This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.
Application of Li and Selig railway formation design method to expansive soil

V. Blanchet, Y.W. Tun, L. Yang & E. Lo
WSP Geotechnics and Tunnels, Brisbane, Australia

ABSTRACT: Highly expansive soils in floodplains were encountered extensively along the Narrabri to North Star rail section of the Inland Rail Project. Li and Selig’s formation design method with two criteria for design criteria was adopted. The first criterion check applied against allowable deviator stresses, whereas the second criterion is a calculation of cumulative plastic settlement over many cycles related to the number of trains against project settlements acceptance limits. Finite Element Analysis (FEA) was carried out to model applied deviatoric stresses with the Mohr-Coulomb plasticity model in Plaxis 3D. Allowable deviator stresses were also adopted from a design undrained shear strength of subgrade which is from laboratory and field testings. Changes in subgrade moisture content and degree of saturation due to flooding and infiltration were assessed using Seep/W. A relationship was established between the change in undrained shear strength and variation in moisture content. In the calculation of cumulative plastic deformation, project-specific design parameters $a$, $b$ and $m$ were also obtained from one cyclic triaxial testing and compared with results in Li and Selig’s papers.

1 INTRODUCTION

1.1 Narrabri to North Star Inland Rail project

The Narrabri to North Star (N2NS) Project is in northwest New South Wales and comprises an upgrade of the existing rail track as part of the Inland Rail Programme. It starts north of Narrabri Junction at kilometrage 575.000 and terminates at North Star approximately 186 km north at kilometrage 760.500.

The project comprises the upgrading of the N2NS existing railway line to allow for heavier train loads, increased train speeds frequency and tonnage (up to 30 tonne Axle Load (TAL)).

The terrain is gently undulating with the alignment crossing several broad floodplains, overland flow paths and smaller creeks. Numerous existing culverts and underbridge locations are generally associated with these geomorphic features.

The railway track design involves the development of an alignment through this area to achieve the required flood immunities and meet the ARTC Inland Rail Basis of Design (BoD) setting the rail track and formation performance requirements.

1.2 N2NS project overview

Black soils (expansive soils) along the alignment of the N2NS portion of the Inland Rail project were recorded. The N2NS laboratory results indicated that traditional 4-days soaked California Bearing Ratio (CBR) results were of generally between 2 and 3 for a 9 kg surcharge.

Furthermore, the design based on soaked CBR does not meet ARTC’s BoD requirements for a mechanistic approach to the formation design.

1.3 Design methodology

The purpose of this study is to present developed methodology to calculate stresses and strains under cyclic loading within formation and subgrade soils. It also covers the effects of cyclic loading as per ARTC Inland Rail BoD. Consideration of the effects of moisture change and undrained shear strength are given.

There are several methods of rail formation design such as the Li and Selig method, the AREMA method, the British Rail method, and the International Union of Railways (UIC) method. A Li and Selig approach was adopted for the N2NS detailed design project.

2 METHODOLOGY

Li and Selig’s (2016) approach was adopted for designing rail formations in expansive soil. Three significant improvements on Li and Selig’s (2016) are presented as follow:
- The introduction of 3D Finite Element Analysis using a Mohr-Coulomb (M-C) plasticity model for simulating applied deviatoric stress $\sigma_d$ in subgrade,
- The establishment of a project-specific correlation between change in moisture $\text{w}$ and change in undrained shear strength $S_u$, driving the allowable deviatoric stress $\sigma_{da}$, and
- The development of a project specific $a$, $m$ and $b$ value, derived from cyclic load testing to calculate cumulative plastic deformation.

2.1 Subgrade failure mechanisms and design criteria

Several publications stated that the most common track failure caused by large repetitive stresses in the formation and natural subgrade is either a progressive shear failure or excessive plastic deformation as illustrated in Figures 1 and 2.

Figure 2 shows a “w” shaped feature which is an evidence of excess cumulative displacement. This is consistent with observations noted on the excavation face of approximately 60% to 70% of the N2NS test pits carried out under the existing track (between the sleepers) as illustrated in Figure 3.

It was also noted during the investigation that moisture conditions of the cohesive soil within the existing rail formation were significantly moister, and generally had a lower strength compared to cohesive subgrade outside the footprint of the rail embankment which was observed to be drier and desiccated at the surface.

The two formation design criteria from the Li and Selig method are summarised as below:

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

Criterion 2: $\rho \leq \rho_a$

where $\sigma_d$ = applied deviatoric stress due to train loads at a subgrade surface; $\sigma_{da}$ = allowable deviatoric stress; $\rho$ = a cumulative plastic deformation; and $\rho_a$ = an allowable cumulative plastic deformation.

2.2 Deviator stress due to train loads at subgrade surface

An applied deviatoric stress is an essential parameter for the Li & Selig method. This parameter is required for both criteria. The detailed procedure of the methodology to obtain this parameter is not presented in the paper, but can be found in Yang (2018). The Mohr-Coulomb (M-C) plasticity model in Plaxis-3D FEA was used to calculate the incremental change in deviatoric stress distribution in the subgrade with depth (Figure 4) due to train loading.

![Image](https://via.placeholder.com/150)

Figure 3. Excessive subgrade plastic deformations were found during N2NS geotechnical site investigations.

![Image](https://via.placeholder.com/150)

Figure 4. Deviator stress increment with depth (structural fill thickness is 250 mm and recompacted/improved subgrade thickness is 250 mm) (Top of ballast = Bottom of sleeper layer)

Table 1. Summary of adopted increment in deviatoric stress at top of layers due to train loads for the formation design based on Plaxis-3D analysis

<table>
<thead>
<tr>
<th>Formation / Subgrade</th>
<th>Deviatoric Stress Increment, $\Delta(\sigma_d - \sigma)$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
</tr>
<tr>
<td>Top of structural fill</td>
<td>64</td>
</tr>
<tr>
<td>Mid-depth of structural fill</td>
<td>- 69</td>
</tr>
<tr>
<td>Top of re-compact ed subgrade*</td>
<td>63</td>
</tr>
<tr>
<td>Top of In-situ subgrade</td>
<td>51</td>
</tr>
</tbody>
</table>

*Input undrained shear strength parameter is 75 kPa for re-compact ed/improved subgrade cohesive layer.

Figure 4 shows the change in deviatoric stress due to train loads with depth in the subgrade. The FEA
results indicate that the change in calculated deviatoric stress in cohesive soils is less than the change in deviatoric stress through capping, and structural fill. This pattern is similar to that in the Li & Selig design chart (Li, 2016). Table 1 summarised the range of change in deviator stress for each considered layer of the model.

2.3 Design criterion 1

In the design criterion 1, the deviator stress \(\sigma_d\) due to the trainload at subgrade surface must be lower than the allowable deviator stress \(\sigma_d\). \(\sigma_d\) was calculated using an FEA 3D Mohr-Coulomb (M-C) model from Plaxis-3D. The analysis indicates an applied deviator stress \(\sigma_d\) for expansive soil of 50 kPa (Table 1).

The allowable deviator stress \(\sigma_{dL}\) is based on the threshold stress, which is the maximum deviatoric stress at which the rate of strain accumulation under cyclic load is constant (i.e. stable strain accumulation). This is generally taken as half the ultimate compressive strength.

In this paper, the above generalisation was adopted as an expansive soils property. Limited triaxial testing was carried out to assess the threshold stress and undrained shear strength.

Existing moisture conditions and associated change in undrained shear strength with future change moisture (and suction) were also considered as per ARTC Inland Rail BoD: an increase in moisture content will be accompanied by a decrease in suction and a decrease in undrained shear strength. The approach and the results are presented in the following section.

2.3.1 GeoStudio Seep/W

Seep/W analysis was carried out to assist in the assessment of water infiltration within the rail formation in the subgrade from flooding events and rainfall. Change in moisture conditions, which includes change in moisture content and suction, drives volumetric variation and change in undrained shear strength, \(S_u\).

Using the results from hydrology modelling, a flooding scenario for a typical embankment section was modelled in GeoStudio Seep/W Version 2012. A rise in groundwater to top of capping over a period of 1-day, flood level remaining at the top of capping for 3-days and a gradual draw down to a toe of embankment over a period of 15 days, followed by one year (365-days) with no additional ingress of water was modelled in Seep/W. The change in degree of saturation was output from the Seep/W analysis after 365-days following the end of the drawdown.

In general, a ratio of horizontal to vertical saturated hydraulic conductivity (permeability) of 2 to 5 was adopted for all modelled soils. The output of Seep/W gives the volumetric moisture content \(w\) within the formation and subgrade layers.

This analyses results indicated an increase in moisture content in the subgrade by 20 to 30% from 25% to 30% below the rail, 365-days following the end of drawdown. The calculated change in moisture content is used to calculate change in degree of saturation, as described in the following sections.

2.3.2 The degree of saturation and moisture content

The degree of saturation \(S\) following the change (increase in this instance) in moisture content was calculated by Eq 1. For the same subgrade soil, \(G_s\) and \(\gamma_{dry}\) were assumed constant. If \(S\) increases from 94% to a fully saturated state, \(w\) also increases from 25% to 30% (Table 2). This increase in the moisture content was validated against measured moisture content and degree of saturation from the cyclic laboratory test (Table 2) and series of laboratory dry density results.

\[ S = \frac{w \times G_s \times \gamma_{dry}}{\gamma_w \times G_s - \gamma_{dry}} \]

where \(w\) = moisture content; \(S\) = degree of saturation; \(\gamma_w\), \(\gamma_{dry}\) = unit weight of water and dry unit weight of soil respectively; and \(G_s\) = specific gravity of spoil particle. The change in moisture content was used in the assessment of the potential for change in undrained shear strength as presented below.

2.3.3 Effect of moisture content on undrained shear strength

The reduction in undrained shear strength with increase in moisture content was considered in the design. \(S_u\) can be approximated using the Atterberg Limits and moisture content \(w\) data using methods from literature, as follows:

\[ S_u = 170e^{-4.61(1-w)} \] (Wroth and Wood, 1978) and

\[ S_u = 138e^{-4.61(1-w)} \] (Koumoto and Houlshy, 2001)

where \(S_u\) = undrained shear strength; \(I_L\) = the Liquidity Index, which is a function of Atterberg Limits (liquid limits \(L_l\) and plastic limit \(P_l\)) and Moisture Content as follows: \(I_L = (w-P_l) / (L_l-P_l)\) (Figure 5).

Figure 5. A summary of Atterberg Limits test results for natural formation subgrade in the N2NS project. The highlighted region is the data set which is used in Figure 6 plot.

Figure 5 shows that the tested subgrades are generally classified as high plasticity clay. For each Atterberg limit test carried, the corresponding undrained
shear was calculated for each correlation and was plotted in Figure 6 (black line for Wroth and Wood, and green line for Koumoto and Houlsby).

![Figure 6](image)

Figure 6. A reduction in undrained shear strength over moisture content.

A site-specific correlation was also developed for this project, as presented in Figure 6. In this figure, 121 blue circular dots represented correlated undrained shear strength based on field testing results (Vane shear testing, pocket penetrometer and dynamic cone penetration tests and validated with triaxial unconsolidated undrained tests). The field moisture content was obtained from laboratory testing for the same location as the field-tested samples that were used to derive the undrained shear strength. Best fit curves were developed using plotted data (blue dash line in Figure 6).

As shown in Figure 6, a significant proportion of the data had a high \( w \) and \( S_u \) of less than 50 kPa in places. This is consistent with the observation made at the time of the site investigation, suggesting that moisture had accumulated within the cohesive subgrade, with the ash serving as a conduit for water ingress, but also reducing evaporation.

It is noted that the Wroth and Wood method addresses fully saturated clay samples. The clay soils at N2NS are mostly from 90 to 95% degrees of saturation, suggesting that the two literatures correlation may be conservative for samples within 85% to 95% of saturation range (moisture content between 20% to 25%).

At lower moisture content, the two previous studies by Wroth and Wood (1978) and Koumoto and Houlsby (2001) result give a higher undrained shear strength than the N2NS site-specific correlation. The site-specific correlation presented in Figure 6 was adopted to assess the relationship between soil strength, plasticity and moisture content for detailed formation design.

In Figure 6, the average moisture content of the expansive soil is between 20% and 25% (blue shaded area). The fully saturated sample has a \( w \) value of 30% (light yellow highlighted area). Both studies, and the project specific correlation, indicate that undrained shear strength reductions are higher at a lower degree of saturation (and at higher strength) than when closer to saturation (and lower strength).

2.4 Design criterion 2

Subgrade progressive shear failure and excessive plastic deformation under repeated loading in subgrade comprising fine grained soils can be related to subgrade cumulative plastic strains \( (\varepsilon_p) \), and deformation \( (\rho) \), as represented in Eq 3-4 (Li, 2016).

\[
\varepsilon_p(\%) = a \left( \frac{\sigma_d}{\sigma_s} \right)^m N^b
\]

\[
\rho = \int_0^\tau \varepsilon_p \, dt
\]

where \( \varepsilon_p \) = the cumulative plastic strains; \( N_p \) = the number of repeated stress applications during the design life; \( \sigma_d \) = soil deviatoric stress; \( \sigma_s \) is soil compressive strength (onsite testing and triaxial laboratory results); \( \rho \) is cumulative plastic deformation; \( T \) is layer thickness; and \( a, b, m \) = cyclic load test parameters.

2.4.1 Cyclic load test

The objective of the cyclic load test was to duplicate the project-specific Li & Selig design parameters \( a, b \) and \( m \). The parameters were compared with those of several earlier studies. It was an experimental investigation performed to study stability threshold stress under the repeated cyclic loading of an expansive soil sample.

The U50 “undisturbed tube” was taken in test pits as part of the N2NS geotechnical site investigation. It was cut to produce samples approximately 140 mm in length. During the cyclic loading stages, data were recorded for five cycles in every 500 cycles. At the completion of the cyclic triaxial, the undrained shear strength was measured using a laboratory vane shear for the top and bottom of the sample. Sample measurements are summarised in Table 2.
Figure 7. Comparison between the literature and N2NS results for the relationship between moisture content and undrained shear.

Staged cyclic loading testing with 10000 cycles for each stage was carried out. For each stage (5 in total) a different cyclic deviatoric stress was applied:

- stage 1 - cyclic deviator stress = 30kPa;
- stage 2 - cyclic deviator stress = 50kPa;
- stage 3 - cyclic deviator stress = 70kPa;
- stage 4 - cyclic deviator stress = 90kPa and
- stage 5 - cyclic deviator stress = 110 kPa.

The measured cumulative axial strains for each stage are presented in Figure 7. In our test for each stage, the cumulative axial strains for the first five load cycles then at every 500 load cycles are recorded. Laboratory results for the sample tested are presented in Table 2.

Table 2. Cyclic triaxial laboratory results

<table>
<thead>
<tr>
<th>Measurements</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial gravimetric water content, w</td>
<td>24.0</td>
<td>%</td>
</tr>
<tr>
<td>Final gravimetric water content, w</td>
<td>30.0</td>
<td>%</td>
</tr>
<tr>
<td>Specific gravity, Gs</td>
<td>2.63</td>
<td></td>
</tr>
<tr>
<td>Initial void ratio, e0</td>
<td>0.67</td>
<td></td>
</tr>
<tr>
<td>Initial degree of saturation, Sr</td>
<td>94.2</td>
<td>%</td>
</tr>
<tr>
<td>Initial dry density, ρ_dry</td>
<td>1.57</td>
<td>t/m³</td>
</tr>
<tr>
<td>Undrained shear strength (top), S_u*</td>
<td>82.0</td>
<td>kPa</td>
</tr>
<tr>
<td>Undrained shear strength (bottom), S_u*</td>
<td>62.0</td>
<td>kPa</td>
</tr>
</tbody>
</table>

The data for Figure 8 was generated from the cyclic laboratory results shown in Figure 7, and present the cumulative axial strain for each stage of cyclic compressive load, plotted in log scale. The purpose of Figure 8 is to obtain the Li and Selig parameters a, b and m used in the calculation of cumulative displacement.

In Figure 8, the power constitutive model equation for each load stages is shown with R-squared (R²) value (the line of best fits). It is the best-fitted curve when the compressive strength is 30 kPa.

Among these three parameters, the b value has more consistent results between different cyclic stages compared to parameters a and m. The mean value for b is 0.21 (Refer to the equations from Figure 8). Coefficient a has the most significant uncertainty and variability. This coefficient conceptually represents the soil plastic strain. The coefficient m value is larger than 1 for fine-grained soils, which indicates the softening phenomenon of deviator stress on soils.

One cyclic load test was undertaken, and there are two unknowns. The following procedure is adopted to obtain these parameters.

The parameter m vs a graph, is plotted in Figure 9. In the graph, three different coefficient A with different stress stages were plotted. The intersection between these three lines gives the project-specific m and a.

Figure 9. Parameters a and m obtained from the cyclic load test

Table 3. Comparison between N2NS and Li & Selig

<table>
<thead>
<tr>
<th>Parameters</th>
<th>N2NS Lab Results</th>
<th>Li &amp; Selig</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>0.88</td>
<td>0.82</td>
</tr>
<tr>
<td>b</td>
<td>0.21</td>
<td>0.12</td>
</tr>
<tr>
<td>m</td>
<td>2.85</td>
<td>1.30</td>
</tr>
</tbody>
</table>

* At subgrade surface (below capping layer).

Table 3 shows the comparison between the N2NS laboratory results and Li & Selig parameters for high plasticity clays. It can be seen N2NS laboratory results are within the range of Li & Selig (2016) recommended parameters.
3 N2NS DESIGN CASES

3.1 ARTC design requirements

Per ARTC Inland Rail Basis of Design, design life is 50 years, and design loading is as defined by the Concept of Operation (number of trains, train type and axle load, 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 no greater than 200 mm at the top of capping over the design life of any location”.

For this publication, the following rail formation was adopted: a ballast depth of 250 mm, a capping layer thickness of 200 mm and axle loads are 30 TAL at 80 km/h. The 30TAL rail load configuration adopted in this design is in accordance with AS5100.2 (2017) and comprises a group of vehicles with four axles each having a load of 300 kN and has axle spacing of 1.7 m, 1.1 m and 1.7 m.

A dynamic load allowance is considered for the design axle load. A dynamic wheel load is used by applying an impact factor (dynamic load factor) to the static wheel load as recommended by AREMA (2012). The adopted impact factor is 1.44 to account for dynamic loading.

3.2 Design case & geotechnical parameters

The design methodology presented above was adopted in the N2NS formation design where Criteria 1 and 2 are targeted to conform with the ARTC Inland Rail BoD. An example is shown in Table 4 for an equivalent undrained shear strength profile with depth (for a 250mm thick structural fill, and 250mm cohesive recompacted/improved layer overlying cohesive natural soil) meeting the ARTC Inland Rail BoD “desirable” performance criteria. The effect of moisture change due to ingress of water or drying as results of engineering fill placement is not included in this calculation, and cumulative displacement are calculated using a, b and m parameters from Li & Selig (2016).

Table 4. Summary of analysis results (refer to Figure 4 for strata thicknesses)

<table>
<thead>
<tr>
<th>Measurements</th>
<th>Subgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fill</td>
</tr>
<tr>
<td>Undrained shear strength, $S_u$ (kPa)</td>
<td>85</td>
</tr>
<tr>
<td>Static compressive strength, $\sigma_s$ (kPa)</td>
<td>170</td>
</tr>
<tr>
<td>Parameter $a$</td>
<td>0.64*</td>
</tr>
<tr>
<td>Parameter $b$</td>
<td>0.10*</td>
</tr>
<tr>
<td>Parameter $m$</td>
<td>1.70*</td>
</tr>
<tr>
<td>Deviator stress due to trains load at the top of the subgrade, $\sigma_d$ (kPa)</td>
<td>85</td>
</tr>
<tr>
<td>Allowable deviator stress of subgrade $\sigma_{d, a}$ (kPa)</td>
<td>85</td>
</tr>
<tr>
<td>Plastic deformation, $\rho$ (mm)</td>
<td>2.2</td>
</tr>
</tbody>
</table>

4 CONCLUSION AND DISCUSSIONS

Analysis for a formation mechanistic approach to formation and use undrained shear strength should consider the effects of strength change with moisture content. In this study, we have demonstrated that an increase in moisture content of 20% to 30% could result in reduction of strength by approximately 50%. Reduction of strength decreases with degree of saturation increase. Change in strength with change in moisture at low suction should be further studied.

Seepage analysis and consideration for unsaturated behaviour should be given as part of the geotechnical investigation and testing planning stage to allow for tailored testing to support a mechanistic design approach and the detailed modelling of unsaturated behaviour. Early establishment of moisture conditions in the field is necessary to establish a cost-efficient laboratory testing schedule and earthworks methodology.

The single cyclic load test result indicates that the parameters $a$, $b$ and $m$ are in reasonable accordance and are possibly less conservative than the parameters recommended by Li & Selig. Further cyclic load testing is recommended to improve confidence in the interpretation of the site-specific $a$, $b$ and $m$ parameters as the project progresses towards detailed design.

5 ACKNOWLEDGEMENTS

The support from the Australian Rail Track Corporation is gratefully acknowledged.

REFERENCES