GROUND IMPROVEMENT AND VALIDATION FOR STAGE 1 IMEX EARTHWORKS

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ABSTRACT

This paper presents a case study of the use of an alternative ground improvement technique to treat contaminated uncontrolled fill other than traditional "remove and replace" for major earthworks for the proposed container Import and Export (IMEX) Terminal at Moorebank, Sydney. First, a brief discussion of the options considered for the ground improvement including removal and replacement and the Impact Roller Compaction (IRC) method will be presented. The local geological setting and the historical form of the existing Stage 1 IMEX Terminal site will be described, with the geotechnical model and associated design engineering parameters being surmised. The key points in the development of a technical specification are presented to take account of the performance requirements, including on-site IRC trial and validation testing. The primary validation measures adopted comprise plate load testing, cone penetration profiling (CPT), insitu density testing, dilatometer (DMT) testing and proof rolling after IRC treatment. Surcharging was considered for the remediated contaminated land areas where the details of the remedial treatments were not available at the Stage 1 IMEX development stage to ensure there will be no issues resulting from long-term settlement. At the time of writing this paper the Stage 1 IMEX works have been completed and are operational. The monitoring results indicate the performance of the site is satisfactory.

1 INTRODUCTION

Sydney Intermodal Terminal Alliance (SIMTA) plans to operate an Import/Export (IMEX) inland rail terminal. The Moorebank Precinct East (MPE) Stage 1 IMEX Number 1 project involves the design, construction and commissioning of all civil, rail, utilities, and other infrastructure works required to provide an IMEX Terminal and the associated rail infrastructure on the Moorebank Intermodal Precinct Terminal (MIPT) Land.

The completed terminal features:

- A terminal entry Optical Character Recognition (OCR) portal, truck processing gates, exit OCR and weigh in motion facility;
- 8 No. truck/Auto-shuttle interchange pens;
- Administration/Control facility and services structures;
- 4 No. - approximately 650 m long rail sidings;
- Automatic Stacking Crane (ASC) and Cantilever Automatic Stacking Crane (CASC) operations; and
- Eastern, central and western container stack areas.

The IMEX terminal will operate 24 hours per day, 365 days per year, other than during maintenance shutdowns. It will be initially operated manually, transitioning to fully automated operations thereafter.

The design considered the operational requirements of the Terminal Operating System (TOS) and Plant and Equipment (P&E) required for fully automated operations. SIMTA will be responsible for specifying the TOS and the major P&E.

The site is located approximately 28 km to the south west of Sydney. The site was previously occupied by the Defence National Storage and Distribution Centre (DNSDC), and comprised multistorey buildings, large storage warehouses, roads, pavement areas, outdoor sitting areas, grassed areas and a refuelling depot. The Stage 1 Proposal is located to the east of Moorebank Avenue and is an approximately rectangular parcel of land, about 400 m wide (east to west) and 850 m long (north to south). The Proposal also includes a rail link, approximately 3.5 km long, connecting the site to the Southern Sydney Freight Line (SSFL). The Stage 1 site and rail corridor footprint are presented in Figure 1.

Figure 1: Terminal Footprint (image © Google Maps)

Arcadis was originally engaged by the client as a technical reviewer. Following the commencement of this review Arcadis was approached by the client to derive an alternative solution to the traditional "remove and replace" of the existing fill option the other party was advising for the entire site as the final solution. This paper describes a case study of the ground improvement technique using the impact roller compaction (IRC) method and the validations by means of various insitu testing methods.

2 GEOLOGICAL SETTING AND LOCAL GEOLOGY

The IMEX site is relatively flat, at an elevation of approximately RL+14 m to RL+17 m AHD. Moorebank Avenue runs along the western boundary of the site, with DNSDC land bordering the north, east and southern boundaries.

The site was previously occupied by the DNSDC, which has been relocated. The site is grassed with scattered trees, occupied by low rise warehouses and administration buildings. Due to the former use of the site as a military base which was backfilled with dredged material from Georges River, unexploded ordinance (UXO) risks had to be considered during the geotechnical investigation, including completion of UXO scanning and clearance at each test location prior to testing commencing.

According to the published 1:100,000 Penrith Geological Map (NSW Department of Minerals, 1991), the IMEX site is underlain by alluvial sediments over rock. Adjacent to the Georges River the alluvial sediments are Quaternary (Holocene) age (<10,000 years) (Qha). These lie above a stratum of Tertiary (Pliocene) age fluvial deposits, consisting of clayey quartzose sand and clay (Ta). The geological map indicates that the underlying rock conditions in the area are either Triassic Hawkesbury Sandstone (Rh) or Ashfield Shale (Rwa).

The fill on the IMEX site was generally found to be about 0.5 m to 1.2 m thick, comprising mainly a mixture of sand, clayey sand, sandy clay and clay, with localised depths of up to approximately 2.3 m.

Interbedded sand and clay units were present beneath the fill to depths of up to 23 m, the depth at which CPTs refused on inferred rock. The sands are medium dense or denser and the clays stiff, very stiff or hard. Some CPTs refused at shallower depths, potentially due to the presence of ironstone layers within the alluvium.

The depth to bedrock within the site is about 17 to 23 m below existing ground level (RL -2 m to -8 m). The rock generally became deeper to the north and west, which is consistent with the published geological map.

Groundwater was at about 5 to 7 m below the existing ground level (based on observations in CPTs), equivalent to about RL 8 to 10 m AHD. The water level at a weir in Georges River to the north west of the site is about RL + 5 m AHD.

3 GROUND TREATMENT OPTIONS AND CONSIDERATIONS

3.1 GEOTECHNICAL DATA AND MODEL

The existing fill thickness profile was derived from reviewing the logs and cone penetration test results and undertaking a method of interpolation (kriging) between investigation locations (Figure 2). The derived fill thickness ranged between 0.5 m and 1.2 m, with the middle part being about 2.3m deep, based on the data provided by factual reports (Douglas, 2016 and Golder, 2017a, 2017b, Arcadis, 2017a). Some localised deeper zones of fill were identified from this assessment. These were not all confirmed during the stripping or the preliminary earthwork activities for the site.

3.2 GROUND TREATMENT CONSIDERATION AND PREFERRED OPTION

Following the review of the ground conditions, Arcadis developed an alternative solution through a workshop with the client. A cost benefit and risk assessment of three possible options were carried out during the workshop, including: 1) remove and replace of all fill materials that were not placed in accordance with either an Australian Standard, Specification or lacked appropriate compaction records; 2) dynamic compaction by dropping a very heavy weight that is dropped from significant height; and 3) the impact roller compaction (IRC) method. The option of "remove and replace" would require treatment and disposal of the potential contaminants contained within the uncontrolled fill. This would result in relatively high cost of removal of the fill material due to the presence of potential contaminants. The dynamic compaction using heavy falling weight would require a specialised plant and mitigation measures for potential noise impacts. Option 3 whereby using the impact roller compaction (IRC) method has been used for compacting both sandy and clayey soils to depths ranging between 2 m to 5 m (C. Lee et al., 2015, Choy et al., 2019). After consultation with the specialist contractors (Landpac, 2017) and the cost savings analysis it was decided to use the IRC method for the treatment of the uncontrolled and potentially contaminated fill materials for this project site.

4 IRC METHOD AND PERFORMANCE CRITERIA

4.1 IMPACT ROLLER COMPACTION METHOD

Impact roller compaction (IRC) method is a rapid controlled form of impact-oriented compaction that provides in-situ improvement to mixed fill, landfills and low strength natural soils. This allows development to proceed on high level footings rather than on more expensive deep foundations. The use of the IRC method is often guided by experience in similar soils and applications. The method works by dragging a weight along the ground. The module (weight) of varying shape is connected to the frame by a system of linkage arms that allow the module freedom of movement within its frame and linkages. Once the tow unit commences forward movement, the module is dragged forward and begins to rotate due to friction and soon reaches its operating speed. The energy delivered to the ground results in ground modification (e.g. Avalle et al., 2007). Dependent on the prevailing ground conditions and the characteristics of the impact roller, the effects are measurable by means such as surface settlement, or a relative gain in compaction or soil strength. Landpac, for example, produced the Continuous Impact Response (CIR) advanced visualisation, which will be discussed further in the following sections.

4.2 CRITERIA OF IRC PERFORMANCE REQUIREMENTS

Based on the assessment of the test pits, boreholes and laboratory testing results across the Stage 1 IMEX area it was expected that the impact roller compaction (IRC) treated ground would be improved to achieve the following:

- The treated foundation soils across the site would achieve a minimum ultimate bearing capacity of 400 kPa and a stiffness / Young's Modulus (E') of 40 MPa for all pavement, train and crane rail areas over a typical depth of 5 meters; and
- The compacted foundation soils would achieve a minimum 98% standard dry density (SDD) at the field moisture content.

It was recognised that these criteria would be reviewed and updated during the IRC ground treatment process since insufficient investigation data was available, in, around the areas where the existing building(s) and buried underground services were present.

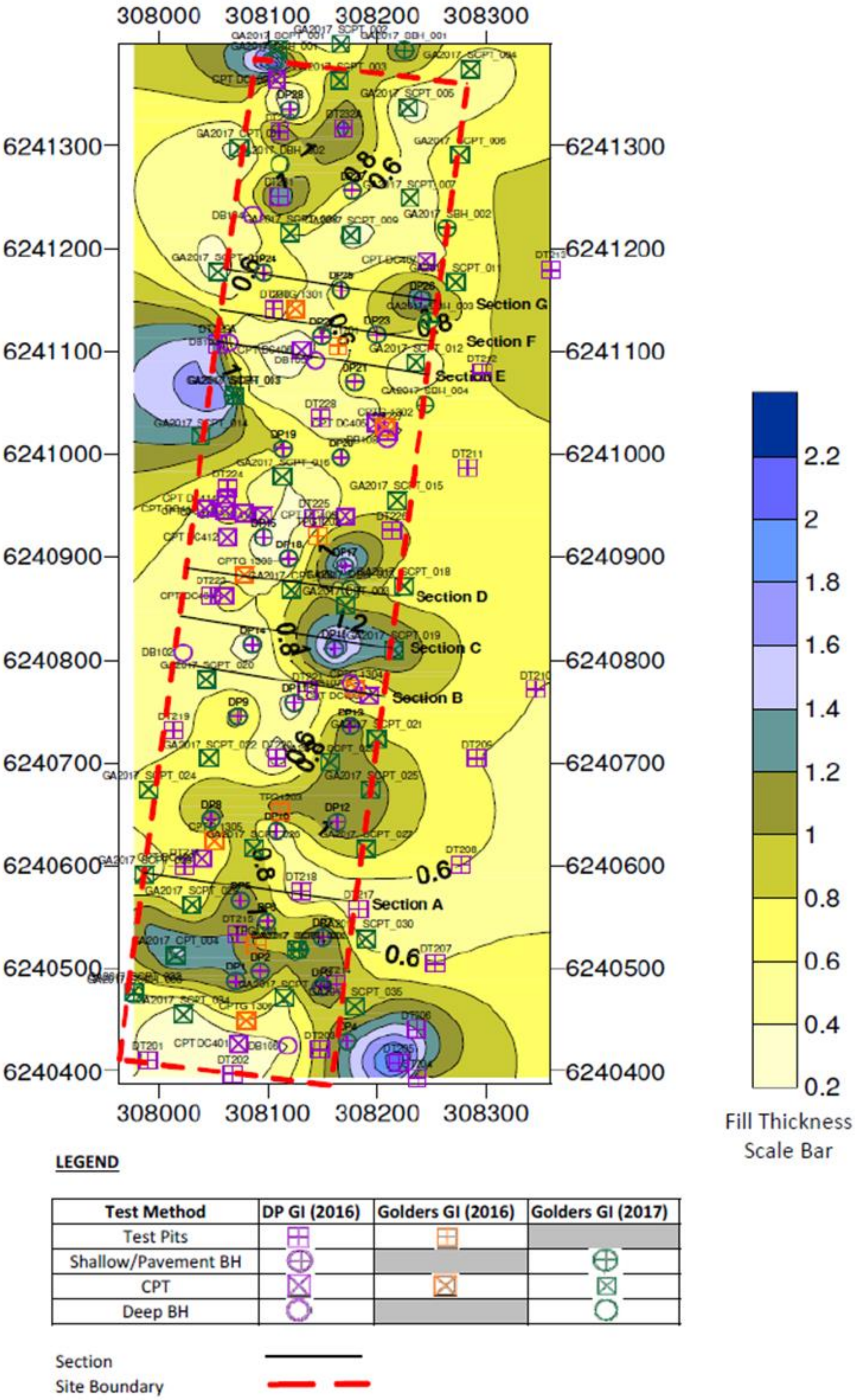


Figure 2: Interpreted fill thickness across site footprint

5 GEOTECHNICAL MODEL AND PARAMETERS

5.1 GENERAL GEOTECHNICAL PROFILE

The engineering properties of the various soil and rock units identified in the previous geotechnical investigations were assessed using test pit logs, borehole logs, cone penetration test (CPT) logs, and in-situ and laboratory test results.

The geological soil units were assessed to be pavement, existing fills, alluvium and bedrock as shown in Table 1. The bedrock has been subdivided into three sub-units, with two for siltstone/shale and one for sandstone. A generalised geological profile with detailed descriptions and associated thicknesses for each unit based on the geotechnical interpretative report (Arcadis, 2017b) is also reproduced in Table 1.

Table 1: Summary of identified geological units, descriptions and approximate thickness

Geological Unit	Material Type	Description	Approximate Thickness (m)
P	Pavements	Generally concrete, although asphaltic concrete encountered in some locations.	0.1-0.2
F	Fill (Existing)	Sand, clay and gravel mixes, generally moderately compacted to well compacted with some soft to firm clay layers identified and lack of appropriate compaction records and certification as per AS3798	< 2.3 (Average of 0.7)
AC or AG	Quaternary Alluvium: Cohesive (AC) Granular (AG)	Clay and sandy clay, generally stiff to hard, with some localised areas of soft to firm clay, as well as dense to very dense clayey and silty sand bands (Unit AG). The alluvium becomes siltier with depth.	< 18
SH-5	Class V Siltstone	Extremely low to low strength siltstone, extremely and highly weathered, extremely low and low strength.	0.4 –1.2 (average of 0.65)
SH-3	Class III and better Siltstone	Medium strength siltstone, moderately weathered, with interbedded medium to high strength sandstone	Unknown
SS-2	Class II sandstone	Medium and high strength, slightly weathered and fresh	Unknown

5.2 GEOTECHNICAL DESIGN PARAMETERS FOR VARIOUS STRUCTURES

Table 2 presents a summary of the design parameters that were derived from either testing results or published correlations.

For purposes of design a minimum design California Bearing Ratio (CBR) of 2% for the re-compaction of existing local fill up to approximately 2 m from finished surface level was adopted. The new fill placed i.e. Select Fill is assumed to have a minimum CBR of 15% and a Plasticity Index of 25.

Table 2: Geotechnical design parameters for various materials

Material Unit	Unit Weight (kN/m^3)	Undrained Conditions		Drained Conditions			
		Cu (kPa)	Eu (MPa)	ν	C' (kPa)	Φ' (degs)	E' (MPa)
Ballast	20	-	-	0.3	0	42	80
Capping Layer	19	-	-	0.3	0	40	80
Concrete	24	-	-	0.15	-	-	32000
Select Fill*	20	-	-	0.3	0	35	50-80
Fill- Existing	20	65	26	0.3	0	26	20
Fill – Existing (LB)***	19	45	18	0.3	0	24	15
Fill - Treated**	20	-	-	0.3	0	35	40
Alluvium – Cohesive	19	100	41	0.3	2	28	35
Alluvium – Granular	19	-	-	0.3	0	36	80
Siltstone Class V	20	-	-	0.25	5	28	80
Siltstone Class III	24	-	-	0.2	60	30	500
Sandstone Class II	24	-	-	0.2	100	30	1000

* Select fill shall be sandstone type material from New M5 tunnel spoil or similar, which will be placed and compacted in accordance with AS3798, with a minimum density of 98% SDD

** This unit refers to the existing fill of variable materials whose shear strength presented is an equivalent value after dynamic compaction treatment.

*** This unit represents the lower bound parameters for the existing fill material (Unit F)

For reinforced concrete pavements, the upper subgrade was stipulated to have a design CBR of 33% with a lower subbase of heavily bound granular material of a minimum 4 MPa and a base comprising of a minimum 40 MPa concrete. Similar design parameters were recommended for the heavy - duty concrete block paver pavements, however, the heavily bound granular material of 9MPa (C8/C10) was adopted in lieu of 4MPa. These major elements were on the proviso that the New Fill would achieve the minimum characteristic CBR of 15% for the subgrade. For pavement structural design purposes, the combination of CBR15% fill over CBR 2% fill not thicker than 2m below Final Surface Level, was adopted.

6 SETTLEMENT AND LOADING CRITERIA

The total settlement and differential settlements for various key structural elements were developed by interaction with the operational requirements from the supplier and the published guidelines for stack container yards, crane rails, ATV area, loco area and loco shifter. The specific requirement details adopted are summarised in Table 3.

Table 3: Settlement and loading criteria for Stage 1 key structural element

Area	Loading	Settlement Criteria
5 Stack Container	40kPa based on 25T (20ft) container reduced by 20%	1.4% between any supports; 6.55m between container beam supports=92mm allowable deflection; and 2.44m width of container=34mm
	250kPa at ends of containers for a 1.8m width	
3 Stack Container	33kPa based on 25T (20ft) container reduced by 20%	1.4% between any supports; 6.55m between container beam supports=92mm allowable deflection; and 2.44m width of container=34mm
	250kPa at ends of containers for a 1.8m width	
Crane Rail A (C-ASC – outside leg)	325kN/m	40mm vertical differential between legs Maximum 1:500 longitudinal differential settlement
Crane Rail B (C-ASC – inside leg)	325kN/m	40mm vertical differential between legs Maximum 1:500 longitudinal differential settlement
Crane Rail C (ASC – west leg)	293kN/m	40mm vertical differential between legs Maximum 1:500 longitudinal differential settlement
Crane Rail D (ASC – east leg)	293kN/m	40mm vertical differential between legs Maximum 1:500 longitudinal differential settlement
ATV Areas	66kN/m	5mm between adjacent wheels
Loco Area	264kPa load on rail sleepers	70mm between rails 9mm longitudinal differential along 10m section of track, i.e. 1:1111**.
Loco Shifter	54kPa uniform load	5mm differential settlement in the longitudinal direction between the Loco Shifter and the approaching rail*

* Differential settlement in this location will be managed using an approach slab in the transition zone between the rail and the Loco Shifter

** Longitudinal differential limits taken from Australian Rail Track Corporation (ARTC) Specification ETG-05-01: Track Geometry

7 TECHNICAL SPECIFICATION REQUIREMENTS

7.1 OVERALL STRATEGY OF SPEIFICATION DEVELOPMENT

To ensure the requirements of the earthworks were met several key points were set out in the technical specifications. These include: 1) to ensure clearance of all services; 2) field trials of the IRC to check if the foundation performance can be achieved; 3) a comprehensive suite of testing to validate the earthworks; and 4) proof rolling after completion of IRC to identify soft spots and undertake remedial actions where required. A series of hold points and witness points were incorporated into the technical performance requirements to validate the effects of the IRC process.

It was highlighted in the specification the Geotechnical Testing Authority (GTA) must submit the testing method statement and detailed procedures for approval by the Geotechnical Inspection and Testing Authority (GITA) in accordance with AS3798. The approved method statements by GITA must be submitted to the Superintendent for review and approval. GTA must submit the testing method statement and detailed procedures for approval by GITA. The approved method statements by GITA must be submitted to the Superintendent for review and approval.

Upon approval of the GITA final report a formal certification of all earthworks and testing stating full compliance with the project specifications must be issued to the Superintendent.

7.2 VALIDATION TESTING REQUIREMENTS

The testing described herein presents the minimum scope of testing for the trial area to assess if the design requirements set out on the specification have been achieved. The field testing and report was undertaken by Golder Associates.

7.2.1 Cone Penetration Testing (CPT)

A minimum of five (5) locations nominated by the GITA for cone penetration testing to assess the zone of improved ground following on from the IRC treatment. The depth of investigation with the CPT must be to a minimum depth of 5 m. An acceptable CPT profile would be determined by the Designer and approved by the Principal after the trial.

7.2.2 In-situ Density Testing

A minimum of five (5) locations were selected for in-situ density testing to assess the IRC improved subsurface profile. The testing surface may require cutting down to below the depth of surface disturbance associated with impact rolling (envisaged to be approximately 300 mm). Typically, three tests were to be undertaken at each location, namely at depths of 0.3 m, 0.6 m, and 0.9 m below ground surface. Where fill was encountered at depths greater than or equal to 2 m, additional density tests at 1.2 m, 1.5 m and 2 m must be undertaken. This was accompanied with field moisture content assessments.

7.2.3 Plate Bearing Tests (PBT)

Five (5) number plate bearing tests (PBTs) were to be undertaken at locations nominated by the GITA. The test locations were to be spread out over the area of the trial foundation area. Each test was to have three cycles, the first to the ultimate bearing pressure (400kPa), the second to 1.5 times the ultimate bearing pressure (600kPa) and third cycle to maximum load / failure – excessive deflection. The plate size was to be nominally 750 mm diameter. Location and details of the tests were to be recorded with the test results. The plate bearing tests were carried out in accordance with BS 1377-9-1990, and were accompanied with in-situ density testing and field moisture content assessments.

7.2.4 Surcharging and Monitoring

Remediation works by means of removal and replace was previously undertaken for areas where petrol tanks were buried. However, there was no compaction data available to confirm what degree of compaction level and associated testing for the areas was achieved with the exception for the material being noted as Bringelly Shale fill on the monitoring records. As such 2 metre of surcharge was placed above the finished level for a month to assess the existing compaction condition of the remediated areas. A total of 6 settlement plates was installed prior to fill placement and surcharging to gather the displacement information for further assessment.

8 FIELD TRIAL AND VALIDATION TESTING

8.1 IMPACT ROLLER COMPACTION

8.1.1 Methodology

Prior to impact roller compaction (IRC) commencing, the existing buildings and structures were demolished, and the exposed ground surface was levelled and stripped of topsoil. A "pre-IRC" proof roll of the stripped surface was carried out to identify, excavate and replace materials which were unlikely to be improved by IRC (e.g. saturated materials, organic soils or unsuitable materials).

After the trial of varying number of passes using IRC it was found that 15 passes using a 3 m wide, 15 tonne, pentagonal dynamic compaction drum towed by a 4 wheeled tractor, as shown in Figure 3, was the best IRC treatment method for the subject site. This drum operates with a drop height of 150 mm to provide 22 kJ of energy during impact rolling. Dynamic response of the ground to the roller was monitored continuously via the Continuous Impact Response (CIR) system, which generated a "coloured map" identifying the level of soil response after the different number of passes. CIR has been adopted by others to verify IRC treatment, as discussed by Kelly and Gil (2012), and further details relating to CIR technology are presented by Landpac (2017). Following the IRC treatment, the surface was smoothed using a smooth drum roller and a "post-IRC" proof roll was also carried out to identify potential areas of concern which were initially identified using the CIR system. Deflective areas which were observed during the post-IRC proof roll generally corresponded to areas which registered low or very low responses on the CIR plots (represented by yellow and red shading in Figures 5a) and 5b) which typically were related to former service trenches or low lying areas where there was a tendency for water to pond. These areas could not be improved by IRC and were remediated using conventional methods of removal and replacement of compacted fill as shown in Figure 4.

The locations for geotechnical validation testing were nominated using both the CIR system and observations during the post-IRC proof roll. At each test location, CPT, PBT and in-situ density testing was carried out within 1 m of each other. Dilatometer testing (DMT) was also carried out during the initial IRC trial area, located towards the southern portion of the site only.

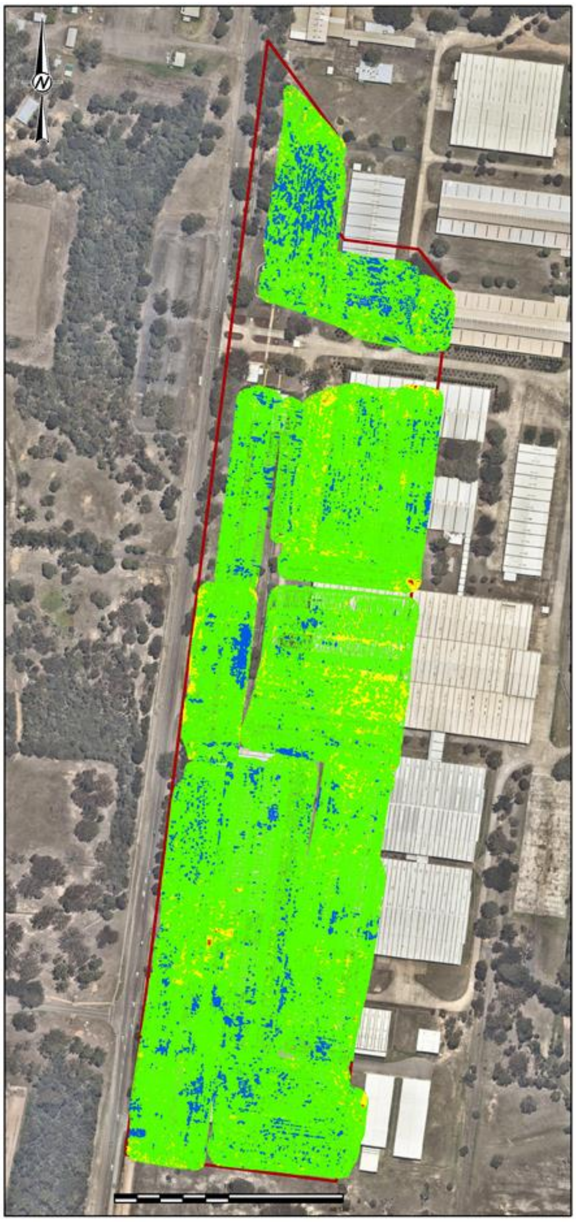
8.1.2 Interpretation of Results

Figures 5a) and 5b) compare the CIR plots after completion of the first and fifteenth pass of IRC, respectively. In general, the extent of poorly compacted areas (i.e. yellow and red areas of the CIR plot, which are unlikely to achieve the required compaction and/or subgrade stiffness), was observed to reduce with more passes of IRC. Conversely, a larger proportion of the site was observed to be well compacted (as represented in blue and green, which indicate areas which were likely to achieve the level of compaction and/or subgrade stiffness) after additional passes of IRC. The CIR monitoring system and output was demonstrated to be an efficient method of subgrade responses to the IRC actions. It should be noted that the relatively lower compaction on the south west edge in Figure 5b) was due to lack of confinement as well as corner effects where the tractor and drum had no further space to maintain the same speeds and compaction effort.

The results of in-situ field density testing indicate the subgrade generally achieved, on average, a density ratio of about 100% SDD after 15 passes of IRC.

Figure 3: Tractor and pentagonal drum used

Figure 4: Removal and replace of areas post proof rolling



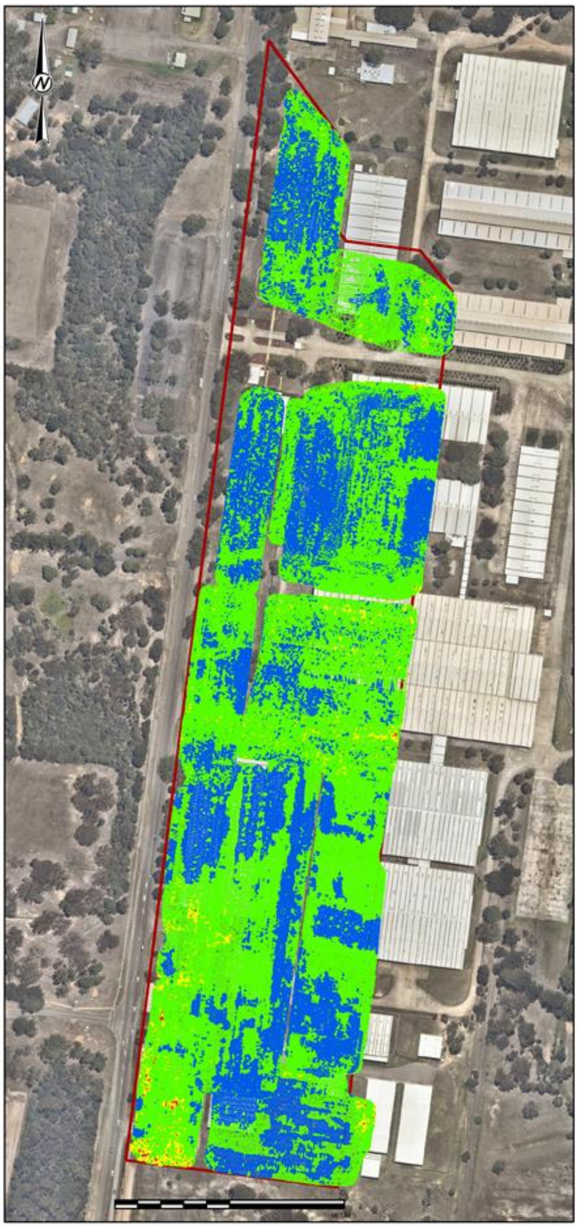


Figure 5(a): CIR plot after first IRC pass **Figure 5(b): CIR Plot after 15th IRC pass**

Ground stiffness is a strain dependent soil parameter which tends to be high where strains are small. Due to this strain dependency, there can be a large variation in stiffness moduli (E') obtained for the same material, depending on the method and strain level induced during testing. For this reason, the E' values interpreted from DMT testing, which induce smaller strains during testing, are typically higher than those obtained from the interpretation of CPT and PBT tests completed in the same material. The E' of the soil has been assessed using CPT data by estimating the constrained modulus of the soil (M) based on Robertson (2012) and then converting this to E' using the formula E' =0.74 x M, assuming a Poisson's ratio of 0.3.

Figure 6 compares the average E' interpreted from various field-testing methods in general, the results indicate IRC to be an effective means of preparing a uniform and suitable foundation, which has achieved the design Young's Modulus value of 40 MPa required as per specification requirement.

Figure 7 summarised the lower bound and upper bound and the average inferred Young's Modulus values with depth, indicating the lower value is about 20 MPa, occurring at depths of 0.5m to 1.5m. All the remaining values are greater than 40 MPa.

Figure 8 and Figure 9 shows the lower bound and upper bound values inferred from plate bearing tests and CPT profiles respectively, noting that the lowest value of Young's Modulus is approximately 20 MPa that only occurs at certain depths.

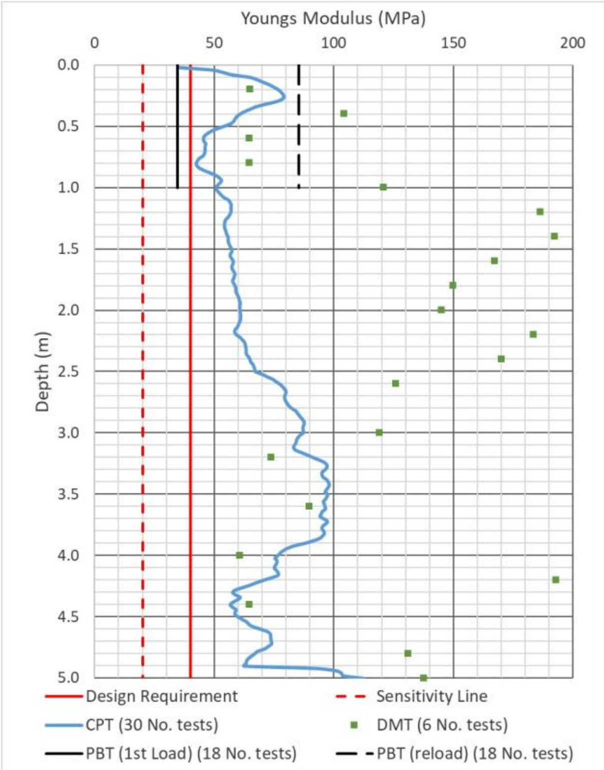


Figure 6: Average Modulus from various testing

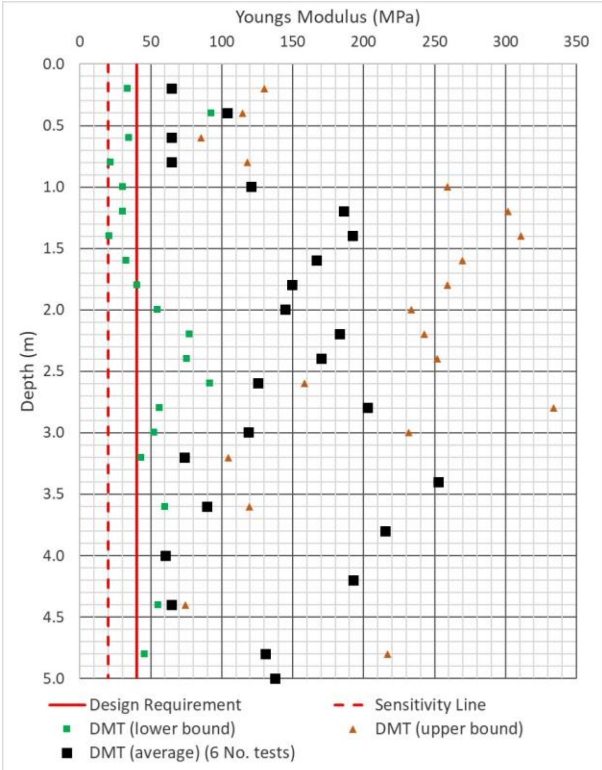


Figure 7: Lower and upper Modulus from DMT testing

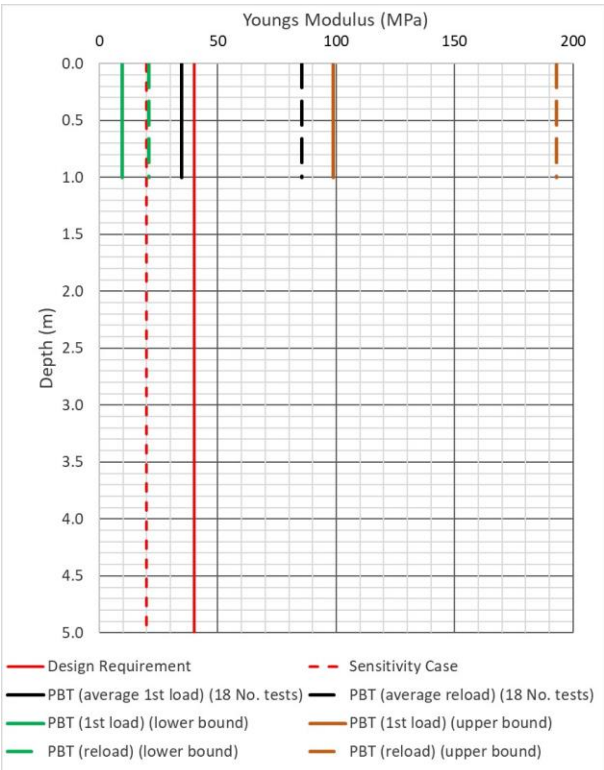


Figure 8: Modulus from various plate bearing testing

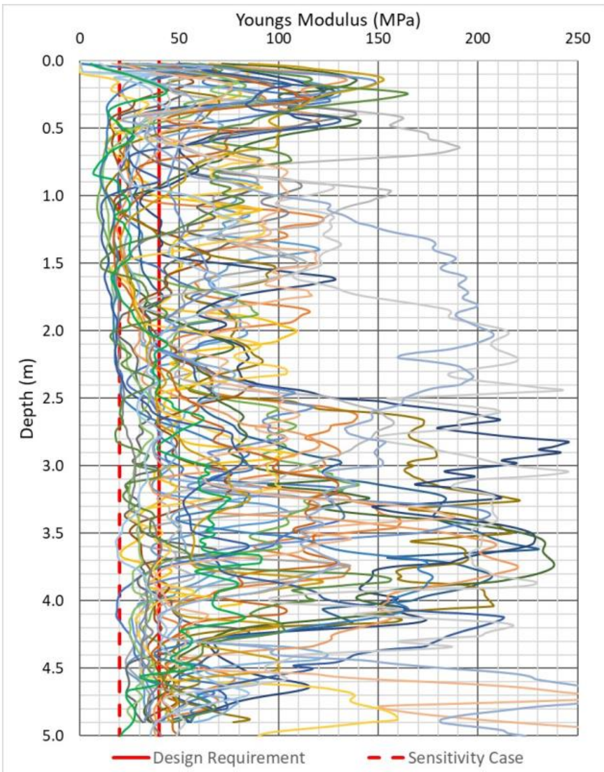


Figure 9: Modulus profiles from CPTtesting

Figure 10 presents the lowest Young's Modulus profile of CPT profile (blue line), with Figure 11 showing the stepped profile with depth in detail. It can be assessed that the Young's Modulus using a weighted average over 5m depth is 62.2 MPa, which is greater than the design values of 40 MPa.

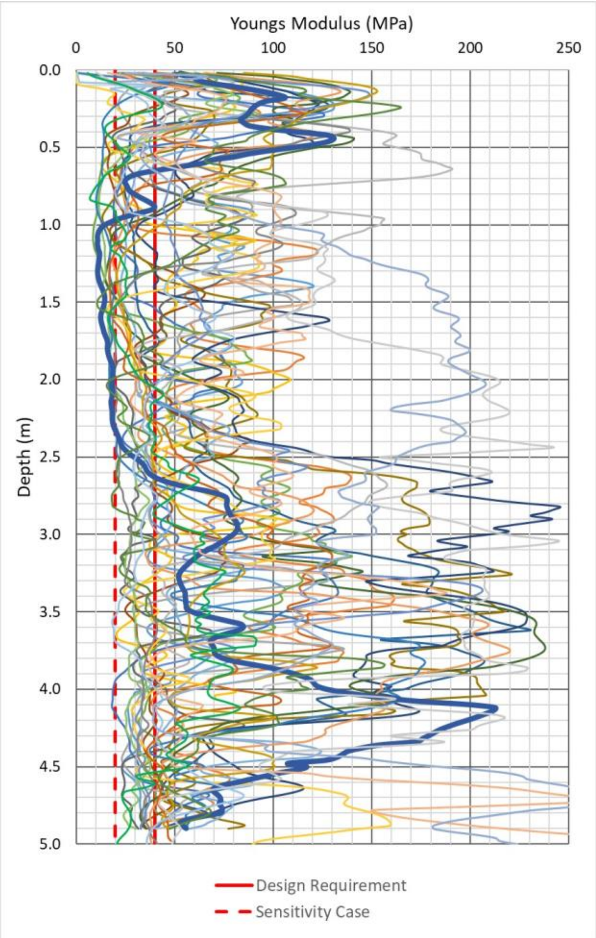


Figure 10: Modulus profiles from CPT testing

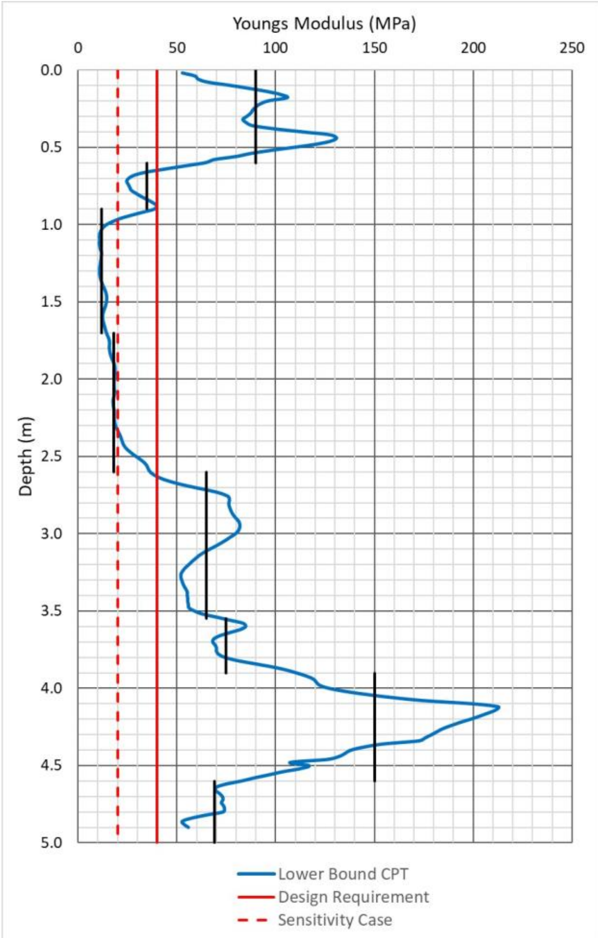


Figure 11: Modulus profile from lower bound CPT

8.1 CONVENTIONAL COMPACTION

Once the subgrade of the site had been proven and compacted using the IRC treatment method, the site levels were raised by construction of a sandstone fill platform, predominantly using sandstone fill material sourced from Sydney tunnelling projects that were running at the same time as this work. Some key issues that require consideration when re-using sandstone from tunnels are as follows:

- Variation in moisture content – depending on the source and status of the source project imported materials may be wet or dry of optimum moisture content;
- Different rock materials may be available depending on the stage of project and source of the materials;
- Excavation methods – blasting vs road header production will impact the grading of the imported fill;
- Presence of other materials mixed in the fill needs to be considered and specifications may allow a small amount of concrete and metal to account for this; and
- Average results of laboratory testing conducted on sandstone fill material used in IMEX (prepared to 98 % standard maximum dry density [SMDD] where applicable) were as follows:
  - CBR = 45%;
  - Optimum Moisture Content = 9.5%;

- o 56% finer than 37.5 mm, 37% finer than 2.36 mm and 10% finer than 75 microns. Note some oversized particles were occasionally observed on site and were excluded from lab testing. i.e. visual inspection of site material required;
- o Liquid limit of 24% with a Plasticity Index of 9%;
- o Phi' = 37 degrees; and c' = 21 kPa.

9 SENSITIVITY ASSESSMENT USING PLAXIS 3D MODELLING

Given the variations of the inferred Young's modulus from the CPT profiling, DMT and plate bearing testing it was decided to undertake three dimensional modelling to assess the differential settlements of the container yard and the crane track formations. This sensitivity assessment was carried out using Plaxis 3D and comprised consideration of unbalanced loading along the yard and the track formations together with a variable ground profile.

The results of the sensitivity assessment, as illustrated in Figure 12, indicated that the differential settlement requirements set out in Table 3 can be satisfied with the potential profile variations at each of the crane legs, that is the lower bound at one leg and the normal condition at the leg of the crane rail track formation.

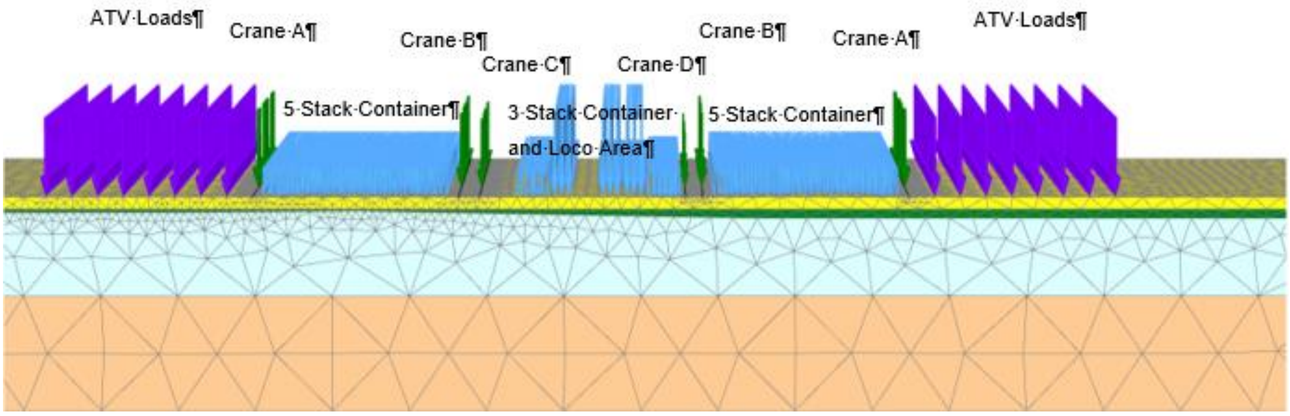


Figure 12: Plaxis 3 model for ATV loads, crane rail loads, container stack loads

10 CONCLUSIONS

The ground improvement work for the Stage 1 IMEX site has been successfully completed in early 2019. The site validation testing results show that the required bearing pressure would be satisfactory with the predicted settlements within the tolerable limits. The client is fully aware of and agreed that any potential on-going settlement post construction will be managed as part of the operation and maintenance works. Should the actual total settlement and differential settlements induced be not acceptable to the operational limits of crane, track and container stack yard pavement or crane rail formation appropriate remediation actions will be undertaken accordingly as part of the long-term maintenance works.

11 DISCLAIMERS

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12 ACKNOWLEDGMENTS

Many staff from Arcadis, Fulton Hogan, Golder, Qube and Tactical worked cooperatively on the Stage 1 IMEX project, both directly and indirectly, contributing to the successful completion of the project. The authors are grateful to everyone who contributed to this project. Gratitude is owed to Qube and Tactical project managers and director, who provided guidance and support and timely approvals during design and construction phases. The reviewer(s) of this paper provided valuable comments. Their contributions are greatly appreciated.

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