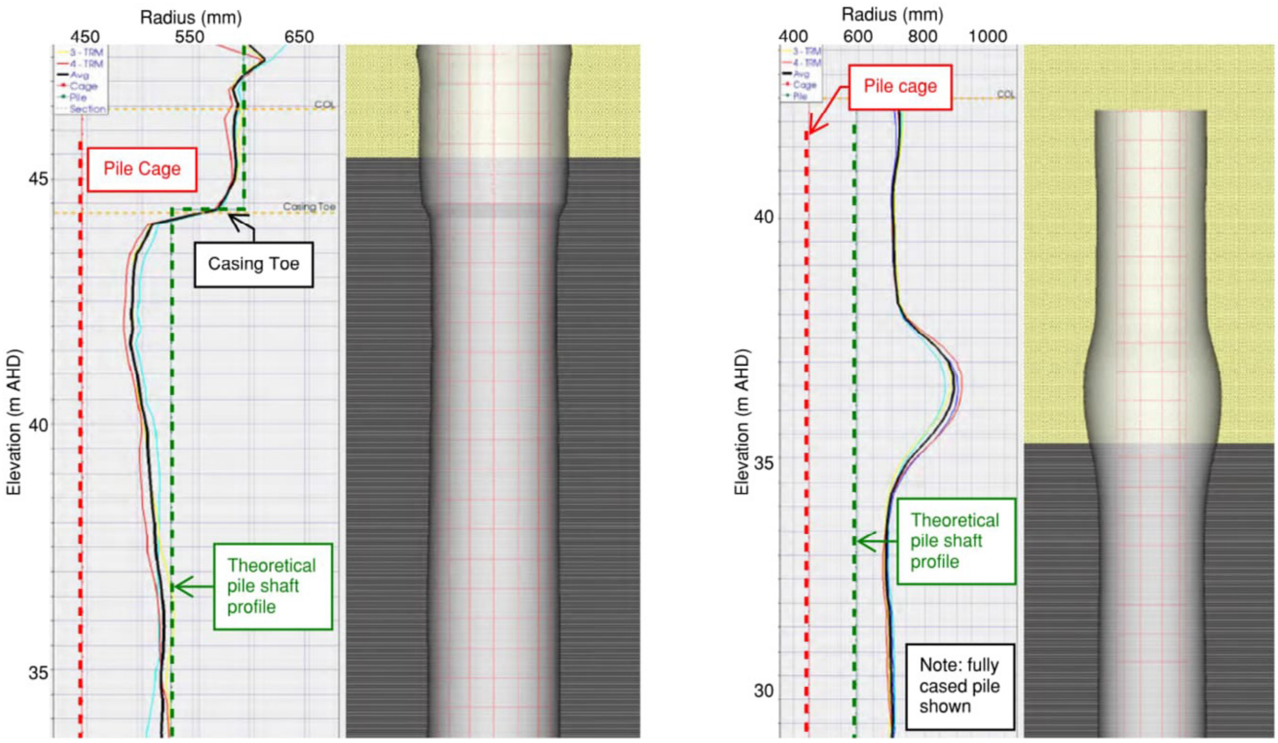


Figure 15: TIP test extracts showing necking (left) and bulging characteristics (right)

6.2 MOVEMENT MONITORING

Wall movement and ground-borne vibration were monitored during construction of the rail corridor to verify wall performance and to form a suitable proxy for monitoring ground movement impact to adjacent third-party assets (typically residential road infrastructure and underground services). Three forms of instrumentation were adopted as follows:

1. Optical Prisms – An upper row of prisms located at the mid-point of the capping beam, and a lower row offset about 1 m above finished surface level (FSL). The prisms were spaced between about 7 m to 10 m c/c, and attached to steel pins (10 mm diameter, 100 mm long) that were cast into the concrete during the wall construction. Survey of the prisms was conducted using a Total Station.
2. Inclinometers – Automated in-place inclinometers (IPI) were installed at select locations perceived to be of a higher risk, based on a review of ground conditions, retained height, predicted movement, and proximity to third-party assets. The inclinometers were typically installed in PVC reservation tubes that were cast into the pile, however several locations required installation in boreholes behind the pile due to construction challenges. The string of tilt sensors was spaced at 1 m c/c extending from the pile head to below the point of curvature, with the spacing increasing to 2 m c/c towards the pile toe. Data loggers provided real-time monitoring which was uploaded to a cloud network.
3. Vibration Sensors – Sensors were used to monitor ground-borne vibration induced by the construction works against the threshold limits that were set based on industry guidance or asset owner specification. Sensors were not expected to be at a fixed locations but instead be positioned at the minimum offset between the relevant asset and the nearest source of activity.

Recommendations for each wall were documented in a Ground Movement Impact Assessment (GMIA) and Instrumentation and Monitoring Plan (IMP) and communicated on the Issued-For-Construction (IFC) drawing packs. Measurements were required as soon as practical to establish a baseline, and then at regular time intervals during bulk excavation and/or changes in loading behind the wall. Monitoring Review Limits (MRL) were developed based on the analysis results for two primary excavation stages, 1) bulk excavation to Finished Surface Level (FSL); and 2) 1.05 m planned over-excavation below FSL. The MRLs were separated into three escalation levels based on movement magnitude, and with increasing response requirements as follows:

- *Action* – Less than 70% of the design movement value; continue to review trends.
- *Design* – Greater than 70% but less than 100% of the design movement value; increase monitoring frequency, additional assessments as required based on likelihood and criticality of further movements.
- *Alarm* – Greater than 100% of the design value; inspection by design team, stop-work if deemed necessary, modified construction methods, design adjustments.

General trends of the monitoring results received for several walls are provided below:

- Survey readings from the optical prisms indicate the walls have performed well, with actual wall movements generally remaining within or below the Action MRLs at practical end-of-construction. This is considered a reflection of the robust design approach, the influence of cemented strata behind and/or in front of the walls, and the influence of construction controls which reduced temporary excavation footprints and maintained passive soil in front of the walls during the over-excavation scenario.
- Due to procurement issues, the in-place inclinometers were typically installed after bulk excavation had progressed. Manually operated inclinometers were conducted to supplement the data, however, in general the full pile movement was not captured. Whilst the ability to compare actual movement against predictions is limited, the instrumentation assisted to confirm wall movements had stabilised at the end of bulk excavation works (as expected) compared to the fluctuations observed in the prism survey data caused by larger equipment tolerances and temperature effects.
- One known trigger of an Alarm level was caused from over-filling of the upper slope above a wall to permit construction of a noise wall at the project boundary. The Alarm was localised to the lowest retained section of the wall and impact to design and nearby assets were considered negligible. However, recommendations were provided to mitigate adverse loading progressing to the taller retained sections.
- No known impact to the functionality of third-party assets caused by wall movements or ground-borne vibrations.

7 CONCLUSIONS

The Yanchep Rail Extension (YRE) alignment spans 15 km to the north of the existing Butler Station and traverses regions well known for its challenging and inherently variable karstic ground conditions. Due to the height of the proposed unsupported, cantilevering and relatively slender contiguous piled structures required, in conjunction with the potentially variable and adverse ground conditions anticipated, considerable pile and soil movements were expected to occur. There is limited precedence of for such structures to be construction within these ground conditions and it was prudent that post-peak strength loss was appropriately considered in design.

In lieu of there being a suitable, standardised PLAXIS 2D constitutive soil model, the *Strain Softening Method* was developed and adopted in the YRE design. This method was an intuitive, simplified and repeatable analytical approach which facilitated consideration of the impacts of localised regions of strain softening. This approach indicated that post-peak strength loss can result in up to 10% to 20% of additional lateral pile movements and bending moments, compared to when the strain softening phenomenon is ignored. There were, however, several limitations of this method which were discussed above.

Various karstic features were encountered during the geotechnical investigations and construction along the YRE alignment, which comprised voids and subsequent concrete loss during piling. Spatial uncertainty and variability made complete karst risk identification and mitigation by conventional investigation difficult and inefficient. A combination of downhole camera inspections, instrumentation and monitoring, diligent piling supervision, GPR and use of targeted drilling and grouting regimes were adopted to manage the residual risks encountered during construction. The geotechnical, structural and construction teams worked collaboratively to agree upon a quick and efficient decision tree process to reduce the collective risk.

Pile integrity testing and wall movement and vibration monitoring were specified and undertaken to verify wall performance and formed a proxy for monitoring ground movement impacts to sensitive nearby assets. The results of the testing and monitoring indicated negligible adverse effects to surrounding infrastructure.

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Any opinions, findings, conclusions and recommendations in this paper are those of the authors only and do not necessarily reflect the views of the other parties listed above.

CRediT authorship contribution statement

Ken Chen: Formal Analysis, Methodology, Validation, Visualization, Writing – original draft, Writing – review & editing. **Michael Wiechecki:** Formal Analysis, Methodology, Validation, Visualization, Writing – original draft, Writing – review & editing. **Jessica Dalton:** Methodology, Writing – original draft, Writing – review & editing. **Philip Davies:** Methodology, Supervision, Writing – original draft, Writing – review & editing.

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