Geosynthetic reinforced column supported embankments and the role of ground improvement installation effects

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Abstract: For geosynthetic reinforced column supported embankments (GRCSE) supporting a high embankment, lateral forces associated with lateral sliding and embankment stability often govern the acceptability of a given design under serviceability conditions. Frequently, the complex soil–structure–geosynthetic interaction, the size, and the three-dimensional nature of a GRCSE necessitate the use of numerical analysis to assess embankment performance relative to serviceability criteria. However, traditional finite element method techniques used to model serviceability behaviour are limited in their ability to model the geotechnical mechanisms associated with column installation, equilibration, and group installation effects. These installation effects are examined herein based on a GRCSE field case study located in Melbourne, Australia, that has been extensively instrumented. The role that these installation effects have on the performance of the GRCSE is highlighted and the behaviour of the columns supporting the embankment is emphasized. It is shown that cracking of the unreinforced columns supporting the embankment is likely inevitable and that the reduction of lateral resistance provided by the columns should be accounted for in design. The suitability of various numerical approaches currently used in design to model the columns supporting the GRCSE, and the embankment itself, are discussed and recommendations are made.

Key words: geosynthetic, field case study, column-supported embankment, installation effects.

Introduction

The design of geosynthetic reinforced column supported embankments (GRCSEs) requires the consideration of a number of limit state conditions. These typically include, but are not limited to, the following: embankment stability, lateral sliding, column group capacity and extent, vertical load distribution (soil arching), reinforcement strain, and foundation settlement (Rogbeck et al. 2004; BSI 2010; Lawson 2013). The focus of this paper is on the global-scale mechanisms that lead to horizontal deformation, such as lateral sliding, embankment stability, and edge instability (column group extent) based on a field case study located in Melbourne, Australia. These mechanisms are particularly important in high embankments, such as the field case study considered herein, where lateral forces are larger.

Assessment of the global-scale mechanisms in a GRCSE is dependent on the type of ground improvement (or semi-rigid inclusions) adopted, and for a GRCSE there are a number of options available. Some examples from field case studies include drilled displacement columns (DDCs) (Briançon et al. 2008; Briançon and Simon 2011; Fok et al. 2012; Wong and Muttuvel 2012), high-speed piles (van Duijnen et al. 2010), timber piles (Hsi 2001; van Eekelen et al. 2010), and rammed aggregate piers (Abdullah and Edil 2007).
All of these options require the installation of semi-rigid (columns) or rigid (piles) inclusions that cause full-displacement installation effects.

Despite the increasing use of DDCs to support geosynthetic-reinforced embankments, the role of installation effects on the behaviour of GRCSEs, and on the installed columns themselves, has not received significant attention. The design methods that do exist for DDCs are largely concerned with the determination of bearing behaviour from empirical methods, which have mostly been developed from in situ load test results. These approaches relate unit base and shaft resistance to cone penetration test (CPT) tip resistance ($q_t$) or standard penetration test (SPT) blow count (N) (e.g., Bustamante and Gianselli 1993, 1998; NeSmith 2002; Brettmann and NeSmith 2005). While these methods may provide a means to calculate bearing capacity, they provide limited insight into installation effects.

A review of numerous studies that have investigated the global-scale behaviour of GRCSEs (Massé et al. 2004; Liu et al. 2007; Jenck et al. 2009; Chatte and Lauzon 2011; Ariyarathne et al. 2012; Zhang et al. 2013; Bhasi and Rajagopal 2015) indicates that numerical analysis techniques are by far the preferred method of analysis for assessing serviceability behaviour. The study of global-scale GRCSE mechanisms through full-scale case studies as well as laboratory-scale physical modelling and centrifuge modelling is far less common. However, current finite element method (FEM) numerical techniques are limited in their ability to model large-strain problems such as pile installation (Więckowski 2004) and generally advanced methods such as Coupled Eulerian-Lagrangian (CEL) (Qiu et al. 2011; Pucker and Grabe 2012) or point-based (meshless) methods such as smooth particle hydrodynamics (SPH) (Bui et al. 2008) or the material point method (MPM) are required. In some cases, “work-around” methods are adopted within a FEM analysis to account for installation effects. Wong and Muttukul (2012) and Gniel and Haberfield (2015) consider the columns supporting a GRCSE as “geotechnical” elements, which are assumed to be cracked and are modelled as plate elements with little to no bending stiffness. However, in many cases, installation effects are ignored entirely, or not acknowledged, and no position is taken as to the effect that this assumption has on the numerical output.

Whilst this shortcoming may sound like a trivial matter, it is well known from field-scale observations of pile installation effects, particularly for driven piles (Randolph et al. 1979; Coop and Wroth 1989; Lehane and Jardine 1994; Eigenbrod and Issigoni 1996) and through the use of analytical methods such as cylindrical cavity expansion techniques (Carter et al. 1979; Randolph and Wroth 1979; Randolph 2003) and the strain path method (Baligh 1985; Sagaseta and Whittle 2001), that the installation of full-displacement piles can induce lateral displacement and excess pore-water pressure (pwp) in the vicinity of the pile shaft. This has also been shown recently for an isolated full-scale controlled modulus column (Suleiman et al. 2015) and for a large-diameter cast in situ concrete pipe pile (Liu et al. 2009) that were both extensively instrumented. A limited number of studies have described group installation effects: Poulos (1994) investigated the effects of driving a pile adjacent to a pile in clay. O'Neill et al. (1982) investigated installation effects for a group of steel tube piles, and Kitazume and Maruyama (2007) assessed the stability of a group of deep soil mixed columns. The cumulative column installation effects, which are relevant for GRCSEs due to the large number of columns or piles installed in a dense array, are considerably more difficult to assess. However, it is thought that a reasonable approximation and insight into group installation effects can be gained by considering the installation of multiple columns within the framework of cavity expansion theory; this is done to assist with interpretation of the field data.

One of the benefits frequently cited to justify the construction of a GRCSE is the rapid speed of construction permitted compared with unsupported geosynthetic reinforcement embankments, where it is often necessary to monitor the excess pwp responses as the embankment is raised, to ensure that an adequate factor of safety for embankment stability is maintained. As embankment load in a GRCSE is (predominately) transferred directly to the founding unit through load distribution in the load transfer platform (LTP), behaviour of the soft soil is generally considered to be of little importance when considering embankment behaviour. However, it is shown here that the excess pwp that develops due to ground improvement works is considerable. Furthermore, where embankment construction proceeds rapidly, and immediately after ground improvement works, this excess pwp in the soft soil underlying the embankment can be expected to reduce the factor of safety for slope stability; increase embankment deformation through enhanced lateral sliding, due to the reduction in effective stresses along the sliding plane; and in addition, impart considerably larger structural actions (bending moments and shear force) on the columns supporting the embankment.

The objective of this paper is to demonstrate the manner in which group installation effects affect global behaviour of a GRCSE and the columns that support the embankment. In addition, the appropriateness of a number of numerical modelling techniques used to simulate the performance of a GRCSE are discussed, based on the field case study data presented and an interpretation of installation effects.

Ground improvement using drilled displacement columns

In recent years, DDCs have seen increased use largely due to the increased torque capacities of modern piling equipment, with penetration depths of up to 30 m now achievable with drilling tool diameters typically between 200 and 550 mm (Larisch et al. 2013). This form of ground improvement goes by a number of names: auger pressure-grouted displacement piles is the term used frequently in the North American context, while drilled displacement piles or the proprietary-named Controlled Modulus Column is used frequently in the European context. Prezzi and Basu (2005) provide an overview of the European and North-American nomenclature. Throughout this paper, the more generic term drilled DDCs is used for the ground improvement works associated with this case study to distinguish this technique from piling applications.

The construction of a DDC is similar in many respects to the nondisplacement continuous flight auger (CFA) pile and has largely evolved from this piling technique. The main difference is that the DDC drilling tool fully displaces the soil during the installation phase. NeSmith (2002) provided a detailed description of the DDC construction process. Similar to a CFA pile, installation data recorded during installation generally comprise time, depth, mast inclination, torque, drilling stem rotation rate, and penetration rate. During the concreting phase, additional data are recorded and include concrete slurry pressure and lifting speed, which is often used to infer an “as-built” DDC, profile. The use of data-acquisition hardware for the monitoring and verification of the DDC installation is an integral part of the ground improvement works, which enables delineation of the subsurface materials and increased confidence that founding conditions and column integrity are achieved. These construction-related aspects form an important part of the GRCSE design; the interested reader is referred to Piscsalko and Likins (2004) and NeSmith and NeSmith (2006) for a more expansive discussion.

Installation effects: general behaviour

The installation effects associated with a driven pile are well described by Randolph (2003) and summarized briefly here. The three fundamental mechanisms described below, in a generalized form, are applicable to full-displacement columnar elements used...
Cylindrical cavity expansion theory is broadly applicable (Yu 2013) as associated with a GRCSE. The zones along a pile shaft where cylindrical cavity expansion occurs are indicated in Fig. 1. These zones broadly resemble embankment performance, the excess pwp and displacement field for the soil mass surrounding a driven pile are derived in the Supplementary Data section. Examples of excess pwp solutions at various times are shown in Fig. 2; these solutions are used to assess the piezometer data presented later in this paper. The soft soil underlying the GRCSE is the Coode Island Silt; the material properties indicated in Fig. 2 are from King et al. (2016). The axisymmetric solutions presented indicate an initial excess pwp of 121 kPa at the column–soil interface, which reduces to 0 kPa at radius R, where R = 2.54 m. As excess pwp dissipates radially outward with time t, excess pwp (u) is reduced at the column/soil interface and increases at values of radius r > R, although this increase is minor.

To describe the displacement field, the following expression for initial radial soil displacement (Δr) was proposed by Carter et al. (1980):

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\Delta r = \sqrt{r^2 + \beta r_0^2} - r
\]

where β is the displacement ratio, which is the ratio of net to gross cross-sectional area of a pile being driven, and r0 is the pile radius. For a solid (driven) pile β = 1; however, for open-ended steel tube piles the value of β is <1 and depends on the extent to which a soil plug develops in the base of the pile during installation. The DDC should theoretically have a value of β = 1, similar to a driven pile. However, it will be shown that the displacement fields that develop due to DDC installation are overpredicted when β = 1 is used. The radial displacement field from eq. (1) is shown in Fig. 2 along with the long-term radial consolidation (t → ∞) solution. The time-dependent radial displacement during the equilibration phase is the sum of the initial radial displacement and time-dependent radial consolidation. The closed-form analytical solution for cylindrical cavity expansion in a (drained) cohesive-frictional material by Yu and Carter (2002) is also shown for comparison; this solution approximates the long-term case where radial consolidation is complete (i.e., end of equilibration phase).

Cylindrical cavity expansion theory was developed further by Randolph et al. (1979) to describe the stress distribution for an expanding cylinder in a work-hardening elastoplastic soil model (“modified Cam clay”). When an isolated full-displacement pile is driven into a low overconsolidation ratio (OCR) soil, the initial vertical and horizontal effective stresses (σv and σh, respectively) are modified considerably within radius R, with the radial effective stress (σr) becoming the major principal stress and both the vertical (σv; intermediate principal stress) and circumferential effective stresses (σc; minor principal effective stress) reducing considerably. For the North Dynon case study (described below), the centre-to-centre spacing is equal to or less than the radius R. As a result, the stress distributions arising due to the installation of DDCs are expected to overlap for the numerous closely spaced DDCs installed as part of the ground improvement considered herein.

Field case study

The North Dynon embankment examined herein was one of four GRCSEs constructed as part of the Regional Rail Link project in Melbourne, Australia, in 2012. The North Dynon embankment is a 60 m long widened GRCSE; a farther 120 m of the widened embankment has more favourable subsurface conditions and did not require ground improvement. The embankment is founded on a 2 m thick fill unit overlying a sequence of Quaternary aged sediments (Yangtze Delta sediments) overlying the Silurian-aged siltstone–sandstone of the Melbourne formation (Neilson 1992). The

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1 Supplementary data are available with the article through the journal Web site at http://nrcresearchpress.com/doi/suppl/10.1139/cgj-2017-0036.
near-surface Coode Island Silt is of particular importance owing to its wide spatial distribution, low undrained shear strength ($s_u$, typically increases from about 15 to 40 kPa at depth) and its considerable thickness of up to 25 m in parts. The geological profile is complex and highly variable, with the Coode Island Silt varying from 7 m to 15 m along the length of the embankment; the underlying stiff to very stiff clays of the Fishermens Bend Silt unit form the founding unit for the columns. A detailed description of the subsurface conditions is presented in King et al. (2016, 2017a). The North Dyonon embankment is a widened embankment and comprises a lower and upper level LTP with a gabion wall around the periphery of the embankment. Ground improvement works comprised the installation of 450 mm diameter DDCs installed in multiple stages (Fig. 3) with 1 m square cast-in-place column heads. The lower level LTP was completed after stages 1 and 2 ground improvement works (146 DDCs installed) while the upper level was completed following stages 3a and 3b ground improvement works (104 DDCs installed). The instrumentation was installed in two areas (Nos. 1 and 2; Fig. 3), a cross section at the location of area No. 2 is shown in Fig. 4. This paper presents post-construction survey data as well as inclinometer, piezometer, and tiltmeter data. Additional earth pressure cell (EPC) and strain gauge data have been presented in a complimentary paper where the localized LTP behaviour was examined (King et al. 2017a). A similar embankment cross section through area No. 1 is presented in King et al. (2017a) along with a more detailed description of the GRCSE design. One of the other four GRCSEs (a back-to-back tied wall embankment) constructed as part of the Regional Rail Link has also been described separately by Gniel and Haberfield (2015).

### Piezometer

A Geotechnical Systems Australia (GSA) vibrating wire piezometer (model 1200) was installed within the Coode Island Silt beneath area No. 1 to a “reduced level” (R.L.) (relative to mean sea level in Australia) of −3.45 m. The piezometer was installed using a “push-in” place method immediately prior to the commencement of the lower-level stage 2 works. The piezometer was housed in a GSA push-in cone and advanced progressively through the subsurface materials using a 1 in. (1 in. = 25.4 mm) steel pipe and a casing, which slides over the push-in housing. The long-term hydrostatic pwp is 39 kPa and indicates a groundwater level of about 0.6 m R.L., although this shows some seasonal variation. It was proposed to install additional piezometers in both instrumentation areas; however, this could not be done due to installation difficulties (drilling difficulties and obstructions in the fill unit) and constraints related to the construction timeline.

### Vertical inclinometers

Vertical inclinometer Nos. 1 and 2 were installed using a specialist drilling contractor to advance the boreholes using a combination of solid auger and then washbore drilling techniques. The inclinometer casing has a 70 mm outer diameter and 58.5 mm inner diameter and was grouted in accordance with the procedure described by Mikkelsen (2002). Inclinometer No. 1 is 22.14 m in length with the toe of casing at R.L. = −20.19 m (adjacent DDCs were installed to R.L.s between −14.0 and −16.2 m). The top 250 mm below the surface of the piling hardstand had a sacrificial protective casing around it. The inclinometer No. 2 casing is 18 m in length with the top of the casing at R.L. = 2.10 m and toe at R.L. = −15.9 m (adjacent DDCs were installed to depths of between R.L. = −12.41 and −14.13 m). Inclinometer readings were taken at 0.5 m intervals using the four-pass method recommended by Mikkelsen (2003). The data have been converted to a local longitudinal–transverse co-ordinate systems.

### Tiltmeters

Three Slope Indicator – micro electromechanical systems (MEMS) bi-axial tiltmeters were cast in-place at various depths within DDC C15 to measure the post-construction tilt of the column. This DDC is in the outer row of columns, beneath the gabion wall in area No. 2. Inclinometer No. 2 is offset 1.6 m from DDC C15 where the tiltmeters are installed to enable calibration between the two. The MEMS tiltmeters measure 32 mm in diameter by 190 mm in length and have an inclination range of ±10° with resolution of 9 arc seconds, which is temperature corrected. The tiltmeters were mounted at various intervals within a 5.91 m length of 50 mm diameter polyvinyl chloride (PVC) pipe installed immediately after the DDC installation, similar to the installation of a reinforcement cage. The base of the PVC pipe was left open to avoid the instrumentation becoming buoyant during installation. Tiltmeters Nos. 1, 2, and 3 were installed at R.L. = −3.0, −0.57, and 1.71 m (within the DDC head), respectively.

### Field case study — installation effects

#### Porewater pressure

Data from the piezometer in area No. 1 (Fig. 5) show the long-term pwp dissipated to a hydrostatic condition over a period of approximately one year. Two major increases in pwp were
observed during construction: the first occurred the day after installation of the columns (day 57) and coincided with the commencement of stage 2 works and the second coincided with the stage 3b works. During stage 2, the pwp increased by approximately 70 kPa and a maximum pwp of 127 kPa was measured. An approximately exponential decay of pwp was observed after installation. The piezometer readings were taken manually during this time, making detailed assessment of installation effects difficult. The data recorded during stage 3b works were recorded on 4 h intervals, allowing a more detailed analysis.

The measured increases in pwp are due to (i) increase in the subsoil stress due to the embankment load and (ii) increases due to column installation. Regarding point (i), stress acting in the area between the column heads zone was measured by EPC5 (the location of this EPC is shown in the Supplementary Data section, Fig. S3). By assuming a 1.5V:1H stress distribution (where V is vertical and H is horizontal) with depth through the fill and Coode Island Silt units, the applied vertical stress acting at the level of the piezometer can be estimated. This applied stress acting at R.L. = −3.45 m is shown in Fig. 5 and indicates that during stage 2 works the increase in pwp (70 kPa) is due to column installation alone as the subsoil stress was 0 kPa at this time. During stage 3b the increase in pwp was 70 kPa and the calculated increase in applied stress due to embankment loading is just 6 kPa. The majority of the measured pwp increase can therefore be attributed to column installation effects and shows a response similar to that observed by O’Neill et al. (1982) for a driven pile group installed into an overconsolidated clay.

A detailed plan view in Fig. 6 indicates the location of the columns installed near area No. 1 during the stage 3b works between

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**Fig. 3.** North Dynon embankment: plan view.

**Fig. 4.** Embankment cross section: area No. 2. RL, reduced level.
days 130 and 134. The response of the piezometer during stage 3b works is presented in Fig. 7a and the responses of the EPCs and strain gauges are shown in Fig. S2 in the Supplementary Data section. The instrumentation layout in area No. 1 is shown in King et al. (2017a). A daily increase in pwp can be observed during the working hours (0700–1700); at the completion of the day’s work, partial dissipation of excess pwp occurs overnight. The daily data indicate upper maxima of measured pwp in the range of 125–130 kPa and is consistent with the maximum pwp observed on day 57 during stage 2 works.

To provide a quantitative interpretation of the pwp data, the pwp response due to the installation of 10 columns between 11 and 12 September was modelled based on the cylindrical cavity expansion theory outlined above and in further detail in the Supplementary Data section. Over an area of 9 m by 7 m, and based on the Coode Island Silt properties at R.L. = −3.45 m (depth of piezometer), $s_u = 25$ kPa, and shear modulus, $G = 3200$ kPa (King et al. 2016), the time-dependent axisymmetric equations for each column installation were solved. Based on these parameters, axisymmetric solutions plotted at various times after installation are presented in Fig. 2; these solutions provide an indication of the magnitude of pwp increase in the vicinity of an isolated column as well as the rates of pwp dissipation. The calculated excess pwp surfaces were superimposed in a spreadsheet application to assess the cumulative installation effect. Solutions at day 131 midday and 1600, day 132 at 0800 and 1600, and day 133 at 0800 are presented (Figs. 8a, 8b, 8c, 8d, and 8e, respectively), representing the major periods of pwp installation and equilibration. Figure 8f compares measured and calculated excess pwp. The excess pwp was calculated as the absolute pwp reading less the initial pwp of 59.9 kPa (see Fig. 5). The magnitude of discrepancy in pwp between the calculated solutions and measured pwp is considerable and the zone of influence predicted by cylindrical cavity expansion theory is not consistent with the observed data.

The initial pwp distribution (Fig. 2) reduced to 0 kPa at a radius of 2.54 m. This is not consistent with the increase in pwp observed following the installation of columns J2 and H3 located at a radial distance of 13.9 and 8.1 m, respectively. Similarly, increases in pwp were observed on days 133 and 134 due to columns installed at radial distances greater than 12.4 and 11.4 m, respectively. Increases in pwp at a radial distance >2.54 m were predicted to occur as part of the equilibration phase (Fig. 2); however, these increases were minor and did not increase until a number of days after installation. The cylindrical cavity expansion theory described above was developed for an elastic–perfectly plastic soil model to predict excess pwp; shear-induced pwp is not described by this approach. Some portion of the difference in calculated and observed pwp can be attributed to this effect; however, for slightly overconsolidated soil such as the Coode Island Silt, this contribution is expected to be relatively small (Randolph 2003). It follows...
then that the difference arises due to the initial pwp distribution adopted at the time of installation. It is suggested that this difference is due to hydraulic fracturing of the Coode Island Silt in the vicinity of the column shaft. This behaviour has been shown previously for displacement piles by Massarsch and Brooms (1977), Randolph et al. (1979), and Asaoka et al. (1994). The hydraulic fracturing of soils due to deep mixed columns by Shen et al. (2003, 2004) and in grouting applications by Gottardi et al. (2008) and Marchi et al. (2013) is also well documented.

Lateral deformation

Two inclinometers described lateral deformation of the embankment. Inclinometer No. 2 was installed beyond the embankment footprint to assess both column installation effects and long-term embankment behaviour. Inclinometer No. 1 was installed within the embankment footprint to assess installation effects and was decommissioned at the completion of stage 1 works. Data from inclinometer No. 1 is described by five phases, where a phase is one or more column installations followed by a reading. The lateral displacement measured by inclinometer No. 1 is similar to data from Inclinometer No. 2 and is presented in the Supplementary Data section.

The location of inclinometer No. 2 and its axes, along with adjacent installed columns, is presented in Fig. 9. Information relating to the installed columns and inclinometer readings is presented in Table 1 and includes the radial distance \( r \) between inclinometer and installed column, \( r/D \) ratio \( D \) is column diameter, and time between installation and a subsequent inclinometer reading. The incremental transverse lateral displacement for the three phases and subsurface conditions are shown in Fig. 10 along with the predicted lateral displacement based on eq. (1). The general shape of the lateral displacement profile (due to installation) is consistent among the various phases and is consistent with data from inclinometer No. 1 (data and analysis presented in Supplementary Data section). A large outward lateral movement in the transverse direction was measured, particularly near the upper surface of the Coode Island Silt, and the magnitude of lateral movement generally correlates with the radial distance. A maximum of lateral displacement of up to 35 mm is generally observed at about 2 m below the fill – Coode Island Silt interface (R.L. = −2 m). For inclinometer No. 1 the maxima is up to 60 mm. It appears that both the overlying stiff fill unit and column socket in the stiff Fishermens Bend Silt act as lateral restraints. However, Larisch et al. (2014) also observed similar behaviour in a soil profile comprising stiff clay overlying hard clay. It is inferred, therefore, that this displacement response is dominated by soil heave during the first few metres of penetration (i.e., resembles spherical cavity expansion) and is not due entirely to subsurface conditions. The observed lateral displacement profile results in three points of inflexion in the profile; this has important implications when assessing structural response of the column later in this paper.

Assessment of the transverse lateral displacement alone provides only limited insight into the three-dimensional nature of the problem. A vector diagram showing the directional and magnitude of lateral movement at R.L. = −3.55 m is presented in Fig. 11. Displacement vectors are inversely scaled based on the radial distance (i.e., vector length ≈ 1/r) and a resultant vector for each phase is shown. The directional component of soil movement was predominantly in the transverse direction away from the embankment (−ve Y-axis) with a minor component of movement in the longitudinal direction (+ve X-axis). The resultant phase vectors do not align precisely with the measured soil movement, although the general agreement is good and the difference is attributed to the nonsymmetric arrangement of columns over a larger area than that shown in Fig. 9 (see Fig. 3).

Cumulative installation effects

Multiple columns were installed for stages 3a and 3b works; these installation effects are assessed through the data from inclinometer No. 2. Dates of the inclinometer readings and their relationship with the various stages of ground improvement works are shown in Table 2. Various cumulative and incremental inclinometer profiles are plotted in the transverse (Y-axis) direction (Fig. 12) and the locations of the tiltmeters are also indicated (described further below). The longitudinal movement (X-direction) during this period was minimal. The first three cumulative profiles show the lateral displacement due to stages 1, 3a, and 3b works. The fourth profile shows the long-term behaviour over a period of 660 days (days 160–741). The lateral displacement measured during this period corresponds to the period where the Coode Island Silt has undergone long-term equilibration and is observed as 10 mm of movement towards the embankment. A further five incremental profiles are shown that describe the installation and (or) equilibration phases associated with stages 3a and 3b works and the long-term condition. For these phases, installation causes movement away from the embankment and equilibration causes movement towards the embankment. The response due to the stage 1 works (Fig. 10) was the same.

The equilibration causes inward movement of the columns that is in a direction opposite to the outward movement of the embankment caused by lateral spreading and (or) embankment instability. The equilibration imposes lateral movement and loadings on the columns that are not typically considered, or recognized, when describing GRCSE behaviour based on numerical analysis that ignored installation effects.
Field case study — global-scale mechanisms

When designing a GRCSE, there are a number of limit states conditions to be considered. For the North Dynon embankment considered here, the ultimate axial capacity of the individual columns, and the column group capacity, was investigated by King (2017) based on dynamic load test results and these results indicate sufficient single, and group, column capacity. Despite this, the observed vertical settlement of the gabion wall was between 10 and 40 mm post-construction (average 24 mm) over a period of about 1 year post-construction. The larger values of settlement correspond with sections of gabion wall that are highest (eastern end in Fig. 3). This post-construction settlement is explained by the post-construction development of arching described in detail by King et al. (2017a). Normalized settlement is plotted relative to arching development, quantified by the stress reduction ratio (subsoil stress / overburden pressure), in Fig. 13 and the agreement is excellent. At the completion of embankment construction, the stress reduction ratio was about 0.7 and this decreased to a value of 0.1 as maximum arching conditions developed at about day 600 onwards. This is consistent with the column load increasing from 164 to 340 kN for the unit cell in area No. 2 described in King et al. (2017a); a 110% increase. This highlights the effect that the post-
construction development of arching can have on the total embankment settlement. This result is not entirely unexpected. The lateral deformation of the embankment also shows similar time-dependent post-construction behaviour; this is however, more difficult to explain and is examined over the remainder of the paper.

Lateral sliding, overall stability and column group extent limit states (Fig. 14) may cause lateral deformation of a GRCSE. For this case study, the column group extent is satisfied by ensuring the limit states relating to the soil reinforced gabion wall construction (i.e., reinforcement failure or pull-out, analysis of connections, and allowable eccentricity) are met (see EBGEO (German Geotechnical Society 2011) for a full list of limit states relating to soil-reinforced walls). The present study focuses on the assessment of overall stability and lateral sliding mechanisms.

**Long-term embankment behaviour**

The long-term lateral movement of the embankment is assessed through tiltmeter and inclinometer data as well as post-construction survey data. The magnitude of lateral displacement (days 182–894) is indicated with survey vectors in Fig. 15 and shows between 10 and 25 mm of movement. The survey markers are located at mid-height of the gabion wall, which varies from 5 to 6.5 m high (survey markers 1–4) and between 3 and 4 m (survey markers 5–10). All of the 10 survey markers show consistent behaviour and indicate that the entire length of the gabion wall, and by inference a portion of the embankment itself, has moved in a southerly direction with larger movement generally occurring near the southeast end where the wall height is greatest. A large component of lateral deformation is in the longitudinal direction and this cannot be explained by (transverse) rotation of the soil-reinforced gabion wall, as is often the case. Furthermore, lateral movement along the length of the gabion wall is not consistent with embankment behaviour caused by the overall stability or column group extent limit states, as no significant wall rotation has been observed and the lateral movement is occurring out-of-plane (i.e., it is not occurring in the direction normal to the gabion wall).

The long-term lateral movement (days 161–741) at the base of gabion wall, and through the subsurface profile, is shown in Fig. 16 based on the data from inclinometer No. 2 (offset 1.6 m from the gabion wall). The inclinometer data indicate about 10 mm of outward transverse movement and 5 mm of longitudinal movement (at R.L. = 2 m), a result of 11 mm movement in an approximate southerly direction that is consistent with survey data (survey marker 7) at this location. The uniform lateral movement at the base and mid-height of the gabion wall suggests a lateral sliding mechanism, not wall rotation, is primarily responsible for the post-construction movement.

In addition to the outward lateral movement at the base of the LTP (R.L. = 2 m), there is also considerable lateral movement...
within the Coode Island Silt in an inward direction; the opposite to the outward transverse movement observed during DDC installation (see Figs. 10 and 11) that is due to radial equilibration of the installed columns. The lateral sliding and column equilibration mechanisms occur in an approximately equal direction with a neutral axis in the transverse direction at about R.L. = −1 m.

The long-term tiltmeter measurements (Fig. 17) show the tilt in the outer row DDC C15, supporting the gabion wall in area No. 2 (see Fig. 9). The transverse axis of the two tiltmeters shows long-term outward rotation consistent with the inclinometer profile (Fig. 16). Between days 161 and 741, the rotation of tiltmeter No. 2 and in the Y-axis direction was −0.36° and −0.32°, respectively. Assuming the DDC column and head rotate uniformly about a neutral axis at R.L. = −1 m, the lateral displacement at R.L. = 2 m (base of LTP) is 17 mm based on the average of the two tiltmeter readings. In the longitudinal direction, the angle of tilt measured by the inclinometer above R.L. = 0 m is just +0.02° (72 arcseconds).

The rotation of tiltmeter Nos. 2 and 3 in the X-axis direction is +6 and +79 arcseconds, respectively, at a resolution of ±9 arcseconds. On this basis, it is inferred that the column head, LTP, and gabion wall are translating laterally with minimal rotation in the longitudinal direction. Due to the complex nature of the long-term displacement profile of the column, it is difficult to back-calculate a deflected profile based on discrete rotational measurements (tiltmeter data). Despite this, the agreement among the inclinometer, tiltmeters, and survey data are considered good.

In Fig. 18, the long-term lateral movement of the gabion wall is shown along with the excess pwp. Both displacement and pwp were normalized with respect to the maximum lateral displacement and maximum excess pwp, respectively. Lateral movement of the embankment stabilized by around day 400, which was consistent with the general dissipation of the excess pwp. While the radial equilibration of the columns was governed by the dissipa-

---

**Table 2. Long-term inclinometer No. 2 readings.**

<table>
<thead>
<tr>
<th>Ground improvement stage</th>
<th>Date</th>
<th>Days elapsed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline</td>
<td>13 March 2013</td>
<td>−51</td>
</tr>
<tr>
<td>Stage 1 works (installation)</td>
<td>14–27 March (four readings)</td>
<td>−50 to −37</td>
</tr>
<tr>
<td>Stage 1 works (equilibration)</td>
<td>3 April</td>
<td>−30</td>
</tr>
<tr>
<td></td>
<td>17 May</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>21 May</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>28 May</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>4 June</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>20 June</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>25 June</td>
<td>53</td>
</tr>
<tr>
<td></td>
<td>5 July</td>
<td>63</td>
</tr>
<tr>
<td>Stage 3a works (installation)</td>
<td>5 to 14 July (no readings)</td>
<td>63–71</td>
</tr>
<tr>
<td>Stage 3a works (equilibration)</td>
<td>23 July</td>
<td>81</td>
</tr>
<tr>
<td>Stage 3b works (installation)</td>
<td>11 to 17 September (no readings)</td>
<td>131–137</td>
</tr>
<tr>
<td>Post-construction (equilibration)</td>
<td>10 October</td>
<td>160</td>
</tr>
<tr>
<td></td>
<td>14 May 2015</td>
<td>741</td>
</tr>
</tbody>
</table>

**Fig. 12. Long-term data: inclinometer No. 2.**

![Graph showing long-term movement and pwp](image-url)
tion of excess pwp, it is shown here that the lateral sliding mechanism was also influenced by the dissipation of excess pwp. Based on the inclinometer, tiltmeter, and survey data, the long-term lateral movement of the GRCSE can be described by

1. Outward lateral (block) sliding of the embankment due to the out-of-balance active earth pressure force acting on the gabion wall; the plane of sliding is, however, not sharply defined, but represents a broad shear zone in the fill and upper portion of the Coode Island Silt, and
2. Inward lateral movement in the Coode Island Silt associated with radial equilibration of the numerous installed DDCs.

Both of these mechanisms act in approximately opposite directions, and are related to the dissipation of excess pwp beneath the embankment due to column installation effects. This was measured directly through piezometer data, observed indirectly as heave–uplift, and predicted based on cylindrical cavity expansion theory. The large build-up of excess pwp beneath the GRCSE present at the completion of embankment construction is expected to have greatly reduced the effective stresses (in particular vertical effective stress) in the upper portion of the Coode Island Silt, and as a result, reduced the lateral resistance provided by the Coode Island Silt. This is inferred to have aided the lateral sliding mechanism and is consistent with the lateral sliding largely ceasing as excess pwp in the upper portion of the Coode Island Silt dissipated.
by about day 400. Dissipation of excess pwp in the middle of the Coode Island Silt is expected to have taken much longer, which is consistent with the tiltmeter data that indicate that the rotation of the columns did not stabilize until about day 700 as the equilibration of the columns continued — about 2 years after the completion of the ground improvement works.

It follows that the critical period with respect to embankment stability was the end of construction when the embankment was at full height and excess pwp due to ground improvement works had not dissipated greatly. This is particularly the case where a GRCSE has been constructed quickly, which is a benefit frequently cited to justify its use in the first instance. Due to the reduction in effective stress in the upper portion of the Coode Island Silt, the lateral sliding mechanisms must be resisted by the flexural capacity of the DDCs and the passive resistance provided by the Coode Island Silt and fill unit around the periphery of the embankment. The combined effect of outward lateral sliding and inward equilibration subjects the columns to considerably greater internal stresses than would otherwise be expected if installation effects were ignored.

**Structural response of drilled displacement columns**

As instruments to measure bending moments directly are not currently available, it is common to calculate the bending moment indirectly from curvature and material properties as follows:

\[ M = \psi EI \]

where \( M \), \( \psi \), \( E \), and \( I \) are the bending moment, curvature, modulus of elasticity, and moment of inertia of the column, respectively. Where the bending moment exceeds the cracking bending moment, \( M_{cr} \), cracking of the column will occur and as a result the cross-sectional area is reduced. The effective moment of inertia, \( I_e \), for a cracked section is calculated as follows (Branson 1977):

\[ I_e = \left( \frac{M_{cr}}{M} \right)^3 I_g + \left[ 1 - \left( \frac{M_{cr}}{M} \right)^3 \right] I_g \leq I_g \]

---

**Fig. 17.** Long-term tiltmeter data.

**Fig. 18.** Lateral displacement of embankment and pwp dissipation during the post-construction period.
where $I_g$ and $I_{gr}$ are the gross moment of inertia and moment of inertia of a cracked transformed section, respectively.

The flexural failure of an unreinforced concrete column is a brittle failure mechanism and the use of eqs. (2) and (3) is not strictly valid, and for this reason the calculated crack depth should be considered as approximate only and is likely a lower bound. For inclinometer-derived lateral displacement profiles, curvature can be calculated using eq. (4); however, typically $dz/dw$ (where $z$ and $w$ are the curvilinear abscissa along the column or pile and displacement, respectively) is assumed to be close to zero and the approximate expression (eq. (5)) is adopted. Curvature can be calculated directly from a lateral displacement profile using eq. (5); however, whilst the inclinometer profile with depth may appear “smooth”, it comprises discrete inclination readings with depth. As a result, the direct use of eq. (5) generally leads to erratic and unrealistic results. To calculate the bending moment profile more accurately, it is necessary to fit a continuous curve to the displacement profile (i.e., curve-fitting techniques are required) (Ooi and Ramsey 2003).

$$
\psi = \frac{d^2w}{dz^2} - \frac{1}{1 + \left(\frac{d\psi}{dz}\right)^2}^{1/2}
$$

$$
\psi = \frac{d^2w}{dz^2}
$$

Ooi and Ramsey (2003) compared 12 curve-fitting methods applied to 60 sets of inclinometer data and concluded that a piecewise cubic curve fitting over a five-point window generally resulted in the best estimate of back-calculated bending moment profiles. The piecewise cubic curve-fitting approach fits a cubic polynomial $w = A_2z^2 + B_2z + C_2 + D_2$ (where $A_2$, $B_2$, $C_2$, and $D_2$ are constants) over a moving window of adjacent data points. In the Supplementary Data section, the five-point data window recommended by Ooi and Ramsey (2003) is shown along with larger data windows (9, 11, and 13 points) and higher-order polynomial approximations (Fig. S10) based on the lateral displacement profile in Fig. 16. Using the average of multiple piecewise cubic functions at a data point, the curvature can be calculated explicitly from eq. (6).

$$
\psi \approx \frac{d^2w}{dz^2} = 6A_2 + 2B_2
$$

The use of larger data windows results in increased “smoothing” of the datasets. Engineering judgement is required to obtain an appropriate balance between a profile that is “smoothed” and localized behaviour. The higher order polynomials generally provide a good fit of the displacement profile; however, the bending moment profiles show erratic and unpredictable behaviour at the top and bottom and are not utilized here; Ooi and Ramsey (2003) made a similar recommendation. The 15-point data window is adopted herein as the preferred method based on analysis of the post-construction lateral displacement profile (Fig. 16) presented in the Supplementary Data section.

The post-construction survey of the embankment indicated that the outward deformation of the embankment was approximately within the allowable limits that satisfy a zero tensile strain (Fig. S9) (i.e., about 20 mm), if it is assumed that the columns supporting the GRCSE rotate uniformly outwards due to lateral sliding alone (this is shown in detail in the Supplementary Data Section). A zero tensile strain condition occurs where the tensile stress induced in the column due to flexure is less than, or equal to, the axial (compressive) stress ($N$) (see Fig. S9). However, the bending moments induced in the DDCs are subjected to the combined effects of lateral sliding and equilibration acting in (approximately) the opposite direction (Fig. 16) and when these actions are considered, not only are tensile stresses induced in the column, but cracking of the column occurs. Due to the rotational restraint provided by the DDC head, and to a lesser extent the fill unit, as well as the rotational restraint at the bottom of the column due to the socket into the Fishermens Bend Silt founding unit, flexural cracking is predicted beneath the column head ($-R.L. = -1$ m), above the location of maximum lateral deflection ($-R.L. = -4$ m) and above the column socket ($-R.L. = -9$ m). Post-construction surveys does little to reveal the additional loadings due to radial equilibration occurring below the ground surface.

The behaviour of DDC C15 has, however, been subject to additional phases of ground improvement works, which occurred after the column was installed, in addition to the post-construction lateral sliding and equilibration. In Fig. 19a the behaviour of DDC C15 during the construction phase is shown (installation to day 161); this includes about 50 mm of lateral movement due to the stage 3a and 3b works. Between R.L. $= -6$ and $-9$ m, $M_{cr}$ is exceeded. The long-term condition (installation to day 741) is shown in Fig. 19b and includes the additional displacement due to lateral sliding and equilibration. The long-term behaviour of the DDC shows flexural cracking at (i) the upper portion of the shaft (due to the rotational rigidity of the DDC head) and (ii) at mid depth (due to the combined action of lateral sliding and equilibration).

Effects on drilled displacement columns during installation

Installation effects arising as columns are installed in a dense grid is a problem that affects ground improvement works where semi-rigid inclusions are installed. These effects are not well understood. The installation layout plan adopted for the North Dynon embankment was a square array of columns on a grid size varying from 2 to 2.5 m. A “hit-1 miss-1” approach was adopted for column installation (Fig. S11) similar to that described by Plomteux and Porbaha (2004). There are two problems that arise: (i) where adjacent columns are installed in immediate succession and the concrete in the previously installed column has not set, the resulting lateral displacement acts to “squeeze” the previously installed “wet” concrete column, resulting in a loss of cross-sectional area (i.e., column “necking”) and (ii) where a column is installed adjacent to a partially or fully cured concrete column the previously installed column is subjected to lateral displacement, which imposes bending moments and shear forces.

Necking of drilled displacement columns

To assess column necking, the lateral displacement profiles due to the installation of DDC C15 (inclinometer No. 2 — phase 3 in Fig. 10 and Table 1) and DDC D10 (inclinometer No. 1 — phase 3 in Fig. 5 and Table 1) installed at radial offsets of 2.04 and 2.41 m, respectively, are considered. The measured maximum lateral displacement resulting from the installation of these columns was about 20 and 10 mm for C15 and D10, respectively (Fig. 20a). Lateral displacements of between 40 and 60 mm were measured following the installation of DDCs at offsets of 1.71 and 1.12 m, respectively. If displacement profiles of this magnitude are imposed on a DDC, prior to the column setting, lateral translation of the liquid column and simultaneous compression of the cross-sectional area, due to the imposed lateral stresses, can be expected. Soil arching will have a beneficial effect, re-distributing lateral stresses around a liquid column and reducing the lateral displacement in a manner analogous to compression of a tunnel subjected to vertical loading. However, relative displacement between the column interface and the surrounding soil is required to mobilize soil arching (Iglesia et al. 2013). For a 2 m square array of columns (2.8 m diagonal spacing), the worst-case scenario is the installation of the four diagonally adjacent columns (each at 2.8 m) and an increase in all-round radial stresses on the slurry column. The
sum of free-field displacements in each direction (about 5 mm) results in a reduction in diameter of 10 mm that is less than 5% of the column diameter; this ignores the development of hoop stresses in the soil around the periphery of the column. A hit-1 miss-1 basis is reasonable and this is supported by field observations. However, at closer spacings, say 1.5 m centre-to-centre (2.1 m diagonal spacing), where free-field displacements were measured to be about 20 mm for an isolated column, it may be necessary to adopt a hit-1 miss-2 approach to increase the spacing between successive column installations to reduce the potential for loss of cross-sectional area in the column.

Column installation stresses

The bending moment profiles calculated from the lateral displacement profiles due to the installation of DDCs C15 and D10 are shown in Figs. 20b and 20c, respectively, where these are calculated based on a zero axial stress condition during ground improvement works (column self-weight is ignored). The column is assumed to move laterally with the soil mass; the lateral displacement measured by the inclinometer is therefore considered representative of the column lateral movement. The column is assessed to crack to a varying extent between about R.L. = −1 and −5 m; however, whether this cracking developed in the field is dependent on the extent to which the concrete has hardened and the ductility of the early-set concrete. For a column situated within a 2 m by 2 m array, bending moments will develop cumulatively due to the installation of adjacent columns. Vector diagrams showing the displacement of an element of soil due to the installation of DDCs is presented in Figs. 11 and S61 for inclinometer Nos. 1 and 2, respectively, and provide an indication of the amount of lateral displacement that the soil mass (and by inference a column) undergoes due to the installation of the surrounding columns. In addition, lateral displacement of >20 mm and nearly 10 mm were measured due to stage 3a and 3b work (Fig. 12), with a measurable change in column tilt (Fig. 17).

A typical plot of time-dependent concrete strength (Gilbert and Mickleborough 1990) indicates that concrete develops around a third of its strength after 3 days, and 75% after 7 days. Given the rate of concrete strength development, the time generally taken to complete multiple passes of installation as part of a ground improvement works program and the considerable amount of lateral displacement imposed on a column due to the installation of columns, it would seem inevitable that cracking of unreinforced columns will occur to some extent. The authors consider it is highly unlikely that any practically viable, and theoretically sound, ground improvement approach could be employed to mitigate these detrimental effects associated with column integrity during installation. Whilst the development of flexural cracking is not likely to impact the ability of these columns to carry vertical load in their intended manner, the analysis presented does however raise questions about the ability of the installed DDCs to provide lateral resistance to embankment lateral loading and to resist lateral sliding and global instability.

Discussion

Implications on the numerical modelling of GRCSEs under serviceability behaviour

For the North Dynon embankment case study considered here, as with many other GRCSEs, numerical methods are needed to assess the vertical and horizontal deformation of the embankment under serviceability conditions. This is particularly impor-
tant for high embankments where lateral deformation under serviceability conditions may govern the acceptability of the proposed design. However, the traditional FEM techniques that are often utilized by design engineers are limited in their ability to explicitly model column installation effects and radial equilibration, yet alone group effects and in-turn the affect that these have on the performance of the other columns and the embankment itself. These group effects are seldom modelled explicitly as part of routine design, if at all. These mechanisms include

1. Lateral displacement due to the installation of multiple columns in a dense array leading to induced internal forces and bending moments; the potential for column cracking is extremely high, if not inevitable, in unreinforced columnar elements.

2. The build-up of excess pwp beneath a GRCSE due to ground improvement, a reduction in effective stresses (in particular the vertical effective stress, \( \sigma'_v \)), and by inference, enhanced lateral sliding due to the reduction in lateral resistance. This will further induce internal forces and bending moments in the columns supporting the embankment; the potential for column cracking is high.

3. Radial equilibration of the columns, occurring as a group effect after ground improvement works. This acts in a direction opposite to lateral sliding and greatly increases the internal column forces and bending moments; the potential for column cracking is extremely high.

These three mechanisms are all related to, and caused by, installation effects. The inability to assess this behaviour in GRCSEs is not due to the lack of rigor in the structural assessment, but the failure to explicitly model the geotechnical mechanisms involved due largely to the current limitations in the numerical techniques readily adopted in practice. The cavity expansion theories offer an analytical tool, or framework, to describe the governing mechanisms involved, to interpret and evaluate these effects, and by extension, a tool to evaluate group effects. However, the use of cylindrical cavity expansion theory to examine both excess pwp and lateral deformation, as presented here, has limitations. At best, it provides a first-order assessment of the behaviour of installed columns. A solution to the problem of column (or pile) installation effects, and the group effects affecting a GRCSE, is extremely complex. Advances in mesh-less methods (material point method, smooth particle hydrodynamics, discrete element method, etc.) as well as other advanced numerical techniques, such as the CEL, have enabled researchers to simulate the installation of an isolated DDC (Qiu et al. 2011; Pucker and Grabe 2012; Busch et al. 2013) with mixed success. However, the extension of these methods to model three-dimensional group effects on the scale of a typical GRCSE requires considerable computational power and further research to accurately model the coupled hydromechanical response of soft soils, which is the unit of most interest when assessing the installation effects.

Despite the numerous limitations of traditional FEM techniques (and advanced numerical techniques at this time) when modeling serviceability behaviour of a GRCSE numerically, solutions to these problems are required in practice. Recognition of the role that installation effects play in the performance of a GRCSE is an important starting point, and from that, an understanding of the need for “work-around” methods, at this time, to assess serviceability behaviour within a traditional FEM framework. Given the difficulties in explicitly modelling these mechanisms, and the many unknowns that cannot be directly accounted for numerically, a risk-based approach, which reflects this uncertainty, is therefore warranted.
This rationale largely underpins the design approach of modeling DDCs as “geotechnical” elements as described by Gniel and Haberfield (2015) and Wong and Muttuvel (2012). This approach models the columns using plate elements with reduced bending stiffness to simulate the vertical stiffness and the loss of lateral resistance due to column cracking. The lateral spreading of the embankment is assessed by progressively reducing the plate element bending stiffness and assessing the dependency of the embankment deformation to the bending stiffness provided by the columns. The combined effects of lost bending and lateral shear capacity of a cracked column can be simulated, if required, by embedding the plate element in a thin finite element strip with a width of about 2/3 of the column diameter and material properties similar to that of cracked concrete. Typically, values of about 2% uncracked moment capacity are adopted; a reasonable level of engineering judgement is required to assess the reliance on bending stiffness relative to the potential for column cracking. This is one such approach, which is time-consuming but does provide a more rational approach to dealing with installation effects associated with column integrity and behaviour. However, even with this approach, the build-up in excess pwp and the corresponding reduction in lateral resistance are not explicitly accounted for.

By comparison, numerous studies (Liu et al. 2007; Jenck et al. 2009; Ariyarathne et al. 2012; Zhang et al. 2013; Bhasi and Rajagopal 2015) have modelled the construction and post-construction GRCSE performance with DDCs as “wished into place” structural elements (either a plate element or linear-elastic material), with gross cross-sectional properties (i.e., uncracked material properties) and have numerically simulated embankment construction starting with the LTP construction. This approach ignores installation effects, equilibration, and the potential for columnar cracking, and as a result overestimates the lateral resistance provided by the installed columns. It is difficult to conceive a scenario where the ground improvement for a typical GRCSE could be undertaken and a dense array of unreinforced columns installed without inducing considerable shear stresses and bending moments, and to a varying extent, cracking of the columns due to the ground improvement works alone. This does not include the effects of radial equilibration or any form of loading applied by the embankment itself. Serviceability limit state design principles typically adopt partial factors equal to unity for actions, soil parameters, and resistances at working loads. However, analysis of field data presented herein suggests that the basis for using gross cross-sectional column properties, without strength or stiffness reduction, at working loads (i.e., for serviceability assessment) is poorly supported by experimental observation and has a weak theoretical basis.

An alternative approach suggested by a number of authors (Mase et al. 2004; Chatte and Lauzon 2011) is to design the unreinforced DDCs as “structural” elements satisfying a zero tensile strain condition to limit the internal stresses and bending moments. A zero tensile strain condition is invalidated, in nearly all cases, during ground improvement works prior to the embankment construction even beginning, and due to the geotechnical mechanisms that are not explicitly modelled. Despite the findings of the numerical analysis, there can be little confidence that the as-built columns satisfy this zero-tensile strain condition.

A column-supported embankment or piled embankment?

It is the authors’ experience that referring to these ground improvement elements as “piles” or as a “piled embankment” frequently leads to unnecessary confusion in design scenarios. Furthermore, this belies the design intent of a GRCSE, and that is as a ground improvement option where unreinforced semi-rigid columns are installed and modelled numerically as “geotechnical elements”, which reflects the risk of columns cracking. From a design perspective there is (presumably) a higher level of redundancy than a piled structure, and as a result less onerous pile testing requirements are appropriate. This is the situation for most GRCSEs; as such, the term column, or alternatively the more general term “semi-rigid inclusion”, should be used.

There are of course certain design scenarios where the geosynthetic reinforced embankment is required to be supported on piles. The piles may comprise driven pre-cast concrete piles, or DDCs with a reinforcement cage installed for example, that have large shear and flexural capacity and should be modelled as “structural elements” that satisfy traditional pile design requirements. Regardless of the piling technique adopted, the nomenclature should reflect the design intent of either a column-supported or pile-supported geosynthetic reinforced embankment.

“Ground improvement effect” and the load transfer platform

The role that subsoil settlement plays in the development of soil arching and on the performance of the LTP was assessed by King et al. (2017a) for the case study considered here and in King et al. (2017b) where serviceability behaviour is considered in greater detail. At this time, there still remains considerable uncertainty in assessing the time rate of subsoil settlement beneath the LTP due to (i) uncertainty in the applied load (due to soil arching) acting on the subsoil and (ii) ground improvement affecting subsoil behaviour. The ground improvement effect includes the radial consolidation of soft soil surrounding individual columns, the build-up of excess pwp, and the settlement interaction between columns and the soft soil. However, a great number of researchers have sought to address this first item — soil arching — and considerable progress has been made. At this time, it would seem apparent that the difficulties in accurately describing the ground improvement effects represent a greater barrier to fully understanding LTP behaviour.

Summary of findings

A number of findings have been outlined based on the assessment of the case study considered herein

- Total settlement of a GRCSE is affected by the development of maximum arching (see King et al. 2017a) due to the increase in load acting on the column. In the vast majority of embankments, this occurs post-construction.
- For the typical centre-to-centre spacing adopted for a GRCSE, the stress, displacement, and pwp fields arising due to installation effects will overlap; installation effects influence the behaviour of a GRCSE as a group effect.
- Excess pwp will develop due to the embankment load distribution in the LTP. However, in most cases, the build-up of excess pwp due to ground improvement works will be considerably greater.
- The cumulative lateral displacement due to the ground improvement works can be expected to impose considerable lateral, shear, and bending stresses on previously installed columns. Due to the slender unreinforced nature of the columns typically adopted for GRCSEs, these bending moments will cause cracking of the columns during the installation phase. It is unlikely that a feasible ground improvement program could be implemented to avoid these detrimental effects.
- The dissipation of this excess pwp (equilibration phase) leads to an inward movement of the soil mass beneath the embankment and imposes additional lateral–shear stresses and bending moments on the supporting columns. The assumption that the columns supporting a GRCSE will form an outward cantilever shape during the post-construction phase is an oversimplification of the true behaviour and ignores this equilibration behaviour.
- Numerical analysis used to predict the imposed bending moments in columns supporting a GRCSE cannot be validated by post-construction survey data alone.
- The rapid construction of most GRCSEs limits the time available for the dissipation of excess pwp (due to ground improve-
significant quantity of data has been presented highlighting the


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