Design of Ground Improvement using Continuous Flight Auger Columns for Railway Embankment

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ABSTRACT
The upgrade of the Glenfield Junction, in south-west Sydney, Australia, involves the construction of new viaducts and associated approach embankments of up to 7.6m height over soft to firm alluvium deposits. It is a challenge to construct new tracks with existing live tracks nearby within tight timeline satisfying both stringent railway geometrical tolerance and performance. Ground treatment by semi-rigid inclusion technique involving the installation of Continuous Flight Auger (CFA) columns, in conjunction with Load Transfer Mat was adopted.
This paper presents the analytical and numerical finite element analyses adopted in assessing the behaviour of the embankment prior and after ground treatment works. It emphasizes on the challenges in predicting the settlements of the embankment and the various numerical modelling approaches to assess the performance of ground treatment by CFA columns both longitudinally and transversely. It is demonstrated that the CFA columns can be treated as vertical reinforcement within a composite 'ground structure' having equivalent improved strength and deformation properties.

RÉSUMÉ
La mise à niveau de Glenfield Junction dans le sud-ouest de Sydney, Australie, implique la construction sur un sol composé de dépôts d'alluvion compressible de consistance variable de mou à ferme, de nouveaux viaducs et de remblais d'approche de 7.6m de hauteur. Il est délicat de construire de nouvelles voies autour et dans la très proche proximité de voies ferrées en opération en respectant á la fois les règles strictes de tolérance géométrique et des objectifs de performance. Le traitement du sol par technique d'inclusion semi-rigide impliquant l'installation de colonnes CFA avec un tapis de transfert de chargement a été adopté comme la méthode de choix.
Cet article décris et présente la méthode ainsi que les résultats d'analyse obtenu notamment par l'utilisation du logiciel éléments finis sur le comportement des remblais avant et après les travaux de traitement de sol. Cet article met l'accent sur la difficulté d'anticiper le degré de tassement des remblais ainsi que sur les diverses techniques d'analyse numériques mesurant l'efficacité du traitement des sols par colonnes CFA à la fois longitudinalement et transversalement.

1 INTRODUCTION

The South West Rail Link (SWRL) is a response to issues of reliability and passenger growth on the metropolitan rail network. The project is being delivered by Transport Construction Authority (TCA) on behalf of the NSW Government. The Glenfield Transport Interchange (GTI) Component of the project is being delivered by the Glenfield Junction Alliance (GJA), led by TCA in partnership with Parsons Brinckerhoff, MacMahon Contractors, Bouygues Travaux Publics and MVM Rail. Construction of a new The Up East Hills Line (UEHL) at Glenfield Junction forms part of GTI works.

The section of the UEHL in question extends over a length of about 1km between station 32+029km and a41+833km[1] (refer to the site plan in Figure 1). It is to be constructed on an embankment rising from the existing ground level to about 7.6m at the Northern Flyover, which is being constructed to separate the UEHL from the Main South Line (MSL). The UEHL requires the construction of three retaining walls to accommodate the UEHL as it nears the Northern Flyover due to the narrow corridor between the existing MSL and the proposed South Sydney Freight Line (SSFL). The eastern retaining wall (RW03) runs parallel to the MSL over a length of about 70m with the maximum height of 7.5m to a minimum height of 5.0m. The western wall (RW05) is located adjacent to the SSFL, extending over a length of about 190m, with a maximum height of 7.6m to a minimum height of 1.3m. Wall RW04 connects at right angles to both RW03 and RW05, spanning over a width of 12.25m.

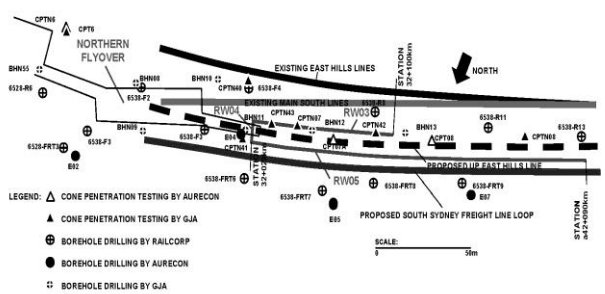

Figure 1. Site plan

The UEHL site is generally characterised by 'Alluvial Terraces' (Qpn), 'Ashfield Shale' (Rwa), 'Modern Alluvium' (Qha) and 'Hawkesbury Sandstone' (Rh). Alluvial Terraces in this region are identified as Pleistocene and Holocene age sediments including sand, silt and clay of highly variable thicknesses and consistency. The depth to the top of rock is about 7m below ground surface. However there is a risk of excessive settlements and instability between 32+029km and 32+070km due to the presence of uncontrolled fill and layers of soft to firm clay

[1] "a" stands for adjusted kilometrage adjustment, at 32+190.845km a 8869.155m is added to become a41+060km.

up to 2.1m thick within the alluvial deposits. Ground improvement using unreinforced Continuous Flight Auger (CFA) columns in conjunction with a load transfer mattress (LTM) were adopted to improve stability and reduce settlements. Reinforced Soil retaining walls (RSW) were specified because of their inherent flexibility and proven performance on poor founding materials.

2 SITE GEOTECHNICAL CHARATERISTICS

2.1 Geotechnical Investigations

The design was based on the site geotechnical investigations carried at the following stages:

- Project planning phase (2001 – 2002), by RailCorp;
- Concept design phase (2008 – 2009), by Aurecon Australia;
- Detailed design phase (2009 – 2010), by GJA.

Overall, 39 boreholes were drilled and five test pits together with 10 cone penetration tests (CPTs) were performed near and along the alignment of the UEHL. Those boreholes were terminated at various depths, ranging from about 2m to 17m below the existing ground surface. The completed geotechnical tests (refer to Figure 1 for the test locations) are summarised in Table 1.

Table 1. Summary of Geotechnical Tests

Test Type	RailCorp	Aurecon	GJA
Borehole	18	12	9
Cone Penetration Test	-	3	7
Test Pit	-	-	5

2.2 Overview of Site Geotechnical Conditions

In general, the subsurface profile comprises fill, alluvium, and/or residual soil, and weathered rock.

Fill of up to 3.1m thick was observed from EHL 32.029km to EHL a41.833km. Alluvium is present immediately beneath the fill from EHL 32.029km to EHL a41.110km and extends to a depth of up to 4m below the existing ground surface. Residual soil underlies the fill and alluvium, where present. The thickness of the residual unit is 0.4m and 1m below the alluvium between EHL 32.029km to EHL 32.040km, and between 32.090km to 32.110km, respectively. Between EHL a41.100km to EHL a41.833km the residual unit extends to a depth of up to 4m immediately below the fill. Weathered rock was encountered to underlie alluvial/residual soil at depth range from 0.7m to 7.5m below the existing ground surface at the borehole locations. Rock typically comprises weathered shale over sandstone.

A soft to firm alluvial layer is of particular interest and concern in terms of settlement potential and was encountered in variable thicknesses (0.2m to 2.1m) from EHL 32.029km to EHL a41.230km. The most significant presence of this soft to firm layer (2.1m thick) within the footprint was encountered in CPTN07 at EHL 32.055km.

Figure 2 shows the typical subsurface conditions beneath the approach ramp. It was concluded that the alluvium and the identified soft to firm layer, in combination with the non-engineered fill across the site, would result in excessive total settlements and settlement gradients.

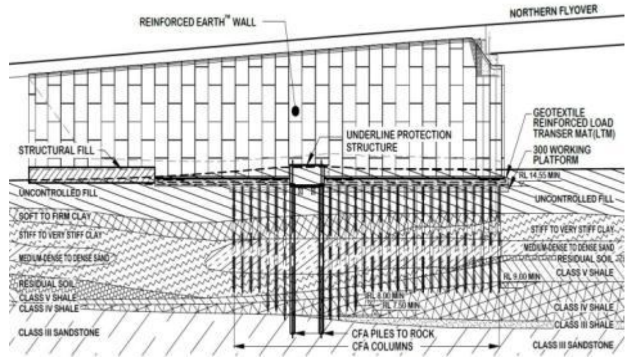

Figure 2. Surface profile and ground improvement

Groundwater was encountered within alluvial and residual materials in a number of boreholes of the exploratory locations and the measured groundwater levels varied from 1.3m to 3.2m below ground.

3 DESIGN METHODOLOGY

3.1 Design Constraints and Design Criteria

The proposed embankment will be constructed over existing uncontrolled fill and natural soils of highly variable consistency/density to provide a railway corridor leading up to the viaduct. The presence of uncontrolled fill and soft to firm clay layers with the natural soils would cause excessive vertical and lateral ground displacements, not only potentially affecting the railway track performance (both existing and proposed railways) but also imposing additional loads on the piles supporting the viaduct structure for the bridge approach area. The stability and serviceability of the railway embankment, together with the impact of the embankment construction on adjacent existing and proposed structures (i.e. railway lines, overhead wiring structures, drainage systems and pile foundations), represent constraints in geotechnical design.

The presence of the underline crossing ULX12 traversing almost perpendicular to the rail alignment at approximately EHL 32.060km has been identified and requires structural protection from loading imposed by the proposed railway embankment and to prevent differential settlement from occurring. A concrete slab spanning over the ULX supported on a piled foundation is adopted to carry the embankment loading. The differential settlement and future performance of rail structure around the protection structure needs to be considered in the design.

The design life set by RailCorp for the new railway embankment is 100 years. However, reconditioning of the railway ballast will be required for a period of 20 years, the following stipulated design settlement criteria:

- Tolerable maximum long-term settlement is limited to 50mm over 20 years post construction.
- Tolerable maximum change in grade anywhere in both longitudinal and transverse direction of the railway embankment is limited to 1%.
- Negligible differential settlements within bridge approach areas, such as those immediately east of the Northern Flyover Viaduct structure.

The other constraint is, based on construction plan, the project designers had only four months to complete their design to satisfy the above settlement requirements whilst ensuring there was no significant impact on the existing railway lines, structures and train operations.

3.2 Design Concept

3.2.1 Ground Treatment Options

Ground treatment by preloading was excluded from the beginning of the design due the constraint of construction plan. Various other ground treatment solutions were investigated during the detailed design stage as part of value management and engineering procedure, including the following techniques:

- Vibro-dynamic replacement involving the installation of stone columns in the ground;
- Use of displacement columns;
- Deep Soil Mixing (DSM) technique;
- Installation of driven timber piles;
- Adoption of concrete columns in the form of Continuous Flight Auger (CFA) unreinforced columns; and
- Leave the ground as it is (no ground treatment works).

The preferred ground treatment option was selected based on design requirements, site conditions/restraints and economics.

Ground treatment options involving the installation of stone columns and displacement columns were discarded mainly due to the concerns over the impact of the ground treatment works on the nearby railway embankments supporting MSL and SSFL. The DSM technique (wet method) requires a relatively wide area for plant set-up and involves a relatively high mobilisation cost; hence, it was omitted due to site and economical constraints. Timber piles were not adopted as they were unable to satisfy project durability requirements. Option that involves 'do nothing (no ground treatment)' was not feasible as the project design settlement criteria could not be met. Ground treatment involving CFA columns became the method of choice as it represents the most cost effective option (i.e. lowest installation and establishment costs) and its impact on the adjacent railway embankments is considered to be negligible (Merry and Power, 2011).

3.2.2 Design Procedures

The CFA columns were designed as ground inclusions. The design should ensure the load from the embankment be effectively transferred to the columns through the soft compressible soil layer to a much stiffer underlying stratum to prevent punching of the columns through the embankment fill, causing differential settlement at the surface of the embankment. Two key elements for a successful design are column arrangements (diameter, spacing and pattern), and load transfer mat (LTM).

The methodology for geotechnical design includes assessment of short-term and long-term performance of the fill embankment under serviceability and ultimate conditions. Due to the complexity of the system and the interaction between the treatment components including CFA columns, LTM and reaction from subsoils, the geotechnical analyses were carried out in general accordance with the following steps:

- Review of subsurface ground conditions underlying the UEHL to assess whether poor, compressible soil layers are present within footprint of the proposed railway embankment. Subsurface conditions in the vicinity of the footprint were assessed taking into account the available geotechnical information to provide a global view of the ground settlement characteristics;
- Assessment of settlement for compliance with the design criteria prior to and post ground treatment works;
- Design of ground treatment, where required;
- Assessment of global stability of embankment for compliance with the stability criteria targeting at:
  - A minimum factor of safety (FoS) against temporary global slope instability of 1.3;
  - A minimum FoS of 1.5 against global instability post construction under permanent loading;
- Assessment of the adequacy of foundation material following treatment to support the proposed reinforced earth walls and to provide global stability.

4 GEOTECHNICAL DESIGN

4.1 Settlement Assessment

To assess the need of ground treatment, a preliminary assessment of settlement potential over 20 years was carried out using an EXCEL spreadsheet developed based on the one-dimensional consolidation method developed by Raymond and Wahls (1976) and Mesri and Godlewski (1977), without the ground treatment works and with shallow excavation and replacement. Considering the subject site is a floodplain area with fluctuating water levels within the alluvial and residual strata, the soils were assumed to be fully saturated in view of the design life in the one dimensional settlement calculations.

The embankment fill will be retained by RSWs (Walls 03, 04 and 05). A Reinforced Earth™ wall system was proposed with steel reinforcing strips because of the negligible creep behaviour of steel and its proven performance when subject to vibration under high dead and live loads. With this system, internal settlement due to the self weight of the embankment fill was estimated to be 0.1% of the embankment height over the 20 year design life based on previous local engineering experience. External primary settlement of the granular material and some external primary settlement of the cohesive material are expected to take place during the construction period. Therefore, the contribution of the granular material settlement has been removed from the long term settlement. In this preliminary assessment, external creep is assumed to start after completion of embankment construction and is calculated based over the 20 year design life.

The results of the preliminary settlement assessment indicated that the total settlement after completion of embankment would be up to 110mm for an embankment height of 7.5m. The settlement design criteria stated in section 3.1 cannot be met with simple excavation and replacement.

4.2 Ground Improvement by CFA Column

Pile/column-supported embankments, consisting of foundation soil, piles/columns, geosynthetics, and embankment fill, have been increasingly used for embankments. The advantages of this technology are rapid construction, small vertical and lateral deformations, and global stability and have been used in a number of applications worldwide to solve many geotechnical problems.

Ground treatment in the form of CFA columns was adopted over a 30m section of the Northern Flyover Approach Ramp as the settlement criteria cannot be met and the embankment loading would be at its maximum with the embankment between 6 and 7.5m high, and an additional 11m of CFA columns was required to transition (to meet the differential settlement criteria) from the ULX12 crossing to the remained excavation and replacement ground treatment zone; resulting in total length of 41m of CFA columns.

The CFA column was 450mm in diameter columns at 1.8m staggered spacing with the properties presented in Table 2. The preliminary CFA column pattern and spacing was estimated using conventional geotechnical practice and in accordance to British Standard "Code of practice for strengthened/reinforced soils and other fills" (BS8006, 1995). The concrete columns are designed to carry the vertical load from the embankment and train load (surcharge) without failures and have adequate factors of safety against lateral sliding and global instability under ultimate limit state load conditions. Each column is required to carry the load of the column's tributary area. Figure 3 and Figure 4 show the typical CFA column arrangement.

4.3 Load Transfer Mat

BS8006 is the British Standard method used for design of embankments with reinforced soil foundations on poor ground and it is the most widely used method. In BS8006, the arching is assumed to be semi-spherical dome and it is independent of the type and strength properties of the embankment fill. A spreadsheet was established to calculate the distributed vertical load acting on the reinforcement between the piles W_T (Figure 5) following BS 8006 code for the determination of the required basal reinforcement tensile strength. The method developed by Hewlett and Randolph (1988) was used for cross-check.

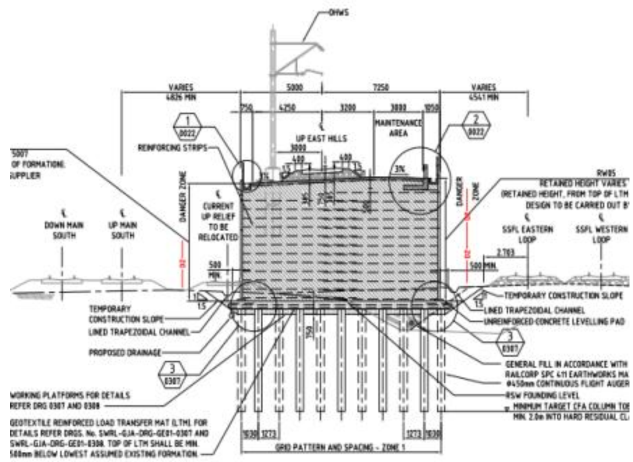

Figure 3. CFA Column Supported Embankment

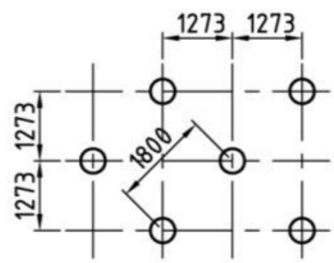

Figure 4. CFA grid pattern and spacing.

Table 2. Material properties for CFA columns and LTM

CFA Columns	LTM
Diameter: 450mm	Thickness: 450mm
Spacing: 1.8m Centre-to-centre	Material grading requirements: Max particle size is 65mm, max passing 19mm sieve is 50% and max passing 75μm sieve is 5%
Target column length: 5.5m-7m	Minimum unit weight: 20kN/m^3
Characteristic concrete strength: 10MPa at 7 days, 15MPa at 28 days	Minimum effective friction angle: 40^o
Concrete stiffness: 20GPa	Basal reinforcement requirements: Min tensile strength of 250kN/m at 6% strain in both longitudinal and transverse directions Max allowable long-term creep of 2% over 100 years Min normal stiffness 5,000kN/m

Vertical loads from the RSW ramp are transferred and distributed to the columns through a Load Transfer Mat (LTM) constructed directly onto the 300mm thick gravel working platform created for the CFA piling rig. The top of the LTM, a 450mm thick, geosynthetics reinforced granular soil layer, is founded below the base of the RSW ramp to transfer and distribute the vertical loads from the embankment to the columns. Since the reinforced soil walls are expected to behave as a rigid soil block, they are not considered to impose any significant horizontal loading on the underlying treated ground. The 300mm thick working platform will comprise of the same material as the LTM and is designed to support a maximum working pressure of 350kPa. The CFA columns are designed as ground inclusions and as such, they are not governed by the piling code and not subject to load testing.

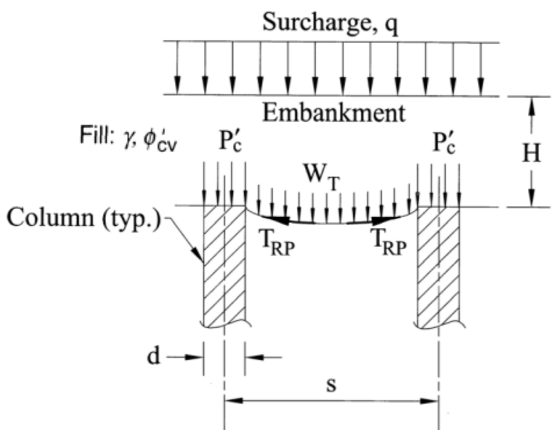

Figure 5. Variables as defined by BS 8006

4.4 Numerical Modelling

The mechanisms of load transfer in CFA supported embankments are a combination of the soil arching effect in the embankment fill, the reinforcement effect of the geosynthetics, and the load transfer between CFA columns and surrounding soils as a result of their stiffness difference.

To increase the design confidence and to provide cost efficiency, numerical modeling using the commercially available finite element analysis (FEA) software PLAXIS 2D Version 9 was undertaken to assess the adequacy of the proposed ground improvement by CFA columns and to optimise the extent of CFA columns required and column spacing.

The purpose of the FEA was also to assess settlement, bearing capacity and load distribution and the impact on adjacent existing rail lines, MSL and SSFL, respectively.

The ground models have been assessed based on available geotechnical information presented in Section 2. The FEA model in the transverse direction (i.e. perpendicular to the alignment of the Up East Hills Line) has been selected at EHL 32.050km, where the embankment is approximately at its highest (7.5m high) and in close proximity to the existing and future track lines. The FEA was also performed in longitudinal direction to assess the differential settlement potential along the alignment.

Figure 6 and Figure 7 show the finite element models set up for the transverse direction and longitudinal direction respectively.

As embankment was formed by reinforced soil walls, it can be treated as a solid block. To simplify the numerical model, the loads from self weight of embankment, railway structures and train were applied as uniformly distributed loads directly on top the LTM.

Existing live tracks has also been incorporated into the Plaxis model through construction sequence to investigate the impacts between the live tracks and construction of new embankments.

Three approaches were adopted to simulate the performance of CFA columns during and post construction for both transverse direction and longitudinal direction. These are a) plate elements (Figures 6 and 7); b) soil elements (Figure 8) and c) equivalent soil blocks (Figure 9).

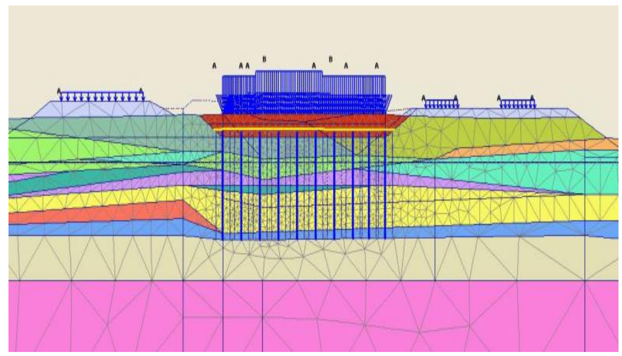

Figure 6. PLAXIS Model: Transverse Direction (Plate Elements)

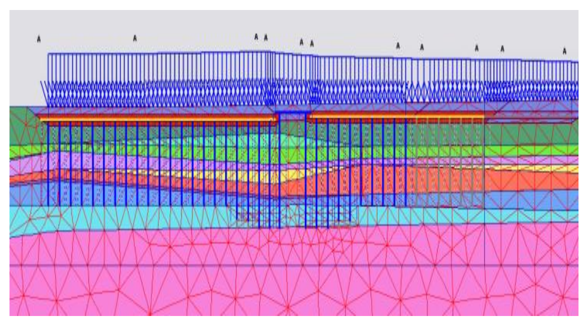

Figure 7. PLAXIS Model: Longitudinal Direction (Plate Elements)

In Plaxis numerical modelling, plate elements are generally used to model the behavior of retaining walls; therefore, it was used as key approach in the design. The equivalent axial stiffness, EA, and flexural rigidity, EI were specified according to the diameter of CFA columns and column spacing. In soil element approach, CFA columns were treated as ground inclusion elements with soil material having high stiffness equivalent to specified concrete material; while in equivalent soil block approach, the soil layers with the CFA column treated area were considered to act like an equivalent soil block, with both CFA column stiffness and soil stiffness were considered to obtain the equivalent stiffness of the mass block. Modeling approached by using soil elements and equivalent soil block were used to further validate the results obtained by modeling of plate elements.

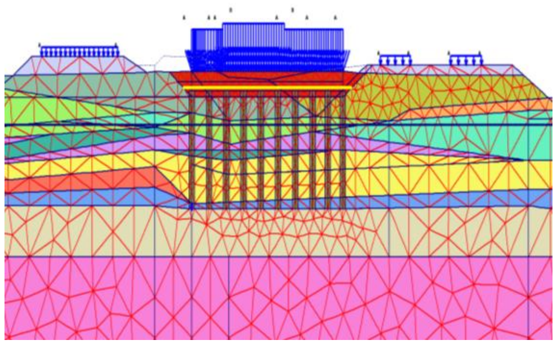

Figure 8. PLAXIS Model: Transverse Direction (Concrete Column as Soil Elements)

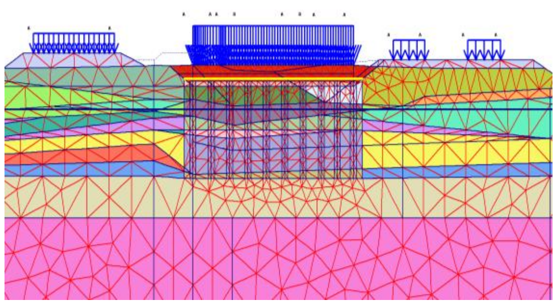

Figure 9. PLAXIS Model: Transverse Direction (Equivalent Soil Block)

The structural design of the ULX12 protection consists of 16 No. (2 rows of 8 piles) 450mm diameter reinforced CFA piles socketed 0.5m into and bearing on Sydney Class III Sandstone. A 600mm thick reinforced concrete slab will span about 3m between the 2 rows of 8 No. piles for the protection of ULX.

This structural ULX protection will provide a "hard point" that will require transition measures to meet differential settlement criteria. This has been addressed in the longitudinal FEA (Figure 7).

It is evident that the embankment and train loading of the UEHL is distributed through the LTM on to the columns and supporting soil (Figure 10). Arching between the columns is displayed. This is consistent with the transverse FEA preformed with 3 different approaches to model the ground treatment, a soil block of the improved system, structural elements of the CFA columns and soil elements for the CFA columns. All three techniques provided comparable results. Similar results were obtained from the longitudinal direction analysis.

The calculated long term total settlements within CFA zone, after ground treatment works, are summarised in Table 3. Our settlement analysis indicates that the differential settlement including the bridge approach area is within the tolerable maximum change in grade in both longitudinal and transverse direction (<1%) as set out in Section 3.1 (Figure 11).

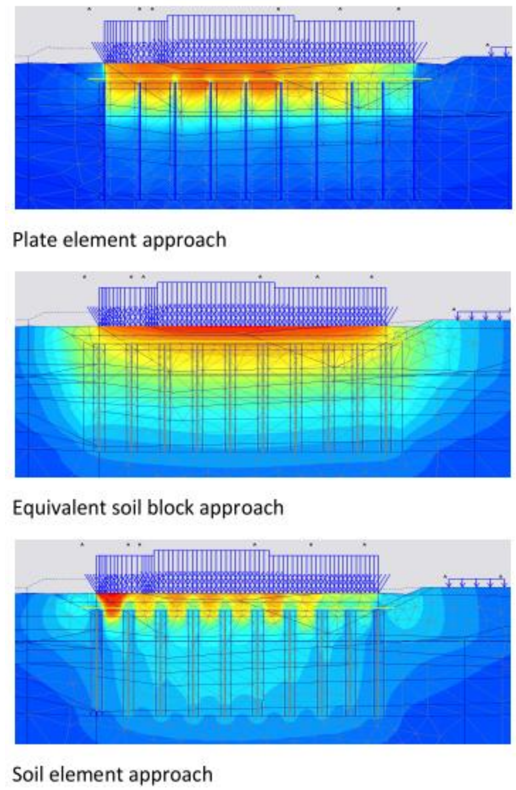

Figure 10. Post construction vertical settlements computed from three different approaches – transverse direction

Table 3: Predicted long-term settlement after ground treatment works in CFA zone

EHL Chainage (km)	Embankment Height (m)	Founding Depth (RL)	Long-term Settlement (mm)
32,029-32,050	7.5 – 6.6	9.0	10.5 – 16.6
32,050-32,052		8.0	10.9 – 11.2
32,052-32,055	6.6 – 6.2	7.5	8.7 – 11.8
32,055-32,062			7.0 – 8.3
32,062-32,064	6.2 – 5.8	8.0	6.5 – 7.0
32,064-32,070		9.0	6.0 – 7.2

Notes:
1) Long-term settlement 20 years post construction.
2) ULX protection slab at chainages between 32,052 and 32,055 approximately.

Shear failure stress behavior are considered one of the most representatives of the failure characteristics exhibited by the model. The relative shear stress is the ratio of the calculated shear stress induced in the soil by the loads to the available shear strength.

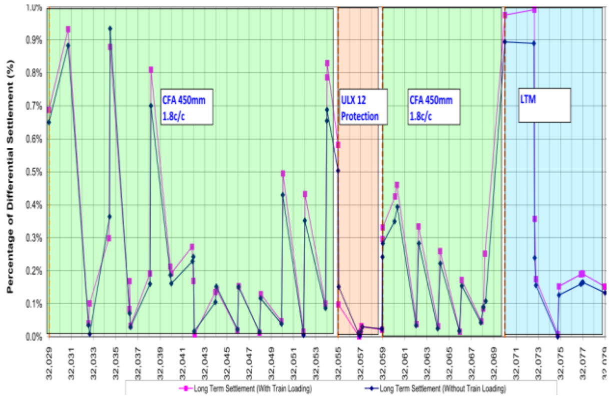

Figure 11. Differential Settlement within ground treatment zone by CFA

Figure 12 shows the plots of the relative shear stress from three different modeling approaches. It provides an indication of how much the strength of the foundation support system (CFA columns and LTM) has been mobilized following the embankment construction. The relative shear stress is shown in the figure as a variation in colour from blue (no mobilisation or no change in stress on the materials) to red (full mobilisation of the material strength). It can be seen that the strength of LTM layer is almost fully mobilized around the top and toe of plate element, indicating the functioning of CFA and LTM support system.

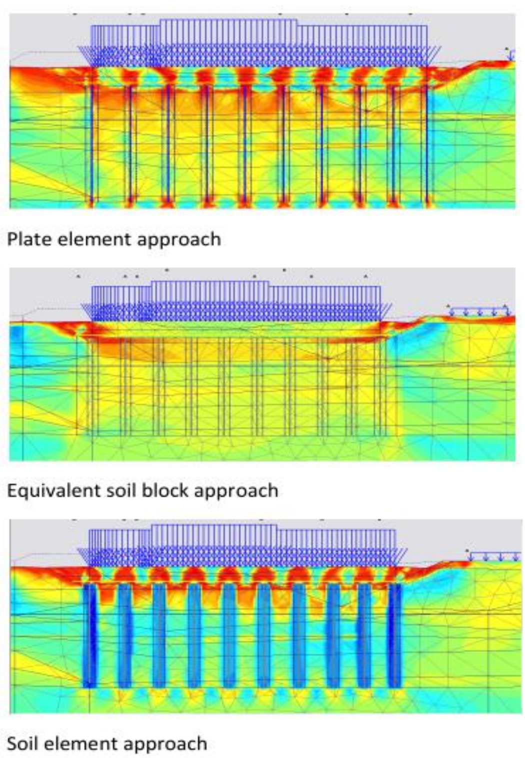

Figure 12. Relative shear stress plots

4.5 Interfacing Between Existing and Future Existing Rail Lines

The existing MSL and future SSFL (construction to be completed prior to UEHL) are adjacent to the proposed UEHL. The impacts of construction of the UEHL embankment supported on CFA columns on the existing and future adjacent rail lines, MSL and SSFL, respectively, were also investigated by FEA analysis.

The analysis was carried out using staged construction and drained parameters to assess the long term conditions. Considering the possible construction sequence, following five load cases were modeled:

1. SSFL with train loading under serviceability limit state (SLS) load conditions – settlement prior to construction of UEHL
2. SSFL, UEHL embankment with train loading under SLS load conditions – settlement due to construction of UEHL and train loading
3. SSFL with train loading, UEHL embankment under SLS load conditions – impact due to SSFL with train loading on UEHL embankment
4. SSFL with train loading, UEHL embankment with train loading under SLS load conditions – settlement due to train loading under both lines in operation
5. SSFL, UEHL embankment with train loading under ultimate limit state (ULS) load conditions

The results are summarised in Table 4.

Table 4: Predicted settlement impact on adjacent rail lines

Load Case	Maximum Settlement at the Top of Embankment (mm)	
	SSFL	MSL
1	36	0
2	2	4
3	11	3
4	11	5
5	3	5

The FEA indicates that there is no significant impact on the adjacent rail lines along the entire alignment.

4.6 Stability Assessment

Global slope stability analyses were performed, using the commercially available computer program SLOPE/W (GeoStudio 2007). The results indicated that the embankments, despite that they are under the influence of flooding, were assessed to have adequate factors of safety against global slope instability.

The reinforced soil walls, supported by CFA columns/LTM system, were designed as permanent structures with a minimum global factor of safety of 1.5 for long-term. The assessed factor of safety against global instability is considered to be adequate.

5 CONCLUSION

The geotechnical design presented here demonstrates the successful adoption of ground treatment system by CFA columns and geosythetics for timely project delivery and with constraints of live tracks close-by. The CFA column/LTM system was designed in combination with method following BS8006 code and two-dimensional finite element analysis.

It is demonstrated through FEA that the CFA columns can be treated as vertical reinforcement within a composite 'ground structure' having equivalent improved strength and deformation properties. Numerical analyses also validated the distribution of the bearing loads through the LTM partially to the CFA columns via concrete-to-ground sidewall friction and partially to the in-situ ground between the CFA columns.

Site construction of the CFA columns started late January 2011, the CFA columns and ULX protection structure are installed (Figure 13). The performance of the CFA columns /LTM system and settlement of the RSW ramp will be monitored.

Figure 13. Construction of CFA columns/LTM system under progress

6 ACKNOWLEDGEMENT

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