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Observed and predicted behaviour of clay foundation response under the Sunshine Motorway Trial Embankment

Comportement observé et prédit de réponse de fondation d'argile sous l'Autoroute de Soleil Remblai D'essai

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ABSTRACT
In 1992, Queensland Department of Main Roads was commissioned to monitor and interpret the findings of a fully instrumented trial embankment in Area 2A of the Sunshine Motorway, South East Queensland. This embankment was used to observe the foundation response upon loading, and evaluate the effectiveness of various ground improvement techniques on the soft, organic clays characteristic of this region. The principal focus of this paper is to compare the numerical predictions obtained from the finite difference analysis with field data obtained during the construction phase. The numerical analysis was undertaken using the finite difference program, FLAC, and deformations predicted on the basis of a fully coupled Biot consolidation model. Vertical drains installed in the trial embankment were modelled by converting each system into a number of equivalent parallel drain walls, and adjusting the coefficient of permeability to account for the smear generated through the drain installation.

RÉSUMÉ
Dans 1992, le Département de Queensland de Routes Principales a été commandé pour contrôler et interpréter les conclusions d'un entièrement instrumented le remblai d'essai dans le Secteur 2A de l'Autoroute de Soleil, Queensland de l'est du sud. Ce remblai a été utilisé pour observer la réponse de fondation sur le chargement, et évaluer l'efficacité de diverses techniques d'amélioration de sol sur la caractéristique d'argiles douce et organique de cette région. Le foyer principal de ce papier sera obligé à comparer les prédictions numériques obtenues de l'analyse de différence finie les données de champ a obtenu pendant la phase de construction. L'analyse numérique a été entreprise l'utilisation du programme de différence fini, FLAC, et les déformations ont prédit en se basant sur un modèle de consolidation de Biot entièrement couplé. Les drains verticaux installés dans le remblai d'essai ont été modelés en convertant chaque système dans plusieurs murs drains parallèles équivalents, et ajuster le coefficient de perméabilité pour expliquer le peturbation produit par l'installation drain.

1 INTRODUCTION

The design methods that exist for the analysis of ground improvement through preloading in conjunction with vertical drain installation are well established. However, when designing any of these improved systems, a more holistic approach incorporating the local knowledge of soil conditions and their behaviours should be used (Bo et al., 2003).

Presently, there are very few case studies presenting soft clay projects within Australia. The objective of this paper is to present and discuss a classic case study of a trial embankment constructed in South East Queensland.

2 PROJECT BACKGROUND

The Sunshine Coast is one of Australia's fastest growing regions. Continued economic and population growth in this region has placed increasing pressure on the region's main traffic corridor, the Sunshine Motorway.

With this pressure, and projections that the population of key centres in the region will approximately double in the next thirty years *(Planning Information and Forecasting Unit, Maroochy Shire Council)*, it will be necessary to reclaim previously unfavourable sites for development.

The case study analysed within this paper deals with a trial embankment constructed on soft clay foundations in Area 2 of the Sunshine Motorway, Stage II. It is located in the Maroochy Shire, Queensland, Australia (see Fig. 1).

Area 2 is 4.7 km in length and is subdivided into two areas – Area 2A and Area 2B. It extends from the lowlands that lie adjacent to the Maroochy River, to approximately 1.5 km south of the Yandina-Coolum Road.

Initial site investigations for the Motorway revealed small sections of very soft[1], extremely compressible, saturated marine clays of high sensitivity scattered along the proposed development route. Obviously, such soil conditions presented difficulties in the development of the new alignment.

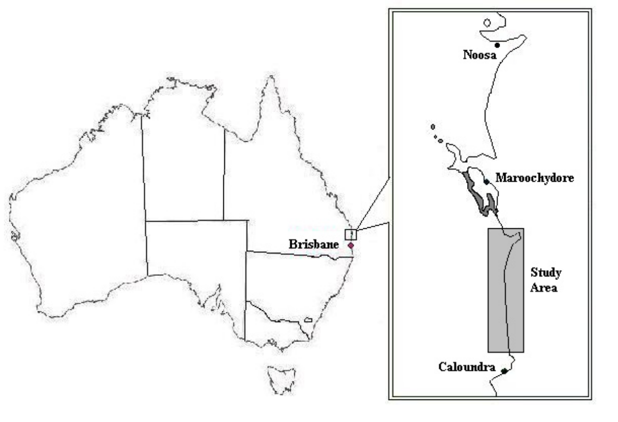

Figure 1. Map of Australia showing the approximate location of the Maroochy Shire, and the study area within it (inset)

[1] Consistency was defined using Main Roads specified conventions

In order to assess the feasibility of using vertical drains to reduce the consolidation time of these marine clays, a trial embankment was constructed in Area 2A of the proposed route.

3 FULL SCALE TRIAL EMBANKMENT

The trial embankment constructed measured approximately 90 m in length and 40 m in width, and incorporated 3 separate sections (see Fig. 2). These sections were identified as Sections A, B and C respectively.

Sections A and B were the two primary sections of the trial embankment and each measured 35 m in length. They represented a vertical drain spacing of 1 m and a no drains case respectively. Section C, an intermediate case, was approximately 20 m in length, and employed vertical drains spaced 2 m apart. The vertical drains in both Sections A and C were installed in a triangular grid pattern.

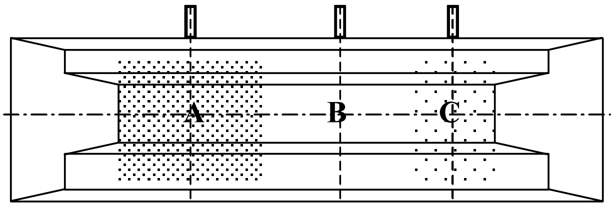

Figure 2. Plan View of Trial Embankment Design

The embankment fill consisted of a loosely compacted granular material with an average bulk density of 1.99 kg/m^3, and was placed using a multi-staged approach.

Berms were constructed to the design width of 5 m on the instrumented side and 8 m on the opposite side.

Half of the cross-section below the trial embankment was intensively instrumented to capture relevant data. The instrumentation consisted of inclinometers, horizontal profile gauges, sondex settlement systems, piezometers (pneumatic, standpipe and vibrating wire), earth pressure cells, strain gauges and other probes.

The locations of the pneumatic piezometers (PP) and settlement gauges (SC) used for verification are shown in Figure 3 below.

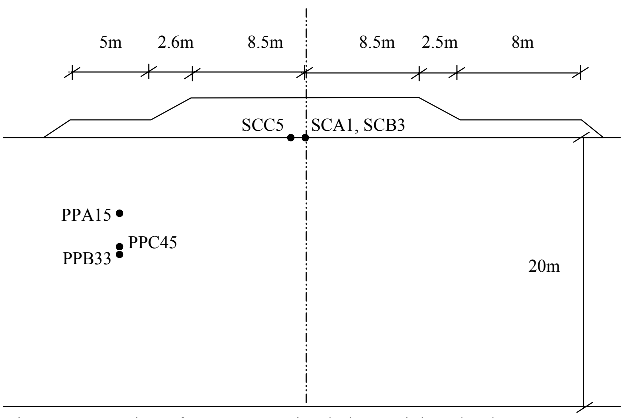

Figure 3. Location of Instrumentation below Trial Embankment

The generated pore pressures and deformations below each of the sections of the embankment were predicted using a finite difference analysis, and compared to results obtained in situ.

4 FINITE DIFFERENCE ANALYSIS

Analysis of this embankment was undertaken using the numerical modelling package, FLAC. FLAC (Fast Lagrangian Analysis of Continua) is a two-dimensional explicit finite difference program used extensively for modelling complex geotechnical problems.

A fully coupled (Biot) consolidation model was adopted for modelling the foundation response, as it has been found to most realistically represent the actual field behaviour of similar soft clays (Indraratna et al., 1997).

4.1 *Model Assumptions and Boundary Conditions*

The depth of foundation modelled was 20 m. This boundary was considered to be rigid as the sand layer below the overlying soft clay layer was dense enough to neglect any deformations associated with it.

Lateral boundaries of the finite difference mesh were each placed 150 m from the embankment centreline, and fixed in the horizontal direction. Boundary effects were kept to a minimum by exceeding the vertical dimension of the model by five times in the horizontal direction.

Both the bottom and top surfaces of the clay were assumed as free draining, and the water table at the ground surface. Although results from standpipe piezometers installed in the trial embankment indicated slight variability in the height of the water table, the latter of these two assumptions was considered reasonable as variations were only minor, and overall, the mean height corresponded to ground level.

4.2 *Preloading Construction*

Construction of the trial embankment was simulated through the application of a series of surcharges to the uppermost boundary. Due to the unsymmetrical nature of the embankment loading in both sections modelled, symmetry could not be exploited and thus, the full width of embankment was modelled.

The assumed rate of loading for the embankment as compared to the actual construction sequence of each section is detailed in Figure 4.

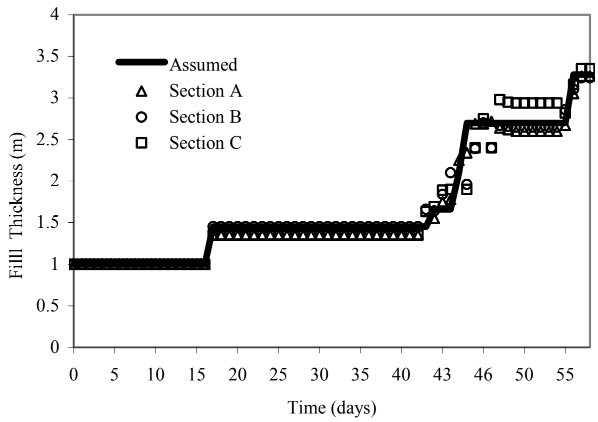

Figure 4. Assumed Construction History for Trial Embankment

4.3 *Subsoil Conditions*

In order to determine the foundation properties characteristic to the proposed development route, field sampling was undertaken. Laboratory testing, in the form of triaxial and oedometer testing, was carried out on samples from soft soil areas to assess the strength and consolidation parameters of the foundation.

From analysing the laboratory data available, generalised values for a number of soil properties were determined (see Tables 1 and 2 below). For all other parameters (Poisson's ratio, cohesion, friction angle), a value was assumed from published soft clay values.

Table 1: Average Soil Properties for Foundation Clays

Property	Average Value
Wet Density, ρ_w (kg/m^3)	1491
Poisson's Ratio, υ	0.3
Cohesion, c (Pa)	13.5×10^3
Friction Angle, ϕ (degrees)	35
Horizontal Young's Modulus, E_h (Pa)	7.62×10^5
Vertical Young's Modulus, E_v (Pa)	2.67×10^5

Table 2: Laboratory and Field Permeabilities for Foundation Clay

Property	Average Value
Horizontal Coefficient of Permeability, $k_{h,lab}$ (m/s)	2.08×10^{-9}
Vertical Coefficient of Permeability, $k_{v,lab}$ (m/s)	7.31×10^{-10}
Horizontal Coefficient of Permeability, $k_{h,field}$ (m/s)	1.31×10^{-9}
Vertical Coefficient of Permeability, $k_{v,field}$ (m/s)	4.95×10^{-10}

Laboratory testing of the horizontal and vertical samples obtained in situ revealed anisotropy in both the permeability and stiffness of the soil.

The anisotropic permeability value (k_h/k_v) calculated from laboratory values was 2.85. However, a sensitivity study using back calculations of permeability from the field measurements suggest that both the vertical and horizontal permeability values are lower than derived from the laboratory testing, and the actual anisotropy existent between the vertical and horizontal permeability is closer to 2.65.

The ratio between soil stiffness in the horizontal and vertical directions (E_h/E_v) was approximately 2.85, and was accounted for in the models by using a cross shear modulus, G_{xy}, This modulus for anisotropic elasticity was determined by using the formula by Lekhnittskii (1981):

$$G_{xy} = \frac{E_x E_y}{E_x(1+2\upsilon_{xy}) + E_y} \tag{1}$$

where G_{xy}=cross shear modulus (N/m^2), E_x=elastic Young's Modulus in the plane of isotropy, E_y= elastic Young's Modulus perpendicular to the plane of isotropy and ν_{xy}=Poisson's ratio for normal stress in the plane of isotropy due to uniaxial stress in the perpendicular plane.

4.4 *Vertical Drains*

In order to model the vertical drains in plane strain, the vertical drain system was converted into equivalent drain walls, and the horizontal permeability of the soil adjusted to account for smear using the permeability matching method outlined by Hird et al. (1992).

The matched permeability derived from this method are given by the following equation:

$$k_{pl} = \frac{2k_{ax}}{3\left[\ln(n/s) + (k_{ax}/k_s)\ln(s) - 0.75\right]} \tag{2}$$

where k_{pl}=horizontal permeability of the soil in plane strain, k_{ax}=horizontal permeability of the soil in the axisymmetric condition, k_s=horizontal permeability of the soil in the smear zone, n=ratio of the unit cell width to drain radius, and s=ratio of the smear zone width to drain radius.

Well resistance was taken into account through the discharge capacity applied to the vertical drains.

Details of the vertical drain system used in Sections A and C are shown below in Table 3.

Table 3: Parameter Values Related to Drain Behaviour

Item	Section A	Section C
Vertical Drain Material	Nylex Flodrain	Nylex Flodrain
Drain Length, H (m)	10.0	10.0
Spacing, S (m)	1.0	2.0
Equivalent Drain Radius, r_w (m)	0.035	0.035
Smear Radius, r_s (m)	0.250	0.250
Horizontal Plane Strain Permeability, k_{pl} (m/s)	1.697×10^{-10}	1.496×10^{-10}

5 ANALYSIS OF RESULTS

The results produced for surface settlements and pore pressure generation compared favourably with the actual field measurements recorded by the Queensland Department of Main Roads (1991a, 1991b, 1992) for foundation response under the Sunshine Coast Motorway Trial Embankment site.

5.1 *Surface Settlements*

The predicted and observed settlements under each embankment section are shown in Figures 5 and 6. The settlement points for comparison were obtained from settlement gauges SCA1, SCB3 (both under the centreline) and SCC5 (1m left of centreline).

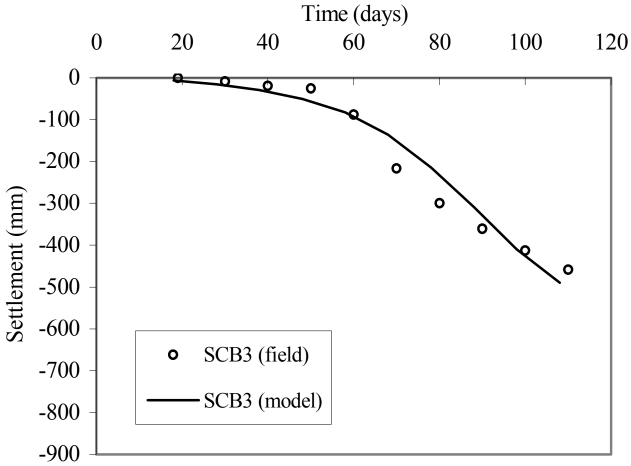

Figure 5. Predicted and Observed Settlements under Section B (Untreated) of the Trial Embankment

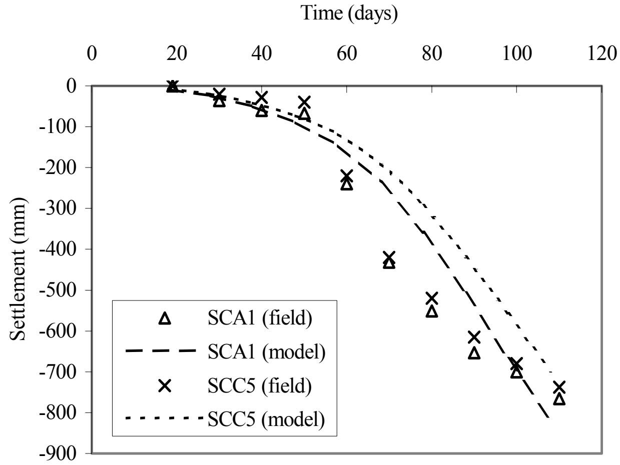

Figure 6. Predicted and Observed Settlements under Sections A and C (Treated) of the Trial Embankment

As demonstrated in the Figures 5 and 6, the behaviour of the foundation response was predicted well.

The settlements produced by the sections installed with vertical drains (Sections A and C) show a much faster rate of consolidation, than the untreated section (Section B). Minimal difference was evident in both the observed and predicted values for Sections A and C, and therefore, it can be seen that any benefits derived from installing vertical drains at a 1 m spacing as compared to a 2 m spacing, will be minimal and is heavily negated by the smear effect produced by their installation.

The divergence observed between the settlement-time relationships for the 1m and 2m drain spacing predicted by the model is larger than that observed in the field. The authors attribute this to physical problems associated with vertical drains such as clogging of the drain filter material and/or kinking of the drain core.

The overall consolidation time for t_{100} was reduced from a predicted 49 years, for the untreated section, to approximately 200 days for both of the treated sections suggesting that consolidation for both systems would finish within the specified 300 day completion period.

Assumption of a single homogeneous layer for the FLAC model has contributed to the predicted settlement values differing from the field values, and the point of inflection for the settlement-time relationship occurring later in the model than in the field. However, the model is considered as an idealised situation, and thus, these differences are not considered significant.

5.2 *Pore Pressures*

Table 4 indicates the predicted versus observed pore water pressures for piezometers PPA15, PPB33 and PPC45 at 110 days. These piezometers were located 14m left of centreline, and 6.5 m, 9.5 m and 9.0 m below ground level respectively.

Table 4: Predicted and Observed Pore Pressures at 110 days

Section Instrument Number	Depth Below Ground Level (m)	Observed Total Pore Pressure (kPa)	Predicted Total Pore Pressure (kPa)
PPA15	-6.5	88.8	91.06
PPB33	-9.5	116.7	125.4
PPC45	-10.0	115.4	125.9

Furthermore, a plot of total pore pressure versus time for PPC45 is shown below in Figure 7.

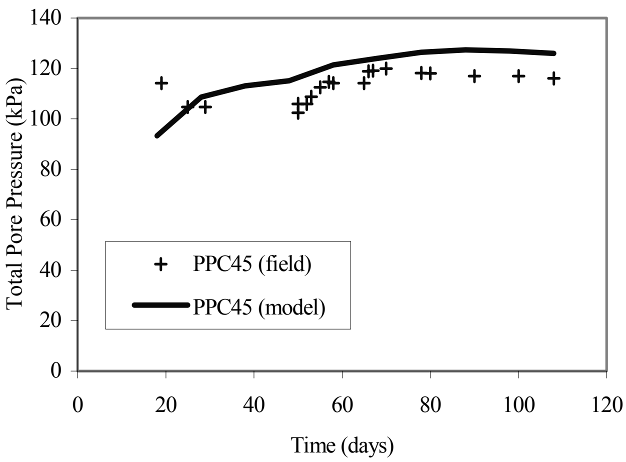

Figure 7. Pore pressure dissipation plot for PPC45 during embankment construction phase

As shown, the pore pressures behaviour predicted using the numerical model compared favourably with in situ measurements. The maximum deviation in predicted vs. actual pore water pressure measurement was recorded at PPC45.

6 CONCLUSIONS

At present, there are very few published case studies that present projects in Australia that involve ground improvement. This paper presents a classic case history of a trial embankment that was built on soft, organic clay in Area 2A of the Sunshine Coast Motorway in South East Queensland, Australia. It details the deformation behaviour and pore pressure response below a fully instrumented trial embankment during its construction phase.

The primary objective of constructing this trial embankment was to assess the performance of vertical drains in reduce the consolidation time of the marine clays within this region.

In order to achieve this purpose, the embankment was considered in three sections. Two of the sections (Sections A and C) were installed with prefabricated vertical drains at spacings of 1 m and 2 m respectively, and the third was left untreated (Section B). Each vertical drain system adopted a triangular grid pattern arrangement.

The numerical analysis was undertaken using the finite difference modelling package, FLAC, and each of the sections was modelled separately as a fully coupled plane strain case.

The results predicted from the analysis compared favourably with the field measurements of pore pressure and settlement.

A sensitivity study carried out on each of the models using coefficients of permeability back calculated from the field measurements suggest that both the vertical and horizontal permeability values are lower than derived from the laboratory testing, and the actual anisotropy ratio between the vertical and horizontal permeability in the field is closer to 2.65.

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