Design of Railway Embankment over Soft Ground

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ABSTRACT

This paper discusses the geotechnical design and performance monitoring of a 2.6 km long railway embankment constructed over soft ground at Hexham, located approximately 16 km northwest of Newcastle and 160 km north of Sydney. The rail alignment traverses over soft ground with soft soil thickness of more than 25 m on the southern end and about 12 m on the northern end of the project corridor. There were challenges in relation to the geotechnical design including (a) design of adequate formation to reduce the risk of soft subgrade failure under cyclic loading (b) excessive settlement of soft ground with time, impacting the performance of the railway tracks in the long term and (c) presence of waste material in the site, as between the 1960s and 1990s, the area was the site of a coal washery and loading facility. Hence, the site has already been preloaded with varying thickness of coal reject fill material placed in isolated areas along the proposed rail corridor, causing potential differential settlement of the proposed rail tracks. Therefore, geotechnical design of formation needs to be carried out by adopting appropriate subgrade modifications in order to reduce (a) the rail formation thickness above the existing ground surface and (b) post construction settlement. Selection of appropriate subgrade modifications requires careful consideration of number of factors including ground conditions, available construction time, performance requirements and cost.

A number of subgrade modification methods including remove and replace, rigid/semi-rigid columnar inclusions, and mass stabilisation using cement or lime have been considered to reduce the post construction settlement as well as to provide a stable formation for the railway embankment. Each modification method has its own advantages and limitations. After discussion with construction team and the Australian Rail Track Corporation (ARTC), "remove and replace" has been selected as a suitable method considering (a) relatively low cost and easy to construct; (b) reduction in primary settlement due to removal of material; and (c) significant testing regime and possible installation difficulties through coal reject fill associated with columnar/mass stabilisation methods. The assessed post construction settlement has been provided to ARTC for the development of maintenance regime. An approach of tamping the tracks to maintain the track geometry within ARTC Standards has been adopted.

The rail tracks have been constructed and in operation successfully from late 2014. Settlement monitoring has been carried out during and after construction. The observed settlements are reasonably consistent with the design prediction. In addition, dynamic deflection of the track has been monitored during the operation of tracks and compared with the design prediction. This paper summarises the geotechnical site investigation, development of design parameters, selection of subgrade modification method, formation design, and back calculation of settlement and dynamic deflection to compare with monitoring data.

1 INTRODUCTION

The Hexham Relief Roads (HRR) project, commissioned by the ARTC, involves the construction of five Relief Roads (i.e. railway tracks) located next to the existing up and down coal roads and associated infrastructure at Hexham in the NSW Hunter Valley, approximately 16 km northwest of Newcastle as shown in Figure 1. The tracks are intended to accommodate trains generally comprising two or three locomotives and up to 91 wagons requiring a minimum standing room of 1.67 km.

The proposed tracks are located on a floodplain consisting of soft soils, leading to potential significant post construction settlement. In addition, a portion of the footprint of the proposed tracks have already been preloaded by coal washery fill (i.e. existing fill) placed between the 1960s and 1990s, leading to potential differential settlement along the alignment. The magnitude of total and differential post construction settlement will dictate the number of interventions and maintenance cost during the design life of the project. Provision of adequate formation over soft subgrade is also important to reduce the risk of subgrade failure as well as to meet the dynamic deflection requirements of the tracks during operation. Hence, the assessment of adequate formation and prediction of the long term settlement become critical elements in the design of railway embankment over soft ground. This paper summarises the geotechnical investigation and interpretation, design criteria, design approach, performance monitoring and back analyses results.

Figure 1: Project location

2 GEOTECHNICAL INVETIGATION AND GEOTECHNICAL MODEL

2.1 Geotechnical Site Investigations and Laboratory Testing

The alignment of the HRR is located over the low-lying flood plain of the Hexham Swamp and comprises Quaternary Aged sediments overlying the Permian aged Tomago Coal Measures which comprise near horizontally bedded shale, mudstone, sandstone, tuff and coal rock types. An array of geotechnical investigation was carried out in the HRR area to develop the site geology and geotechnical design parameters. A number of cone penetration tests (CPT) including dissipation tests, boreholes and test pits were carried out as part of the investigation to classify the material and to assess strength profile with depth. The laboratory test data on samples retrieved from boreholes located within the HRR area have also been analysed for the derivation of compression and strength parameters. The assessment of design parameters for geotechnical design is summarised in the following sections.

2.2 Design Parameters for Geotechnical Design

Undrained Strength Parameters

The undrained shear strength of soft soils (s_u) has been assessed based on the CPT data adopting an empirical cone N_{kt} factor of 14. The adopted N_{kt} value has been assessed from the calibration of CPT data with in-situ vane shear strength results. Project corridor has been divided into a number of zones for geotechnical design purposes. As an example, a geological long section developed for a zone within a low lying area is provided in Figure 2. Strength parameters for existing fill material were assessed from direct shear test results.

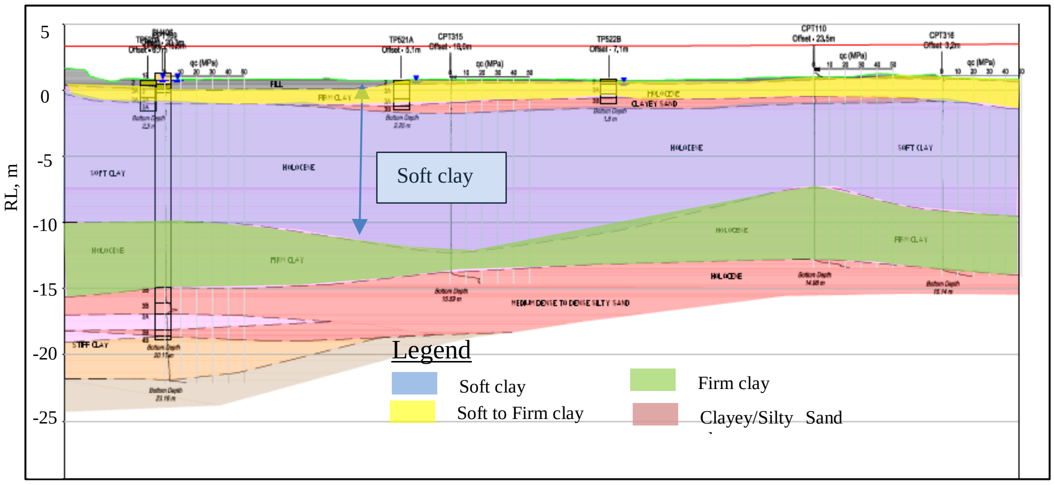


Figure 2: Geological long section developed based on site investigation data (typical section of the site)

Compressibility Parameters

The compression parameters including Compression Ratio (CR), Recompression Ratio (CRR) and Creep Strain Rate ($C_{\alpha\varepsilon}$) have been assessed based on the results of laboratory oedometer tests. The quality index (based on Lunne et al., 2006) of individual test samples have been adopted to assess the data quality and to eliminate poor quality samples from the interpretation. The creep strain rate for normally consolidated clay ($C_{\alpha\varepsilon(NC)}$) has also been assessed based on typical correlations between creep index (C_α) and compression index (C_c) proposed by Mesri and Godlewski (1977). The creep strain rate for over consolidated clay ($C_{\alpha\varepsilon(OC)}$) has been assessed using the approach proposed by Ladd (1989), modified by Wong (2007). As shown in Figure 3, the relationship has been further enhanced with the extended consolidation test results from HRR project.

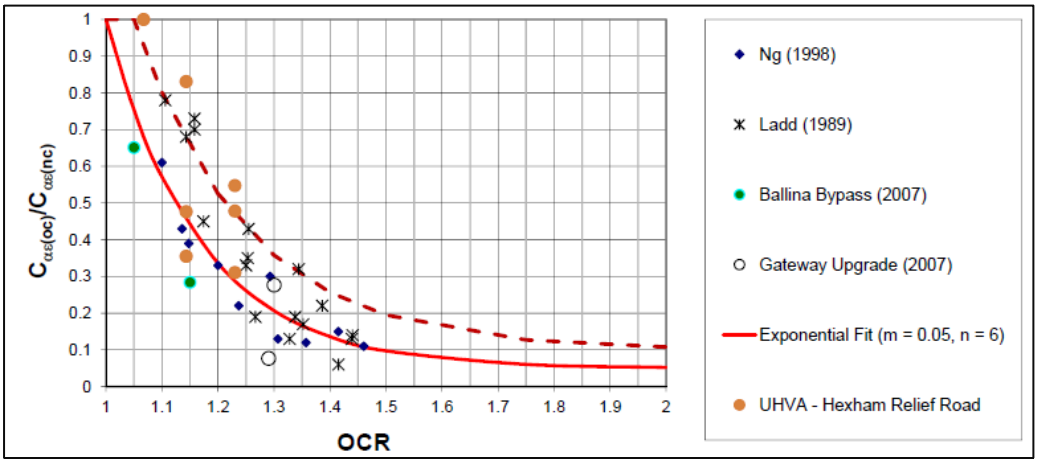


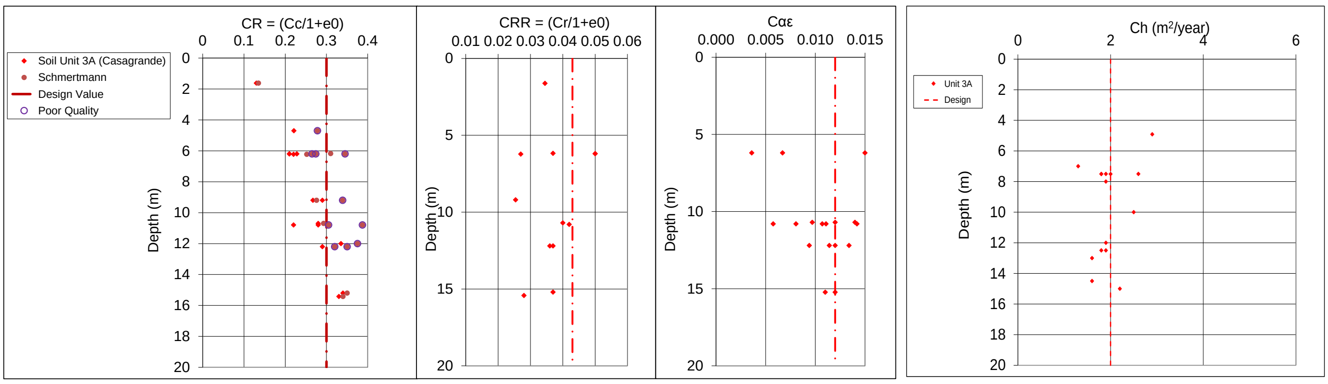
Figure 3: Creep strain ratio versus overconsolidation ratio

The Over-Consolidation Ratio (OCR) has been estimated from s_u values based on published correlations (Ladd and DeGroot, 2003) presented in Equation 1 below.

$$s_u = 0.22\sigma'_{v0} OCR^{0.85} \tag{1}$$

where, σ'_{v0} (kPa) is the in-situ vertical effective stress.

The coefficient of horizontal consolidation (C_h) values were estimated based on the pore pressure dissipation test results from the CPTu tests. As shown in Figure 4, C_h for normally consolidated soil layers 5 m below the existing ground level is approximately 2 m^2/year based on the results. For over consolidated clay, C_h of 20 m^2/year has been adopted for design. The coefficient of vertical consolidation (C_v) has been taken as half of the coefficient of horizontal consolidation (C_h) based on the depositional nature of the stratum and experience with similar stratigraphy along the East Coast of Australia.



(a) (b)

Figure 4: (a) Assessed compression parameters and (b) Assessed coefficient of horizontal consolidation

3 GEOTECHNICAL DESIGN

Geotechnical design of the railway formation is summarised in the following sections.

3.1 Formation Design

The formation design for the HRR project has been carried out adopting two main design criteria summarized below for a railway track configuration shown in Figure 5.

Step 1 - Strength Criterion

The formation or sub-ballast should be adequate to distribute the axle loadings in order to reduce the risk of subgrade failure due to cyclic wheel loadings.

Step 2 - Operational Criterion

Deflection of rail tracks under cyclic wheel loadings (i.e. dynamic deflection) should be maintained within an allowable limit to achieve operational requirements. In this step, dynamic deflection has been predicted adopting the formation thickness assessed from the Step 1 to assess whether it is within the allowable values of 3.2 mm to 6.4 mm. Where this was not satisfied, formation thickness has been increased until the dynamic deflection is reduced within the allowable value.

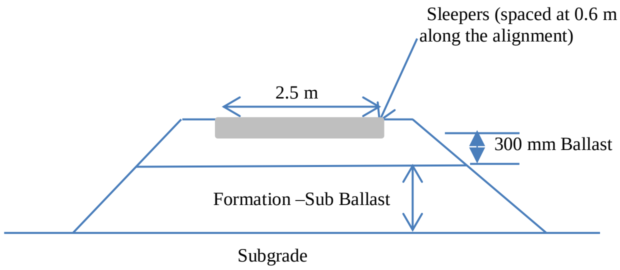


Figure 5: Components of a Rail Track - HRR Project

During the design development, a cost effective option of placing rail formation on top of the existing ground, utilising the existing fill material as part of formation, has been considered (i.e. minimum cut option). Due to the placement of coal reject fill material during the period between 1960 and 1990, the existing ground is in higher elevation than the existing up coal track. It was advised by ARTC maintenance team that the level difference between the existing up coal track and the adjacent proposed new track should be maintained at 1.2 m or less for the future maintenance purposes. Level difference between the proposed track with the “minimum cut” option and the existing up coal track is more than 1.2 m in the majority of the alignment and hence, this option has not further been explored.

After eliminating the “minimum cut” option, a number of subgrade modification methods have been explored with an intention to develop a cost effective formation design as well as to reduce the post construction settlement. Risks and opportunities of each subgrade modification method have been carefully weighed in the process of selecting an appropriate subgrade modification method. A summary of the subgrade modification methods considered during the design process and the cons and pros of each method is summarised in Table 1.

In the selection of appropriate subgrade modification method, the following factors have been taken into account:

(a) Relative cost benefits of each method (i.e. capital and maintenance cost);
(b) Risks of installation works with columnar inclusions due to obstructions within the existing fill, vibration impact on adjacent operating tracks and potential delays in construction;
(c) Rigorous testing regime and trial requirements associated with ground inclusions and cement mixing panels may increase construction period; and
(d) It was also noted columnar inclusions and cement stabilised panels still require some cut of existing ground to meet level difference between the proposed new track and the existing up coal track as well as material double handling.

Considering the above factors and risks and opportunities outlined in Table 1, it was decided by the project team to proceed with the “remove and replace” option for the detailed formation design.

Table 1: Risks and opportunities of various subgrade modification methods

Subgrade modification Method	Risks	Opportunities
Method 1: Remove and Replace	• Material removed shall need to be disposed. • Placement fill is likely to be under water. However, if the rock fill is used, this may not be an issue.	• Straight forward construction. • The material to be placed can be used as formation fill. • Provide an effective drainage for formation and no requirements for capping layer (i.e. rock fill). • Removal of existing fill/natural material and placement of the rock fill material to construct formation will reduce the post construction settlement as the amount of fill above the existing ground level is reduced. • Cheaper than Methods 2 to 4.
Method 2: Partially penetrated Concrete Injected Columns (CIC)	• Consolidation settlement due to platform fill between columns may form a mushroom effect on the surface. Appropriate structural geofabric can be placed within the formation fill to account for this effect. • After installation of CICs, working platform must be cleaned and formation must be built with structural geofabric. Spoil management is required. • Installation difficulties through the existing fill material due to potential obstructions.	• Formation thickness will be thinner than Method 1. • Consolidation and creep settlement of the railway embankment will be lower than Method 1. • Installation is quicker than Methods 3 and 4. • Required strength and stiffness parameters can be achieved with reasonable control. • Dynamic deflection will be lower than all other methods.
Method 3: Partially penetrated Stone Columns (SC)	• Consolidation settlement due to platform fill between columns may form a mushroom effect on the surface. Appropriate geofabric reinforcement can be placed within the formation fill to account for this effect. • After installation of SCs, working platform must be cleaned and formation must be built. Spoil management is required. • Additional granular material for column construction may be required to account for the increase in column diameter due to expansion of soft clay caused by vibro-compaction during installation. • Installation is slower compared to CIC.	• Formation thickness will be thinner than Method 1. • Consolidation and creep settlement of the railway embankment will be lower than Method 1. • Required strength and stiffness parameters can be achieved with reasonable control. • Dynamic deflection will be lower than Method 1.
Method 4: Partially penetrated Cement mixing panels	• Laboratory testing must be carried out to assess the dosage. It may be different to field conditions. Field trial is therefore required. • Consolidation settlement due to platform fill between panels may form a mushroom effect on the surface. Appropriate reinforcement can be placed within the formation fill to account for this effect. • After installation of panels, working platform must be cleaned and formation must be built. Spoil management is required. • Installation is slower compared to CIC.	• Formation thickness will be thinner than Method 1. • Consolidation and creep settlement of the railway embankment will be lower than Method 1. • Dynamic deflection will be lower than Method 1.

For the remove and replace option, the exposed subgrade mostly had a California Bearing Ratio (CBR) value of 1% or less, particularly within the low-lying areas. Formation configuration for a subgrade with CBR value of 1% or less is not part of the ARTC standards. Therefore, a specific formation design has been carried out adopting the design criteria summarised earlier in this chapter. Formation design has been carried out in two steps to achieve the “strength criterion” and the “operational criterion” as summarised below.

<u>Step (1): Designing for Strength Criterion</u>

Loads acting on sleepers through rails have been assessed using an approach provided in Selig and Waters (2000) for an axle configuration shown in Figure 6. A cyclic load allowance factor (F_d) computed from the relationship reported by Li and Selig (1998) has been applied to these loads to assess the cyclic loads (i.e. equivalent static wheel load) acting at each sleeper (Equation 2).

$$F_d = \left[1 + 0.0052 \frac{V}{D_w}\right] \quad (2)$$

Where, V (km/h) is the train speed and D_w (m) is the diameter of wheel. The wheel diameter was assumed to be 0.95 m.

The assessed cyclic wheel load at each sleeper have then been applied in a finite element model to assess the load distribution through the formation fill and ballast, as well as cyclic deviatoric stress induced within soft subgrade. The finite element analysis has been carried out using commercially available computer program PLAXIS 2D.

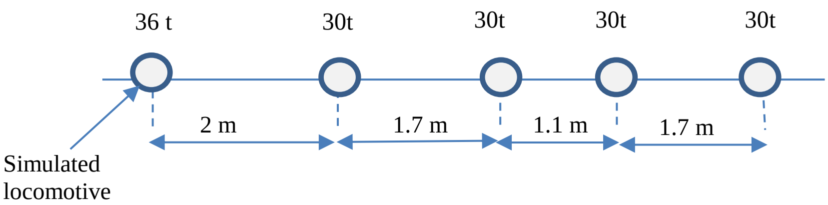


Figure 6: Axle configuration considered in formation design (as per AS5100.2)

The computed cyclic deviatoric stress has been compared to the strength of subgrade to assess the likelihood of subgrade failure. For this purpose, a term "cyclic stress ratio" has been introduced as shown in Equation 3 below.

$$CSR = \frac{Cyclic \quad Deviatoric \quad Stress}{Static \quad Deviatoric \quad Strength} \qquad (3)$$

A limiting or threshold CSR value to reduce the risk of subgrade failure has been developed using published data. Raymond (1978) summarizes some important aspects of cyclic loads and safe bearing pressure for rail formation design. Raymond presented cyclic triaxial test results on clay samples conducted by the British Rail and illustrated that clay samples do not fail under cyclic loadings when the CSR is about 0.5 or below, irrespective of number load cycles.

Shahin, *et.al.* (2011) concluded based on triaxial cyclic tests on normally consolidated clay that there is a threshold cyclic stress above which clay samples will fail, without reaching resilient status. They reported that when the CSR is less than 0.63, no sign of effective stress failure was observed. However, plastic deformation increases significantly when CSR is more than 0.56 (increases approximately from 1.8% to 4.3%). There are other studies reporting various threshold CSR values (e.g. 0.6 by Ni, *et.al*, 2012). Based on these information, a threshold CSR of 0.5 has been adopted for the formation design of HRR project. A design example is provided below.

<u>Design Example (Hypothetical case)</u>

An example of a train operating with four 30 tonne axles and a 36 tonne axle locomotive as recommended in AS5100.2 has been considered here. A train speed of 80 km/h and wheel diameter of 0.95 m has been adopted. The following design parameters for soft clay subgrade, sleeper and formation fill and design assumptions have been adopted for the example.

- A soft subgrade with 10 kPa undrained shear strength at the ground surface and linearly increasing in strength with depth at 1.2 kPa/m.
- Groundwater is at the ground surface.
- Formation fill consists of rock fill with a resilient modulus of 300 MPa (with two layers biaxial geogrid with ultimate tensile strength of 30 kN/m).
- Ballast is 300 mm in thickness with a resilient modulus of 150 MPa.
- Sleeper length is 2.5 m.
- Sleepers are spaced at 600 mm centre to centre.

The design approach described above has been applied to assess the maximum deviatoric stress within subgrade under cyclic wheel loadings. The assessed CSR with various formation fill thickness is presented in Figure 7. Based on the relationship provided in Figure 7, formation fill thickness can be selected to reduce the risk of subgrade failure. For this example, a formation thickness of 1.15 m is required if a threshold CSR of 0.5 is selected.

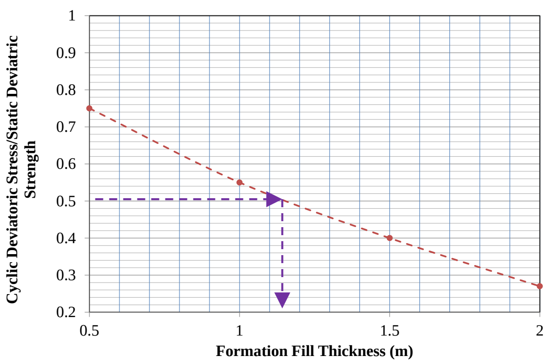


Figure 7: Variation of CSR against formation fill thickness

In addition to the above assessments, a bearing capacity assessment has been carried out based on Meyerhof (1974). The assessed factor of safety against bearing failure is approximately 2 and considered reasonable.

Step (2): Designing for Operational Criterion

The assessed formation thickness from Step (1) has then been used to assess the dynamic deflection of the track under train loads. When the deflection is more than the allowable limit, the formation thickness has been increased to reduce the dynamic deflection within the allowable deflection.

3.2 Settlement Assessment

Assessment of settlement for tracks due to primary and creep settlement of soft subgrade has been carried out such that ARTC can develop a tamping program to meet prescribed differential settlement criteria. This program was subject to revision through the future settlement monitoring. The fill thickness including formation rock fill and ballast above the existing ground level along and across the rail tracks varies due to the undulating terrain. This will cause differential settlement along and across the alignment. A one dimensional finite difference program for consolidation analysis has been used to assess the settlement with time for various zones along the alignment. The assessed settlement along the alignment has been reported for various time periods so that the tamping regime can be developed. An example of the assessed settlement along the alignment for down coal road during a period between 3 months and 9 months after commissioning of tracks is presented in Figure 8.

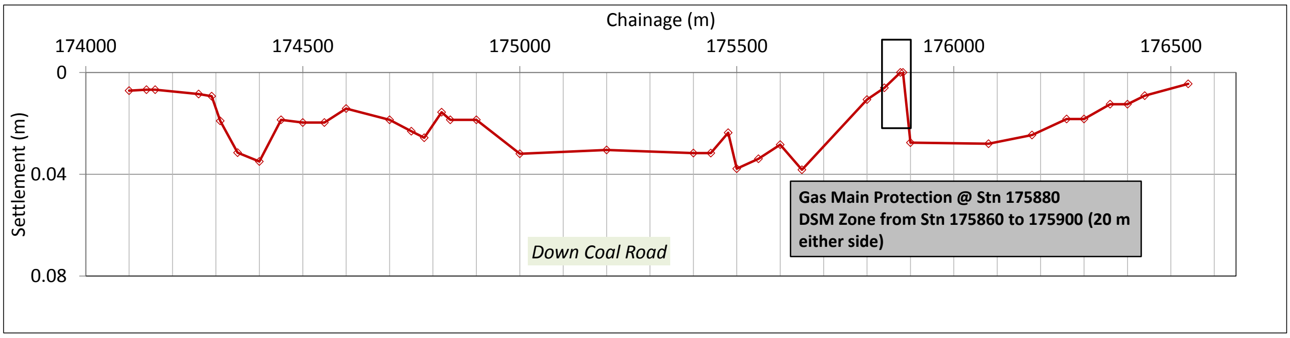


Figure 8: Assessed settlement along the alignment during a period between 3 months and 9 months after commissioning of tracks

4 PERFROMANCE OF CONSTRUCTED TRACKS

Construction of rail tracks has been completed and the tracks have been in operation from late 2014. Monitoring of tracks and embankments have been carried out during and after construction in order to compare the performance against the design prediction. The following have been carried out as part of this monitoring works:

(a) Pile Driving Monitor (PDM) survey technique has been used to record the dynamic deflection of operating tracks. This was required to assess whether the track formation is performing consistent with the predicted dynamic deflection.
(b) Inclinometer data to confirm that the lateral movement of subgrade has stabilsied.
(c) Settlement monitoring using settlement plates. This is important to assess whether settlement will need to be further revised based on the monitoring data such that ARTC can revise the prepared tamping regime.

Monitoring of dynamic deflection

The detailed description of the use of PDM in HRR project has been published elsewhere (Muttuvel and Sasiharan, 2015) and brief details associated with back analysis are provided here. The observed dynamic deflection of Relief Road 4 is presented in Figure 9.

For the initial review of dynamic deflection data against the design prediction, back analysis has been carried out for two selected locations using PLAXIS 2D. Original geotechnical design parameters adopted for the formation rock fill and soft subgrade have not been changed for the back analysis purposes. The assessed track vertical movement (i.e. dynamic deflection) for a train on down coal track with empty wagons and for a train on Relief Road 4 with fully loaded wagons are 3.3 mm and 5.3 mm, respectively. This is comparable to the observed dynamic deflection using PDM presented in Table 2 and Figure 9.

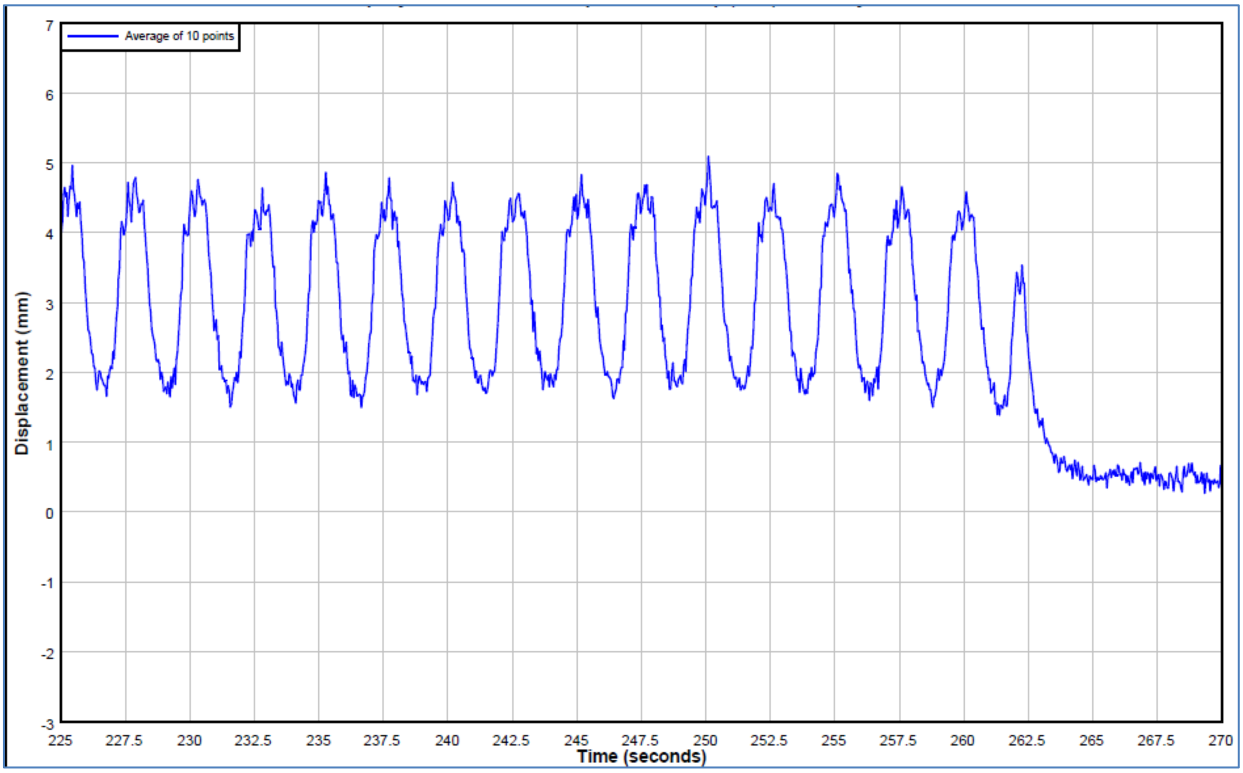


Figure 9: Observed dynamic deflection of Relief Road 4 with fully loaded wagons
– 30 Tonne axle – Train speed 25 kmph

Table 2: Measured dynamic deflection based on PDM survey

Location	Track	Train No.	No of Locos	No of Wagons	Measured Maximum Deflection when a "single train passes" (mm)[1]	Back analysed deflection (mm)
176,000km (Plain track)	Up Coal (RR4)	XRN004/XRN027/ XRN005	3	96	5.0	5.3
176,500km (Turnout)	Down Coal	5032/5008	2	88	4.0	3.3

[1] "Single train passes" means that no trains are in operation on adjacent tracks during the time of measurements.

The observed inclinometer data indicates that the lateral deformation has stabilised and are only 3 mm to 5 mm in magnitude. Therefore, it was evident that progressive failure of subgrade has not occurred. Settlement monitoring data has been compared against the predicted values. For an example, the observed settlement within a zone has been plotted in Figure 10. For back analysis purposes, the original design parameters adopted in the design has not been changed. As presented in Figure 10, the predicted settlement is similar to the observed settlement before a significant change in the observed settlement was recorded between September and December 2014. No apparent reasons such as construction activity or change in benchmark have been identified for the cause of such increase in settlement. In addition, similar observations have been made at some other settlement plate locations. Therefore, this sudden increase in settlement is mostly due to the internal compression of the rock fill induced due to commencement of train operations. After December 2014, it appears that the rate of settlement is of similar order of magnitude to that predicted as shown in Figure 10. However, more readings are required before concluding the predicted settlement values are comparable to the observed values.

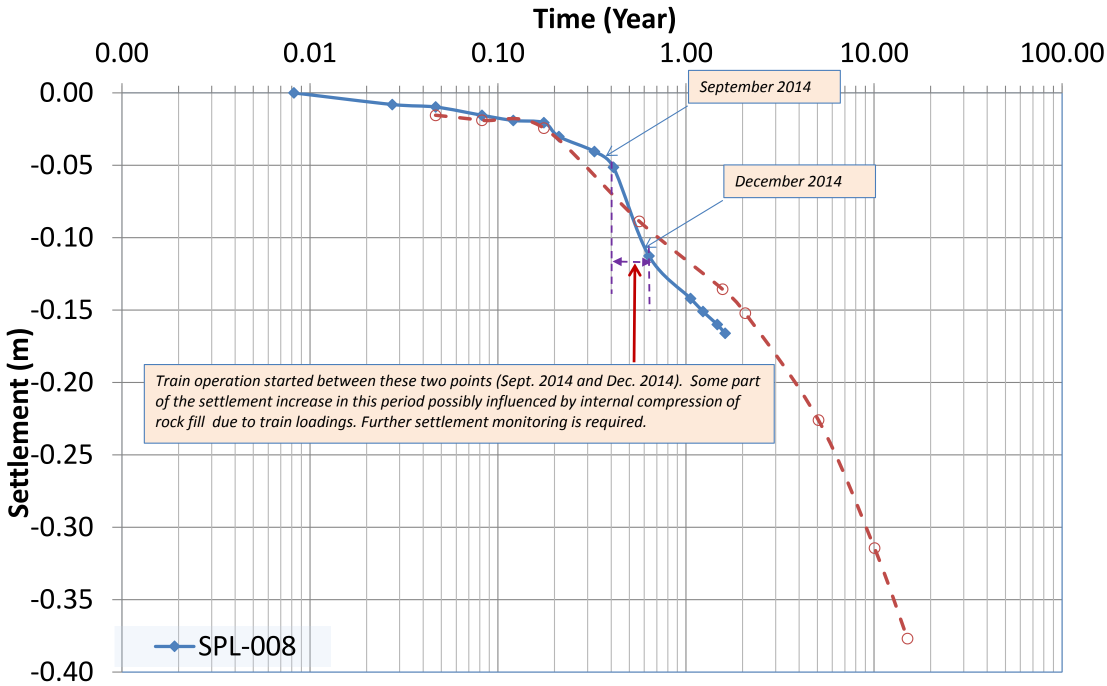


Figure 10: Example settlement curve for a high settling zone (Approx. Chainage 174,410)

5 CONCLUSIONS

The design of formation requires consideration of a number of key aspects including construction and maintenance cost, long term settlement, operational requirements such as dynamic deflection and dissipation of dynamic load through granular layer to reduce the risk of subgrade failure. A step by step design procedure and a design example have been presented to outline two important aspects of the formation design. In addition, this formation design procedure has been successfully applied on the Hexham Relief Road project as summarised in this paper.

The design predictions have been compared with observations gathered during construction and operation of Hexham Relief Roads. The monitored dynamic deflection using a technique of PDM indicated that the formation has been adequate to achieve the required operational performance criteria. The inclinometer data indicates that the lateral movement stabilised after rail operations. Considering (a) similar order of observed dynamic deflection to the prediction and (b) no progressive movement of the embankment toe, it can be concluded that the intended purpose of the formation is achieved. The back analysis results based on the settlement data gathered over 1.5 years indicate that the predicted rate of settlement appears to be in line with the prediction. The rail tracks have been allowed to settle over time with an appropriate maintenance tamping regime, without compromising operations.

6 ACKNOWLEDGEMENT

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