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The nature, design considerations and construction performance of reactive soils for the Northern Missing Link Project, Bowen Basin

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ABSTRACT

Approximately half of the 69km of the Northern Missing Link (NML) rail line has been constructed on reactive soils. Through a variety of remote sensing techniques, field reconnaissance & mapping, subsurface investigations and laboratory testing the extent and nature of reactive soils (Gilgais whose origins may be alluvial or residual basalt) over the rail alignment has been defined. Challenges have arisen during the design phase in the assessment of these soils geotechnical properties, in design of appropriate extensive earthworks, including reuse within the rail embankment, dealing with potential detrimental shrink-swell characteristics and the prediction of performance over the design life of the project. A variety of techniques were considered to deal with significant shrink-swell issues, with height of embankment, construction of toe berms and limited dig-out-and-replace defined as the key solutions. Deriving appropriate solutions to these soils has had significant influences on overall rail alignment, the mass-haul, cost and programme.

The construction phase of the works was severely delayed by the 2008-2009 wet summer, but this allowed the dry cracking clays to regain moisture and reheal. During construction, the interpreted extents of the reactive soils was confirmed allowing a reassessment of the most appropriate subgrade preparation procedure, redefinition of the earthworks specification and ultimately the most practical and appropriate reuse of these excavated materials.

Keywords: railway, Gilgai, shrink-swell, earthworks design, construction

1 INTRODUCTION

The NML is part of the Goonyella to Abbot Point (GAP) Expansion Project a rail line to service the Bowen Basin coal mines. It will increase export capacity out of the Port of Abbot Point, from 21 to 50mtpa. The project involves the design and construction of 69km of new rail (termed the NML), three passing loops, a ballast siding, 11 rail bridges, two road-over-rail bridges, 2.3Mm^3 of earthworks and some 80 culverts varying in size from 1 to 2.5m diameter with up to a dozen pipes. The construction of the NML reached practical completion in December 2011 and is now fully operational.

The published soil maps indicate that the NML alignment is covered by a variety of reactive clays defined as either Gilgaied or deep to shallow cracking clays. The published geological maps show the alignment underlain by Permian sedimentaries, and Tertiary sedimentary and volcanic rocks, including basalt lava flows.

Where reactive, moisture sensitive clays have been deposited (Gilgais) or developed (basaltic residual soils) on flatter more distal colluvial slopes and flood plains, Gilgai's develop. These have a characteristic hummocky appearance with deep polygonal desiccation cracks (Plate 1) and 'melon holes' prevalent. These surface features develop in arid or semi-arid environments by marked seasonal changes due to wetting and drying within the soil profile. Fox (2000) indicates for the NML alignment the soil suction profile in these deposits could be in the range of 2.5 to 3m depth. The periodic wetting and drying and associated changes in moisture content and associated swelling and shrinking effects results in the development of fissuring (often exhibiting polished or slickensided surfaces) and cracking possibly over the full depth of the soil profile (see Figure 1). The cracks may be extensive and wide at the surface (Plate 1) and allow avenues for root growth and for blown granular dominated soils to be deposited within them. As a result, sealed cracks within the soil profile are often lined or distinguished by the occurrence of orange-yellow vertical sand bands. Another characteristic feature is the self-mulching of the uppermost surface layers producing a sugar-cubed effect (Plate 2).

The afore mentioned repeated shrinking and swelling, tends to churn the surface, mixing the organic matter into the upper soil horizon, creating what is known as a 'self-mulching' soil.

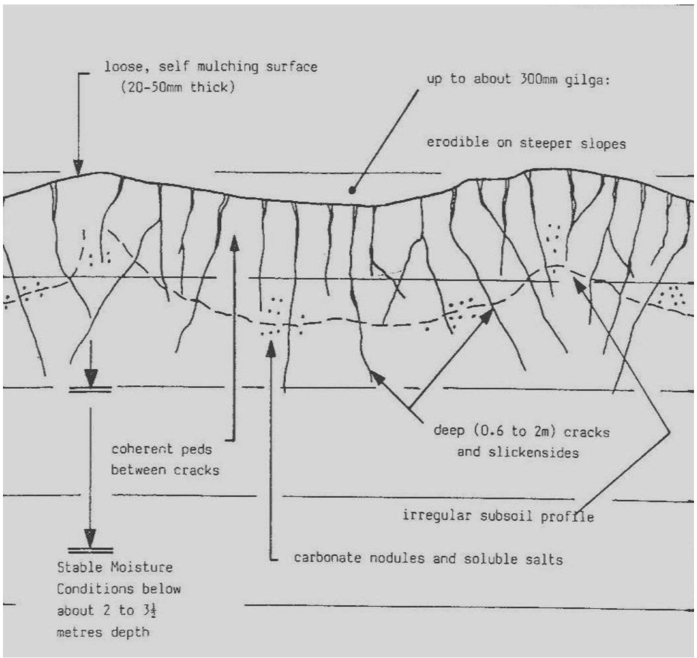


Figure 1 Gilgai Typical Characteristics (Wallace 1988)

Plates 1& 2 Gilgai Occurrence

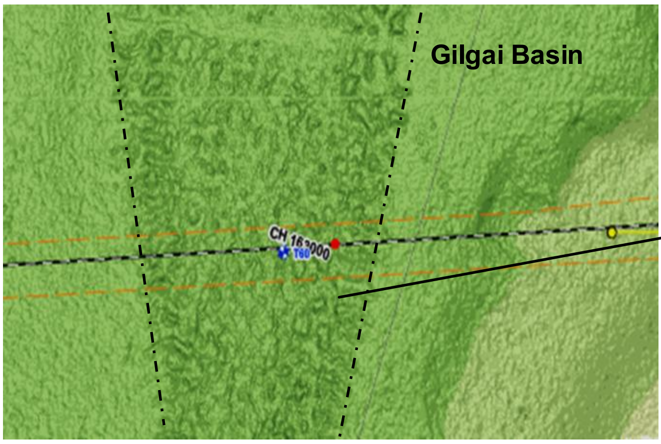
2 INVESTIGATION & IDENTIFICATION OF REACTIVE SOILS

A detailed desk study including aerial photography interpretation and review of LiDAR relief shade maps (see Figure 2), coupled with published soil maps, enabled a preliminary assessment of the extent of reactive soils to be made. This initial model was further refined by field mapping including the use of changes in vegetation patterns (blue grass and acacia bush prevalent on reactive soils – see Plate 3), from which a subsurface investigation was designed and implemented. Test pitting (to a maximum of 3.7m, and generally at 400-500m spacing) and Dynamic Cone Penetration (DCP) tests, at 250m spacing, were undertaken primarily to identify the depth and character of cracking/fissuring, to recover suitable samples for laboratory testing (including index, compactions, Californian Bearing Ratios (CBR), shrink-swell, and lime demand tests), and for definition of effective CBRs for rail pavement design (AustRoads 2008). The subsurface investigations revealed reactivity to depths exceeding 2m characterised by cracking, fissuring, and slickensides on discontinuity faces. The cracking has allowed roots to penetrate the full profile thickness. These investigations were carried out between late March and July 2008 prior to the abnormally wet season experienced in summer 2008/2009. Based on the remote sensing, field mapping and subsurface investigations 34.25km of the 69km rail alignment (50%) was found to encompass reactive soils either occurring as Gilgai basins (melon holes up to 900mm deep) or gently undulating grasslands composed of residual basaltic soils.

3 GEOTECHNICAL DESIGN CONSIDERATIONS

One of the main cost driving issues for the NML was the reactive soil character and its potential effects on the proposed works & related mitigation measures. The main design considerations were cuttings, embankments, shrink-swell behaviour (particular directly below low embankments or at grade works and turnouts) and reuse in earthworks.

All cuttings required for the alignment in these materials were <3m and as such the main controlling factors to ensure performance over time was to minimise potential detrimental effects of rain drop action and overland flow (OLF). To this end, slope batters were designed at 1v:3h with all batters vegetated.



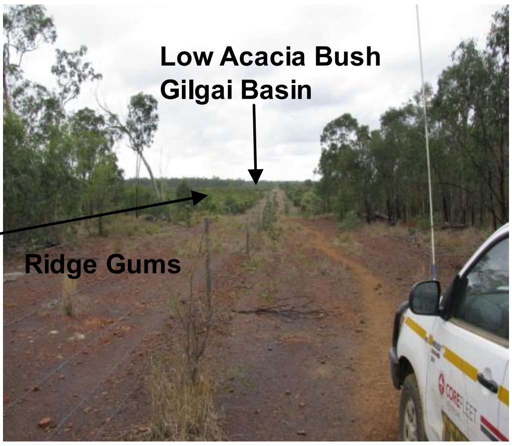


Figure 2 LiDAR Relief Shade Maps - Gilgai Identification

Plate 3 Gilgai Basin Vegetation

The shrink-swell behaviour of reactive soils is a result of a change in the moisture content during dry and wet periods causing the clay minerals to expand or contract. However, at the equilibrium moisture content, little or no volume change is likely to occur. Ideally, the construction moisture content should be at or near the material equilibrium moisture content for these materials. This is, however, not always practical as research shows that the equilibrium moisture contents of these soils can be as high as 40% above the standard compaction optimum moisture content of the material (Look, 1995). Such high construction moisture contents are likely to compromise the strength requirements of the embankment and cause trafficability issues. It is therefore essential that a balance between the shrink-swell and strength requirements be achieved.

The shrink-swell movements are more prevalent within the upper portion of the soil profile that is subject to environmental influences (referred to as the environmental influence zone). This influence zone as indicated previously can be as deep as 3m in a semi-arid environment (Fox, 2000).

An initial heave is likely to occur if the foundation material is dry of the equilibrium moisture content at the time of construction. Little can be done to prevent these initial movements other than to remove and replace or to modify the material. Apart from initial movements, seasonal shrink-swell movements may occur at the edge of the embankment footprint as a result of seasonal influences. These movements are likely to extend to below the embankment and may cause edge flexing and longitudinal cracking. If not adequately controlled, this could be an ongoing occurrence. The central area beneath the embankment is likely to tend towards an equilibrium moisture condition, provided that the verge is thick enough to prevent seasonal drying.

The insitu materials along the centreline of the alignment were assessed for their potential for volume change. The assessment included an evaluation of the material properties and swell tests . Based on the test results the Residual Basalt appeared to be the most reactive, however although the Gilgai soils indicate a less reactive material, the formation of depressions in the landscape suggested severe surface movements.

There are a number of methods to predict the potential heave of expansive materials available internationally. Most, if not all of these methods are empirical in nature and highly dependent on the input parameters. The Australian Standard Method 2870 – 1996 is the National Standard to determine the site classification of residential slabs and footings. The method determines the potential surface movement based on the Shrink-Swell Index and depth of moisture suction change of the material.

Estimates of the potential initial and seasonal movements for the recommended embankment cross-sections were determined. It is important to note that the heave estimations are based on the assumption that the foundation is dry of equilibrium conditions at the time of construction. The general findings of this analysis were : a) full depth basaltic clay (highly reactive, Iss: 6% – 9%) - potential initial movements typically ranged between 45 to 150 mm, with seasonal movements ranging between 15 to 45 mm; b) full depth clay (less reactive, Iss: 3% - 6%) - potential initial movements typically ranged between 25 to 100 mm, with seasonal movements ranging between 10 to 30 mm; and c) limited depth clay over weathered rock (assume Iss: 3% to 9%) – potential initial movements typically ranged between 0 to 80mm, with seasonal movements ranging between 0 to 40 mm. The initial movements are likely to occur over a 5 year period. The seasonal shrink-swell movements are, however, likely to be ongoing.

3.1 The effect of remove and replace on the potential for heave

Removing the reactive material and replacing it with non-reactive fill material was designed to significantly reduce the surface movements. This was achieved by a) removal of the most expansive material in the upper section of the profile and replacing it with a stable fill material; b) changing the suction profile of the material; c) the fill material provides cover to the clay and reduces environmental influence; and d) the fill material provides an overburden pressure that can help to resist the potential heave pressures.

3.2 The effect of overburden pressure on the potential for heave

The effect of overburden pressure to reduce the potential heave of a material is difficult to quantify. Literature suggests a conservative approach of applying a reduction factor of 30% per metre of embankment, ignoring the 1st metre of embankment (TRH 9, 1982). This approach was used to determine the estimated surface movements . It should be noted that swell pressures of as high as 280kPa have been measured on some of the basaltic soils. One metre of embankment typically equates to approximately 20kPa of overburden pressure. This would equate to an embankment height of approximately 15m to resist the potential swell if the expansive material is exposed to moisture ingress. However, the cover provided by the embankment material was deemed to reduce the influence of moisture on the expansive clays and therefore a lower embankment height would be required.

The potential heave estimates show that the seasonal movements were generally within tolerable limits i.e. below the 30mm QR National target values, except for the very highly expansive materials with a Shrink-Swell Index of 9%. The calculated potential initial movements were, however, in excess of 30mm and likely to adversely affect the operational and maintenance requirements of QR National. It should be noted that these movements were perceived likely to only occur where the foundation material was dry and cracked at the time of construction. It was therefore recommended that the design methodology adopted included mitigation measures to address the initial and seasonal movements.

4 COPING WITH SHRINK-SWELL BEHAVIOUR

A number of treatment alternatives to improve reactive soil stiffness for capping layer design and to cater effectively with potential detrimental shrink/swell characteristics (initial and seasonal movements), as well as to optimise material usage along the alignment, were reviewed from which the choice of dig-out-and-replace was chosen as the optimum methodology. The potential alternative solutions considered included: a) lime stabilisation (of a 300mm thickness of the Gilgai or basaltic residual soils under the capping layer and increasing this to 500mm directly below the edges with a typical improvement value of CBR 20); b) the use of heavy compacting plant; and c) excavation of buffer trenches or moisture barriers (Tarong Transport Alliance 2007).

The adopted dig-out-and-replace design solution recommended was as follows: a) remove and replace dry and cracked material at the base of the embankment to a depth as determined by the Geotechnical Representative on site. This approach was considered to be pragmatic and allowed taking into account the existing moisture condition of the materials at the time of construction. The removal of dry and cracked material would be irrespective of embankment heights; b) remove and replace material at the foundation of the embankment where the height to formation level is less than 1550mm. This removal depth varied between 0 and 900 mm, depending on the height of the embankment; c) construction of 2500mm wide x 500mm thick berms at the toe of the embankment to reduce the environmental influence at the edges of the embankment (by increasing the flow path length of over land flow to limit significant changes in moisture content of reactive core or underlying reactive soils, as well as increasing overall stability); d) reactive soils are only placed in zoned embankments where it is covered by a minimum of 2500mm non-reactive material; and e) remove and replace reactive material below culverts and loop turn-outs to a depth based on the thickness and moisture condition of the reactive soil (in turn-outs due to more stringent settlement targets a 2000mm thick zone of material was removed and replaced). In addition, as previously indicated, the Gilgai basins were also characterised by the presence of large roots in the upper horizons. From previous project experience a maximum 2% (total mass) of organic matter was used as a control, with dig-out-and-replace undertaken where this was exceeded during construction.

5 CONSTRUCTION OBSERVATIONS

When your construction site is 69km long in a remote part of central Queensland, ground conditions, and cut and fill distribution become critical to your project success. A significant cost driving and risk issue for the construction and performance of the NML was the reactive soil characteristics and its effects on the proposed works and related mitigation measures.

5.1 Extent of reactive soils

One of the critical early construction tasks was to confirm embankment foundation conditions to ensure that construction works were not delayed and to allow optimisation of the mass haul. The extent of the reactive soils was confirmed by additional test pitting onsite and laboratory analysis allowing a reassessment of the most appropriate subgrade preparation procedure. The geotechnical mapping undertaken to determine the extent of the reactive soils was found to be highly accurate with just under half of the 69km of new rail underlain by reactive soils.

Two classes of expansive soils were identified, black soils (derived from basalt weathering profiles) and Gilgai, (termed locally as “melon holes”) occurring in larger areas of floodplains commonly indicated by well-developed hummocks and hollows at the ground surface. Typical soaked CBRs in these reactive materials ranged from 1% to 3%, Liquid Limits 75% to 100% with Plasticlty Index >35%, however crack depths were typically only 200 mm below the Topsoil, not up to 1.5 m as initially observed during the field investigations undertaken after several years of drought. The crack depth was much less than anticipated due to the preceding very wet summer allowing the dry cracking clays to regain moisture and swell. This confirmed the adopted dig-out-and-replace design solution as the most cost effective as typically only the upper 200 mm of dry cracked material needed to be removed and replaced, which also removed the worst of the root zone (Plate 4). The underlying soil was found to be at or near the optimum moisture content to a depth in excess of 4m.

Plate 4 Typical Gilgai Cracked Soil & Root Zone

5.2 Reuse of reactive soils in a zoned embankment

Although only a 200mm thickness was excavated and replaced to prepare the embankment foundations, this still produced a significant amount of very low quality material that either needed to be spoiled or incorporated into a zoned embankment (Plate 5). Due to space constraints and limited fill sources as much material as practical needed to be reused for embankment construction.

Plate 5 Gilgai Boxout

A zoned embankment developed during the design phase required the reactive materials to be placed in the core of the embankment fully encapsulated by non-reactive material. Strict moisture and compaction guidelines were enforced on the placement of this core material to prevent any future changes in soil moisture resulting in shrink or swell. It was found if the *in situ* moisture content of the clay was close to the optimum moisture content compaction and moisture requirements were easily achieved however care was required not to over compact the material as it had a tendency to commence heaving. If however, the material was found to be dry of optimum it was almost impossible to add moisture to meet the moisture requirements as added water would only penetrate a few mm into the material, leaving the bulk of the material dry and unusable. This was also an issue during the ground surface treatment of the embankments on reactive clays as the strict moisture requirements of the foundations could not be met when material was dry of optimum. This resulted in a significant volume of reactive clay that was allocated for use as core construction having to be spoiled. This resulted in massive spoil dumps having to be constructed and the importation of additional material.

6 CONCLUSION

Due to reactive soil shrink-swell behaviour, and associated potential detrimental effects on long linear infrastructure projects, the importance of accurate definition of the horizontal extents and related soil suction profiles of these soils is vital in the adoption of appropriate designs and related construction methodologies. The identification of optimum design solutions for reactive soils on the NML has been defined by the interpreted ground conditions, the rail alignment, mass-haul considerations, the construction program, cost and QR National acceptance criteria. The adopted techniques are believed to significantly reduce the risks of the shrink-swell characteristics of reactive soils causing potential detrimental settlement or distortion of the formation. During construction, the vertical cracking extents of the reactive soils were found to be significantly less confirming boxout and replace as the most appropriate subgrade preparation procedure, with strict control of moisture content and compaction.

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