Proceedings of the 7th International Conference on Geotechnical and Geophysical Site Characterization
Barcelona, 18 - 21 June 2024

When did geotechnical data become a point of view: a case of numerical analysis vs site data

Burt Look

AGTRE Pty Ltd, Queensland, Australia
lookburt@agtre.au

ABSTRACT

The construction of a permanent bridge required a temporary bridge parallel to the permanent bridge alignment. Preloading of the permanent bridge abutment occurred under a separate early earthworks contract. That fill was removed, and the temporary structure was constructed. The fill was then replaced behind the abutment. Movement of the southern abutment of the permanent bridge was identified but with a gap of several weeks in survey monitoring due to a XMAS break period. The abutment had moved towards the river and temporary bridge. Potential causes for movement were investigated by additional investigation adjacent to and far away from the temporary bridge piles.

Post movement tests carried out included: Dilatometer Testing (DMT) to assess for shear zones (if any) for slope instability, Cone Penetration Testing (CPTu) to assess strength changes (if any) to proximity of piles and new inclinometer readings. INSAR data was also obtained.

A 3-D finite element analysis (FEA) carried out by a consultant matched the measured lateral displacements at the adjacent bridge. Based on that correlation, it was concluded that the removal of the close-ended temporary piles was the main cause of the excessive movement, and this initiated a contractual claim. Correlation is not causation. This case study provides a background on the site data and numerical analysis. The FEA did not include much of the site observational and site data post movement. Given the FEA was given the same credence as the site data, this suggests that data is now considered a point of view.

Keywords: Finite element analysis, Pile removal; DMT, CPTu; shear zones, INSAR

1. Introduction

The construction of a permanent bridge required a temporary bridge construction parallel to the permanent bridge alignment. Preloading of the permanent bridge abutment occurred under a separate early works earthworks contract. That fill was removed by the bridge contractor and the temporary bridge structure was constructed. The fill was then replaced behind the southern bridge abutment. The permanent bridge is supported by two 1050 mm diameter bored piles at the abutments, and four 1200 mm diameter bored piles for the piers. The temporary bridge used 2 No. 760mm driven closed end steel tubes 8.8m apart.

Movement of the southern abutment of the permanent bridge was identified but with a gap of several weeks in survey monitoring over a XMAS break period. The Southern abutment had moved in a north easterly direction towards the river and temporary bridge.

Chan et al. (2023) provides site and construction details leading to these movements. Some of the potential causal factors suggested for the abutment's movement included:

- Reduction in shear strength (loosening) of the ground due to temporary pile removal vibration
- Removal of support at toe of the slope (scour of the riverbank)
- Increased loading at crest of slope (filling directly behind abutment)
- The lack of wick drains between the wingwall and the abutment causing differential settlement
- Global slope instability of the riverbank
- Extreme tidal variation causing rapid drawdown
- Possible presence of deep sensitive clay layers and a pre-existing shear plane.
- Ground movement due to collapse of pile holes (i.e. soil loss) during temporary pile removal

Chan et al. (2023) used a 3-D PLAXIS finite element analysis (FEA) to model "pile hole collapse during the temporary pile removal". They concluded that the removal of the close-ended temporary piles which occurred in that gap period was the cause of the excessive movement. A contractual claim then resulted based on that FEA. Legal case studies such as this is seldom reported. Given the 2023 paper is now in the public domain, then further discussion is warranted on the other potential mechanisms and is presented in this paper.

Following the movement, additional tests were carried out including

- Dilatometer Testing (DMT) to assess for shear zones (if any) for slope instability
- Cone Penetration Testing (CPTu) to assess strength changes (if any) with proximity of piles
- A new inclinometer installation and readings.
- INSAR data was also obtained.

This case study provides a background on the site, site observations, the river geomorphology, nearby bridge historical data, and geotechnical data acquired after movement was observed. The numerical analysis did not reference these post movement site observations, data, and background. Given the FEA was the basis of the contractual claim and given the same credence as site data by several geotechnical engineers then this suggests that data is now considered a point of view.

1.1. Background of site

River Geomorphology

The site is located at a river confluence (Figure 1). The new permanent bridge is located downstream of two existing bridges. A heritage bridge was constructed in the 1930s and located furthest west and a more recent bridge constructed in the 1990s located between the heritage bridge and the new bridge. The temporary bridge was located downstream of the new permanent bridge.

Figure 1. Bridge site is at river confluence

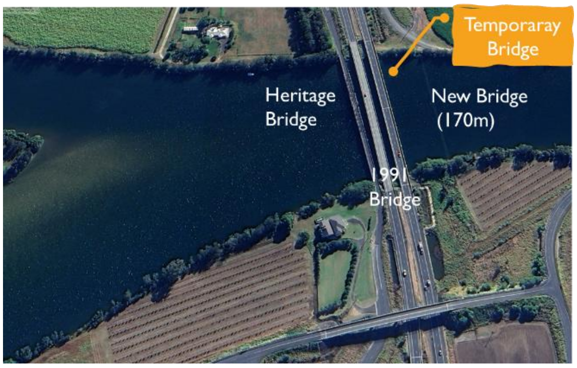


Figure 2. Permanent bridges with temporary bridge

The river diverges after the bridges which aids in the faster river flows near the narrow bridge crossings due to reduced width. Turbulent flow and higher velocities are associated at river constrictions. Ghobadian and Bajestan (2007) describe how the combined flow from 2 river branches cause extensive variation in flow pattern and creation of vortex. Because of these, scour holes develop just downstream of the river confluence. These phenomena can accelerate the rate of bank erosion and may causes failure of the bridge or structures nearby.

At this site evidence of recent riverbank erosion was evident with exposed tree roots (Figure 3) shortly after the observed bridge movement. A January 2019 survey measurements compared to December 2018 showed local loss at the crest of the riverbank.

Scour depths of 7m for the 100-year average recurrence interval (ARI) at the river piers was predicted by the hydraulic design engineers. At the abutment for the 100-year ARI, no scour depth was considered applicable since scour protection was designed for such events. At the time of the movement no scour protection was in place, but a 1.1% probability event had occurred during this period.

Figure 3. Riverbank erosion at the abutment

The river width is 225m at the new bridge location with the bed level just below RL -6m. Comparison of the riverbed surveys between 1991 and 2017 show at the location of the new permeant bridge, the river below RL -6 had changed from 60m to 120m width. This doubling in width was associated with the river now being 0.8m deeper since the construction of the 1991 bridge.

Stratigraphy at bridge site

The southern bridge abutment is underlain by Holocene age sediments comprising:

- An upper firm / soft clay unit of 4 – 5m thickness
- Overlies 9 – 10m thick interbedded loose silty sand and soft / firm clay
- The lower sediments of 8 to 10m thick stiff clays
- An older alluvium of very stiff clay and dense sand are encountered below these upper sediments
- Weathered rock units are encountered below around RL- 23m

The groundwater level is at approximately 1m to 2m below existing ground level (~ RL3) at the abutments.

Rainfall and tidal variation

Rainfall data sourced from the nearest publicly available weather monitoring station (Figure 4) shows a period of significant rainfall occurred within the period that site movement is inferred to have commenced. No survey observations were taken for the Xmas period, although pile removal was carried out.

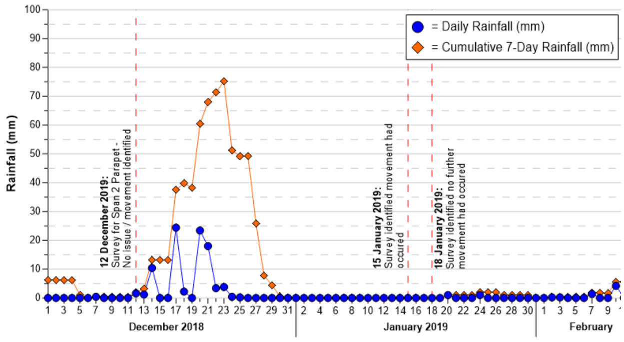


Figure 4. Rainfall data – daily (blue) and cumulative 7-day rainfall totals (orange)

The rainfall is not considered extreme but is associated with a high tide of 2.04m which occurred at the same time and has a 1.1% probability based on 1095 days of record i.e., 12 events over that period.

The tidal variation with a 0.7% high tide probability was shown to contribute to a landslide at site on the Brisbane river where the width was (coincidentally also) 225m (Look, 1999). Variation of 2.5m scour during an extreme flood event along the Brisbane river was also noted at another site, with that material being replenished within 1 month between surveys of such events.

2. Timeline and nearby issues

2.1. Heritage Bridge

Figure 5 summarizes the historical events. The survey on 12 December 2018 showed no movement after the removal and back filling of the 6 land-based temporary piles closest to the bridge abutment which occurred over an 8-day period. The removed piles between 10 December and 10 January 2019, when movement was first observed, was 9m to 75m distance from the temporary abutment. A few of those water-based piles were closer to the permanent bridge piers than those removed near the abutment. The survey did not show similar movements adjacent to those river piles.

There is no date reporting of the services trenches relocation that occurred in front of the bridge abutment.

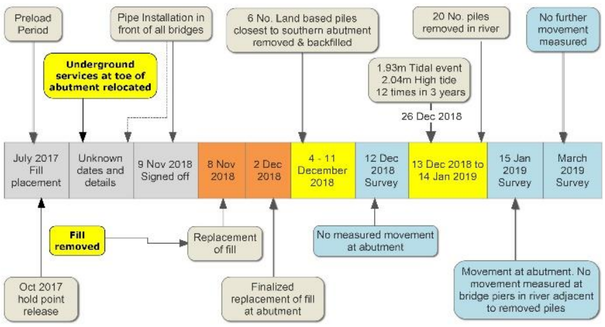


Figure 5. Time history after the fill period, its removal and reinstatement and dates of survey

2.3. Heritage Bridge

The review of the historical data shows that the heritage bridge had commenced movement in January 2014 (earliest available photo records) with 2017 repair works. In 2019 another (new?) movement at that heritage bridge abutment is evident (Figures 6 and 7).

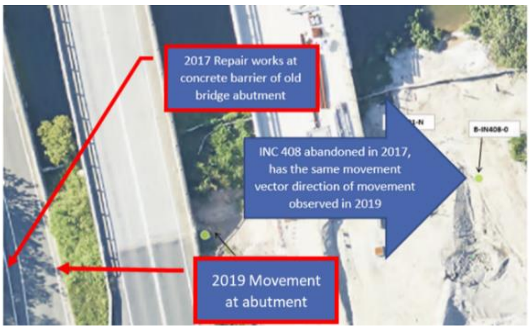


Figure 6. Repair works undertaken

Figure 7. 2019 Concrete barrier movement abutment of the heritage bridge

2.4. 1991 Existing Bridge

The adjacent 1991 bridge underwent relevelling in 2014 based on historical Google photos and again in 2020 at the southern abutment (Figure 7). These photos show the ongoing history of settlement at the adjacent south bound bridge abutment.

Thus all 3 bridges needed some repair at the southern abutment although the subject was the new bridge.

Figure 8. 2020 relevelling of 1991 adjacent bridge with 150mm drop in the relieving slab

3. Geotechnical data post movement

The CPTs and DMTs in 2019 was used to define local ground variations. Importantly the locations specifically targeted direct comparison of material properties close, between and far away from the temporary pile locations.

This comparative analysis was essential to inform causation. If collapse adjacent to the temporary pile locations was a potential causation, then a reduced strength (from collapsing soil) would be evident. The CPT measurements showed no loss of strength for proximity to piles.

The DMTs were carried out for locating shear zones (if present). This was its main and arguably only intent. All the 10 DMTs showed shear surfaces.

The CPTs and DMTs were compared and sorted into 3 categories:

1. Adjacent (< 2m from a temporary pile);
2. Between (2-5m from a temporary pile); and
3. Far > 5m from temporary pile.

3.1. CPTu data

The test data showed no loss of strength immediately adjacent to the temporary pile locations. Many of the CPTs showed sensitive clays at similar depths (Figure 9).

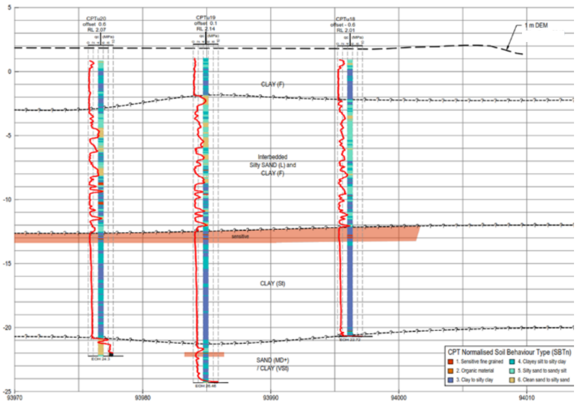


Figure 9. CPTU data showing 30m continuous sensitive clay layer near interface of layers

While the sensitive clays are predominantly at about RL-13, there were 2 anomalies at CPT 6 and CPT 12 which also showed a thin sensitive clay layer at RL-22. These two CPTs are close to each other and behind the bridge abutment.

The 2019 CPTs show alternating weak zones predominantly at 12m to 16m depth. This occurs both in the area of filling and even in areas remote from the filling and temporary bridge. Even in the fill areas there are alternating layers of weak and strong. A fill should not improve the ground in alternating layers.

CPT stratigraphy data in 2019 show the weak layers occur across the site even for CPTs 50m apart. The results show the upper layer soil strength near the temporary piles is nominally stronger than CPTs located at some distance away. This suggests that driving of the temporary piles created a volumetric expansion and soil strengthening in the near field.

An approximate equivalency of strength for all CPTs occurs at depth. The piles were in the ground for about 10 to 11 months before being removed and backfilled with gravel. These CPT findings are the opposite of the hypothesis that the ground was weakened in the area of the temporary piles. The intent of the CPTs were to compare soil properties associated to nearby and distant from pile locations. This data discounts any weakening of the ground at the pile locations.

3.2. DMT data

The DMTs were carried out as it is superior for locating shear zones (if present). An established method of assessing slip zones with the DMT is the $K_D \approx 2$ Method (Marchetti, 1997). The Normally consolidated (NC) clay bands, remolded by the sliding, then reconsolidated under the weight of the overlying soil, are recognized by using $K_D \approx 2$ as the identifier of the NC zones. As this site has been surcharged the clay is expected to have some light over-consolidation.

Figure 10 illustrates the methodology of identifying dormant failures zones. Knowing its ability to define shear zones, the use of the DMT was suggested in early site meetings following the observed movements. That was its main and arguably sole purpose.

All 10 DMTs showed shear zones at depth irrespective of proximity to the temporary piles (Figure 11). These results suggest dormant slip surface extends to the base of the sensitive clays (~RL-14) identified in the 2016 and the 2019 CPTs and the inclinometers.

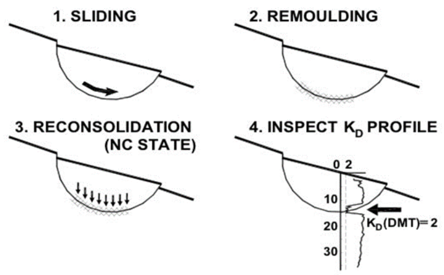


Figure 10. DMT Method of K_D ~ 2 representing failure zones (Marchetti, 1997).

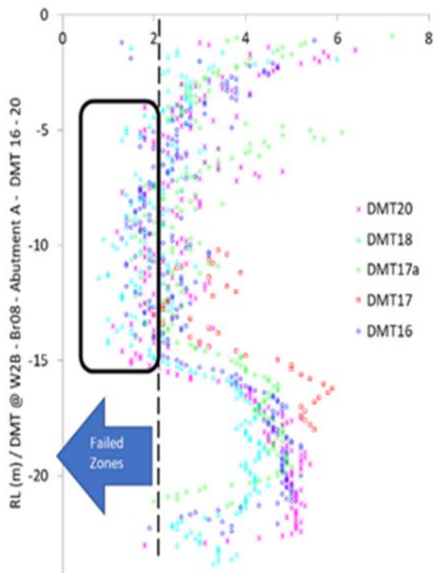


Figure 11. Low K_D values by DMT location.

If one accepts that a low K_D result is a failed zone, then by extension all the DMTs are in a failed zone whether the test is close or far from the abutment. Potential dormant slip planes occur even 30 – 35m away from the bridge abutment. The DMTS were also used to compare strength profiles strength profiles

1) adjacent to temporary pile locations
2) between those pile locations and
3) far away from the pile locations.

Table 1 summarizes the 10 - percentile, quartile, median and coefficient of variation for the “weak” upper and middle alluvium. These results 10 weeks after the temporary pile removal show that DMTs closer to the removed temporary piles measure a nominally stronger top clay and interbedded material. The piles were in place for approximately 1 year prior to removal and same day gravel backfilling. These adjacent tests show strength increases with proximity to the temporary pile locations and assumed due to driven pile installation.

The possibility that hole collapse has occurred during removal led to a *reductio ad absurdum* as adjacent soil strength to the removed pile has been shown to improve with proximity to the pile locations. For a collapse mechanism to be valid the CPT and DMTs MUST show a strength reduction with proximity to the pile location.

Table 1. Comparison of DMT strength adjacent to and far away from temporary piles

Layer	Adjacent to pile	Between piles	Far away from piles
Top clay - Cohesion (kPa) 10%/ 25%/ 50%	11/ 13/ 18	11/ 14/ 16	10/ 12/ 14
COV (%)	39%	29%	27%
Middle Interbedded silty sand and clay Friction angle (°) 10%/ 25%/ 50%	30/ 32/ 33	31/ 32/ 33	29/ 30/ 32
COV (%)	6%	5%	7%

3.3. Inclinometer data

The hold point for the preloading was released in September 2017. There was a mistaken belief that releasing the hold point meant complete abandonment of all monitoring. Removal of all embankment material and reloading at a later stage (December 2019) is not a standard practice. To install and then not monitor (albeit at reduced intensity) and then subsequent abandonment shows a lack of understanding of the intent of geotechnical instrumentation.

Up to the October 2017, the nearest inclinometer INC408 inclinometer moved 21.4mm on 20 September 2017. This magnitude (less than 25mm) was used as a part basis for the removal of the surcharge. Yet this inclinometer was incorrectly placed in a low movement zone at the side instead of in front of the embankment on the river side as originally planned.

Peak readings typically occur at or shortly after peak load has been placed. Filling stopped on 4 August 2017, yet lateral movement was continued up to the date of the last reading 9 weeks later. The inclinometer INC408 data abandoned in 2017 provides evidence of movement to the depth of the sensitive clay. In hindsight, it correctly 'predicts' the depth of the sensitive clay which has now been comprehensively recognised by recent CPTs and 10 DMTs in the area (Figure 12).

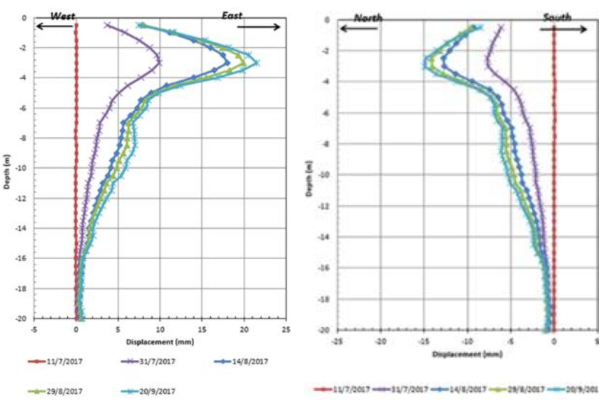


Figure 12. INC 408 which was used only in 2017

The new inclinometer installed after the observed movement showed little movement and suggested the movement had stabilized. During construction monitoring, a settlement of 234mm was measured at the hold point release based on the 3.15m height if fill. Note this does not mean the settlement was 100% complete. An assumed unit weight of 20 kN /m^3 was applied in this settlement monitoring.

3.4. INSAR data

Interferometric synthetic aperture radar (INSAR) data was obtained. The satellite passes every 2 weeks at this location and shows movement starting during the fill placement (Figure 13) and before removal of the temporary piles. This line-of-sight measurement is at the top of bridge deck and at road level while the surveyor measured lateral movement below the bridge deck. Thus, movement magnitude was not expected to coincide.

This data weas obtained 9 months after all other data were available. Note on 12 December, the surveyor had not reported any movement.

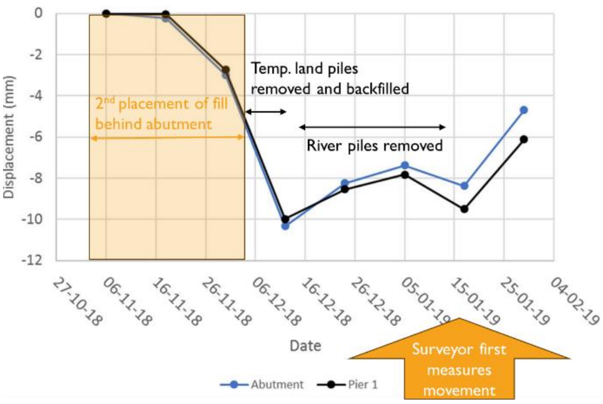


Figure 13. INSAR data at two points on bridge

3.5. Fill construction records

An assumed unit weight of 20 kN/m^3 for embankment was applied in the design. This is a typical value, but when imported fill from residual soils or weathered rock is used, the density is greater and can affect the settlement and stability analysis as described in Look (2021). During bridge construction, the initial preload fill was removed to have a level access area for construction of the bridge. The wet density of the placed fill was:

- 2.09 median (range 1.99 to 2.14 t / m^3 based on 51 results) for the initial fill which was then removed, and later replaced with a fill of
- 2.22 median (range 2.15 to 2.30 t / m^3) based on 15 results close to the bridge abutment.

Table 2 compares the measured field bulk density (γ), and the fill pressure with the historical pressures at the various periods of assessment. The assumed unit weight of 20 kN/m^3 applied initially for design was used throughout, even at the forensic stage and with available measured values during construction.

Both the settlement and stability would be affected by an assumed vs actual unit weight. The replaced fill has a ground pressure increase of 116.5% (73.4kPa / 63 kPa). This was therefore a first time loading due to differing density values. The 16.5% "extra" ground pressure matches the time of placement of the final 2 compacted layers. No settlement monitoring was undetaken during that placement, as settlement had been assumed to have been completed 2 years prior with the hold point release.

Table 2. In situ quality (density) tests for different sites

Stage	Measured γ, t/m^3	Fill pressure kPa
Design. Settlement & slope stability analysis	Unknown Assumed 2.00	3.15m x 20 kN / m^3 = 63 kPa
First filling to 3.15m and release of hold point during monitoring	2.08	69 kPa
Removal of fill for temporary bridge construction	N/A	4.8kPa
Replacement of 3.15m fill. No hold points. Assumed no further settlement	2.22	73.4 kPa
Forensic analysis by various parties	2.00	3.15m x 20 kN / m^3 = 63 kPa

The timing of these final layers being placed matched the start of movement as noted by INSAR satellite line of sight measurements analysis. The factor of safety reduced by 0.02 to 0.11 depending on the type of model analysis when the actual fill pressure is used.

This was therefore the first time the ground experienced that ground pressure. Yet the design, the hold point monitoring and forensic investigation by various experts used the assumed value of 20 kN / m^3 in various stability and finite element analyses.

4. Analytical models

4.1. Slope Stability

Both circular and non circular slope analyis was carried out for long term and short term conditons. The non circular would typically be 0.1 to 0.4 less than the circular analysis. The various slope slope stability modelling completed at the Abutment showed:

- Adopting the 'peak' strength parameters for all material units show a stable condition.
- Including the sensitive clay in the slope stability changes the Factor of Safety (FoS) from 1.47 to 1.08 for effective strength parameters.
- When erosion is included in the slope stability analysis, the results show that if 2.0 m or greater, of material was scoured from the riverbed then the design Abutment arrangement would present slope stability issues (FoS $\leq$ 1.0). Scour depths of 7m for the 100-year ARI at the river piers was predicted by the hydraulic engineers – and a significant event (1.1% tidal probability) occurred at the gap period.

4.2. Probability analysis

The 20 CPTs and 10 DMTs provided a significant local data at this abutment area, and which was not previously available. Figure 14 shows the spread of results and best fit and normal probability density functions (PDF) from 130 DMT results for the undrained cohesion between the temporary piles. A median cohesion of 17 kPa was used in the short-term analysis and the soil variability included for a probability analysis. These results and others for the interbedded layer were presented in Table 1

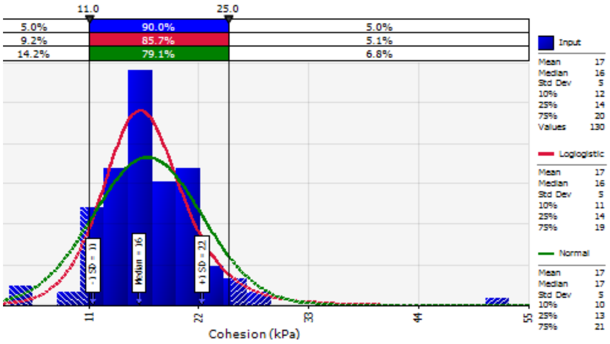
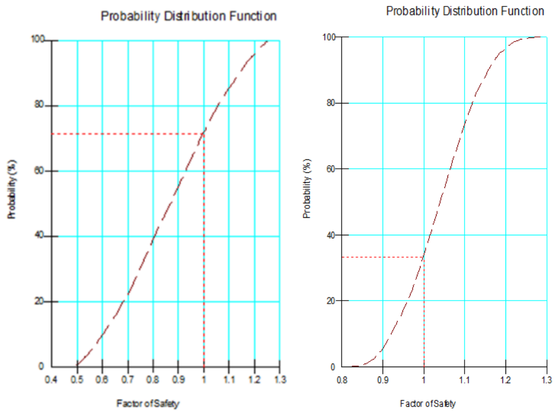


Figure 14. Undrained cohesion between piles

Figure 15 shows the probaility of failure for the case of a rapid drawdon and service trench in front of the embankment using this new and extensive soite data. There is a 33.3% and 71.4% probability of falure for the former and latter, respectively



(a) Rapid Drawdown (b) Trench

Figure 15. Probability of failure if (a) rapid drawdown or (b) trench is used in analysis

4.3. Numerical analysis

Chan et al. (2023) uses a Plaxis 3D analysis to model "pile hole collapse during the temporary pile removal".

"This was based on the hypothesis that when the closed-ended tubular piles supporting the temporary bridge were extracted from the ground using vibration, the extraction created a void at each pile location forming a temporary "vacuum". With the assistance of vibration, the soft clays and water charged sandy soils were greatly disturbed and caved into the void."

This model required:

- An unfilled hole to be assumed
- Created a "square" equivalent hole (Figure 16)
- Collapse aligned with the observed movement.

Given a round hole FEA model did not collapse a "square" equivalent hole with a forced side displacement aligned in the direction shown was used in the PLAXIS 3D model (Figure 16).

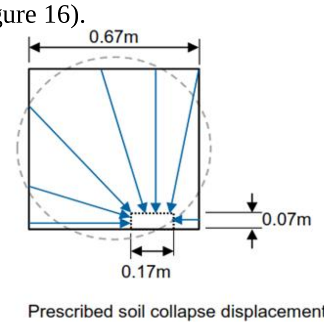


Figure 16. Temporary hole model for land piles

Figure 17 shows the reality of the gravel which was in the "unfilled" hole. Construction dockets show the material was delivered at the time of the temporary pile removal. The pile removal typically took 30 minutes, and the gravel was then placed. The photo records 2 months after, do not show any surface collapse of this gravel backfill material since placement.

Figure 17. Gravel back fill placed at OSP4B

The nearest temporary piles to the permanent bridge sill beam were 4.6m (2A), 5.0m (3A) and 12.1m (4A). The locations are shown in Figure 18. Although the rationale for the forced displacement was the adjacent fill, the 3.1 m fill at the temporary abutment was 8.8m to 35.2m distance from SP1 to SP 4 row of piles, respectively (Figure 19). Some (but not all) of that fill had already been removed prior to the pile removal.

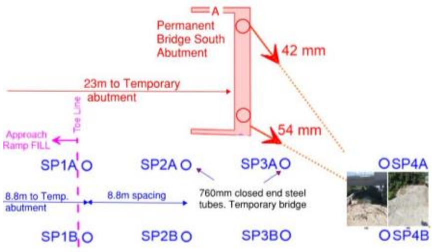


Figure 18. Permanent bridge (red) and temporary pile positions (blue) with movement vectors (vectors not to scale)

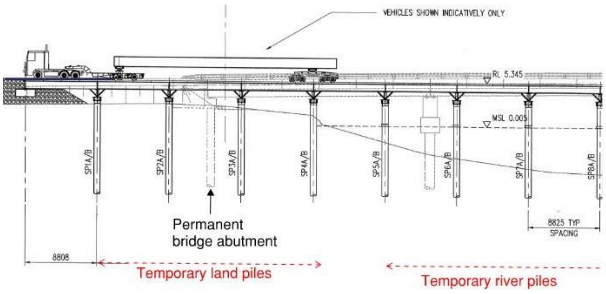


Figure 19. Temporary bridge piles

Chan et al. (2023) showed *"the 3D FEA was able to match the lateral soil movements and settlements at the survey benchmarks reasonably well.*

Since the 3D FEA of the temporary pile hole collapse mechanism was able to simulate the soil movement and pile displacement at various locations, it can be deduced that collapse of the pile holes during the extraction of the closed-ended temporary piles near the permanent bridge could be one of the key contributors to the observed movements."

The representative soil model for the analysis used the values shown in Table 3. This ground model does not match the 20 CPTs and 10 DMTs at this abutment area as part of the forensic investigation. There was no 9.0m sand layer and the lower 10 – percentile value from those tests are all above 29˚ friction angle (refer Table 1). Effective stress parameters are used despite "collapse" occurring during temporary pile removal.

The top 4 - 5m clay layer (Figure 9) with a median undrained cohesion strength of 14 kPa and 18 kPa far from and adjacent to temporary pile location (Table 1) does not form part of the model.

Table 3. Representative soil model used in FEA

Layer	Young's Modulus (MPa)	Cohesion (kPa)	Friction Angle (˚)
Top 1.5m – Desiccated surface	15	5	30
Middle Holocene – 3.5m Interbedded sand / clay	4	0	28
9.0m sand	7	0	28
3.0 m clay	4	0	28

5. Discussion on FEA and measurements

The FEA was verified as the predicted movements match the movements at the bridge abutment. However, given the calibration by induced movement and in direction of the movement, can the PLAXIS 3 D model be used as "predicting" collapse?

The terms verification and validation are often used interchangeably. Validation is the process to ensure that the model is representing the real world as much as possible to be considered an accurate model. Lees (2016) discuss validation of FEA for accuracy by plausibility checks for input, assessing accuracy of outputs, and key elements of the observational method. At this site, the model accuracy should address the following 3 sets of considerations.

5.1. Input requirements

a) A void from the extracted pile, when site data and records show gravel backfill occurred on the same day. How does a 100%-hole collapse account for the evidence of the placed gravel
b) A "calibration" of direction of movement to align with measured. Can this be then used as a "prediction"?
c) A square hole was modelled when a circular hole model could not fail

d) Soil parameters do use the data from the adjacent and targeted 20 CPTs and 10 DMTs
e) The DMT and CPT data adjacent to the removed piles show an increase in strength (attributed to pile installation)
f) "Collapse" contradicts the DMT and CPT data that showed no loss of strength adjacent to the pile "void."

5.2. Site test data considered not relevant to the FEA included:

a) The sensitive clays were observed in several CPTs but was not present in all the tests. One cannot say sensitive clay exits everywhere.
b) All 10 DMTs showed shear zones. This K_d approach was called an interpretation.
c) The inclinometer depth (both abandoned and new) matching the shear zone and depth of sensitive clay identified by the DMT and CPT, respectively, was not considered sufficient evidence.
d) The unit weight difference for the backfill material assumed and measured was not considered significant, although the surcharge fill construction now varied from design
e) The INSAR data shows movement starting before the temporary pile removal and during abutment fill loading was not considered relevant matching. That data was above the bridge deck while the critical movement was below the deck
f) Low FoS based on toe erosion. The 1.1% probability event did not prove bed erosion occurred as this is transient and an unrealistic mechanism. Both the historical and measured deepening of the riverbed was considered not relevant although design allows for 7m scour
g) A service line trench between the abutment and river was signed off 1 month before the observed movement. But the date and details of installation could not be verified.
h) The 2 adjacent bridges undergoing repairs prior and after this incident was "out of scope" for this study as this involved other parties
i) These were "missing" wicks based on aerial photos and drawings, but site personnel stated these wicks had been placed below the abutment.

5.3. Output aspects

a) Outputs from FEA show movement increases further away from the temporary piles.
b) The model shows predicted movement of 1.0m at the riverbank and 0.5m at the old bridge abutment. This is over 30m away. No such movements have been measured.

6. Conclusion

Legal case studies such as this is seldom reported. However, a 2023 paper suggesting the benefits of a 3D FEA was written on this case study. That paper suggested that the 3D FEA model adequately demonstrated the potential contribution of collapse from removal of temporary piles. Such discussion in the public domain, allows alternative potential factors to now also be presented. Albert Einstein said, "If I were to remain silent, I'd be guilty of complicity." Many engineers from varying companies agreed to the collapse cause. Therefore, it is highly recommended to read that 2023 paper in conjunction with the site background in this paper for a balanced viewpoint.

That FEA analysis occurred after field tests specifically targeted to demonstrate a void failed to show adjacent weakness to this piles. Instead, the tests showed findings of shear zones and sensitive clays at depth which was not previously known. Such data was considered a point of view. Assumed conditions are required for input which contradicts site observations and measurements. Calibration to movements and then finding the model correctly predicts movement is a circular argument.

Alternative analysis based on sensitive clays and shear zone show low factors of safety. When bed erosion is considered, factors of safety are below 1.0. When high tidal events or trenching in front of the embankment is analyzed a high probability of failure is predicted, using the test data variability from the CPTs and DMTs.

Even when shown movements are occurring on the 2 adjacent bridges, such observations are considered out of scope for this bridge defect assessment. The INSAR data by a consultant who did not know specific dates and other details was discounted. That INSAR analysis show movement starting before the temporary pile removal. The INSAR movement coincides with the second back filling behind the permanent bridge abutment with a heavier unit weight material than previously placed. A significant tidal event which could cause bed erosion also occurred after the first observed INSAR movement.

The FEA study was to confirm a possibility. Correlation is not causation. Yet that correlation was the basis of a legal claim. This paper provides a background on the site data observations and numerical analysis. The latter was given the same credence as the site observations and measurements with that data considered a point of view.

References

Chan K, Poon B and Yip G (2023). Potential factors causing the movement of a newly constructed bridge. *Proceedings of the 14th Australia and New Zealand Conference on Geomechanics*, Cairns (ANZ2023).

Ghobadian R. and Shafai M. (2007). Investigation of Sediment Patterns at River Confluence. *Journal of Applied Sciences*, 7: pp 1372-1380.

Lees, A. (2016). Geotechnical finite element analysis: a practical guide. *ICE publishing*, London

Look B (1999). The Probability of Failure of the Brisbane Riverbank at St Lucia. *2nd International Conference on Landslips, Slope Stability and the Safety of Infrastructures*, Singapore, pp 209-216.

Look, B. (2021). Compaction density of Residual Soils and Weathered Rock. *Australian Geomechanics Journal*, Vol 56, No.1, pp 61 – 75.

Marchetti S. (1997), "The Flat Dilatometer: Design Applications" *Proceedings of the 3rd International Geotechnical Engineering Conference*, Cairo, pp 421 – 448.