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A mechanistic formation design in cohesive soil based on Li and Selig Method

Un dimensionnement mécanistique pour la couche de forme et couche de fondation des voies de chemin de fer dans des sols argileux inspirer de la méthode développée par Li & Selig.

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ABSTRACT: Rail formation design is conventionally based on an empirical approach, relying on subgrade CBR or subgrade soil types. Some formation design methods are based on subgrade soil strength, such as threshold stress or bearing capacity. A more recent and robust method developed by Li et al. (2016) considered both subgrade strength and cumulative plastic strain under cyclic train loading. This paper presents the procedure, challenges and formation design results in the application of this method for the Inland Rail Project, one of Australia's largest freight railway projects. Three-dimensional Plaxis modelling was undertaken to analyse deviatoric stress in formation and subgrade due to train loading. Allowable deviatoric stress was adopted based on laboratory and field testing. A methodology to interpret Li and Selig *a*, *b* and *m* parameters from cyclic load triaxial tests has been developed and project-specific design parameters were selected on several soil types (including lime treated subgrade). Changes in unsaturated cohesive subgrade moisture content due to flooding and infiltration were assessed using Seep/W. A relationship between the change in undrained shear strength and variation in moisture was established to determine long-term design parameters in subgrade. The Li and Selig method was used to optimise ground treatment design (removal of existing weak subgrade and its replacement with structural fill or lime stabilised fill) to meet the project design specifications, including axle load, speed, tonnage, design life, and maximum plastic deformation over the 50 year design life. The results of this paper indicate that mechanistic formation design gives client and rail operator the opportunity to select the performance desired while allowing a balance between capital expenditure and rail maintenance operations through the design life.

RÉSUMÉ: La conception des couches de fondation et couches de forme pour des voie ferrées est classiquement basée sur une approche empirique. D'autres méthodes sont basées sur la résistance des sols de fondation, comme la contrainte de seuil ou la capacité portante. Une méthode plus récente et plus robuste développée par Li et al. (2016) considère à la fois la résistance de la plate-forme ferroviaire et la déformation plastique cumulative sous les charges cycliques ferroviaire. Cet article présente les procédures, les défis et les résultats de la conception de la formation dans l'application de cette méthode dans le cadre du projet Inland Rail, l'un des plus grands projets de chemin de fer de fret en Australie. Une modélisation tridimensionnelle aux éléments finis (Plaxis 3D) a été entreprise pour analyser les contraintes dans les sous-couches et couches de forme due à la charge du train. La contrainte admissible a été adoptée sur la base d'essais en laboratoire et sur le terrain. Une méthode pour interpréter les paramètres a, b and m de Li et Selig est présenté ainsi que des paramètres de conception spécifiques au projet sélectionnés à partir d'essais triaxiaux à charge cyclique sur plusieurs types de sol (y compris la couche de forme traitée à la chaux). Les changements dans la teneur en humidité de la fondation cohésive non saturée en raison des inondations et des infiltrations ont été évalués à l'aide de Seep/W. Une relation entre la variation de la résistance au cisaillement non drainée et la variation de l'humidité a été établie pour déterminer des paramètres de conception à long terme des sols de fondation. La méthode a été utilisée pour optimiser les besoins en terrassement et éliminer couche de fondation en stabilisant la couche de forme à la chaux tout en répondant aux spécifications du projet comprenant la charge à l'essieu, la vitesse, le tonnage, la durée de vie et la déformation plastique maximale tolérée. Les résultats de cet article indiquent qu'un dimensionnement mécaniste des couches de fondation et couches de forme donne au client et à l'opérateur ferroviaire la possibilité de sélectionner les performances souhaitées tout en permettant un équilibre entre les dépenses en capital et le cout des opérations de maintenance ferroviaire tout au long de la durée de vie d'un réseau ferré.

KEYWORDS: Rail formation design, deviatoric stress; cyclic load triaxial test; lime stabilised clays; unsaturated soil.

1 INTRODUCTION

Rail formation traditionally considers California Bearing Ratio (CBR) and its empirical relationship to developed rail formation design. However, it does not consider the rail traffic, such as the number of trains and tonnage movement throughout the design life.

In cohesive soil equilibrium, suction and moisture change do vary throughout the design life (often 50years for national infrastructure projects) and affect the performance of the rail.

Often a CBR soaked for a set period is specified to provide a minimum performance under "wetter" conditions. Repeatability of CBR, particularly in cohesive soil, is low (Breytenbach et al. 2010), especially where CBR values lie between 1 and 3. Volumetric, strength and stiffness changes with moisture change, drive the unsaturated cohesive soil behaviour. Earthwork specification requirements based on CBR can also reduce construction efficiencies as it can take several weeks or more to obtain results in remote locations due to the sample preparation, test procedures, and resource constraints.

Alternative mechanistic formation design methods have been developed such as Li and Selig method (Li et al., 2016) which considers both subgrade strength and plastic deformation combined with design traffic tonnage, axle load and speed.

The two formation design criteria from the Li and Selig method are summarised as below (Eq. 1 and Eq. 2):

$$\text{Criterion 1: } \sigma_d \leq \sigma_{da} \tag{1}$$

$$\text{Criterion 2: } \rho \leq \rho_a \tag{2}$$

where σ_d = applied deviatoric stress due to train loads at a subgrade surface; σ_{da} = allowable deviatoric stress; ρ = a cumulative plastic deformation; and ρ_a = allowable cumulative plastic deformation.

Criterion 1 purpose is to prevent progressive shear failure, whereas Criterion 2 limits plastic deformation. The acceptable plastic deformation can be established depending on the project requirements such as design, budget, operation, and maintenance. A design procedure for rail formation in cohesive soil is presented in this paper. It follows an approach similar to that of Li and Selig with the following four significant improvements:

- Stress distribution within the rail formation modelled using three-dimensional finite element analysis that allows the integration of non-homogenous subgrade, structural fill properties and alternative design.
- Cyclic load triaxial testing to develop material parameters for the numerical modelling and formation performance calculation,
- The incorporation of unsaturated soil behaviour (change in strength associated with time dependent moisture change) based on unsaturated laboratory testing and analysis.
- A relationship between undrained shear strength and moisture content established specific to the project and incorporated in the design methodology.

On the Australian Rail Track Corporation (ARTC) Inland Rail Program, a formation performance defined as a cumulative formation deformation throughout the design life is required by the ARTC Inland Rail Basis of Design (BoD). This paper presents two rail formation designs developed for the Narrabri to NorthStar section of the Inland Rail program. One is based on structural fill with a minimum strength and stiffness requirement with depth governing the depth of excavation and replacement required. The second is based on a lime stabilised treatment of cohesive soil won from the site to provide a "structural bridging" layer and distribute stress to the firm to stiff existing subgrade.

2 METHODOLOGY / DESIGN PROCEDURES

2.1 *Step 1: Ground model and failure mechanism*

As the method under discussion uses numerical models to establish stress and strains within the formation and subgrade (such as fill or natural ground in low height embankment or cuts), it is essential to establish a ground model and associated input geotechnical parameters.

Several publications have stated that the most common track failure caused by repetitive stresses in the formation and natural subgrade is either excessive plastic deformation (Figure 1a) or progressive shear failure (Figure 1b).

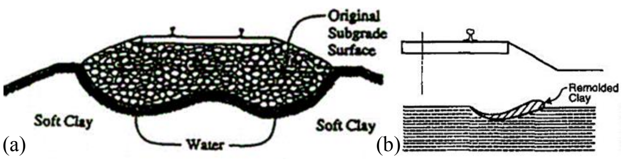

Figure 1. Subgrade failure modes (Li et al., 2016).

Figure 1(a) shows a w-shaped feature, evidence of excess cumulative displacement and progressive shear failure mechanisms. In cohesive soil, the ground model needs to consider the impact of existing moisture variations and moisture change following construction and throughout the design life. The new moisture equilibrium in the long term (30 to 40years after construction) is likely to be wetter than at the time of construction, particularly where the formation and subgrade are comprised of fine-grained soils even in arid and semi-arid environments. The authors noted such behaviour at two brownfield construction projects of the Inland Rail (Parkes to Narromine and Narrabri to North Star in New South Wales), and two rail formation investigation sites (near Tom Price in Pilbara – Western Australia and near Cloncurry – far north west Queensland).

Geotechnical Investigations need to be carefully planned to collect data informing the design process and give due consideration for seasonal, climate conditions and drainage.

In cohesive soil for low height embankment or brownfield formation/ renewal, geotechnical testing to establish existing moisture condition distribution with depth (suction and moisture content) within the existing formation subgrade zone of influence is recommended. Undisturbed sampling, field and laboratory testing may include but not be limited to suction testing at close vertical spacing (0.25m) combined with undrained shear strength, moisture content and specific gravity. The zone of influence of rail load varies depending on the materials. A rule of thumb of 5m (approximately 2 times sleeper length) from the top of the designed rail level is suggested.

The performance of fill material is equally important within the zone of influence of a rail load. Reconstructed samples of anticipated fill material are recommended to increase the level of confidence in the fill analysis parameters.

2.2 *Step 2 Establishing design parameters*

The allowable deviatoric stress σ_{da} is based on the threshold stress, which is the maximum deviatoric stress at which the strain accumulation rate under cyclic load is constant (i.e. stable strain accumulation). This is generally taken as half the unconfined compressive strength for cohesive soil and can be obtained from in situ field testing and laboratory testing on undisturbed and reconstructed samples of the proposed fill material.

The undrained shear strength is moisture-dependent and varies with weather pattern, seasonal change, rain, and flood event as time passes. In-situ undrained shear strength can be established using vane shear testing and triaxial tests. It is critical to capture moisture content and suction when measuring the undrained shear strength. Change in moisture with time can be calculated using transient analysis in commercial software such as Seep/W or Vadose/W with input from the unsaturated soil laboratory testing (suction tests, soil water retention curves, specific gravity, measurement of air entry value close to saturation).

It is recommended that cyclic load triaxial tests on (1) undisturbed samples within the zone of influence of the rail load and (2) fill material (structural and stabilised) using samples at representative compaction, and density anticipated, be undertaken for establishing unconfined compressive strength, stiffness parameters including resilient modulus (M_r) used in the numerical modelling and other parameters (a, b and m) used in the cumulative plastic deformation calculation.

2.3 *Step 3 Establishing the applied deviatoric stress*

The applied deviatoric stress due to train loads σ_d can be calculated using three-dimensional Finite Element Analysis (3D FEA). It allows the modelling of non-homogenous formation design and alternatives to structural fill material traditionally recommended.

The selection of the constitutive model is critical and driven by soil behaviour. As detailed by Yang et al. (2019), a Mohr-Coulomb elastic-plastic constitutive model is preferred over elastic model. In 3D FEA using Plaxis, the Undrained-C method is considered more suitable for modelling cohesive subgrade under transient dynamic train loading to assess the applied deviatoric stress. Another benefit of using this method is that the undrained shear strength input can be measured in the field. In this method the Resilient modulus is input as stiffness. Effects of dynamic train loading is modelled as adding a dynamic load factor to the design axle load.

2.4 *Step 4: Design criterion 1*

The deviatoric stress σ_d due to the train load at the subgrade surface must be lower than the allowable deviatoric stress σ_{da} for each layer of the formation zone of influence.

$$\text{Criterion 1: } \sigma_d \leq \sigma_{da} \tag{3}$$

If the criterion is not met, change the geotechnical treatment until it is met. Treatment to improve stress distribution may include thicker layers of formation material, or different material or the inclusion of geogrid to achieve a better distribution of applied stresses within the zone of influence of rail load. Once Criteria 1 is satisfied, proceed to Criteria 2.

Consideration for existing moisture conditions and change in undrained shear strength (and hence σ_{da}) with anticipated change moisture conditions (and suction) over the design life should be given. The time for a change in moisture is difficult to establish with certainty, given the complexity and variables involved. Seep/W and Vadose/W can be used to model a range of change in moisture with time associated with flood and rain events.

The results can be used in combination with the undrained strength to moisture relationship to allow for a strength reduction overtime throughout the design life. A material-specific undrained strength to moisture relationship can be established using a series of field and laboratory testing.

2.5 *Step 5: Design criterion 2*

Design Criterion 2 is a calculation of cumulative plastic deformation (rut depth) within the formation (at the underside of ballast) for a number of loadings (traffic tonnage, which includes the number of trains, train type, and axle load and life cycle as a function of time). This can be obtained from the Concept of Operation (or similar) from the asset owner/operator.

The calculated cumulative plastic deformation through the design life need to be less than the allowable plastic deformation. The allowable cumulative plastic deformation can be established by or in collaboration with asset owner considering asset and maintenance requirements combined with capital expenditure cost. Where a higher cumulative plastic deformation is selected by the client it is recommended a level of engineering judgement is applied to cumulative plastic strains.

The formation of cumulative plastic deformation is related to the cumulative plastic strains (ε_p) and deformation (ρ), as represented in EQ 2-3 (Li et al., 2016).

$$\varepsilon_p = a \left(\frac{\sigma_d}{\sigma_s}\right)^m N^b \tag{4}$$

$$\rho = \int_0^T \varepsilon_p \, dt \tag{5}$$

where ε_p = the cumulative plastic strains; N = the number of repeated stress applications during the design life; σ_d = soil deviatoric stress; σ_s is soil compressive strength; ρ is cumulative plastic deformation; t is layer thickness; T is zone of influence thickness; and a, b and m = cyclic load test parameters.

2.6 *Step 6: earthworks specification and compliance testing that directly corelate to the design parameters assumed in the analysis*

As the method relies on strength and resilient modulus, particularly in cohesive soil, it is critical that the earthworks specifications and associated testing regime reflect the design intent and assumptions to ensure that the ground conditions encountered in the field, including the engineering fill, are as expected and in accordance with the assumption forming the basis of the design.

Measurement of strength and stiffness for existing conditions or embankment fill/ formation material (including compaction) in-situ as construction progresses is recommended. Traditional methods of assessing existing subgrade or foundation level (prior filling) may involve using the Dynamic Cone Penetration Test (DCP), Vane shear testing, or plate load testing. These methods have limitations and are either crude, time-consuming or unpractical for mixed material such as gravelly clay, placing constraints to construction program and methodology.

Fill material performance and compaction are traditionally validated using characterisation testing based on grading Atterberg limits and soaked CBR combined with nuclear densometer and compaction curves. CBR and relative density can take time associated with the preparation of the sample for these tests. This is exacerbated for variable material.

Recent progress has seen the development of alternative devices for measuring strength and stiffness, such as Panda, and Light Weight Deflectometer (LWD), which allow direct results from the field, thus saving time during construction. Trials are recommended to calibrate readings to project-specific ground conditions and compacted fill material, but these devices significantly reduce the need for time-consuming laboratory testing, reporting and management of non-compliant results. Similarly, these benefits also apply to engineered fill compaction test results such as nuclear densometer traditionally used. Furthermore, these alternative devices can be easily combined with intelligent compaction. These in-field tests can also be automated and georeferenced to achieve near-live reporting of results using the GIS platform.

2.7 *Implication of rail traffic speed*

Work by Yang et al (2009) indicates that for speeds up to 30km/hr the dynamic loading effect is minimal. From 30km/hr to 80km/hr, the dynamic loading effect needs to be considered (as is for this method using AREMA dynamic load factor). For speeds higher than 80km/hr further study is recommended to assess the effect of resonance on the stresses and strains in the formation and subgrade.

3 APPLICATION FOR N2NS

3.1 *ARTC Inland Rail formation design requirements*

Per ARTC Inland Rail Basis of Design, design life 50 years, design Tonne Axle Loading (TAL) and traffic tonnage rising to 45.7 Mtpa (megatonne per annum) are as defined by the Concept of Operation, detailing number of trains, train type and length, speed and lifecycle as a function of time. The ARTC Inland Rail BoD indicates: "It is desirable for the rut depth no greater than 50 mm at the top of capping. It is essential that rut depth to be no greater than 200 mm at the top of capping over the design life at any location". The ARTC Basis of Design for the Inland Rail also requires a mechanistic formation design where cumulative plastic deformation is calculated.

3.2 *N2NS project overview*

The method presented in the previous section was developed for and applied to the Narrabri to North Star (N2NS) Project in northwest New South Wales, which comprises an upgrade of the existing rail track as part of the Inland Rail program. The project starts north of Narrabri Junction at kilometrage 575.000 and terminates at North Star approximately 186km north at kilometrage 760.500.

The project comprises the upgrading of the N2NS existing railway line to allow for heavier trainloads (up to 30 TAL), increased train speeds (80km/hr), frequency and increased traffic tonnage through the design life course (50 years).

The terrain is gently undulating, with the alignment crossing several broad floodplains, overland flow paths and smaller creeks. Numerous existing culverts and under bridge locations are generally associated with these geomorphic features. Exiting rail embankment were generally low (0.5m to 1.5m high) constructed from the track cess drain site won soil. Black soils (heavy reactive soils) often exhibiting gilgai geomorphic features along the N2NS alignment were recorded.

3.3 *Failure mechanism and investigation findings*

Over 300 test pits were excavated within the existing formation between September and October 2017 (from shoulder to shoulder). A typical "*w*" shape feature was noted on most test pits as evidence of progressive shear failure mechanism, see Figure 2 and Figure 3.

Figure 2. Photo of typical soil profile recorded during N2NS geotechnical investigation showing *w* shape (dash orange), ash and progressive ballast degradation (dash white line).

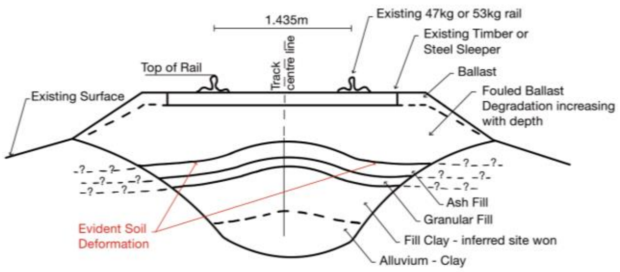

Figure 3. Sketch of a typical test pit exposure for the N2NS geotechnical investigation.

The detailed recording of the test pit features was essential to the ground model as well as undisturbed sampling (push tube U_{50} and U_{75}), vane shear and Dynamic Cone Penetrometer (DCP) testing. It was also noted during the geotechnical investigation that the cohesive soil within the existing rail formation was significantly moister and generally had a significantly lower strength than cohesive soil outside the footprint of the rail embankment, which was observed to be drier and desiccated at the surface and of higher strength with depth.

Undisturbed sampling for suction testing at 0.25m interval was carried out. The results illustrated in Figure 4 indicate a suction profile generally constant with depth, ranging from 1.5 to 2.5pF below the existing formation (below the bottom of ash) and reflecting the "abnormal moisture conditions". A normal moisture condition that would typically apply to exposed cohesive soil would be expected to vary seasonally and range from 4 to 6pF with an equilibrium soil suction value of 4.4pF (Barnett and Kingsland, 1999) in the Moree region (semi-arid).

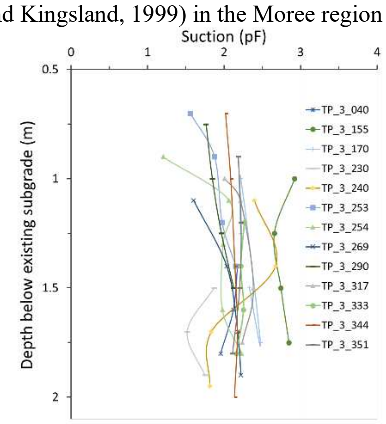

Figure 4. Graph showing typical laboratory suction test results within the existing N2NS subgrade from test pits spread across the alignment.

The presence of ash, also noted on most test pits, is interpreted as a transition layer between the reactive clay fill and the ballast. While the ash may have performed initially, the close to saturation conditions noted during the late 2017 investigation indicate that over time the ash generally retained moisture that seeped into the cohesive soil below the existing formation and reduced the strength of the insitu subgrade.

3.4 *Design parameters*

Fine-grained material is abundant along the alignment and often constitutes a default cost-effective material necessary for creating the embankment supporting the rail. The moisture and undrained shear strength of the cohesive soil and its change over the asset life, drive its performance. The undrained shear strength of cohesive soil is crucial to Steps 3 to 6 previously described. A relationship between undrained shear strength and moisture content specific to the project has been previously published (Blanchet et al., 2019). The ingress of moisture within the existing material and engineered cohesive fill was incorporated in the analysis.

The gradual strength reduction with time anticipated during the design life was evaluated using transient seepage analysis for a range of flood and rainfall scenarios. The flood scenarios were based on flood modelling developed as part of the project. Flood scenarios included water rise to the top of capping, remaining for 3 days, followed by a gradual drawdown for fifteen days and an infiltration period of one year. A rainfall event was considered a 13.5mm/hr event for a 6-hours period followed by an infiltration period of one year.

The analysis results indicate an increase in typically 8 to 10% volumetric moisture content within the formation and increases up to 30% towards the embankment edge outer 1m. The analysis did not consider the implication of evaporation or repetition of the flood scenarios. Further research is recommended to assess the impact of evaporation and multiple rainfall/ flood events on moisture change with time.

A series of cyclic loading triaxial tests was carried out on existing undisturbed cohesive subgrade, structural fill and lime stabilised fill to estimate the resilient modulus and parameters a, b and m used in the calculation of cumulative plastic deformation. The results are summarised in Table 1.

Cyclic loading triaxial testing on undisturbed insitu clay subgrade samples located within the zone of influence of the proposed rail upgrade has been previously presented (Blanchet et al., 2019). Cyclic loading triaxial testing on a reconstituted structural fill sample from the Parkes to Narromine Section of the Inland Rail and an undisturbed lime stabilized (2% quicklime) compacted site won fill sample (U_{100}) obtained from N2NS section is summarised in Table 1.

Table 1. Cyclic loading triaxial testing – Undisturbed samples

Parameters	*Existing Cohesive Soil*	*Structural Fill*	*Lime Stabilized Fill*
Compressive strength (kPa)	140	725	782
Resilient Modulus M_r (MPa)	15 to 20	82 to 100	63 to 109
a	0.88	0.84	0.5
b	0.21	0.021	0.0053
m	2.85	0.23	0.225

All the samples were saturated and isotropically consolidated to 30kPa. Staged undrained cyclic load testing with 10,000 cycles for each stage (with deviatoric stress 30kPa, 50kPa, 70kPa, 90kPa and 110kPa) was carried out. For each stage, the frequency was set at 2Hz. A similar procedure previously presented (Blanchet et al., 2019) was adopted to evaluate *a, b* and *m* parameters. The material parameter *b* was primarily independent of the deviatoric stress levels and could be determined by the exponent of the power model of the cumulative total strain and load cycles (Li et al., 2016). Figure 5 presents the plots of cumulative axial strain vs load cycles for each stage and the best-fit power model of each stage for structural fill and lime stabilised soil. The average exponent *b* was found to be 0.021 and 0.0053 for structural fill and lime stabilized fill respectively. Both b values are more than 10 times smaller than the *b* of insitu clay subgrade. This indicates that most of plastic deformation of the structural fill and lime stabilized fill is completed within first few load cycles. Figure 5 also shows lime stabilized fill performs better in term of minimizing plastic strain although plastic strain within both formation material (<0.7% for structural fill and <0.4% for lime stabilized fill) is less than that from the insitu clay subgrade (generally <2% or 3%). This highlights the main function of the structural fill and lime stabilized fill which is to distribute and reduce train load stresses and strains within the subgrade. Plastic deformation contribution of the structural fill and lime stabilized fill particularly under cyclic train loads is expected to be insignificant.

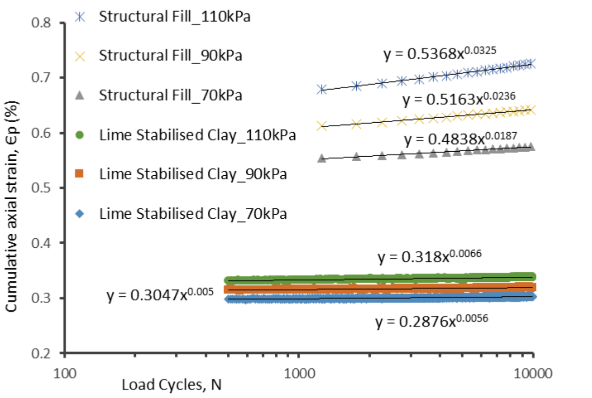

Figure 5. Cumulative total axial strain vs load cycles of each applied deviatoric stress stages (70kPa, 90kPa, 110kPa) for lime stabilised fill and structural fill.

Using the same parameter b and soil compressive strength for each applied deviatoric stress, different values of material parameters a and m could be selected for each stage and are plotted in Figure 6. The intersection of these plots gives the project-specific material parameters *a* and *m*.

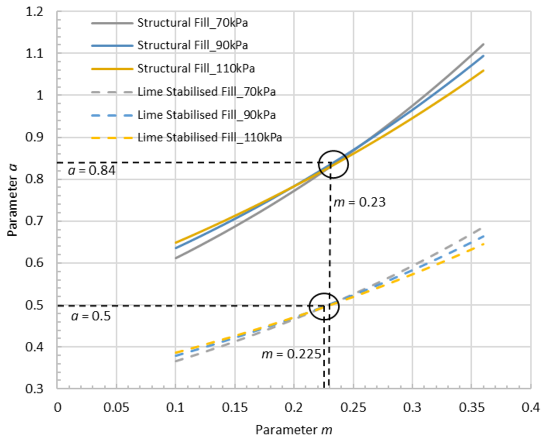

Figure 6. Determination of material parameters *a* and *m* for structural fill and lime stabilised fill

3.2 *Applied deviatoric stress*

A three-dimensional finite element model of the rail track was created using Plaxis3D and used to model train load deviatoric stress distribution within the formation and subgrade. A Mohr-Coulomb Constitutive model was selected with both saturated and unsaturated conditions (Yang et al., 2019). Dynamic wheel load was introduced by applying a dynamic load factor of 1.44 to the static wheel load recommended by AREMA (2012). The model was 14m parallel to the rail track and 20 m perpendicular to the rail track. The depth of the model in the vertical direction was set at 6.5m. Fixed boundary conditions was applied except for vertical direction at upper surface which was set as free boundary. The finite-element mesh comprises a total of 61,080 elements. The rail and sleepers were modelled using plate elements and solid element was used for ballast, formation and subgrade. Materials properties were presented in Yang et al., 2019.

Two models were analysed (Model 1 and Model 2). Model 1, comprising a structural fill layer below capping, has previously been presented (Yang et al, 2019) and is summarised in Table 2. Model 2 comprised a lime stabilised layer below the capping. A similar approach to that Model 1 was adopted for the analysis. Calculated applied deviatoric stress for Model 2 are presented in Table 3. Li and Selig method requires that the subgrade strength (allowable deviatoric stress) to be greater than the applied deviatoric stress (Criteria 1) presented in Table 2 and 3 for Model 1 and 2 respectively. The results following 3D FEA iteration indicate that Model 2 allows a weaker subgrade (S_u = 50kPa input in the analysis giving 37kPa applied deviatoric stress) closer to the formation level compared with Model 1 (Subgrade S_u = 75kPa input in the analysis giving 60kPa applied deviatoric stress) to satisfy Criteria 1 requirement.

Table 2. N2NS FEA analysis results for Model 1

Strata	*Thickness (mm)*	*Calculated applied deviatoric stress (kPa)*
Structural fill (top*)	250	75 (110)
Cohesive Engineered Fill (top*)	250	62 (66)
Existing cohesive subgrade (top*)	500	53 (57)
Existing cohesive soil (bottom*)	4000	20 (20)

*Value in bracket for unsaturated result, top for top of layer, bottom for bottom of layer

Table 3. N2NS FEA analysis results for Model 2

Strata	*Thickness (mm)*	*Calculated applied deviatoric stress (kPa)*
Lime treated (top*)	250	90
Lime treated (top*)	500	79
Existing cohesive soil (top*)	250	37
Existing cohesive soil (bottom*)	4000	18

* top for top of layer, bottom for bottom of layer

3.2 *N2NS design*

Two designs were developed (Design 1 and Design 2). Design 1 using Model 1 results is summarised in Table 4. Design 1 comprises a structural fill layer and excavation and replacement of existing firm clays to achieve a minimum strength profile with depth. The quantum of excavation is based on the minimum strength profile with depth requirement and needs to be validated on-site during excavation works in combination with DCP/ Pandas at regular spacing.

A higher strength profile (30% higher) was required in areas prone to flooding or at higher risk of moisture ingress and degradation of cohesive soil strength within the formation over time (Blanchet et al 2019). Design 1 calculations indicated approximately 75mm cumulative plastic deformation within ARTC.

Table 4. N2NS Formation Design 1

Strata	*Thickness (mm)*
Ballast	250
Capping	200
Structural fill	250
Fill/ natural cohesive soil achieving a minimum undrained strength 100kPa in areas prone to flooding or moisture change and 60kPa for areas less exposed to moisture change.	500
Natural cohesive soil with a minimum strength of 60kPa	4000

Table 5. N2NS Formation Design 2

Strata	*Thickness (mm)*
Ballast	250
Capping	200
Lime treated soil achieving a minimum unconfined compressive strength of 700kPa	750
Existing cohesive fill/ natural soil with a minimum undrained strength of 50kPa	4000

Design 2 using Model 2 results is summarised in Table 5 and was developed in collaboration with ARTC to reduce importing a large amount of structural fill. Design 2 is a single treatment design through the alignment and comprises a 750mm thick site won stabilised layer bridging over the firm soils. The site won comprised a mix of the existing formation material and cess drain from within the corridor improving overall project sustainability.

The Design 2 calculation indicated approximately 50mm of cumulative plastic deformation meeting ARTC desirable acceptance criteria. In Design 2, no strength reduction to the subgrade undrained shear strength (50 kPa) was considered as the subgrade is already nearly saturated at that strength.

4 CONCLUSIONS

The mechanistic design methodology presented is an engineered approach allowing material re-use and the incorporation of moisture change with time. It can be used to develop a design that significantly reduces the environmental impact associated with the importation of material. The absence of the CBR requirement also improves construction efficiencies by reducing double handling and time lag between the sampling and laboratory result. However, it requires:

- A rigorous approach to assess change in moisture and the associated change in strength and stiffness, and deviatoric stress distribution within formation and subgrade due to train load.
- Carefully planned geotechnical investigation fully integrated with the design. The inclusion of unsaturated laboratory testing and a cyclic loading test is strongly recommended.
- Site trial to validate alternative earthworks design and associated assumptions and ongoing construction phase insitu strength validation testing.

This design methodology which estimates rail formation performance over time allows a balance between capital expenditure and operation and maintenance cost critical in high capital expenditure projects such as Inland Rail. It requires close collaboration between the project delivery team members and acceptance from the operation and maintenance team. The methods can be deployed to renewal and maintenance programs to assess future formation deformation based on traffic tonnage and existing rail formation geotechnical conditions.

5 ACKNOWLEDGEMENTS

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