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Geotechnical challenges and design optimisation for embankments on compressible foundations: A case study of the Hexham Straight Widening project

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ABSTRACT: The construction of embankments on compressible foundation materials presents significant challenges in controlling post-construction settlement. This paper discusses the geotechnical challenges encountered during the design and construction of Ironbark Creek bridge approach embankments for the Hexham Straight Widening project, where ground treatment strategies were refined to meet performance requirements and manage construction constraints. The site lies on a low-lying floodplain underlain by Tertiary and Quaternary sediments of the Hunter River Valley. These reflect a complex depositional history involving erosion, channelisation, sea-level fluctuations, and a meandering riverbed channel. A key challenge was to develop cost-efficient ground treatments that would satisfy stringent pavement performance criteria while improving whole-of-life project costs. The Concept Design proposed retaining walls and concrete-injected columns (CIC) to support critical-path bridge piling. In later design stages, the embankments supported by CICs were refined with preloading, surcharge, and prefabricated vertical drains (PVDs) to accelerate consolidation. Construction staging was adjusted to optimise geotechnical outcomes, including longer preload durations. Traffic was diverted onto the new bridge for at least one year during construction. This enabled the existing bridge to be replaced and allowed for an extended preloading. Upon completion of the bridge replacement, pavement corrections and the final wearing course were constructed. The extended preload period helped reduce settlement risks. Settlement observed during preloading was compared with predictions, and site monitoring data were used to calibrate design parameters. These were revised to reflect observed settlement behaviour. Assessment of the degree of consolidation using the Asaoka method were consistent with theoretical expectations. The calibrated PLAXIS model confirmed that Total Residual Settlement met project requirements. This paper presents a detailed geological interpretation based on Leapfrog modelling. It explores the interpretation of the transitional soils unit, the iterative design process informed by monitoring and back-analysis, and broader implications for geotechnical performance and constructability. The findings highlight the importance of data-driven refinement and collaborative geotechnical in complex floodplain environments.

KEYWORDS: consolidation settlement, prefabricated vertical drains, preloading, surcharge, design optimisation, settlement monitoring, back-analysis.

1 INTRODUCTION

The Hexham Straight Widening (HSW) project, located in the Hunter River delta of New South Wales, involves the upgrade of a six-kilometre section of the Pacific Highway (Maitland Road) through Hexham, expanding it from four to six lanes. This corridor forms part of the National Land Transport Network and is critical for freight and commuter movement. Key works include construction of a new southbound bridge and replacement of the 1962 twin bridges with a new northbound structure.

A major challenge in this project was the construction of approach embankments over soft, compressible soils within a low-lying floodplain. These ground conditions imposed strict limitations on embankment stability and post-construction settlement, especially under tight construction timeframes. At Ironbark Creek, preload and surcharge durations were limited to two months in critical transition and abutment zones, due to the bridge works being on the project's critical path.

To address these challenges, the design focused on delivering effective ground treatment within the available timeframe while meeting stringent performance criteria. Geotechnical objectives included controlling total and differential settlement, maintaining embankment stability, and achieving pavement performance targets over a 40-year design life.

This paper outlines the strategies adopted to manage settlement risk and optimise geotechnical outcomes under programme constraints.

2 HISTORICAL BRIDGE DEVELOPMENT AND SITE ACTIVITIES

The Ironbark Creek crossing has experienced multiple bridge alignments and modifications since the late 19th century, each leaving structural and geotechnical legacies. The first formal structure, a timber truss bridge built in 1875, operated for 63 years before being replaced due to structural limitations and increasing traffic. Its northern approach closely matches the alignment of the proposed southbound bridge, suggesting the possible presence of remnant foundations or disturbed ground.

In 1938, a concrete-steel bridge was constructed west of the original alignment to remove a sharp northern bend. This bridge failed during the 1955 Hunter Valley floods, with its central span collapsing in 1956 (Figure 1). A temporary prefabricated Bailey bridge was installed later that year. At the same time, a new five-span timber bridge was built along the original alignment, this time with the sharp bend removed. Construction during this phase introduced fill materials likely still present in the subsurface profile.

A permanent five-span steel girder and reinforced concrete bridge was constructed in 1962. Aerial imagery reveals remnants of earlier structures, including the 1965 temporary bridge and timber bridge components, which are likely still buried at the site. Figure 2 shows the aerial view highlighting these features.

3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

The soil and geology along the alignment are the result of complex erosional and depositional processes associated with a buried paleo-valley system, likely a former tributary of the Hunter River. Ironbark Creek is the dominant geomorphological influence, with recent sedimentation also affected by estuarine and deltaic activity.

The deepest sediment deposits are found near Ironbark Creek. The rock-soil interface dips to the northwest at about 35° to horizontal and is interpreted as a stepped profile consisting of horizontal rock platforms and vertical joints, rather than a continuous slope. Tertiary to Quaternary sediments unconformably overlie weathered Permian bedrock, with a thick residual soil layer present across much of the site.

Geotechnical boreholes show highly variable bedrock depths, ranging from 1.1 m to 42.1 m below the ground level. Near the northern abutment, bedrock was not encountered to depths of 55 m, suggesting a deep incised paleo-valley likely representing a relict channel of the Hunter River. The thickness of the soil in this area is greater than elsewhere due to intense erosion and deposition.

Figure 1. Ironbark Creek Bridge collapsed during 1956 (State Library of NSW)

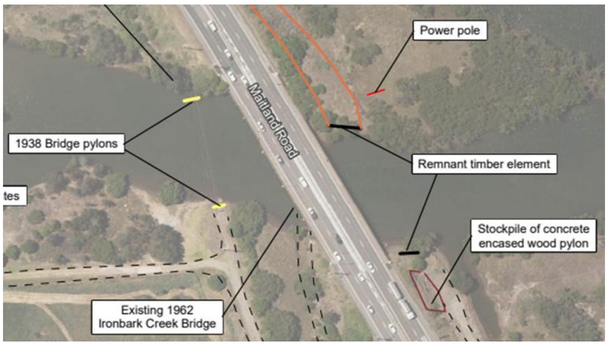

Figure 2. Aerial image of the Ironbark Creek Bridge

Table 1. Geological units

Unit	Description
Unit 1 - Fill	Fill comprising slag, ash, gravel, and other uncontrolled fill from historical road construction. Highly variable including granular and cohesive layers, poorly compacted in places.
Unit 2 - Upper Sediments	Very soft to soft silt and clay within Ironbark Creek riverbed.
Unit 3 - Transitional Estuarine Soils	3.1A: Soft to firm sandy clay/silt, low-medium plasticity. 3.1B: Very loose to loose clayey sand/sandy silt. 3.2A: Stiff or better sandy clay or sandy silt. Low to medium plasticity. Typically, over-consolidated. 3.2B: Medium dense to dense clayey sand/silty sand. Fine to medium grained sand.
Unit 4 - Coarse Grained Estuarine Soils	4.1: Very loose to loose sand, clayey sand and silty sand with shells. Fine to medium grained. 4.2: Medium dense to dense sand. Fine to medium grained sand. Trace of shells.
Unit 5 - Fine Grained Estuarine Soils	5.1: Soft to firm clay with shells and silt. High plasticity. 5.2: Stiff or better clay with shells and silt. High plasticity.
Unit 6 - Residual Clay	Stiff to hard residual clay. Extremely weathered in parts.

3.1 *Soil stratigraphy*

The subsurface stratigraphy is summarised in Table 1. The assigned geotechnical units are based on interpretation of field investigation data, laboratory testing and 3D geological modelling using Leapfrog (Figure 3). The 3D model helped the design team understand the distribution of soils and to visualise the geology and ground conditions across the project area. The model integrates historical map-based data with current and new geotechnical investigation results, pre-existing topography, and site bathymetry. Zonation of units has been developed based on the depositional history, geological setting, and site development history.

Subsurface conditions at the southern approach embankment comprised Fill (Unit 1) up to 1 m deep, overlying soft estuarine soils (Unit 3.1A) to depths of around 10 m, underlain by Residual soil (Unit 6). At the northern approach embankment, the ground profile consisted of a surficial fill layer overlying interbedded transitional estuarine soils (Unit 3), including interbedded soft to firm sandy clay and silt materials up to depths of around 10 m. These were underlain by coarse-grained estuarine soils (Unit 4) to depths of approximately 25 m, which were interbedded with and/or overlying typically stiff estuarine soils (Unit 5) extending to depths of up to 50 m, underlain by Residual soil (Unit 6). Unit 3.2A and 3.2B was typically present in the upper soil profile of the northern approach embankment and were interbedded with Unit 3.1A and 3.1B.

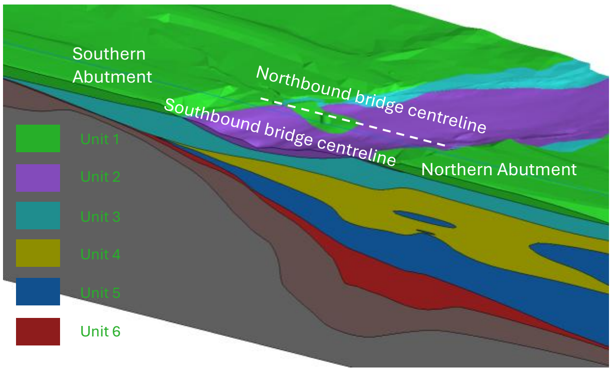

Figure 3. Leapfrog long section through the Northbound bridge, showing a visualisation of the geological units in the area of Ironbark Creek Bridge.

3.2 *Estuarine soils (Unit 3)*

The estuarine soils, which is critical to the ground treatment design, were divided into sub-soil units using borehole logs, laboratory tests (particle size distribution and Atterberg limits), in situ strength tests (SPT and CPT), and assessment of material behaviour using empirical and theoretical methods. Soil types at each CPT location were interpreted with depth using the Soil Behaviour Type (SBT) Index developed by Robertson and Wride (1998), as shown in Figure 4. The SBT index, I_c, was calculated as:

$$I_c = ((3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2)^{0.5} \tag{1}$$

where Q_t is the normalised cone resistance and F_r is the normalised friction ratio.

Figure 4 illustrates that, within the upper 10 m, the distribution of clayey silt to silty clay (pink) and silty sand to sandy silt (yellow) differs notably between the Northern and Southern Abutments. These differences reflect variation in estuarine deposits. Note that rock was encountered at approx. 10 m at the southern abutment. Soil behaviour type classifications based on Robertson (2010) are shown in

Figures 5 and 6 for each location, using the normalised SBT chart. The detailed calculations of normalised cone resistance and normalised friction ratio are provided in Robertson (2010).

3.3 *Northern Abutment*

At the Northern Abutment, the SBT Index in the upper 10 m generally ranges between 2.2 and 3.0, indicating transitions among silty sand to sandy silt, clayey silt to silty clay, and occasional zones approaching clayey soils. The SBT Index curve exhibits strong vertical variability, highlighting a heterogeneous profile with frequent shifts in soil behaviour. Thin, recurring layers of clayey silt to silty clay (pink) are interbedded with thicker silty sand to sandy silt (yellow), suggesting alternating low and moderate-permeability materials. Although silty sand to sandy silt dominates, the presence of thin clayey layers indicates reduced permeability and variable compressibility, both critical for settlement and drainage assessment.

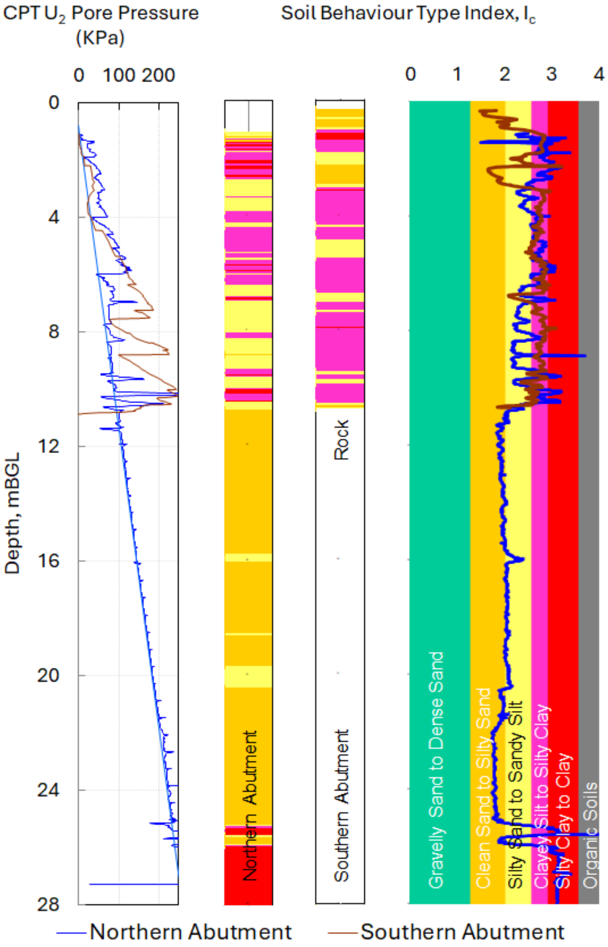

Figure 4. Cone penetration pore pressure and soil behaviour type classification for North and South Abutments.

3.4 *Southern Abutment*

In contrast, the Southern Abutment shows a more uniform distribution. Clayey silt to silty clay occurs as a thicker, more continuous layer, mainly between 3 m and 7 m depth. Silty sand to sandy silt layers are less frequent and thinner, generally restricted to shallow depths. This suggests a finer grained, more homogeneous profile in the upper 10 m, with reduced permeability and increased compressibility compared to the Northern Abutment. These variations are significant for ground improvement design and settlement control.

3.5 *Comparison with pore pressure and soil behaviour data*

Figures 4 show CPT pore pressure (U_2) measurements and corresponding SBT classifications. While these classifications generally align with observed stratigraphy, some discrepancies were observed between CPT interpretations, laboratory particle size results, and borehole logs, even at closely spaced locations. These inconsistencies were most pronounced in the upper estuarine soils at the Northern Abutment.

For example, elevated CPT U_2 pore pressures indicated low-permeability materials, consistent with fine-grained soils. The SBT method, however, classified these zones as sand-silt mixtures. Meanwhile, laboratory tests identified them as clayey silty sand, and borehole logs described them as sand. These differences highlight the limitations of relying on a single method and the importance of integrating multiple data sources for accurate interpretation.

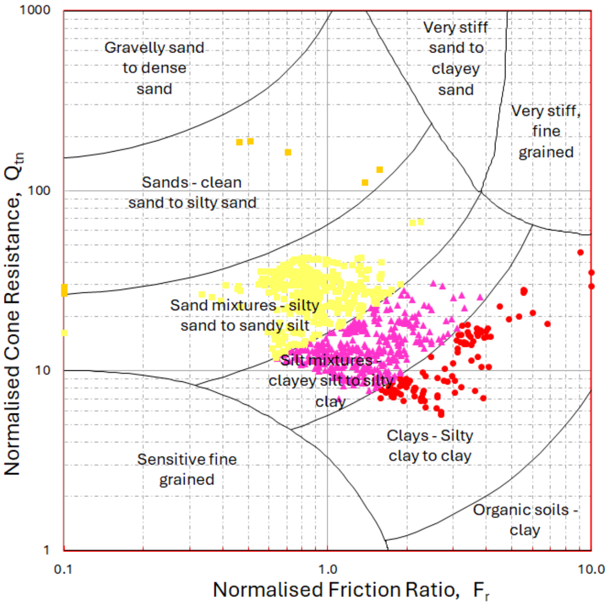

Figure 5. Classification of Estuarine soils at Northern Abutment.

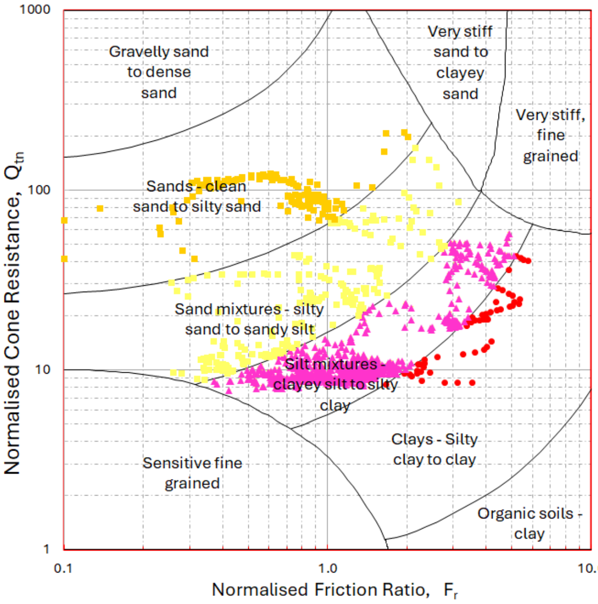

Figure 6. Classification of Estuarine soils at Southern Abutment.

3.6 *Schneider (2012) interpretation*

CPT data were further evaluated using the soil behaviour classification framework by Schneider (2012). The detailed calculations of normalised net cone resistance, *Q* and CPT friction ratio, *F (%)* are provided in Schneider (2012). Figure 7 shows that most estuarine soils at the Northern Abutment fall within Zone 3, with local occurrences in Zones 1b and 2. Zone 3 corresponds to transitional soils, interbedded clay, silt, and sand, consistent with the observed heterogeneity. The SBT index, range mostly from 2.0 to 3.0, matching silt mixtures and clayey materials, with occasional sand-like behaviour. Pore pressures are moderate and relatively stable with depth (Figure 4), indicating better drainage characteristics and lower soil sensitivity. These traits suggest a stiffer, more heterogeneous soil profile with improved consolidation performance.

In contrast, the Southern Abutment shows more scattered CPT data extending into Zone 1b, which includes sensitive and organic clays. The I_c values exceed 3.6 in many zones, consistent with soft organic or sensitive clays. The U_2 pore pressures are significantly elevated, often above 150 kPa, and persist over a greater depth range, indicating poor drainage, higher compressibility, and increased susceptibility to undrained behaviour.

Overall, the Southern Abutment is underlain by softer, more compressible, and more sensitive estuarine soils, requiring PVD ground improvement and longer preload durations. The Northern Abutment, while still comprising transitional soils, features stiffer and better-drained materials, offering comparatively more favourable conditions for settlement control and foundation stability.

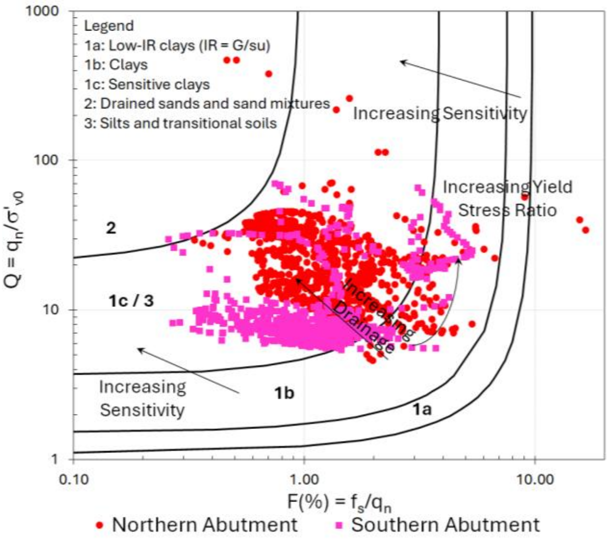

Figure 7. Schneider (2012) plot for the North and South Abutments.

3.7 *Shear strength and over-consolidation ratio*

A CPT cone factor (N_{kt}) of 14 was adopted for the project by calibrating the CPTu-derived shear strength profile against corrected field vane shear test results. This value closely aligns with the N_{kt} of 13.7 reported by Jones (1995) in their assessment of alluvial soils in Newcastle. However, site-specific vane shear results were preferred over CPT-interpreted undrained shear strengths where available, due to the observed variability of the N_{kt} factor.

Figure 8 presents the interpreted undrained shear strength, over-consolidation ratio (OCR), and pre-consolidation pressure from CPT data for the North and South Abutments, with back-analysed values shown as dashed lines.

The Northern Abutment shows higher undrained shear strength, particularly at shallow depths. In contrast, the Southern Abutment exhibits lower shear strength, with a marked reduction at greater depths. The OCR profile at the Northern Abutment is also higher, indicating over-consolidation and a history of greater loading. At the Southern Abutment, the OCR generally decreases with depth, ranging from 4 to 5 in the upper 3 m, reducing to around 2 at mid-depths, and reaching values as low as 1.3 (near normally consolidated) at deeper levels.

Figure 8 also includes the coefficient of consolidation from dissipation tests, alongside the soil behaviour type classification for both abutments. The Southern Abutment shows lower coefficients of consolidation, especially at shallow depths. Below 3 m, values from both abutments are of similar magnitude. However, the back-analysed coefficient of consolidation at the Northern Abutment is more than double that of the Southern Abutment and significantly higher than the dissipation test results.

This discrepancy may be partly attributed to sampling bias during dissipation testing at the Northern Abutment. The SBT plots show that tests were mostly conducted within clayey silt and silty clay layers, which likely produced results similar to those at the Southern Abutment, where these materials form a thicker, more continuous layer between 3 m and 7 m depth.

Overall, the Northern Abutment exhibits better geotechnical stability due to higher shear strength and OCR, while the Southern Abutment may require additional reinforcement and surcharging to reduce post-construction creep settlement.

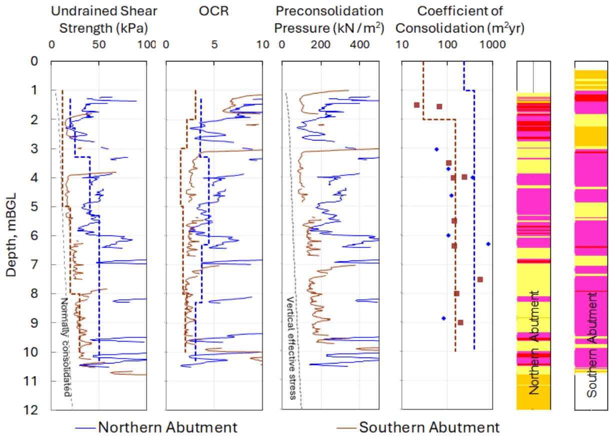

Figure 8. Shear strength, over-consolidation ratio (OCR), pre-consolidation pressure and coefficient of consolidation interpreted from CPT data for the North and South Abutments, with the back-analysed values shown as dashed lines.

3.8 *Compressibility and creep behaviour*

The modified compression ratio (CR) and recompression ratio (CRR) were interpreted from oedometer test results using the Casagrande (1936) reconstruction method. Sample disturbance was evaluated using the Terzaghi *et al.* (1996) and Lunne *et al.* (1997) methods, and poor-quality samples were excluded from the data set. No significant variance in the CR and CRR was observed across different estuarine soils near Ironbark Creek.

Modified secondary compression index (Cαe) values were derived based on empirical correlations and extended oedometer tests. The extended oedometers indicated a lower ratio of Cαe /CR, typically around 1%. Accordingly, the lower end of the empirical correlation range was adopted.

4 DESIGN CHALLENGE

The main design challenge was limiting post-construction settlement to meet strict pavement criteria within a tight construction schedule and limited preload and surcharge durations. Settlement limits included a maximum differential settlement of 0.5% for flexible pavements and residual settlement under 50 mm within 20 m of bridge abutments.

The short time limited the degree of primary consolidation which could occur before pile installation and pavement works, such that the design had to account for residual settlement and lateral soil movement while meeting settlement performance targets.

5 GROUND IMPROVEMENT OPTIMISATION STRATEGIES

5.1 *Initial design*

The Concept Design used retaining walls and concrete-injected columns (CICs) to support piles and reduce settlement. However, meeting the strict settlement criteria (0.5% differential and 50 to 200 mm residual) while maintaining cost efficiency was challenging. This led to several design iterations with a focus on optimising whole-of-life costs.

5.2 *Alternative ground treatment strategies*

A location-specific ground improvement strategy was developed for the northern and southern abutments to account for the variability in ground conditions and the anticipated settlement behaviour at each location. The treatment approach, particularly the preload and surcharge durations, was formulated in close collaboration with the construction team to align with the sequencing of earthworks, temporary works, and piling activities at each abutment.

The design also capitalised on the one-year traffic switch period, during which traffic was diverted to the new southbound bridge to enable demolition and reconstruction of the existing northbound bridge. Final pavement surfacing was deferred until completion of the new northbound structure, allowing an extended preload period without impacting the overall construction programme. This approach reduced the risk of long-term settlement and enabled design refinement through ongoing monitoring.

5.2.1 *Southern Abutment*

Prefabricated vertical drains (PVDs) combined with surcharge loading were implemented at the southern abutment due to the presence of a softer and more compressible soil profile compared to the northern abutment. PVDs were installed at 1.2 m spacing in a triangular grid to accelerate consolidation and enhance overall stability.

Smear effects were incorporated into the design, following the methods of Indraratna and Redana (1998). A smear zone diameter to drain diameter ratio of 5 was adopted, and the smear zone permeability was assumed to be one-fifth that of the surrounding undisturbed soil.

Variable surcharge heights were applied across the abutment and transition zones, with a maximum height of 1.7 m. Surcharge durations of two months at the abutment zone and six months at the transition zone were adopted. PVDs were not installed in the transition zone, as the extended preload period and reduced thickness of compressible soils were considered sufficient to achieve the required settlement performance.

5.2.2 *Northern Abutment*

A preload and surcharge strategy was adopted for the northern abutment. A 1.5 m surcharge was applied for four months in the abutment zone, while a six-month preload period was applied in the transition zone. PVDs were not considered necessary given the relatively stiffer ground conditions and longer preload durations.

6 PRELOAD RELEASE ASSESSMENT

Monitoring data were back-analysed to validate settlement predictions and refine key design parameters. The degree of consolidation achieved was assessed against design targets, and calibrated PLAXIS models were used to verify compliance with residual settlement criteria.

Ground settlement observed during the preload and surcharge period was reviewed and compared with the predicted settlement response. Settlement plate data were assessed using the Asaoka Method (1978) to estimate the total primary settlement and degree of consolidation. In parallel, back-analysis was conducted in PLAXIS 2D to adjust the soft soil consolidation parameters to better reflect the monitored settlement magnitudes and rates. These back-analysed parameters were subsequently used to revise the predicted total residual settlement over a 40-year period, to demonstrate compliance with the project brief.

Parameter calibration was guided by both site investigation results and instrumentation data. Key modifications included adjusting the over-consolidation ratio (OCR) to match observed settlement trends from settlement plates and extensometers; modifying the drained shear strength, stiffness, and compressibility parameters of the soil units to align with measured settlement magnitudes; and calibrating the coefficient of consolidation (c_v) and hydraulic conductivity parameters to reflect the observed rate of consolidation.

The final back-analysed parameters adopted for the southern and northern abutments are presented in Tables 2 and 3, respectively. The OCR was calculated using the SHANSEP relationship, with design values of S = 0.22 and m = 0.95 adopted for the analysis. Compressibility parameters adopted during detailed design also provided for information.

The back-analysed time-settlement curves, derived from monitored settlement data, are presented in Figures 9 and 10. These figures also include settlement recorded during the construction of the new northbound bridge, when traffic was temporarily diverted onto the completed southbound bridge for a one-year period. This diversion enabled an additional year of preload at the design embankment height prior to pavement correction and final surfacing. The extended preload duration further reduced the risk of post-construction settlement. Ongoing instrumentation monitoring during this period confirmed performance compliance.

Table 2: Back analysed parameters (Southern Abutment)

Depth (m)	Unit	OCR	CR	CRR	Cαe	c_v (m^2/yr)
0-1	1	-	-	-	-	-
1-2	3.1A	3.10	0.193	0.035	0.006	15
2-3	3.1A	2.19	0.193	0.035	0.006	15
3-5	3.1A	1.52	0.193	0.035	0.006	75
5-8	3.1A	1.76	0.193	0.035	0.006	200
8-8.5	3.1A	2.17	0.193	0.035	0.006	200
8.5-9.5	3.1A	2.00	0.193	0.035	0.006	200

Original Design Values for 3.1A: CR = 0.17, CRR = 0.030

Table 3: Back analysed parameters (Northern Abutment)

Depth (m)	Unit	OCR	CR	CRR	Cαe	c_v (m^2/yr)
0-1.3	1	-	-	-	-	-
1.3-2.3	3.1A	3.60	0.193	0.035	0.006	120
2.3-3.3	3.1B	3.57	0.130	0.012	0.004	200
3.3-5.3	3.1B	4.41	0.130	0.012	0.004	200
5.3-6.3	3.1B	4.46	0.130	0.012	0.004	200
6.3-8.3	3.1B	3.71	0.130	0.012	0.004	200
8.3-10.3	3.1B	3.03	0.130	0.012	0.004	200

Original Design Values for 3.1B: CR = 0.12, CRR = 0.010

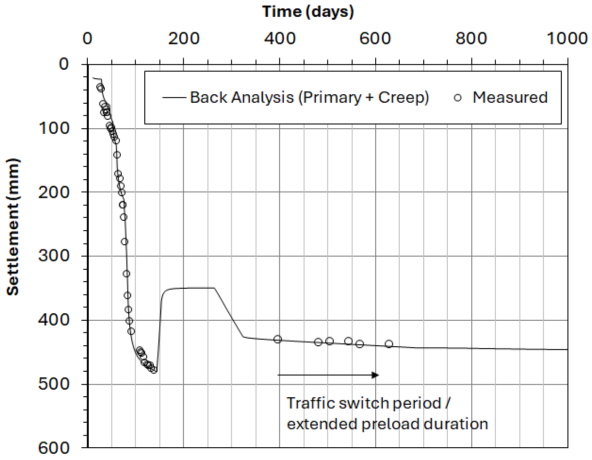

Figure 9. Back analysed time settlement curve vs. settlement plate monitoring data for the Southern Abutment

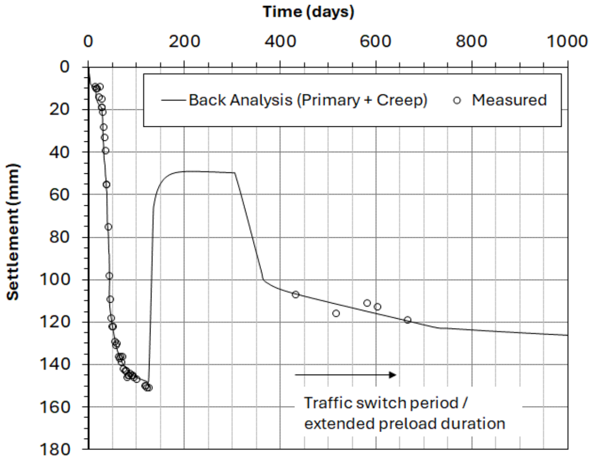

Figure 10. Back analysed time settlement curve vs. settlement plate monitoring data for the North Abutment

7 ENVIRONMENT AND SUSTAINABILITY

The ground treatment was designed to reduce the construction footprint and limit impacts on nearby sensitive receivers, including waterways and habitats. Switching from concrete-injected columns to preload and surcharge with PVDs reduced concrete use, lowering carbon dioxide emissions associated with cement production. This change also eliminated chemical leaching risks from concrete into soil and groundwater and preserved natural groundwater flow. The adopted method offered a more sustainable and environmentally responsible ground treatment solution.

8 CONCLUSION

The Hexham Straight Widening project demonstrated effective management of settlement and stability in compressible floodplain soils through a combination of preloading, surcharging, prefabricated vertical drains (PVDs), and geosynthetic reinforcement. A key factor in the success of the ground treatment strategy was the integration of geotechnical design with the construction programme. Close collaboration with the construction team enabled ongoing design refinement during delivery, aligning treatment measures with evolving ground conditions and programme constraints.

Recognising that the Southern Abutment lay on the project's critical path, while the Northern Abutment had greater programme flexibility, informed a targeted approach. PVDs were installed at the Southern Abutment to accelerate consolidation and allow surcharging to be completed before bridge works commenced. In contrast, PVDs were omitted at the Northern Abutment without compromising performance, as preload durations could be extended to suit the construction schedule. The one-year traffic diversion onto the new southbound bridge enabled replacement of the existing bridge while maintaining preload at the northbound embankment, reducing long-term settlement risk.

This project also highlights that dissipation tests may not fully capture the variation in consolidation behaviour due to sampling bias. At the Northern Abutment, most dissipation tests were conducted within clayey silt and silty clay layers, resulting in coefficients of consolidation similar to those at the Southern Abutment. However, back-analysed values were significantly higher, indicating that the dissipation test results underestimated the true variability. This underscores the need to interpret dissipation data alongside soil behaviour type to obtain more representative consolidation parameters.

9 ACKNOWLEDGEMENTS

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