# GEOTECHNICAL ASPECTS OF THE MID-WEST RAIL UPGRADE PROJECT IN WESTERN AUSTRALIA

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## ABSTRACT

This paper presents a description of the geotechnical aspects associated with the design and construction of the Mid-West Rail Upgrade project in the Mid-West region of Western Australia. It covers aspects of site investigation on existing track formation and track duplication alignments, assessment of the capacity of existing bridge foundations, methods for assessment of the suitability of existing rail formation and upgrades required, analysis of railway behaviour in salt lake areas and recommended ground improvement schemes as well as design of formation-to-bridge transition zones. The paper presents results from a constructability study carried out to assess the stability of operational existing track during construction of duplicated portions of the line. It also describes methodology and results of borrow source investigations for both main sections of the project and geotechnical related issues that arose during the construction stage.

## 1 INTRODUCTION

Brookfield Rail is a provider of rail infrastructure in Western Australia including the Mid-West region. In this region, Brookfield's primary customers are resource companies developing new major iron ore projects. Such projects require bulk haulage of iron ore for the mine-to-port segment of their supply chains with the product ultimately exported through the Port of Geraldton. The Mid-West Rail Upgrade project is one of these major projects.

Physically the project was divided into two main sections of rail. The north-south oriented section (N-S section) comprised the rail from Tilley siding (north of Morawa, WA) to the Mullewa junction and is approximately 100 km in length. The east-west oriented section (E-W section) from Mullewa junction to Narngulu Yard. (about 8 km east of Geraldton, WA) is approximately 95 km in length An additional short section of rail (approximately 8 km) between Narngulu Yard and the port of Geraldton was also included in this project. The location and alignment of the two rail sections is presented on Figure 1.

The upgrade of existing rail infrastructure was to cater for the current and future requirements of Brookfield's clients. The works were undertaken in two stages:

i) Stage 1 of the upgrade involved strengthening of sections of the existing track between Narngulu Yard and Mullewa (E-W section).
ii) Stage 2 of the upgrade involved providing improved track infrastructure, from Tilley to Mullewa, to enable long term transportation of iron ore produced from the mines east of Tilley.

Both stages included a mixture of upgrade of existing track combined with construction of new track on duplicated formation. The upgraded sections typically included geotechnical and ground improvement works, formation upgrades, new and upgraded drainage systems, bridge assessment and strengthening, and new rail track construction. While the long term expected traffic task on the N-S railway is 32 TAL, the initial operations would be limited to 21 TAL. This introduced different design requirements and complex timing for design and construction of works.

This paper presents a description of the geotechnical work associated with the investigation, design and construction of the Mid-West Rail Upgrade project. The work included:

- site investigation on existing formation and for duplication alignments,
- methods for assessment of the suitability of existing rail formation,
- design of typical upgrades required on existing formation to support increased load requirements,
- assessment of the geotechnical capacity of existing bridge foundations,
- analysis of railway behaviour in salt lakes areas and recommended ground improvement schemes, and
- design of formation-to-bridge transition zones


Figure 1: General location of Mid-West Rail Upgrade project.

Results from a constructability study carried out to assess the stability of operational existing track during construction of the duplication of the line are also discussed. In addition, the methodology and results of borrow source investigations for the two main sections of the project are provided. Finally a discussion of significant geotechnical related issues that arose during the construction stage is presented.

## 2 GENERALISED GEOLOGY

The following publicly available geologic maps (GSWA, 1983) were reviewed and used as a basis for preliminary assessment of the ground conditions underlying the rail to be upgraded:

- Tilley to Mullewa alignment (N-S section): 1:250,000 scale Perenjori and Yalgoo sheets
- Narngulu to Mullewa alignment(E-W section): 1:250,000 scale Geraldton-Houtman Abrolhos sheet

Review of the geologic maps indicated the N-S section of the rail alignment to be underlain by Archaean age granite and granitic gneiss basement rock. The E-W section of rail alignment was mapped as being underlain by Precambrian age granulite bedrock overlain by Mesozoic age lithified sedimentary rocks primarily of the Champion Bay Group.

Typically, the basement rock underlying both sections of the project has a surface weathering mantle of varying thickness which can range from 0 m to 10 m or greater. Locally, lateritic ferricrete and associated gravels and sands overlie the weathered basement rock surface. These materials represent chemical deposition of iron and alumina in the upper portions of a paleo water table. This deposit is believed to have formed roughly 40 million years ago.

Overlying both the weathered bedrock and the lateritic deposits are relatively recent sediments derived primarily from erosion of the underlying materials.

## 3 SITE INVESTIGATION

### 3.1 EXISTING INFORMATION

Prior to design, a suite of geotechnical information for the project was supplied by Brookfield Rail. This work included reports prepared previously by other consultants. The investigations involved visual examination of materials along the alignment, dynamic cone penetrometer testing and limited laboratory materials testing for the purposes of preliminary site characterisation.

### 3.2 ADDITIONAL SITE INVESTIGATIONS

Additional site investigations typically comprising series of test pits (TPs) with dynamic cone penetrometer test probing (DCP) and electric friction cone penetrometer test probing (CPT), were conducted by AECOM along both existing formation and proposed duplication alignments to develop the interpretation of the subsurface ground conditions for use in the detailed design of the proposed embankment and formation upgrades. This work was completed in May 2011.

The locations and target depths of the investigations were guided by preliminary horizontal and vertical rail alignment options prepared by AECOM's rail engineering team and the magnitude of the proposed design rail loadings. The results of the investigations and subsequent geotechnical analyses guided the rail engineering team's decisions on whether to strengthen the existing formation or construct along new alignments. This in turn helped to focus the geotechnical investigations in subsequent campaigns to gather better site-specific information for design.

### 3.2.1 Test Pits and Dynamic Cone Penetrometer Testing

Test pitting was carried out in order to assess the subsurface conditions along the proposed rail corridor. A total of 195 test pits were excavated at select locations to depths ranging from 0.5 m to 2.1 m. The test pits were typically excavated along new alignments due to site access restrictions to the existing formation and concerns about weakening the existing formation. However limited test pits were excavated at the edge of existing sleepers to assess the condition of the existing ballast and rail formation.

The excavations were advanced using an 18 tonne excavator (CAT318) to collect bulk and disturbed samples for visual assessment and laboratory testing. Excavators were selected for the project due to previously encountered difficulties with lighter equipment (backhoes and mini-excavators) experiencing shallow refusal in locally extensive laterite deposits in natural ground found during the early stage investigations. These laterite deposits are known to form 'hard caps' over softer sediments and weathered rock, potentially leading to overestimates of foundation strength conditions. Following the completion of logging and sampling, the test pits were backfilled and compacted by tamping with the excavator bucket.

Dynamic cone penetrometer (DCP) tests were also conducted adjacent to the test pits in accordance with Australian Standard AS 1289.6.3.2 (1997). All test pit locations were referenced by using a handheld GPS and checked against offsets from chainage markers along the rail corridor.

### 3.2.2 Electric Friction-Cone Penetrometer Tests (CPTs)

A total of 90 rail mounted CPTs were conducted to depths in the range 0.2 m to 8.0 m below top of formation. Other 110+ CPTs were conducted on duplication alignments. The target depths were typically selected to be to depths at least equal to the height of the existing formation but in practice most probings stopped well short of this due to refusal on very dense granular materials or rock strength materials. A modified rail trolley was used to carry out CPT testing along the centreline of the existing formation along the existing rail alignment and for study of constrained level crossings.

CPT locations were positioned by AECOM's field engineer by use of a handheld GPS and checked against offsets from chainage markers along the rail corridor.

In addition to the investigations described above, the work included specific site investigations for the assessment of the capacity of existing bridge foundations, geotechnical design of crossings of salt lake areas (details provided below) and the geotechnical assessment of constrained level crossings. Figure 2 illustrates the different types of site investigation techniques used in the project.

(a)

(b)

(c)

Figure 2: (a) CPT rig mounted on a slepeer trolley for on-formation site investigation of existing track structure; (b) DCP testing at Narngulu Yard; (c) Borehole drilling at Eradu bridge for assessment of capacity of existing pile foundations.

# 4 LABORATORY TESTING

A laboratory testing program was carried out on selected samples from the test pits to characterise in situ materials and as bulk test pit samples to characterise potential fill materials. All scheduled testing was carried out in NATA registered laboratories. The following types of tests (carried out according to Australian Standards) were conducted:

- classification testing: Atterberg limits (AS1289 3.1.1, 3.2.1 and 3.3.2), linear shrinkage (AS1289 3.4.1) and particle size distribution (AS1289 3.6.1),
- California Bearing Ratio (CBR) (AS1289 6.1.1),
- multi-stage direct shear tests on remoulded samples (AS1289 6.2.2)
- point load strength index (AS4133 4.1)
- compaction testing: compaction tests (AS1289 5.2.1).

Direct shear test results were used to assess the strength characteristic of *in situ* and borrow materials soils planned to be re-used as an embankment fill.

# 5 ENGINEERING PROPERTIES

## 5.1 SOIL STRENGTH

Soil strength information was derived from CPT and DCP test data.

The peak friction angle of granular (sandy and gravelly) soils based on CPT probing was assessed using the methods of Robertson and Campanella (1986), Olsen and Farr (1984), Baldi (1985) and Mimura, 2004 (all referenced by Lunne *et al.*, 1997).

The estimated *in situ* undrained shear strength of fine-grained, cohesive soils based on CPT probing was assessed using two methods – the cone factor and the SHANSEP methods. A conservative value of undrained shear strength was selected for design on the basis of these two methods.

The DCP test data was interpreted using the methods presented in Look (2007). A conservative strength for the *in situ* soils was adopted for design based on the 25$^{th}$ percentile angle of internal friction derived from both CPT data and DCP test data, as the majority of soils are sandy in nature. The DCP test interpreted strength results were compared to the CPT probing interpreted data at adjacent testing sites and it was assessed that the 25$^{th}$ percentile approach provided a reasonable correlation between the two methods. The testing indicated a surprising degree of spatial consistency in the in situ strength of the existing formations. An angle of internal friction of 36 degrees was typically adopted in the subsequent analyses for the entire alignment. An angle of internal friction of 35 degrees was assumed for common fill material.

## 5.2 MOISTURE REACTIVITY

A relationship between plasticity index and linear shrinkage has been presented by Hazelton and Murphy (2007) for determining the volume change potential. A correlation for arid to semi-arid climate was adopted for this interpretation. A plot of the plasticity index and linear shrinkage indicates that the soils have generally low potential for volume change with limited areas of soils with medium potential for volume change at specific chainages. A plot of plasticity index and linear shrinkage for a section in the N-S alignment is presented as Figure 3.

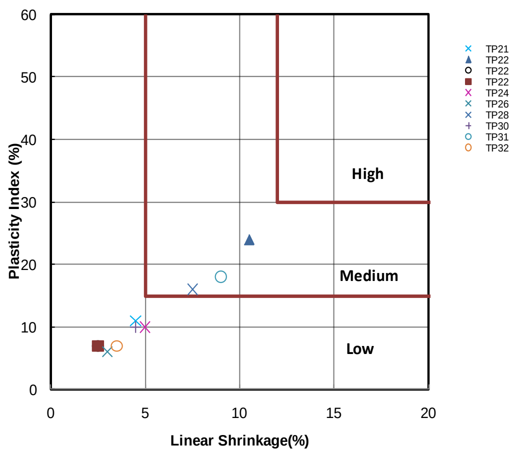


Figure 3: Qualitative assessment of volume change potential.

# 6 RAILWAY GEOTECHNICS

## 6.1 RAILWAY LOADINGS

Generally, the proposed long-term design static axle loads is 21 tonnes for track upgrade sections (including existing bridges discussed in section 6.5) and 32 tonnes for new duplication alignments. In addition to the static load, dynamic loads were considered in the analytical models used for slope stability assessments. Dynamic load factors were estimated using Eisenmann's formula. For duplication alignments, a dynamic load factor of 1.50 (reflective of new track and new formation) was adopted as the basis of design assuming the following average conditions:

- Design Speed = 80 km / hr (max.)
- Upper confidence limits = 97.7% (t = 2)
- Track condition factor (Eisenmann method) = 0.2 (good track condition).

The track condition factor was selected in consultation with Brookfield Rail's operations team inputs to reflect the anticipated long term maintenance regime expected to be applied to the rail over its design life.

The rail seat load (R) was estimated based on dynamic load and load distribution factors in accordance with AS1085.14, Cl 4.2.3. Using various track structure variables such as sleeper spacing and depth of clean ballast as inputs, the applied pressures were calculated based on AS1085.14, Cl 4.2.4 and Brookfield Rail Narrow Gauge Code of Practice (equations 3.9, 3.10 and 3.11). Different railway track foundation design methods were considered (Burrow *et al.,* 2007; Li and Selig. 1998a, 1998b). The factor of safety (FoS) achieved was calculated based on the assessed ultimate bearing pressure and the calculated applied pressure. The results of final FoS for various track structures were reported separately. A typical calculation of load distribution is presented as Figure 4.

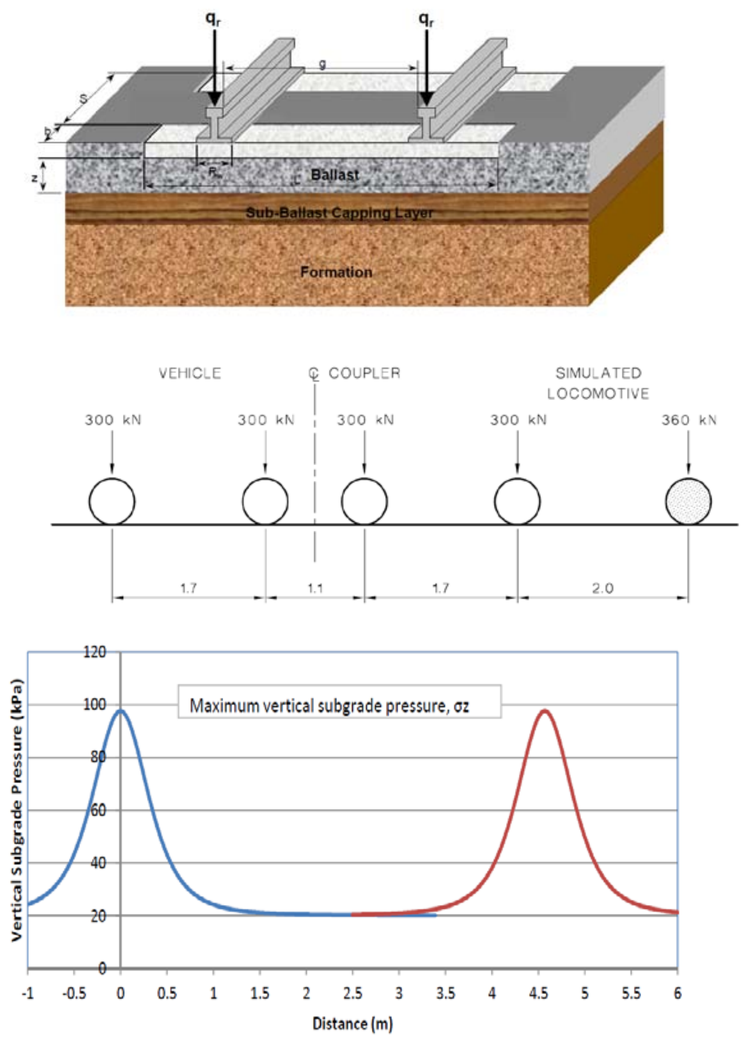

Figure 4: Evaluation of load distribution from ballast contact pressure to vertical subgade pressure (inset (b) from AS 5100.2—2004).

Following a similar approach, a dynamic load factor of 1.69 was established as the basis of design for upgrade of existing formation (e.g. E-W section alignment between Narngulu and Mullewa). This higher value reflected uncertainty in conditions along existing embankments, a lower track condition factor and greater variability in results from site investigations along the alignments.

## 6.2 DESIGN APPROACH FOR EXISTING FORMATION

Typical geometry requirements for formation included a formation width of at least 6 m wide and batter slopes no steeper than 1V:2H (for both banks and cuttings).

### 6.2.1 Allowable bearing pressure

The allowable bearing pressure of the subgrade was calculated as the ultimate bearing capacity (point at which shear failure of the subgrade occurs) divided by a factor of safety. The factor of safety takes into account shear failure of the soil and mitigates against excessive settlement. For the purposes of these analyses, the factors of safety for shear failures and settlement are taken as 2 and 1.67, respectively (Doyle, 1980). This results in an overall global factor of safety of 3.33.

The allowable bearing pressure of the formation was checked using a two-dimensional limit equilibrium method in the software Stabl (Version 6H) (Achilleos, 1988). The analyses undertaken assuming that the critical failure surface can be approximated by a circular arc whose mobilising and resisting forces are approximated using the Bishop method of analysis and assumptions. The results were checked against the random failure surface generator method in Stabl which typically approximated a log spiral failure surface.

Assessment of the allowable bearing pressure assuming a two dimensional failure surface of unit width is conservative as it ignores the shear resistance of the ends of any failure surfaces that would occur as three-dimensional failures in reality. A key advantage of using Stabl is that the actual embankment geometry can be assessed, particularly the loss of confinement in the vicinity of slopes.

The ultimate resistance of the formation to bearing failures was estimated by increasing the surcharge load distributed over a sleeper width until a factor of safety of 1 was achieved. This is the point at which the mobilising forces are equal to the resisting forces. The allowable bearing pressure is taken as the trial surcharge value divided by the overall global factor of safety of 3.33.

Estimation of peak capping stresses at the underside of ballast (capping level) was undertaken as a separate exercise as part of the track design considering appropriate ballast thicknesses in order to limit stresses at the capping level to acceptable values.

### 6.2.2 Embankment stability

In addition to bearing failures, the maximum allowable pressure applied by the rail loads could be limited by the overall embankment or embankment foundation stability. Considering the age of the existing railway (80+ years) and an increase of axle load from 16 TAL to 21 TAL it was considered unlikely for existing embankments to undergo significant short or long term settlement under the new design axle loading condition. This was confirmed by two dimensional finite element analyses using short term elastic modulus values derived from the CPT probing data and hand calculation of long term settlement estimates due to creep assuming onerous sustained peak loading conditions. Other railroad settlement models (Dahlberg, 2001) were also considered.

As such, only the stability of embankments under the new axle load was assessed. It was generally considered that as long as adequate stability is ensured, the settlement caused by the train load may be approximated as proportional to the axle load (excluding specific sections in salt lake areas). Two approaches were used for this assessment:

- A limit equilibrium analytical approach similar to the above mentioned method was used to assess the critical factor of safety against slope failure.
- Embankment stability analysis was conducted by means of finite element modelling using a phi/c reduction approach in which the shear strength of the soil units is reduced by a reduction factor until imminent failure is apparent.

The embankment stability was assessed under both static and earthquake loading conditions. The seismic condition was assessed with pseudo-static analyses considering 50% of the Peak Ground Acceleration (PGA) of 0.09 *g*, in accordance with AS 4678-2002 and AS 1170.4-2077. Three different water levels were considered for the static analyses: water at embankment toe level, fully submerged and "flow" condition (upstream water at crest level and downstream water at toe level). The water level was considered at the embankment toe level for the seismic case as the likelihood of an earthquake of the design magnitude (10 % chance of occurring in a 50 year period) and a peak flood (> 1 in 20 to 50 year flood) occurring at the same time during the life of the embankments was considered remote.

The minimum Factors of Safety (FoS) presented in Table 1 were targeted for different loading and water conditions.

Table 1: Target minimum factors of safety values.

| Ground Condition Profile | Static Conditions | | | Seismic Condition PGA = 0.09 g |
|---|---|---|---|---|
| | Water at Toe | Fully Submerged | Flow (upstream at embankment crest, downstream at toe) | Water at Toe |
| 50th percentile | 1.45 | 1.3 | 1.2 | 1.2 |
| 100th percentile | 1.3 | 1.15 | 1.1 | 1.1 |

Overall, the results indicated that embankments with height less than 1.5 m (including both the cases where the rail is on grade or in a cut situation) are likely to have adequate stability under 21 TAL without additional stabilisation measures.

For embankments with heights exceeding 1.5 m the analyses showed that stabilisation measures were required for the proposed increased loading. The recommended stabilisation measures included toe berms, embankment widening and slope flattening. The general fill material sourced from sandy borrow pits along the alignment to widen and flatten existing oversteep slopes was specified to be compacted to achieve an angle of internal friction of at least 36° to obtain the required factor of safety. In the event of an earthquake, excessive slope movements may occur requiring subsequent maintenance or repairs and this was accepted by Brookfield Rail as being an acceptable risk. A typical sketch of embankment strengthening is presented as Figure 5.

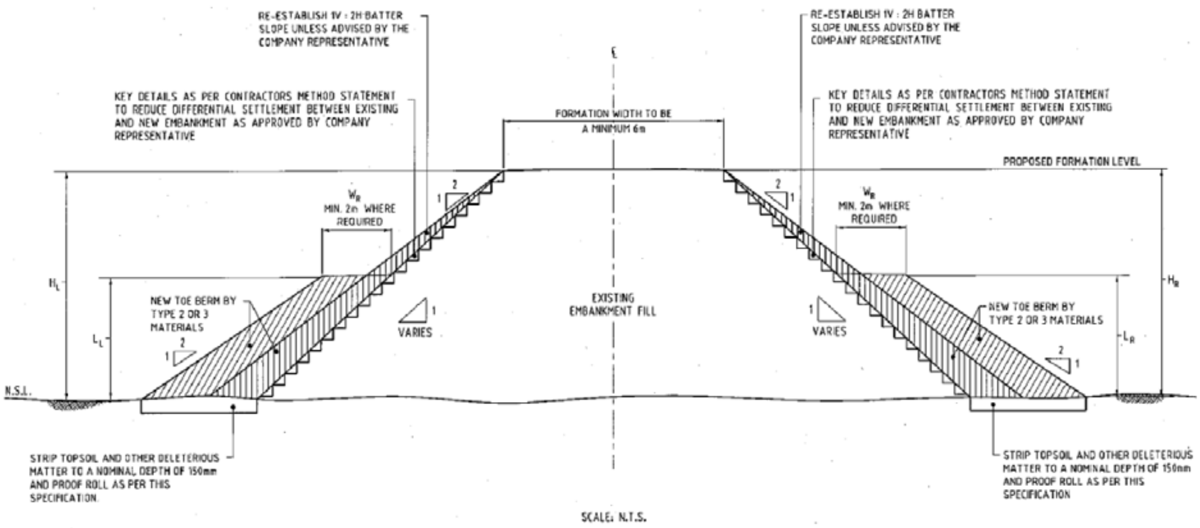


Figure 5: Typical cross-section of existing embankment strengthening – additional berms

### 6.2.3 Embankment erosion

Visual observations of the existing slopes indicated that the sandy materials for construction are susceptible to erosion in the form of rilling and occasionally deep gullying, particularly where runoff from the capping layer is discharged down the slope. Areas covered with vegetation have proved to be less susceptible to erosion. As part of the upgrade works it was recommended to provide erosion protection on newly constructed slopes (including embankment widening) to avoid future slope instability and compromises to track stability.

## 6.3 DESIGN APPROACH FOR NEW EMBANKMENTS (TRACK DUPLICATION

A geotechnical assessment including field investigation and engineering was carried out to facilitate earthworks track design duplication for 32 TAL between Tilley and Canna and between Mullewa to Sullivan on the north-south alignment.

In general, the subsurface ground profile was inferred to comprise medium to very dense granular materials with lenses or layers of clayey soil. Groundwater was observed at limited locations.

The geotechnical assessment of embankment stability was carried out based upon a 6.0 m wide embankment formation at the top and batter slopes of 1V:2H using a similar methodology adopted for track upgrade. The results indicated that all sections analysed with up to 4.0 m high duplicated embankments will have adequate stability under 32 TAL without extra stabilisation measures.

Some areas in this section were identified as requiring hard ripping to remove shallow materials for preparation of embankment foundations. The excavatability assessment is discussed in Section 8.1.

### 6.4 NEW EMBANKMENT ON SALT LAKES AREAS - IRWIN RIVER AND TRIBUTARIES

A 7.5 km section between Sullivan and Mullewa crosses the Irwin River and its tributaries. This section is characterised by the presence of clayey materials from shallow depths and up to 10 m deep with groundwater levels between 0.3 m and 1.0 m below the surface (at the time of investigation). Specific assessment of embankment stability and prediction of settlements was carried out using finite element methods and a staged construction methodology.

The selected criteria for total and differential settlement were based on the Brookfield Rail Code of Practice for design speeds of 80 km/hr (loaded) and 90 km/hr (empty).

Modelling results indicated long term settlements due to embankment weight and train loading would exceed prescribed limiting values if no ground improvement were undertaken on the foundation materials. A ground improvement scheme involving replacement of soft/loose materials with common fill material, compacted according to the specification, was proposed to achieve the settlement criteria.

Recommendations regarding verification of ground conditions by DCP testing with spacing not greater than 30 m to detect soft spots were implemented during construction. Displacement monitoring at regular intervals was also recommended for this section.

### 6.5 ASSESSMENT OF EXISTING BRIDGE FOUNDATIONS

A geotechnical assessment of the foundations of two existing bridges on the E-W section of the alignment between Narngulu Yard and Mullewa was carried out. The objective of the assessment was to evaluate the capability of the foundations of the bridges to support the increased design loads which resulted from the upgrade of the track from 16 toone to 21 tonne nominal axle loads.

While the long term expected traffic task on the N-S railway is 32 TAL, the initial operations would be limited to 21 TAL for potentially several years. Maximum design load in the E-W section is 21 TAL. Given the limited estimated remaining life of the superstructure of the subject bridges, Brookfield Rail did not consider foundation strengthening to be an option.

A general description of the bridges and their respective foundation systems is presented below:

- Eradu Bridge (CH 55) consists of an eight span structure with a total length along the rail track alignment of about 118 m. The bridge is supported by spread footings in Abutment 1 (Geraldton side) and Piers 1 to 3, spread footings and Jarrah piles in Piers 4 to 7 and spread footings and concrete piles in Abutment 2 (Mullewa side). A sketch from the original design drawings is shown as Figure 6.
- The bridge at CH 103 consists of a three span structure with a total length along the rail track alignment of about 11.5 m. The piers and the abutments are orientated at an angle of 135° (Figure 7) with respect to the longitudinal axis of the rail track. The bridge is supported by shallow foundations resting on sandstone.

The geotechnical assessment complemented a detailed structural investigation of superstructure and substructures of both bridges to determine strengthening requirements for a design axle load of 21 tonnes per axle and no speed restrictions for loaded or empty trains. The assessment included evaluation of bearing capacity (Poulos & Davis, 1980), settlement of foundations and stability of abutments. Rock engineering properties were inferred from site investigation data and an analysis based on the Hoek & Brown criterion (Hoek *et al.,* 2002) using the program Roclab V1.031 (Rocscience, 2009).

The geotechnical assessment concluded that the existing foundations of the two bridges were suitable for the proposed track upgrade and the resulting increase in the design loads providing that the ground conditions at the tested locations are consistent with the conditions beneath the existing foundations and that the assumed geometry for piers and abutments is consistent with that provided in the as-built drawings. For the particular case of Eradu Bridge, erosion protection was recommended in the existing sloping ground at the front of the abutments considering the importance of maintaining this slope on embankment stability. A survey and assessment of the integrity of both the Jarrah piles and the concrete piles of Abutment 2 was also recommended.

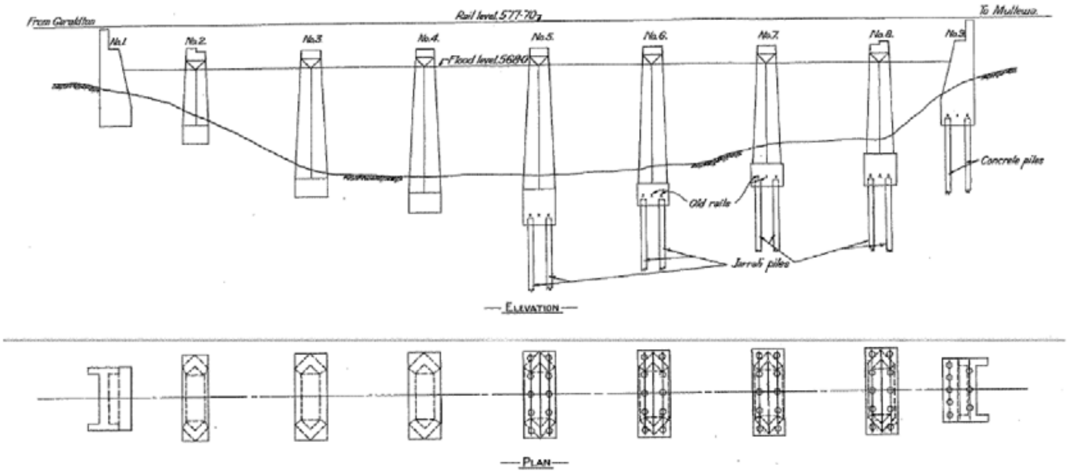


Figure 6: Foundation details for Eradu bridge as per design drawing (1934).

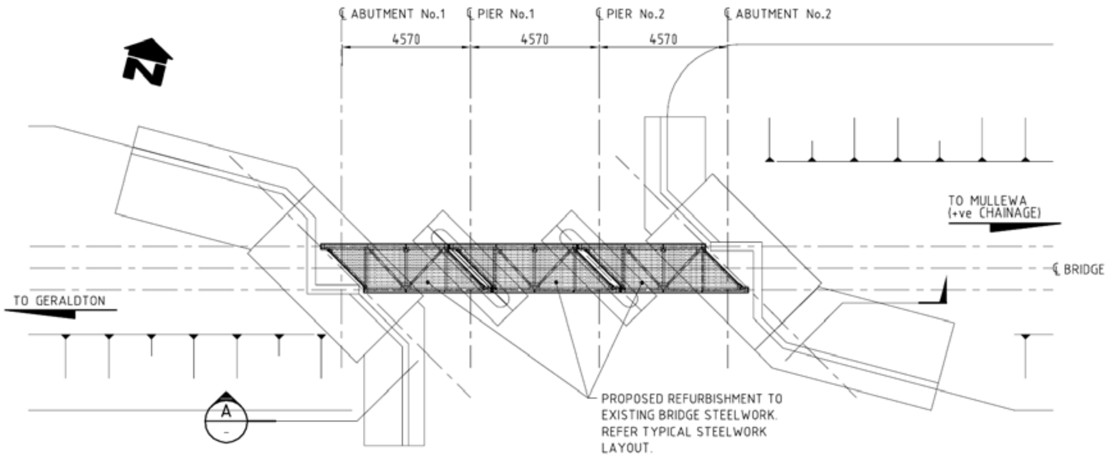


Figure 7: Plan view of rail bridge at CH103 (East-West alignment) indicating orientation of abutments and shallow foundations.

### 6.6 REGULAR FORMATION TO BRIDGE TRANSITION ZONES

Bridge approaches of railway track are locations that often require frequent track maintenance, especially under heavy axle load train operations. Track geometry degradation or rough track geometry is the most common problem associated with railway bridge approaches. It often appears as rough vertical surface and cross-level, and can cause significant vehicle–track interaction or large dynamic wheel–rail forces. The main objective of the transition zone from regular formation to fixed track structure is to prevent uneven track deflections due to large change in track stiffness (Gallego Giner & Lopez Pita, 2009; Li & Davis, 2005). If differential settlement occurs it may lead to adverse dynamic vehicle/track interaction. Increased forces acting on the track cause or accelerate uneven settlement, which leads to even higher forces that are detrimental to track components.

A review of transition zones from upgraded track formation to existing bridges was carried out using the finite element method. Several transition schemes were considered and an alternative involving replacement of up to 1 m of material of the existing formation with layers of selected fill and high strength geogrid layers in between was selected. The selection was based upon technical, cost benefit analysis, ease of construction and minimising rail operation downtime considerations. An iterative design process was carried out to determine the optimum number of layers to minimise potential differential settlements. Differential settlements below 5 mm were calculated for the selected scheme. A sketch of the solution is indicated in Figure 8.

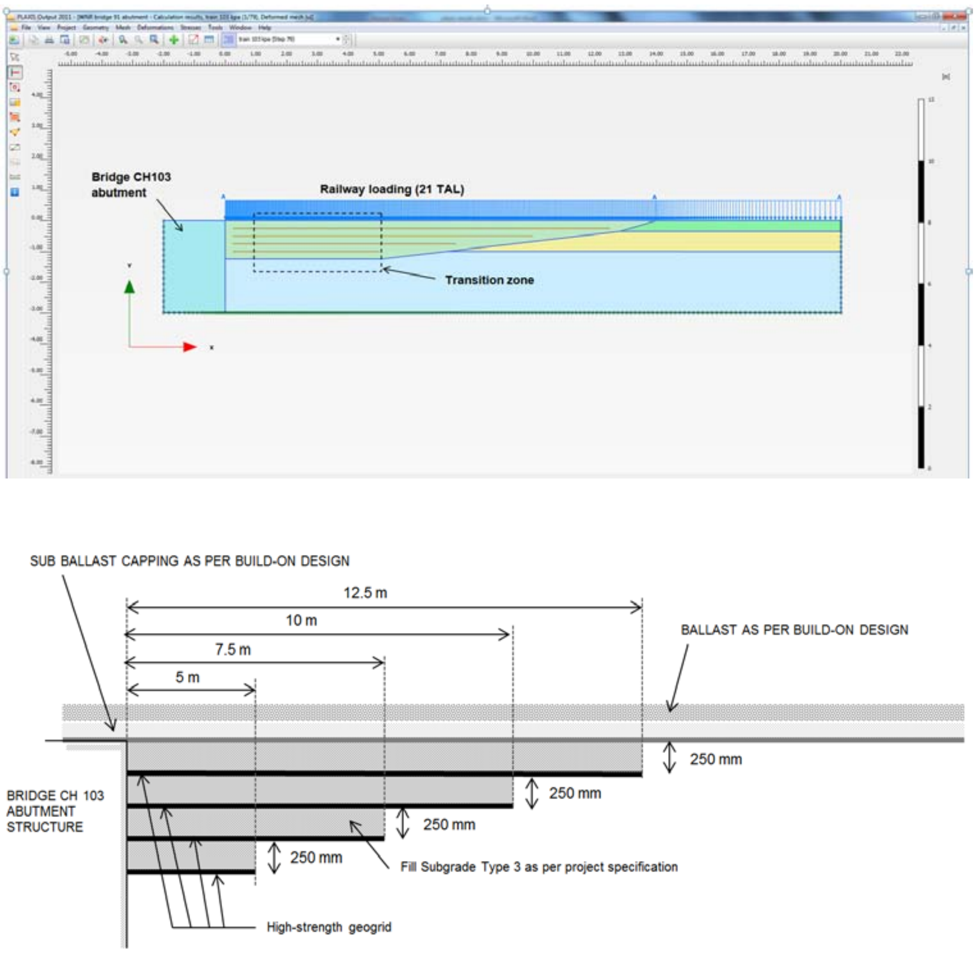


Figure 8: Formation to bridge transition at CH103 Bridge (E-W section): (a) FEM model; (b) Sketch of recommended solution.

# 7 BORROW SOURCE INVESTIGATIONS

A preliminary investigation of potential borrow material sources was carried out to confirm availability of suitable construction materials for both the N-S section and the E-W section of the project. Borrow source areas were targeted with preference for those areas adjacent to the rail line showing potentially favourable geological characteristics. Borrow source area spacing was maintained wherever possible to minimise haulage distances between specific chainages. The investigation was conducted in two phases (one for each section) initiated with a two day site visit during which potential borrow sources were identified and discussed with local area property owners. Selection of potential property owners was conducted by Brookfield Rail and based primarily on proximity to the existing rail corridor as well as property size (larger properties were considered to have greater potential to contain usable material sources).

Following the initial source identification and visual inspection of potential target sites, AECOM assessed each location via a gridded pattern of test pits and produced a preliminary estimate of potential material types and volumes from each source investigated. Only potential source locations that were located in previously disturbed land (i.e. excavate in existing paddocks without native vegetation) were included in this investigation.

## 7.1 FIELD INVESTIGATION OF POTENTIAL MATERIAL SOURCES

Field work was carried out at 18 locations on the East-West alignment and 40 locations on the North-South alignment. Test pits were dug at each location using an excavator. Test pit locations and depths were selected onsite based on the following criteria:

- Pit locations exclusively in degraded open paddock (no native bush),
- Maximum vertical extent of suitable material (as assessed in the field) or limiting depth of excavation equipment.

Logging and sampling (disturbed bulk samples) was conducted at each pit location to verify material properties and allow identification of areal variability in materials present. Laboratory testing included: particle size distribution (PSD), Atterberg limits, shear box testing, optimum moisture content (OMC), modified maximum dry density (MMDD) and California bearing ratio (CBR).

### 7.2 REPORTING OF INVESTIGATION RESULTS

Analysis and reporting of the material source investigation included the following basic information for each potential source:

- Identify the borrow source location and extent of resources
- Identify potential haul route(s) for transport of material to the proposed work as well as chainage intercepts along the existing rail corridor
- Following receipt of laboratory results from materials testing, assess materials classification information against the material grading selection.
- Classify each borrow source in accordance with material types provided in the specification (sub ballast capping or general fill).
- Prepare draft pit management plans for each client approved borrow source.

Overall the investigation proved an estimated volume of 2,000,000 m$^3$ of common fill type material, approximately 8% of which meets Subgrade or Sub Ballast Capping Material Type requirements. It should be noted that the earthworks materials specification was prepared considering the results of the borrow source investigations in conjunction with engineering requirements as per the basis of design for the project. The basis of design included specifications for common fill, subgrade, sub ballast capping and surface rock protection.

## 8 CONSTRUCTION ISSUES

### 8.1 EXCAVATABILITY

The excavatability of materials in areas requiring cuts for new formations was assessed as a function of strength and geometric average spacing of discontinuities of the rock mass, using the excavatability graph as derived by Pettifer and Fookes (1994). Point load tests were undertaken on selected rock samples collected from spoil materials which were excavated as cobbles and boulders. Average spacing was observed from the test pit walls where possible. A typical plot of excavatability data is presented as Figure 9.

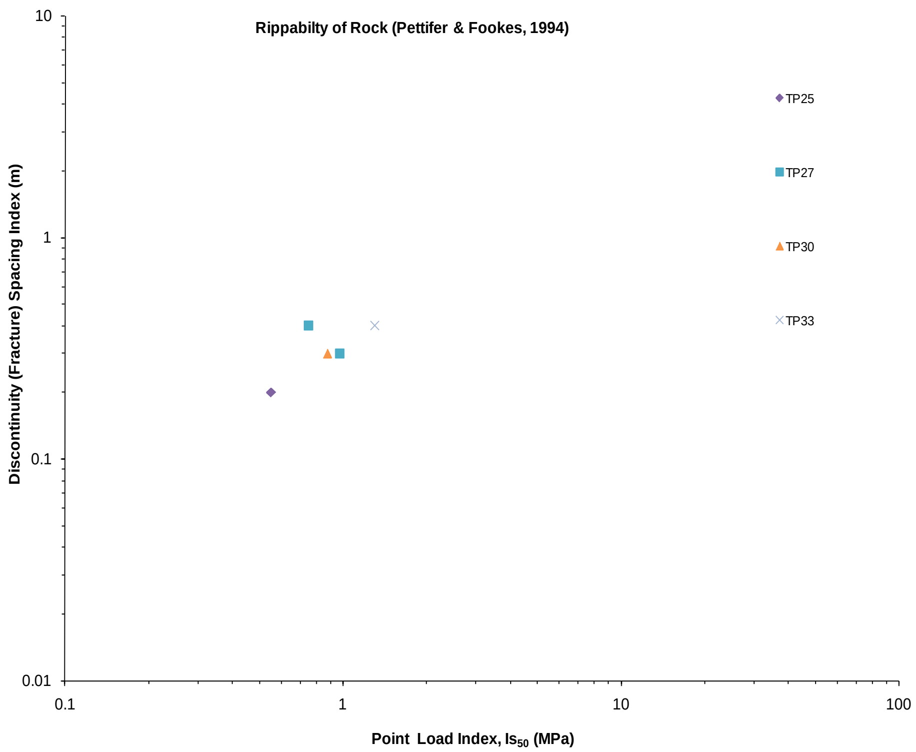


Figure 9: Excavatability assessment for a section in N-S alignment between Sullivan and Mullewa.

Assessment results indicated that the majority of proposed cuts would be directly excavatable with normal construction equipment without pre-treatment. Only limited areas showed the presence of hard ripping materials within depths of embankment foundation preparation. These high strength areas corresponded to cobbles of lateritic soil and highly to moderately weathered Gneiss with wide joint spacing (> 1 m). Heavy excavation plant was required in those sections.

### 8.2 EARTHWORKS

The main challenge encountered during earthworks was related to compaction moisture content control to mitigate the risk of saturation induced collapse and swelling of fill material. This was particularly difficult during periods of hot, dry weather with high ambient temperature in the mid 40's. Other issues affecting earthworks were regularly occurring high wind conditions and the availability of suitable water resources (some bores having very high salinity).

### 8.3 STABILITY OF EXISTING LINE DURING CONSTRUCTION OF DUPLICATION TRACK

A project requirement established that existing rail lines should be maintained operational during construction of duplication alignments. Geotechnical analyses were carried out to examine this stability issue relevant to sections of the N-S section alignment.

Review of the rail design drawings for the areas of duplication from Tilley to Canna and Sullivan to Mullewa showed that there were two potential embankment stability issues with regard to the construction of the new rail line while maintaining the existing line active:

- Type 1: Excavation of cut adjacent to the existing line. For this case, the maximum depth of excavation was 3 m
- Type 2: Excavation of boxed out areas for embankment foundation improvements adjacent to the existing line. This refers mainly to areas in the Irwin River and tributaries areas where special treatment of the foundation was required (Section 6.4).

Analyses were undertaken using the software Plaxis 2D (version 8.6). Plain strain analyses were undertaken using 15-node triangular elements. A staged construction approach was employed to simulate the activities during the construction of the rail embankment. The slope stability from the analyses was then assessed to determine the existing slope performance and the necessity for ground improvement or temporary works schemes.

Parameters for numerical analyses were selected from available CPTU and DCP testing and laboratory results from samples in test pits.

Factors of safety at different stages were calculated using a phi/c reduction method. Although the construction stage of the duplication line represented a temporary condition, the target factor of safety was maintained at 1.5 for any stage of the construction work based on consideration of the effects of potential failures or excessive deformations. An example output of stability analysis is presented as Figure 10. This study provided input to recommendations for suitable slopes and construction sequences in particular sections of the alignment.

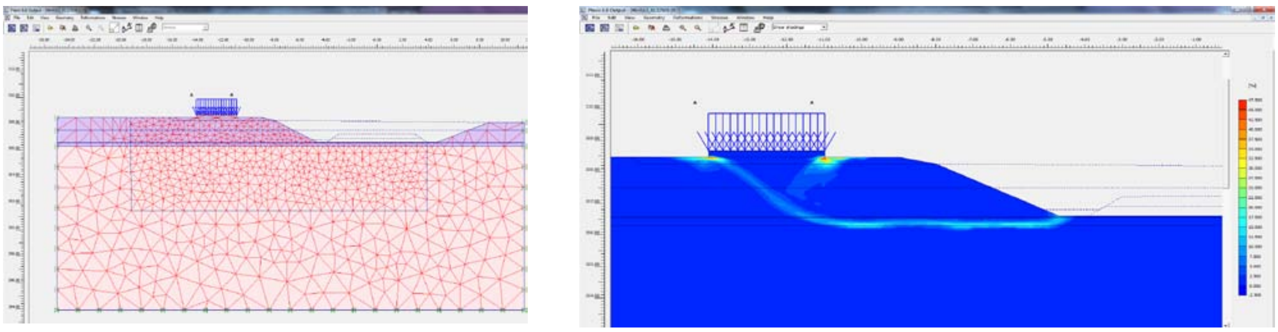

Figure 10: Construction stage before proposed embankment is placed: (a) FEM model; (b) Shear strain shading indicating FoS > 1.5 under train load and water level at surface of maximum excavation depth.

The results of the constructability study were also used to define management of groundwater at some sections (temporarily lowered during construction) and also to confirm/redefine requirements of foundation treatment in the areas of the Irwin River and its tributaries. Operations at reduced speed were recommended if water conditions became more critical (i.e. shallower) that those assumed in the analysis.

## 9 CONCLUSIONS

This paper presents the geotechnical aspects of the upgrade of existing rail infrastructure in the Mid-West region of Western Australia to better serve the current and future rapid growth in demand for iron ore exports from Western Australia. The successful completion of this project has aided successful integration of freight and export operations in the Mid-West. These developments will open Geraldton and the Mid-West region to customers across the globe, potentially strengthening long-term economic prosperity in the region and maintaining employment opportunities.

From a geotechnical perspective the Mid-West Rail Upgrade project included a complex mixture of upgrades to existing track combined with construction of new track and track duplication. The work involved integration of site investigations on existing formation and for duplication alignments, borrow source investigations, assessment of the capacity of existing bridge foundations, assessment of the suitability of existing formation and upgrades required to carry increased loading requirements, analysis of railway behaviour in salt lake areas and recommended ground improvement schemes and design of formation-to-bridge transition zones. A constructability study using finite element method was carried out to assess the stability of operational existing track during construction of the duplication of the line. Geotechnical support was provided to the client during the construction stage of the project.

The project is being commissioned at the time of this writing (August 2012).

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