

GEOTECHNICAL STUDIES FOR THE ADELAIDE RAIL REVITALISATION PROJECT: A CASE STUDY COMPARING IN-SITU AND LABORATORY TESTING OF BALLAST AND SUBGRADE MATERIALS

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SUMMARY

The South Australian Government has committed \$2 billion over the next 10 years to the Rail Revitalisation Project for electrification of its Adelaide passenger rail network. This includes an extensive rejuvenation program comprising ballast, rail and sleeper replacement. The Department for Transport, Energy and Infrastructure (DTEI) commissioned Parsons Brinckerhoff (PB) to undertake geotechnical and environmental investigations over 104 km of rail network to provide information on track and drainage conditions and contamination levels of ballast and subgrade materials for the track reconstruction works.

The investigation program covered a wide range of Adelaide soils and comprised over 100 boreholes and 700 test pits excavated through the ballast into the subgrade. Laboratory tests were undertaken on ballast and subgrade samples and included soaked and unsoaked California Bearing Ratio (CBR), total suction, and soil and pavement material classification tests. In-situ testing included pocket penetrometers, Dynamic Cone Penetration (DCP), Clegg Hammer, Light Weight Falling Weight Deflectometer (LWFWD) and Humboldt GeoGauge stiffness test methods.

The paper presents details of the investigative methods undertaken to collect and present the data, compares laboratory tests with in-situ test results, describes the main causes for track stability problems and recommends potential design and construction solutions.

1 INTRODUCTION

With the aim of increasing patronage of the rail network by using electric trains and as part of the Rail Revitalisation Project, the Department for Transport, Energy and Infrastructure (DTEI) commissioned Parsons Brinckerhoff (PB) to undertake geotechnical and environmental investigations of the Gawler, Tonsley, Grange, Outer Harbor and Noarlunga rail corridors. This rail network is operated by TransAdelaide. The main causes for track stability problems are generally poor track drainage, poor subgrade materials (both fill and natural), fouled ballast and insufficient ballast thickness. The project involves installing gauge convertible concrete sleepers and improving the ballast, capping and subgrade conditions on the TransAdelaide rail network. Comprehensive geotechnical and contamination investigations provide essential information to deliver a sustainable track formation.

2 SITE INVESTIGATION PROGRAM

The site investigation was undertaken in three stages comprising a track walkover, a test pit program and a borehole program. The frequency and rationale for the geotechnical fieldwork program was based on best practices for rail

investigations and sampling Adelaide's highly reactive soils. This program was determined jointly by PB and DTEI prior to commencing the work. All fieldwork was conducted under strict safety procedures required when operating within the rail corridor, with all field staff undertaking Track Safety Awareness training.

Documentation provided to DTEI included a Project Execution Plan, Risk Management Plan, Quality Management Plan and Health, Environment and Safety Plan.

Access to the track for test pitting and drilling was limited to the period between the last train service of the night and the first train service of the morning, generally between midnight and 5 am. Additional activities such as track maintenance, also limited the time for fieldwork on some nights. Field teams were rostered for 10 nights work then 2 nights off based on TransAdelaide track protector rosters. A large dedicated team was formed which allowed rotation of field staff into support roles in the office over the 6 month field program carried out during winter and spring. Information from field staff was transferred each morning to office support staff. This process ensured continuous laboratory testing and reporting progress to meet challenging and changing tight project schedules.

2.1 Track Walkover

A daytime track walkover by a Geotechnical Engineer was undertaken prior to all night time intrusive works. The walkover involved assessing track conditions, drainage performance, confirming access points and identifying locations for specific geotechnical investigations.

The proposed test locations were confirmed during the walkover, cleared of underground services and marked out on the sleepers. A Geospatial Information System (GIS) series of coloured maps were used to mark the locations of all test sites, drainage points of interest, contamination, ballast fouling and track pumping areas observed during the walkover. All points of interest were input into the GIS database after the walkover.

During the walkover a number of areas of pumping and fouling of the track were identified. The areas of pumping tended to be where evidence of greater than 50 mm of vertical movement had been occurring over 4 or so sleepers. Movement was identified by gaps between sleepers and ballast, mud on the rails and sleepers, and materials pushed up by sleeper movement. In extreme cases water was visible in the gap between the sleeper and the ballast.

From review of the underground service plans and observations made during the walkover, some of the pumping areas coincided with underground assets. Areas identified as fouled ballast during the walkover contained excessive fines visible in the ballast matrix (Figure 1).



Figure 1: Walkover inspection with track pumping

2.2 Test Pit Program

The test pit program included the excavation of over 700 test pits to a maximum depth of 3 m. All test pits were supervised and logged by a Geotechnical Engineer. All encountered ballast and soil units were logged and pocket penetrometer tests were also undertaken on cohesive materials. Representative bulk samples were collected from each test pit and submitted for

laboratory testing. Each test pit was photographed for visual reference.

Test pits locations were generally located at approximately 100 m intervals where dual tracks occurred and alternated between up and down tracks on both sides of the tracks.

The test pits were excavated by a backhoe using a 300 mm wide bucket and were located parallel to the rails at the end of the sleepers (Figure 2). Test pit and lithology depths were recorded from the top of the adjacent sleepers. Test pits were first excavated through the ballast to the top of the natural material.



Figure 2: Test pit excavations

A Dynamic Cone Penetration (DCP) test was then undertaken to a depth of at least 900 mm or refusal, recording the number of blows per 100 mm of penetration (Figure 3). DCP results provide a direct correlation to in-situ subgrade strength such as the California Bearing Ratio (CBR)^[1].

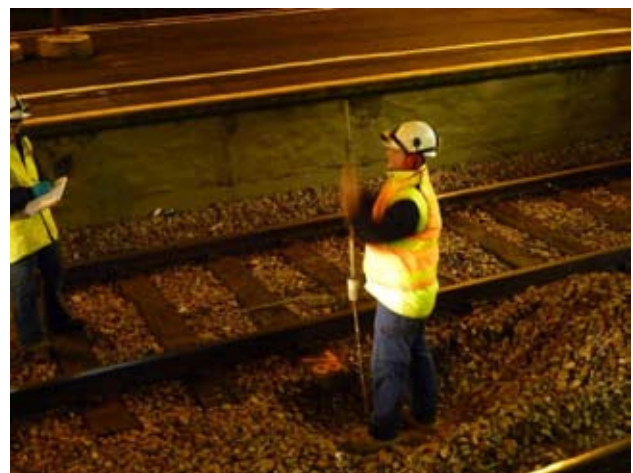


Figure 3: DCP testing into subgrade

The backhoe bucket was subsequently used to scrape ballast material from between the sleepers where it was shovelled into sample bags for laboratory testing. This was found to be the best methodology for sampling the ballast and fines.

Generally the excavations through the ballast encountered several “layers” of ballast and other formation materials (sub-ballast, capping and subgrade), including slags, coal cinders, ash,

gravels and soils placed during past construction, maintenance and upgrades of the tracks.

Where a clearly defined upper clean ballast layer was encountered the layer was separately sampled. In almost all cases the ballast was fouled (except for the surface layer of material) and was generally comprised of one or more layers of ballast (and sub-ballast) material excavated from above the natural material (see Figure 4 for an example).



Figure 4: Layers of ballast, sub-ballast, cinders, fill and subgrade

At many test locations a clearly defined layer of subgrade fill material was encountered above the natural material and was identified as such. In rail track terminology 'subgrade' materials may include both fill and natural materials, whilst geotechnically they are separate materials and were described, where possible, separately in the reports.

After ballast sampling and DCP testing, the test pits were extended to the final depth of at least 0.5 m into the underlying natural material, with samples taken from the backhoe bucket. Pocket penetrometer tests were conducted on both the test pit walls and on selected relatively undisturbed samples of cohesive material collected from the backhoe bucket.

The test pits were then progressively backfilled and compacted so as to retain the original order of layered materials, with compaction being continuously undertaken by the backhoe bucket. The backfilled material was tested using the DCP to confirm that compaction achieved a density equal to or greater than the pre-excavation conditions.

Test pits excavated within railway station confines occasionally encountered cement stabilised subgrade materials on which the backhoe and DCP refused and no subgrade or natural material samples could be collected.

2.3 Borehole Program

The borehole program included the drilling of over 100 boreholes to 3 m depth. The boreholes were

drilled using the dynamic push tube technique with a 4WD utility mounted drill rig (Figure 5).



Figure 5: Drilling into rail formation

The boreholes were drilled between the rails and adjacent to test pit locations, at a spacing of approximately 500-1000 m spacing along the railway line and were supervised and logged by a Geotechnical Engineer. The boreholes were initially extended through the ballast using a large diameter solid auger. Once the hole was cleared of ballast by the auger, the boreholes were extended using hollow push tube sampling to 3 m depth. The materials were extruded into plastic core trays, photographed, logged and sampled for geotechnical laboratory testing including soil suction tests. Pocket penetrometer tests were undertaken on cohesive materials.

2.4 Test Location Survey

All test pit and borehole locations were surveyed during the services check and walkover. A RTK GPS provided horizontal accuracy of ± 0.1 m to the Map Grid of Australia coordinate system (MGA94) and vertical accuracy of ± 0.5 m to Australian Height Datum (AHD). Cross checking of the RTK GPS accuracy was undertaken by picking up known survey points along the rail corridor.

3 LABORATORY TESTING

Samples were generally collected in two 20 kg bags from each test pit, one containing ballast and one containing subgrade or natural materials.

3.1 Ballast

The top section of all ballast material encountered at each test pit location was sampled and tested for particle size distribution (PSD) and particle density (t/m^3). One in five ballast samples were tested for Los Angeles Abrasion (LAA) to determine compliance to ballast strength specifications. One in ten ballast samples were also tested for particle shape to determine the extent of degradation of the ballast.

The laboratory testing was undertaken to determine compliance to ballast specifications and provide material classification for potential re-use

of the material in accordance with DTEI specifications.

Results from the laboratory testing were also used to determine the extent of ballast fouling. Selig and Waters^[2] provide a fouling index, F_i , based on the results of ballast grading where $F_i = P_4 + P_{200}$ where P_4 = percentage passing the 4.75mm sieve, and P_{200} = percentage passing the 0.075mm sieve.

Categories of fouling are reproduced as Table 1 below.

Category	Fouling Index F_i
Clean	<1
Moderately clean	1 to <10
Moderately fouled	10 to <20
Fouled	20 to <40
Highly fouled	≥ 40

Table 1: Fouling Index Categories ^[1]

Separate samples of the ballast were also tested for contamination.

3.2 Subgrade and Natural Materials

Each test pit subgrade or natural material sample was tested for 4 day soaked California Bearing Ratio (CBR), compaction to standard maximum dry density (SMDD), optimal moisture content (OMC), Atterberg Limits, field moisture content (FMC), and particle size distribution (PSD). One in two samples was also tested for un-soaked CBR to provide design correlation parameters. CBR samples were compacted to 95% SMDD with a 4.5 kg surcharge. Natural soil samples from the boreholes were tested for PSD, Atterberg Limits and total soil suction.

The laboratory testing was undertaken to provide bearing capacity design parameters of the subgrade and natural materials for rail pavement design and also to provide material classification for potential material re-use in accordance with DTEI specifications. Separate samples of the subgrade and natural materials were also tested for contamination.

4 RAILWAY FORMATION

Design of railway earthworks is highly dependent on the geotechnical properties of the subgrade soil materials (both fill and natural) that support the railway formation. The depth of ballast and track formation depends on the strength of the underlying natural soils, with weaker soils requiring thicker track formations to distribute the loads.

Soils with poor drainage are generally more saturated and weaker and also require thicker track formations. A comprehensive understanding of the soils and drainage conditions along the existing track formation was essential to designing the most economical upgrade with the least maintenance for the Adelaide rail network.

4.1 Current Specifications

The design of earthworks for the TransAdelaide network is based on Part 1040 Formation and Earthworks^[3] specifications for capping, structural, general and unsuitable fill criteria. These also reflect similar Australian Rail Track Corporation (ARTC) specifications. The encountered subgrade materials (both fill and natural) were all assessed for suitability of meeting these specifications.

General fill, structural fill and capping materials are all specified based on gradings, characteristics of the materials clay fraction (such as Liquid Limit, Plasticity Index and activity indices) as well as soaked CBR values. A minimum of 250 mm of ballast below the sleeper soffit is required on at least 200 mm of capping material (CBR>50%).

Depths of capping and structural fill are dependent on the soaked CBR of the fill or subgrade materials. Subgrade materials with soaked CBRs ranging from 1% to 3% require a 1,000 mm structural fill layer, while subgrade materials with soaked CBRs ranging from 3% to 8% require a 500 mm structural fill layer. Subgrade or fill materials with a soaked CBR exceeding 8% may have direct placement of capping materials with no structural fill layer.

Unsuitable materials (soaked CBR values less than 1%) found in the subgrade are to be removed to a depth of at least 300 mm prior to replacing with 1300 mm of structural fill.

4.2 Current Track Condition

As the entire Adelaide rail network was constructed prior to the 1980's, track formation configurations of capping, structural and general fill were rarely encountered during the investigation program.

The investigation program encountered many ballast and subgrade configurations along the railway line. Generally the ballast was clean on the surface only and fouled ballast was underlain by sub-ballast layers of slag and coal cinders, and even buried ballast and sub-ballast layers. The depth of the ballast and sub-ballast materials generally exceeded the height of the rail embankment, which indicated that the track formation was forming a "bath" configuration similar to that sketched in Figure 6.

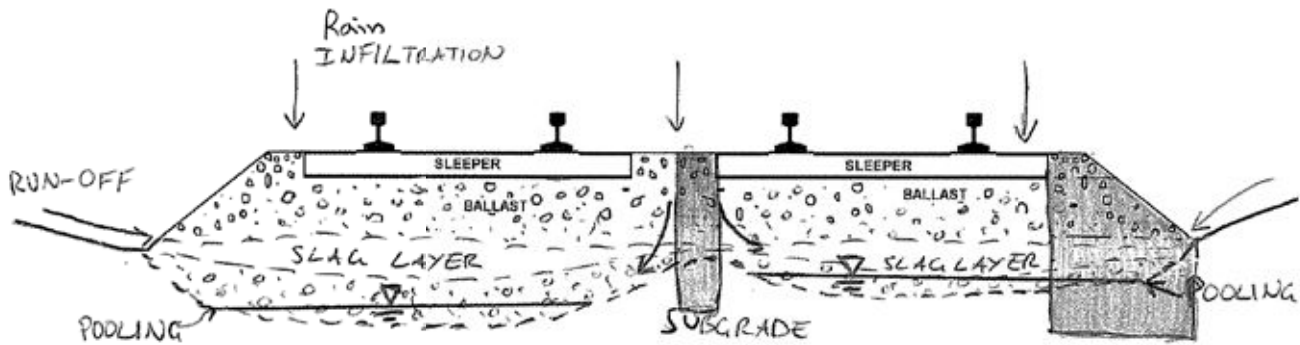


Figure 6: Typical track formation configuration

This rail track configuration increases infiltration of rainwater, not only from the poorly drained sides of the embankment but also through the ballast material, into the sub-ballast and subgrade materials.

The location of a ‘perched’ water table (Figure 6) within the ballast formation severely weakens clay rich subgrade materials, further facilitating the settling (or punching in) of the ballast formation into the subgrade. Track formation settlement also causes additional fines to migrate into the ballast and sub-ballast material. This process results in track pumping (Figure 1).

Erosion of cut batters containing dispersive soils was also identified as a source of ballast fouling along the Noarlunga line.

The investigation program identified a number of drainage issues along the track. Drainage systems were generally not well constructed causing blockage in low points due to the lack of suitable outlets. Other areas were found to be poorly maintained and contained build up of silt and fine materials with no mechanism to divert water from the track formation. Culverts were also blocked, damaged or eroded due to lack of head and wing walls (see Figure 7).



Figure 7: Poor drainage and blocked culvert

4.3 Laboratory and In-situ Test Results

A summary of the laboratory and in-situ test results on ballast and subgrade materials are presented below.

4.3.1 Ballast

Results from 652 ballast samples were assessed for compliance to specifications outlined in Section 4.1 (Class N for track traffic between 1 million and 6 million gross tonnes per annum^[3,4]) and are summarised below:

- 12% of the ballast did not meet LAA strength requirements of less than 30%.
- 35% of the ballast did not meet particle shape requirements of less than 30%.
- 19% of the ballast did not meet particle density requirements of greater than 2.5 t/m³.
- The average fouling index F_1 was 16 and ranged from 0 to 81. Table 2 below demonstrates the variability of the fouling index over the network and major lines.

Category	Fouling Index F_1	Total Network	Gawler Line	Noarlunga Line
Clean	<1	4%	2%	8%
Moderately clean	1 to <10	28%	26%	34%
Moderately fouled	10 to <20	35%	38%	31%
Fouled	20 to <40	30%	32%	24%
Highly fouled	≥ 40	3%	2%	3%

Table 2: Network Fouling Index

- Only 4% of the ballast samples meet grading specifications.
- The percentage of fines (<75µm) in the ballast averaged 4% and ranged from 0% to 37%.
- The percentage of sand fraction (>75µm to 4.75mm, based on the fouling index criteria)

in the ballast averaged 8% and ranged from 0% to 46%.

The above results indicate that commonly the fouling of the ballast may be more influenced by the mechanical breakdown of ballast materials generating sand size fractions, combined with varying amounts of ingress of silts and clays from the subgrade, side drainage and slopes.

4.3.2 Subgrade and Natural Materials

Results from 642 subgrade samples were assessed for compliance to specifications outlined in Section 4.1 and are summarised in Table 3.

Property	unit	Ave	Min	Max
Fines (<75µm)	%	59	2	96
Sand (75 µm to 2.36mm)	%	33	2	98
Gravel (2.36mm to 63mm)	%	7	0	96
Field Moisture content	%	19.2	2	37
Optimum Moisture content	%	19	7.5	36
4 day soaked CBR	%	4.1	0.5	31
Unsoaked CBR	%	6.3	1	30
Standard Maximum Dry Density (SMDD)	t/m ³	1.7	1.31	2.04
Plasticity Limit (PL)	%	17	8	48
Liquid Limit (LL)	%	39	30	104
Plasticity Index (PI)	%	22	1	81
Linear Shrinkage (LS)	mm	9.7	0	69
Pocket penetrometer (PP)	kPa	247	25	600+
DCPT	Blows/100 mm	4.8	0	28
	CBR %	10.2	0	75

Table 3: Summary of subgrade results

- Field moisture contents were high reflecting the winter and spring seasonal conditions when tested (wettest winter in four years).
- Field moisture contents were generally slightly higher than optimum moisture contents.
- 83% of the field moisture contents were higher than the Plastic Limit for the soils, with an average of 3.4% higher.
- Over 50% of the soaked CBR results were less than 3% while only 20% of unsoaked CBR results were less than 3%, as shown by Table 4.
- 89% of unsoaked CBR results were higher than soaked CBR results, with an average of 2.5% increase.

CBR range	Soaked CBR % of samples	UnSoaked CBR % of samples
<3%	51%	20%
≥3% - <8%	43%	60%
≥8%	6%	20%

Table 4: Summary of subgrade CBR results

- In-situ CBR results derived from DCP tests were on average 6% higher than soaked CBR and 4% higher than unsoaked CBR values.
- A small percentage of subgrade materials would meet structural fill requirements (minimum of 8% CBR) and no materials would meet capping material requirements (min 50% CBR).
- The rail network covers the full range of soils but was dominated by clays and clayey sands, as shown by Table 5.

Unified Soil Classification	Percentage of samples
CI, intermediate plasticity Clays	26%
CL, low plasticity Clays	26%
SC, clayey Sands	23%
CH, high plasticity Clays	15%
GC, clayey Gravels	4%
SP/SW, Sands	2%
GP, Gravels	1%
ML, Silts	1%
SM, silty Sands	1%
GM, silty Gravels	<1%

Table 5: Summary of subgrade soil units

- Clays recorded standard maximum dry densities (SMDD) in the lowest range, between 1.31 t/m³ to around 1.62 t/m³, whilst the sands and gravels ranged from 1.4 t/m³ to 2.04 t/m³.

As described above, the majority of the natural clay soils had high moisture contents due to poor drainage along the railway corridor that did not facilitate the diversion or drainage of winter seasonal rains from underneath the track formation.

4.4 Track Upgrade Design Criteria

While standard DTEI CBR testing protocols in South Australia (and most other Australian states) recommend using samples soaked for 4 days, the results of the testing program allows design engineers to challenge this approach and recommend using unsoaked samples in this case because testing was undertaken during winter and spring. By conducting in-situ testing and comparing these results with both soaked and unsoaked samples, design engineers are able to

prove that the unsoaked samples could be used as the design basis for track formation remedial upgrade works.

Since the strength of the formation is related directly to the CBR values of the subgrade, the higher unsoaked CBR results are beneficial to the upgrade program. For example, on the Gawler Line the use of unsoaked CBR sample results has the potential for only about 12% of the formation needing the highest level of remediation (CBR <3%) when constructed in conjunction with appropriate drainage works, compared to 50% if the more traditional soaked CBR values are used.

Design options for remedial works to improve the track formation and subgrade conditions currently being considered include one or a combination of many of the following:

- ballast cleaning to remove clay, silt and sand materials;
- removal and total track replacement of all fouled ballast, sub-ballast and historical subgrade fill materials such as slags, coal cinders and other deleterious materials;
- improvement of drainage systems within the track corridor to divert water away from the track formation and foundations;
- installation of sub-surface drainage systems to de-water the track corridor;
- improving the bearing capacity of subgrade materials various stabilisation methods (granular, in-situ or pugmill mixing with lime, fly ash, cement or other admixtures);
- application of geofabric and/or geogrid products to reinforce, support, seal or isolate poor subgrade conditions;
- removal and replacement of low strength subgrade materials with structural fill;
- removal and replacement of low strength subgrade material with fouled ballast and sub-ballast materials (encapsulation method); and
- static compaction of ballast, sub-ballast and subgrade fill materials into the subgrade (granular stabilisation).

The following design strategies are currently being considered to optimise the track upgrade within budget and schedule constraints;

- improve subgrade materials to reduce track formation thickness;
- improve the drainage system within the rail corridor to reduce moisture ingress into highly sensitive clayey soils;
- assess the relationship between in-situ and laboratory test results to potentially increase recommended subgrade design values

(soaked to unsoaked CBRs) and use mechanistic design approaches; and

- conduct trials of remediation methods.

5 DATA PRESENTATION

5.1 Reports

Reports were produced for sections of the network; 4 sections along the Gawler line, 3 sections for the Noarlunga line, and one each for the Tonsley, Grange, Outer Harbor and Adelaide Yards sections. The reports prepared were subject to a continuous improvement process throughout the project to ensure that the final product met DTEI's strict requirements.

The testing program involved collecting data along 104 km of the rail network at approximately 100 m intervals and presenting the large amount of data and engineering parameters in a format that facilitated quick and sound engineering judgement. The standard method of presentation of geotechnical data was not suitable for the extent of work required, so the PB team and the DTEI design engineers developed a new approach.

All field data was presented on a custom A3 sheet template which included the test pit log, a photograph of the encountered ballast and subgrade profile from the test pit, DCP test results with depth, pocket penetrometer results from cohesive soils and laboratory test results with depth. Engineering design parameters were also displayed on the A3 sheets which included design CBR and bearing capacity as required by DTEI.

An example of this A3 sheet is presented in Figure 8. The new A3 reporting sheet replaced five pages of information which reduced the size of each report by several hundred pages and facilitated the effective transfer of design information.

5.2 GIS Database

Upon completion of each section of rail track all site investigation information was compiled and placed in a Geotechnical Data Management System (GDMS), which was linked to a site-specific GIS (see Figure 9). The GDMS was used to provide the framework for the GIS to access the geotechnical information relating to the Rail Revitalisation Project. The GIS was compiled on a DVD that was able to be distributed to contractors for the tender, design and construction phases of the project.

The information on the DVD included walkover observations, underground services locations, geotechnical and environmental site investigation data, laboratory results and reports. It also included geological, topographic, cadastral and soil maps and links to photographs taken during the project (see Figure 10).

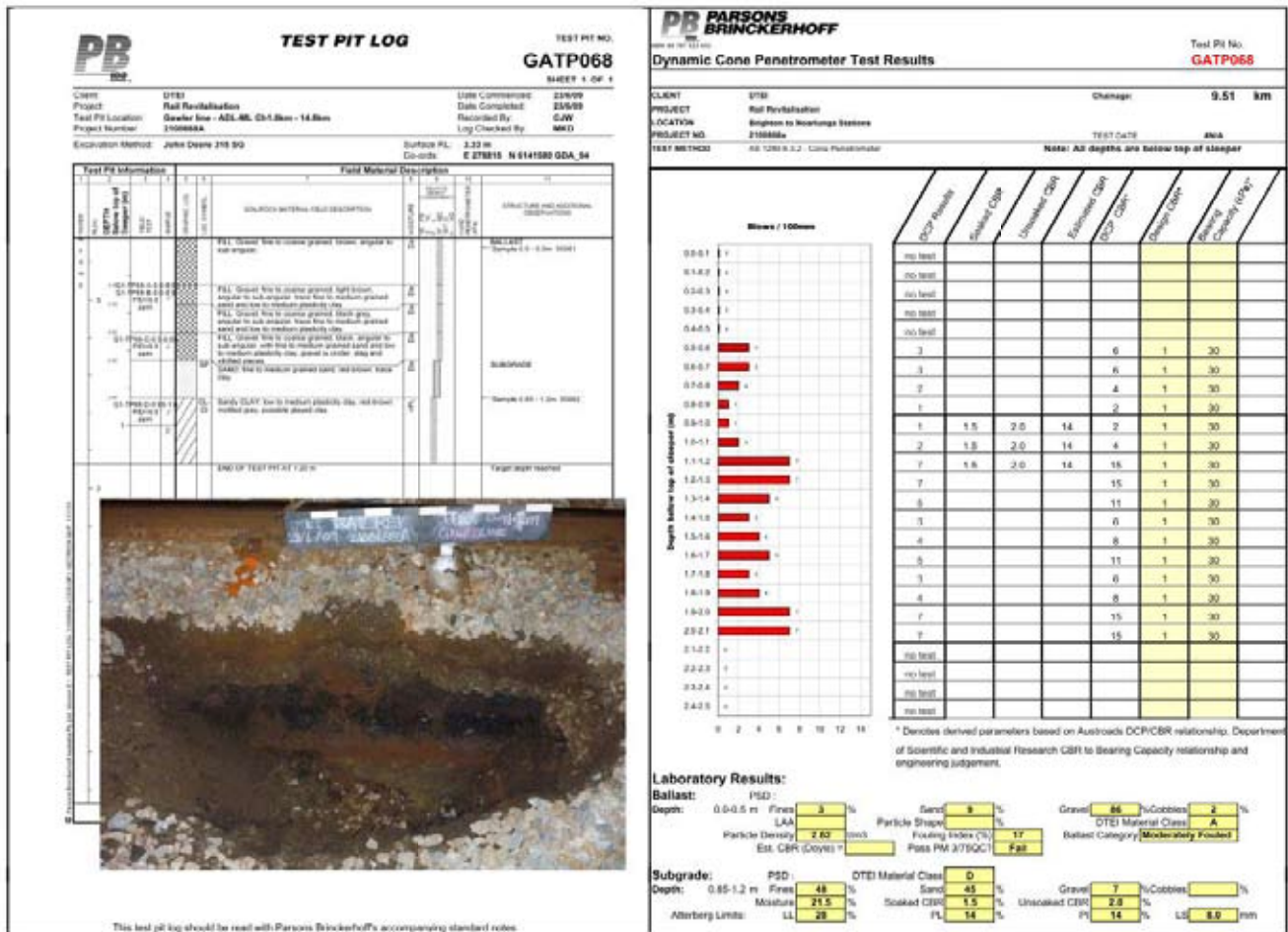


Figure 8: A3 datasheet



Figure 9: GIS database

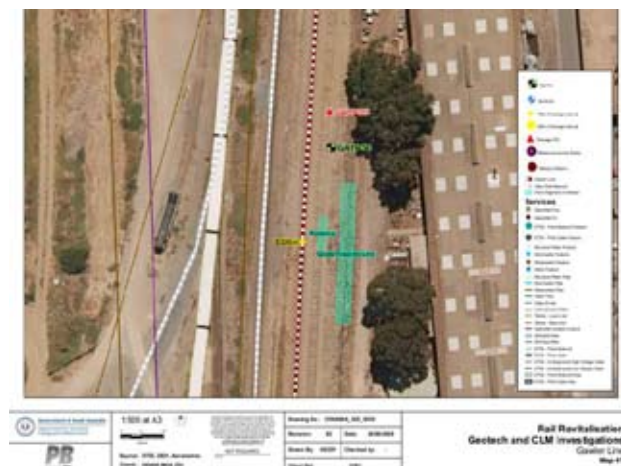


Figure 10: GIS plans

Knowledge management which is an important consideration when working on large projects can be made more efficient with the use of GIS. Linking a GDMS to GIS allows multiple users access to spatially presented geotechnical and environmental data. With careful planning, distributing the data within the GIS onto a DVD had significant cost savings in terms of time spent searching for data.

The reduction in paper usage by presenting the data on DVD also had environmental benefits.

6 BALLAST STRENGTH TESTING

The recent upgrade of the Belair line completed in 2009 involved the full replacement of ballast and sub-ballast materials. This approach would be cost-prohibitive if applied to the remainder of the network. DTEI asked PB to research and provide recommended testing methods for existing ballast

materials which could then be used to define those areas of the rail network that new ballast may be placed directly onto existing fouled ballast.

6.1 Desktop Study

DTEI engineers needed to determine if the design bearing capacities of the retained ballast material were adequate for placing 250 mm of new ballast. Standard strength and compaction tests are not suitable because of the coarse grading of the existing ballast. PB researched Australian and international best practice and provided DTEI with recommendations for field trials and potential design criteria using several field test methods. Large scale laboratory tests and plate load tests were not considered due to their disruptive test methods on existing track formations.

In-situ ballast strength testing methods were investigated that could be done without disturbing the track formation, were easy and safe to use and provided relevant track design data and quality control criteria during construction works. Three test methods were recommended for a three stage trial program on two different sections of the rail network.

6.2 Stage 1

The first stage included ballast testing at various test pit locations and compared three different types of testing equipment:

- 1) Humboldt GeoGauge to measure the in-situ stiffness (3 to 70 MN/m) and Young's Modulus (26 to 610 MPa) to 200 to 300 mm depth;
- 2) Zorn ZFG2000 light weight falling weight deflectometer (LWFWD) with 300 mm base plate to measure the dynamic deflection modulus to 500-600 mm depth; and
- 3) 4.5 kg Clegg Hammer to determine approximate CBR values to about 100 mm depth. A 20 kg Clegg Hammer with the larger 150 mm base plate more suitable for the ballast material was not available for these trials.

Tests were undertaken on various sub-locations at 19 test pit sites over a 10 day roster. The sub-locations were:

- ballast surface on the shoulder;
- ballast surface between the sleepers;
- ballast at 100 mm below sleeper soffit in the shoulder; and
- top of subgrade level in the shoulder.

The 4.5 kg Clegg Hammer, with a base plate of 50 mm was not suitable for testing the large (<75 mm) ballast material. The larger 20 kg Clegg Hammer with a 150 mm diameter base plate would be more suited for ballast testing.

The Humboldt GeoGauge was found to be more susceptible to seating conditions within the coarse ballast which caused more variable results to occur at the same location. This required a thin bed of dry sand on the ballast material to provide reliable contact (Figure 11).



Figure 11: Humboldt GeoGauge

The LWFWD recorded more consistent results at each location, was less susceptible to ballast material variability due to its larger base plate (300 mm) and required no prior surface preparation (Figure 12). The first three seating blows prior to the three recorded blows provided sufficient surface preparation.



Figure 12: Zorn ZFG2000 LWFWD

The bearing capacity of the ballast materials tested over the 19 sites ranged from 90 kPa to 220 kPa (CBR 5% to 23%) for the GeoGauge and 110 kPa to 360 kPa (CBR 7% to 56%) for the LWFWD, with the LWFWD generally recording higher values than the GeoGauge even though the depth of testing is deeper.

The bearing capacity of the subgrade materials ranged from 60 kPa to 220 kPa (CBR 3% to 23%) for the GeoGauge and 90 kPa to 220 kPa (CBR 5% to 23%) for the LWFWD. Both test equipment provided results which were higher than soaked CBR values of 2% to 5% and unsoaked CBR

values of 3% to 11% but similar to DCP results recorded at these 19 test pit locations.

The 4.5 kg Clegg Hammer on the subgrade materials recorded bearing capacities between 50 kPa and 150 kPa (CBR 2% to 11%), which closely matched the range of laboratory CBR values. However, direct comparison of results at most test locations was also highly varied.

Overall the LWFWD provided the most consistent testing method and is sufficiently robust for use in the field for investigation, design and construction phases.

6.3 Stage 2

The second stage of the field program involved undertaking further field trials with the LWFWD during reconstruction works of a short section of rail line. The reconstruction works included recovery of rail, removal of timber sleepers and ballast materials and the preparation of a new grade level. The rail track was then rebuilt using new dual gauge concrete sleepers, placement of a minimum of 250 mm of clean ballast and return of the existing rail.

Five test pit locations were identified within the reconstruction works. Tests using the LWFWD were undertaken at four sub-locations within the track formation adjacent to each of the test pit locations, consisting of:

- the existing ballast surface between the sleeper and the rail;
- the soffit level of manually removed sleepers in in-situ undisturbed ballast materials;
- the soffit level between the sleepers in in-situ undisturbed ballast materials; and
- the final grade level.

A set of three tests were conducted at each sub-location. At the final grade level, two series of three tests were also conducted, the first three tests prior to compaction and the second series of three tests after compaction and prior to placement of clean ballast (Figure 13).



Figure 13: LWFWD testing during track works

The bearing capacity of the ballast material derived from the LWFWD ranged from 140 kPa to 250 kPa (CBR 10% to 26%, see Table 6).

Test Site	Surface Crib between sleepers (kPa)	Sleeper Soffit (kPa)	Soffit level between Sleepers (kPa)
1	140–170	180–220	210–250
2	180–210	200–230	190–240
3	160–200	170–210	160–210
4	160–190	160–240	-
5	180–230	210	-

Table 6: Range of ballast bearing capacities from LWFWD testing

A summary of the results is provided below:

- crib surface level ballast strengths are lower than ballast strengths directly below the base of the sleeper soffit;
- crib surface level ballast strengths are lower than ballast strengths at the sleeper soffit level between sleepers;
- ballast strength varied between directly below the base of the sleeper and within the crib at the same level;
- all test locations had depths of ballast between 0.55 m to 1 m which suggests that the LWFWD tests conducted at the surface may not have been influenced by the underlying subgrade material;
- ballast strengths marginally increased with increasing depth of ballast overlying subgrade materials;
- ballast strength at the final grade level was lower than surface or sleeper soffit ballast strengths. Construction activity on the ballast without appropriate compaction reduced pavement strengths (see Table 7);

Test Site	Grade level materials	Bearing capacities
1	Fouled ballast	170–180 kPa
2	Fouled ballast	160–240 kPa
3	Fouled ballast	130–180 kPa
4	Subgrade	70–100 kPa
5	Not tested	-

Table 7: Range of grade level bearing capacities from LWFWD testing

- pavement strengths varied across the track width, at all levels, but was generally (but not conclusively) higher towards the edge of the

track compared to the centre of the track formation; and

- the strength of the ballast increased with increasing compactive effort (i.e. the number of passes of a roller).

6.4 Ballast Strength Findings

These trials demonstrated that ballast materials may be tested for in-situ strength using a variety of specialist equipment that is currently available. Using a combination of equipment the strength of the existing ballast materials may be determined to a depth of between 300 mm to 500 mm from the crib surface. This information may be used by design engineers to provide track formation design parameters that are currently not available using the conventional soil testing methods.

PB recommended two methods, for strength testing of the ballast and track formation to DTEI: the 20 kg Clegg Hammer and the LWFWD. The LWFWD is currently used in Europe and the United Kingdom^[5] for design and construction works. These two test methods could be implemented during railway construction works to provide Quality Control testing similar to road works test methods. These test methods are currently being considered by DTEI for inclusion into their standard specifications for railway construction works.

7 Conclusions

The Adelaide Rail Revitalisation Program is a major investment aimed at replacing timber sleepers with dual gauge concrete sleepers, remediation of the ballast and new track formations. The upgrade will facilitate increased train speeds, track durability and ride comfort, which in turn will facilitate greater patronage of the train services. The track upgrade is also essential for the planned electrification of the rail network which will provide new rolling stock, reduced energy use and further increased patronage.

The comprehensive geotechnical investigation program outlined in this paper has provided the essential data necessary to design track improvements in an efficient and economical manner. The variety of in-situ, laboratory and field tests over the network has provided a best practice benchmark on which the Rail Revitalisation project could be founded.

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