Optimisation of bridge approach treatment via staged construction

Optimisation du traitement de remblais d'accès à des ponts par phasage des travaux

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ABSTRACT: This paper describes an improved approach to bridge embankment transition design and construction staging that was utilised to overcome financial and programme challenges associated with the proposed initial design solution for bridge approach embankments. An alternative staged approach was developed for construction, comprising improvement of the strength and compressibility characteristics of the soft soil foundation by surcharging techniques in combination with use of prefabricated vertical drains (PVD) and high strength geotextile. Unreinforced continuous flight auger (CFA) columns were installed after surcharging to achieve smooth transition at bridge approach embankments. During construction, the behaviour of the foundation under load was closely monitored and back analysis of the performance of the improved foundation was undertaken. Construction stage design optimisations were then made to satisfy the design criteria using actual monitoring data. This approach to bridge embankment transition design provided ability for the entire subsurface profile to accommodate the applied embankment loading. As a result, major cost, programme and environmental benefits were realised during construction by avoiding the installation of approximately 88,900 lineal metres of concrete foundation piles that were specified in the initial design.

RÉSUMÉ : Cet article décrit une approche améliorée pour la conception et le phasage des travaux de remblais d'accès aux ponts. Cette approche a été utilisée pour répondre aux contraintes financières et de planning associées à la solution initiale proposée. Une approche alternative en termes de phasage des travaux a été développée et comprenait l'amélioration de la résistance et de la compressibilité du sol de fondation (argile molle) par l'installation de remblais de chargement, de drains verticaux préfabriqués et de géotextiles haute performance. Des colonnes en béton ont été installées après la période de chargement pour assurer une transition en douceur au niveau des remblais d'accès au pont. En phase construction, le comportement du sol de fondation sous la charge était étroitement contrôlé et une évaluation de la performance en ce qui concerne l'amélioration actuelle du sol de fondation a été réalisée. Sur la base des mesures effectuées sur chantier, certains paramètres de conception ont été optimisés en phase travaux pour satisfaire aux exigences du projet. Cette méthode de conception des remblais d'accès aux ponts a fourni à l'ensemble du sous-sol la capacité de supporter le chargement qui s'applique sur le remblai. Ainsi, de conséquents gains financiers, de temps et environnementaux ont été réalisés en phase travaux puisque cette solution a évité l'installation d'environ 88,900 mètres de pieux en béton, spécifiés dans les études initiales.

KEYWORDS: Ground improvement, bridge approach transition treatment, prefabricated vertical drain, CFA column, preloading.

1 INTRODUCTION

As a state government initiative, the AUD $1.88B Gateway Upgrade Project in Brisbane Australia involves the design, construction, operation and 10 year maintenance (DCOM) of a new Gateway Bridge, existing Gateway Bridge refurbishment, 12km of motorway upgrade and 7km of new motorway.

Located along Brisbane's north south arterial transportation corridor, the project provides improved connectivity to infrastructure such as Brisbane's Trade Coast region, Airport and the Port of Brisbane. Construction completion for the entire project occurred during November 2010.

Delivered by Queensland Motorways Limited (QML) in partnership with Leighton Abigroup Joint Venture (LAJV) and principal designers Maunsell SMEC Joint Venture (MSJV), the project involved construction and refurbishment of 30 bridge structures. Fourteen (14) of these bridges are located within the Brisbane Airport Interchange precinct, which is characterised by soft, compressible foundation soils up to 20 m in thickness, with road embankment heights up to 13m.

Initial design for the bridge approach treatment in this area comprised use of various forms of piled embankment supported by a mixture of approximately 4,900 continuous flight auger (CFA) piles, displacement auger piles, pre-stressed concrete piles and dynamic replacement columns.

Following cost and program analysis, an alternative staged ground treatment approach was proposed and adopted for the construction of 14 of the 28 bridge approaches within the Airport precinct. This paper focuses on one such abutment (denoted as BR25A) within this area. Site based geotechnical characteristics are identified together with key aspects of the initial and alternative design approach, summary of the alternative design methodology, comparison between predicted and actual ground settlement and outcomes successfully delivered through utilisation of a staged approach to ground treatment.

2 GEOTECHNICAL CHARACTERISTICS

2.1 *Subsurface conditions*

Geotechnical investigations indicated that the Airport Interchange is underlain by up to 20m Holocene (upper and lower) and Pleistocene alluvial deposits.

Upper Holocene alluvium within the Airport Interchange area was characterised by variable deposits of clay and silt (UH-C) and sands (UH-S). Lower Holocene alluvium (LH-C) was found to be of more uniform composition, comprising compressible silty clay to up to 20m depth.

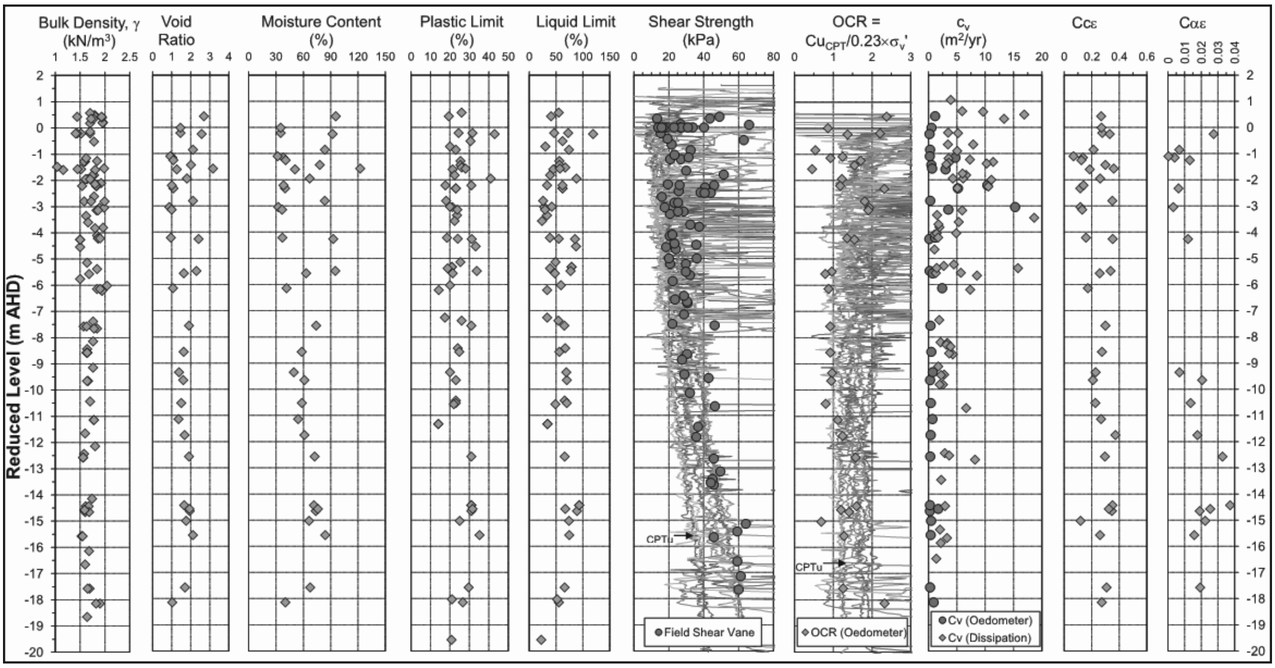

Figure 1: Soil test results - Airport Interchange precinct

Deposited during previous lower sea level, Pleistocene clay (P-C) was found to be characterised by less compressible stratum including stiff to hard clays and medium dense (or denser) sand layer. The ground water level was observed to be within 1-2m of the natural ground surface. The design ground water table was assumed at the ground surface level prior to construction (approximately RL 1.3m).

2.2 *Geotechnical design parameters*

The geotechnical parameters adopted for design and back-analysis of BR25A bridge approach are summarised in Table 1. Design parameters were derived taking into consideration the potential variability in the ground conditions and were calibrated against monitoring results during construction stage. The coefficient of consolidation in the horizontal direction (c_h) was assumed to be $2c_v$ and this ratio was found to be appropriate based on the back-analysis of field measurements.

Site investigation data indicated variation in strength, compressibility and hydraulic conductivity with depth and location within the Airport Interchange area. Field results from this vicinity indicate that the undrained shear strength (Cu) of the compressible clay increases with depth from approximately 10kPa to 60kPa. Cu values derived from piezocone were calibrated against the shear strength determined from the field shear vane. For geotechnical design, a characteristic Cu value of 20 + 0.6z1 (kPa) for UH-C and 23.6 + 2.7z2 (kPa) for LH-C was selected, where z1= 0 at RL 0 and z2= 0 at RL -6. Over-consolidation ratios (OCR) were derived from Oedometer and piezocone data. Figure 1 shows field and laboratory test results.

3 ALTERNATIVE DESIGN DETAILS

The alternative design philosophy involved initially improving the shear strength and compressibility characteristics of the soft soil by 6 months preloading in combination with placement of 4.3m surcharge. High strength geotextile (2 layers of WX600/50) and prefabricated wick drains (1.0m triangular pattern) were utilised for stability control. Refer to Figure 2 for schematic design arrangement nominated during design stage 1.

To facilitate construction haulage, a 2m high temporary berm in the longitudinal direction was proposed and this stabilising effect was incorporated in the design. The use of temporary berm achieved a reduction in the high strength geotextile requirement for stability control.

Following conclusion of preload, installation of final settlement transition treatment was anticipated, following review of actual performance of the embankment during preloading. The ground transition treatment for the alternative approach comprised 3 transverse rows of unreinforced concrete CFA columns (0.6m diameter on a 2.5m square grid with a UCS of 40MPa) overlain by a 20m long geotextile reinforced mattress to provide adequate pavement transition (see Figure 3). Two layers of WX1100/100 were specified in the longitudinal direction and one layer of WX200/50 in the lateral direction for the geotextile mattress. As a Stage 3 optimisation, 1m of embankment fill was excavated and replaced with lightweight fill (flyash) to increase the final over-consolidation ratio of the foundation soils and decrease preload period from 6 months to 2.4 months.

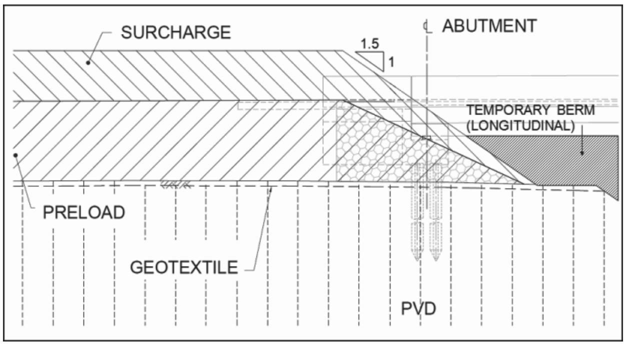

Figure 2: Typical stability and settlement control (schematic)

4 ALTERNATIVE DESIGN METHODOLOGY

The alternative design comprised a 3 staged approach to design, which occurred across the design and construction stages for the BR25A bridge approach.

4.1 *Stage 1 methodology*

Stage 1 involved undertaking design calculations to predict the required ground treatment to meet the settlement and stability criteria for the bridge approach transition. To meet the prescribed settlement criteria of 50mm (max) at the abutment

Table 1: Geotechnical Design Parameters

Layer	Depth (top of layer) (m)	γ_t (kN/m^3)	c' (kPa)	Φ' (°)	E' (MPa)	υ	OCR	$C_{c\varepsilon}$	$C_{r\varepsilon}$	$C_{\alpha\varepsilon}$	c_v (m^2/yr)
Fill	0.0	16.5 (16.5)	0 (0)	30 (30)	15 (15)	0.3 (0.3)	-	-	-	-	-
UH-C	0.5	17.0 (17.0)	2 (2)	27 (27)	-	-	2.5 (2.5)	0.3 (0.2)	0.03 (0.03)	0.01 (0.01)	6.0 (3.0)
UH-S	5.2	17.0 (17.0)	0 (0)	30 (30)	10 (10)	0.3 (0.3)	-	-	-	-	-
UH-C	5.5	17.0 (17.0)	2 (2)	27 (27)	-	-	2.0 (2.0)	0.3 (0.2)	0.03 (0.03)	0.01 (0.01)	6.0 (2.5)
LH-C	8.0	17.0 (17.0)	2 (2)	27 (27)	-	-	1.5 (1.5)	0.3 (0.2)	0.03 (0.03)	0.018 (0.01)	2.5 (1.5)
P-C	20.5	17.0 (17.0)	2 (2)	27 (27)	15 (15)	0.3 (0.3)	-	-	-	-	-
P-S	21.9	19.0 (19.0)	0 (0)	38 (38)	40 (40)	0.3 (0.3)	-	-	-	-	-

Note: Figures shown in brackets are values used in back analysis

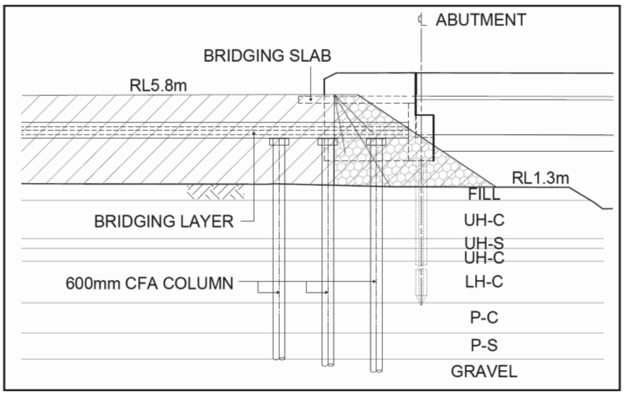

Figure 3: Typical operational stage bridge approach transition design arrangement proposed during Stage 1

location at the conclusion of the 10 years maintenance period in combination with 0.5% change in grade on the pavement surface, 6 months preloading with surcharge was proposed (refer section 3). Methodology for settlement assessment and calculation was undertaken in accordance with the method outlined by Hsi *et al.* (2008). Permeability numerical matching technique (Hird *et al.* 1992) was adopted to model the consolidation behaviour of wick drains in plain strain FEA using PLAXIS.

Suitability of the CFA column arrangement was evaluated using PLAXIS by comparing the predicted principal stresses with the allowable compression and tensile strength. Unreinforced CFA columns were modelled as non-porous elastic-perfectly plastic model with a tension cut-off at its tensile strength and plasticity parameters were obtained from compressive and tensile strengths.

In PLAXIS analysis, the two-dimensional stresses were converted into principal stresses (σ_1' and σ_3') in the two major and minor principal axis directions. The major principal stress value (σ_1') was compared with the factored unconfined compressive strength (UCS) and factored tensile capacity of concrete. A factor of safety of 2 was adopted for both compressive and tensile strengths. Structural adequacy was considered to be met if the principal stresses induced in the columns were less than the respective compressive and tensile capacities.

Soil structure interaction of bridge approach transition treatment was analysed in PLAXIS. The prescribed settlement method was used to analyse the differential settlement within the transition zone due to creep effect. Post construction creep settlement was firstly estimated. Subsequently, the ground behind the CFA columns was then prescribed to settle by an amount equal to the estimated post construction settlement. The embankment change in grade over any 4m length of pavement due to differential settlement was then calculated.

To meet the stability criteria (minimum safety factor of 1.2 in short term and 1.5 in long term), the embankment construction was constrained at a rate of 1m per week. Accordingly, strength gains due to consolidation of the UH-C and LH-C layers were able to be considered in the design. Time rate of consolidation of the UH-C and LH-C layers was further accelerated by use of PVD's. High strength geotextile in combination with lateral stability berms was utilised to provide additional stability control during construction. Stability analysis and design of soil reinforcement were carried out in accordance with the method outlined in Hsi and Martin (2005).

4.2 *Stage 2 methodology*

Stage 2 involved constructing the embankment using the design arrangement in combination with monitoring of the actual embankment performance during the preload period based on an observational approach. The objective of Stage 2 was to validate the design assumptions and ensure the safe and economical construction of the embankment by controlling the filling rate. Two settlement plates and markers, one vibrating wire piezometer, three inclinometers and one extensometer were implemented at BR25A to monitor the embankment performance during filling and preloading.

4.3 *Stage 3 methodology*

Conducted in parallel with Stage 2, Stage 3 involved back analysis during construction to validate the proposed transition design with respect to the settlement criteria. Additionally, the predicted date of preload removal was refined and investigation into opportunities to optimise the design from a cost and time perspective was undertaken. Back analysis was conducted using actual construction stage monitoring data.

To validate the magnitude of primary settlement, back analysis comprised initial modelling of the actual rate of embankment construction in PLAXIS and then comparing the primary settlement obtained from the modelling to the settlement actually observed in the field (see Figure 4). Calibrations were then made to the soil model to achieve an acceptable match between observed and predicted behaviour. The magnitude of primary settlement inferred by the Asaoka (1978) method using a constant time step of 7 days was compared to the actual field data and numerical predictions as an additional validation check on the degree of consolidation achieved. A further validation was undertaken by comparing the actual degree of excess pore water pressure dissipation recorded by the piezometer against the degree of excess pore water pressure dissipation calculated by FEM during stage 3 back analysis (see Figure 4). The magnitude of creep settlement was estimated based on the methods described in Mesri and Feng (1991), Mesri *et al.* (1997) and Stewart *et al.* (1994) and compared with the design criteria. The recommended preload duration was then refined to ensure that the predicted post construction settlement met the design criteria.

5 RESULTS

As shown in Figure 4, the magnitude of primary settlement predicted in Stage 1 was significantly greater than the actual primary settlement recorded during Stage 2 field monitoring. Compressibility and consolidation parameters were calibrated (calibrated parameters shown bracketed in Table 1) to achieve a good agreement between Stage 2 actual settlement results and settlement back calculated at Stage 3. From iterations during the Stage 3 back calculation, the source of the difference between Stage 1 and Stage 3 settlement predictions was partly attributed to the higher modified compression index $C_{c\varepsilon}$ and modified recompression index $C_{r\varepsilon}$ adopted during Stage 1 design. As a result, the modified secondary compression index $C_{\alpha\varepsilon}$ was also amended. As a further validation check, the primary settlement was also calculated using the Asaoka (1978) method. Using this method, primary settlement of approximately 1.79m was estimated, which compared reasonably well to the Stage 3 back calculated primary settlement estimate (1.80m).

The degree of excess pore pressure dissipated as measured by the piezometer during Stage 2 was compared against the degree of excess pore water dissipation from the Stage 3 back calculation. This comparison provided an additional validation check in relation to the estimated degree of consolidation of the compressible soils. A reasonable agreement between the measured (Stage 2) and back calculated degree of excess pore water dissipation (Stage 3) of the compressible soil was observed (see Figure 4).

6 CONCLUSIONS

An alternative staged approach to design and construction successfully achieved reductions of over 88,900 lineal metres of ground improvement piling that was specified in the initial design. BR25A approach has been presented as a ground treatment design case study; providing key geotechnical considerations, design methodology and a comparison of actual embankment performance with design predictions.

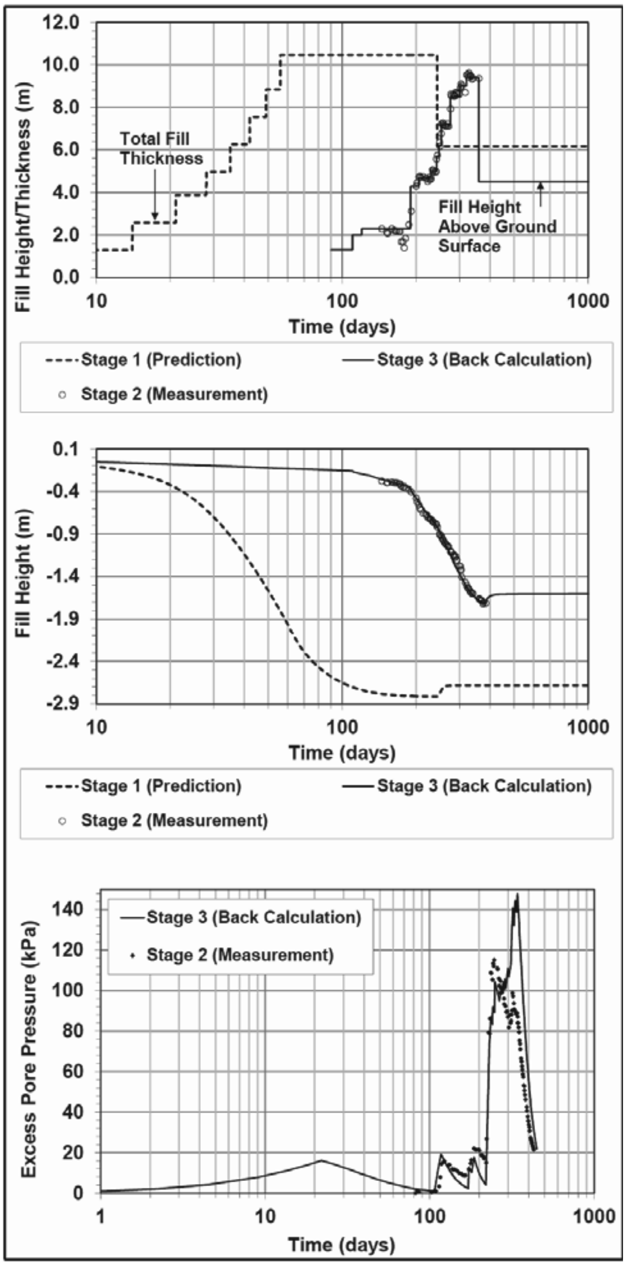

Figure 4: Predictions compared to construction monitoring

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