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7 September 2018

Our ref: GEOTLCOV24072AH-L02-Rev3

CPB Contractors Pty Ltd
Chatham Avenue, Moorebank
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[redacted]

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Moorebank Precinct East Stage 1 RALP No. 1 – Glenfield Waste Services Construction Impact Assessment Report

1. Introduction and Objective

CPB Contractors Pty Ltd (CPB Contractors) has been engaged by Sydney Intermodal Terminal Alliance (SIMTA) to design and construct the Stage 1 – RALP No. 1 works package, herein referred to as the Rail Link, which forms part of the Moorebank Precinct East Development. Coffey Geotechnics Pty Ltd (Coffey) has been engaged by CPB Contractors to provide geotechnical and contamination support for this project.

Coffey has previously prepared a CIAR for the Rail Link (ref: GEOTLCOV24072AF-AV, dated 9 November 2017 and 8 May 2018 (GEOTLCOV24072AH-L02). Changes to the design associated with the realignment of the Rail Link through the GWS Facility, and replacement of the dynamic compaction with the surcharge ground improvement technique, has triggered the need to update the CIAR. This document supersedes the previous version of the CIAR, and provides an assessment of the potential impacts and resulting mitigation measures for the revised design of the Rail Link (refer Attachment A) which will pass through the GWS Facility.

This Glenfield Waste Service Construction Impact Assessment Report (CIAR) has been prepared in response to development consent conditions issued by the Planning Assessment Commission of NSW under approval # SSD 6766. Specifically, Condition C5 states that:

> *"Prior to the commencement of construction of the Rail Link within the Glenfield Waste Facility licensed premises, the Application shall prepare an assessment report of the proposed impacts of construction on the Glenfield Waste Facility licenced premises...*
>
> *….*

The Applicant must provide the assessment report to the EPA for review and approval at least 6 weeks prior to the commencement of construction. A copy must also be submitted to the Secretary for information. No works are permitted to commence within the Glenfield Waste Facility licenced premises without the EPA's written approval, unless otherwise agreed by the Secretary"

The objective of this report is to assess potential impacts associated with construction activities to be carried out on the Glenfield Waste Services Facility (GWS Facility) as required in Condition C5. The requirements of Condition C5 are addressed in detail in Section 3 of this report. This document has been prepared with consideration of the NSW EPA (2016), Solid Waste Landfills.

2. Proposed Construction Procedure and Sequence at the GWS Facility

The works undertaken within the GWS Facility would be contained within the project boundaries of the Rail Link. The rail corridor will traverse along the eastern edge of the GWS Facility so as to minimise disturbance to the Facility. The Rail Link will exit the GWS Facility via a bridge constructed across the Georges River. Attachment A contains a set of design drawings showing the proposed alignment through the GWS Facility.

The rail formation through the GWS Facility will consist of a pre-loaded section, a reinforced earth wall and embankments. The different types of construction methods were chosen to minimise the extent of impact to the working landfill site and through consultation with the landowner. Reinforcement structures such as earth walls, terra-mesh walls and rock anchors are required to support the embankment in areas, however there are no existing landfill cells where these ground improvement techniques will be employed.

The design through the GWS Facility has recently been modified since the November 2017 version of the CIAR document was prepared. These changes have been a result of discussions between the proponent of the Rail Link and the landowner of the GWS Facility, particularly around the design of the Rail Link through the GWS Facility and how it aligns with the future landfill cells and finished ground levels. The major changes are listed below:

- A viaduct will no longer be constructed over a portion of the Rail Link adjacent to the existing leachate pond. This section of the track will be built as a reinforced earth wall, with rock anchors or similar.
- The construction of reinforced earth wall on the section of Rail Link adjacent the existing leachate pond will cause the embankment to encroach onto the footprint of the leachate pond. As such, the existing leachate pond will be relocated to the north of the GWS Facility, between the southern and northern connectors. The existing leachate pond would need to be relocated prior to implementing these ground improvements and relocation of the leachate dam in GWS Facility is subject to EPA consent. The relocation of the leachate pond, which will be undertaken by the landowner of the GWS Facility, will also provide the GWS Facility with additional landfill capacity, which is the preference of the landowner. Further detail on these changes is provided in Section 2.3 below.
- Additional capacity in the stormwater basin located adjacent to the northern connection is required by the landowner of the GWS Facility once the final landfill design is included. This has resulted in a realignment and extension of the footprint of the existing basin. To account for this the volume of the existing stormwater basin will be increased by along the northern connection, and increase the total capacity of the basin to support stormwater management during GWS's future landfilling operations. Further detail on these changes is provided in Section 2.4 below.

- Placement of an additional geosynthetic liner on the current capped surface existing landfill cells to abate infiltration into the underlying landfill cell, and also contribute towards reduction in leachate production associated with future landfilling in adjacent cells.
- Excavation of soils and construction of an embankment adjacent to the Southern Connector to a base level of RL 3.0m AHD. The embankment is required for track stability purposes, and to allow for the future construction of the "Cell X" landfill cell by GWS.
- A new haul road into the landfill pit located adjacent to the existing leachate pond. The existing haul road is located with the construction footprint of the Rail Link. The construction of the new haul road will be undertaken by the landowner of the GWS Facility prior to the commencement of construction of the Rail Link in that section.
- Dynamic compaction will no longer be utilised as a ground improvement over existing landfill cells, as referenced as the Rail Links preferred ground improvement treatment in previously endorsed versions of the CIAR.

2.1. SSFL Connection

The works for the southern and northern connections to the Southern Sydney Freight Line (SSFL) will be undertaken in consultation with ARTC around availability of possession of rail infrastructure, particularly for relocation of signalling infrastructure and the installation of new turnouts on both northern and southern connections.

The construction methodology for the connections to the SSFL is as follows:

- Establish a construction platform within the project boundaries for the Rail link through the eastern edge of the GWS Facility and connection to the SSFL.
- Access to the SSFL work zone is expected to be primarily via the GWS Facility. The existing Cambridge Avenue will be utilised as the main access road during construction activities.
- Establish possession times with ARTC and Sydney Trains and plan for the installation of the turnouts and the relocation of any signalling, HV or other essential rail services.
- Undertake earthworks to establish the rail formation within the safe zone adjacent to the SSFL.
- During possessions undertake the works to remove the existing sections of SSFL on both northern and southern connections and install the turnout and sufficient rail to allow ongoing works to be undertaken outside of possession times within Rail Corridor.

2.2. Earth Embankment / Pre-loading

Where the rail alignment is over the completed landfill cells, it is proposed to be constructed as an embankment within the GWS Facility using pre-loading/surcharge for ground stability.

2.2.1 Pre-Loading Surcharge

The pre-loading surcharge methodology will be required where a geosynthetic and/or clay liner exists (i.e. from approximately 40.440km to 40.740km). The methodology will involve:

- Construction of an embankment on the existing landfill.
- Overfilling the embankment by up to an average of 9 metres.
- This surcharge will remain in place for up to 3 - 6 months, depending on design requirements, to allow for the settlement of the landfill.
- Once complete, the surplus material used to surcharge landfill will be removed and used elsewhere on site as construction material for the RALP.

- The track formation will be constructed on this embankment.

Pre-loading surcharge was selected as the preferred construction method to minimise detrimental impacts on the existing geosynthetic and clay liner. Refer to Section 3 Item (e) for additional details.

The remaining sections of the Rail Link within the GWS Facility will not be constructed over active or capped landfill cells.

2.3. Leachate Pond

The landowner of the GWS Facility has advised the proponent of the Rail Link that the existing leachate pond will be relocated by the landowners of the GWS Facility to the space between the northern and southern connections of the Rail Link. This will provide the GWS Facility with additional future landfill capacity in the area of the existing leachate dam. Due to the relocation of the leachate pond, there is no longer a requirement for a viaduct be included in the design, as the purpose of the viaduct was to ensure the existing leachate pond was not impacted on by the Rail Link. Therefore, with the removal of the leachate pond, a reinforced earth wall will now be constructed, which we understand will provide additional future landfill capacity to the landowner of the GWS Facility.

The proposed location of the new leachate pond is in the northern section of the GWS Facility, between the northern and southern connector (refer Attachment A). The process to relocate the leachate pond, which will be undertaken by GWS under their planning approvals, would generally follow the steps below:

- Excavation of new leachate pond, including appropriate handling of excavated material.
- Installation of the pond liner, and appropriate certification of new leachate pond.
- Construction of appropriate leachate pipe work infrastructure for new leachate pond. This pipe infrastructure will be placed in locations and elevation such that it will not be impacted by the future construction of the Rail Link. The design of the new pipe infrastructure is being coordinated with the RALP design to mitigate such impacts.
- Pump leachate from existing leachate pond to new leachate pond.
- Decommissioning and excavation of existing leachate pond including the geosynthetic basal liner of the leachate pond to allow for construction of the reinforced earth wall and the rail embankment..

The commissioning of a new leachate pond and the decommissioning of the existing leachate pond by GWS will require the landowner of the GWS Facility to seek approval from the Environmental Protection Authority (EPA).

The relocation of the leachate pond will be completed by the landowner of the GWS Facility prior to the commencement of construction of the reinforced earth wall required for that section of the Rail Link. No RALP construction works will commence in the area of the existing leachate pond until the new leachate pond has been commissioned, and the EPA and GWS have given approval for works to proceed.

GWS will continue to have access to the leachate pond during the course of construction activities of the Rail Link.

2.4. Stormwater Basin

Additional capacity in the stormwater basin located adjacent to the northern connection is required by the landowner of the GWS Facility once the complete landfill design has been finalised. This has resulted in a realignment and extension of the footprint of the existing basin. The stormwater basin, which previously required only a small readjustment due to the design of the Rail Link slightly impacting on the capacity of the basin, will now require a more significant extension and realignment to allow for the proposed final landfill levels. This significant extension and realignment will be undertaken by the landowner of the GWS Facility under their planning approvals.

The general sequence of construction of the realigned stormwater basin is summarised below.

- Expand the existing stormwater basin in a southerly direction to the capacity desired by the owners of the GWS Facility for their future operations. Drainage to, and capacity in the stormwater basin shall be designed so that it is sufficient for the operation of the GWS Facility.
- Install dam wall in basin to allow for minor filling operations on western side of old basin for RALP.
- Commence filling operations to the western part of the old stormwater basin to enable construction of the northern connection of the Rail Link to the Southern Sydney Freight Line.
- Additional drainage pipelines/swales and access routes, as requested by the owner of the GWS Facility, to the stormwater pond will be constructed as part of the Rail Link construction for GWS Facility operations. Access to the extended stormwater basin for the owner of the GWS Facility will be maintained throughout the RALP construction period.
- Install new, or adjust existing pipes to drain stormwater to new extended stormwater basin, to minimise impact from the Rail Link construction activities.

Management of the stormwater basin and any dewatering is the responsibility of the licensee for the basin, namely GWS Management.

GWS will continue to have access to the surface water basin during the course of construction activities of the Rail Link.

The capacity of the new pond will increase to account for stormwater management requirements associated with the future requirements of GWS's landfill operations.

No Rail Link surface water is to be directed to or discharged offsite via the existing stormwater basin. All Rail Link surface runoff is to be separately controlled and discharged offsite in accordance with s120 of the Protection of the Environment Operations Act 1997.

3. Construction Impact Assessment and Proposed Mitigation Measures

Potential construction impacts associated with the works to be carried out on the GWS Facility and proposed mitigation measures are summarised in the table below.

<table>
<tr><th>NSW Conditions of Consent (SSD 6766)</th><th>Activities undertaken and proposed mitigation measures and commitments</th></tr>
<tr><td colspan="2">Condition C5 Contamination</td></tr>
<tr><td>a) Targeted intrusive investigations to determine contamination pathways and to develop mitigation, management and/or remediation options based on those investigations.</td><td>A supplementary contamination assessment was undertaken in February 2016 for the Moorebank Intermodal Rail Link, of which the Rail Link through the GWS Facility forms part of. The scope of the investigation within the GWS Facility included:
• Excavation and sampling from 15 test pits
• Sampling from 8 geotechnical boreholes
• Installation and sampling from 4 newly installed groundwater monitoring wells. The monitoring wells were installed as such both the shallow groundwater within the Alluvium and the deeper groundwater within the Sandstone.
• Collection of 2 surface water samples from the leachate dam and surface water pond
• Installation and sampling from 2 newly installed gas monitoring wells adjacent to former landfill cells

The supplementary contamination assessment was designed based on the existing JBS&G (2015) investigation, of which four test pits were excavated along this section of the Rail Link. Investigation methodology, results, findings and recommendations are provided in the Land Contamination Status Report, prepared by Coffey, dated 13 July 2018 (Ref: GEOTLCOV24072AH-Rev05).

Based on the results presented in the Land Contamination Status Report (LCSR), it was concluded that localised remediation will be required in the following parts of the proposed Rail Link alignment with the GWS land: lead impact along the Southern Connection section and installation of gas vents to minimise gas ingress and/or accumulation within underground service pits or trenches. The proposed remediation works are summarised in the Remediation Action Plan (RAP (Coffey, 2018)) specifically prepared for this project. (Ref: Coffey, 2018, GEOTLCOV24072AH-R02-Rev03).

Since the preparation of the LCSR, and the RAP, the design through the GWS Facility has been modified, with the significant changes summarised within Section 2 above. The LCSR and RAP have subsequently been updated to reflect the changes to the recent redesign of the Rail Link. The following additional contamination investigations were completed within the GWS Facility as a of the redesign:

• Excavation and sampling from 26 test pits within the southern connector, northern connector, and the proposed leachate and stormwater pond</td></tr>
</table>

NSW Conditions of Consent (SSD 6766)	**Activities undertaken and proposed mitigation measures and commitments**
	• Installation and sampling from a newly installed groundwater monitoring well adjacent the Georges River • Re-drilling, installation and groundwater sampling from two monitoring wells (BH5 and BH18) • Excavation and sampling from 1 test pit within the proposed location of the stormwater pond extension, and 1 test pit within the proposed location of the new leachate pond.
b) Details of the quantity of landfilled waste to be removed, the location from where it will be removed, the methodology to be utilised and the estimated timeframe for removal and reburial.	Based on information provided by CPB Contractors, the current design levels are such that excavation works are not expected to extend to depths that will require the relocation of material within the project boundary or removal of landfill waste. Notwithstanding the above, erring on the side of caution it is reasonable to consider that a limited quantity of the landfilled waste may be exposed during the construction of the working platform for the surcharge / preloading activities in the southern end of the GWS Facility, between approximate Chainage 40650 to CH4070 (MB2S), as shown in Attachment B, where earth embankment is proposed to be constructed over former cells. The volume of waste to be exhumed (if any) is unknown at this stage although is estimated to be relatively small (i.e. less than 500m^3). Exhumed waste (if any) would be transported to, and placed within open and operational landfill cells within the existing GWS Facility. The timeframe for re-burial is not known but given the Facility will remain an operational landfill located adjacent to Rail Link, it is expected that re-burial would take place within a matter of days following exhumation in accordance with GWS operating procedures. The surcharge/preloading activities are expected to take approximately 3-6 months to complete, with waste exhumed (if any) only during the initial part of that period.

NSW Conditions of Consent (SSD 6766)	Activities undertaken and proposed mitigation measures and commitments
c) Proposed measures to mitigate odour impacts on sensitive receivers, including an undertaking to apply daily cover to any exposed waste in accordance with Waste benchmark technique 33 of the NSW EPA 1996 Solid Waste Landfills Guideline	As noted above, the construction methodology is not expected to expose buried waste materials, although the ground improvement proposed may give rise to odour emissions. Given that the nearest sensitive receivers are approximately 700m west and 1km south of the site, it is considered that potential odour impacts on sensitive receivers during construction period will be minimal and will be in line with existing operations within the GWS Facility. Notwithstanding the potential for odour impacts on offsite sensitive receptors being minimal, for the comfort of construction workers, one or more of the odour control measures outlined below will be implemented, should odour causing waste material be excavated: a) Should odour be considered a nuisance to construction workers, then spray the odour causing material with odour suppressant (e.g. BioSolve™ or similar). b) Should the odour causing material need to be temporarily stockpiled onsite for later reburial elsewhere or offsite disposal, then the material will be covered with suitable plastic sheeting or other construction spoil that is free of odour causing waste material and also not contaminated by chemical constituents. c) To the extent practicable, minimise the time the odour causing material is maintained in a temporary stockpile.
d) Details of impacts on pollution control and monitoring systems including existing groundwater and landfill gas bores and their subsequent repair / replacement	With reference to the design drawings (Attachment A), several existing GWS monitoring wells are located within close proximity to the construction zone. CPB Contractors will endeavour to avoid causing damage to the existing groundwater and gas monitoring bores that were installed for pollution control and monitoring systems. To avoid impact to landfill infrastructure, CPB Contractors is to identify landfill infrastructure on specific site environmental plan(s), design drawings, and relevant Work Pack(s) covering the landfill site, as well as fence off landfill infrastructure from construction activities, and undertake regular monitoring to ensure fences are functional and landfill infrastructure is protected from construction activities. Construction personnel working within the landfill site are to be “toolboxed” on this requirement. Should any damage to monitoring wells occur these will be repaired or replaced as soon as practicable. Where replacement bore needs to be reinstalled, they will be installed as close to original locations as possible. The screen interval of the replacement well will be similar to the well that it is replacing. These works will be done in consultation with GWS and EPA as required.

e) The methodology proposed to ensure that the landfill barrier system disturbed in the removal process is replaced/ repaired to ensure its ongoing performance. The Applicant shall detail matters such as sub grade preparation and specifications, liner installation/reinstallation procedure and CQA procedures	The ground improvement methodology outlined in Section 2 above was specially selected because it allowed the construction of the Rail Link embankment without exposing buried waste, and it prevented physical damage to the existing leachate barrier and collection system. To mitigate potential impacts to the existing landfill cells and potential leachate generation issues arising from the proposed surcharge works, the following measures will be implemented: • Placement of a low permeability geosynthetic liner over the existing landfill cell to restrict infiltration of surface water. Runoff shall be channelled towards swale drains (away from landfill operations) constructed adjacent to the rail embankment. A CQA report will be provided to the EPA as requested for the installation of the Geosynthetic liner over existing landfill areas (surcharge/preload areas). The CQA report should be prepared in accordance with the NSW EPA (2016) Solid Waste Guidelines, and AS3905. • Placement of General/Selected Embankment Fill to a maximum thickness of 9m, to promote gradual settlement of the landfilled waste. Coffey had assessed this solution and determined that it meets the primary objective of the EPA Environmental Guidelines for Solid Waste Landfills (2016), in that rainwater infiltration is significantly reduced. The Memorandum and Infiltration Assessment from Coffey has been included in Attachment D. Coffey have prepared the following documents which provides an assessment of: • The effect of dynamic compaction impact on the clay liner (GEOTLCOV24072AG-AG). • The effect of the surcharge ground improvement technique on the GCL liner (GEOTLCOV24072AG-AH). The design within the GWS Facility (in particular the design over the landfill cells) has changed since these two documents were prepared (most notably, the replacement of dynamic compaction ground improvement technique with pre loading surcharge described in Section 2.2). Coffey's review of the current design concludes that the proposed ground improvements would not alter the conclusions outlined within these two documents. The current geotechnical design for the landfill cell area of the GWS Facility is detailed in the following report: • Ground Treatment Design between CH40,440 and CH 40,740 (MBS2) (N01031-GRW-DRP-GEO-0001-05). The above report is included in Attachment D. A groundwater monitoring program is proposed to be implemented to monitor groundwater conditions during, and for a minimum of three months after the pre loading surcharge works.

NSW Conditions of Consent (SSD 6766)	Activities undertaken and proposed mitigation measures and commitments
f) A commitment to providing the EPA with a CQA report within 60 days of the completion of the works referred to in (d) above	CPB Contractors confirms that details of replaced bores will be provided to EPA within 60 days of the completion of the works referred to in (d) above.
g) An overview of any access and/or materials / equipment storage arrangements with Glenfield Waste Facility in relation to the construction of the project, and operation and maintenance of the Rail Link.	Access to the GWS Facility during construction will be from Cambridge Avenue. A construction materials and equipment storage area will be located at the Western Compound and the location nominated in agreement with GWS as per Attachment C. There are no current plans to establish any material or equipment storage facilities for the operation and maintenance of the Rail Link within the GWS Facility. Access to the Rail Link through the GWS Facility is subject to an agreement between SIMTA and GWS, and is expected to a combination of public roads to the GWS site, GWS internals road and Maintenance Access Roads along the Rail Link. The landowners of the GWS Facility intend to construct an alternate access route from Cambridge Avenue through the GWS Facility under the Eastern East Hills Line underpass, which avoids the existing weighbridge. Once completed under their planning approvals, the landowners of GWS have instructed CPB to use this access route as its primary access route through the GWS Facility. As such, it will be added to the CPB EPL as the designated main access route. Once RALP works are complete, the access route will be handed back to GWS for their ongoing operations. A secondary access route will be via the back entrance to the GWS Facility, located off Railway Parade, near where it meets the roundabout. This access route will only be used where the Eastern East Hills Line underpass is not suitable for use and will be subject to the approval of the landowner of the GWS Facility. Both proposed access routes avoid the existing weighbridge for purposes of GWS Facility operations.
h) Details of any other expected or potential impacts to the licensed area and options for management and mitigation of those impacts (i.e., leachate management and surface water runoff, potential impacts on the Georges River during works, dust etc.).	The GWS Facility is currently operating under two Environment Protection Licences (EPL); EPL # 4614 & EPL # 20974. EPL #20974 relates to resource recovery operations south of the East Hills Railway Line (EHRL) and EPL #4614 relates to operations north of EHRL where the Rail Link is proposed to traverse. A variation to CPB Contractor's EPL (EPL # 20966) is required for the construction activities associated with the Rail Link through the GWS Facility, although details of the license applicable to GWS were not available at the time this report was compiled. CPB's EPL includes the scheduled activity "extractive activities" and does not include a number of the other scheduled activities that are included in the GWS EPL. Once CPB receive an EPL for the construction of the Rail Link section which passes through the GWS Facility, the

NSW Conditions of Consent (SSD 6766)	Activities undertaken and proposed mitigation measures and commitments
	requirements of the EPL will be incorporated into the management plans for the project. The proposed construction zone will not form part of the GWS Facility operations during the construction period. The construction zone of the Rail Link will be delineated from GWS operations. Delineation will include fencing, bollards, barriers or other mechanisms as appropriate for a specific location. Since the proposed RALP construction works will be undertaken under an EPL separate to the extant GWS EPLs and a project specific Construction Environmental Management Plan, appropriate environmental mitigation measures will be adopted during construction works to minimise potential impacts during construction. The preloading/surcharge works may give rise to a temporary increase in leachate levels beneath the preload embankment, however this increase is expected to only last for a short period of time, which is less than the total duration for preloading. Additionally, as discussed in Item (e) above, a groundwater monitoring plan is proposed to be implemented during the construction activities. As discussed in Section 2 above, the extension of the stormwater basin, including installation/adjustment of associated infrastructure (i.e. pipes), by the landowners of the GWS Facility will be undertaken prior to the commencement of construction of the Rail Link. As discussed in Section 2.3 above, the existing leachate pond will be relocated by the landowner of the GWS Facility prior to the commencement of construction of the Rail Link in this area. The proposed location of the new leachate pond is in the northern section of the GWS Facility, between the northern and southern connector (refer Attachment A) of the Rail Link. The process to relocate the leachate pond would generally follow the sequence outlined in Section 2.3. GWS will continue to have access to the surface water pond and new leachate pond during the course of the Rail Link construction activities. Pipelines will be constructed connecting the landfill cells and the new leachate pond. Surface water within the Rail Link will be diverted away from the landfill, and this water will be channelled towards the stormwater pond. Run-off from the landfill would not be channelled towards the stormwater pond.

NSW Conditions of Consent (SSD 6766)	Activities undertaken and proposed mitigation measures and commitments
i) Details of and proposed mitigation measures for the long term management of the Rail Link (eg, subsidence or gas issues)	Coffey was engaged by CPB Contractors to assess and report on the Rail Link design to ensure it is geotechnically and structurally stable, as well as confirm landfill infrastructure is not adversely impacted during both construction and operation of the Rail Link. The report titled “Ground Treatment Design between Ch 40,440 and Ch 40,740 (MB2S)” (Coffey, July 2018) considered geotechnical and structural stability issues and confirmed the validity of the Rail Link design and appropriateness of design parameters through the GWS landfill Facility. The intrusive investigation has identified a lead contamination hotspot in fill soils beneath the Southern Connection. The extent of this material has been confirmed and will be capped as part of the Rail Link in accordance with the approved RAP. Ground gases are expected to be an issue that will be applicable during operational phase. There exists the potential for gas ingress and accumulation in underground service pits. The RAP (Coffey, 2018) recognises the need to have provision for gas mitigation in these pits and includes appropriate mitigations measures to mitigate this risk. Some works to be undertaken within these pits during maintenance service may need to be treated as confined space where personal protective equipment such as gas monitoring and/or personal monitoring sensors may be required. The Southern Connection section of the Rail Link will be constructed adjacent to the future Cell X, where waste will be placed. The potential landfill gas ingress into service pits within the Southern Connection associated with the future placement of waste within Cell X will be managed by the Rail Link operator in accordance with a long term Environmental Management Plan. Potential post-construction intervention (i.e. re-levelling with the addition of ballast and tamping) has been assessed as part of the geotechnical detailed design activities. Assuming that intervention will take place when post-construction settlement exceeds 100 mm or differential settlement exceeds 0.25% since the last intervention, the anticipated interventions are at 0.5, 1.3, 4.4, 19, and 40 years after track commissioning. About 5 years after track commissioning, re-tamping of ballast may need to take place at more frequent intervals than assessed above, for normal rail operation reasons rather than post-construction secondary consolidation settlement which is expected to have slowed down significantly at this time. Hence, during the maintenance works carried out after about 5 years from track commissioning, re-adjustment/re-levelling of the track is planned to or may need to be carried out.

4. Closing Remarks

Whilst design details and construction methodologies are subject to change, the overall impact assessments and corresponding mitigation measures outlined herein, is expected to by-and-large remain unchanged and relevant for the detailed design and construction phases.

Should a significant change to the design and/or construction methodology outlined above to be modified significantly, then a revision of this document will be undertaken and issued to the relevant stakeholders in accordance with Condition C5 (SSD 6766).

Please do not hesitate to contact the undersigned should you have any queries regarding this draft report.

For and on behalf of Coffey

[redacted]

[redacted]

Attachments

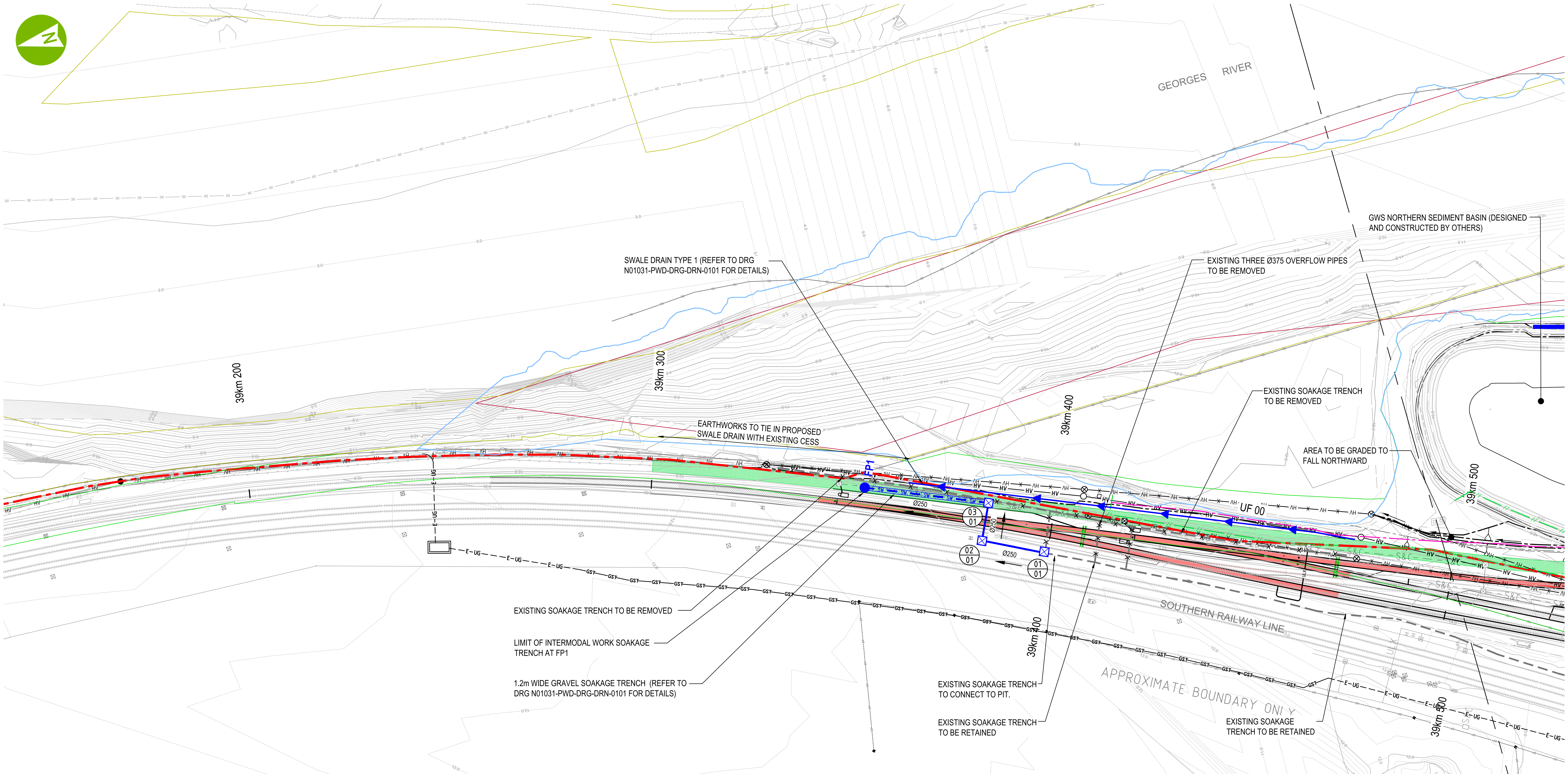
Attachment A – Design drawings

Attachment B – Proposed cut section within GWS Facility

Attachment C – Construction compound location and proposed access route

Attachment D - Landfill design assessment documents

Attachment A – Design drawings



PLAN
1:500

LEGEND:

KEY PLAN
N.T.S

Ø250	RALP STORMWATER PIPE (SLOTTED), SIZE & FLOW DIRECTION
Ø900	GWS STORMWATER PIPE SIZE & FLOW DIRECTION
	GWS DRAINAGE CHANNEL
	RALP STORMWATER PIT/INSPECTION COVER
02 / 01	STORMWATER PIT NUMBER / STORMWATER LINE NUMBER
	RALP Ø375 FRC HALF PIPE
	RALP DRAINAGE CHANNEL
	GWS PRE-CLOSURE DRAINAGE CHANNEL
Ø250	RALP STORMWATER PIPE (UNSLOTTED), SIZE & FLOW DIRECTION
	SOAKAGE TRENCH
FP	RALP FLUSHING POINT
	RALP HEADWALL
GPT	GROSS POLLUTANT TRAP
	Ø150 DOWNPIPE (15.0m SPACINGS)
	GWS STORMWATER PIT
	SCOUR PROTECTION
	BATTER CHUTE
	GWS POST-CLOSURE DRAINAGE CHANNEL
	TRENCH GRATE
	PROPOSED TRACK ALIGNMENT
	SIMTA RAIL CORRIDOR
Ø300	GWS PIPE SLEEVE SIZE & FLOW DIRECTION
	GWS HEADWALL
	CONTOUR RALP
	PROPOSED ACCESS ROAD
	FUTURE INTERMODAL TERMINAL
	GABION WEIR
Ø450 C	RALP STORMWATER CULVERT, SIZE, NAME AND FLOW DIRECTION
SS	SUBSOIL DRAINAGE PIPE
S&C	SIGNALLING & COMMS
	EXISTING SOAKAGE TRENCH
	PROPOSED RALP AND GWS SHARED ACCESS ROAD
	VOLUNTARY PLANNING AGREEMENT (VPA) BOUNDARY
	RIPARIAN VEGETATION MANAGEMENT
	AREA OF CULTURAL VALUE
	PROPOSED GWS ACCESS ROAD
	2015 EIS APPROVED CONSTRUCTION BOUNDARY
	SIMTA BIODIVERSITY OFFSET AREA
	RAILCORP/ARTC RAIL CORRIDOR
	FUTURE NOISE WALL
	CADASTRAL BOUNDARY
	FENCE

SCALE 1:500 (5 0 10 20m)

NOTES:

1. REFER TO SERVICES & UTILITIES PACKAGE FOR EXISTING AND PROPOSED UTILITIES.

ARTC DRAWING No | EDMS No | EDMS REV

aurecon
www.aurecongroup.com

CLIENT: CPB CONTRACTORS; SIMTA SYDNEY INTERMODAL TERMINAL ALLIANCE

REV	DATE	REVISION DETAILS	APPROVED
[redacted]			

SCALE	SIZE
AS SHOWN	A1

DRAWN [redacted]
DESIGNED [redacted]
CHECKED [redacted]

PRELIMINARY
NOT FOR CONSTRUCTION
APPROVED
DATE

PROJECT: MOOREBANK INTERMODAL TERMINAL DEVELOPMENT PACKAGE 1- RALP No.1

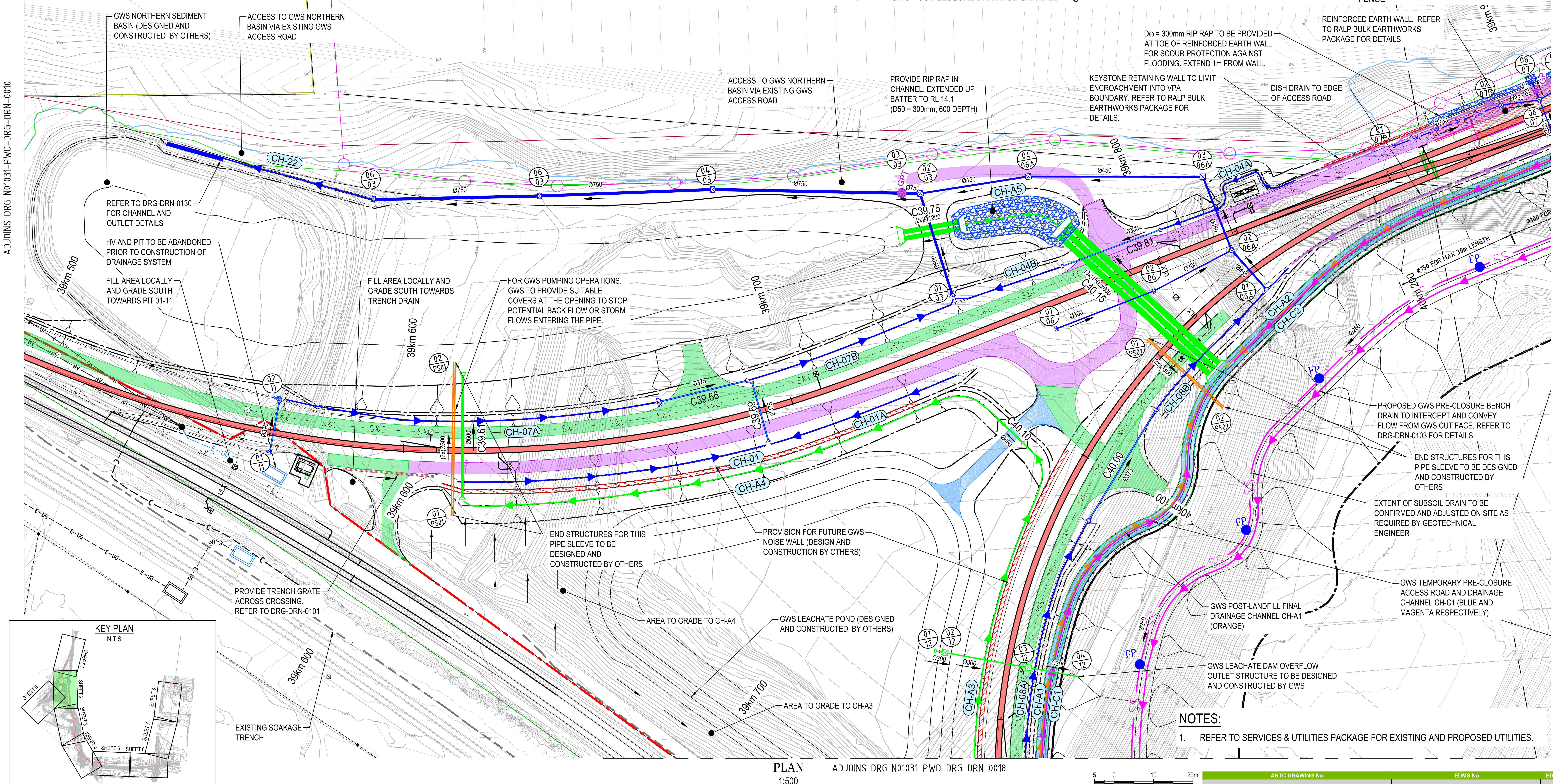
TITLE: DRAINAGE GENERAL ARRANGEMENT PLAN SHEET 1 OF 9

DRAWING No.	PROJECT No.	ZONE	TYPE	DISC	NUMBER	REV
	N01031	PWD	DRG	DRN	0010	E1

Plot Date: 3/7/2018 5:21:43 PM Office: AUSYD Filename: C:\PW_WORK\ASHLEY.LITTLE\ANZ_PROD\0029075\N01031-PWD-DRG-DRN-0010.DWG

LEGEND:

Ø250	RALP STORMWATER PIPE (SLOTTED), SIZE & FLOW DIRECTION
Ø900	GWS STORMWATER PIPE SIZE & FLOW DIRECTION
	GWS DRAINAGE CHANNEL
	RALP STORMWATER PIT/INSPECTION COVER
02 / 01	STORMWATER PIT NUMBER / STORMWATER LINE NUMBER
	RALP Ø375 FRC HALF PIPE
	RALP DRAINAGE CHANNEL
	GWS PRE-CLOSURE DRAINAGE CHANNEL
Ø250	RALP STORMWATER PIPE (UNSLOTTED), SIZE & FLOW DIRECTION
	SOAKAGE TRENCH
FP	RALP FLUSHING POINT
	RALP HEADWALL
GPT	GROSS POLLUTANT TRAP
	Ø150 DOWNPIPE (15.0m SPACINGS)
	GWS STORMWATER PIT
	SCOUR PROTECTION
	BATTER CHUTE
	GWS POST-CLOSURE DRAINAGE CHANNEL
	TRENCH GRATE
	PROPOSED TRACK ALIGNMENT
	SIMTA RAIL CORRIDOR
Ø300	GWS PIPE SLEEVE SIZE & FLOW DIRECTION
	GWS HEADWALL
	CONTOUR RALP
	PROPOSED ACCESS ROAD
	FUTURE INTERMODAL TERMINAL
	GABION WEIR
Ø450 C	RALP STORMWATER CULVERT, SIZE, NAME AND FLOW DIRECTION
SS	SUBSOIL DRAINAGE PIPE
S&C	SIGNALLING & COMMS
	EXISTING SOAKAGE TRENCH
	PROPOSED RALP AND GWS SHARED ACCESS ROAD
	VOLUNTARY PLANNING AGREEMENT (VPA) BOUNDARY
	RIPARIAN VEGETATION MANAGEMENT
	AREA OF CULTURAL VALUE
	PROPOSED GWS ACCESS ROAD
	2015 EIS APPROVED CONSTRUCTION BOUNDARY
	SIMTA BIODIVERSITY OFFSET AREA
	RAILCORP/ARTC RAIL CORRIDOR
	FUTURE NOISE WALL
	CADASTRAL BOUNDARY
	FENCE



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ADJOINS DRG N01031-PWD-DRG-DRN-0018

SCALE 1:500

NOTES:

1. REFER TO SERVICES & UTILITIES PACKAGE FOR EXISTING AND PROPOSED UTILITIES.

Plot Date: 3/7/2018 5:21:47 PM Office: AUSYD Filename: C:\PW_WORKASHLEY.LITTLE\ANZ_PROD\...\N01031-PWD-DRG-DRN-0011.DWG

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SIMTA SYDNEY INTERMODAL TERMINAL ALLIANCE

CLIENT	REV	DATE	REVISION DETAILS	APPROVED
	[redacted]			

SCALE	SIZE
AS SHOWN	A1

DRAWN [redacted]
DESIGNED [redacted]
CHECKED [redacted]

PRELIMINARY
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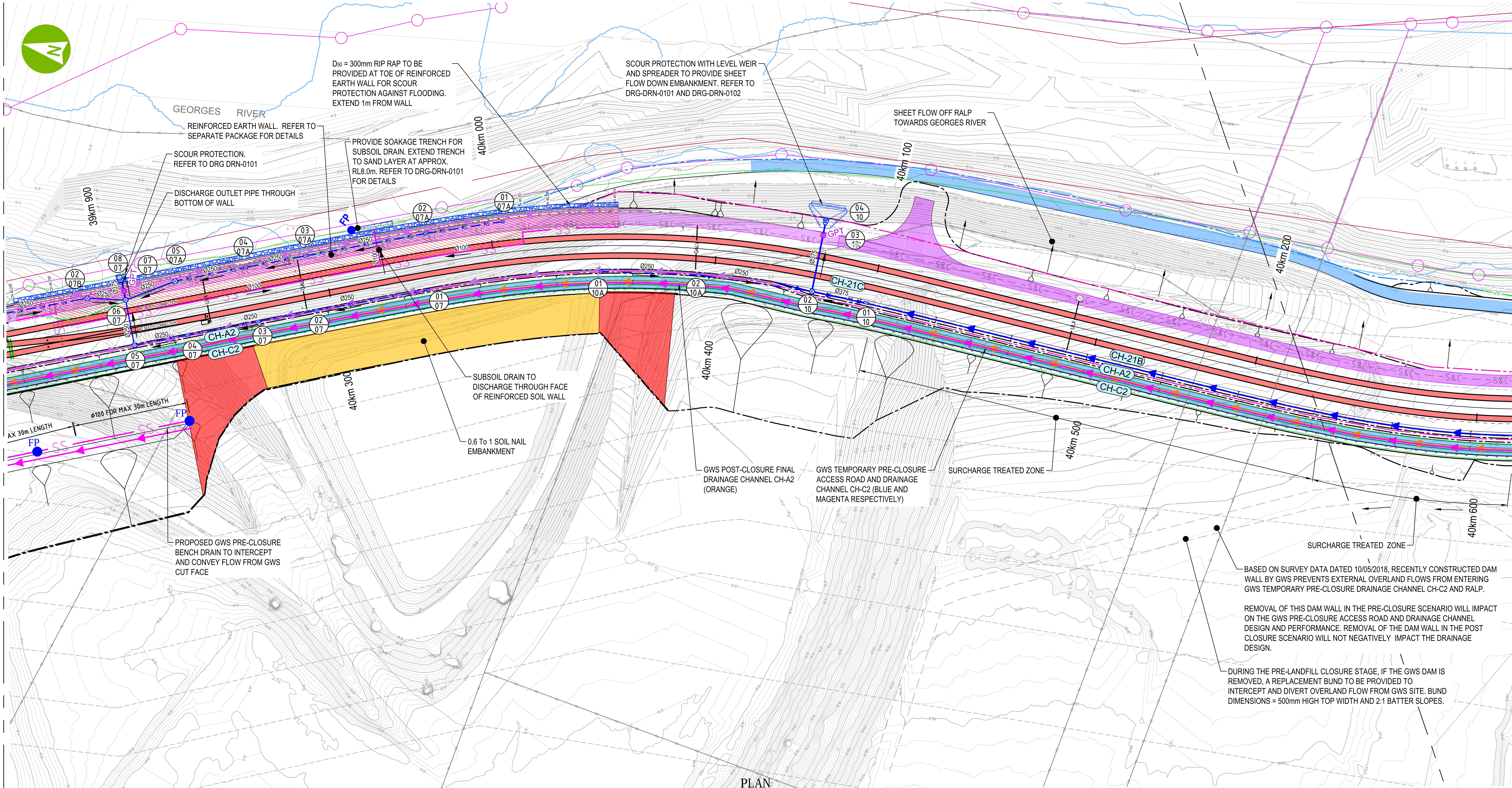
ARTC DRAWING No | EDMS No | EDMS REV

PROJECT: MOOREBANK INTERMODAL TERMINAL DEVELOPMENT PACKAGE 1- RALP No.1

TITLE: DRAINAGE GENERAL ARRANGEMENT PLAN SHEET 2 OF 9

DRAWING No.	PROJECT No.	ZONE	TYPE	DISC	NUMBER	REV
	N01031	PWD	DRG	DRN	0011	E1



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LEGEND:

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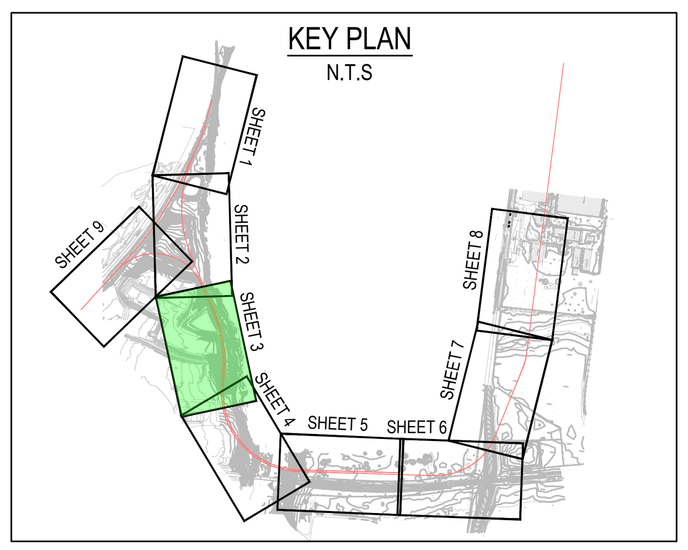
Ø900 GWS STORMWATER PIPE SIZE & FLOW DIRECTION	SOAKAGE TRENCH	PROPOSED TRACK ALIGNMENT	SIGNALLING & COMMS
GWS DRAINAGE CHANNEL	FP RALP FLUSHING POINT	SIMTA RAIL CORRIDOR	EXISTING SOAKAGE TRENCH
RALP STORMWATER PIT/INSPECTION COVER	RALP HEADWALL	Ø300 GWS PIPE SLEEVE SIZE & FLOW DIRECTION	PROPOSED RALP AND GWS SHARED ACCESS ROAD
02/01 STORMWATER PIT NUMBER / STORMWATER LINE NUMBER	GPT GROSS POLLUTANT TRAP	GWS HEADWALL	VOLUNTARY PLANNING AGREEMENT (VPA) BOUNDARY
RALP Ø375 FRC HALF PIPE	Ø150 DOWNPIPE (15.0m SPACINGS)	CONTOUR RALP	RIPARIAN VEGETATION MANAGEMENT
RALP DRAINAGE CHANNEL	GWS STORMWATER PIT	PROPOSED ACCESS ROAD	AREA OF CULTURAL VALUE
GWS PRE-CLOSURE DRAINAGE CHANNEL	SCOUR PROTECTION	FUTURE INTERMODAL TERMINAL	PROPOSED GWS ACCESS ROAD
	BATTER CHUTE	GABION WEIR	2015 EIS APPROVED CONSTRUCTION BOUNDARY
	GWS POST-CLOSURE DRAINAGE CHANNEL	Ø450 C RALP STORMWATER CULVERT, SIZE, NAME AND FLOW DIRECTION	SIMTA BIODIVERSITY OFFSET AREA
			RAILCORP/ARTC RAIL CORRIDOR
			FUTURE NOISE WALL
			CADASTRAL BOUNDARY
			FENCE

KEY PLAN
N.T.S



NOTES:

1. REFER TO SERVICES & UTILITIES PACKAGE FOR EXISTING AND PROPOSED UTILITIES.
2. REFER TO REINFORCED EARTH WALL DESIGN DRAWING N01031-PWS-DRG-GEN-0070

SCALE 1:500 (0 10 20m)

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REV	DATE	REVISION DETAILS	APPROVED
[redacted]			

SCALE	SIZE
AS SHOWN	A1

DRAWN: [redacted]
DESIGNED: [redacted]
CHECKED: [redacted]

PRELIMINARY
NOT FOR CONSTRUCTION

APPROVED
DATE

ARTC DRAWING No | EDMS No | EDMS REV

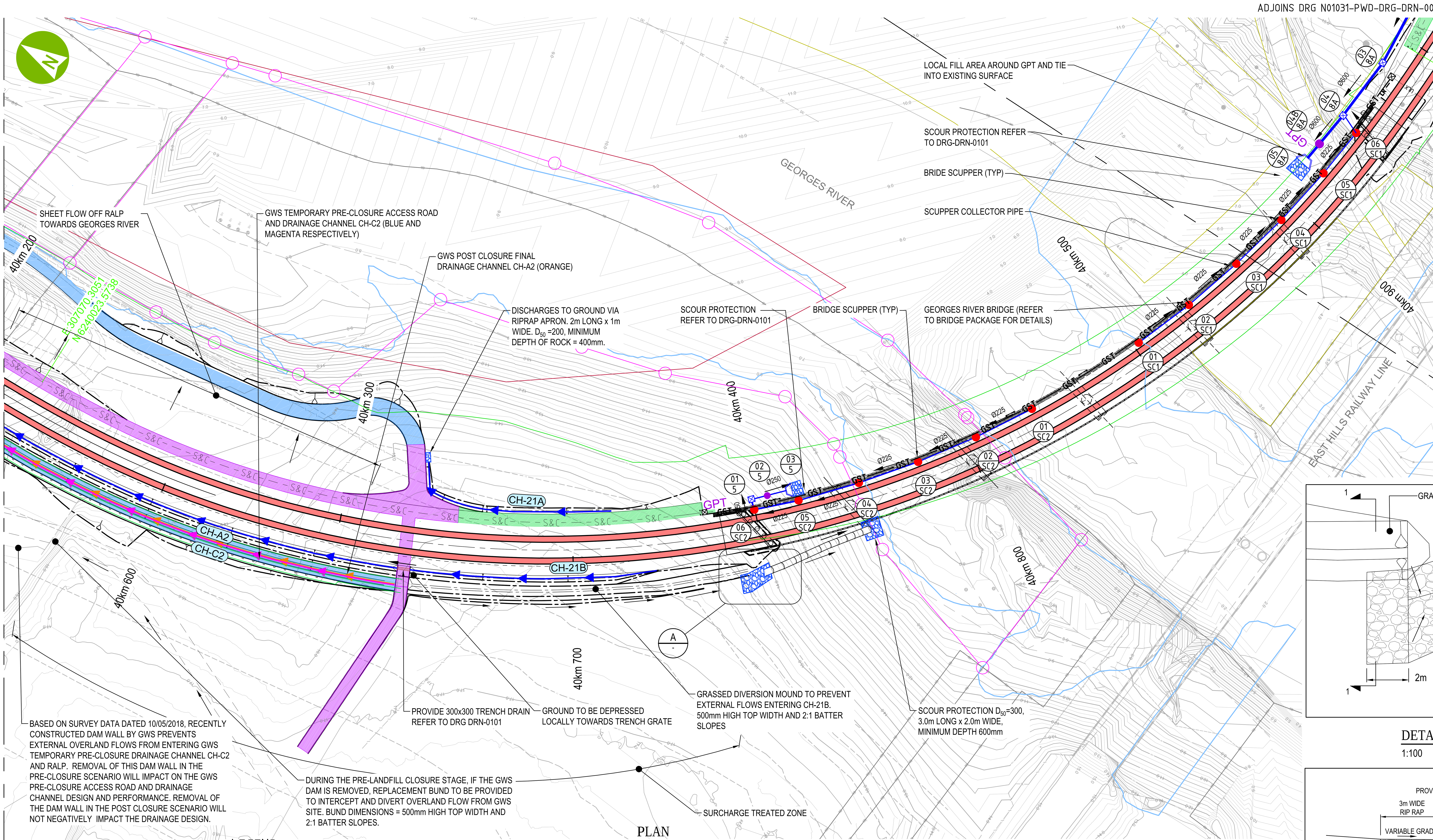
PROJECT: MOOREBANK INTERMODAL TERMINAL DEVELOPMENT PACKAGE 1- RALP No.1

TITLE: DRAINAGE GENERAL ARRANGEMENT PLAN SHEET 3 OF 9

DRAWING No.	PROJECT No.	ZONE	TYPE	DISC	NUMBER	REV
	N01031	PWD	DRG	DRN	0012	E1

ADJOINS DRG N01031-PWD-DRG-DRN-0014

ADJOINS DRG N01031-PWD-DRG-DRN-0012



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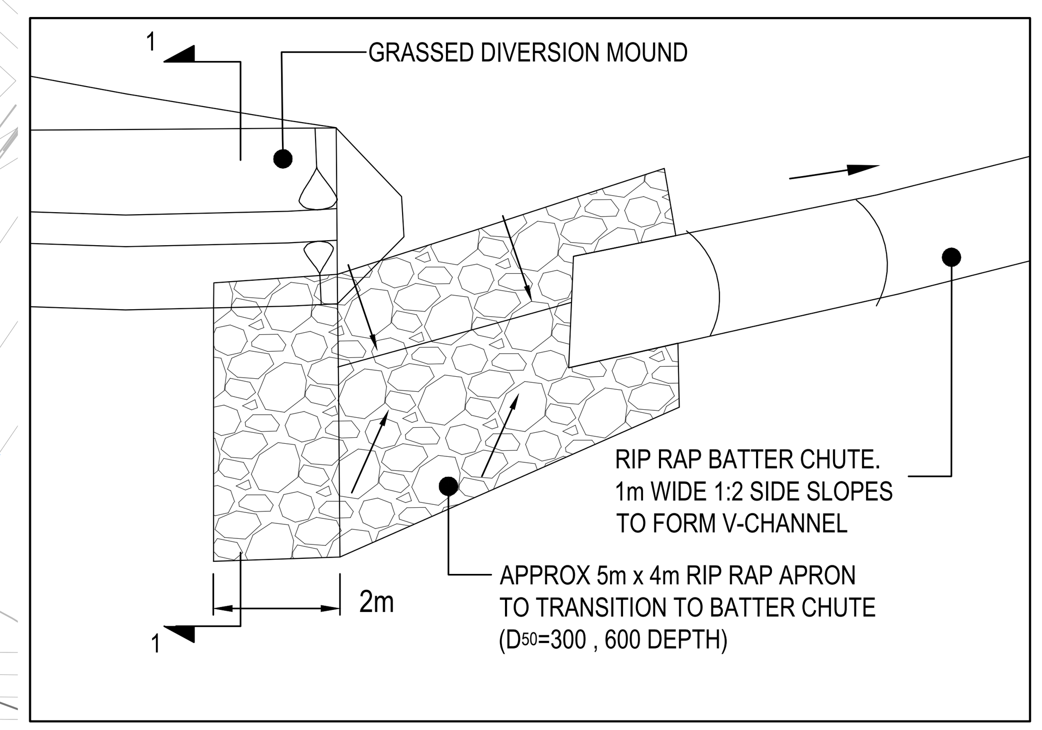

DETAIL A
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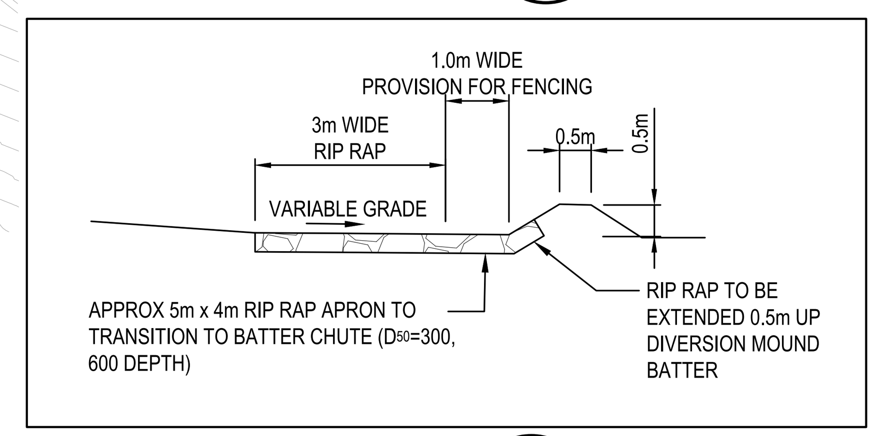

SECTION 1
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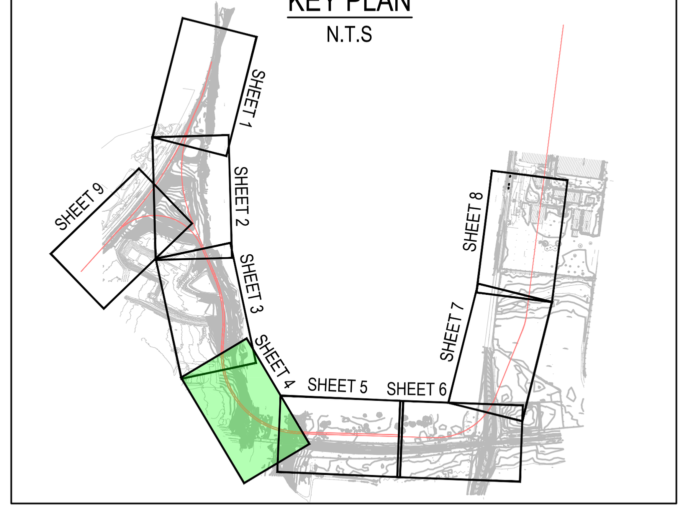
KEY PLAN

N.T.S



LEGEND:

- Ø250 — RALP STORMWATER PIPE (SLOTTED), SIZE & FLOW DIRECTION
- Ø900 — GWS STORMWATER PIPE SIZE & FLOW DIRECTION
- GWS DRAINAGE CHANNEL
- RALP STORMWATER PIT/INSPECTION COVER
- STORMWATER PIT NUMBER / STORMWATER LINE NUMBER (02 / 01)
- RALP Ø375 FRC HALF PIPE
- RALP DRAINAGE CHANNEL
- GWS PRE-CLOSURE DRAINAGE CHANNEL
- Ø250 — RALP STORMWATER PIPE (UNSLOTTED), SIZE & FLOW DIRECTION
- SOAKAGE TRENCH
- FP — RALP FLUSHING POINT
- RALP HEADWALL
- GPT — GROSS POLLUTANT TRAP
- Ø150 DOWNPIPE (15.0m SPACINGS)
- GWS STORMWATER PIT
- SCOUR PROTECTION
- BATTER CHUTE
- GWS POST-CLOSURE DRAINAGE CHANNEL
- TRENCH GRATE
- PROPOSED TRACK ALIGNMENT
- SIMTA RAIL CORRIDOR
- Ø300 — GWS PIPE SLEEVE SIZE & FLOW DIRECTION
- GWS HEADWALL
- CONTOUR RALP
- PROPOSED ACCESS ROAD
- FUTURE INTERMODAL TERMINAL
- GABION WEIR
- Ø450 C — RALP STORMWATER CULVERT, SIZE, NAME AND FLOW DIRECTION
- SS — SUBSOIL DRAINAGE PIPE
- S&C — SIGNALLING & COMMS
- EXISTING SOAKAGE TRENCH
- PROPOSED RALP AND GWS SHARED ACCESS ROAD
- VOLUNTARY PLANNING AGREEMENT (VPA) BOUNDARY
- RIPARIAN VEGETATION MANAGEMENT
- AREA OF CULTURAL VALUE
- PROPOSED GWS ACCESS ROAD
- 2015 EIS APPROVED CONSTRUCTION BOUNDARY
- SIMTA BIODIVERSITY OFFSET AREA
- RAILCORP/ARTC RAIL CORRIDOR
- FUTURE NOISE WALL
- CADASTRAL BOUNDARY
- FENCE

SCALE 1:500 (5 0 10 20m)

NOTES:

1. REFER TO SERVICES & UTILITIES PACKAGE FOR EXISTING AND PROPOSED UTILITIES.

ARTC DRAWING No	EDMS No	EDMS REV

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CPB CONTRACTORS

SIMTA SYDNEY INTERMODAL TERMINAL ALLIANCE

CLIENT	REV	DATE	REVISION DETAILS	APPROVED
	[redacted]			

SCALE	SIZE
AS SHOWN	A1

DRAWN: [redacted]

DESIGNED: [redacted]

CHECKED: [redacted]

PRELIMINARY
NOT FOR CONSTRUCTION

APPROVED
DATE

PROJECT: MOOREBANK INTERMODAL TERMINAL DEVELOPMENT PACKAGE 1- RALP No.1

TITLE: DRAINAGE GENERAL ARRANGEMENT PLAN SHEET 4 OF 9

DRAWING No.	PROJECT No.	ZONE	TYPE	DISC	NUMBER	REV
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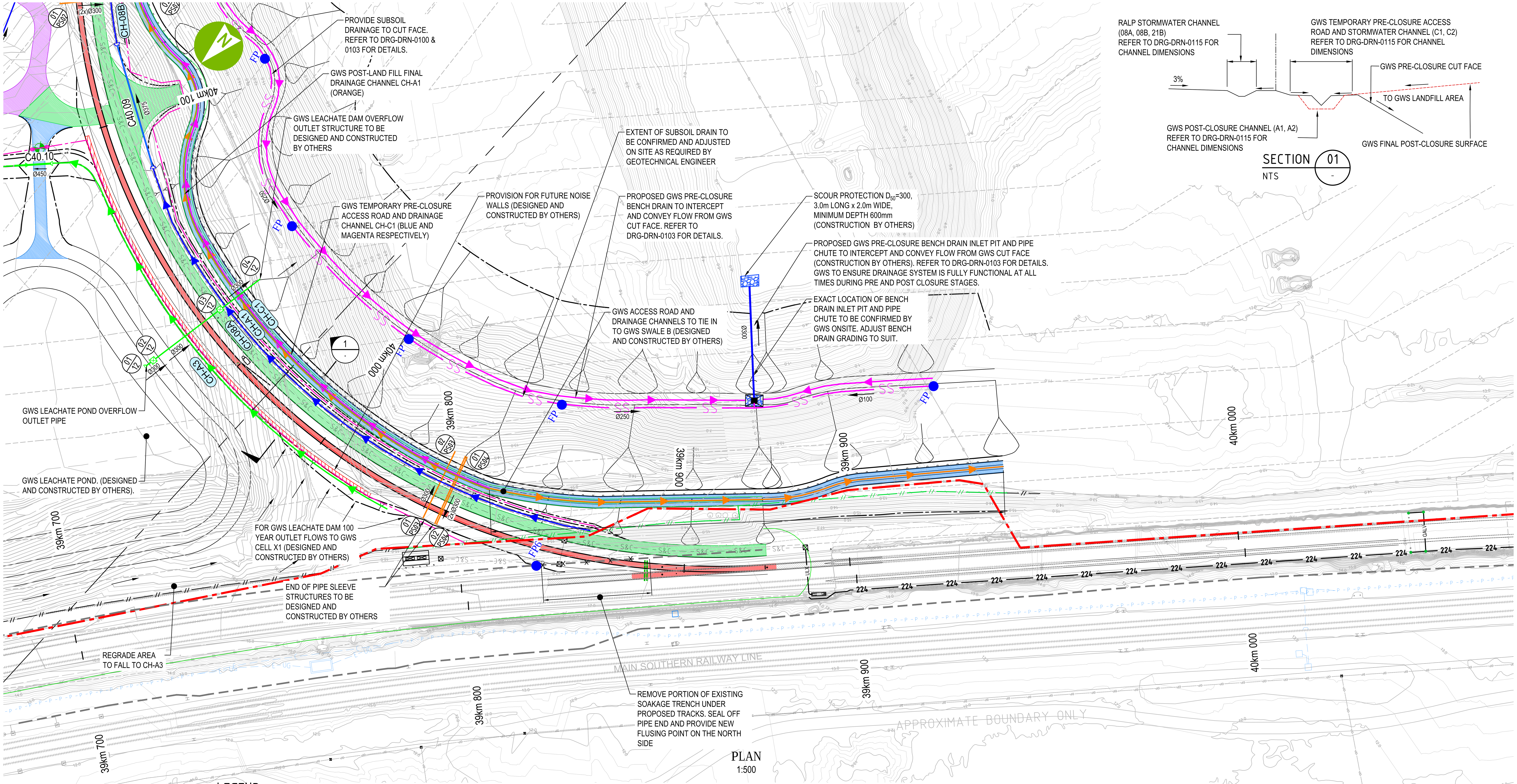

LEGEND:

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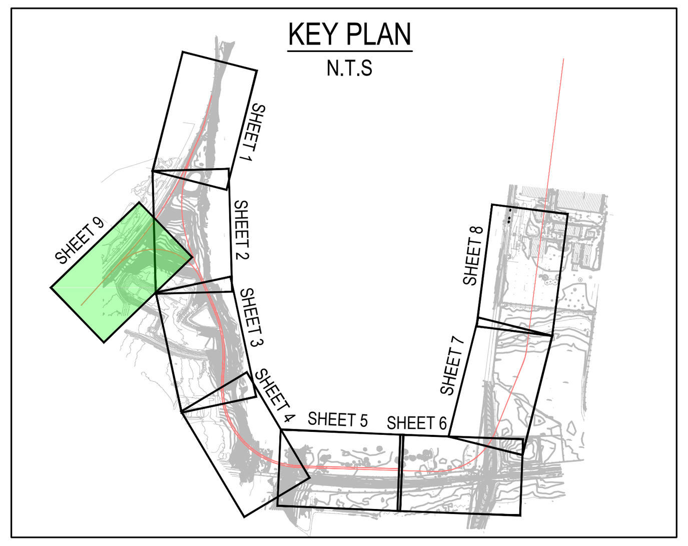
Ø900	GWS STORMWATER PIPE SIZE & FLOW DIRECTION		SOAKAGE TRENCH		PROPOSED TRACK ALIGNMENT	S&C	SIGNALLING & COMMS
	GWS DRAINAGE CHANNEL	FP	RALP FLUSHING POINT		SIMTA RAIL CORRIDOR		EXISTING SOAKAGE TRENCH
	RALP STORMWATER PIT/INSPECTION COVER		RALP HEADWALL	Ø300	GWS PIPE SLEEVE SIZE & FLOW DIRECTION		PROPOSED RALP AND GWS SHARED ACCESS ROAD
02	STORMWATER PIT NUMBER	GPT	GROSS POLLUTANT TRAP		GWS HEADWALL		VOLUNTARY PLANNING AGREEMENT (VPA) BOUNDARY
01	STORMWATER LINE NUMBER		Ø150 DOWNPIPE (15.0m SPACINGS)		CONTOUR RALP		RIPARIAN VEGETATION MANAGEMENT
	RALP Ø375 FRC HALF PIPE		GWS STORMWATER PIT		PROPOSED ACCESS ROAD		AREA OF CULTURAL VALUE
	RALP DRAINAGE CHANNEL		SCOUR PROTECTION		FUTURE INTERMODAL TERMINAL		PROPOSED GWS ACCESS ROAD
	GWS PRE-CLOSURE DRAINAGE CHANNEL		BATTER CHUTE		GABION WEIR		2015 EIS APPROVED CONSTRUCTION BOUNDARY
			GWS POST-CLOSURE DRAINAGE CHANNEL	Ø450 C	RALP STORMWATER CULVERT, SIZE, NAME AND FLOW DIRECTION		SIMTA BIODIVERSITY OFFSET AREA
							RAILCORP/ARTC RAIL CORRIDOR
							FUTURE NOISE WALL
							CADASTRAL BOUNDARY
							FENCE

KEY PLAN
N.T.S



NOTES:

1. REFER TO SERVICES & UTILITIES PACKAGE FOR EXISTING AND PROPOSED UTILITIES.

SCALE 1:500 — 5 0 10 20m

ARTC DRAWING No	EDMS No	EDMS REV

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CLIENT: CPB CONTRACTORS — SIMTA SYDNEY INTERMODAL TERMINAL ALLIANCE

REV	DATE	REVISION DETAILS	APPROVED
[redacted]			

SCALE	SIZE
AS SHOWN	A1

DRAWN: [redacted]
DESIGNED: [redacted]
CHECKED: [redacted]

PRELIMINARY
NOT FOR CONSTRUCTION

APPROVED
DATE

PROJECT: MOOREBANK INTERMODAL TERMINAL DEVELOPMENT PACKAGE 1- RALP No.1

TITLE: DRAINAGE GENERAL ARRANGEMENT PLAN SHEET 9 OF 9

DRAWING No.	PROJECT No.	ZONE	TYPE	DISC	NUMBER	REV
	N01031	PWD	DRG	DRN	0018	E1

Plot Date: 3/7/2018 5:22:04 PM Office: AUSYD Filename: C:\PW_WORK\ASHLEY.LITTLE\ANZ_PROD\00292753\N01031-PWD-DRG-DRN-0018.DWG

Attachment B – Proposed cut section within GWS Facility

[redacted]

rev	description	
A	ORIGINAL ISSUE	[redacted]

20 0 20 40 60 80 100

Scale (metres) 1:2000

drawn	[redacted]
approved	[redacted]
date	[redacted]
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client: CPB CONTRACTORS PTY LTD

project: MOOREBANK INTERMODAL RAIL LINK
PROPOSED CUT SECTION WITHIN GLENFIELD WASTE SERVICE SECTION
MOOREBANK, NSW

title: GLENFIELD WASTE SERVICES CONSTRUCTION
IMPACT ASSESSMENT REPORT

project no: GEOTLCOV24072AF-AV | figure no: ATTACHMENT B | rev: A

Attachment C – Construction compound location and proposed access route


LEGEND
Project Access Roads
Rail Design
RALP Compounds
0
0.125
0.25
0.5 Km

Attachment D – Landfill design assessment reports

coffey
A TETRA TECH COMPANY

Level 19, Tower B, 799 Pacific Highway
Chatswood
NSW 2067 Australia

[redacted]

coffey.com

Electronic Transmission

To	[redacted]	From	[redacted]
Email address	[redacted]	Date	13 April 2017
Company	CPB Contractors	Reference	GEOTLCOV24072AF-BN Rev 1
cc	[Cc]	Pages	1 of 5
Subject	Infiltration Performance Assessment - Rail Embankment within GWS Landfill area		

[redacted],

1. Introduction

The existing landfill cover layer will be disturbed during Dynamic Compaction (DC) works. Prior to the DC, a minimum 500 mm thick working platform will be added on the existing cover layer from Ch 40,560 to Ch 40,740 (MB2S). The anticipated settlement during DC work is about 1 m. Hence, the minimum overall thickness (approximately at Ch 40,680) of soil layer over the landfill after the DC work and rail embankment construction will be over 3 m. This soil layer will include rail capping layer, rail embankment, working platform and existing cover layer.

In accordance with the EPA guidelines, the sealing layer over landfill should be at least 600 mm thick, with an in-situ hydraulic conductivity of not more than 1×10^{-9} m/sec. The sealing layer is recommended to achieve “infiltration from the base of the final cap to be less than 5% of the annual rainfall".

An infiltration analyses has been carried out using commercially available software SEEP/W, at a critical rail embankment section (i.e. at Ch 40,680) to assess the infiltration performance within the rail embankment footprint and hence, compare with the EPA performance requirement as detailed above.

2. Model

Rail embankment at Ch 40,680 consists of following soil layers immediately above landfill and as shown in Figure 1:

- 150mm thick rail capping layer;
- 500mm thick formation layer (Structural fill);
- 500mm thick general embankment layer;
- 500mm thick working platform layer; and
- 1.5m thick existing cover layer.

The existing cover layer and working platform will be subjected to DC tamping and hence the level of compaction will be increased.

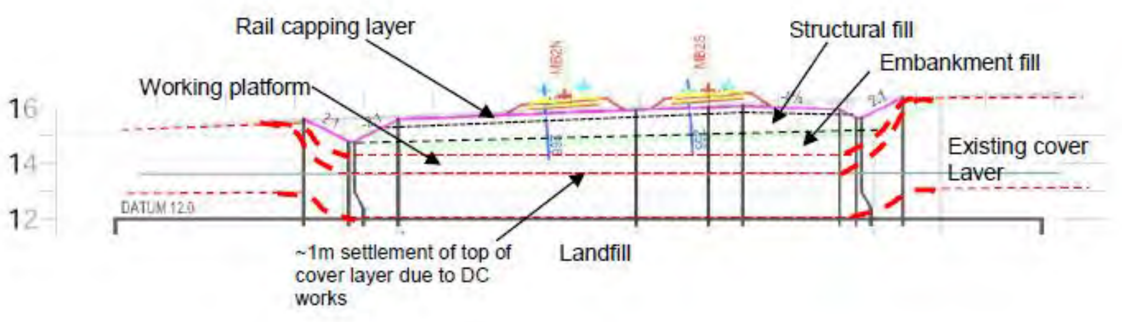


Figure 1: Rail embankment and subsurface layers at Ch 40,680

Our simplified SEEP/W model representing these conditions is presented in Figure 2 below:

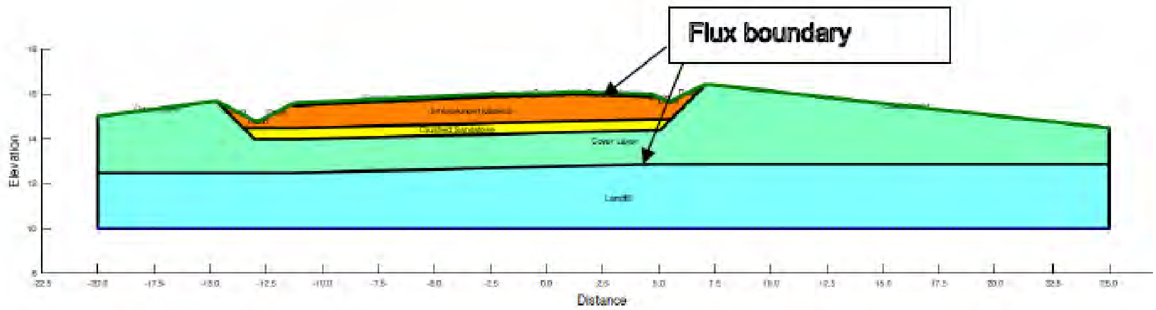


Figure 2: SEEP/W model at Ch 40,680

3. Infiltration parameters and rain events

3.1. Infiltration parameters

In-situ infiltration tests and laboratory permeability tests have been conducted on the existing cover layer material. As reported in memo GEOTLCOV24072AF-BP (refer Attachment 1), the assessed in-situ hydraulic conductivity of existing cover layer is in the order of 10^{-7} m/sec. Laboratory permeability test carried out on soil samples from existing cover layer provided saturated hydraulic conductivity in the order of 10^{-9} m/sec. As the existing cover layer is subjected to DC tamping including an ironing pass to address shallow soil disturbance, the anticipated in-situ hydraulic conductivity of the existing cover layer after DC works would be lower than 10^{-7} m/sec

However, in this analysis, in-situ hydraulic conductivity of 10^{-7} m/sec was adopted for existing cover layer.

Infiltration parameters adopted for the other soil layers are presented in Table 1

Table 1: Infiltration parameters

Layer	Layer thickness, (m)	Saturated permeability (m/s)	Volumetric water content[(1)]	Typical unsaturated soil characteristic curve[(2)]
Rail capping layer	0.15	10^{-9}	0.4	Clay
Structural fill/Embankment fill	1	10^{-5}		Gravel
Working platform	0.5	10^{-4}		Gravel
Existing Cover layer	1.5[(3)]	10^{-7}		Gravel

Note:

1) Typical value for well compacted gravel has been adopted.

2) Unsaturated soil characteristic curves have been assumed based on anticipated behaviour of the layers. Conservatively assumed gravely behaviour for layers other than the capping layer.

3) Although expected thickness of existing landfill cover layer is about 2m, a lower thickness is assumed.

3.2. Rainfall data

Annual rainfall data relevant to GWS landfill area has been assessed from two weather stations namely “Bankstown airport” and “Ingleburn”. Considering the higher average annual rain fall and number of rainy days, data from Bankstown airport weather station has been used for this analysis. Table 2 below summarise the rainfall data for year 2016 and average over years 1968 to 2016:

Table 2: Summary of annual rainfall data

	Year 2016	Average
Annual rainfall (mm)	973	895
Number of rain days/year	107	116

In addition to above annual rainfall events, two isolated rain events were considered in the analysis:

Table 3: Isolated rainfall events

Event	Description
Rain Event 1: Long duration rainfall event	Cumulative rainfall of 152mm over 17 days. With rain occurs in 4 consecutive days
Rain Event 2: High intensity rainfall event	Cumulative rainfall of 293mm over 3 days.

4. Analysis methodology and results

Steady state flow analysis was carried out based on following assumptions:

- Considering that the rail formation has been constructed with appropriate cross falls and longitudinal drainage is provided at the toe of the embankment, no water ponding is anticipated. During periods of rainfall saturated conditions at the surface of the rail capping layer are assumed for the full day for each day of rainfall. Unsaturated conditions with no water ingress are assumed to occur on days with no rainfall; and

- Evapotranspiration is not modelled. However, adopting only rainy days to assess the average annual flow is considered reasonable.

Results of steady state seepage analysis carried out using SEEP/W are summarised in Table 4 below:

Table 4: Results of steady state seepage flow analysis

Steady state flow	**Through capping layer**	**Through existing cover layer below rail embankment foot print**
m^3/day/m	0.009 over width of about 16m(1)	0.017 over width of about 45m(1)
mm/day (rain day only)	0.56	0.38
Annual rainfall event		
mm/year during anticipated rainy days	60	41
% as annual rain fall	6.2%	4.2%
Isolated rainfall events	~1.5%(2)	<1%(2)

[1] Refer Attachment 2: SEEP/W output plot for flow through the formation
[2] % as cumulative rainfall during the isolated rain event

5. Conclusion

Based on above assessment the assessed infiltration through existing cover layer in to landfill is about 4.2% of the average annual rainfall. Considering the conservative infiltration parameters adopted and evapotranspiration is not modelled exclusively, assessed infiltration of 4.2% of the average rainfall is considered conservative.

As the infiltration percentage in to the landfill is less than 5% of the average annual rainfall, no additional sealing layer is required within the embankment footprint.

Should you have any queries please do not hesitate to contact the undersigned.

For and on behalf of Coffey

[redacted]

[redacted]

Attachments:

Attachment 1: GEOTLCOV24072AF-BP – Factual test results on infiltration rate and hydraulic conductivity of existing cover layer

Attachment 2: SEEP/W output plot for flow through the formation

Attachment 1: GEOTLCOV24072AF-BP – Factual test results on infiltration rate and hydraulic conductivity of existing cover layer

Level 19, Tower B, 799 Pacific Highway
Chatswood
NSW 2067 Australia
[redacted]
coffey.com

Electronic Transmission

To	[redacted]	From	[redacted]
Email address	[redacted]	Date	10 April 2017
Company	CPB Contractors Pty Ltd	Reference	GEOTLCOV24072AF-BP
cc		Pages	3 + 3 Attachments
Subject	Factual Test Results on Infiltration Rate and Hydraulic Conductivity for existing cover layer of GWS landfill		

1. Introduction

As requested by CPB Contractors (CPB), Coffey has carried out a fieldwork on 23 March 2017 within the Glenfield Waste Service (GWS) facility as part of the Moorebank Intermodal Rail Link (MIRL) project. The fieldwork was carried out to undertake a number of in-situ tests and collect soil samples for the laboratory tests for the measurement of infiltration rate and hydraulic conductivity of the existing cover layer of the landfill. The works were commissioned by CPB Contractors Pty Ltd (CPB) in order to characterise the cover layers and refine the assessment of infiltration performance of these layers within the landfill area treated by Dynamic Compaction (DC) from Ch 40,560 to Ch 40,740 (MB2S).

This correspondence summarises the factual results of the in-situ and laboratory tests. The interpretation provided was undertaken to process the raw data for the assessment of parameters in accordance to the relevant standards and published literature.

2. Fieldwork and Laboratory Testing

The in-situ tests undertaken during the aforementioned fieldwork comprise the following:

- Two Double Ring Infiltration (DRI) Tests in accordance to the ASTMD3385-03 (Standard Test Method for Infiltration Rate of Soils in Field using Double- Ring Infiltrometer) to measure the incremental infiltration rate.
- Four Inversed Auger Hole (IAH) Tests or “Porchet Method” to measure saturated hydraulic conductivity (K).
- Two Field Density Testing (FDT) using the nuclear moisture-density gauge in accordance to AS1289.5.8.1 – 2007 to measure the field dry density and field moisture content values.

Locations of the abovementioned tests are shown in Figure 1 in the attachment. The FDT tests were carried out next to the locations of DRI tests. Two bulk samples were collected from the same locations as those of the FDT tests and transported to our NATA-accredited laboratory. Those samples were tested for the following:

- Two compaction tests using standard compaction to measure the Standard Maximum Dry Density (SMDD) and Standard Optimum Moisture Content (SOMC) in accordance to AS1289.5.1.1 – 2003, AS1289.2.1.1 – 2005 and AS1289.5.4.1 – 2007.

- Two Falling Head Permeability (FHP) tests to measure hydraulic conductivity of samples compacted to SMDD in accordance to AS1289.6.7.2 – 2001.

The test locations and materials observed at these test locations are summarised in Table 1 below.

Table 1 – Test locations and descriptions of observed materials

Test Location	Easting (m)	Northing (m)	Corresponding Chainage (MB2S)[a]	Material Description [b]
LC1	307065	6239996	40,585	Gravelly SAND with some clay
HA1	307067	6239993	40,588	Gravelly SAND with some clay
HA2	307071	6239978	40,610	A mix of gravel, sand and clay
HA3	307077	6239944	40,650	Gravelly SAND with some clay
HA4	307083	6239917	40,672	Gravelly CLAY with some sand
LC2	307088	6239913	40,675	Gravelly CLAY with some sand

Note:

a. The corresponding chainage is approximate only based on projection of coordinates
b. Based on observations of materials near the surface

The results of DRI tests at locations LC1 and LC2 are presented as Figures 2 to 3 in Appendix A. The results of IAH tests at locations HA1 to HA4 are presented as Figures 4 to 7 in Appendix B. The results of FDT, FHP and other laboratory tests are presented in Appendix C.

3. Conclusions and Limitations

The factual results of in-situ and laboratory tests are summarised in Table 2 below.

Table 2 – Summary of in-situ and laboratory testing results

Test ID/Test Locations	Infiltration Rate (cm/hr)		Saturated Hydraulic Conductivity (m/s)	Field/Laboratory Dry Density (t/m^3)
	Peak	End of Test		
In-situ Testing				
DRI – LC1[b]	0.26	0.024	Approx. 2×10^{-7} to 3×10^{-7} (Note a)	2.09
DRI – LC2[b]	0.21	0.07		1.80
IAH – HA1[b]	Not measured		3.8×10^{-7}	2.09
IAH – HA2	Not measured		3.7×10^{-7}	Not measured
IAH – HA3	Not measured		2.8×10^{-7}	Not measured
IAH – HA4[b]	Not measured		2.6×10^{-7}	1.80

Test ID/Test Locations	Infiltration Rate (cm/hr)		Saturated Hydraulic Conductivity (m/s)	Field/Laboratory Dry Density (t/m^3)
	Peak	End of Test		
Laboratory Testing				
FHP – LC1	Not measured		6.6×10^{-9}	2.09[c]
FHP – LC2	Not measured		4.5×10^{-10}	1.96[c]

Note:

a. Adopted hydraulic conductivity values provide typical depths of saturation profile. Hence, adopted hydraulic conductivity values are considered reasonable in comparison to the values of nearby IAH tests.
b. Locations LC1 and LC2 are situated within a close proximity to the locations HA1 and HA4, respectively
c. Denotes laboratory measured dry density

The following limitations should be noted:

- The permeability of the soil is also influenced by other properties of the soil including but not limited to the in-situ void ratio and dry density of the soil layers; and
- Subsurface soil conditions may vary over a distance; and
- Subsurface soil conditions may vary over the depth including any localised presence of soil layers with varying permeability.

The attached document entitled “Important Information about Your Coffey Report” presents additional information on the uses and limitations of this report. Should you have any queries, please do not hesitate to contact the undersigned or [redacted]

For and on behalf of Coffey

[redacted]

[redacted]

Attachments

Appendix A – Results of Double Ring Infiltration Tests
Appendix B – Results of Inversed Auger Hole Testing
Appendix C – Results of Laboratory Testing

Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Important information about your **Coffey** Report

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way. Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

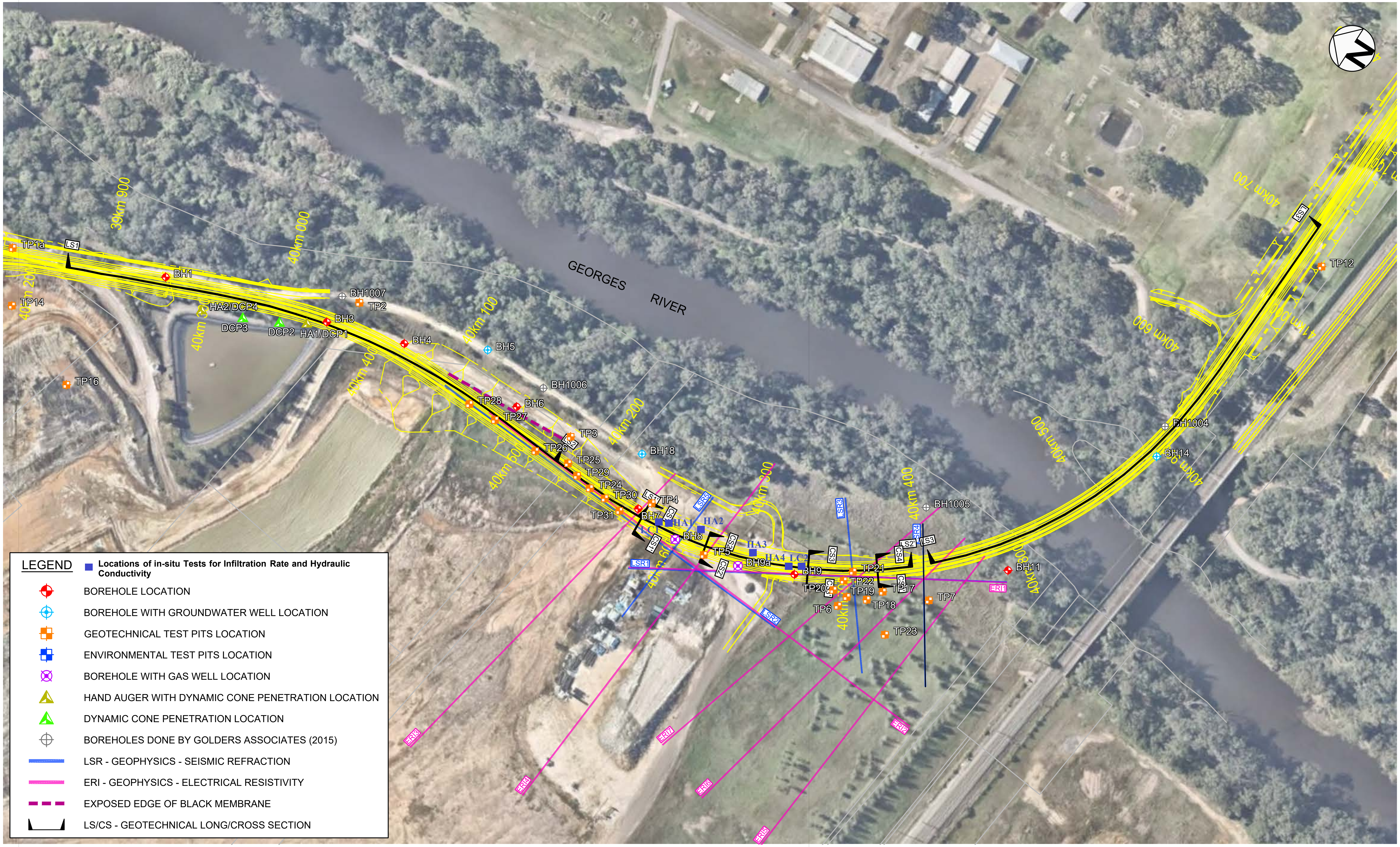
Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction,
speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.



LEGEND

- Locations of in-situ Tests for Infiltration Rate and Hydraulic Conductivity
- BOREHOLE LOCATION
- BOREHOLE WITH GROUNDWATER WELL LOCATION
- GEOTECHNICAL TEST PITS LOCATION
- ENVIRONMENTAL TEST PITS LOCATION
- BOREHOLE WITH GAS WELL LOCATION
- HAND AUGER WITH DYNAMIC CONE PENETRATION LOCATION
- DYNAMIC CONE PENETRATION LOCATION
- BOREHOLES DONE BY GOLDERS ASSOCIATES (2015)
- LSR - GEOPHYSICS - SEISMIC REFRACTION
- ERI - GEOPHYSICS - ELECTRICAL RESISTIVITY
- EXPOSED EDGE OF BLACK MEMBRANE
- LS/CS - GEOTECHNICAL LONG/CROSS SECTION

PLOT DATE: 27/06/2016 10:55:48 AM DWG FILE: F:\GEOTECHNIC\S1 PROJECTS\GEOTLCOV240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-AM.DWG

revision	no.	description	drawn	approved	date
	[redacted]				

20 0 20 40 60 80 100

Scale (metres) 1:2000

drawn	[redacted]
approved	[redacted]
date	27 / 06 / 16
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	GEOTECHNICAL INVESTIGATION LOCATION PLAN SHEET 2 OF 4
project no:	GEOTLCOV24072AF-AM
figure no:	FIGURE 1
rev:	C

Appendix A – Results of Double Ring Infiltration Testing

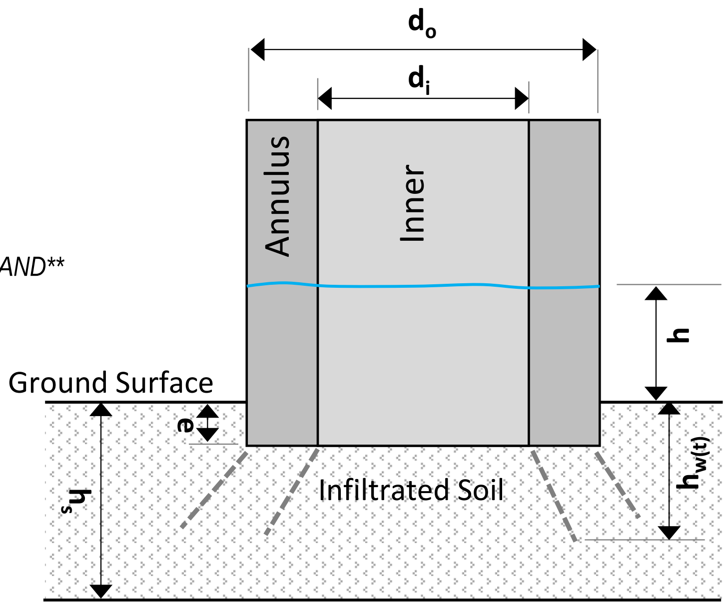
DOUBLE RING INFILTRATION TEST (ASTM D3385-03)

TEST ID = *LC1*
Location = *Ch 40,585 (near centreline)*
TEST Date = *23-Mar-17*

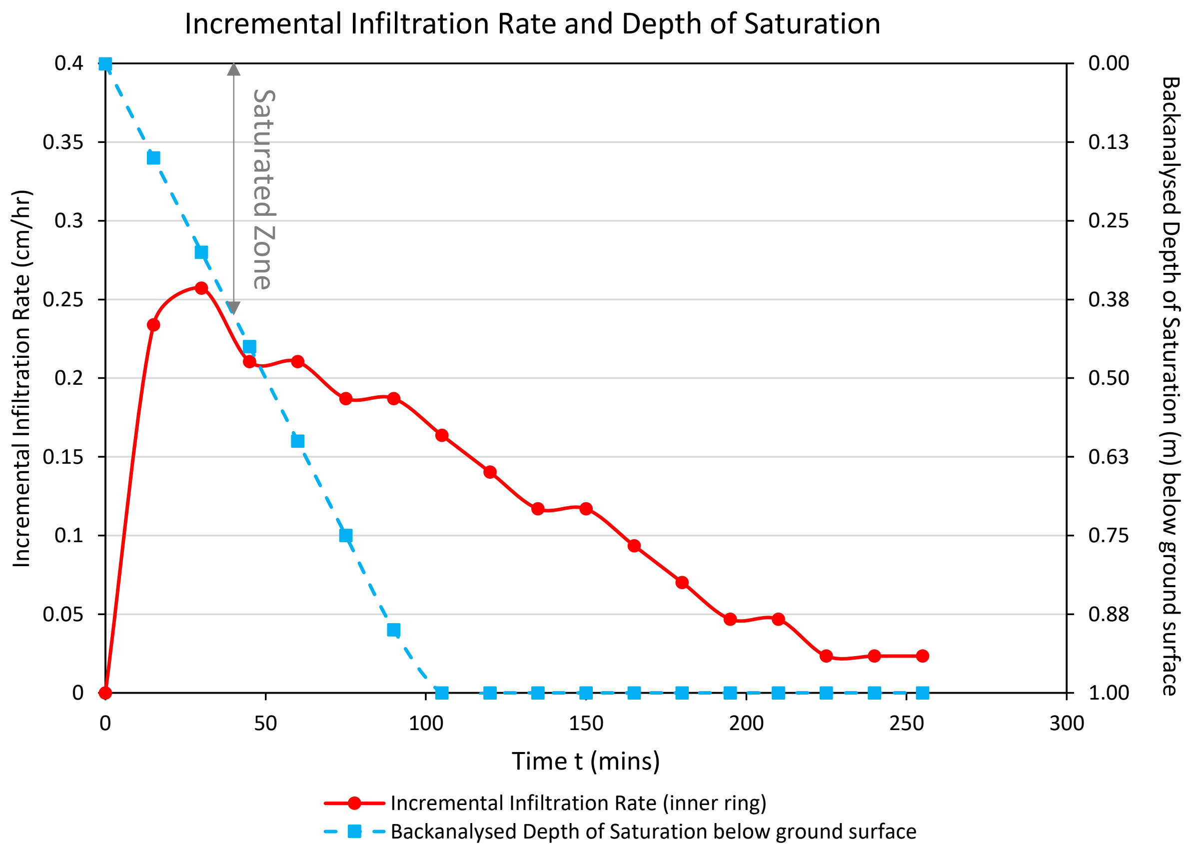
Inner diameter of inner ring, d_i (mm) = *330*
Inner diameter of outer ring, d_o (mm) = *650*
Depth of embedment into ground, e (mm) = *150*
Water height above Ground Surface, h (mm) = *290*
Thickness of soil being infiltrated, h_s (m) = *1**
Description of soil being infiltrated = *Gravelly SAND***
Water Temperature at the start of test (C) = *17*
Weather during the test = Cloudy

$h_{w(t)}$ = Depth of saturated soil at time t

** Based on nearby borehole (see Fig. 1)*
*** Based on near surface observation*



Hydraulic Conductivity (K) (m/sec) = *3.80E-07* (assumed based on Inversed Auger Hole Test)



Note: Depth of Saturation below the ground surface was backanalysed by matching the value of Hydraulic Conductivity adopted from laboratory testing or other in-situ testing

drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	CPB Contractors Pty Lrd		
approved			project:	Moorebank Intermodal Rail Link		
date	4/10/2017					
scale	NTS		title:	Double Ring Infiltration Test Result - LC1		
original size	A4		project no:	GEOTLCOV24072AF	figure no:	2

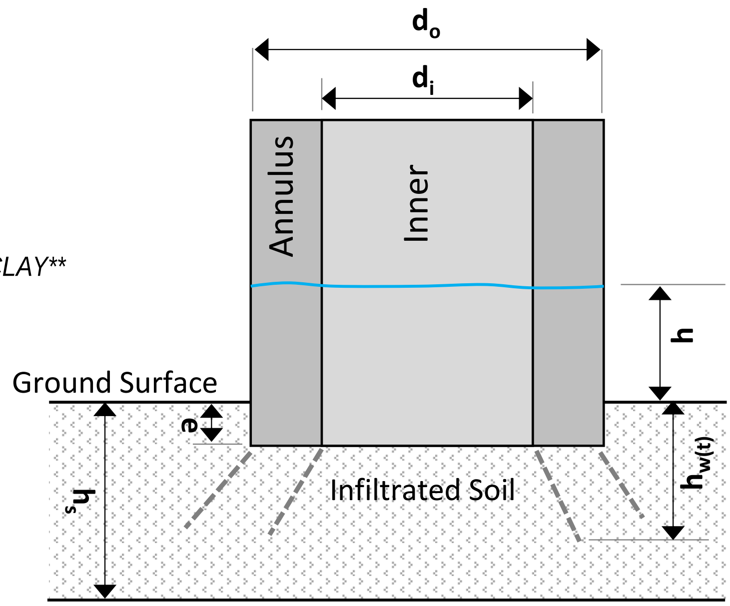
DOUBLE RING INFILTRATION TEST (ASTM D3385-03)

TEST ID = *LC2*
Location = *Ch 40,675 - MB2S (near centreline)*
TEST Date = *23-Mar-17*

Inner diameter of inner ring, d_i (mm) = *330*
Inner diameter of outer ring, d_o (mm) = *650*
Depth of embedment into ground, e (mm) = *150*
Water height above Ground Surface, h (mm) = *270*
Thickness of soil being infiltrated, h_s (m) = *2.5**
Description of soil being infiltrated = *Gravelly CLAY***
Water Temperature at the start of test (C) = *17*
Weather during the test = Cloudy

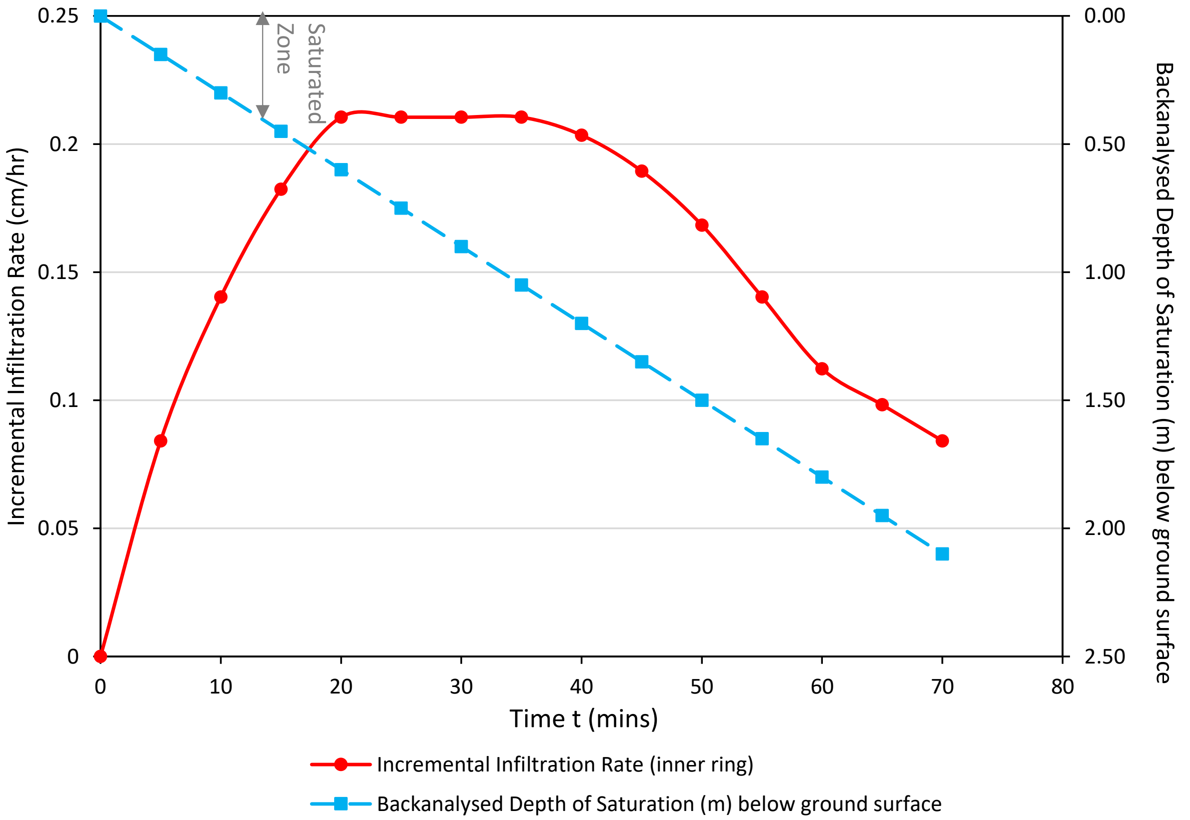
$h_{w(t)}$ = Depth of saturated soil at time t

** Based on nearby borehole (see Fig. 1)*
*** Based on near surface observation*



Hydraulic Conductivity (K) (m/sec) = *2.60E-07* (assumed based on Inversed Auger Hole Test)

Incremental Infiltration Rate and Depth of Saturation



Note: Depth of Saturation below the ground surface was backanalysed by matching the value of Hydraulic Conductivity adopted from laboratory testing or other in-situ testing

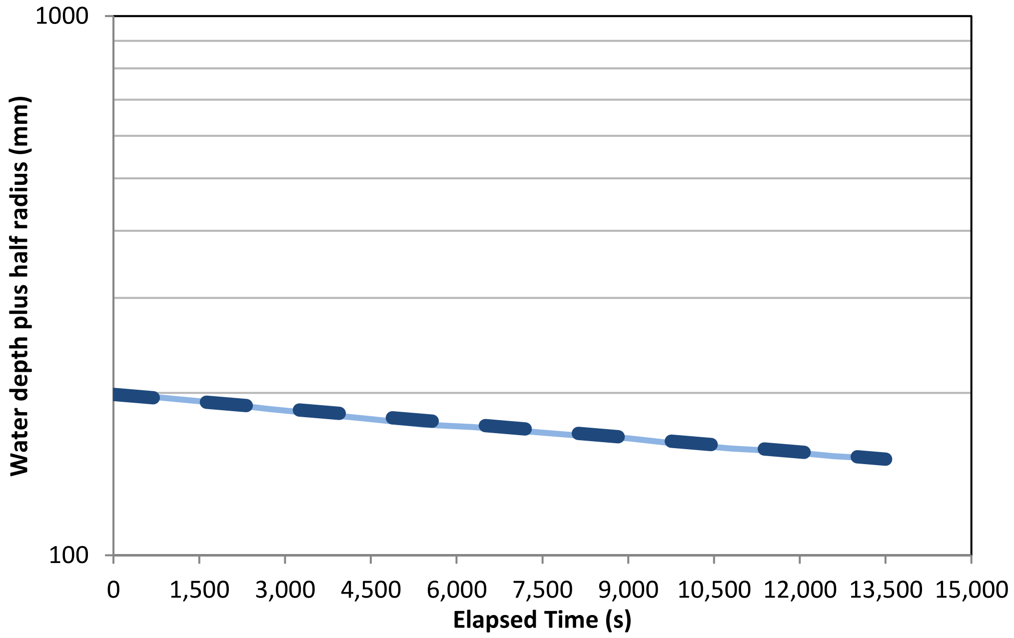
drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	**CPB Contractors Pty Ltd**	
approved			project:	**Moorebank Intermodal Rail Link**	
date	4/10/2017				
scale	NTS		title:	Double Ring Infiltration Test Result - LC2	
original size	A4		project no:	**GEOTLCOV24072AF**	figure no: **3**

Appendix B – Results of Inversed Auger Hole Testing

HOLE DIMENSIONS	
Diameter (mm)	75
Depth (mm)	180

INVERSED AUGER TEST

MEASUREMENTS	
Time (hr:min:secs)	Depth (mm)
0:00:00	0
0:01:00	0
0:03:00	0
0:07:00	1
0:15:00	3
0:30:00	7
0:45:00	12
1:00:00	16
1:15:00	20
1:30:00	24
1:45:00	26
2:00:00	29
2:15:00	32
2:30:00	34
2:45:00	38
3:00:00	41
3:15:00	43
3:30:00	46
3:45:00	48



Match interval	
Initial time	0:00:00
Final time	3:45:00

Intepreted Permeability
3.8E-07 m/s
3.3E-02 m/d

Note: Method described by: RJ Oosterbaan and HJ Nijland in *Determining the saturated hydraulic conductivity*

See chapter 12 of *Drainage Principals and Applications* ILRI Publication 16 2nd Edition 1994

drawn	[redacted]
approved	
date	10-Apr-2017
scale	AS SHOWN
original size	A4

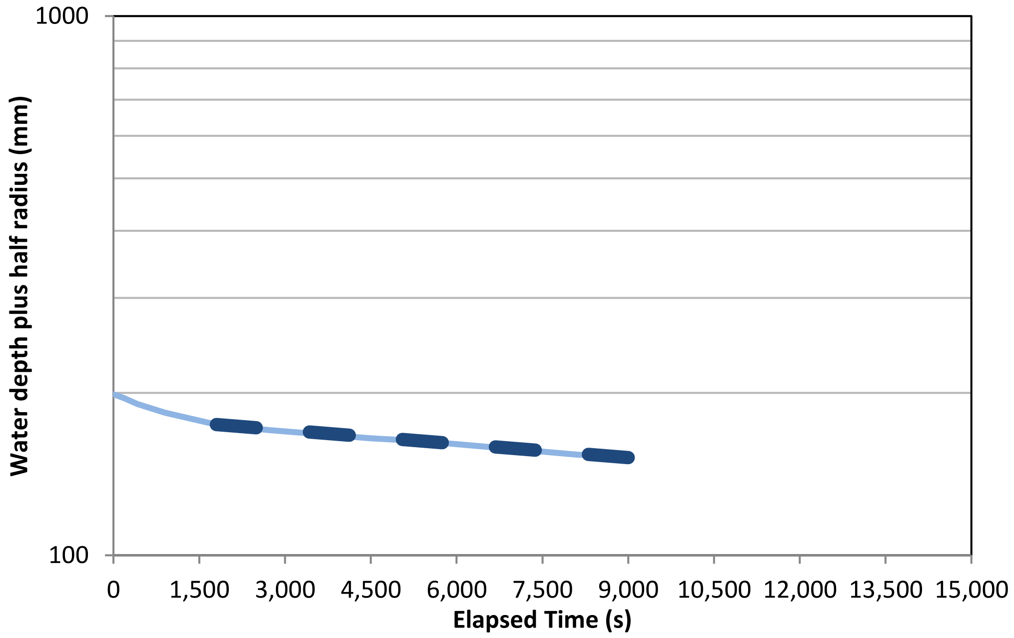
coffey
A TETRA TECH COMPANY

client:	**CPB Contractors Pty Ltd**
project:	**Moorebank Intermodal Rail Link** **Moorebank**
title:	Inversed Auger Testing - HA1
project no:	**GEOTLCOV24072AF**
figure no:	**4**

HOLE DIMENSIONS	
Diameter (mm)	75
Depth (mm)	180

INVERSED AUGER TEST

MEASUREMENTS	
Time (hr:min:secs)	Depth (mm)
0:00:00	0
0:01:00	1
0:03:00	3
0:07:00	8
0:15:00	15
0:30:00	24
0:45:00	28
1:00:00	31
1:15:00	34
1:30:00	36
1:45:00	39
2:00:00	42
2:15:00	45
2:30:00	47



Match interval	
Initial time	0:30:00
Final time	2:30:00

Intepreted Permeability
3.7E-07 m/s
3.2E-02 m/d

Note: Method described by: RJ Oosterbaan and HJ Nijland in *Determining the saturated hydraulic conductivity*

See chapter 12 of *Drainage Principals and Applications* ILRI Publication 16 2nd Edition 1994

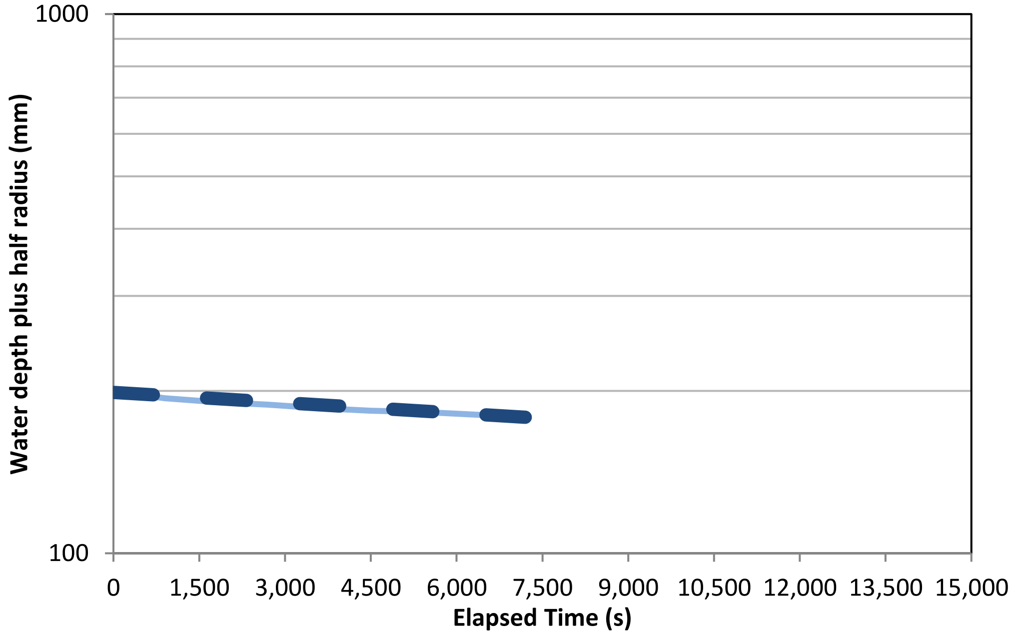
drawn	[redacted]	client:	**CPB Contractors Pty Ltd**
approved		project:	**Moorebank Intermodal Rail Link** **Moorebank**
date	10-Apr-2017	title:	Inversed Auger Testing - HA2
scale	AS SHOWN	project no:	**GEOTLCOV24072AF**
original size	A4	figure no:	**5**

coffey
A TETRA TECH COMPANY

HOLE DIMENSIONS	
Diameter (mm)	75
Depth (mm)	180

INVERSED AUGER TEST

MEASUREMENTS	
Time (hr:min:secs	Depth (mm)
0:00:00	0
0:01:00	0
0:03:00	1
0:07:00	2
0:15:00	5
0:30:00	8
0:45:00	10
1:00:00	13
1:15:00	15
1:30:00	16
1:45:00	18
2:00:00	20



Match interval	
Initial time	0:00:00
Final time	2:00:00

Intepreted Permeability
2.8E-07 m/s
2.4E-02 m/d

Note: Method described by: RJ Oosterbaan and HJ Nijland in *Determining the saturated hydraulic conductivity*

See chapter 12 of *Drainage Principals and Applications* ILRI Publication 16 2nd Edition 1994

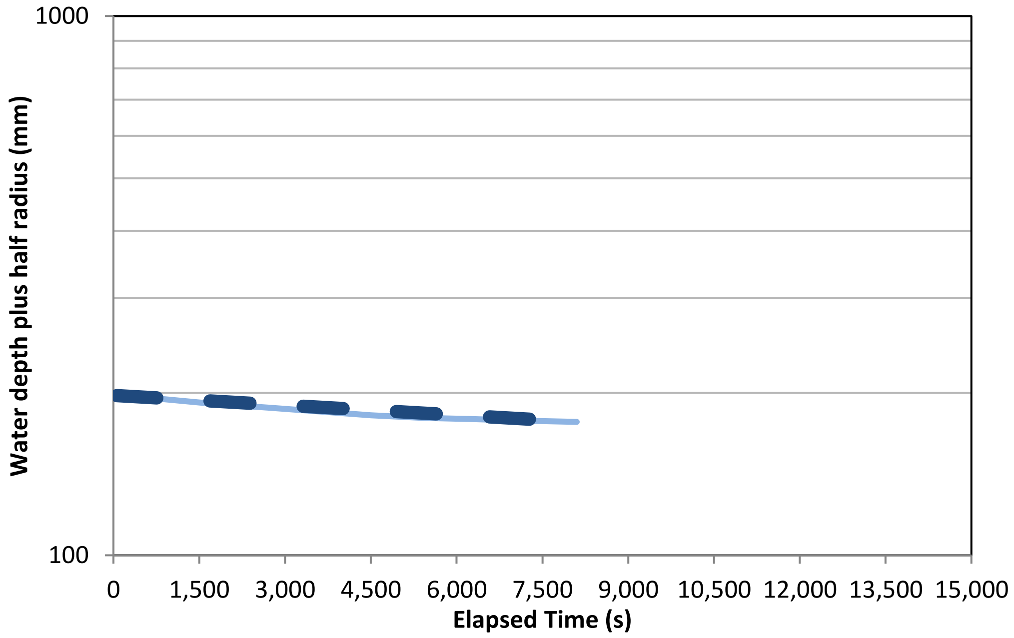
drawn	[redacted]	client: **CPB Contractors Pty Ltd**
approved		project: **Moorebank Intermodal Rail Link** **Moorebank**
date	10-Apr-2017	
scale	AS SHOWN	title: Inversed Auger Testing - HA3
original size	A4	project no: **GEOTLCOV24072AF** figure no: **6**

coffey
A TETRA TECH COMPANY

HOLE DIMENSIONS	
Diameter (mm)	75
Depth (mm)	180

INVERSED AUGER TEST

MEASUREMENTS	
Time (hr:min:secs)	Depth (mm)
0:00:00	0
0:01:00	1
0:03:00	1
0:07:00	2
0:15:00	4
0:30:00	8
0:45:00	11
1:00:00	14
1:15:00	17
1:30:00	19
1:45:00	20
2:00:00	21
2:15:00	22



Match interval	
Initial time	0:01:00
Final time	2:15:00

Intepreted Permeability
2.6E-07 m/s
2.3E-02 m/d

Note: Method described by: RJ Oosterbaan and HJ Nijland in *Determining the saturated hydraulic conductivity*

See chapter 12 of *Drainage Principals and Applications* ILRI Publication 16 2nd Edition 1994

drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	**CPB Contractors Pty Ltd**
approved			project:	**Moorebank Intermodal Rail Link** **Moorebank**
date	10-Apr-2017			
scale	AS SHOWN		title:	Inversed Auger Testing - HA4
original size	A4		project no: **GEOTLCOV24072AF**	figure no: **7**

Appendix C – Results of Laboratory Testing

Sydney Laboratory

Coffey Services Australia Pty Ltd
31 Hope Street
Melrose Park NSW 2114

ABN 55 139 460 521
Phone: +61 (2) 9352 5000

Report No: DDR:SYDN17W00961
Issue No: 1

Dry Density Ratio Report

Client:	Coffey Services Australia Pty Ltd (Chatswood) Level 19, 799 Pacific Highway Chatswood NSW 2067
Principal:	
Project No.:	754-SYDN00058AA
Project Name:	754-GEOTLCOV24072AF - 754 EARLY SERVICES- MOOREBANK
Lot No.:	**TRN:**

Accredited for compliance with ISO/IEC 17025 - Testing.

The results of the tests, calibrations and/or [redacted]

(Senior Geotechnician)
NATA Accredited Laboratory Number:431
Date of Issue: 6/04/2017

Sample Details

Location:	Glenfield Tip
Client Request ID:	
Specification Requirements:	
Field Test Procedures:	AS 1289.5.8.1
Laboratory Test Procedures:	AS 1289.5.1.1, AS 1289.2.1.1, AS 1289.5.4.1
Sampling Method:	AS1289.1.2.1 Clause 6.4 (b)
Source:	Ex. Site
Material:	Fill

Sample Data

Sample ID	SYDN17S-02181	SYDN17S-02182
Field Sample ID	00001	00002
Date Tested	26/03/2017	26/03/2017
Time Tested	09:00	09:20
Location	L.C.1	L.C.2
Easting	0307064.8	0307088.7
Northing	6239995.9	6239913.4
RL	13.49	16.20
Soil Description	Gravelly SAND	Gravelly CLAY
Field and Laboratory Data		
Depth of Test (mm)	300	300
Depth of Layer (mm)	300	300
Compactive Effort	Standard	Standard
AS Sieve Size (mm)	19.0	19.0
Oversize Wet (%)	0	0
Oversize Dry (%)	0	0
Field Moisture Content (%)	8.8	15.6
Field Wet Density (t/m³)	2.28	2.08
Field Dry Density (t/m³)	2.09	1.80
Lab Result from Test No.	SYDN17S-02181	SYDN17S-02182
Maximum Dry Density* (t/m³)	2.09	1.81
Optimum Moisture Content* (%)	10.0	14.5
Moisture Ratio (%)	87.0	108.5
Moisture Variation	1.5 dry	1.0 wet
Density Ratio (%)	**100.0**	**99.5**
legend * adjusted for oversize material		

Comments

Artarmon, Sydney Laboratory

Coffey Services Australia Pty Ltd
ABN 55 139 460 521
31 Hope Street
Melrose Park NSW 2114

Phone: +61 (2) 9352 5000

A TETRA TECH COMPANY

Maximum Dry Density Report

Report No: MDD:ARTA17S-00207
Issue No: 1

Client:	Coffey Services Australia Pty Ltd (Chatswood) Level 19, 799 Pacific Highway Chatswood NSW 2067
Principal:	CPB Contractors
Project No.:	754-ARTA00035AF
Project Name:	754-GEOTLCOV24072AF - 754 Sydney Lab External Proj
Lot No.:	**TRN:**

Accredited for compliance with ISO/IEC 17025 - Testing.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.

[redacted]

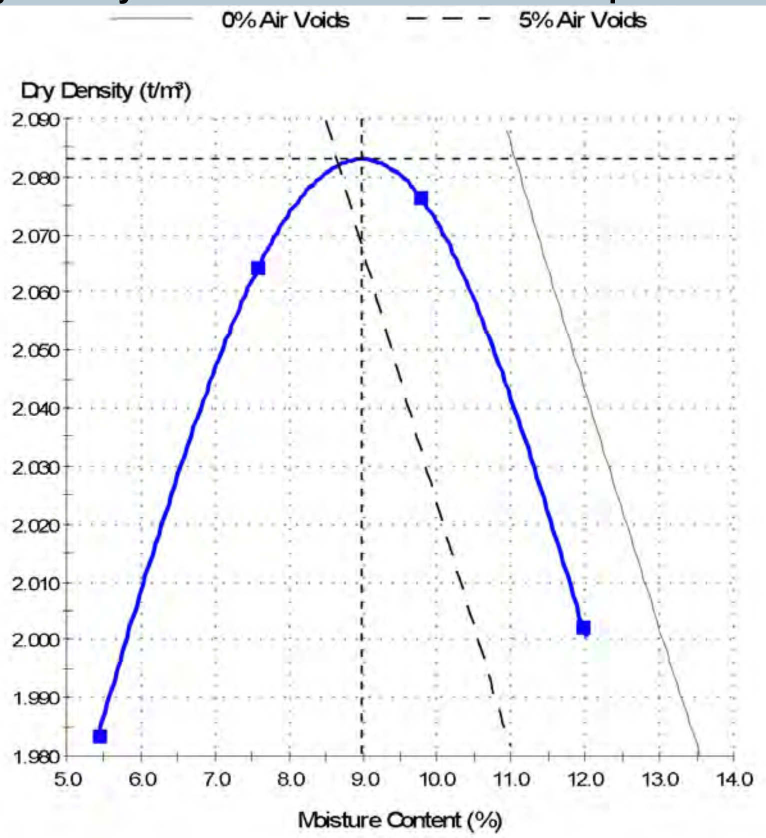
(Specialised Testing Manager)
NATA Accredited Laboratory Number:431
Date of Issue: 7/04/2017

WORLD RECOGNISED ACCREDITATION

Sample Details

Sample ID:	ARTA17S-00207	**Sampling Method:**	Submitted by client
Date Sampled:	23/03/2017	**Material:**	Subgrade
Date Submitted:		**Source:**	Ex Job Site
Date Tested:	3/04/2017	**Specification:**	No Specification
Project Location:	Moorebank, NSW		
Sample Location:	LC1 (0.05 to 0.30 m)		

Dry Density - Moisture Content Relationship



Test Results

AS 1289.5.1.1

Standard MDD (t/m³): **2.08**

Standard OMC (%): **9.0**

Retained Sieve 19.0mm (%): 0

Comments

Coffey Corporate Services Pty Ltd
31 Hope Street, Melrose Park, NSW 2114
ph: (61 2) 9352 5000
ABN: 92 114 364 046

test results - falling head permeability report

client:	*Coffey Services Australia Pty Ltd (GEOTLCOV24072AF)*	job no:	*754-ARTA00034AA*
principal:	*CPB Contractors*	laboratory:	*Melrose Park*
project:	*Moorebank Intermodal Rail Connection*	report date:	*7th April, 2017*
location:	*Morrebank, NSW*	test report:	*IOLT 9730*
test procedure:	*AS 1289.6.7.2*	test date:	*03/04/17 to 07/04/17*

Sample Identification	REMOULDED DRY DENSITY t/m^3	REMOULDED MOISTURE CONTENT (%)	REMOULDED FALLING HEAD PERMEABILITY cm/sec	REMOULDED FALLING HEAD PERMEABILITY m/sec
LC1 **(0.05 to 0.30 m)** Artarmon Sample Number: ARTA17S-00207	2.08	9.0	6.6×10^{-7}	6.6×10^{-9}

Notes:
1. *Specimen recompacted to 100% of Standard Maximum Dry Density and at Standard Optimum Moisture Content.*
2. *Specimen tested with Distilled Water.*
3. *0.0 % material retained on the 19mm sieve*
4. *0 kPa pressure was applied to the specimen.*
5. *Sample received from Client*

Remarks:

F:\2. TECHNICAL\INFO-TESTING\01. Laboratory_754-QEXT-SYDN-FY17\754-ARTA00035AF - Moorebank, NSW\[LC1_0.05-0.30_FHP.xls]Sheet1

NATA Accredited Laboratory

No 431

Date: 7th April, 2017

[redacted]

Approved Signatory: [redacted]

Artarmon, Sydney Laboratory

Coffey Services Australia Pty Ltd
ABN 55 139 460 521
31 Hope Street
Melrose Park NSW 2114

Phone: +61 (2) 9352 5000

coffey
A TETRA TECH COMPANY

Report No: MDD:ARTA17S-00208
Issue No: 1

Maximum Dry Density Report

Client:	Coffey Services Australia Pty Ltd (Chatswood) Level 19, 799 Pacific Highway Chatswood NSW 2067
Principal:	CPB Contractors
Project No.:	754-ARTA00035AF
Project Name:	754-GEOTLCOV24072AF - 754 Sydney Lab External Proj
Lot No.:	**TRN:**

NATA
WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.

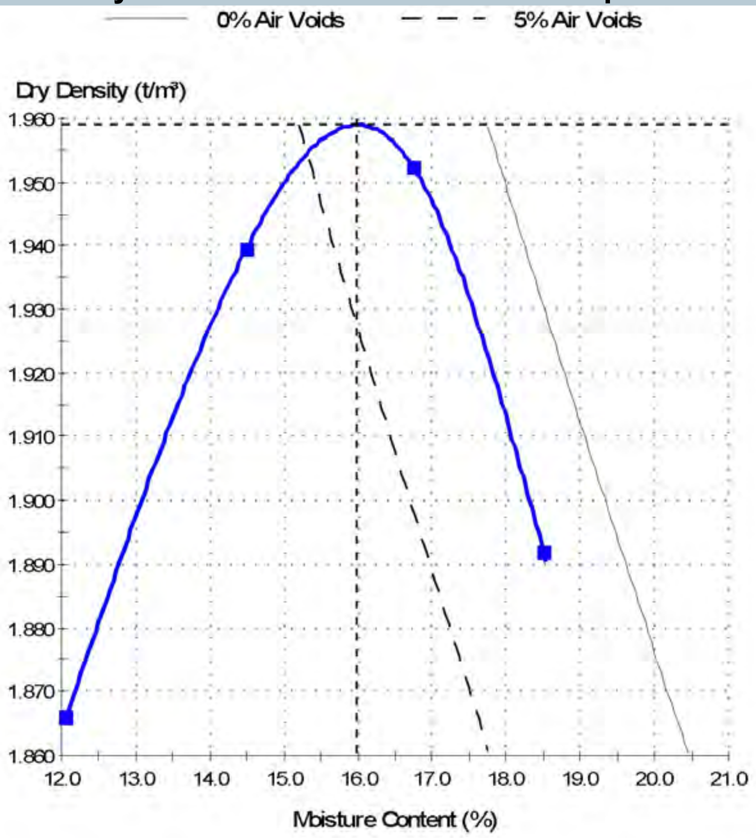
[redacted]

(Specialised Testing Manager)
NATA Accredited Laboratory Number:431
Date of Issue: 7/04/2017

Sample Details

Sample ID:	ARTA17S-00208	**Sampling Method:**	Submitted by client
Date Sampled:	23/03/2017	**Material:**	Subgrade
Date Submitted:		**Source:**	Ex Job Site
Date Tested:	3/04/2017	**Specification:**	No Specification
Project Location:	Moorebank, NSW		
Sample Location:	LC2 (0.05 to 0.30 m)		

Dry Density - Moisture Content Relationship



Test Results

AS 1289.5.1.1

Standard MDD (t/m³):	**1.96**
Standard OMC (%):	**16.0**
Retained Sieve 19.0mm (%):	0

Comments

Coffey Corporate Services Pty Ltd
31 Hope Street, Melrose Park, NSW 2114
ph: (61 2) 9352 5000
ABN: 92 114 364 046

test results - falling head permeability report

client:	*Coffey Services Australia Pty Ltd (GEOTLCOV24072AF)*	job no:	*754-ARTA00034AA*
principal:	*CPB Contractors*	laboratory:	*Melrose Park*
project:	*Moorebank Intermodal Rail Connection*	report date:	*7th April, 2017*
location:	*Morrebank, NSW*	test report:	*IOLT 9731*
test procedure:	*AS 1289.6.7.2*	test date:	*03/04/17 to 07/04/17*

Sample Identification	REMOULDED DRY DENSITY t/m^3	REMOULDED MOISTURE CONTENT (%)	REMOULDED FALLING HEAD PERMEABILITY cm/sec	REMOULDED FALLING HEAD PERMEABILITY m/sec
LC2* *(0.05 to 0.30 m) *Artarmon Sample Number: ARTA17S-00208*	*1.96*	*16.0*	4.5×10^{-8}	4.5×10^{-10}

Notes:
1. *Specimen recompacted to 100% of Standard Maximum Dry Density and at Standard Optimum Moisture Content.*
2. *Specimen tested with Distilled Water.*
3. *0.0 % material retained on the 19mm sieve*
4. *0 kPa pressure was applied to the specimen.*
5. *Sample received from Client*

Remarks:

F:\2. TECHNICAL\INFO-TESTING\01. Laboratory_754-QEXT-SYDN-FY17\754-ARTA00035AF - Moorebank, NSW\[LC2_0.05-0.30_FHP.xls]Sheet1

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025
The results of the tests, calibrations, and/or measurements included in this document are traceable to Australian/national standards.

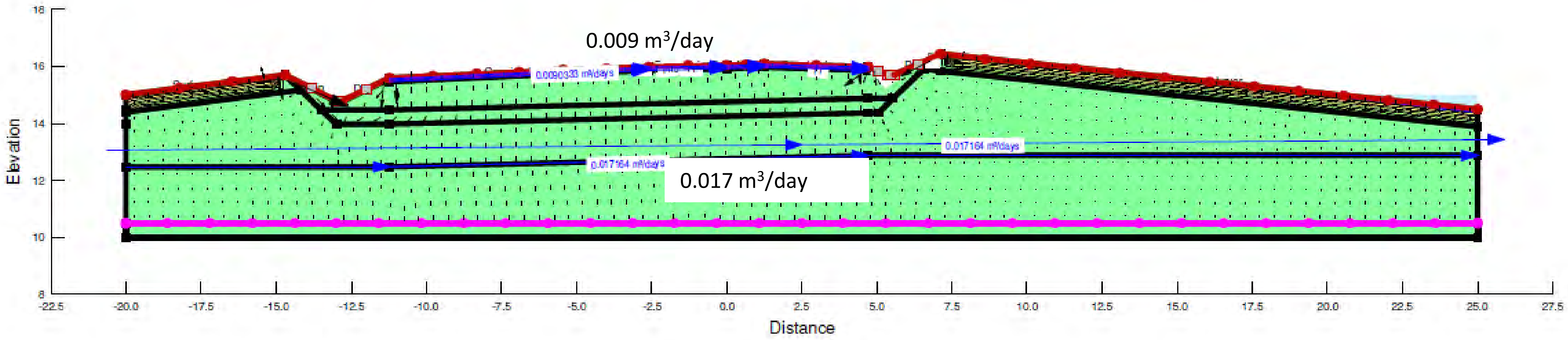
NATA Accredited Laboratory

No 431

Date: 7th April, 2017

Approved Signatory: [redacted]

Attachment 2: SEEP/W output plot for flow through the formation



drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	CPB Contractors Pty Ltd
approved			project:	**Moorebank Intermodal Rail Link** **Infiltration assessment**
date	11/04/2017			
scale	NTS		title:	SEEP/W output for flow through the rail formation
original size	A4		project no: **GEOTLCOV24072AF**	figure no: **A2-1**

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Electronic Transmission

To	[redacted]	From	[redacted]
Email address	[redacted]	Date	27 March 2018
Company	CPB Contractors Pty Ltd	Reference	GEOTLCOV24072AG-AG
cc	[redacted]	Pages	1 of 3
Subject	Moorebank Intermodal Rail Link – Dynamic compaction impact on clay liner		

[redacted]

1. Introduction

CPB Contractors (Pty Ltd) has been commissioned by Sydney Intermodal Terminal Alliance (SIMTA) to undertake the design and construction of the Moorebank Intermodal Rail Link (MIRL) as part of the Moorebank Intermodal Terminal Development. The project involves land preparation and construction of the proposed rail line from the existing South Sydney Freight Line (SSFL) to the proposed SIMTA Intermodal Facility over an approximate length of 2.4 km from the northern connection up to the proposed terminal.

Design of the Ground Improvement has been presented in the detailed design Report Ref: N01031-GRW-DRP-GEO-0001-02 dated 19 May 2017. The design report provides the geotechnical ground treatment design for a section of embankment over the transition and landfill area from Ch 40,427 (MB2S) to Ch 40,740 (MB2S).

This memorandum explains further the design intent of the Dynamic Compaction (DC) between Ch 40,560 to Ch 40,740 (MB2S) and clarifies the approach to achieve and validate the design intent.

Please note that Dynamic Compaction will not be used over the areas with existing geosynthetic liner.

2. DC Design intent

The design intent of the Dynamic Compaction for the MB2S (old landfill area) is as follows:

- To densify the waste materials to reduce short and long-term settlement of the proposed rail embankment.
- To conduct the densification in a manner that there will be no permanent deformation at the existing waste liner level.

The above design intent will be achieved by limiting the depth of treatment of the waste fill beneath the embankment, and also limit the impact energy of individual blows (i.e. via larger pounder and lower drop height) where the embankment is situated over the batter (or wall) of the landfill. The methodology adopted in the design and construction to validate the above is described in detail below.

3. Dynamic compaction of municipal waste

Soil densification by dynamic compaction (DC), also known as dynamic deep compaction is a well-known compaction method. The method is successfully used for more than 30 years worldwide for compaction of landfills and municipal waste sites not only to improve the waste properties but also to increase the available cell volume (refer to Table 1).

Table 1 – Worldwide dynamic compaction data of municipal waste (after Van Impe and Bouazza, 1996)

Site[a]	Area (m^2)	Depth[b] (m)	Age (years)	*W* (t)	*H* (m)	*A* (m^2)	*E* (t·m/m^2)	Settlement (m)
East London ,U.K.[1]	—	6.5	40	14	14	4	260	0.58
Redditch, U.K.[1]	22 000	5–6	15	15	20	4	260	0.50
Hertfordshire, U.K.[1]	30 000	8	15–20	15	20	4	260	0.50
Cwmbran, U.K.[2]	12 500	5–10	9	15	17	—	120–250	0.35–1
Coutances, France[3]	—	7–10	<15	—	—	—	500	1.75
Arkansas, U.S.A.[4]	18 000	6–12	5	18	28	—	876	1.6–2.5
Chicago, U.S.A.[5]	—	6	30	6	11		130–260	0.3–0.46
Skokie, U.S.A.[6]	9 500	6–15	14	15	18	1.5	560	1.1–1.2
Haifa, Israel[7]	14 000	25	20	11	15	4	—	—
Bell Lane, U.K.[8]	13 500	4–5	6–10	17	15	3.5	281	0.7–1
Poyle, U.K.[9]	—	6	<10	15	20	—	150	0.6
Tulsa, U.S.A.[10]	90 000	6	—	18	23	3.7	312	0.66
Arkansas, U.S.A.[11]	—	5–12	3	18.2	28	—	852	1–3
Indiana, U.S.A.[11]	—	7.5–90	13	18.2	30	—	666	0.75–0.9
New York, U.S.A.[12]	—	6–7	—	15	20	—	167	0.6
North-Shield B, U.K.[13]	—	5	15–25	8	16	—	50	0.46
North-Shield A, U.K.[13]	—	5	15–25	15	10	—	92	0.46
Maldegem, Belgium[14]	30 000	4–8	20	13	10	—	15–55	0.4–0.5

Note: W, weight of pounder; H, height of fall; A, area of pounder.
[a]Sources: 1, Charles et al (1981); 2, Downie and Treharne (1979); 3, Gambin (1981); 4, Welsh (1983); 5, Lukas (1985); 6, Stienberg and Lukas (1984); 7, Frydman and Baker (1987); 8, Perelberg et al. (1987); 9, Sherwood (1987); 10, Snethen and Homan (1991); 11, Lukas (1992*b*); 12, Gifford et al. (1992); 13, Mapplebeck and Fraser (1993); 14, Van Impe (1994).
[b] Denotes the depth of the landfill

The municipal waste consists of different materials where a large part of the constituents has a high void ratio compared to naturally occurring soils. The waste during the DC improvement is compacted by repeated, systematic application of high energy using a drop of a heavy weight (pounder) on the landfill surface. The imparted energy is transmitted from the ground surface to the deeper waste which forces the waste particles into a denser state by inducing settlement and deformation. The settlements are larger at the waste surface gradually decreasing with the depth to the point where no settlements and no deformation occur.

The depth in which DC does not affect waste densification and settlement is called the depth of treatment (DoT) and can be determined using empirical the following relationship

$$DoT = n\sqrt{W\,H}$$

Where, *n* is the empirical value in the range of 0.35 to 0.5 for old landfills, and *W* and *H* are the drop weight (tonne) and drop height of pounder (m), respectively.

The design intent is to terminate the DoT well above the liner and leave the untreated bottom waste layer in the uncompressed and unaffected state so that there is no permanent deformation at the existing waste liner level.

As the base of the GWS landfill has been established by survey and geophysical assessment the design drop height of the pounder was chosen not to affect the last 3-4m above the base of the fill.

Series of tests can be conducted during the trials prior to the main works which gradually increase the height drop while measuring the lateral soil movement with depth to establish the base of the insignificant permanent deformation (e.g. DoT), thus validating the design "n" value or enabling adjustment in accordance with the design intent.

A lighter "ironing" pounder with a larger surface and smaller drop heights will be used in the wall liner areas to reduce the energy impact for each drop and more evenly distribute and dissipate the impact energy to allow the design DoT to be achieved. The number of drops would increase thus the total energy introduced to the layer will remain as designed.

In order to ensure an effective and well spread transfer of the applied energy a stiffer imported construction platform will cover the existing GWS landfill ground surface so that the localised deep settlements below the designed depth of treatment are avoided.

As the depth of the bottom liner on the majority of the site is well beyond the practically recommended DoT of 10 m for the DC method and the liner is placed on competent strata we believe that design intent of 10 m treatment for the full depth landfill and 2 to 9m at the landfill wall will be realised.

Dynamic compaction is used for more than 30 years for the improvement of the municipal waste. The close cooperation of the designer with experienced DC specialist contractor during the DC trials and timely reporting the site observation and monitoring will allow the compaction of the waste materials while achieving the design intent.

We trust this information satisfies your present requirements. Please contact [redacted] or the undersigned should you require any clarifications.

For and on behalf of Coffey

[redacted]

[redacted]

Level 19, Tower B, 799 Pacific Highway
Chatswood
NSW 2067 Australia

t:
f:

coffey.com

Memorandum

Recipient name	[redacted]	**Recipient company**	CPB Contractors Pty Ltd
Copied recipients	[redacted]	**Memo date**	27/03/2018
Author	[redacted]		
Project number	GEOTLCOV24072AG-AH		
Memo Subject	**Moorebank Intermodal Rail Link - Analysis to assess GCL liner performance and recommendations on appropriate liner system**		

[redacted]

1. Introduction

Coffey has previously carried out geotechnical ground treatment design for a section of embankment over the transition and landfill area from Ch 40,427 (MB2S) to Ch 40,740 (MB2S) of the Moorebank Intermodal Rail Link (MIRL) as part of the Moorebank Intermodal Terminal Development. A section of the embankment between Ch 40,445 (MB2S) and Ch 40,740 (MB2S) is located over landfill area. Issue for construction (IFC) Drawings and Report (Ref: N01031-GRW-DRP-GEO-0001-03 dated 28 September 2017) have been submitted.

The above design recommended that infiltration performance of the rail embankment is well within the performance stipulated in EPA guideline with no additional sealing or capping layers required.

However, this approach has not been accepted by SIMTA/GWS/EPA, and GWS have requested a GCL liner system solution placed beneath the proposed embankment to limit infiltration from the embankment into the existing landfill. EPA Guidelines (2016) stipulates that a clay liner having a minimum thickness of 600 mm and a saturated permeability less than 10^{-9} m/sec is required. EPA also permits the use of a geosynthetic clay liner (GCL) provided it is used in composite with a geomembrane. In accordance with the GWS request for a GCL liner Coffey has carried out analysis and provide recommendations on an appropriate liner system and its performance under the high embankment/surcharge loading and subsequent differential settlement of the rail tracks due to settlement of land fill caused by on-going creep settlement and future placement of landfill against the rail embankment

To facilitate the commissioning of the Moorebank RALP Nº1 Rail Line in accordance with SIMTA time lines we have incorporated following design changes into the redesign of the preload/surcharge ground treatment for the revised transition zone construction and into the current settlement and differential settlement impact assessment:

- The time for preloading/surcharging has been reduced from 6 months to 3 months;
- The embankment width on the tip side has been increased by 5.25 m to accommodate an access track;
- The tip will be filled in future to the level of the proposed railway embankment in accordance with the model provided by Aurecon (Ref GWS 12d Rev E).

1.1. Proposed Liner System

EPA guideline stipulate double protection incorporating an HDPE liner and GCL. That is, the liner system (from bottom up) should comprise:

- A bearing layer which could be the existing capping layer after stripping, smoothing and compaction. However, maximum particle size should be less than 13.2 mm with more than 90% passing 2.4 mm and more than 30% passing 75 microns. If the existing subgrade does not meet this requirement, an imported bedding clay layer satisfying the above having a compacted thickness of 300 mm should be provided.
- A GCL (either BENTOMAT® DN or equivalent)
- Textured HDPE geomembrane (minimum 2 mm thickness)
- A cushion geotextile layer (Bidim A24 or equivalent) as a protective layer
- First layer of the embankment with 300 mm compacted thickness having a maximum particle size of 13.2 mm and 90% passing 2.4 mm.

2. Assessment Methodology

We have carried out a Plaxis 2D finite element analysis to assess the performance of the selected GCL linear and to assess the impact of adjacent filling on the proposed railway. We have carried out the analysis at Ch 40+540, which we consider is a critical section for the analysis due to the thick landfill layer and high embankment fill including surcharge.

The analysis was carried out considering Soft Soil (SS) and Soft Soil Creep (SSC) models in-built in Plaxis for landfill. SSC model was used to assess the future creep settlement. The analysis considered the nominal age of the landfill to be 10 years (for calibration of the numerical model with the design calculations). Updated mesh analysis was considered in order to account for the anticipated large deformations.

The liner system was assumed to be BENTOMAT® DN or equivalent with textured HDPE geomembrane. The GCL properties are provided in the datasheet shown in Attachment A. This GCL has a grab tensile strength of 8.8 kN/m. The tensile strength of geosynthetic reinforcement is typically reached at 10% strain (based on product datasheets), and we expect that filter fabric layers and GCL to have a strain at failure of greater than 10%. The HDPE geomembrane has larger elongation capacity compared to GCL liner, and therefore, we have conservatively modelled the liner system as a single geotextile element with EA = 88 kN/m for our analysis.

We note that the manufacturer of BENTOMAT® DN states that this GCL has high shear resistance and is suitable for installation on slopes greater than 1V:1.5H. Although the existing ground slope is much shallower, we recommend that a similar GCL be adopted due to relatively high short-term settlement under the preload and long-term creep settlement of the landfill.

The analysis considered embankment construction to occur in one month period to the surcharge level. Two months preload waiting period was allowed before the surcharge is trimmed back to final proposed design level. The filling adjacent to the proposed embankment was considered to occur immediately after the surcharge removal. The typical section considered for analysis is shown in Figure 1 below.

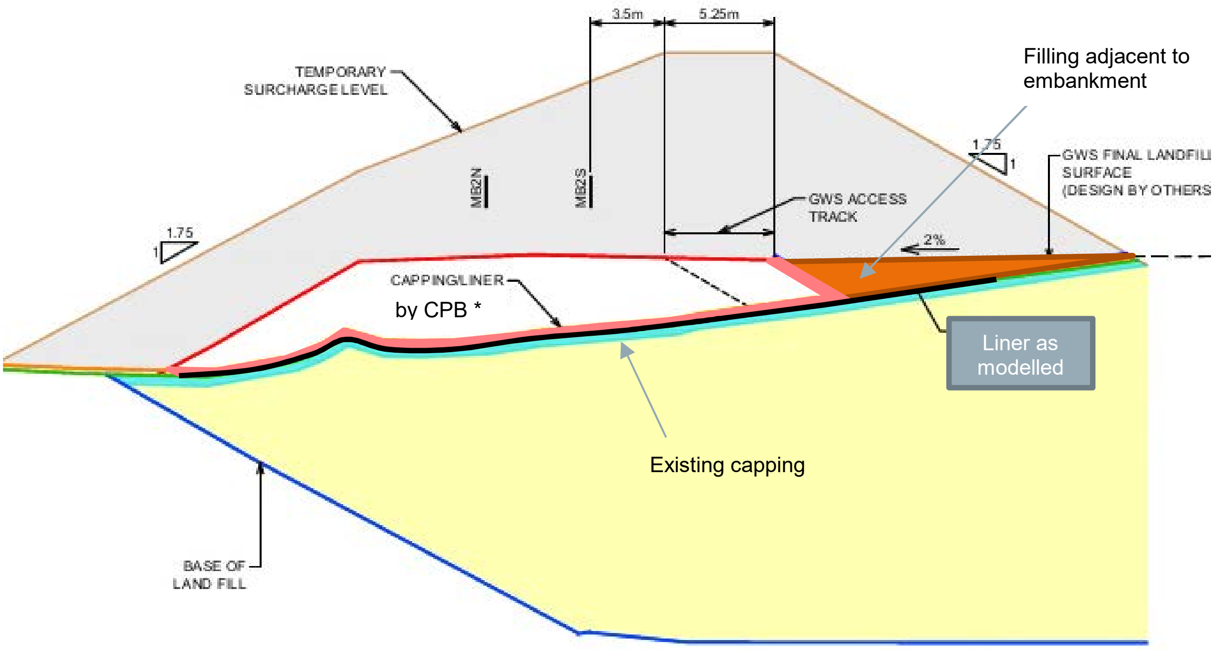


Figure 1: Typical section considered for the Analysis
Note: * final GCL layout to be agreed

3. Results and Recommendations

3.1. Performance of GCL

Maximum tensile forces along the GCL as assessed using Plaxis 2D is presented in Attachment B, along with assessed strain based on the adopted properties for GCL. As can be seen in the plots in Attachment B, the assessed maximum load and strain in the GCL is approximately 1.2 kN/m and 1.4 %, respectively. The computed maximum tensile load and strain are much lower than the quoted strength of the GLC and the expected strain at failure (i.e. > 10%). Furthermore, the computed maximum tensile strain is much lower than the usual limit of 5% to 6% strain recommended under operating conditions for geosynthetics. Therefore, we consider the proposed GCL liner using BENTOMAT® DN or equivalent to be suitable for the purpose.

Should you have any queries in relation to this memorandum, please contact the undersigned on [redacted]

For and on behalf of Coffey

[redacted]

[redacted]

Important Information about Your Coffey Report

Attachments:

Attachment A: Data sheet for BENTOMAT® DN

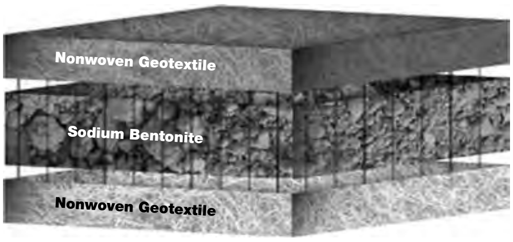
Attachment B: Assessed tensile forces and strain on GCL

Attachment A: Data sheet for BENTOMAT® DN

BENTOMAT® DN

TECHNICAL **DATA**

BENTOMAT® DN is commonly used in some of the most demanding applications including landfills where slopes are as steep as 1.5H:1V. This GCL is reinforced and consists of a layer of sodium bentonite between two heavier weight non-woven geotextiles, making this GCL suitable for applications requiring high internal and interface shear strength.



BENTOMAT® DN certified properties

Material Property	ASTM Test Method	Test Frequency	Required Values
Bentonite Swell Index[1]	D 5890	1 per 50 tonnes	24mL/2g min.
Bentonite Fluid Loss[1]	D 5891	1 per 50 tonnes	18mL max.
Bentonite Mass/Area[2]	D 5993	4,000m²	3.6kg/m² min.
GCL Tensile Strength[3]	D 6768	20,000m²	88N/cm MARV
GCL Peel Strength[4]	D 6496	4,000m²	500N/m min.
GCL Index Flux[4]	D 5887	Weekly	1×10^{-8} m³/m²/sec max.
GCL Hydraulic Conductivity[4]	D 5887	Weekly	5×10^{-9} cm/sec max.
GCL Hydrated Internal Shear Strength[5]	D 5321 D 6243	Periodic	24kPa typ @ 976kg/m²

Bentomat DN is a reinforced GCL consisting of a layer of granular sodium bentonite between two nonwoven geotextiles, which are needlepunched together.

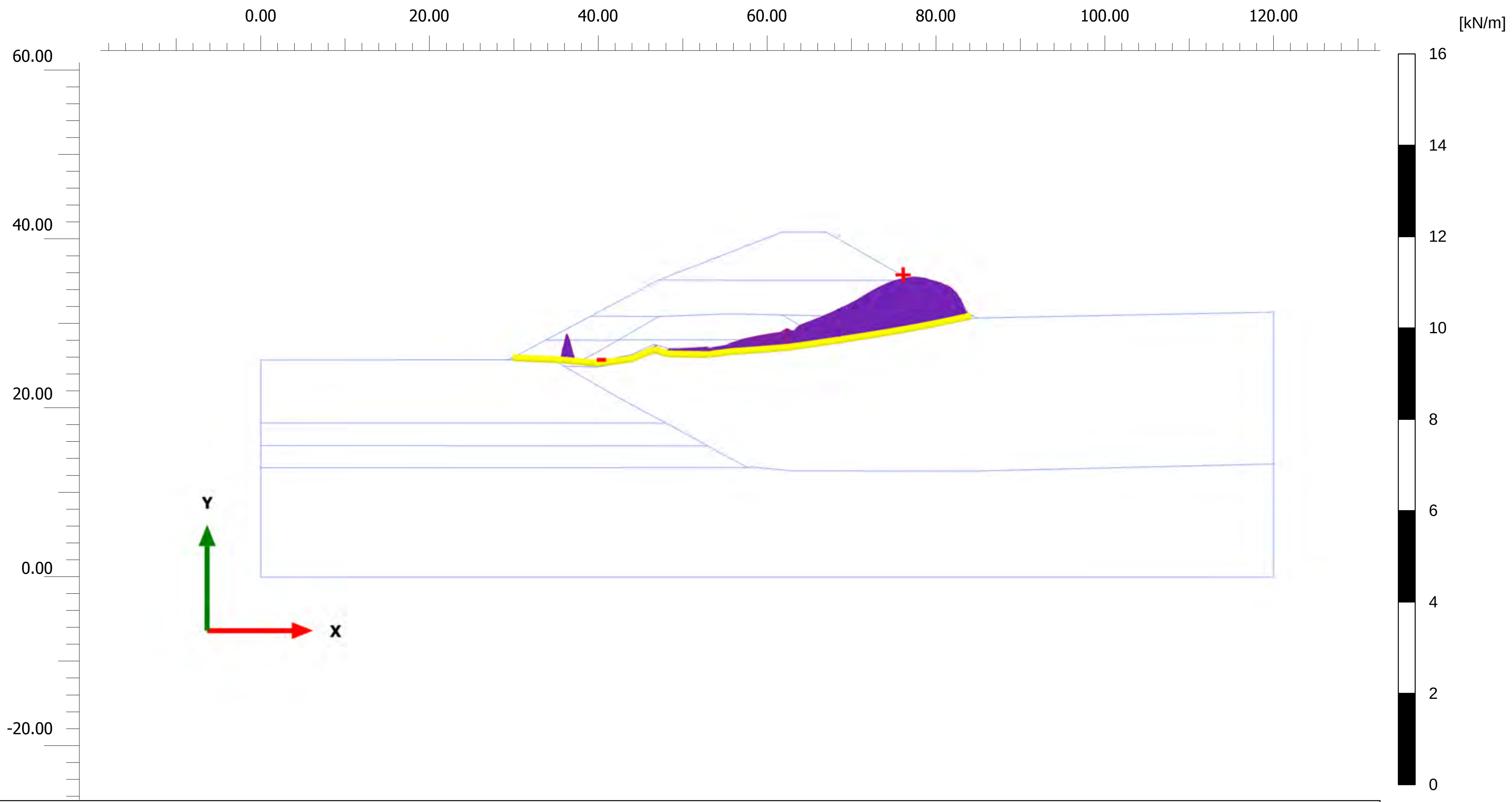
Notes

1. Bentonite property tests performed at a bentonite processing facility before shipment to CETCO GCL production facilities.
2. Bentonite mass/area reported at 0 percent moisture content.
3. All tensile strength testing is performed in the machine direction using ASTM D 6768. All peel strength testing is performed using ASTM D 6496. Upon requst, tensile and peel results can be reported per modified ASTM D 4632 using 4 inch grips.
4. Index flux and permeability testing with deaired distilled/deionized water at 552kPa cell pressure, 531kPa headwater pressure and 517kPa tailwater pressure. Reported value is equivalent to 1×10^{-8} m³/m²/sec. This flux value is equivalent to a permeability of 5×10^{-9} cm/sec for typical GCL thickness. Actual flux values vary with field condition pressures. The last 20 weekly values prior to the end of the production date of the supplied GCL may be provided.
5. Peak values measured at 10kPa normal stress for a specimen hydrated for 48 hours. Site-specific materials, GCL products and test conditions must be used to verify internal and interface strength of the proposed design.

CETCO has developed an edge enhancement system that eliminates the need to use additional granular sodium bentonite within the overlap area of the seam. This edge enhancement is known as SuperGroove™, and it comes standard on both longitudinal edges of Bentomat®DN. It should be noted that SuperGroove™ does not appear on the end-of-roll overlaps and recommend the continued use of supplemental bentonite for all end-of-roll seams.

Attachment B: Assessed tensile forces and strain on GCL

Output Version 2017.1.0.0



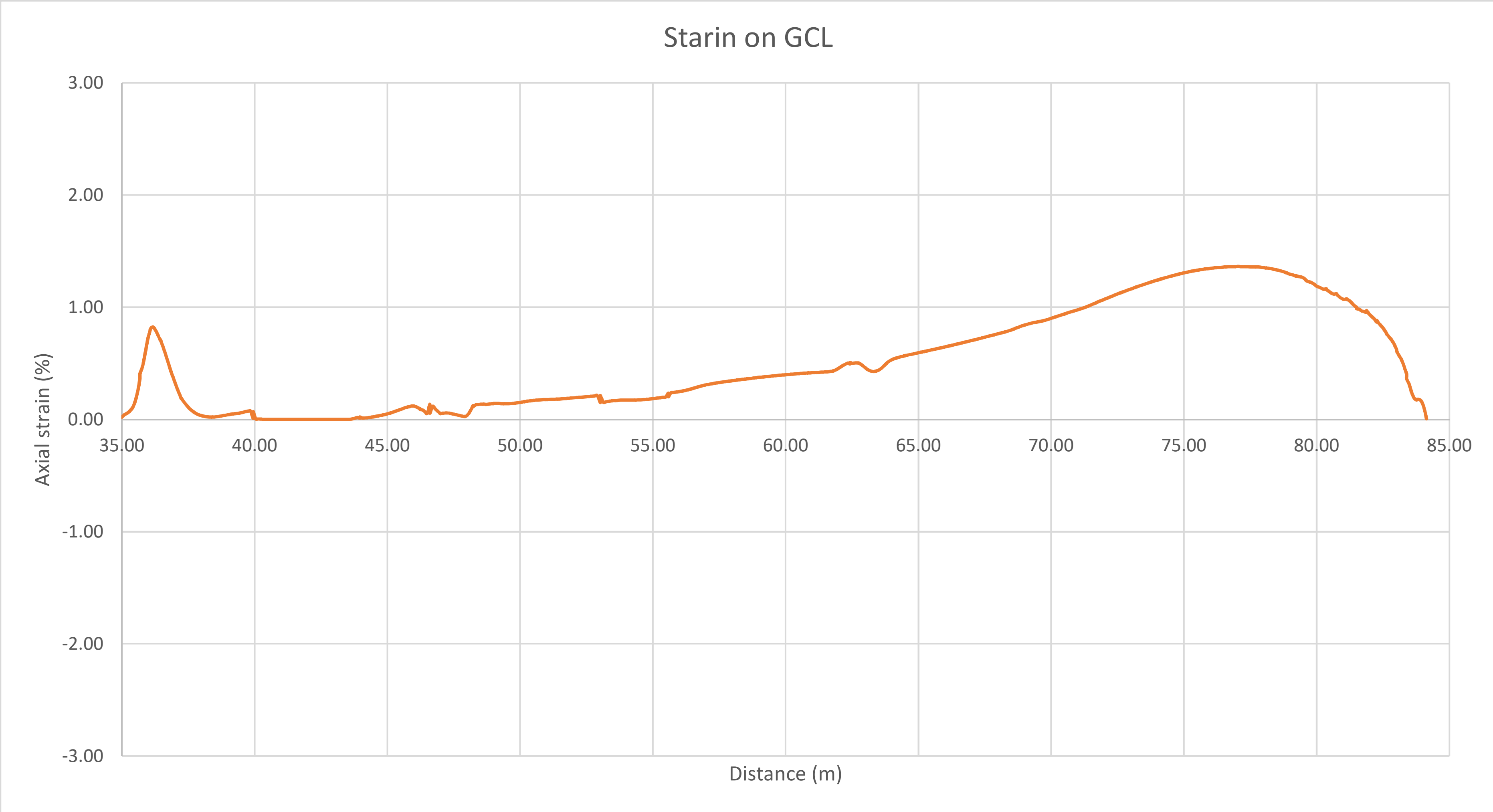
Axial forces N (scaled up 5.00 times) (Time 90.00 day)

Maximum value = 1.199 kN/m (Element 57 at Node 6139)

Minimum value = 0.07898×10^{-3} kN/m (Element 155 at Node 12750)

Project description	Date
Moorebank Intermodal Rail Connection	16/03/2018

Project filename	Step	User name
Moorebank Intermodal Rail ...	16	Coffey Geosciences Pty Ltd


Starin on GCL
3.00
2.00
1.00
0.00
-1.00
-2.00
-3.00
Axial strain (%)
35.00
40.00
45.00
50.00
55.00
60.00
65.00
70.00
75.00
80.00
85.00
Distance (m)

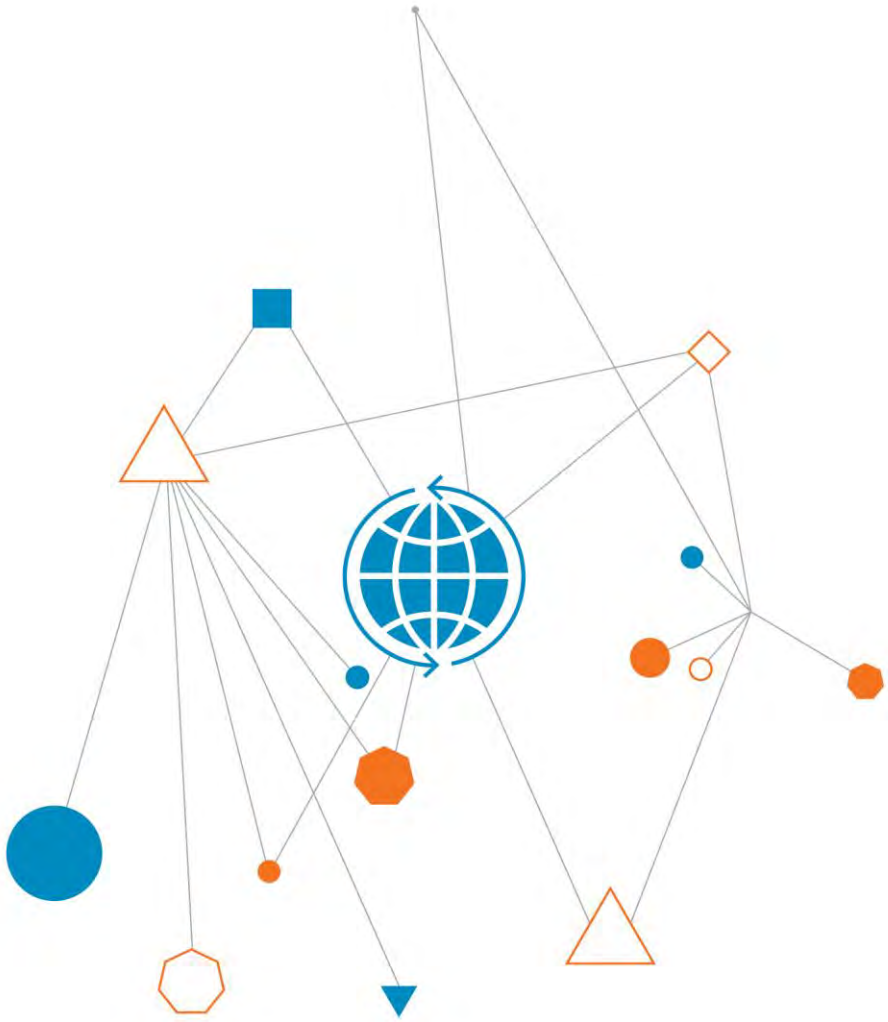
CPB Contractors

Partner:

Moorebank Intermodal Rail Link

Ground Treatment Design between Ch 40,440 and Ch 40,740 (MB2S)

10 July 2018



When you think with a global mind problems get smaller

Moorebank Intermodal Rail Link

Prepared for
CPB Contractors
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10 July 2018

Coffey Reference: GEOTLCOV24072AF-BA

Project Reference: N01031-GRW-DRP-GEO-0001-05

For and on behalf of Coffey

[redacted]

Quality information

Revision history

Revision	Description	Date	Author	Reviewer	Signatory
Original Design based on combination of surcharge and dynamic compaction					
C1	Final Design – Initial issue	17/10/2016	FS/TM	PKW/AP	AP
C2	Final Design – Second Issue	23/10/2016	FS/TM	AP	AP
01	IFC	13/03/2017	FS/TM	PKW/AP	AP
02	IFC – Second issue	19/05/2017	FS/TM	PKW/AP	AP
03	IFC – Third issue	28/09/2017	FS	PKW/AP	AP
Revised Design with Surcharge Only					
04	35% Design - Forth issue with 35% surcharge design for Ch 40440 – Ch 40,740 M2BS	25/06/2018	VN	PKW	VN
05	Final Design (FD-RD) – Initial issue	10/072018	VN	PKW	VN

Distribution

Report Status	No. of copies	Format	Distributed to	Date
[redacted]				
[redacted]				
[redacted]				

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Sketches

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Figures

Pre-IFC-OD Figures

Post-IFC-OD Figures (Revised Final Design)

Appendices

1. Introduction

CPB Contractors Pty Ltd (CPB) has been commissioned by Sydney Intermodal Terminal Alliance (SIMTA) to undertake the design and construction of the Moorebank Intermodal Rail Link (MIRL) as part of the Moorebank Intermodal Terminal Development. The project involves land preparation and construction of the proposed rail line from the existing South Sydney Freight Line (SSFL) to the proposed SIMTA Intermodal Facility over an approximate length of 2.4 km from the northern connection up to the proposed terminal.

The project comprises the construction of two rail tracks which connect to the existing SSFL line at two locations, Northern connection (NC) and Southern Connection (SC). The NC and SC start at Ch 39,300 (MB2N) and Ch 39,900 (MB2S), respectively. Therefore, two control lines namely MB2N and MB2S with different chainages were adopted in the track alignment design. In the western part of the Georges River, the alignment traverses the Glenfield quarry and waste storage facility up to the Georges River crossing. Within this area, the tracks will be constructed on (a) a cut in the vicinity of the existing stockpile and (b) embankment constructed over a landfill improved by ground treatment. The alignment continues from the Georges River along a relatively straight line parallel to the existing East Hills railway line within the land formerly occupied by the Department of Defence (DoD) before it curves northwards in the vicinity of the Moorebank Avenue crossing. The skew crossing ends in an existing Sydney Train property and continues northward along the property owned by DoD. It then crosses the existing Anzac Creek and terminates at approximately Ch 42,140 (MB2S) located adjacent to the terminal.

Coffey Geotechnics Pty Ltd (Coffey) has been engaged by CPB to undertake geotechnical ground treatment design for the rail embankment section from Ch 40,440 to Ch 40,740 (MB2S). The original ground treatment design proposed comprised of surcharge and preloading between Ch 40,440 and Ch 40,550 and dynamic compaction between Ch 40,550 and Ch 40,740, as presented in the issued for construction (IFC) design report N01031-GRW-DRP-GEO-0001-03 dated 28 Sep 2017. The surcharge preloading period was 6 months and track construction period of 5 months were considered in the surcharge treatment design.

In January 2018, modifications to the design and construction program necessitated a change to the surcharge design with a reduced preloading period of 3 months in the Transition Zone from Ch 40,440 to Ch 40,560. In early June 2018 following direction from SIMTA, CPB directed Coffey to undertake geotechnical design using surcharge and preloading in lieu of dynamic compaction for the rail embankment section between Ch 40,550 and Ch 40,740. Therefore, the design for the section between Ch 40,440 and Ch 40,740 reported currently has been progressed in accordance with the following:

- Deletion of a reinforced soil batter at around Ch 40,440. All batters are now at 1.75H:1V except at the end of the ground treatment at the western bridge abutment of the Georges River Bridge at Ch 40,740.
- Surcharge and preloading treatment for a 3 month period followed by a rail track construction period of 4 months.
- Surcharge and preloading treatment for embankment section from Ch 40,440 to Ch 40,550 and from Ch 40,550 to Ch 40,740 in lieu of DC treatment.
- Inclusion of ground treatment beneath the access road on the landfill side.
- Addition of an 80^{o} reinforced soil batter to provide temporary support of the surcharge at the approach embankment of the Georges River Bridge western abutment.

This updated revision 5 report provides the 100% Final Design (FD-RD) of the revised preloading with surcharge for the rail embankment over the transition and landfill area from Ch 40,440 to Ch 40,740 following issue of our 35% design report N01031-GRW-DRP-GEO-0001-04 dated 25 June 2018.

2. Scope of Report

The scope of this report is to present the Final Geotechnical Design of ground treatments within the following areas:

- Transition zone (increasing landfill thickness from west to east): Ground improvement using surcharge between Ch 40,440 and Ch 40,560 (MB2S), and
- Remaining zone (relatively uniform landfill thickness beneath embankment): Ground treatment using surcharge between Ch 40,560 to Ch 40,740 (MB2S).

3. Design Criteria and Standards

3.1. Design Criteria

The main design criteria pertaining to the performance of tracks are given in the Principal's Project Requirements (PPR) of Railway Access and Land Preparation (RALP) No. 1 of Moorebank Intermodal Terminal Development (MITD) dated June 2015. The Performance Specification dated 15 June 2015 (prepared by AECOM) is included as Appendix 8 of the PPR.

Clause 1.2.1 of the PPR specifies that the expected long term post-construction differential settlement of top of the surface is equal to or less than 1:400 over design life of 30 years. As noted in the tender and the Developed Concept Design (DCD) stages, a differential settlement of 1:400 has been adopted for the ground treatment design requirements, considering some maintenance interventions over the design life of 40 years and allow two tamping (or interventions) be carried out during defect liability period of 1 year.

In addition, design is based on the maximum post construction settlement of 500mm as stipulated in the "Head Contract Clarification No 20".

We have previously carried out a design of ground treatment works for the rail tracks over the soft to firm estuarine clay in the Hexham Relief Road project in which the interventions have been adopted to maintain the criteria associated with post construction differential settlement. The summary of this project is provided in Appendix H.

3.2. Engineering Standards and Specification

The following standards and specification were considered in the design:

- Appendix 8 of Performance Specification: Principal's Project Requirements;
- ARTC Standard ETM 08-01 - Earthworks, Formation and Capping Materials (ARTC, 2010) as downloaded from the official ARTC website;
- ARTC Standard ETC 08-01 - Earthworks for New Tracks and Formation Widening (ARTC, 2006) as downloaded from the official ARTC website;
- ARTC Heavy Haul Infrastructure Guidelines – Track, Civil and Structures (2013);
- ARTC ETA-04-01 - Ballast Specification (ARTC, 2007);
- RMS Specification D&C R44 – Earthworks;

- RMS Specification D&C R67 – High Strength Geosynthetic Reinforcement;
- Australian Standard AS5100.2 – Bridge Design Part 2 for Design Loads (2004) was considered in relation to train loadings;
- Australian Standard AS1170.4 – Structural Design Actions – Earthquake Actions in Australia;
- NSW EPA Environmental Guidelines, Solid Waste Landfills, 2nd Edition 2016.

3.3. Order of Precedence

Unless otherwise stated, the following order of precedence has been applied in the design:

- Specific provisions of the Performance Specification;
- ARTC Standard requirements;
- Standard and guidelines adopted by Sydney Trains;
- RMS Standards and guidelines;
- Australian Standards and guidelines;
- Relevant standards, regulations and codes and other documents listed in Appendix A of the Performance Specification;
- Any other relevant Standard Australia codes, standards or specifications not listed in Appendix A of the Performance Specification; and
- Any relevant international codes, standards or specifications not listed in Appendix A of the Performance Specification.

4. Geological Setting and Geotechnical Model

4.1. Regional and Site Geology

The 1:100,000 scale Penrith Geological Map (NSW Department of Minerals, 1991) indicates that part of this site in the western side of Georges River is underlain by Quaternary Alluvium soils with some parts being underlain by older Tertiary Alluvium layers. Part of the site in the eastern side of Georges River is predominantly underlain by Tertiary Alluvium layers. These alluvium soils typically comprise sand, clay and silt. As inferred by the surrounding areas shown on the geological map, the bedrock formation beneath the cover of alluvial soils is comprised of Ashfield Shale or Siltstone of the Wianamatta Group. The Wianamatta Group is in turn underlain by Hawkesbury Sandstone.

Parts of the site located to the west of Georges River have been significantly altered due to past sand quarrying, pond excavation and filling and landfill placement. Based on Consulting Earth Scientist (CES) report (refer CES031101-LAK-15-F Rev 2 dated 31 October 2006), it is understood that the newer landfill located approximately between Ch 40,445 and Ch 40,560 (MB2S) was placed after year 2000. Based on information inferred by the aerial photographs (refer Geotechnical Investigation Report GEOTLCOV24072AF-AM Rev 2 dated 20 July 2016: GIR and Geophysical Investigation Report GEOTLCOV24072AF-AI dated 23 June 2016: GIP), the older landfill (Ch 40,560 – Ch 40,740 of MB2S) has been constructed about 30 years ago. We also understand that the landfill placements took place following the quarrying activity in the alluvial sand. Subsequent investigations (post IFC of the previous dynamic compaction ground improvement design) were carried out and the results presented in Report GEOTLCOV24072AG-AE dated 28 May 2018 (refer post-IFC GI) which included two additional boreholes (BH2021 and BH2022) located near the western abutment of the proposed bridge across Georges River to better define the extent of the landfill.

4.2. Site Investigations and Subsurface Conditions

From Ch 40,445 to Ch 40,560 (MB2S), eight test pits have been excavated as part of the Detailed Design Geotechnical Investigation (DD-GI, refer GIR) to support the characterisation of the fill layer and waste materials. As inferred from the cells presented in the CES report, this area comprises relatively younger landfill materials placed after year 2000. The report also infers a presence of Geosynthetic Clay Liner (GCL) below the waste layers within the newer landfill area. Therefore the site investigation in the form of deep borehole drilling was deemed unsuitable due to the risk of damaging the GCL liner. Additionally, a geophysical investigation using Land Seismic Refraction (LSR) and Electrical Resistivity Imaging (ERI) methods have been carried out (refer GIP) over the newer and older landfill areas.

A number of test pits have been excavated and four boreholes have been drilled within the older landfill area between Ch 40,560 and Ch 40,740 (MB2S) as part of the DD-GI (refer GIR). As inferred by the historical aerial photographs, the age of landfill is likely over 30 years and it is considered an old landfill in which the majority of biological decomposition is likely to have completed (Dimitrios et al, 2013; Van Impe and Bouazza, 1996). It is understood that clay liners may be present below the old landfill although this has not been substantiated. At the western bridge abutment (Ch 40,740), BH2021 encountered 7.7 m of silty clay fill which did not appear to contain landfill material. BH 2022 located a further 25 m east of the western bridge abutment encountered only 1.3 m of clay fill within indication of the presence of landfill. Therefore, we have interpreted the eastern limit of the landfill to be at approximate Ch 40,700 and the fill beyond this point to be general soil and rock backfill on the eastern batter of the old landfill area.

Therefore, the following information have been used to develop the geotechnical model within the landfill area:

- Data from boreholes and test pits from the DD-GI (ref GIR), the geophysical investigation (ref GIP) and the post-IFC geotechnical investigation (ref post-IFC GI);
- The contour data showing the landfill base prepared by Burton & Field Consulting Surveyors (ref Figure 5); and
- Information provided in Consulting Earth Scientists report (CES031101-LAK-15-AF dated 19 February 2007) pertinent to the landfill cells and placement.

It is noted that the contours of landfill base over the old landfill area were generally noted as “estimation only” while the contours over new landfill area were inferred to be more reasonable than those over old landfill area. Based on this information and landfill tip base interpreted from the GIP, we have assessed the landfill thickness and estimated the design parameters for the purpose of the final design. It is assessed that the landfill materials overlies the bedrock layers except on the area where landfill batter is present.

The geotechnical investigation plans and inferred geological sections in the study area are presented in Figures 1 to 4 (from the GIR) and Figures 6 to 11 (from the latest investigations post-IFC-OD) in this report for ease of reference.

The newer landfill continues to the old landfill at approximately Ch 40,560 (MB2S). Based on a number of test pits excavated within the newer landfill area, waste materials have been encountered below the cover layer up to base of the test pits.

In the longitudinal direction, the landfill materials starts at approximately Ch 40,445 (MB2S) and increases in thickness towards south up to Ch 40,460 (i.e. with increasing chainage). The reduced level of landfill base is in the range of RL-2.5 mAHD to -3.5 mAHD below the centreline (between

MB2N and MB2S) of the proposed alignment from Ch 40,460 to Ch 40,695 (MB2S). The base then increases in level approximately from Ch 40,695 (MB2S) until it reaches the original ground surface (i.e. no landfill) at approximately Ch 40,700 (MB2S). Therefore, the portion of the track is likely to be over the landfill batter between Ch 40,695 to 40,700 in the longitudinal direction.

In the transverse direction, the landfill battered upward from the west to east starting approximately below the centreline of MB2N section of the alignment. Therefore, part of the track is likely to be over the landfill batter in the transverse direction. Based on the recent geotechnical investigation (post-IFC-OD) and the geotechnical long section, the thickness of the upper non-waste fill layer (i.e. likely landfill cover layer) is generally 0.5 m within the new landfill area (Ch 40,445 to Ch 40,560 – MB2S) and varies between 0.5 m and 2 m between Ch 40,560 and Ch 40,700 (MB2S) and increasing to about 8 m thereafter.

The geophysical investigation (refer GIP) indicates a steep batter from approximately Ch 40,685 to Ch 40,700 (MB2S) in the southern section of old landfill, in which the landfill materials are bounded by highly conductive materials. During the test pits investigation between Ch 40,690 and Ch 40,710 (MB2S), a layer of fill comprising sandstone gravels and cobbles with some boulders on which the excavation refusal occurred has been consistently observed at depths of 1 m to 2 m below the existing ground level. This was considered in our sensitivity analysis and is presented in Section 5.6 in this report. The interpreted subsurface conditions in the landfill area are shown in Table 1 below.

Table 1 – Interpreted Subsurface Profiles under the Embankment between Ch 40,427 and Ch 40,740 of MB2S

Unit	Material/ Origin	Depth to Top of Unit (m)	Thickness of Unit (m)	Elevation at Top of Unit (m AHD)	Description
1b	Upper Fill	0	0.5 to 2.0 to Ch 40,700 approximately, then up to 8 m thereafter	4.3 to 16.7	Fill comprising granular materials, gravelly clay, silty clay and clay
1a[2]	Landfill[2]	0.5 to 2.0	0 to 16	2.3 to 16.2	Waste materials with variable contents including soil layers[1]
1/2/3	Fill/Alluvium/Residual Soil	See Note (3)			
4/5	Bedrock	*Note: We adopted the base of landfill based on the results of geophysical investigation, contour plan of landfill base, CES report.*			

Note :

(1) Soil layers within the landfill materials can be parts of intermediate landfill covers or intermixed with the waste materials

(2) No landfill exists between Ch 40,427 and Ch 40,445 (MB2S)

(3) These layers were not reached or sufficiently penetrated due to constraints associated with the risk migration of damaging the liner

An average groundwater level of +2.6 mAHD was adopted for the landfill area based on the levels observed in the boreholes and monitoring wells located in the proximity of the landfill area (refer GIR).

4.3. Landfill Characterisation

Landva and Clark (1990) categorised landfill materials into organic and inorganic materials. The organic materials were divided into putrescible and non-putrescible materials depending on the rate of and resistance against biodegradation. The inorganic materials were generally non-biodegradable (inert) although most of metals undergo chemical degradation due to corrosion.

The continuous samples from boreholes BH7 and BH8 along with the Standard Penetration Test (SPT) samples from BH9, BH9a and a number of test pits were used to assess the composition of landfill materials. The logs of boreholes and test pits together with the photographs of the continuous samples are attached in our GIR. The volume percentage of each constituent of landfill materials was assessed using visual observation during the drilling and with further aid of photographs. The weight of each constituent was assessed using the typical dry unit weight for each constituent from published data (Landva and Clark 1990) and the assessed volume of each constituent. The composition of landfill materials is shown in Table 2 in percentage of each constituent by dry weight.

Table 2 – Composition of Landfill Materials by Dry Weight

Constituent of Landfill Materials	Material Description (based on Landva and Clark 1990)	Percentage by Dry Weight (%)
Granular soil (sand, gravel and cobble)	Inorganic (non-degradable)	19
Fine soil (clay and silt)		17
Brick and concrete fragments		3
Glass and ceramics		4
Metal		14
Wooden and timber	Non-putrescible organic (slower rate and highly resistant against biodegradation)	11
Plastic including rigid plastic		17
Textile		2
Others (non-putrescible)[1]		2
Paper and its derivatives	Putrescible organic (readily biodegradable)	3
Organic Materials		7
Others (not identified)[2]		1

Note :

1. These material types (foam and rubber) were non-putrescible and had an individual percentage of less than 2% by dry weight

2. These material types could not be visually identified from the sample during the drilling. They were considered as putrescible organic materials for the analyses purpose

As summarised in Table 2 above, the putrescible organic material (OW) comprised approximately 11% of the old landfill materials by weight (including unidentified material). A reasonable percentage of soil fill may be associated with the placement of intermediate covers. A relatively low amount of putrescible organic materials is also inferred by a higher N-values observed during the Standard Penetration Test (SPT) in BH9a. The SPT tests typically encountered refusal due to solid materials such as metal or large timber.

4.4. Design Parameters

4.4.1. Landfill Properties

Most of the recovered materials from continuous samples (BH7 and BH8) were generally described as dry to moist (as shown in the sample photographs presented in the GIR). Some free water observed in a few samples could mostly be attributed to drilling water used during the sonic drilling. Therefore, a moisture content (w) of 20% is considered as it is in reasonable agreement with the anticipated putrescible organic content (OW) of the landfill (based on the correlation between organic content and water content of landfill materials by Hyun et al. 2011 and Landva and Clark 1990).

Based on the percentage by dry weight of each constituent material presented in Table 2 and dry unit weight of individual constituent (Landva and Clark 1990), the assessed dry unit weight of landfill is 11.5 kN/m^3. The dry unit weight of 11.5 kN/m^3 (and hence bulk unit weight γ_b= 13.8 kN/m^3) adopted in our design is generally consistent with the typical range of γ_b (Dimitrios et al 2006) between 10 kN/m^3 and 15.5 kN/m^3 for landfill that has undergone some compaction during the placement and has some soil cover. The evidence of compaction can also be inferred from the photographs of continuous samples presented in the GIR.

Based on the findings summarised above, we have characterised the landfill within the GWS using a methodology presented by Bareither et al. (2012) as provided below.

1. A non-dimensional parameter, Waste Compressibility Index (WCI), is used to characterise the waste materials (refer Equation 1). This parameter takes into account the variability of biodegradable materials and can be correlated with the compressibility parameters of waste materials.

$$WCI = w\left(\frac{\gamma_w}{\gamma_d}\right)\left(\frac{OW}{100-OW}\right) \qquad (1)$$

   Where OW is the percentage of readily biodegradable (putrescible) materials (i.e. food waste, paper, cardboard, etc) and γ_w is the unit weight of water. Other parameters are as previously defined.

For landfill at GWS, the WCI is 0.021 for the assessed OW of 11%.

4.4.2. Development of Design Parameters

Bareither et al. (2012) provided a correlation between Compression Ratio (CR) and WCI using a large number of data from numerous literatures. In the correlation plot, they also presented upper and lower bound values to take into account the variability.

Based on above, assessed CR for the landfill is in the range of 0.075 and 0.25. Hence we have adopted CR value of 0.15 in the Final Design (FD). To assess the impact of the variability of CR value on the settlement performance of the embankment, sensitivity analyses have been carried out and is presented in this report.

The secondary compression ratio ($C_{\alpha e}$) adopted in our design took into account both mechanical creep and remaining biological creep. The biological creep is caused by the biological decomposition of organic materials. This is influenced by a number of factors such as landfill age, landfill composition, water circulation within the landfill and other conditions that facilitate or retard the biodegradation process.

The assessed values of $C_{\alpha e}$ which are available in literature along with the landfill age and corresponding WCI are compiled and tabulated in Table 3 below.

Table 3 – Summary of secondary compression ratio ($C_{\alpha \varepsilon}$) published in literatures (field and experimental measurements) and the value adopted in the design for unimproved landfill

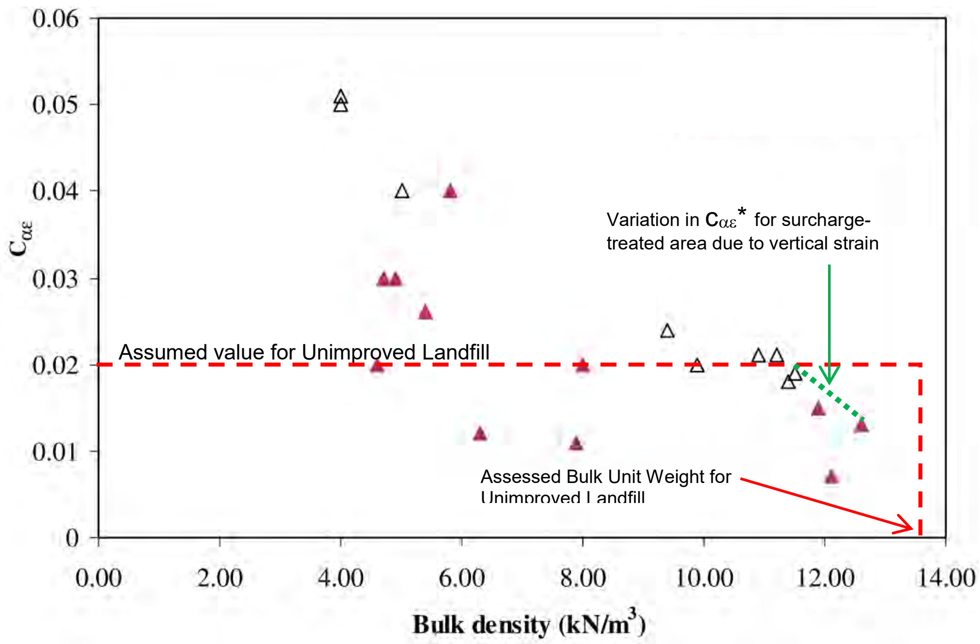
Reference	Age of Landfill or Comment on Decomposition State	Percentage by weight of degradable materials	Assessed WCI[1]	$C_{\alpha e}$
Gabr and Valero (1995)	15 – 30 years	2	0.02 – 0.039	0.015 – 0.023
Landva et al (2000)	15 years (w = 15.6%)	47	0.15	0.012
Hyun et al (2011)	10 years (young)	0.05	0.0001	0.001
KGS (1994)	8 years	3.3	0.008	0.005
Yuen et al (1997)	Fresh	68.7	0.55	0.033
Chen et al (2010)	Inhibited decomposition (bulk unit weight = 7.9 kN/m^3)	56.1	0.718	0.011
	Inhibited decomposition (bulk unit weight = 12.1 kN/m^3)	56.1	0.416	0.0071
DCD and FD Stage (35% - 100%)	***Landfill (about 15 - 30 years old)***	***11[2]***	***0.021[2]***	***0.02[2]***

Note :

1. Some values of WCI are based on those already computed in Bareither et al. (2012) while other values were assessed based on material composition, water content and dry unit weight provided in the cited literatures
2. Values adopted in the Developed Concept Design stage (35%)

The adopted value of $C_{\alpha e}$ for FD stage is generally reasonable in comparison with the typical values published in the literature by taking into account the age and composition of landfill.

Siddiqui et al. (2013) presented a correlation between bulk density (γ_b) and $C_{\alpha e}$ where the authors stated that the $C_{\alpha e}$ increases with the decrease in γ_b (see Sketch 1).



Sketch 1 – Relationship between Bulk Density (γ_b) and $C_{\alpha e}$ (from Siddiqui et al 2013)

A similar trend as above in Sketch 1 is also presented by Chen et al. (2010), Wall and Zeiss (1995) and Sowers (1973). This demonstrates that the reduction in void ratio due to the application of external load (i.e. surcharging or dynamic compaction) in general reduces the value of $C_{\alpha e}$.

For the new landfill treated by surcharge (CH 40,427 to CH 40,560 of MB2S), the improvement in the form of reduction of $C_{\alpha e}$ is induced by the vertical strain (εv) due to the embankment load during the preloading. Therefore, the following relationship is adopted for the transition zone:

$$C^*_{\alpha\varepsilon} = C_{\alpha\varepsilon} - \beta\, \varepsilon_v \; where \; C^*_{\alpha\varepsilon} \geq 0.015 \qquad (1)$$

Where $C^*_{\alpha\varepsilon}$ and $C_{\alpha\varepsilon}$ denote secondary compression ratios for treated and unimproved landfill, respectively. The upper value of 0.02 was selected for unimproved landfill with the age of 15 years or older (see Table 3 above) while the lower limiting value of 0.015 was selected on the basis of the effectiveness of surcharge treatment which is generally lower than that of the DC treatment as reported in a number of case studies presented in literatures (Dimitros et al 2013; Sharma and De 2007) within the same subject area. A value of β of 0.07 was selected based on the trending shown in Sketch 1 above. In Sketch 1, the line associated with of β of 0.07 is located within the higher values of $C_{\alpha e}$ which was adopted to take into account the pre-treatment uncertainties and will be revised following the settlement monitoring during the waiting period.

In the original design for the old landfill treated by Dynamic Compaction (Ch 40,560 to CH 40,740 of MB2S), a vertical strain of 15% is assumed for the design purpose based on literature (Dimitrios et al. 2013). By using the assumed strain, the improved dry unit weight of 13.5 kN/m^3 (bulk unit weight γ_b = 16.2 kN/m^3) was estimated following the DC. Although DC is no longer used in the current revised design, the previously adopted parameters for DC treated ground are included here for reference purposes for the surcharge design, as discussed further below.

Using Sketch 1 and assessed bulk density, we have selected $C_{\alpha e}$ for unimproved and improved landfills as follows:

- $C_{\alpha\varepsilon}$ for unimproved landfill (old and new) = 0.02
- $C^*_{\alpha\varepsilon}$ for treated older landfill (with Dynamic Compaction) = 0.01
- $C^*_{\alpha\varepsilon}$ for treated newer landfill (with Surcharge) = vary between 0.015 and 0.02 as per Equation (1)

A summary of adopted design parameters for the FD stage ground treatment design are presented in Table 4.

Table 4 – Design Compressibility Parameters

Design Parameters	Unit	Embankment Fill to be placed	Existing Soil including fill and residual soil	Landfill (relatively new)	Landfill (relatively old)
Bulk Unit Weight	kN/m^3	21	20	13.8	13.8
Constrained Modulus, M’	MPa	25	15	-	-
Compression Ratio CR (existing unimproved condition)	-	-	-	0.15	0.15
Modified Compression Ratio CR* (due to Surcharge and Dynamic Compaction)	-	-	-	0.075(1)	0.075(1)
Secondary consolidation strain rate $c_{\alpha\varepsilon}$ (existing unimproved condition)	-	Note (2)	Note (2)	0.02	0.02
Modified Secondary consolidation strain rate $c_{\alpha\varepsilon}$* (due to Surcharge or Dynamic Compaction)	-	Note (2)	Note (2)	0.02 – 0.07ε_v, but ≥ 0.015 (surcharge) (3)	0.01(1) (DC)

Notes:

(1) We assumed that compressibility parameters for DC treated landfill is 50% of the compressibility parameters of unimproved landfill (based on published data from Sharma and Anirban, 2007 and Dimitrios et.al. 2013).

(2) The secondary consolidation settlement of non-landfill fill materials was calculated by assuming 0.15% of the total non-landfill fill thickness over 40 years.

(3) Secondary consolidation (creep) strain rate for landfill improved by surcharging has been limited to be not less than 0.015 compared to creep strain rate of 0.01 for old landfill area treated by DC. This

difference in creep strain rate is considered reasonable considering effectiveness of DC treatment and slightly higher moisture content (than that of old landfill) observed for the new landfill.

For the purpose of stability analyses (Limit Equilibrium method), the parameters shown in Table 5 below have been adopted:

Table 5 – Summary of Parameters used in the Stability Analyses

Unit	Material Type	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (degree)
1c	Upper Fill	20	2	33
1a	Landfill (existing/unimproved)	13.6	0	30
2	Alluvium Sand	20	0	33
1	Existing Fill	20	5	30
1b	Landfill (treated)	16	0	32
3	Residual Soil	20	5	26
4	Class V/IV Sandstone	22	10	30

4.4.3. Design Parameters for General Embankment Fill and Select Fill

The embankment fill properties adopted in the design of general embankment (constructed as per R44) are:

- Cohesion = 5 kPa
- Friction angle = 30 degrees
- Unit weight = 21 kN/m^3

The embankment fill to be used in the reinforced embankment should satisfy select fill (constructed as per R44). The select fill properties adopted in the design of reinforced embankment are:

- Cohesion = 0 kPa
- Friction angle = 34 degrees
- Unit weight = 21 kN/m^3

4.4.4. Design Parameters for Geogrid

Multiple layers of structural geofabric with a short term ultimate tensile strength of 100 kN/m (Paralink or equivalent) at 12% strain are proposed to provide short term embankment stability of the reinforced embankment at the bridge approach at Ch 40,740. Based on technical data sheet of Paralink, a material strength reduction factor of 2 has been adopted for the analysis. If different product is used, the strength reduction factor must be confirmed by Coffey. Factor of Safety (FOS) dependent tensile strength has been considered in the stability analysis for embankments in landfill area.

The properties as listed below were used for calculating the bond resistance between geogrid and embankment fill:

- Bond cohesion = 0 kPa
- Bond friction angle = 34°
- Interface factor = 2
- Bond safety factor = 2

Supply and installation (including use of embankment fill material) of geogrid should be as per RMS R67.

4.4.5. Design Parameters for Earthquake Loading

Based on Table 3.2 of AS1170.4 – 2007, the hazard factor (Z) adopted for Sydney is 0.08. The hazard factor is equal to the acceleration coefficient (a_{max}) for an assumed Annual Exceedance Probability (AEP) of 1/500.

For the pseudo-static stability analyses using the method of slices, the additional destabilising force due to earthquake loading is calculated by multiplying the slice weight by the seismic coefficient (k_h). The value of seismic coefficient is equal to the acceleration coefficient (a_{max}) of 0.08.

5. Design Description

5.1. Proposed Works

The proposed work comprises construction of embankment along the landfill area (Drawing No. N01031-GRW-DRG-GEO-0012-02). The design drawings for the final design are presented in Appendix A. It starts with a high embankment of up to 14.4 m high above the existing ground level (EGL) to 2.1 m high above EGL at Ch 40,560 (MB2S) where the older landfill starts. The lower embankment continues to reduce in thickness up to the EGL at approximately Ch 40,660 (MB2S). The embankment thickness increases again from Ch 40,705 (MB2S) up to 2 m at Ch 40,740 near the north-western approach of Georges River crossing.

5.2. Design Developments

This section summarises various design stages associated with the geotechnical design of railway formation on landfill area. The design milestones are:

- Developed Concept Design of Original Design (DCD-OD) (previously submitted);

- Final Design of Original Design (FD-OD):
    - FD-OD initial issue (previously submitted), and
    - FD-OD Second issue (previously submitted).
- Issue For Construction of the Original Design (IFC-OD) (previously submitted);
- Detailed Design of Revised Design in Transition Zone between Ch 40,440 and Ch 40560:
    - Design analyses, drawings and model completed and issued for comments in May 2018 but not reported due to instruction by CPB to further revise the design to preloading with surcharge only (see below);
- Design of Revised Design using preload with surcharge only:
    - 35% Concept Design of Revised Design (CD-RD) reported in N01031-GRW-DRP-GEO-0001-04 dated 25 June 2018;
    - 100% Final Design of Revised Design (FD-RD) – this report.

5.2.1. Developed Concept Design (DCD-OD)

The DCD (35% stage) of the original design was previously submitted under 2 separate covers:

- Ground Treatment Design for Transition Zone south of Viaduct (Ch 40,427 – Ch 40,560 of MB2S): Developed Design (35%) Report, GEOTLCOV24072AF-AN Rev 1 dated 20 May 2016); and
- Ground Treatment Design for Dynamic Compaction treated area (Ch 40,560 – Ch 40,740 of MB2S): Developed Design (35%) Report, GEOTLCOV24072AF-AR Rev 1 dated 20 May 2016).

In the DCD, the design has been developed by considering relevant revisions made after the tender design stage. This included the changes of vertical and horizontal profiles of the alignment. This design has been reviewed by third parties on behalf of SIMTA. The responses to the review comments on the design reports have been provided in Appendix B.

5.2.2. Final Design of Original Design (FD-OD)

FD-Initial issue presented the final geotechnical design, which includes all the engineering standards, specifications, calculation summaries and the final design drawings. The FD included a summary of those items presented in previous submissions, address inter-discipline integration aspects, and document and address any comments received from the SIMTA. The FD included a final version of written report and include a specific ground treatment design as well as the final design drawings. The responses to the review comments on the initial issue of FD report have been provided in Appendix B.

Since the initial issue of FD-OD, the design was updated to incorporate the latest changes to embankment profile. A steep batter of 0.6H:1V is proposed for the western slope of the rail embankment between Ch 40,427 and Ch 40,470 (MB2S). The slope of the batter was then gradually reduced to 1.75H:1V at Ch 40,490. Hence, the embankment from Ch 40,427 and Ch 40,490 (MB2S) was designed as a reinforced embankment with flexible facing.

5.2.3. Issue For Construction of the Original Design (IFC-OD)

The IFC Design of the original design (IFC-OD) for construction purposes and incorporated amendments to the FD-OD to address the comments from SIMTA, other design disciplines and construction team.

5.2.4. Detailed Design of Revised Transition Zone between Ch 40,440 to 40,560

In January 2018, Coffey was engaged to revise the ground treatment design of the Transition Zone between Ch 40,440 to 40,560 to account for the following civil design and construction programming changes:

- A re-design of the SIMTA Rail Alignment through GWS Landfill will remove the Viaduct and replace it with an earthworks embankment with 1.75H: 1H batters. A transition in a steeper reinforced earth embankment before Ch 40,440 is therefore no longer required.
- Additionally Program constraints require the current Transition Zone surcharging to be complete within 3 months for Primary & Secondary Consolidation.
- Inclusion of the construction haul road on the eastern side (i.e. GWS landfill side) in the embankment/ground treatment design.

Coffey provided preliminary design drawings in late February 2018 for CPB review, and following receipt of CPB review comments, Coffey completed the detailed analyses and design of the Transition Zone ground improvement in April/May 2018. Coffey provided design drawings and 12D model of the revised design at the Transition Zone. However, a formal revised design report was not submitted because on 28 May 2018, CPB informed Coffey that SIMTA advised that Dynamic Compaction would not be permitted in the older landfill area due to concerns regarding the risk of detrimental impact to the integrity of the underlying waste cell and liners. Coffey understood that despite justifications given on previous use of Dynamic Compaction on landfill sites and methodology provided on the proposed trial with instrumentation to enable adjustment of the energy of DR blows, DR would not be permitted on this project.

5.2.5. Revised Final Design using Preload with Surcharge (FD-RD)

Following acceptance of Coffey's fee proposal GEOTLCOV24072AG-AK_Rev1 dated 30 May 2018 by CPB, Coffey redesigned the ground improvement from Ch 40,440 to Ch 40,740 using preload with surcharge only.

A 35% Concept Design of the Revised Design (CD-RD) was reported in N01031-GRW-DRP-GEO-0001-04 dated 25 June 2018, and the 100% Final Design of the Revised Design (FD-RD) is presented in this report. Included in this report are some changes to the end of the Transition Zone at Ch 40,550/40,560 to merge with the surcharge profile thereafter, and design of a steep (80^{o} from horizontal) reinforced soil batter at the end of the ground treatment at the bridge approach (i.e. approximately Ch 40,740) together with temporary sleeving of the abutment piles to enable pile construction to take place during the preloading period.

5.3. Design Methodology

5.3.1. General Ground Improvement Design Philosophy for Landfill

Landfill is a highly compressible material with long-term biodegradation creep settlement as well as primary consolidation behaviour. Ground improvement on landfills are typically targeted at removing all primary consolidation prior to construction of the structure, and reducing the long-term post-construction creep settlement by reducing the void ratio and hence the rate of biodegradation of the landfill.

Compared to naturally occurring soft soils, landfill is an “uncontrolled fill” which is expected to have the following adverse properties:

- Much higher variability (composition and compressibility)
- Higher void ratio
- Relatively stronger and stiffer for its void ratio

Due to the above adverse landfill properties, conventional preloading with surcharge thickness of around 2 m would not provide adequate confidence of treating the landfill to an “engineered fill” standard. This is because the conventional surcharge thickness mentioned above is too low to impose sufficient compaction energy to reduce variability within the fill. Under a relatively low static preload, the interlocking nature of the landfill particles may prevent consolidation. However, over the long-term biodegradation will cause weakening of particle interlocks, then other external factors (e.g. groundwater movement, cyclic dynamic train loading and earthquake loading) may cause reshuffling of materials into voids which could pose the risk of potential sudden, localized collapse settlements which is undesirable from a safety perspective for train operations.

Dynamic Compaction was proposed in the original design because this process is known from experience to be able to achieve high energy to a sufficient depth (up to 10 m in the original design) to alleviate the above concerns.

For the Transition Zone between Ch 40,440 and Ch 40,560 (MB2S) where preloading with surcharge was proposed to treat the landfill, the thickness of surcharge was made higher than would be designed using conventional consolidation theory to provide a static load that is equivalent to the dynamic energy imposed by the proposed DC (by considering similar settlement to be achieved during ground treatment). A similar strategy is adopted for the revised design presented in this report for the entire zone (i.e. Ch 40,440 to Ch 40,740), resulting in a surcharge thickness of up to 9 m from Ch 40,560 to Ch 40,700. From Ch 40,700 to the bridge abutment at Ch 40,740, the surcharge thickness is reduced due to the apparent absence of landfill (i.e. only non-waste fill appears to be present).

5.3.2. Settlement Assessment of Landfill Materials

Standard laboratory testing to assess strength and compressibility parameters is not practically feasible for landfill materials due to difficulties in extraction of samples and health, safety and environmental issues. Therefore the design approach adopted for the landfill area is divided into two stages:

a) Pre - Construction Ground Treatment Design

   The pre-construction ground treatment design is based on data from the past experience and extensive literatures on landfill with similar properties and ages as provided in this design report. In general, the compressibility parameters reported are based on the macro settlement behaviour of landfills.

b) Post - Construction Ground Treatment Design

   The post-construction ground treatment design involves refinement of settlement and additional surcharge thickness (if any required) based on settlement monitoring data gathered during the waiting period.

5.3.1.1. Pre-construction ground treatment design

The development of design parameters was previously outlined in Section 4.4 using an extensive data from the literatures and past experience on similar landfill.

Ground treatment requirements have been assessed to achieve the following criteria as discussed in Section 3.1:

- Maximum differential settlement of 0.25% change in grade for a maintenance intervention;
- Maximum post construction settlement of 500mm as stipulated in the “Head Contract Clarification No 20”; and
- Carry out three tampings (or maintenance intervention) during the defect liability period of 1 year for area between Ch 40,427 and Ch 40,740.

Ground treatment design was carried out at “selected locations” considering the following:

- Variation of landfill thickness along the alignment; and
- The variation in landfill thickness due to the landfill batter along the transverse direction to the alignment.

The primary settlement of the landfill due to embankment fill has been assessed using the Compression Ratio (CR) in the following equation:

$$S_{prim} = CR \ H_{Lo} \ \log\frac{\sigma'_{vo} + \Delta\sigma'}{\sigma'_{vo}} \qquad (2)$$

Where H_{Lo} denotes the initial thickness of landfill while σ'_{vo} and $\Delta\sigma$ are the existing effective vertical stresses and effective vertical stress increase due to embankment fill material. Based on literature (Wall and Zeiss 1995, Bareither et al., 2013) and our experience in similar landfill, the primary compression generally completes relatively fast within one month. Hence, for current design, primary consolidation settlement is assumed to be completed in one month period.

The secondary compression of landfill materials is usually assessed to take into account the time-dependent mechanical creep and biological creep. The following equation is used to assess the secondary consolidation settlement of the landfill materials:

$$S_{second} = \ Creep\ strain\ rate * \ H_{L1} \log\frac{t_2}{t_1} \qquad (3)$$

where t_1 and t_2 denote the reference time and end time (i.e. t_2 = the design life + t_1) of secondary settlement respectively. Term H_{L1} is the remaining landfill thickness after the primary consolidation completes. The adopted creep strain rates for improved and unimproved landfill materials are provided in Table 4.

The ground treatment requirements have been designed such that a reduced creep strain rate is achieved in order to meet the settlement criteria discussed above.

5.3.1.2. Post-Construction ground treatment design

Due to uncertainties and the inevitable variability in the ground conditions, it is current industry practice to adopt an observational approach to assess embankment performance. As outlined above, these uncertainties and variability are greater in the landfill area in comparison to the areas underlain by soil.

Embankment performance comprises stability during construction, comparisons of predicted and measured settlements and lateral movement during and construction and waiting periods. The Observational Method is widely used in practice to monitor geotechnical performance and will be a primary tool used in the design and construction of embankments. The Observational Method in geotechnical engineering is an on-going, managed, integrated process of design, construction control, monitoring, review and back analysis which enables the construction team to:

- Regularly re-evaluate design assumptions and predictions; and
- Modify embankment construction during construction based on observations and/or take remedial actions where required or take advantage of better than the predicted performance.

Important steps in the application of this method are listed below:

a) install settlement plates, inclinometers, Hydrostatic Profile Gauges (HPG) and survey monuments to capture movements at critical locations;

b) carry out baseline predictions at selected settlement plate locations where possible;

c) monitor settlement plates, inclinometers, Hydrostatic Profile Gauges (HPG) and survey monuments at an appropriate frequency;

d) perform visual inspections of the embankments during construction for signs of distress; and

e) back-analyse monitoring results to review and reassess settlement performance.

5.3.1.3. Instrumentation and Monitoring

In order to back analyse the secondary compression parameter ($C^{*}_{\alpha\varepsilon}$) for improved landfill, the settlement monitoring must be carried out during the waiting period. We recommend the following instrumentation, construction and monitoring approach:

1. Install the following instrumentation as shown on Drawing No. N01031-GRW-DRG-GEO-0010-09 in Appendix A:

    a. 32 Settlement Plates to monitor the magnitude and rate of settlement. Fewer settlement plates may be adopted if more HPGs and downhole hydraulic vibrating wire settlement sensors (DHVWSS) are used (see (d) below);

    b. 25 survey monuments at the toe of the embankment to monitor the lateral movement. At the embankment, the tip of settlement plates (survey prism attached to the tip) can be monitored during the waiting period to measure the lateral deformation;

    c. 11 inclinometers are proposed at the toe of the batter to assess lateral movement and monitoring of potential embankment instability

    d. 3 Hydrostatic Profile Gauges (HPG) for post-construction settlement monitoring purposes. The HPGs can also be used to measure the settlement during construction and the waiting period, and in particular differential settlement during preloading caused by landfill batters. If more HPG are used together with DHVWSS, the majority of the settlement plates may be replaced by such instrumentation. For example, the 32 settlement plates recommended in (a) above may be replaced by 4 settlement plates and 14 HPG and 2 sets of DHVWSS.

    The locations of settlement plates, survey monuments, inclinometers and HPGs are included in the drawings (refer N01031-GRW-DRG-GEO-0010-09). Once the rail tracks have been constructed, survey monuments (say at every 10 m interval) can be established on the rail sleepers to monitor settlement and lateral movement at rail level.

2. Place the fill up to the top of surcharge level (ref Drawing N01031-GRW-DRG-GEO-0005-03). Placement of the formation shall be as per relevant ARTC standards (ETC-08-01 and ETM-08-01).

3. Monitor the settlement using settlement plates over the proposed waiting period of 3 months.

4. Monitor the lateral movement using inclinometers, survey monuments and settlement plate tips to assess the embankment stability and also to predict the long term lateral movement of the rail lines.

5. Review the monitoring data and reassess the compressibility parameters based on observed primary settlement and secondary consolidation settlement, and provide revised long term settlement prediction and potential changes in intervention regime.

Monitoring data will be reviewed on a weekly basis. It is anticipated that after the first month of readings confirmation of primary consolidation would be possible and some information on creep settlement within the next 2 months of waiting period would allow preliminary validation of creep parameters to be made.

Intermediate back-analysis will be carried out in 2.5 months to predict the post construction settlement. The predicted post construction settlement, together with any recommendations for additional surcharge or extension of waiting period (if required) will be submitted including predicted intervention periods to CPB for their review. The monitoring will be continued until the end of the waiting period. At the end of waiting period, we will refine the settlement prediction as required and report the refined post construction settlement values together with predicted intervention periods to CPB.

The summary of various stages for monitoring and review is presented in Appendix C. The above processes for the surcharge and preloading ground treatment are described in the process control diagrams shown in drawings N01031-GRW-DRG-GEO-0008-09.

Subject to discussions with CPB on its preference, the type and quantity of settlement monitoring will be finalised in the IFC-RD report, and a revised Instrumentation and Monitoring Plan will be provided.

5.3.3. Finite Element Analyses of Post Construction Settlement and Corresponding Lateral Deformation

Finite Element (FE) analyses were carried out using a commercially available computer program PLAXIS 2D version 2016. The FE modelling is used to assess the ratio of lateral deformation and settlement under the alignments.

The post construction settlement occurs due to the long term secondary settlement of the landfill materials. The simulation of secondary settlement in PLAXIS 2D, in general, is carried out using the Soft Soil Creep (SSC) model.

However the SSC model has the following limitation in the modelling of some aspects of secondary settlement.

a. SSC model is known to overestimate the lateral deformation (Fatahi et al., 2013; Grimstad et al., 2014) beneath the embankment and the deformations at a distance away from the embankment; and

b. SSC model only simulates isotropic condition (Sivasithamparam et al., 2015) while the landfill materials are expected to be anisotropic.

<u>Overestimation of Lateral Deformation</u>

Published results (Fatahi et al., 2013) indicate that the assessed ratio between lateral movement at the toe of embankment and maximum settlement under the embankment is relatively higher than that observed in the field.

To compare the above limitations in SSC model, we have model a instrumented embankment over very soft to soft clay constructed as part of the approaches to the Flood Plain Bridge No. 1 (FPB1) of the Pacific Highway Upgrade, Ballina Bypass project (NSW Australia) using SSC model in PLAXIS

2D. Considering numerical modelling using SSC model with appropriate parameters, settlement and associated lateral movement behaviour of soft soil and landfill material is expected to be similar. Hence, deformation under FPB1 embankment is considered appropriate in comparing numerical modelling and likely field behaviour.

FPB1 embankment is underlain by a 13 m thick of very soft to soft estuarine clay. This simulation is detailed in a brief note in Appendix F. From the back-analysis, the ratio of lateral deformation at the embankment toe to the vertical settlement under the centre of embankment was assessed. This ratio was compared against the ratio from the field measurement. The analyses summary and its comparison against field measurement are shown in Table 6. The locations referred to in Table 6 is presented in Sketch 2

Table 6 – Summary of Back-analyses of FPB1

Data source	Maximum Settlement under the centre of embankment (Location C[4]), (mm)	Max. Lateral Deformation at: Location A[1] / Location B/ Location D[4] (mm)	Ratio of Lateral Deformations to Settlement (%)		
			Location A[1,4]	Location B[3, 4]	Location D [3, 4]
PLAXIS 2D using SSC model	49	9 / 6 / 3.6	18	12	7
Field Measurement[2]	60	5 / Not measured / Not measured	8	Not measured	

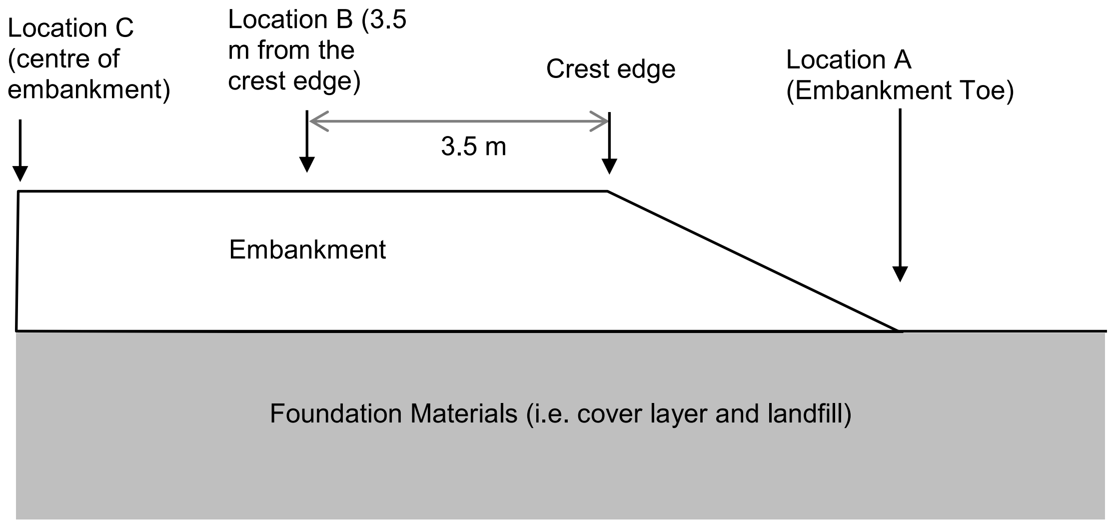
Notes:

(1) Lateral deformation at embankment toe is typically selected for comparison purposes as the maximum lateral deformation usually occurs at this location. This also allows a comparison against published literature.

(2) Settlement was measured with Hydrostatic Profile Gauge installed across the embankment while the lateral deformation was measured using a vertical inclinometer.

(3) Locations B and D have the same offsets to those of MB2S and MB2N from the crest edge of the embankment in MIRL project, respectively.

(4) Refer to Sketch 2 for the illustration and description of various locations except for location D.



Note: The above sketch is not drawn to the scale. Location D (i.e. 8.7 m away from the crest edge is not shown.

The comparison in Table 6 shows that ratio computed using SSC model built in the PLAXIS 2D is substantially greater than that measured in the field.

In literatures (Tavenas et al. 1980, Fatahi et al. 2013 and Grimstad et al. 2014), the ratio (of lateral deformation at the embankment toe to the settlement under the centre of embankment) measured in various field embankment is in the range of 6% to 9%.

PLAXIS 2D analyses have been carried out for the MIRL project at the following typical sections:

<u>MIRL PROJECT</u>

PLAXIS 2D analyses have been carried out for the MIRL project at the following typical sections:

- Ch 40,540
- Ch 40,640

The results are presented in Appendix P and used for assessment of the impact on the landfill capping layer as discussed in Section 5.4.4 which included the movements during construction. Post-construction, the computed lateral deformation at the rail centrelines are less than 30 mm in 40 years.

Based on the analyses for the Ballina Bypass project discussed above, a ratio of lateral deformation to post-construction settlement of 6% has been adopted for the MIRL project, which results in the reported values given in Table 9 (Section 5.4.1) with a maximum value of 31 mm in 40 years.

5.3.4. Assessment of Ballast Thickness due To Long Term Settlement

The assessed maximum post construction settlement in 40 years is in the order of 400 mm. If the rail level is to be adjusted by increasing the ballast thickness, the overall ballast thickness will exceed the maximum standard ballast thickness of 500 mm (as per the ARTC Heavy Haul Guidelines – Track, Civil and Structures).

Therefore, we have considered an intervention strategy which has the following considerations:

- At any given intervention, settlement is partially adjusted while maintaining change in grade in the transverse and longitudinal directions to be equal or less than 0.25 %; and
- Hence, over a period of 40 years, the profile of the top of the rail is allowed to settle while maintaining change in grade to equal or less than 0.25% and maintaining the profile of top of the rail above the lowest compliance rail levels (refer Tables I5 to I7 in Appendix I)

The above procedure, however, still result in a ballast thickness of greater than 500 mm in 40 years in localised areas. Therefore, we carry out a Discrete Element Modelling (DEM) and a Dynamic Deflection to compare the relative performance of thicker ballast in view of its lateral stability and dynamic deflection.

DEM

The DEM methods have been well developed and widely used for the assessment of ballast assembly (Lu and McDowell 2010). Unlike constitutive model in the Finite Element code, the contact model in DEM is a basic model that governs the particle interaction based on Newton's second law. Due to this, DEM is a reliable tool in assessing the behaviour of granular materials especially ballast under most types of loading.

In DEM, ballast materials are modelled as an assembly of random particulates which interact with each other dynamically (i.e. contact appears and disappears). Each of these particles is assigned a set of micromechanical parameters that govern how the materials will interact with each other. For cohesionless particles, a linear contact model with three basic micromechanical parameters has been appropriately assumed. Particle breakage has been considered by randomly introducing bonds at contacts of particles to allow for asperity breakage (Lu and McDowell 2008).

In this case, the DEM is particularly used to assess the relative deformation and movement of ballast assemblies with varying thicknesses under successive train loading. Two cases were considered and the details of the modelling are given in Appendix G (Part G1).

Dynamic Deflection

A dynamic deflection analysis is carried out to assess additional deflection of slightly thicker ballast layer than the maximum standard ballast thickness. Dynamic deflection analysis is required as the DEM does not typically model the overall formation including subgrade. Therefore, PLAXIS 2D is used to compare the deflection of standard ballast thickness (i.e. 500 mm) to thicker ballast of about 650 mm. The details of the dynamic deflection modelling using PLAXIS 2D are given in Appendix G (Part G2).

5.3.5. Embankment Stability

We have carried out assessment of the embankment stability for both short-term during surcharging (maximum embankment height) and long-term (lower embankment height but higher factor of safety requirement) at the following representative sections:

- Ch 40,440 River Side (1.75H:1V batter), Embankment Height = 13.5 m (short-term), = 13 m (long-term)
- Ch 40,640 Landfill Side (1.75H:1V batter), Embankment Height = 10.5 m (short-term), = 1.5 m (long-term)
- Ch 40,740 River Side (80^{o} Reinforced Soil Embankment) at end of surcharge, Embankment Height = 8 m (short-term), = 2 m (long-term)

Global stability using Limit Equilibrium Analyses (LEA) have been carried out using the commercially available computer program Slope/W. Three representative sections have been selected in the stability assessment. Train loading has been modelled in accordance to the Australian Standard AS5100.2 (2004). The following train loadings have been considered for the assessment:

- Four axles (30 T each) with a 36 T simulated locomotive as provided in AS5100.2 (2004).
- Train speed of 60 kmph.
- Wheel diameter of 0.95 m.

For the global stability assessment of embankment, a uniformly distributed load of 100 kPa has been adopted on each track (i.e. over sleeper width).

The LEA analyses were carried out to assess the embankment stability for the following cases:

- Short term loading due to surcharge and construction load with a minimum targeted factor of safety of 1.3
- Long term loading due to embankment and transient train loading with a minimum targeted factor of safety of 1.5
- Seismic and Rapid Drawdown cases with a minimum targeted factor of safety of 1.1

5.3.6. Landfill Capping Design

GWS requires that the new rail embankment be lined with suitable capping to limit seepage inflow into the existing landfill and proposed future landfill against the embankment batter on the landfill side so as to isolate leachate from the external environment and to limit leachate collection requirement at the base of the existing landfill.

The landfill capping design has been carried out based on the following document:

- NSW EPA Environmental Guidelines, Solid Waste Landfills, 2nd Edition 2016 (EPA Guidelines)

The design discussed in this report deals with the capping design only. Design and construction of the leachate collection system associated with the proposed future landfill against the rail embankment will be the responsibility of others.

In relation to the capping design, Section 1.1 of the EPA Guidelines requires the following:

- *"a compacted sub-base 200 millimetres thick to provide a firm, stable, smooth surface of high bearing strength on which to install the liner*
- *a compacted clay liner at least 1000 millimetres thick, with an in situ hydraulic conductivity of less than 1 x 10^{-9} metres/second; for landfills receiving more than 20,000 tonnes of waste per year, the liner should include a geomembrane over the compacted clay; the base liner should have gradients of greater than 1% longitudinally and 3% in transverse directions.*
- *To achieve the required in situ hydraulic conductivity of less than 1 x 10^{-9} metres/second, the clay should have high plasticity and a suitable particle-size distribution, with no particles greater than 50 millimetres in any dimension. Source testing of the material should confirm these properties.*
- *As an alternative to compacted clay, a geosynthetic clay liner may be used, provided it is used in composite with an overlying geomembrane liner.*

- *A protection or cushion geotextile should be used to protect geomembranes from damage by construction equipment and overlying materials".*

Coffey has assumed that the GWS landfill will receive more than 20,000 tonnes of waste per annum and therefore all of the above conditions will need to be met.

In the previous IFC-OD, Coffey has assessed that the existing capping layer over the old landfill area was sufficient to limit infiltration from rainfall based on permeability testing included in Appendix M of this report. However, with the intention of GWS to place future landfill over the existing landfill and against the batter of the rail embankment, the previous assessment no longer applies. Furthermore, due to the high settlement and potentially high differential settlement beneath the embankment batter anticipated during the preload period, the use of a compacted clay liner is unlikely to provide certainty on its performance should cracking occurs due to differential settlement. Therefore, we have adopted the use of a composite capping consisting of the following design for the capping layer:

- A bearing layer which could be the existing capping layer after stripping, smoothing and compaction. However, maximum particle size should be less than 13.2 mm with more than 90% passing 2.4 mm and more than 30% passing 75 microns. If the existing subgrade does not meet this requirement, an imported bedding clay layer satisfying the above having a compacted thickness of 300 mm should be provided.
- A GCL (either BENTOMAT® DN or engineer approved equivalent)
- Textured HDPE geomembrane (minimum 2 mm thickness)
- A cushion geotextile layer (Bidim A24 or equivalent) as a protective layer
- First layer of the embankment with 300 mm compacted thickness having a maximum particle size of 13.2 mm and 90% passing 2.4 mm.

The installation of GCL shall be carried as per manufacturer's guidelines.

- The GCL shall be placed in the transverse direction with adequate laps between two adjacent liners ensuring continuity of the liner during the life time of the embankment. If a lap in the transverse direction is required, care should be taken regarding the gradient of existing ground and the location of the lap to avoid slip between the GCLs.
- GCLs shall be installed so that an outward gradient is maintained in the transverse and longitudinal directions during the life time to drain water to collection points. Concaving of the GCLs will retain water. Water ponding for prolonged period is likely to result in infiltration of water through GCL.

Required quality control and assurance procedures should be in place during the installation of GCLs. Quality Control certificates shall be issued by the GCL manufacturer to the contractor, installer or project engineer, CQA inspector or other designated party for each delivery of material. A qualified Construction Quality Assurance (CQA) inspector shall confirm the installation are carried out as per manufacturer's guidelines and satisfies design and specification.

The impact on settlement and differential settlement during preloading and in the long-term on the landfill capping layer is discussed in Section 5.4.4 below.

5.4. Analyses and Results

5.4.1. Surcharge Requirements, Settlements and Lateral Deformations

The calculation sheets showing the details of settlement analyses are presented in Appendix D. In order to limit the maximum post construction differential settlement to 0.25 % change in grade (1 in 400), it is necessary to carry out the intervention to re-level and re-tamping of ballast. The assessed primary consolidation during embankment construction and waiting period, recommended surcharge thickness, and post-construction settlement at 0.5 year after track commissioning are summarised in Table 7. The contours of assessed primary and secondary settlements are presented as Figure 12 (primary and secondary settlements) and Figure 13 (secondary settlements only).

Table 7 – Summary of Analysis Results for Ch 40,440 – Ch 40,740 of MB2S

Chainage (MB2S)	Design embankment height (1)	Surcharge Thickness (m) (2)	Assessed Primary Consolidation Settlement (mm) (3)	Assessed Secondary Consolidation Settlement 0.25 yr after track commissioning (mm) (3)
40,440	9.5 to 14.4	0.1 to 0.2	6 to 115	up to 5
40,460	9.5 to 10.6	0.1 to 2	10 to 700	up to 15
40,480	8.9 to 10.4	0.5 to 3	180 to 890	5 to 20
40,500	6.8 to 9.1	0.5 to 4.5	410 to 960	10 to 20
40,520	4.8 to 7.2	2 to 6	460 to 985	10 to 25
40,540	3.6 to 3.7	3.7 to 6.5	615 to 1025	15 to 30
40,550	2.9 to 3.2	4 to 7	615 to 1040	15 to 30
40,560	2 to 2.5	9	925 to 990	20 to 25
40,580	1	9	955 to 980	20 to 25
40,600	1.5 to 1.6	9	990 to 1005	25
40,620	1 to 1.2	9	940 to 970	25
40,640	0.5 to 0.7	9	925 to 960	25
40,660	0.1	9	945 to 950	25 to 30
40,680	0.1	9	945 to 950	30
40,700	0.2	9	910 to 960	25 to 30
40,720	0.4	9	645 to 770	10 to 15
40,740	1.6 to 2	5	100	5

Notes:

(1) The fill heights (including capping and ballast thickness) vary across the alignment due to the variability in the existing ground levels and the embankment cross section.

(2) Surcharge thickness is additional to the design embankment height, and varies from the eastern crest edge to western crest edge of the embankment (see typical surcharge cross section shown on Drawing No. N01031-GRW-DRG-GEO-0005).

(3) Assessed settlement vary from the eastern crest edge to western crest edge of the embankment. Settlement values have been rounded to the nearest 5 mm.

The proposed interventions required to satisfy post construction differential settlements are 0.25, 0.5, 1.0, 2.1, 4.2, 9, 19, and 40 years after the track commissioning. These interventions periods do not account for regular maintenance works involving track reconditioning and ballast re-tamping. It is noted that a more frequent ballast re-tamping from 5 years after the track commissioning will be required for normal rail operation other than the requirements pertinent to the post construction differential settlement. The accumulated settlement values assessed at each intervention period for tracks between Ch 40,440 and Ch 40,740 are presented in Table 8 below.

Table 8 – Accumulated secondary settlements at each intervention period at various chainages

Chainage (MB2S)	Assessed accumulated Secondary Consolidation Settlement at various periods after track commissioning (mm) (centre of MB2N / at centre of MB2S)							
	0.25 yr	**0.5 yr**	**1.0 yr**	**2.1 yr**	**4.2 yr**	**9 yr**	**19 yr**	**40 yr**
40,440	6/15	6/21	6/25	7/30	7/38	9/49	13/63	21/84
40,460	6/49	6/68	6/81	7/96	7/120	9/150	13/183	21/220
40,480	15/73	21/101	25/120	30/143	38/178	48/222	60/268	77/319
40,500	29/87	40/121	47/144	56/171	70/213	88/265	108/319	133/377
40,520	35/95	48/133	58/158	69/188	86/234	107/290	130/348	157/410
40,540	58/117	81/162	96/193	115/230	143/286	177/354	213/424	251/497
40,560	81/109	105/155	121/186	140/223	170/280	205/348	243/419	282/492
40,580	88/112	113/159	129/191	148/229	178/286	214/356	252/428	292/503
40,600	88/107	112/153	129/184	148/221	178/278	214/346	251/417	291/491
40,620	86/104	110/150	126/181	145/219	175/275	211/344	249/415	289/489
40,640	90/111	114/158	131/190	151/228	181/286	217/356	256/429	296/504
40,660	102/119	127/167	144/199	164/239	195/298	233/370	273/444	314/521
40,680	102/120	127/168	144/201	165/240	196/299	233/371	273/445	315/523
40,700	Note (1)							
40,720								

Chainage (MB2S)	Assessed accumulated Secondary Consolidation Settlement at various periods after track commissioning (mm) (centre of MB2N / at centre of MB2S)							
	0.25 yr	0.5 yr	1.0 yr	2.1 yr	4.2 yr	9 yr	19 yr	40 yr
40,740								

Notes:

(1) Post construction settlements in the vicinity of Ch 40,700 to 40,740 (MB2S) due to secondary compression of non-waste fill materials are expected to be less than 50 mm / 40 years and landfill is not present in this area

The assessed maximum lateral deformations associated with the secondary settlement at the centre of MB2N and MB2S tracks are shown in Table 9 below. These are the assessed values during any intervention period (ref Table 8 for the intervention periods).

Table 9 – Summary of assessed lateral deformations at track levels during any intervention period

Chainage (MB2S)	Assessed Maximum Lateral Deformations associated with secondary settlement at track levels during any intervention period (mm)		Assessed maximum relative movement over 8m chord length	
	MB2N	MB2S	MB2N	MB2S
40,427	< 2	< 2	<2	2
40,460	1	13	1	2
40,480	5	19	1	5
40,500	8	23	2	8
40,520	9	25	3	9
40,540	15	30	4	10
40,560	17	30	6	12
40,580	18	30	7	12
40,600	17	29	7	12
40,620	17	29	7	12
40,640	18	30	7	12
40,660	19	31	7	12
40,680	19	31	8	13

Chainage (MB2S)	Assessed Maximum Lateral Deformations associated with secondary settlement at track levels during any intervention period (mm)		Assessed maximum relative movement over 8m chord length	
	MB2N	MB2S	MB2N	MB2S
40,700	<3	<3	<2	<2
40,720	<3	<3	<2	<2
40,740	<3	<3	<2	<2

The effect of above lateral movement values on the performance of the rail tracks should be checked by a qualified rail engineer against the requirements stipulated in relevant specifications.

5.4.2. Intervention Strategy

Intervention strategy has been developed to maintain the differential settlement criteria, ballast thickness requirement and the profile of top of rails. As described in Section 5.4.1, the proposed interventions required to satisfy post construction differential settlements are 0.5, 1.0, 2.1, 4.2, 9, 19, and 40 years after the track commissioning.

During the above interventions, the profile of the top of the rails is allowed to settle while maintaining change in grade to less than or equal to 0.25 % and maintaining the profile of top of the rail above the lowest compliance rail levels (ref. Tables I5 to I7 in Appendix I). Verification of the re-adjusted rail vertical alignment based on predicted post construction settlement has been carried out by a qualified rail engineer from Aurecon(refer Appendix I).

We have assessed the required level of the top of rails at any given intervention as presented in Figure I1 (Appendix I). The cumulative ballast thicknesses required at selected intervention periods (9 years, 19 years and 40 years) are presented in Tables I2 to I4 in Appendix I. Table 10 below presents the cumulative ballast thickness after proposed intervention at 40 years.

Table 10 – Summary of Assessed Ballast Thicknesses accumulated over 40 years

Chainage (MB2S)	Assessed Accumulated Ballast Thicknesses After 40 years (mm)	
	MB2N Centreline	MB2S Centreline
40,427	Note (2)	
40,445	374	261
40,460	343	437
40,480	331	504(1)
40,500	307	515(1)

Chainage (MB2S)	Assessed Accumulated Ballast Thicknesses After 40 years (mm)	
	MB2N Centreline	MB2S Centreline
40,520	258	512(1)
40,540	296	571(1)
40,550	280	568(1)
40,560	271	476
40,580	264	465
40,600	267	469
40,620	303	512(1)
40,640	339	546(1)
40,660	438	608(1)
40,680	343	437
40,700	379	289
40,720	393	291
40,740	Note (2)	

Notes:

(1) Thicknesses of Ballast exceeding the maximum standard thickness of 500 mm.

(2) Minimum or no ballast adjustment is anticipated here as no significant settlements are expected other than any ballast adjustment required as per routine maintenance works.

As shown in Table 10 above, the assessed cumulative thicknesses at some locations especially from Ch 40,500 to Ch 40,450 and from Ch 40,620 to Ch 40,660 (MB2S) are generally greater than 500 mm and up to about 610 mm. Therefore, assessment of impact on thicker ballast profiles have been carried out using DEM and FEM (i.e. Dynamic Deflection). Based on a Dynamic Deflection analysis using PLAXIS 2D (see Appendix G), the assessed deflection under train loading for 500 mm and 675mm thick ballast is 4.3 mm and 4.1 mm, respectively.

Degradation of ballast is likely due to the particle breakage. Research finding indicated that ballast breakage will not reduce the stiffness of the ballast, in fact tend to increase the stiffness. Indraratna et. al. (2008) presented that resilient modulus increases with particle breakage. Hence, no significant change in dynamic deflection is expected.

In addition, detailed DEM analysis has been carried out considering particle breakage. DEM simulations (see Appendix G) demonstrates that ballast assemblies of 500 mm to 675 mm thick do not undergo adverse vertical and lateral deformations under successive train loading. Therefore we consider that the presence of 610 mm thick ballast in a localised area is not expected to contribute to additional vertical and lateral movements and increase the likelihood of the instability.

5.4.3. Stability Analyses

We have considered three critical sections at Ch 40,440, Ch 40,620 and Ch 40,740 (MB2S) for the stability assessments:

Section at Ch 40,440: Non-reinforced permanent embankment is on landfill area. Maximum permanent embankment height is about 13 m and short-term surcharge height up to a total of 13.5 m.

Section at Ch 40,620: Non-reinforced embankment is on landfill area. Maximum permanent embankment height is about 1.5 m and short-term surcharge height up to a total of 10.5 m.

Section at Ch 40,740: Non-reinforced approach embankment in on general fill. Maximum permanent embankment height is about 2 m and short-term reinforced soil wall surcharge height up to a total of 8 m.

Analyses results indicated that the geogrid layers as presented in Table 12 are required to achieve the following minimum targeted factor of safety values:

- Long term factor of safety of 1.5;
- Short term factor of safety of 1.3; and
- Factor of safety under seismic loading and rapid drawdown condition of 1.1.

For reinforced soil wall at Ch 40,740, the geogrid layers are at 0.6 m vertical spacing. The geogrid configurations adopted in the analyses are shown in Table 12 below.

Table 11 – Geogrid Configurations adopted in the Analyses for Temporary Batter at Bridge Abutment

Section	Geogrid Requirement
Ch 40,740 (MB2S)	Geogrid vertical spacing at 0.6 m with a minimum ultimate tensile strength of 100 kN/m at 12% strain

Rapid drawdown stability analyses have been carried out at the above critical sections assuming the 100 year flood level of RL 11.5 m. As 100 year flood level will not overtop the existing embankment/Levy, wetting of the western batter of the main embankment can be neglected. However, conservatively a piezometric surface at RL 9 to 9.5 m within the embankment and the piezometric surface at the ground surface have been assumed.

The assessed factors of safety from the stability analyses are summarised in Table 12 below.

Table 12 – Summary of Stability Analyses

Station (MB2S)	Case	Description	Assessed Factor of Safety (FoS)	Target FoS
40,440	Short Term	Embankment Fill, Surcharge Fill and Construction Load	1.7	≥ 1.3
	Long Term	Embankment Fill and Equivalent Train Loading	1.6	≥ 1.5
	Rapid Drawdown	Rapid Drawdown and Equivalent Train Loading	1.1	≥ 1.1
	Seismic Loading	Seismic Loading and Equivalent Train Loading	1.3	≥ 1.1
40,620	Short Term	Embankment Fill and Construction surcharge	1.45	≥ 1.3
40,740	Short Term	Embankment Fill and Construction surcharge	1.52	≥ 1.3

The output of Slope/W analyses are presented in Appendix E.

5.4.4. Impact of Settlement and Differential Settlement on GCL

To assess the impact of the settlement and differential settlement on the GCL, we have performed FEA at two critical sections Ch 40+540 and Ch 40+640, based on the landfill layer thickness and embankment fill height including surcharge.

The analyses were carried out using the commercial software package PLAXIS using its in-built Soft Soil (SS) and Soft Soil Creep (SSC) models for the landfill. Updated mesh analysis was considered in order to account for the anticipated large deformations.

The liner system was assumed to be BENTOMAT® DN or equivalent with textured HDPE geomembrane. This GCL has a grab tensile strength of 8.8 kN/m. The tensile strength of geosynthetic reinforcement is typically reached at 10% strain (based on product datasheets), and we expect that filter fabric layers and GCL to have a strain at failure of greater than 10%. The HDPE geomembrane has larger elongation capacity compared to GCL liner, and therefore, we have conservatively modelled the liner system as a single geotextile element with EA = 88 kN/m for our analysis.

We note that the manufacturer of BENTOMAT® DN states that this GCL has high shear resistance and is suitable for installation on slopes greater than 1V:1.5H. Although the existing ground slope is much shallower, we recommend that a similar GCL be adopted due to relatively high short-term settlement under the preload and long-term creep settlement of the landfill.

The analysis considered embankment construction to occur in one month period to the surcharge level. Two months preload waiting period was allowed before the surcharge is trimmed back to final proposed design level. The filling adjacent to the proposed embankment was considered to occur immediately after the surcharge removal.

Results of the FEA on the capping performance (assessed maximum tensile forces and strains along the GCL) are provided in Appendix P.

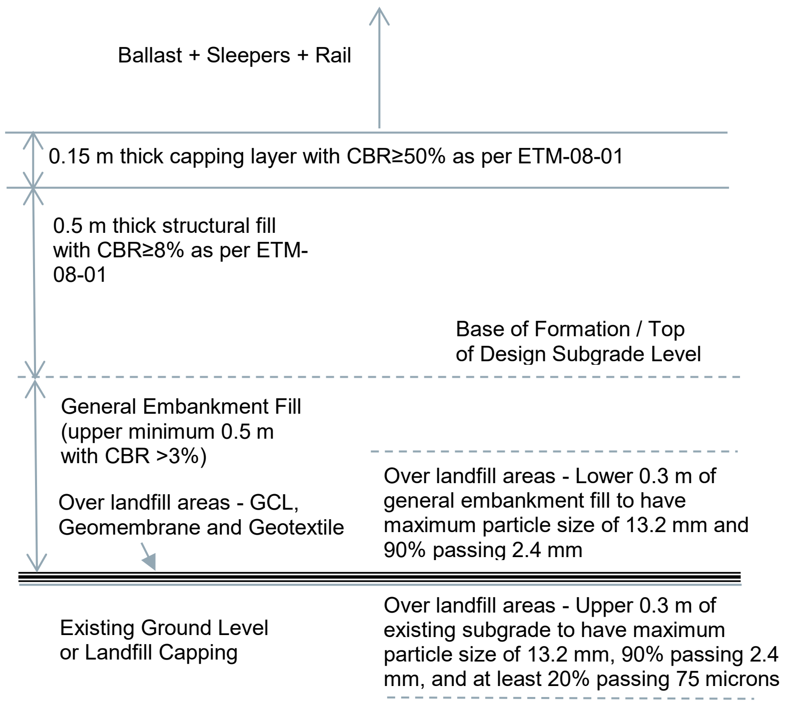
As can be seen in the plots in Appendix P, the assessed maximum load and strain in the GCL at Ch 40+540 is approximately 1.2 kN/m and 1.4 %, respectively. The computed maximum tensile load and strain are much lower than the quoted strength of the GLC and the expected strain at failure (i.e. > 10%). Furthermore, the computed maximum tensile strain is much lower than the usual limit of 5% to 6% strain recommended under operating conditions for geosynthetics. Therefore, the proposed GCL liner using BENTOMAT® DN or equivalent is considered to be suitable for the purpose.

5.5. Construction Staging

5.5.1. General Formation Requirements

The formation requirements shall be in accordance with the ARTC standard ETM-08-01 Earthworks, Formation and Capping Material for the following subgrade CBR values assumed in the design:

As the embankment is constructed using general embankment material (as per R44) or select fill material (as per R67) a minimum CBR of 3% can be adopted for the embankment fill. Based on the ARTC standard ETM-08-01 the formation configuration provided in Sketch 3 should be adopted.



Sketch 3 – Formation Requirements

Adopting a ballast thickness of 300 mm, the minimum formation thickness above a subgrade with CBR > 3% is 0.95 m according to Sketch 3. An additional thickness of 0.5 m of subgrade fill having a CBR > 3% is required below the formation level to isolate any existing landfill that is likely to have lower CBR.

5.5.2. Construction Sequence from Ch 40,440 to Ch 40,640

From Ch 40,440 to Ch 40,640, the design ground surface level (i.e. top of ballast) is at least 1 m above the current ground level and landfill is present from about Ch 40,445. Therefore, provided the embankment fill or the existing capping layer over the landfill have a minimum thickness of 0.5 m and a CBR > 3%, the formation within this section may be constructed following removal of the surcharge to the base of the formation layers (i.e. 0.95 m below top of ballast) after the preload period.

Within this section, we recommend that the following embankment construction sequence:

Step 1. Strip topsoil and vegetation and stockpile for reuse for landscaping purposes.

Step 2. Check that the existing surface layer meets the particle size requirement of the GCL cushioning layer (see Sketch 3 above). If not, over-excavate to a depth of 300 mm and replace with compacted GCL cushioning material.

Step 3. Proof roll the exposed surface by a minimum 8 passes of a vibratory roller having a static weight of at least 10 tonnes.

Step 4. Localised areas that appear wet, spongy or heave excessively (i.e. more than 20 mm visually) shall be over-excavated to 300 mm and backfilled with compacted material (note: where imported GCL cushioning layer is to be placed, the backfill material beneath the GCL cushioning layer may comprise crushed sandstone or similar granular material, but where the existing subgrade is to be used as the GCL cushioning layer, the backfill material shall meet the requirement of the GCL cushioning material).

Step 5. Where required, place and compact the imported 300 mm GCL cushioning layer to a minimum Dry Density Ratio (DDR) of 98% Standard at Optimum Moisture Content (OMC) ± 2%. This compaction requirement also applies to existing subgrade material satisfying the GCL cushioning layer grading.

Step 6. Place the GCL, geomembrane and cushioning geotextile in the following order:

- GCL (either BENTOMAT® DN or engineer approved equivalent)
- Textured HDPE geomembrane (minimum 2 mm thickness)
- Cushion geotextile layer (Bidim A24 or equivalent) as a protective layer

The installation of the GCL, Geomembrane and Geotextile shall be carried as per manufacturer's guidelines.

- The GCL shall be placed in the transverse direction with adequate laps between two adjacent liners ensuring continuity of the liner during the life time of the embankment. If a lap in the transverse direction is required, care should be taken regarding the gradient of existing ground and the location of the lap to avoid slip between the GCLs.
- The river side of the existing GCL with the existing landfill shall be exposed and the new GCL lap over it for a minimum distance of 2 m towards the river.
- On the landfill side, the new GCL shall be placed beyond the toe of the surcharge and at least 3 m past the toe of the final embankment (i.e. after removal of the surcharge), whichever is the greater.
- The Geomembrane shall be placed over the GCL (to the same lateral extent), with all joints field welded according to the manufacturer's specification.

- The cushioning geotextile shall be placed over the Geomembrane (to the same lateral extent) with a minimum overlap of 300 mm between joints.
- Provide temporary soil cover of at least 0.5 m thickness over the GCL, Geomembrane and Geotextile that extends beyond the toes of the surcharge.

Step 7. Place the first layer of general embankment material with 300 mm compacted thickness having a maximum particle size of 13.2 mm and 90% passing 2.4 mm above the landfill capping layer (see Sketch 3 above), to a minimum DDR of 95% Standard at OMC ± 2%. All general embankment fill should have a soaked CBR > 3% (i.e. minimum 4%).

Step 8. Continue placing and compacting general fill in layers, including surcharge fill until the design surcharge level is reached. Settlement of the landfill during embankment/surcharge construction may result in more surcharge being placed than required for design, depending on the rate of settlement achieved. Subject to settlement monitoring and review, termination of surcharge construction may be feasible prior to reaching the design surcharge level.

Step 9. After primary consolidation is complete and sufficient data on secondary consolidation gathered from the monitoring, the surcharge may be removed to the design subgrade level (i.e. base of formation).

Step 10. Construct the formation in accordance with ARTC standard ETM-08-01 Earthworks.

5.5.3. Construction Sequence between Ch 40,640 and Ch 40,705

Between Ch 40,640 and Ch 40,705 the design height is less than 1 m, with the end of the landfill at about Ch 40,700 and we recommend that the landfill capping be extended to Ch 40,705. In some areas, the design height is close to existing ground level. In this situation, pre-excavation and replacement with suitable subgrade material prior to installation of the landfill capping layer will be required to enable the required formation layer to be constructed. The amount of pre-excavation will depend on the settlement anticipated during preloading and uncertainties associated with the settlement prediction.

The predicted primary settlement under the surcharge, however, ranges from 645 mm (less than this where landfill does not exist) to 960 mm. Allowing for uncertainties in settlement prediction, we recommend that 0.6 m of pre-excavation be adopted in the design, with the following construction sequence between Ch 40,640 and Ch 40,705:

Step 1. Excavate to a depth of 0.6 m below existing ground level and stockpile the material for reuse for landscaping purposes.

Step 2. Proof roll the exposed surface by a minimum 8 passes of a vibratory roller having a static weight of at least 10 tonnes.

Step 3. Localised areas that appears wet, spongy or heave excessively (i.e. more than 20 mm visually) shall be over-excavated a further 300 mm and backfilled with compacted granular material (e.g. crushed sandstone).

Step 4. If landfill is found near the excavated surface and causing difficulties in the proof rolling, placement of a heavy geogrid (e.g. bi-directional 60 kN geogrid) may be required to assist proof rolling and compaction of the overlying layers.

Step 5. Place and compact a 300 mm thick layer that satisfies the GCL cushioning layer grading requirements shown in Sketch 3 above as well as ARTC standard ETM-08-01 Earthworks for Structural Fill (i.e. between 20% and 30% passing 75 microns and maximum particle size of 13.2 mm with a soaked CBR of at least 8%). If necessary, use lime stabilisation to achieve this minimum CBR. It is intended that this layer would form the lower zone of the Structural Fill which forms part of the Formation. Compaction of this layer shall meet the requirement of ARTC standard ETM-08-01 Earthworks for Structural Fill.

Step 6. Install the landfill capping layer (i.e. GCL, Geomembrane and Geotextile) as in Step 6 described in Section 5.5.2 above.

Step 7. Place and compact a 300 mm thick layer of material over the GCL that satisfy the grading requirements of the lower 0.3 m general fill layer shown in Sketch 3, and in addition, the material used is to have a minimum soaked CBR of 8% and satisfying the requirements of ARTC standard ETM-08-01 Earthworks for Structural Fill. For example, a fine to coarse grained sand or crusher dust layer would satisfy this requirement. This layer would form the upper zone of the Structural Fill of the Formation layer, giving a total thickness of 600 mm of Structural Fill, allowing for a possibility of 100 mm of Structural Fill to be trimmed off if required depending on the actual settlement achieved during preloading (see Step 12). Compaction of this layer shall meet the requirement of ARTC standard ETM-08-01 Earthworks for Structural Fill.

Step 8. Place and compact general fill surcharge (to a minimum DDR of 95% Standard) over the Structural Fill until the design surcharge level is reached. Settlement of the landfill during embankment/surcharge construction may result in more surcharge being placed than required for design, depending on the rate of settlement achieved. Subject to settlement monitoring and review, termination of surcharge construction may be feasible prior to reaching the design surcharge level.

Step 9. After primary consolidation is complete and sufficient data on secondary consolidation gathered from the monitoring, the surcharge may be removed to the top of the Structural Fill already placed during Step 7. Note that for the above construction sequence, a settlement of 450 mm would result in the top of the Structural Fill being at 450 mm below the original ground level which would allow 150 mm of capping and 300 mm of ballast to be placed for a design surface level equivalent to the existing ground level. This could also be achieved if the actual settlement is only 350 mm, by trimming off 100 mm of Structural Fill following the settlement period.

Step 10. Construct the formation in accordance with ARTC standard ETM-08-01 Earthworks.

5.5.4. Construction Sequence from Ch 40,705 to Bridge Approach

Based on the geotechnical investigation results, landfill does not exist from approximately Ch 40,700 onwards. However, significant thickness of uncontrolled fill exist. The embankment height in this section ranges from 0 m to about 2 m above existing ground level.

To mitigate the risk of post-construction settlement of the uncontrolled fill, surcharging has been proposed up to the bridge abutment at Ch 40,740. Furthermore, due to the low embankment height, partial removal of the uncontrolled fill and replacement with engineered fill is required to accommodate the rail formation layers, provide a stable formation for the temporary reinforced soil batter at the end of bridge approach, and to control differential settlement behind the bridge abutment. The recommended construction sequence in this section is as follows:

Step 1. Strip topsoil and vegetation and stockpile for reuse for landscaping purposes.

Step 2. Excavate the existing fill to the following depths below existing ground level, and stockpile clean fill for reuse as general embankment fill:

- Ch 40,705 to Ch 40,731.5 1 m
- Ch 40,731.5 to Ch 40,747.23 2.5 m
- From Ch 40,747.23 2.5 m to 0 m to intersect existing ground

Step 3. Place and compact general fill in layers (not more than 300 mm for each layer) to a minimum Dry Density Ratio of 95% at OMC ± 2% until 300 mm below the subgrade level.

Step 4. Place and compact the final 300 mm layer of general fill to a minimum DDR of 98% at OMC ± 2%.

Step 5. Place and compact surcharge fill in layers to a minimum DDR of 95%.

Note: In Steps 3 to 5, fill construction will need to incorporate the construction of the geogrid reinforced soil batter at the bridge approach.

Step 6. After primary consolidation is complete and sufficient data on secondary consolidation gathered from the monitoring, the surcharge may be removed to the design subgrade level (i.e. base of formation).

Step 7. Construct the formation in accordance with ARTC standard ETM-08-01 Earthworks.

5.6. Sensitivity Analyses

5.6.1. Compressibility Parameters

Due to variability and uncertainties in landfill design parameters, sensitivity analyses were carried out during the pre-construction ground treatment design in the absence of settlement monitoring data. The range of parameters used in the sensitivity analyses on the compressibility analyses as previously outlined in Section 4.4.2 are shown in Table 13 below.

Table 13 – Cases and parameters considered in the Sensitivity Analyses for Compressibility Parameters

Case	Dry Unit Weight (kN/m^3)	Compression Ratio (CR) for unimproved landfill	Secondary Compression Ratio			Remarks
			Unimproved Landfill ($C_{\alpha\varepsilon}$)	β value	Treated Landfill ($C^*_{\alpha\varepsilon}$) (new/old landfill)[1]	
Design Case Scenario 1	11.5	0.15	0.02	0.07	0.015 (0.01)	Compressibility parameters considered in current design

<table>
<tr><th rowspan="2">Case</th><th rowspan="2">Dry Unit Weight (kN/m^3)</th><th rowspan="2">Compression Ratio (CR) for unimproved landfill</th><th colspan="3">Secondary Compression Ratio</th><th rowspan="2">Remarks</th></tr>
<tr><th>Unimproved Landfill ($C_{\alpha\varepsilon}$)</th><th>β value</th><th>Treated Landfill ($C^*_{\alpha\varepsilon}$) (new/old landfill)[1]</th></tr>
<tr><td>Scenario 2</td><td>11.5</td><td>0.075</td><td>0.02</td><td rowspan="2">0.07</td><td rowspan="3">0.015 (0.01)</td><td rowspan="2">Compression ratio (CR) is varied</td></tr>
<tr><td>Scenario 3</td><td>11.5</td><td>0.25</td><td>0.02</td></tr>
<tr><td>Scenario 4</td><td>11.5</td><td>0.15</td><td>0.02</td><td>0.05</td><td>Lower β value, hence, less improvement in landfill due to surcharging</td></tr>
<tr><td>Scenario 5</td><td>10.6</td><td>0.16</td><td>0.0225</td><td>0.07</td><td>0.0175 (0.0125)</td><td rowspan="2">Higher secondary compression ratio, hence higher settlement</td></tr>
<tr><td>Scenario 6</td><td>9.8</td><td>0.175</td><td>0.025</td><td>0.07</td><td>0.020 (0.015)</td></tr>
</table>

Note :

1. Value outside the bracket indicates the assumed minimum creep strain rate that can be achieved in the transition zone while the value in bracket indicates the assumed creep strain rate that can be achieved due to the dynamic compaction in the older landfill area

The variations in compressibility parameters considered above (in Table 13) influence the assessed primary settlements and post construction settlement. The results of sensitivity analyses are summarised in Table 14 below.

Table 14 – Results of Sensitivity Analyses on Compressibility Parameters

Case	Number of Interventions (1) within 40 years after track commissioning
Design Case (Scenario 1)	8
Scenario 2	9
Scenario 3	8
Scenario 4	9

Case	Number of Interventions [1] within 40 years after track commissioning
Scenario 5	11
Scenario 6	12

Note:

1. More settlement would result in additional interventions (in addition to the design case) and hence increase in the ballast thickness.

The results indicate either the variation in the intervention number or surcharge thickness. The combined effects on both intervention number and surcharge thickness are not assessed as a part of sensitivity analyses. Alternative 2 (i.e. use of additional surcharging) is proposed to adopt as contingency measure to satisfy the settlement criteria (i.e. both total settlement and differential settlement).

The settlement monitoring during the preload period information can be used to refine the above assessment and hence refine the additional surcharge thickness (if required).

5.6.2. Landfill Profile

The side batters of the landfill in the Transition Zone between Ch 40,440 and Ch 40,560, and at the end of the landfill at Ch 40,700 approximately have been taken into account in the surcharge design. Variations of the actual batter profiles to the adopted design profiles will invariably cause different settlement and differential settlement behaviour. Such variations and their impact on settlement are best monitored using proposed HPGs as recommended in Section 5.3.1.3. The monitoring results would be used to adjust surcharge thickness and/or duration of the surcharge as required.

6. Design Integration

6.1. Structures

Along the corridor of subject area (Ch 40,427 – Ch 40,740 of MB2S), no existing structures are present. The foundation design for signalling huts and gantry are to be designed under separate cover by others.

The proposed north-western abutment of Georges River crossing is situated less than 20 m to the southeast of the landfill boundary and no landfill is present below the proposed abutment. The abutment detail proposed by Aurecon as shown in Appendix K (Aurecon Drawing No. N01031-GRW-DRG-BRD-0160-01). However, we have extended the surcharge to Ch 40,740 to treat the uncontrolled fill (albeit non-waste), and a temporary reinforced soil batter at 80^{o} from the horizontal is required to enable the abutment piles to be installed during the preload period.

To mitigate lateral movement to the abutment bridge piles induced by the preloading, we recommend CPB installs temporary enlarged steel casing socketed 2 m below the uncontrolled fill (i.e. toe at about 10 m deep below existing ground level) with an offset gap of 200 mm from the pile face on the landfill side. After lateral movement has ceased, the gap should be grouted with cement grout.

The proposed level crossing at Ch 40,660 will be subjected to post construction settlement. Impact of post construction settlement on this level crossing has been considered by the designers (Aurecon) and included in the earthworks package of IFC submission.

6.2. Alignment Profile

We understand that the alignment design takes into account a number of considerations. As part of intervention strategy, the intervention requirements have been refined to maintain a maximum ballast thickness of 500 mm in 40 years by considering the differential settlement criteria and the lowest compliant level of the top of rail.

6.3. Drainage

The requirement of subsurface drainage beneath the railway tracks has been developed by Aurecon incorporating the post construction settlements provided by Coffey. The settlement contours developed based on the assessed primary and secondary settlement within the landfill area are presented in Figures 12 and 13. Cross section of embankment presenting the change in cross fall of capping layer due to long term settlement (i.e. post construction settlement after 40 years) are presented in Appendix J. As presented in Appendix J, initial cross fall of capping layer has been designed to maintain minimum required cross fall considering the anticipated long term settlement. After anticipated long term settlement of over 400 mm in 40 years, cross fall of capping layer is maintained at 2.6 % or better. Cross sections presented in Appendix J shows the design cross fall and cross fall after 40 years.

Also presented in Appendix J is settlement at swale drainage. Performance of the swale drains due to predicted post construction settlements has been checked by the drainage engineers (Aurecon).

We also recommend that the drainage engineers checks the impact of dishing of the landfill capping layer beneath the embankment crest during preloading as discussed in the following section.

6.4. Landfill Capping Layer

The impact of settlement on the landfill capping layer during preloading and in the long-term has been discussed in Section 5.4.4 and the results presented in Appendix P. A suitable landfill capping comprising a composite GCL, Geomembrane and Geotextile has been recommended to provide the required function of a low permeability layer over the design life of the project.

We note that because of settlement induced by the surcharge, and because the thickness of the landfill generally increase towards the pit, there will be a tendency for dishing of the landfill capping beneath the crest of the surcharge on the landfill side. As such, some water ponding is likely to result where the longitudinal gradient is insufficient to drain the ponded water away. The ponded water will result in some infiltration through the capping layer but the amount and rate of infiltration is expected to be very low and will not be contaminated by landfill leachate. Prior to GWS placing more landfill on the batter of the rail embankment, a new capping layer will be constructed over the batter of the embankment and on the existing ground level on the landfill side, together with a leachate collection system to control seepage into the existing landfill.

Dishing of the landfill capping layer beneath the crest of the embankment as a result of the preloading is expected as shown in Appendix P. As discussed in Section 5.4.4, ponding of water due to this dishing effect is not expected to cause significant infiltration of rainfall into the existing landfill. After construction of the rail embankment, a new capping layer will be placed over the batter of the embankment on the landfill side and integrated with the landfill capping over the existing landfill ground level prior to placement of further landfill by GWS against the batter of the embankment. Nevertheless, we recommend that the drainage designer checks the acceptability of the expected dishing and the potential for minor ponding of water over the landfill capping layer.

6.5. Impact of Settlement on Cross Fall of Rail Formation Capping Layer

Initial cross fall of the rail formation capping layer has been designed by Aurecon to maintain required minimum cross fall over 40 year period. After anticipated long term settlement of over 400mm, cross fall of capping layer is maintained at 2.6% or more. Cross sections supplied by Aurecon and presented in Appendix J show the design cross fall and cross fall after 40 years.

6.6. Utilities

It is understood that a permanent underground CSR conduit is proposed to run through the landfill area. We understand that an assessment of the impact of landfill settlement on the conduit is carried out by Aurecon.

No other proposed or existing utilities within the transition and DC zones (Ch 40,427 – Ch 40,740 of MB2S) are identified.

7. Environmental Considerations

7.1. Environmental Impact Statement (EIS)

An EIS document has been prepared by Hyder Consulting (now Arcadis) dated May 2015. The key environmental issue which is relevant to the geotechnical design is subject to the assumption made in the EIS report as quoted below:

"*Glenfield Waste Facility: It is not intended that the Rail link would disturb or compromise the integrity of the lining or barrier systems that currently exist within the Glenfield Waste Facility*"

In addition to the above, the EIS has made various references to the assumption that the proposed MIRL is not expected to damage any landfill liner and barrier system such that these would lose their functionalities.

The ground treatment options adopted in this report have been chosen to avoid disturbance on the landfill liner and barrier system. We note that the DC treatment option with a preceding DC trial provides an opportunity to assess the variability of ground conditions and flexibility in adjusting the improvement effect during full scale implementation. It is expected that the effective densification of landfill material with DC will terminate at an elevation reasonably (about 4 m) above the base of the landfill. In order words, the significant stress increases within the landfill that are induced by DC are not expected to impact on the landfill liner.

Considering compressible nature of the landfill material, settlement and lateral movement of landfill material at the liner area (i.e. at the stiff base) are minimal. In general, settlement and lateral movement are higher at the top of the landfill and cease at the base (2D PLAXIS analysis results indicated similar behaviour as expected). As minimal movement is anticipated at the base of the landfill, no damage of landfill liner is expected.

7.2. Landfill capping layer

Landfill capping design and construction beneath the embankment has been addressed in Section 5.4.4, 5.5 and 6.4. We recommend that a similar capping be installed on the landfill side of the embankment batter prior to GWS placing additional landfill over the embankment batter. The construction of this new landfill layer and joining to the existing landfill capping will be carried out by others in the future.

7.3. Landfill Gas

It is expected that landfill gas will not be adversely impacted by the operation of MIRL (refer to GEOTLCOV24072AF-AU – Second Draft of Remedial Action Plan dated 8 August 2016 and GEOTLCOV24072AF-AQ – Third Draft of Land Contamination Status Report dated 12 July 2016).

8. Consideration on Project Specific Procedures

With a reference to the Project Specific Procedure Rev. E (dated 9 Feb 2017) issued by CPB, we consider that the geotechnical ground treatment design is consistent with the Management, Controls and Mitigation Measures adopted in the report with respect to the following items:

- The design has considered appropriate specifications (refer Section 3.2);
- Design considered the construction within GWS landfill area. Which include, identification of location of rail embankment in relation to landfill cells. Hence, landfill thickness, extent and its variability in thickness (i.e. location of tip batter) have been considered in the design;
- Design considered the reduction/elimination of any impact on existing environment. Selected ground treatment eliminates excavation of waste material. Existing cover will be in place or where it removed partially, will be covered with working platform and embankment material.
- Selected construction materials are as per the relevant standards; and
- It is expected that no ground treatment work will extend beyond the Project Works Areas into the “No-go” zones specified in the aforementioned report.

We understand that a permanent underground CSR conduit is proposed to run through the landfill area. It is understood that an assessment of the impact of landfill settlement on the conduit is carried out by Aurecon.

9. Operations and Maintenance

The design intends to minimise maintenance requirements associated with intervention periods and achieve the design life and design criteria stipulated in Section 3.1 through careful design. A key focus of the project is to provide a rail connection that does not adversely impact on ARTC freight operations and ensure the rail link and its components can be maintained effectively, safely and with consideration of minimum whole of life costs.

9.1. Intervention: Re-levelling and Re-tamping

We understand that the Operator of MIRL (SIMTA) agrees that the proposed rail formation is a non-standard design and appropriate maintenance such as re-levelling and re-tamping of ballast will be carried out at proposed intervention periods to meet the operational criteria including differential settlement during the design life of the rail track.

Initial two interventions will be carried out by CPB within the Defect Liability Period of one year, while the remaining interventions will be carried out by the Operator of MIRL.

As part of the interventions as well as routine track maintenance purpose, we recommend that survey of rail tracks will be carried out and hence monitor the post construction settlement. In addition, settlement plates and survey markers installed as part of construction stage monitoring can be used to monitor post construction settlement. HPGs are also proposed for the monitoring of post construction settlement.

Predicted post construction settlement and intervention strategy provided in this report can be used for the assessment of re-levelling and re-tamping requirements. Typical settlement trigger levels (i.e. settlement at which next intervention to be performed) and adjustment levels as per the predicted post construction settlements are presented in Table I1 and Table I2 respectively in Appendix I.

Instrumentation and Monitoring specification outlining instrument specification requirements, installation requirements, monitoring process control plan and monitoring frequency (during construction and post construction) has been developed and issued with the previous IFC documentation. The instrumentation and monitoring plan will need to be updated for this revised Final Design, and will be re-issued following discussions with CPB on its preferred choice of settlement monitoring devises.

9.2. Embankment Durability and Design Life

Long term performance of the rail embankment has been assessed considering stability and deformation. Long term stability of rail embankment has been assessed including high groundwater condition and earthquake loadings. Area where embankment is reinforced with geogrid and facing element, appropriate strength reduction factors for geogrid reinforcement have been adopted in assessing the long term stability of the rail embankment during its design life. Long term deformation of the rail embankment is considered and hence, proposed intervention periods to meet the operational criteria including differential settlement during the design life of the rail track. Long term performance of rail embankment ensure that SIMTA freight operation will not be adversely impacted and the rail link and its components can be maintained effectively and safely.

Design life of the earthwork foundation is ensured by following the Track & Civil standard reference in Section 4 (Technical Maintenance Plan) of ARTC Standard ETE-00-03. Intervention periods, Settlement trigger levels and track adjustment levels as detailed in Section 9.1 and Appendix I, should be incorporated in the Operation and Maintenance Manual for the project.

Terramesh or equivalent flexible facing is proposed for 0.6H: 1V reinforced earth embankment batter. The mesh forming the Terramesh or equivalent unit is provided with polymer-coated galvanized alloy steel (Refer manufacturer’s data sheet in Appendix O) having working life of about 120 years. A crushed rock fill cover to fill front of the mesh provides protection (Refer typical details in Appendix O) against soil erosion.

Monitoring requirements are outlined in the Instrumentation and Monitoring Specification to assess and monitor the rail tracks and rail embankment performance during defect liability period and after practical completion of the project.

9.3. Maintenance

Maintenance procedure is detailed in Section 9.1 of Track Design Report: N01031-DRP-TGM-0001(01). Additional maintenance requirements for the rail track over landfill area are detailed here.

9.3.1. Inspection frequency

Minimum applicable inspection frequencies are provided in Section4 (Technical Maintenance Plan) of ARTC Track & Civil Standard ETE-00-03. In addition, inspection should be carried out at proposed re-levelling and re-tamping intervention periods provided in Table I1 and Table I2 in Appendix I.

Furthermore, following visual observations and survey frequencies are proposed (refer Instrumentation and Monitoring Specification: N01031-GRW-GEO-SPE-0001-04) as a measure to mitigate risk associated with rail twist at the commencement of rail operation:

- After first two trains.
- Twice weekly for two weeks.
- Once a week for next two weeks.
- Once a month until practical completion.
- Once a month from practical completion which can be incorporated in the ARTC standard Technical Maintenance Plan.

9.3.2. Inspection Method

ARTC Track & Civil standard ETE-00-02 details the requirement for Patrol, General and Detailed inspection to be performed on the various infrastructure elements. Track patrol inspections should keep a lookout for obvious, abnormal rail defects or conditions as listed in Section 3.7 of ETE-00-02, specifically, Sections 3.7.7 (Ballast), 3.7.8 (Track Geometry), 3.7.9 (Track lateral stability) 3.7.11 (Earthworks) and 3.7.16 (Level Crossings).

9.3.3. Competence of staff engaged on track inspection

Persons carrying out track inspection shall hold the required competencies stipulated in Section 2.3 of ETC-00-02. In addition, experienced geotechnical engineer should be engaged to assess the settlement performance (based on the observed settlement) of the rail embankment at each intervention period or at settlement trigger levels and hence refine future intervention periods.

10. Constructability

Design leaders from Coffey/Aurecon/CPB design and construction team attended weekly meeting discuss design integration and constructability issues.

The latest design drawings have been presented to members from the CPB Contractors project team with appropriate construction experience. CPB and the design team reviewed the drawing packages to ensure that the designs are constructible and to explore areas for improvement. The comments were recorded on the drawings for incorporation into the design documentation.

In addition, the design packages are reviewed by the construction team prior to each of the key milestone dates.

10.1. Key Constructability issues

The following key constructability issue have been identified:

- Preparation of existing ground surface in newer landfill area where GSL liner is exposed; and
- Preparation of benching into existing batter.

Above will be addressed in CPB work packs which will be developed prior to the commencement of major construction works.

11. Safety in Design

The safety in design has been considered where appropriate as summarised below.

- For the stability analysis, appropriate factors of safety as per industry accepted standards have been adopted in the design. Where batters steeper than 1V:1.75H, batter has been reinforced.
- Observational approach has been considered during the design to reduce the risk of embankment instability during construction. Hence, construction sequence has been developed by taking into consideration of safety during construction (Refer Drawing Nos. N01031-GRW-DRG-GEO-0003-02 and N01031-GRW-DRG-GEO-0004-02).
- Intervention periods for re-levelling and re-tamping have been considered in the design to maintain safe operation of the rails.

12. References

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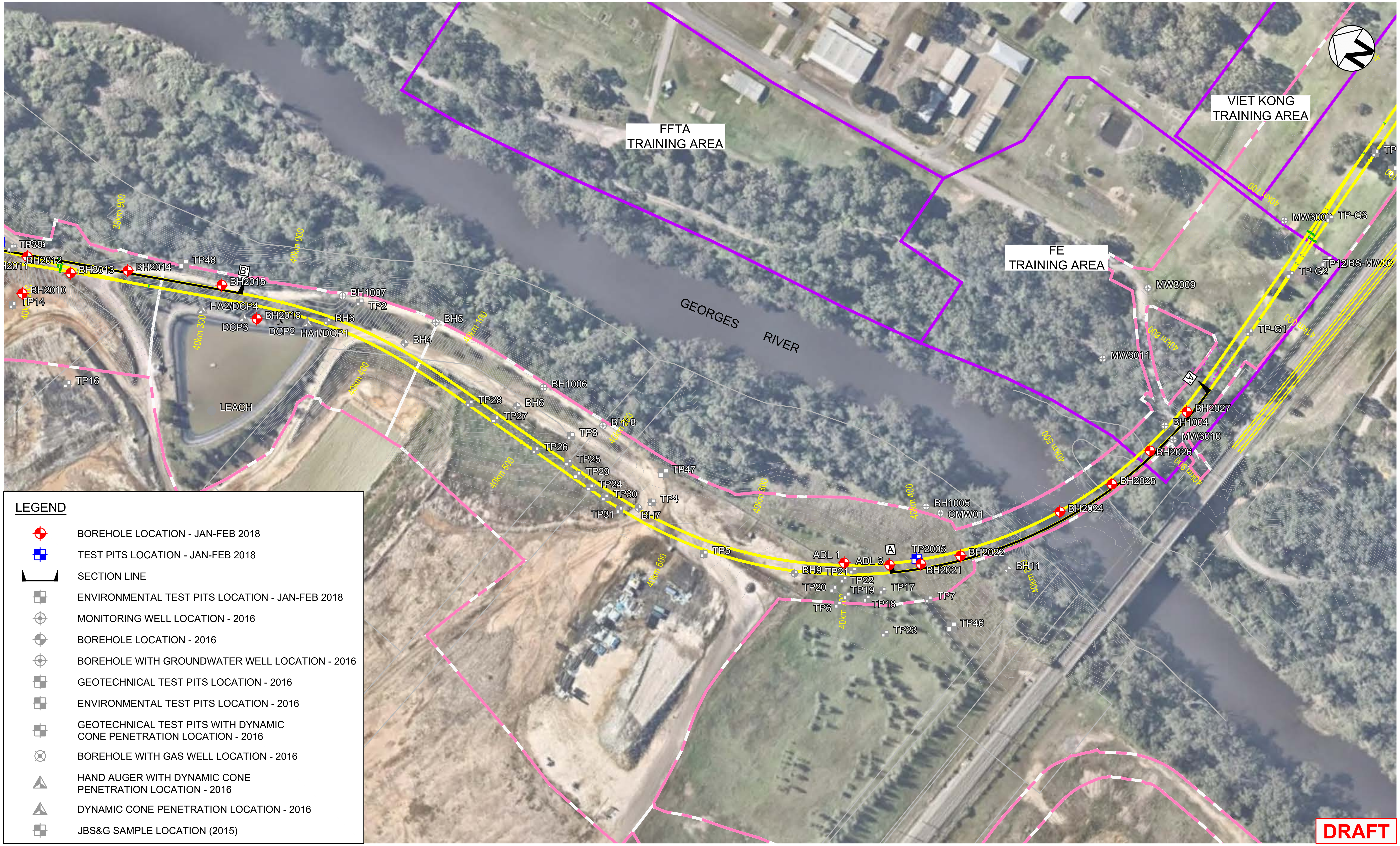

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approved	-
date	9 / 05 / 18
scale	AS SHOWN
original size	A3

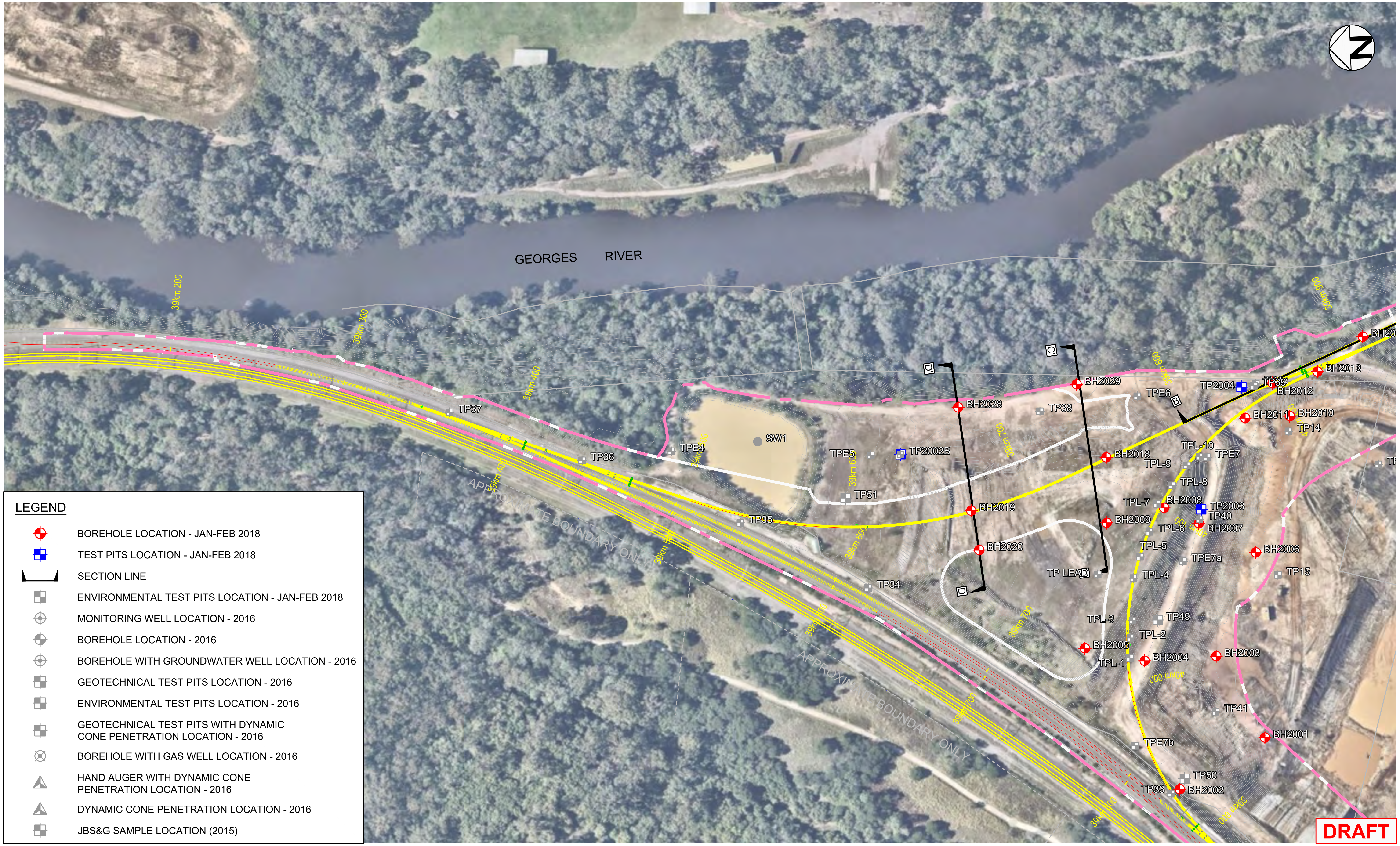
coffey
A TETRA TECH COMPANY

client: CPB CONTRACTORS PTY LTD

project: MOOREBANK INTERMODAL RAIL LINK
RALP NO.1
MOOREBANK, NSW

title: GEOTECHNICAL & CONTAMINATION
INVESTIGATION LOCATION PLAN - SHEET 1 OF 2

project no: 754-GEOTLCOV24072AG | figure no: FIGURE 6 | rev: A



LEGEND

- BOREHOLE LOCATION - JAN-FEB 2018
- TEST PITS LOCATION - JAN-FEB 2018
- SECTION LINE
- ENVIRONMENTAL TEST PITS LOCATION - JAN-FEB 2018
- MONITORING WELL LOCATION - 2016
- BOREHOLE LOCATION - 2016
- BOREHOLE WITH GROUNDWATER WELL LOCATION - 2016
- GEOTECHNICAL TEST PITS LOCATION - 2016
- ENVIRONMENTAL TEST PITS LOCATION - 2016
- GEOTECHNICAL TEST PITS WITH DYNAMIC CONE PENETRATION LOCATION - 2016
- BOREHOLE WITH GAS WELL LOCATION - 2016
- HAND AUGER WITH DYNAMIC CONE PENETRATION LOCATION - 2016
- DYNAMIC CONE PENETRATION LOCATION - 2016
- JBS&G SAMPLE LOCATION (2015)

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A	ORIGINAL ISSUE			

revision

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drawn	[redacted]
approved	-
date	9 / 05 / 18
scale	AS SHOWN
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A TETRA TECH COMPANY

client: CPB CONTRACTORS PTY LTD

project: MOOREBANK INTERMODAL RAIL LINK
RALP NO.1
MOOREBANK, NSW

title: GEOTECHNICAL & CONTAMINATION INVESTIGATION LOCATION PLAN - SHEET 2 OF 2

project no: 754-GEOTLCOV24072AG | figure no: FIGURE 7 | rev: A

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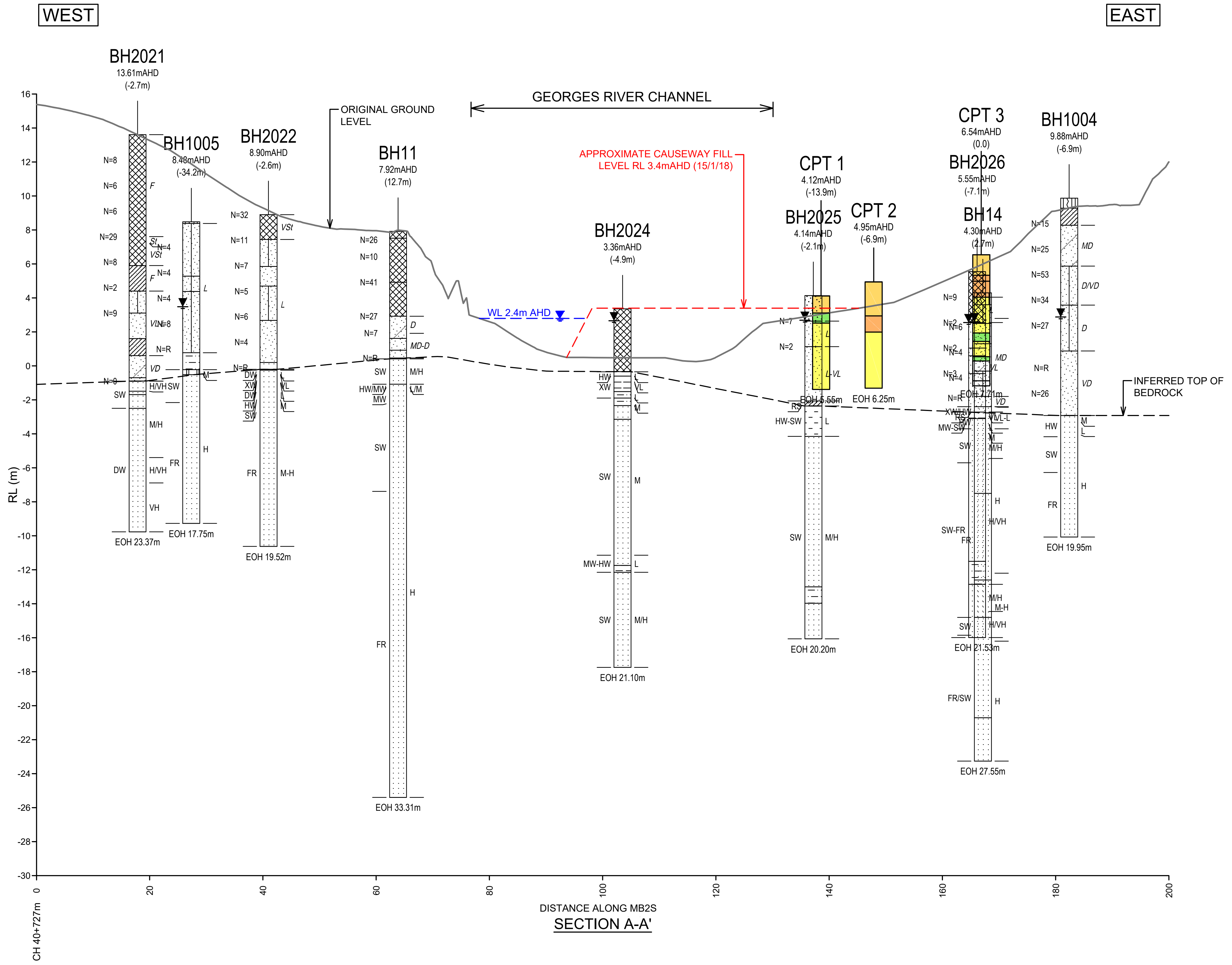

SECTION A-A'

LEGEND

- FILL
- SILTY CLAY
- SAND
- SILTSTONE
- INTERLAMINATED SILTSTONE & SANDSTONE
- INTERBEDDED SILTSTONE & SANDSTONE
- SANDSTONE
- TOPSOIL
- SANDY CLAY
- CLAYEY SAND
- SILTY SAND
- INTERBEDDED SILTSTONE & SANDSTONE
- CLAYEY SILT
- SANDY SILT
- CLAYSTONE
- CLAY
- GRAVELLY CLAY
- NO CORE
- SHALE
- MUDSTONE

UNIT LEGEND

- SANDSTONE FILL (CRUSHED)
- SANDY GRAVEL (HILLWASH ?)
- SANDY GRAVELLY ALLUVIUM
- ORGANIC-RICH LAYER

NOTES:
1. BOREHOLES BH2021, BH2022, BH2024, BH2025, BH2026 AND BH2027 FROM COFFEY JAN-FEB 2018 INVESITGATION.
2. ORIGINAL GROUND LEVEL SURVEY DATA SUPPLIED BY CPB.

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no.	description	drawn	approved	date
A	ORIGINAL ISSUE			

revision

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Vertical Scale (metres) 1:250

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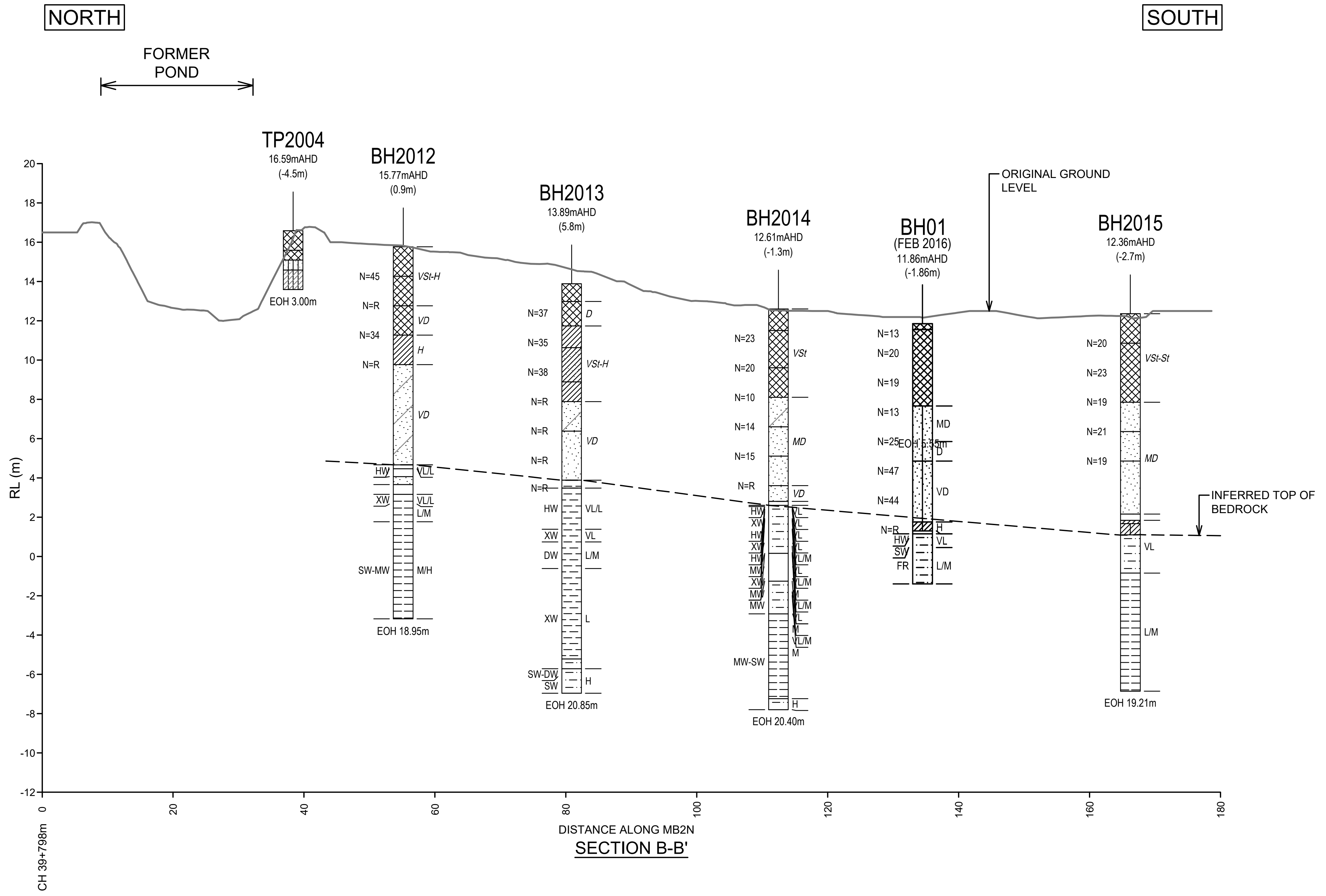
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS PTY LTD
project:	MOOREBANK INTERMODAL RAIL LINK RALP NO.1 MOOREBANK, NSW
title:	SECTION A-A'
project no:	754-GEOTLCOV24072AG
figure no:	FIGURE 8
rev:	A



SECTION B-B'

LEGEND

- FILL
- SILTY CLAY
- SAND
- SILTSTONE
- INTERLAMINATED SILTSTONE & SANDSTONE
- INTERBEDDED SILTSTONE & SANDSTONE
- SANDSTONE
- TOPSOIL
- SANDY CLAY
- CLAYEY SAND
- SILTY SAND
- INTERBEDDED SILTSTONE & SANDSTONE
- CLAYEY SILT
- SANDY SILT
- CLAYSTONE
- CLAY
- GRAVELLY CLAY
- NO CORE
- SHALE
- MUDSTONE

NOTES:
1. BOREHOLES BH2021, BH2022, BH2024, BH2025, BH2026 AND BH2027 FROM COFFEY JAN-FEB 2018 INVESITGATION.
2. BH01 FROM COFFEY FEB 2016 INVESTIGATION.
3. ORIGINAL GROUND LEVEL SURVEY DATA SUPPLIED BY CPB.

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Vertical Scale (metres) 1:250

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scale	AS SHOWN
original size	A3

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A TETRA TECH COMPANY

client: CPB CONTRACTORS PTY LTD

project: MOOREBANK INTERMODAL RAIL LINK
RALP NO.1
MOOREBANK, NSW

title: SECTION B-B'

project no: 754-GEOTLCOV24072AG | figure no: FIGURE 9 | rev: A

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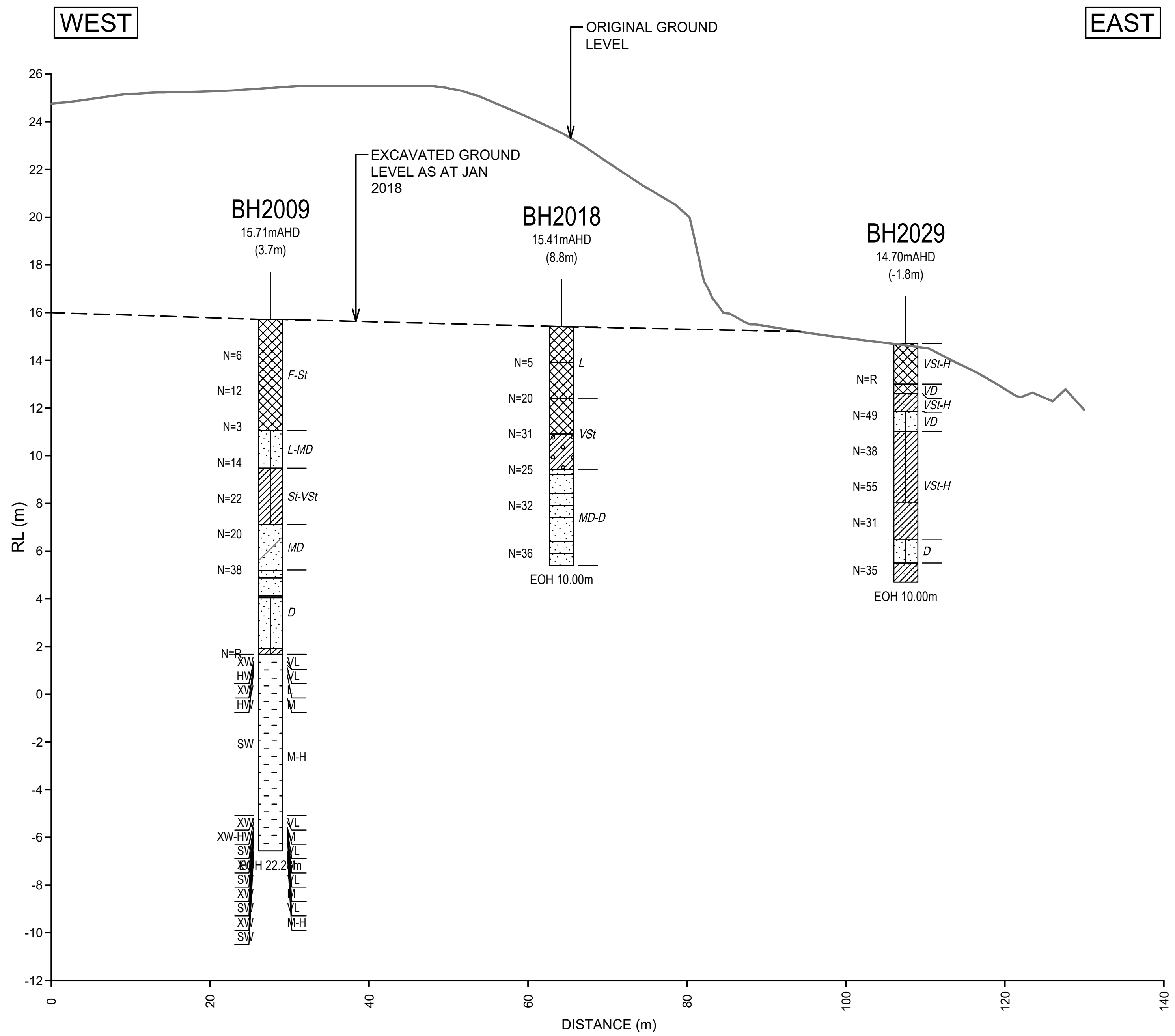

SECTION C-C'

LEGEND

- FILL
- SILTY CLAY
- SAND
- SILTSTONE
- INTERLAMINATED SILTSTONE & SANDSTONE
- INTERBEDDED SILTSTONE & SANDSTONE
- SANDSTONE
- TOPSOIL
- SANDY CLAY
- CLAYEY SAND
- SILTY SAND
- INTERBEDDED SILTSTONE & SANDSTONE
- CLAYEY SILT
- SANDY SILT
- CLAYSTONE
- CLAY
- GRAVELLY CLAY
- NO CORE
- SHALE
- MUDSTONE

NOTES:

1. BOREHOLES BH2021, BH2022, BH2024, BH2025, BH2026 AND BH2027 FROM COFFEY JAN-FEB 2018 INVESITGATION.
2. ORIGINAL GROUND LEVEL SURVEY DATA SUPPLIED BY CPB.

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Vertical Scale (metres) 1:250

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original size	A3

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project:	MOOREBANK INTERMODAL RAIL LINK RALP NO.1 MOOREBANK, NSW
title:	SECTION C-C'
project no:	754-GEOTLCOV24072AG
figure no:	FIGURE 10
rev:	A

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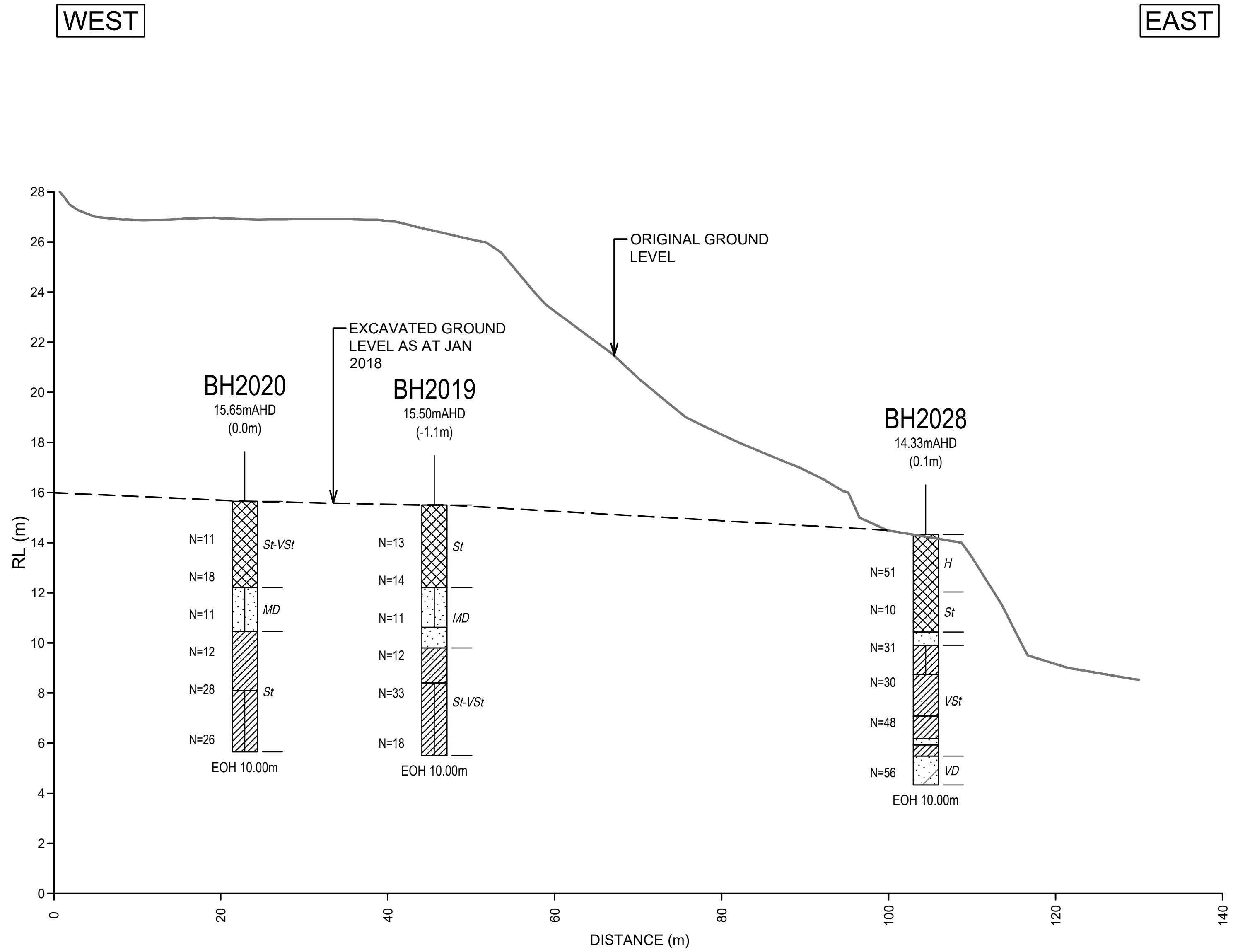
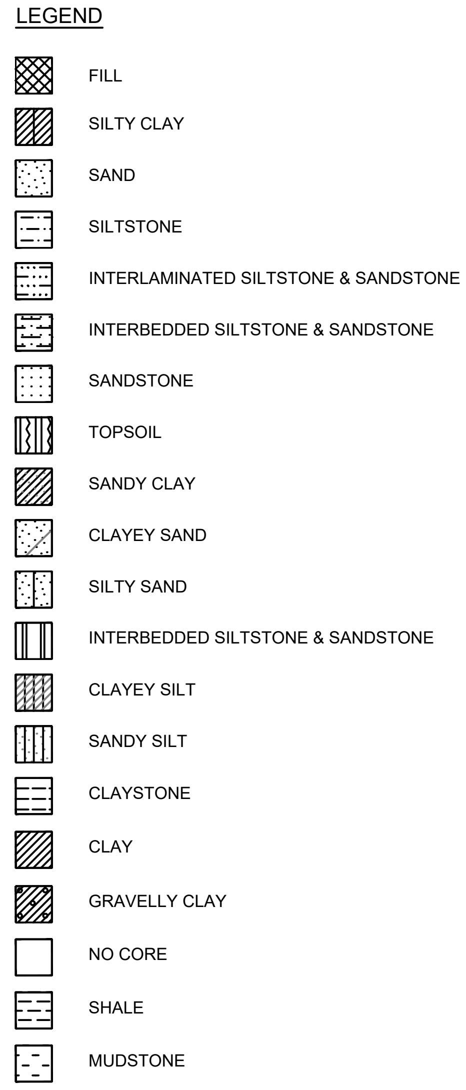


SECTION D-D'



NOTES:
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2. ORIGINAL GROUND LEVEL SURVEY DATA SUPPLIED BY CPB.

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revision no.	description	drawn	approved	date
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A TETRA TECH COMPANY

client:	CPB CONTRACTORS PTY LTD
project:	MOOREBANK INTERMODAL RAIL LINK RALP NO.1 MOOREBANK, NSW
title:	SECTION D-D'
project no:	754-GEOTLCOV24072AG
figure no:	FIGURE 11
rev:	A

Appendix A - Drawings

CPB CONTRACTORS

SIMTA SYDNEY INTERMODAL TERMINAL ALLIANCE

MOOREBANK INTERMODAL TERMINAL DEVELOPMENT - PACKAGE 1 - RALP No. 1

GEOTECHNICAL GROUND TREATMENT

COVERSHEET

LOCALITY PLAN

coffey
A TETRA TECH COMPANY
ABN No: 93 056 929 483

FOR COMMENT

revision

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approved	
date	10 / 07 / 18
scale	NTS
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	COVERSHEET
drawing no:	N01031-GRW-DRG-GEO-0001-09

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MOOREBANK INTERMODAL TERMINAL DEVELOPMENT
GROUND TREATMENT DESIGN
CH 40,427 - CH 40,740 (MB2S)
DRAWING INDEX

FOR COMMENT

revision

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date	10 / 07 / 18
scale	NTS
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	DRAWING LIST
drawing no:	N01031-GRW-DRG-GEO-0002-09

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GENERAL NOTES

GENERAL

G1 THESE DRAWINGS SHALL BE READ IN CONJUNCTION WITH ALL OTHER RELEVANT ENGINEERING DRAWINGS AND SPECIFICATIONS REFERRED IN THIS DESIGN LOT.

G2 THE CONSTRUCTION TEAM SHALL VERIFY SETTING OUT DIMENSIONS SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF CONSTRUCTION.

G3 REFER ANY DISCREPANCY TO THE SITE GEOTECHNICAL REPRESENTATIVES FOR CLARIFICATION BEFORE PROCEEDING WITH THE WORKS .

G4 UNLESS NOTED OTHERWISE, BATTER SLOPES FOR FILL SHALL BE 1.75H:1V OR FLATTER.

G5 ALL DIMENSIONS IN mm UNLESS OTHERWISE SHOWN.

DESIGN INTENT

DI1 FILL EMBANKMENTS FROM CH 40,445 TO CH 40,746 (MB2S) ARE OVERLYING LANDFILL MATERIAL. OR UNCONTROLLED FILL WITHOUT APPROPRIATE GROUND TREATMENT, THE LANDFILL ON WHICH THE EMBANKMENT IS TO BE CONSTRUCTED COULD RESULT IN EXCESSIVE POST CONSTRUCTION SETTLEMENTS.

DI2 SURCHARGE (CH 40,445 TO 40,746 - MB2S) GROUND TREATMENT TECHNIQUE HAS BEEN PROPOSED FOR THE FILL EMBANKMENTS TO ENSURE SATISFACTORY RAIL TRACK PERFORMANCE IS ACHIEVED.

DI3 MAINTAINING STABILITY OF THE EMBANKMENT DURING CONSTRUCTION IS OF PARAMOUNT IMPORTANCE. EMBANKMENT MUST NOT BE FILLED AT AN EXCESSIVELY FAST RATE OF FILLING GREATER THAN 2.1 M/WEEK AS IT CAN CAUSE EMBANKMENT INSTABILITY.

DI4 MONITORING OF THE GROUND BEHAVIOUR WITH RESPECT TO THE SETTLMENTS THE SURCHARGE/WAITING PERIOD IS REQUIRED AS AN INTEGRAL PART OF THE DESIGN PROCESS. THIS OBSERVATIONAL APPROACH IS REQUIRED TO REFINE THE PREDICTED SETTLEMENTS.

DI5 BASED ON THE RESULTS FROM SETTLEMENT MONITORING DURING SURCHARGE/WAITING PERIOD, THE FREQUENCY OF REQUIRED FUTURE MAINTENANCE PERIOD MAY BE MODIFIED TO ENSURE THE PREDICTED POST CONSTRUCTION SETTLEMENT AND DIFFERENTIAL SETTLEMENTS ARE IN ACCORDANCE WITH PROJECT SPECIFIC DESIGN CRITERIA PROVIDED IN DC1 AND DC2.

DESIGN CRITERIA AND ASSUMPTIONS

DC1 THE FOLLOWING POST CONSTRUCTION SETTLEMENT CRITERIA HAS BEEN ADOPTED:

1. MAXIMUM POST CONSTRUCTION SETTLEMENT OF 500mm AS STIPULATED IN THE "HEAD CONTRACT CLARIFICATION" No. 20 OVER A DESIGN LIFE OF 40 YEARS; AND
2. POST CONSTRUCTION DIFFERENTIAL SETTLEMENT ≤ 0.25% CHANGE IN GRADE WITHIN EACH INTERVENTION PERIOD.

DC2 THE POST CONSTRUCTION DIFFERENTIAL SETTLEMENT CRITERIA WAS DEVELOPED ADOPTING THE REQUIREMENTS AS PER SECTION 1.2.1 CLAUSE d(ii) OF THE PRINCIPAL'S PROJECT REQUIREMENTS AND PAST EXPERIENCE IN A SIMILAR PROJECT (HEXHAM RELIEF ROAD PROJECT).

DC3 THE INTERVENTION INVOLVING REBALLASTING AND RETAMPING ARE CONSIDERED TO SATISFY THE ABOVE CRITERIA (ITEM DC1) THROUGHOUT THE DESIGN LIFE. THREE INTERVENTIONS ARE ALLOWED WITHIN THE DEFECT LIABILITY PERIOD OF 1 YEAR AFTER THE TRACK IS COMMISSIONED.

DC4 THE FOLLOWING FACTORS OF SAFTEY (FOS) WERE ADOPTED IN THE GEOTECHNICAL DESIGN TO MEET THE GLOBAL STABILITY REQUIREMENTS FOR THE EMBANKMENT:

1. SHORT TERM (SURCHARGE AND CONSTRUCTION LOADING) ≥ 1.3;
2. LONG TERM (DESIGN EMBANKMENT AND TRAIN LOADING) ≥ 1.5; AND
3. RAPID DRAWDOWN AND SEISMIC ≥ 1.1.

INSTRUMENTATIONS

IN1 MONITORING EQUIPMENT (TUBES, RODS, ETC.) SHALL BE EXTENDED 24 HOURS BEFORE THE NEXT LAYER OF EMBANKMENT IS PLACED IF THIS WILL RESULT IN THE EMBANKMENT REACHING WITHIN 0.5 m OF THE TOP OF THE INSTRUMENTS, AND ALL THE EQUIPMENT SHALL BE CHECKED PRIOR TO FILLING.

IN2 ONLY HAND OPERATED EQUIPMENT TO BE USED WITHIN 1 m OF THE INSTRUMENTATION AND THE INSTRUMENTS SHALL BE KEPT VERTICAL.

IN3 INSTRUMENTS ARE FOR THE PURPOSE OF MONITORING EMBANKMENTS MOVEMENTS DURING AND AFTER CONSTRUCTION ONLY. APPROPRIATE TEMPORARY ACCESS MUST BE PROVIDED TO MONITOR INSTRUMENTS DURING CONSTRUCTION.

IN4 AN OVERSIZE PVC CONDUIT MUST BE INSTALLED TO PROTECT SETTLEMENT PLATE ROD.

IN5 INSTALLATION, PROTECTION, MONITORING AND REPORTING OF INSTRUMENTATION WORKS MUST BE IN ACCORDANCE WITH PROJECT "INSTRUMENTATION AND MONITORING SPECIFICATION" (TO BE ISSUED IN IFC DOCUMENTATION)

FORMATION REQUIREMENTS

FR1 A SUBGRADE CALIFORNIA BEARING RATIO (CBR) OF > 3% IS ASSUMED FOR GENERAL EMBANKMENT FILL AND REINFORCED EMBANKMENT AREAS. FORMATION REQUIREMENTS SHOULD BE AS PER ARTC STANDARD, ETM-08-01.

FR2 SUBGRADE CONDITIONS MUST BE ASSESSED FOR CBR. TEST LOCATION MUST BE BASED ON INFORMATION FROM ANY ADDITIONAL GEOTECHNICAL SITE INVESTIGATION AND SURCHARGE PREPARATION.

SITE INVESTIGATIONS

SI1 LANDFILL THICKNESS ASSUMED IN THE DESIGN IS BASED ON AVAILABLE SITE INVESTIGATION DATA AND SURVEY DATA SHOWING THE BASE OF LANDFILL.

GEOGRIDS - REINFORCED EMBANKMENT AREA

GE1 MANUFACTURE AND INSTALLATION OF STRUCTURAL GEOGRIDS SHALL BE IN ACCORDANCE WITH RMS D&C R67.

GE2 EMBANKMENT FILL MATERIALS SHALL BE PLACED IN ACCORDANCE WITH RMS D&C R44 AND B30.THE SELECT FILL (AS PER RMS D & C R44) HAS BEEN CONSIDERED WITH FOLLOWING STRENGTH PROPERTIES:

- EFFECTIVE FRICTION ANGLE = 34°
- EFFECTIVE COHESION = 0

GE3 CPB WILL CARRY OUT GEOTECHNICAL TESTING ON SITE-WON EXCESS MATERIAL TO DETERMINE THE SOIL PROPERTIES. IF THE SITE-WON MATERIAL SATISFIES GENERAL EMBANKMENT FILL PROPERTIES (AS PER RMS D&C R44) OR BETTER, CPB RESERVE THE RIGHT TO RE-DESIGN THE REINFORCED EMBANKMENT USING SITE-WON MATERIAL.

GE4 SHOULD SITE-WON MATERIAL NOT SATISFY GENERAL EMBANKMENT FILL PROPERTIES (AS PER RMS D&C R44), CPB WILL CARRY OUT GEOTECHNICAL TESTING ON OFF-SITE SOURCED MATERIAL TO DETERMINE THE SOIL PROPERTIES. IF THE OFF-SITE SOURCED MATERIAL BE SATISFIES GENERAL EMBANKMENT FILL PROPERTIES (AS PER RMS D&C R44) OR BETTER, CPB RESERVE THE RIGHT TO RE-DESIGN THE REINFORCED EMBANKMENT USING OFF-SITE SOURCED MATERIAL

GE5 GEOGRIDS SHALL BE PLACED FLAT AND FREE OF CREASES AND LIGHTLY TENSIONED PRIOR TO AND DURING PLACEMENT OF FILL.

GE6 HIGH STRENGTH STRUCTURAL GEOGRIDS MUST SATISFY THE MINIMUM ULTIMATE TENSILE STRENGTH REQUIREMENTS STATED IN RELEVANT DRAWINGS. THE ADOPTED ULTIMATE TENSILE STRENGTH VALUES PROVIDED IN THIS DESIGN LOT ARE BASED ON THE SHORT TERM TENSILE STRENGTH VALUES AT 12 % STRAIN FOR PARALINK OR EQUIVALENT. STRENGTH REDUCTION FACTOR OF 2 IS TO BE APPLIED FOR THE ASSESSMENT OF REQUIRED DESIGN SHORT TERM AND LONG TERM GEOGRID STRENGTH. (CURRENT DESIGN CONSIDERS A LINEAR VARIATION BETWEEN TENSILE STRENGTH AND STRAIN).

GE7 MACCAFERRI'S GREEN TERRAMESH OR EQUIVALENT FLEXIBLE FACING SYSTEM IS TO BE INSTALLED. GEOGRID LAYERS ARE TO BE CONNECTED TO FACING ELEMENTS, AS PER MANUFACTURER'S REQUIREMENTS AND METHOD SPECIFICATION.

LANDFILL CAPPING

LC1 PRIOR TO CONSTRUCTION OF EMBANKMENT OVER EXISTING LANDFILL, INSTALL LANDFILL CAPPING LAYER (COMPOSITE GCL, GEOMEMBRANE & GEOTEXTILE) IN ACCORDANCE WITH DRAWING N01031-GRW-DRG-GEO-0007.

LC2 PRIOR TO PLACEMENT OF FUTURE ADDITIONAL LANDFILL ON EMBANKMENT BATTER, LANDFILL CAPPING LAYER TO BE INSTALLED ON EMBANKMENT BATTER BY OTHERS.

OTHER ITEMS

OI1 DRAINAGE, UTILITIES AND SERVICES WITHIN LANDFILL AREA MAY BE IMPACTED BY THE LANDFILL SETTLEMENTS WHICH SHALL BE TAKEN INTO ACCOUNT OF IN THEIR DESIGN. THESE HAVE BEEN COORDINATED WITH AURECON/CPB.

RELEVANT SPECIFICATIONS

ARTC STANDARD ETM-08-01 (DATED 18 JUNE 2010)

ARTC STANDARD ETC-08-01 (DATED 11 MAY 2006)

ARTC HEAVY HAUL INFRASTRUCTURE GUIDELINE - TRACK, CIVIL AND STRUCTURES - REV 7 (DATED 20 JUNE 2013)

PROJECT SPECIFICATION FOR INSTRUMENTATION AND MONITORING REQUIREMENTS (TO BE ISSUED IN IFC DOCUMENTATION)

APPLICABLE TECHNICAL DOCUMENTS

GEOTECHNICAL INVESTIGATION REPORT (GEOTLCOV24072AF-AM REV. 2).

GEOPHYSICAL INVESTIGATION REPORT (GEOTLCOV24072AF-AI REV. 0)

GEOTECHNICAL GROUND TREATMENT REPORT (GEOTLCOV24072AF-BA/N01031-GRW-DRP-GEO-0001-01).

POST IFC - ORIGINAL DESIGN

GEOTECHNICAL INVESTIGATION REPORT (GEOTLCOV24072AG-AE DATED 28 MAY 2018).

FOR COMMENT

revision

no.	description	drawn	approved	date
[redacted]				

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	NTS
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	GENERAL NOTES
drawing no:	N01031-GRW-DRG-GEO-0003-09

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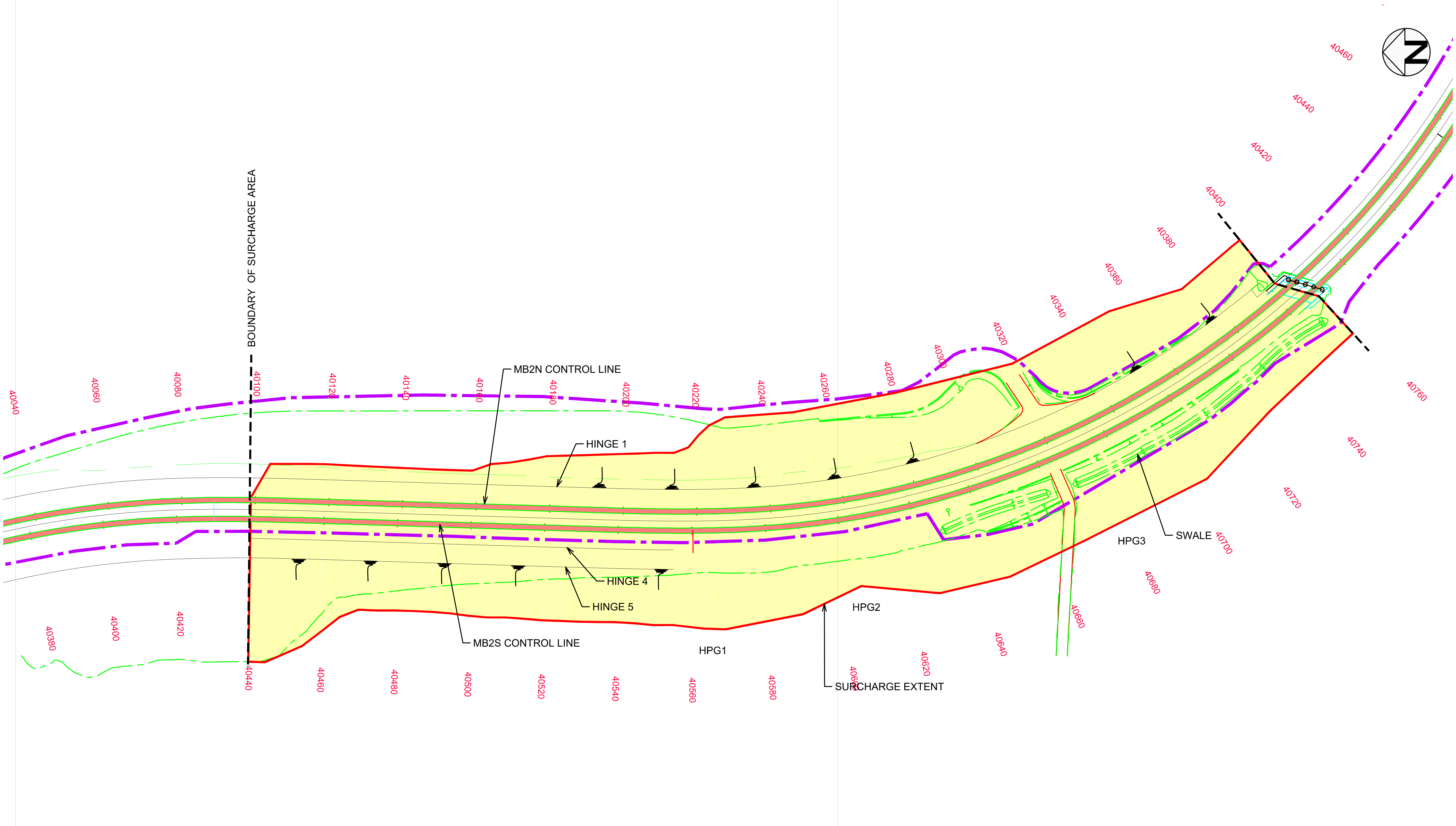

LEGEND

SURCHARGE GROUND TREATMENT

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

10 0 10 30 50

Scale (metres) 1:1000

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey

A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	SCHEMATIC PLAN OF GROUND TREATMENT
drawing no:	N01031-GRW-DRG-GEO-0004-09

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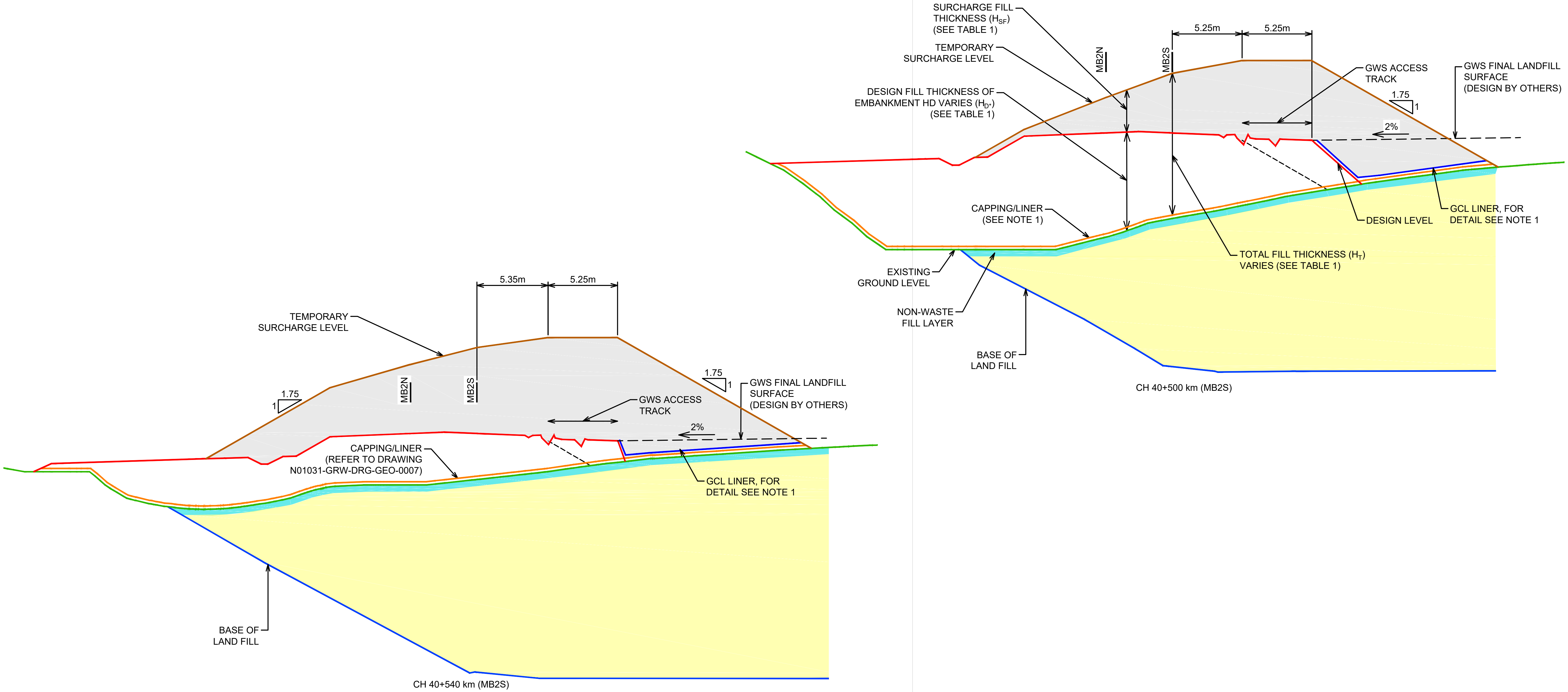


TYPICAL SECTIONS OF SURCHARGE TREATED AREA

LEGEND

- SURCHARGE FILL ABOVE DESIGN LEVEL
- NON-WASTE FILL LAYER
- LANDFILL

NOTE:

1. TO BE INSTALLED BY OTHERS PRIOR TO FUTURE LANDFILL. REFER TO DRAWING N01031-GRW-DRG-GEO-0007 FOR LANDFILL CAPPING REQUIREMENTS.

FOR COMMENT

revision

[redacted]

5 0 5 10 15

Scale (metres) 1:300

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	SURCHARGE DETAILS FOR TRANSITION ZONE
drawing no:	N01031-GRW-DRG-GEO-0005-09

PLOT DATE: 10/07/2018 3:39:59 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA2_TECHNICAL\GEOTECHNICS\1 PROJECTS\GEOTLCOV\240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG

Construction Sequence from Ch 40,440 to Ch 40,640

Step 1. Strip topsoil and vegetation and stockpile for reuse for landscaping purposes.

Step 2. Check that the existing surface layer meets the particle size requirement of the GCL cushioning layer (see Sketch 3 above). If not, over-excavate to a depth of 300 mm and replace with compacted GCL cushioning material.

Step 3. Proof roll the exposed surface by a minimum 8 passes of a vibratory roller having a static weight of at least 10 tonnes.

Step 4. Localised areas that appear wet, spongy or heave excessively (i.e. more than 20 mm visually) shall be over-excavated to 300 mm and backfilled with compacted material (note: where imported GCL cushioning layer is to be placed, the backfill material beneath the GCL cushioning layer may comprise crushed sandstone or similar granular material, but where the existing subgrade is to be used as the GCL cushioning layer, the backfill material shall meet the requirement of the GCL cushioning material).

Step 5. Where required, place and compact the imported 300 mm GCL cushioning layer to a minimum Dry Density Ratio (DDR) of 98% Standard at Optimum Moisture Content (OMC) ± 2%. This compaction requirement also applies to existing subgrade material satisfying the GCL cushioning layer grading.

Step 6. Place the GCL, geomembrane and cushioning geotextile in the following order:

manufacturer's guidelines.

- The GCL shall be placed in the transverse direction with adequate laps between two adjacent liners ensuring continuity of the liner during the life time of the embankment. If a lap in the transverse direction is required, care should be taken regarding the gradient of existing ground and the location of the lap to avoid slip between the GCLs.
- The river side of the existing GCL with the existing landfill shall be exposed and the new GCL lap over it for a minimum distance of 2 m towards the river.
- On the landfill side, the new GCL shall be placed beyond the toe of the surcharge and at least 3 m past the toe of the final embankment (i.e. after removal of the surcharge), whichever is the greater.
- The Geomembrane shall be placed over the GCL (to the same lateral extent), with all joints field welded according to the manufacturer's specification.
- The cushioning geotextile shall be placed over the Geomembrane (to the same lateral extent) with a minimum overlap of 300 mm between joints.
- Provide temporary soil cover of at least 0.5 m thickness over the GCL, Geomembrane and Geotextile that extends beyond the toes of the surcharge.

Step 7. Place the first layer of general embankment material with 300 mm compacted thickness having a maximum particle size of 13.2 mm and 90% passing 2.4 mm above the landfill capping layer (see Sketch 3 above), to a minimum DDR of 95% Standard at OMC ± 2%. All general embankment fill should have a soaked CBR > 3% (i.e. minimum 4%).

Step 8. Continue placing and compacting general fill in layers, including surcharge fill until the design surcharge level is reached. Settlement of the landfill during embankment/surcharge construction may result in more surcharge being placed than required for design, depending on the rate of settlement achieved. Subject to settlement monitoring and review, termination of surcharge construction may be feasible prior to reaching the design surcharge level.

Step 9. After primary consolidation is complete and sufficient data on secondary consolidation gathered from the monitoring, the surcharge may be removed to the design subgrade level (i.e. base of formation).

Step 10. Construct the formation in accordance with ARTC standard ETM-08-01 Earthworks.

Construction Sequence between Ch 40,640 and Ch 40,705

Step 1. Excavate to a depth of 0.6 m below existing ground level and stockpile the material for reuse for landscaping purposes.

Step 2. Proof roll the exposed surface by a minimum 8 passes of a vibratory roller having a static weight of at least 10 tonnes.

Step 3. Localised areas that appears wet, spongy or heave excessively (i.e. more than 20 mm visually) shall be over-excavated a further 300 mm and backfilled with compacted granular material (e.g. crushed sandstone).

Step 4. If landfill is found near the excavated surface and causing difficulties in the proof rolling, placement of a heavy geogrid (e.g. bi-directional 60 kN geogrid) may be required to assist proof rolling and compaction of the overlying layers.

Step 5. Place and compact a 300 mm thick layer that satisfies the GCL cushioning layer grading requirements shown in Sketch 3 above as well as ARTC standard ETM-08-01 Earthworks for Structural Fill (i.e. between 20% and 30% passing 75 microns and maximum particle size of 13.2 mm with a soaked CBR of at least 8%). If necessary, use lime stabilisation to achieve this minimum CBR. It is intended that this layer would form the lower zone of the Structural Fill which forms part of the Formation. Compaction of this layer shall meet the requirement of ARTC standard ETM-08-01 Earthworks for Structural Fill.

Step 6. Install the landfill capping layer (i.e. GCL, Geomembrane and Geotextile) as in Step 6 described in Section 5.5.2 above.

Step 7. Place and compact a 300 mm thick layer of material over the GCL that satisfy the grading requirements of the lower 0.3 m general fill layer shown in Sketch 3, and in addition, the material used is to have a minimum soaked CBR of 8% and satisfying the requirements of ARTC standard ETM-08-01 Earthworks for Structural Fill. For example, a fine to coarse grained sand or crusher dust layer would satisfy this requirement. This layer would form the upper zone of the Structural Fill of the Formation layer, giving a total thickness of 600 mm of Structural Fill, allowing for a possibility of 100 mm of Structural Fill to be trimmed off if required depending on the actual settlement achieved during preloading (see Step 12). Compaction of this layer shall meet the requirement of ARTC standard ETM-08-01 Earthworks for Structural Fill.

Step 8. Place and compact general fill surcharge (to a minimum DDR of 95% Standard) over the Structural Fill until the design surcharge level is reached. Settlement of the landfill during embankment/surcharge construction may result in more surcharge being placed than required for design, depending on the rate of settlement achieved. Subject to settlement monitoring and review, termination of surcharge construction may be feasible prior to reaching the design surcharge level.

Step 9. After primary consolidation is complete and sufficient data on secondary consolidation gathered from the monitoring, the surcharge may be removed to the top of the Structural Fill already placed during Step 7. Note that for the above construction sequence, a settlement of 450 mm would result in the top of the Structural Fill being at 450 mm below the original ground level which would allow 150 mm of capping and 300 mm of ballast to be placed for a design surface level equivalent to the existing ground level. This could also be achieved if the actual settlement is only 350 mm, by trimming off 100 mm of Structural Fill following the settlement period.

Step 10. Construct the formation in accordance with ARTC standard ETM-08-01 Earthworks.

Construction Sequence from Ch 40,705

The recommended construction sequence in this section is as follows:

Step 1. Strip topsoil and vegetation and stockpile for reuse for landscaping purposes.

Step 2. Excavate the existing fill to the following depths below existing ground level, and stockpile clean fill for reuse as general embankment fill:

- Ch 40,705 to Ch 40,731.5 — 1 m
- Ch 40,731.5 to Ch 40,747.23 — 2.5 m
- From Ch 40,747.23 — 2.5 m to 0 m to intersect existing ground level

Step 3. Place and compact general fill in layers (not more than 300 mm for each layer) to a minimum Dry Density Ratio of 95% at OMC ± 2% until 300 mm below the subgrade level.

Step 4. Place and compact the final 300 mm layer of general fill to a minimum DDR of 98% at OMC ± 2%.

Step 5. Place and compact surcharge fill in layers to a minimum DDR of 95%.

Note: In Steps 3 to 5, fill construction at the end of the surcharge will need to incorporate the construction of the geogrid reinforced soil batter.

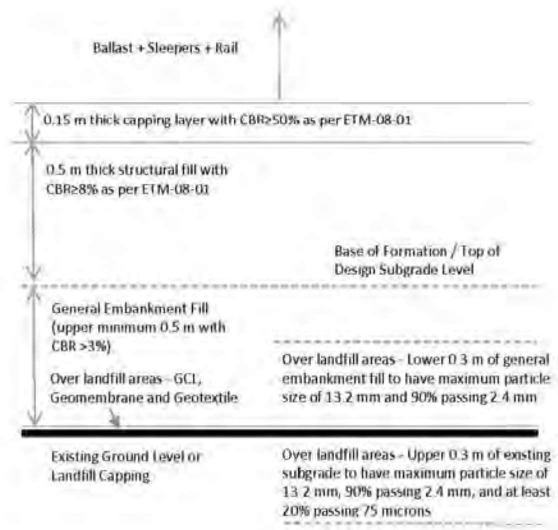
Step 6. After primary consolidation is complete and sufficient data on secondary consolidation gathered from the monitoring, the surcharge may be removed to the design subgrade level (i.e. base of formation).

Step 7. Construct the formation in accordance with ARTC standard ETM-08-01 Earthworks.

CONSTRUCTION STAGING

Typical Formation Profile

The formation requirements shall be in accordance with the ARTC standard ETM-08-01 Earthworks, Formation and Capping Material, and as shown in the following sketch.



FOR COMMENT

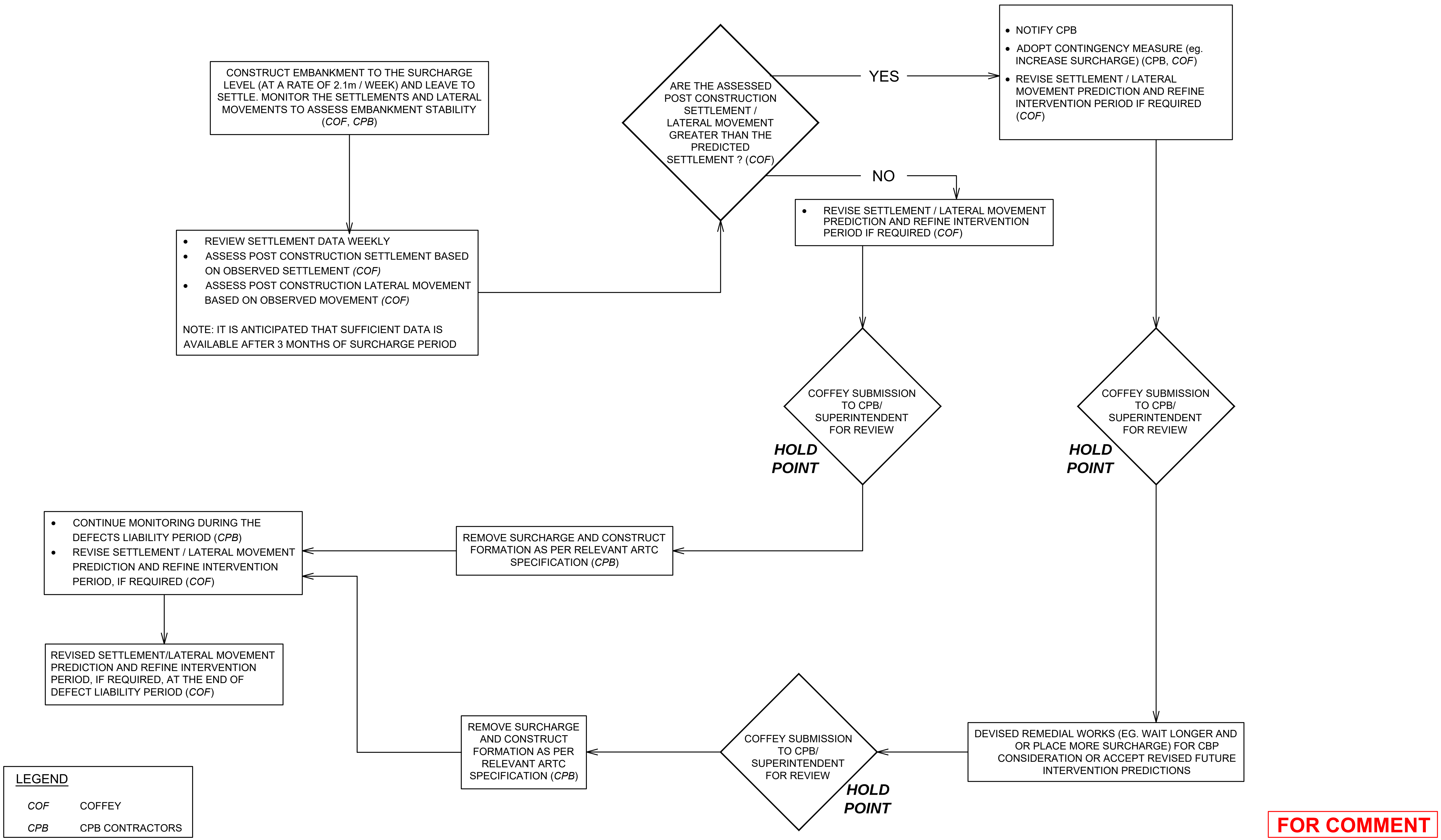
no.	description	drawn	approved	date
[redacted]				

revision

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	NTS
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	CONSTRUCTION SEQUENCE
drawing no:	N01031-GRW-DRG-GEO-0007-09



FOR COMMENT

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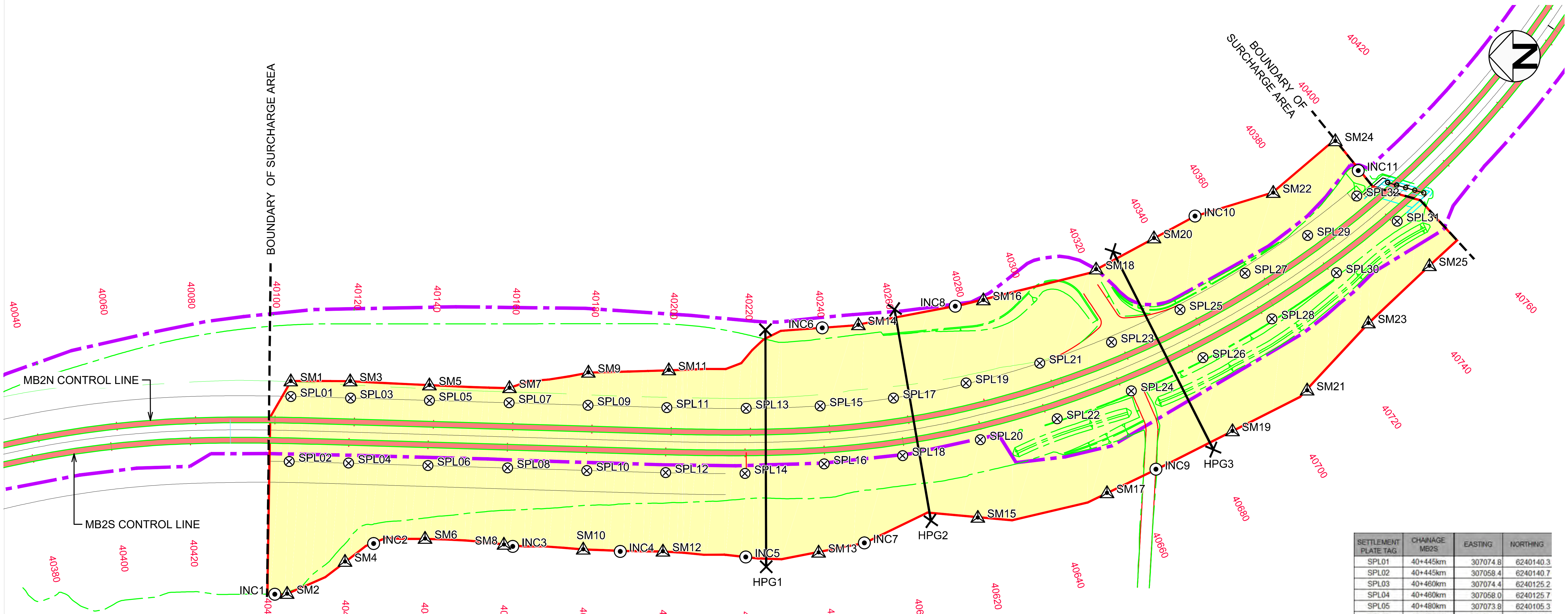
drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	GEOTECHNICAL GROUND TREATMENT PROCESS CONTROL DIAGRAM FOR SURCHARGE WORKS
drawing no:	N01031-GRW-DRG-GEO-0008-09



LEGEND

- SURCHARGE GROUND TREATMENT
- ⊗ SETTLEMENT PLATE LOCATION
- ▲ SURVEY MONUMENT
- ⊙ INCLINOMETERS (INC)
- ×——× HYDROSTATIC PROFILE GAUGE (HPG)

HYDROSTATIC PROFILE GAUGE	CHAINAGE MB2S
HPG1	40+568km
HPG2	40+613km
HPG3	40+673km

INCLINOMETER	CHAINAGE MB2S	EASTING	NORTHING	DEPTH FROM SURFACE
INC1	40+440km	307024.9	6240144.3	TBA
INC2	40+465km	307037.7	6240119.4	TBA
INC3	40+502km	307037.0	6240084.3	TBA
INC4	40+530km	307035.7	6240057.2	TBA
INC5	40+560km	307034.3	6240025.5	TBA
INC6	40+240km	307092.1	6240006.2	TBA
INC7	40+588km	307037.9	6239995.6	TBA
INC8	40+278km	307097.6	6239972.6	TBA
INC9	40+660km	307056.4	6239922.0	TBA
INC10	40+353km	307120.3	6239912.1	TBA
INC11	40+440km	307131.8	6239871.0	TBA

SURVEY MONUMENT TAG	CHAINAGE MB2S	EASTING	NORTHING
SM1	40+445km	307078.7	6240140.3
SM2	40+445km	307025.1	6240141.2
SM3	40+460km	307078.7	6240125.3
SM4	40+460km	307033.2	6240126.6
SM5	40+480km	307077.7	6240105.3
SM6	40+480km	307039.0	6240106.4
SM7	40+500km	307077.0	6240085.1
SM8	40+500km	307037.5	6240086.5
SM9	40+520km	307080.9	6240065.2
SM10	40+520km	307036.3	6240066.5
SM11	40+540km	307081.5	6240044.9
SM12	40+540km	307035.8	6240046.4
SM13	40+575km	307035.5	6240007.1
SM14	40+590km	307092.9	6239997.1
SM15	40+617km	307044.3	6239966.9
SM16	40+630km	307099.2	6239965.5
SM17	40+649km	307050.5	6239934.3
SM18	40+665km	307106.9	6239937.1
SM19	40+680km	307066.1	6239902.7
SM20	40+680km	307114.8	6239922.5
SM21	40+700km	307076.4	6239883.8
SM22	40+720km	307126.2	6239892.4
SM23	40+720km	307093.4	6239868.4
SM24	40+740km	307139.3	6239876.7
SM25	40+740km	307107.8	6239853.0

SETTLEMENT PLATE TAG	CHAINAGE MB2S	EASTING	NORTHING
SPL01	40+445km	307074.8	6240140.3
SPL02	40+445km	307058.4	6240140.7
SPL03	40+460km	307074.4	6240125.2
SPL04	40+460km	307058.0	6240125.7
SPL05	40+480km	307073.8	6240105.3
SPL06	40+480km	307057.4	6240105.7
SPL07	40+500km	307073.2	6240085.2
SPL08	40+500km	307056.8	6240085.6
SPL09	40+520km	307072.6	6240065.3
SPL10	40+520km	307056.1	6240065.6
SPL11	40+540km	307072.0	6240045.4
SPL12	40+540km	307055.6	6240045.7
SPL13	40+560km	307071.8	6240025.4
SPL14	40+560km	307055.4	6240025.7
SPL15	40+580km	307072.4	6240006.7
SPL16	40+580km	307057.8	6240005.7
SPL17	40+600km	307074.4	6239988.2
SPL18	40+600km	307059.9	6239985.9
SPL19	40+620km	307078.2	6239969.9
SPL20	40+620km	307063.9	6239966.3
SPL21	40+640km	307083.2	6239951.3
SPL22	40+640km	307069.2	6239946.9
SPL23	40+660km	307088.5	6239933.2
SPL24	40+660km	307076.2	6239928.2
SPL25	40+680km	307096.7	6239915.9
SPL26	40+680km	307084.6	6239910.1
SPL27	40+700km	307105.9	6239899.5
SPL28	40+700km	307094.4	6239892.7
SPL29	40+720km	307115.4	6239883.7
SPL30	40+720km	307106.0	6239876.4
SPL31	40+740km	307119.0	6239861.1
SPL32	40+740km	307125.4	6239871.3

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

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Scale (metres) 1:1000

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	INSTRUMENTATION SETOUT
drawing no:	N01031-GRW-DRG-GEO-0010-09

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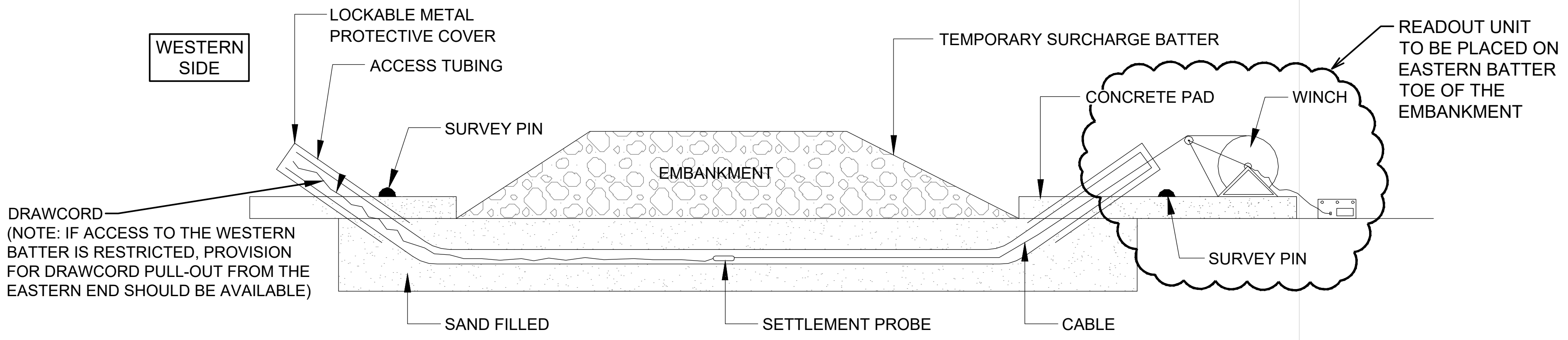
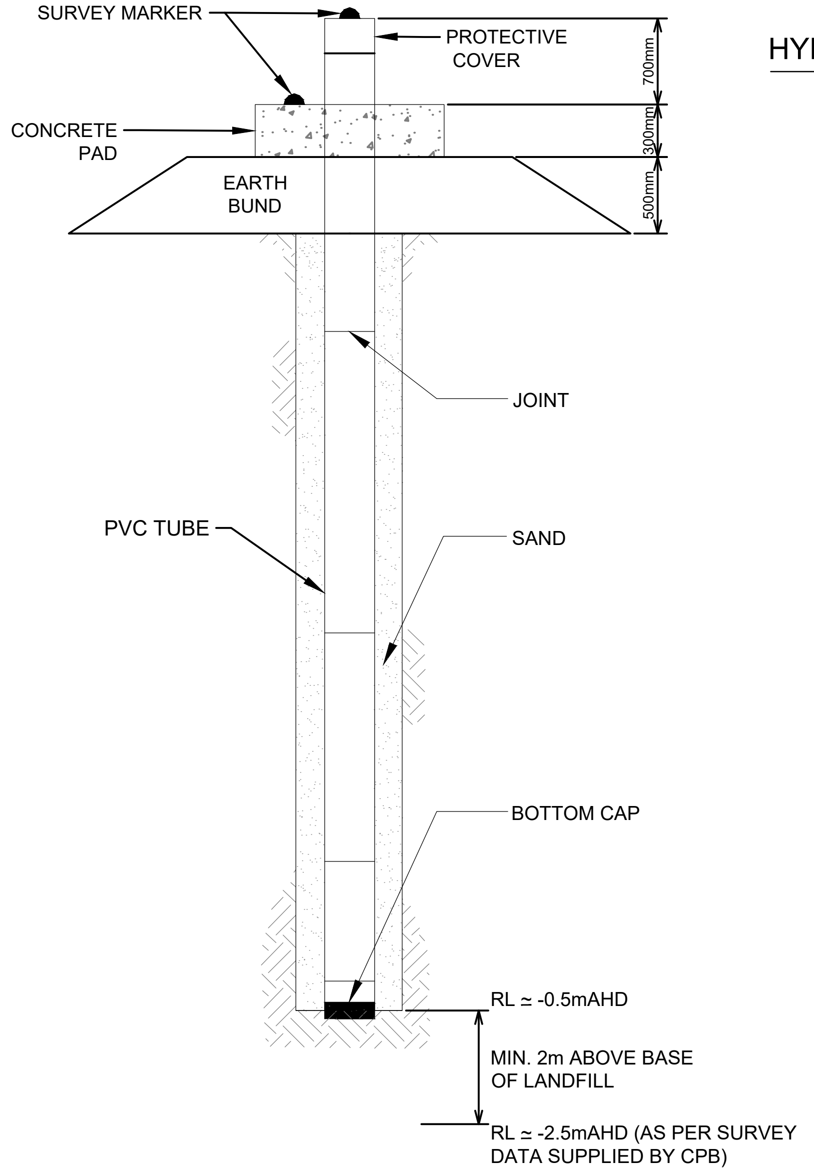


HYDROSTATIC PROFILE GAUGE (HPG)

(NTS)



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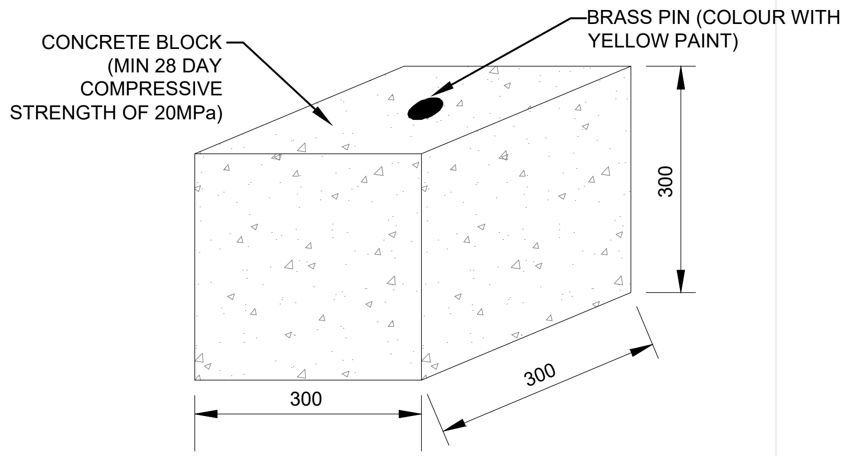
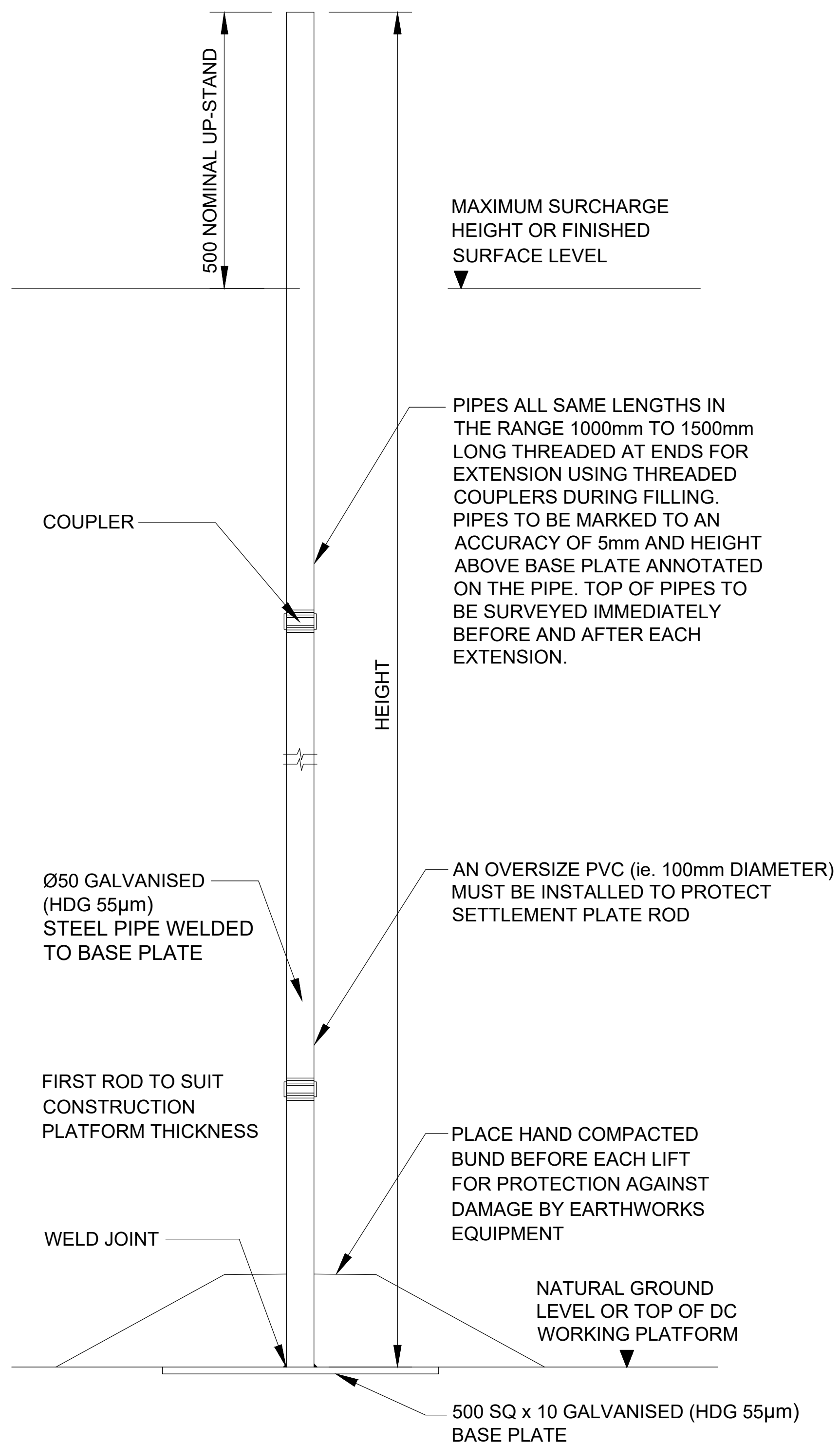
(N.T.S)



SURVEY MONUMENT (SUM)

SCALE 1:10



SETTLEMENT PLATE (SPL)

(N.T.S)

FOR COMMENT

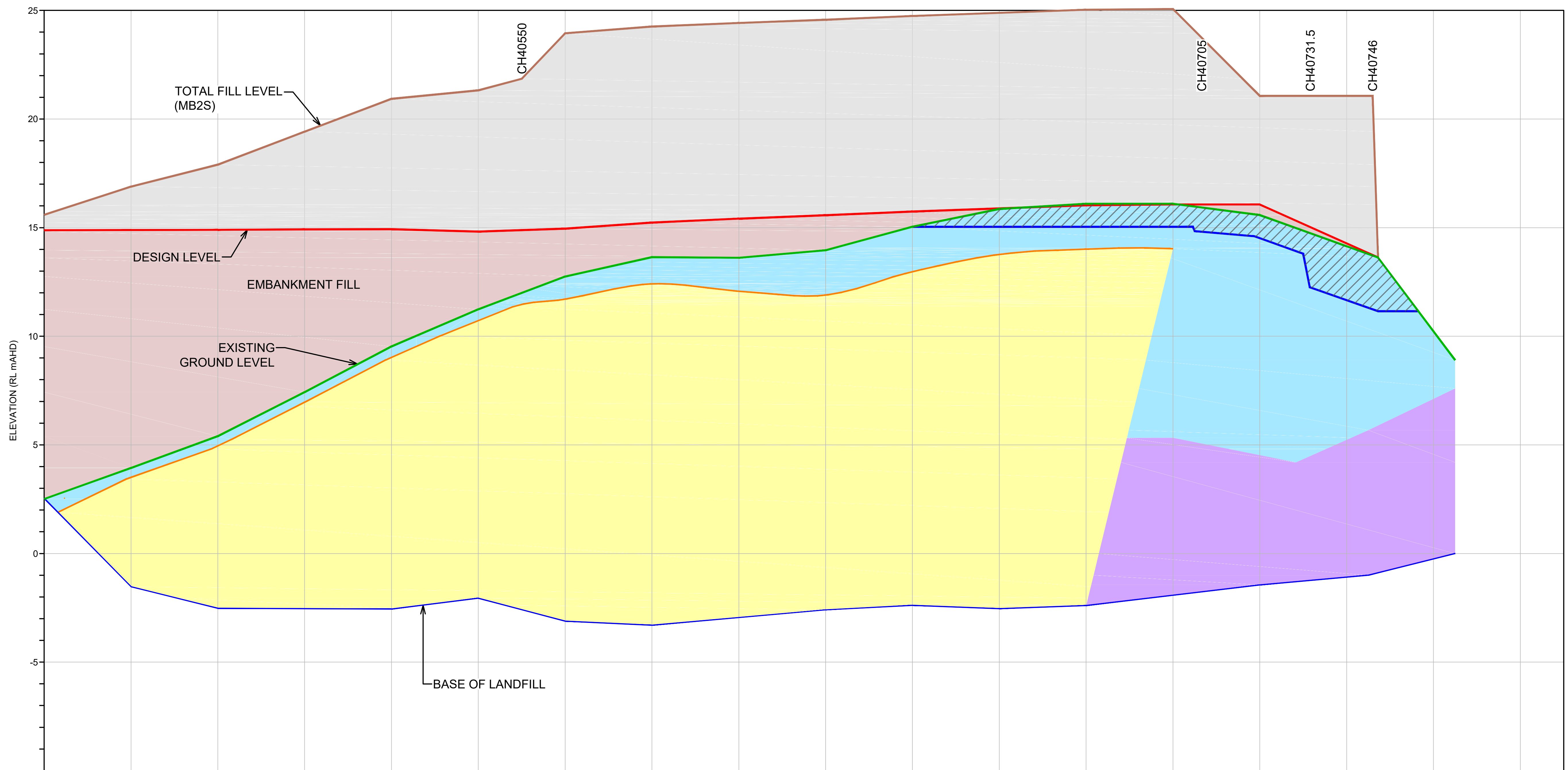
no.	description	drawn	approved	date
[redacted]				

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	TYPICAL INSTRUMENTATION DETAILS
drawing no:	N01031-GRW-DRG-GEO-0011-09

PLOT DATE: 10/07/2018 3:18:32 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA2 TECHNICAL\GEOTECHNICS\1 PROJECTS\GEOTLCOV240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG



CHAINAGE (m) (MB2S)	DESIGN LEVEL (mAHD) MB2S	EXISTING LEVEL (mAHD) MB2S
40440	15.59	
40460	16.89	3.94
40480	17.90	5.40
40500	19.42	7.43
40520	20.93	9.53
40540	21.32	11.24
40560	19.68	12.75
40580	15.41	13.64
40600	15.56	13.61
40620	15.70	13.96
40640	16.04	15.04
40660	16.09	15.86
40680	16.24	16.10
40700	16.31	16.10
40720	16.22	15.58
40740	16.12	14.12
40760		
40780		
40790		

UNIT LEGEND

- SURCHARGE FILL
- FILL TO DESIGN LEVEL
- NON-WASTE FILL LAYER
- LANDFILL
- ALLUVIUM
- OVER EXCAVATION AND REPLACE WITH ENGINEERED FILL FOR WORKING PLATFORM

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

Horizontal Scale (metres) 1:1000 (10 0 10 30 50)

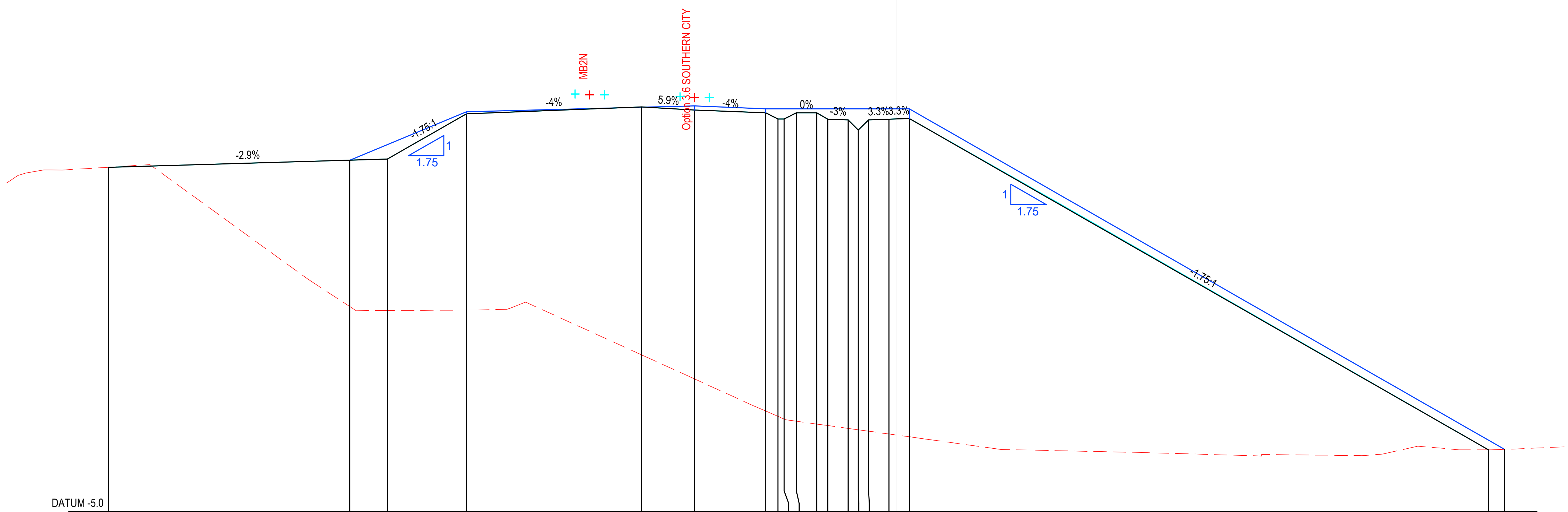
Vertical Scale (metres) 1:200 (2.5 0 2.5 5 7.5 10)

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	GEOTECHNICAL LONG SECTION ALONG THE CENTRELINE
drawing no:	N01031-GRW-DRG-GEO-0012-09

PLOT DATE: 10/07/2018 3:18:44 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA\2. TECHNICAL\GEOTECHNICS\1 PROJECTS\GEOTLCOV240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG



CUT STRINGS	IY02L		IA02L	ES02L	COGL	MB2S	ESR	ID21R	IE21R	IC21R	IB01R	EH1R	DGR	DMR	DJR	[illegible]	EVR	IAR	
SURCHARGE LEVELS		12.231		14.606		14.893	14.753										14.753		-1.832
DESIGN LEVELS	11.885		12.285	14.506	14.847	14.693	14.553	14.253	14.253	14.553	14.553	14.240	14.210	13.710	14.210	14.244	14.277	-1.979	
EXISTING GROUND LEVELS	11.885		4.858	4.88	2.683	1.503	-0.073	-0.334	-0.465	-0.573	-0.71	-0.786	-0.923	-0.992	-1.061	-1.198	-1.336	-1.979	
DESIGN OFFSETS	-28.791	-16.930	-15.086	-11.200	-2.600	0.000	3.500	4.100	4.400	5.000	6.000	6.547	7.547	8.047	8.547	9.547	10.547	38.995	42.720

40440

FOR COMMENT

PLOT DATE: 10/07/2018 3:18:56 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA\2. TECHNICAL\GEOTECHNICS\1.PROJECTS\GEOTLCOV24010\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG

no.	description	drawn	approved	date
[redacted]				

revision

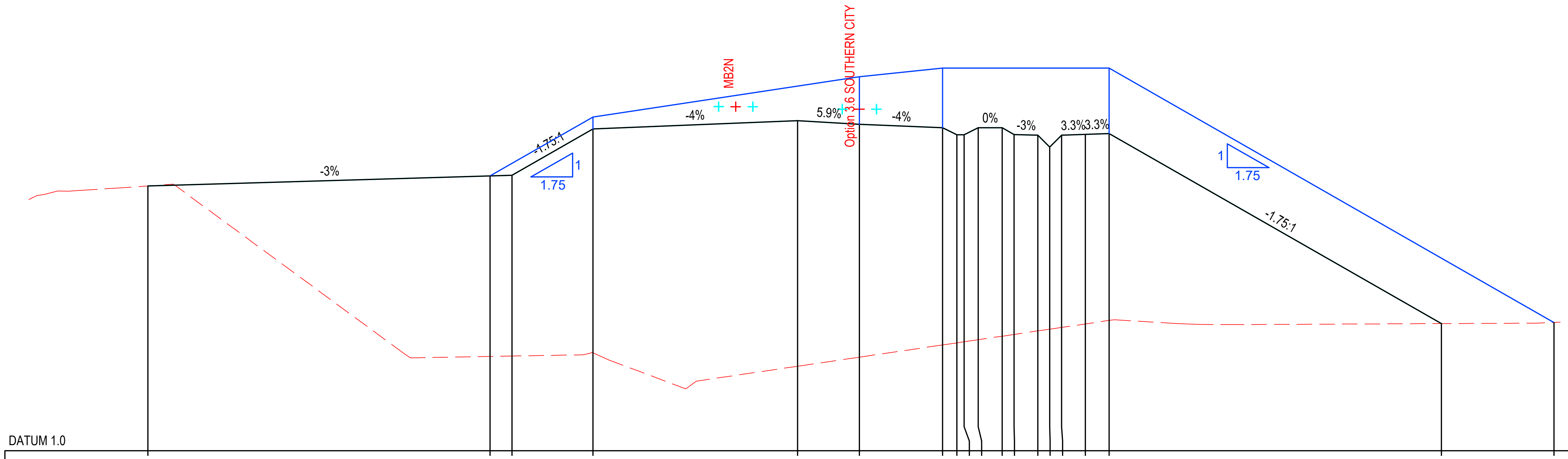
2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	[redacted]
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40440m
drawing no:	N01031-GRW-DRG-GEO-0014-09



	IY02L		IA02L	ES02L	COGL	MB2S	ESR	ID21R	IE21R	IC21R	IB01R	EH1R	DGR	DMR	DJR	[illegible]	EVR	IAR	
CUT STRINGS	IY02L		IA02L	ES02L	COGL	MB2S	ESR	ID21R	IE21R	IC21R	IB01R	EH1R	DGR	DMR	DJR	[illegible]	EVR	IAR	
SURCHARGE LEVELS		12.543		15.020		16.705	17.068										17.067		6.383
DESIGN LEVELS	12.119		12.571	14.518	14.862	14.708	14.568	14.268	14.268	14.568	14.568	14.280	14.250	13.750	14.250	14.284	14.317	6.337	
EXISTING GROUND LEVELS	12.119		4.972	5.11	4.545	4.927	5.441	5.529	5.573	5.661	5.808	5.882	6.029	6.102	6.175	6.322	6.469	6.337	
DESIGN OFFSETS	-29.915	-15.529	-14.607	-11.200	-2.600	0.000	3.500	4.100	4.400	5.000	6.000	6.503	7.503	8.003	8.503	9.503	10.503	24.468	29.200

40460

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

2.5 0 2.5 5 7.5 10

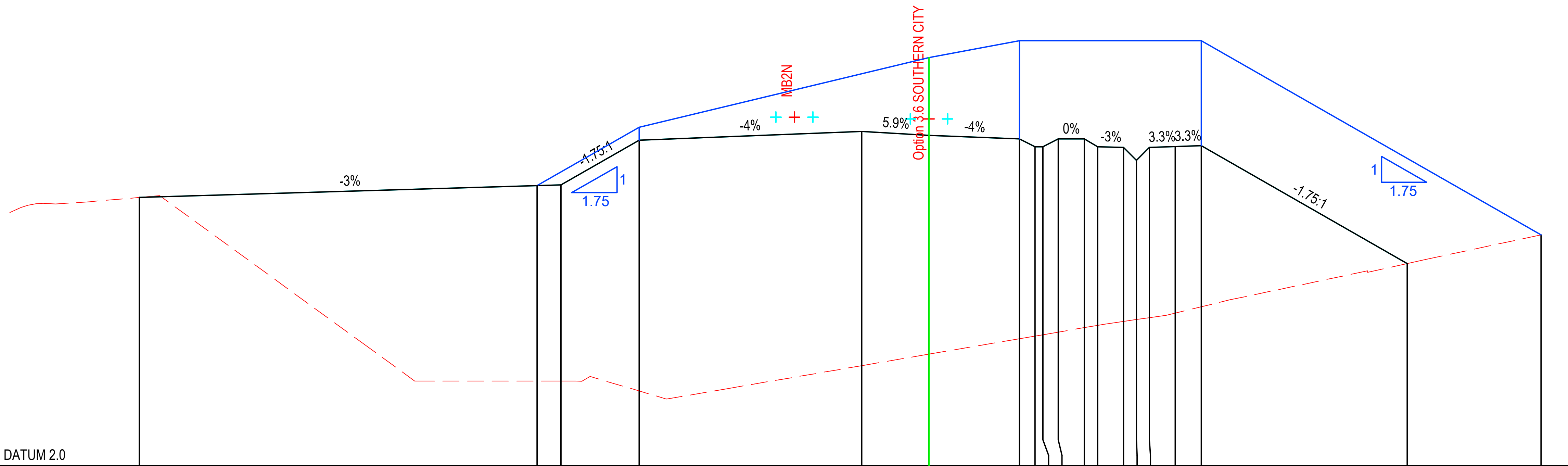
Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40460m
drawing no:	N01031-GRW-DRG-GEO-0015-09

PLOT DATE: 10/07/2018 3:19:07 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA\2. TECHNICAL\GEOTECHNICS\1.PROJECTS\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG



CUT STRINGS	IY02L		IA02L	ES02L	COGL	MB2S	ESR	ID21R	IE21R	IC21R	IB01R	EH1R	DGR	DMR	DJR	[illegible]	EVR	IAR	
SURCHARGE LEVELS		12.815		15.068		17.758	18.481										18.481		10.913
DESIGN LEVELS	12.359		12.842	14.568	14.912	14.758	14.618	14.318	14.318	14.618	14.618	14.320	14.290	13.790	14.290	14.324	14.357	9.807	
EXISTING GROUND LEVELS	12.359		5.264	4.884	5.861	6.305	6.902	7.005	7.056	7.159	7.329	7.418	7.57	7.643	7.716	7.895	8.136	9.807	
DESIGN OFFSETS	-30.525	-15.143	-14.220	-11.200	-2.600	0.000	3.500	4.100	4.400	5.000	6.000	6.521	7.521	8.021	8.521	9.521	10.521	18.484	23.654

40480

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

2.5 0 2.5 5 7.5 10

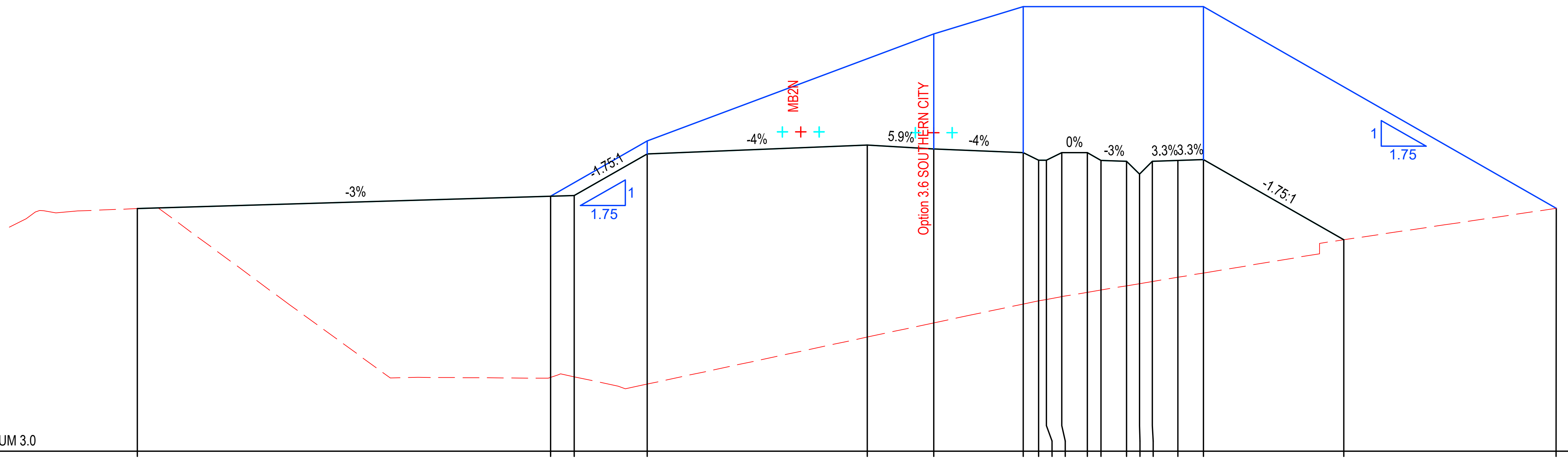
Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40480m
drawing no:	N01031-GRW-DRG-GEO-0016-09

PLOT DATE: 10/07/2018 3:19:19 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA\2. TECHNICAL\GEOTECHNICS\1.PROJECTS\GEOTLCOV240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG



CUT STRINGS	SURCHARGE LEVELS	DESIGN LEVELS	EXISTING GROUND LEVELS	DESIGN OFFSETS
IY02L		12.477	12.477	-31.135
	12.960			-14.975
IA02L		12.989	5.906	-14.052
ES02L	15.118	14.618	5.623	-11.200
COGL		14.962	7.458	-2.600
MB2S	19.308	14.808	8.013	0.000
ESR	20.368	14.668	8.751	3.500
ID21R		14.368	8.872	4.100
IE21R		14.368	8.929	4.400
IC21R		14.668	9.041	5.000
IB01R		14.668	9.207	6.000
EH1R		14.360	9.296	6.539
DGR		14.330	9.462	7.539
DMR		13.830	9.545	8.039
DJR		14.330	9.628	8.539
[illegible]		14.364	9.794	9.539
EVR	20.368	14.397	9.961	10.539
IAR		11.261	11.261	16.026
	12.488			24.328

40500

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

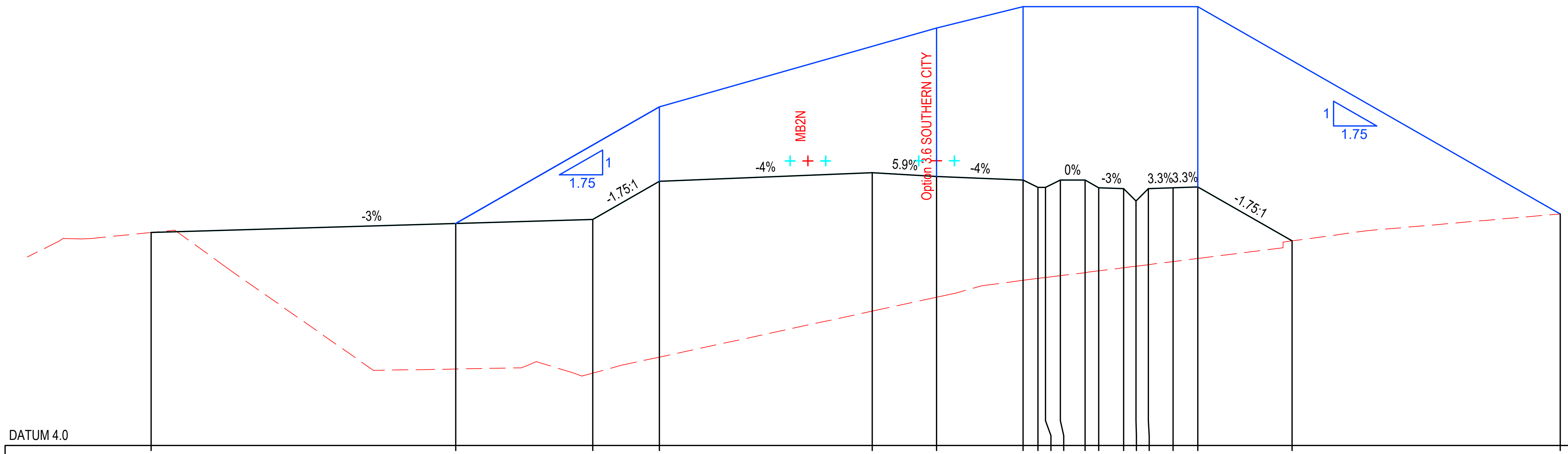
2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40500m
drawing no:	N01031-GRW-DRG-GEO-0017-09



CUT STRINGS	IY02L		IA02L	ES02L	COGL	MB2S	ESR	ID21R	IE21R	IC21R	IB01R	EH1R	DGR	DMR	DJR	BGR	EVR	IAR	
SURCHARGE LEVELS		12.963		17.668		20.858	21.718										21.718		13.348
DESIGN LEVELS	12.598		13.128	14.668	15.012	14.858	14.718	14.418	14.418	14.718	14.718	14.400	14.370	13.870	14.370	14.404	14.437	12.259	
EXISTING GROUND LEVELS	12.598		6.922	7.568	9.425	9.986	10.668	10.743	10.78	10.855	10.979	11.049	11.175	11.238	11.301	11.426	11.552	12.259	
DESIGN OFFSETS	-31.745	-19.434	-13.896	-11.200	-2.600	0.000	3.500	4.100	4.400	5.000	6.000	6.556	7.556	8.056	8.556	9.556	10.556	14.368	25.203

40520

FOR COMMENT

revision no.	description	drawn	approved	date
[redacted]				

2.5 0 2.5 5 7.5 10

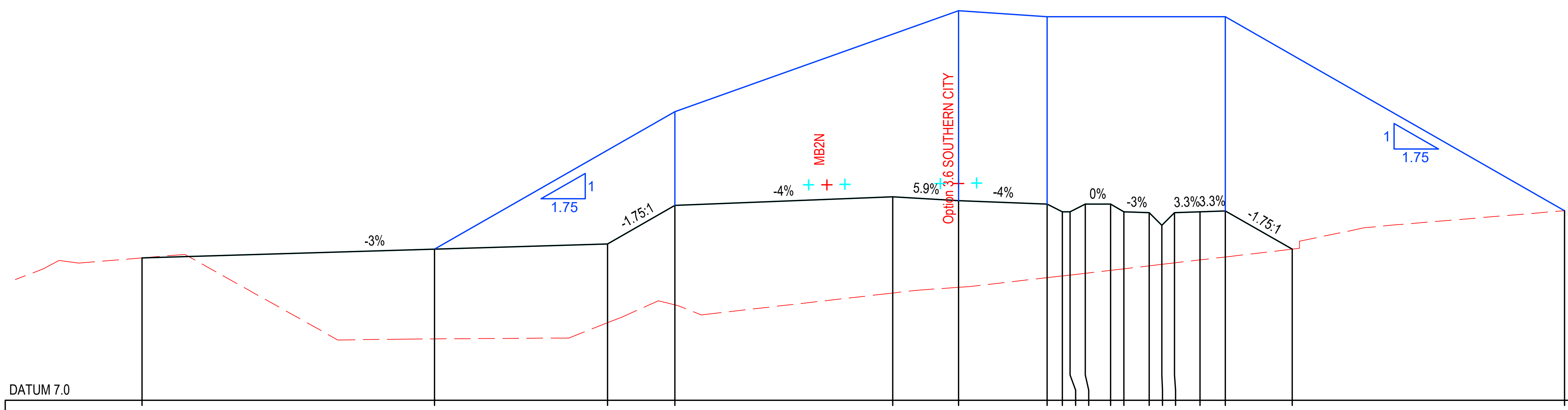
Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40520m
drawing no:	N01031-GRW-DRG-GEO-0018-09

PLOT DATE: 10/07/2018 3:19:21 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA\2. TECHNICAL\GEOTECHNICS\1.PROJECTS\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG



CUT STRINGS	SURCHARGE LEVELS	DESIGN LEVELS	EXISTING GROUND LEVELS	DESIGN OFFSETS
IY02L		12.621	12.621	-32.245
	12.967			-20.693
IA02L		13.173	10.061	-13.860
ES02L	18.392	14.693	10.767	-11.200
COGL		15.037	11.235	-2.600
MB2S	22.382	14.883	11.46	0.000
ESR	22.143	14.743	11.84	3.500
IB21R		14.443	11.908	4.100
IE21R		14.443	11.942	4.400
IC21R		14.743	12.012	5.000
IB01R		14.743	12.129	6.000
EH1R		14.440	12.191	6.529
DGR		14.410	12.308	7.529
DMR		13.910	12.366	8.029
DJR		14.410	12.424	8.529
BSER		14.444	12.541	9.529
EVR	22.141	14.477	12.658	10.529
IAR		12.966	12.966	13.173
	14.486			23.925

40540

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

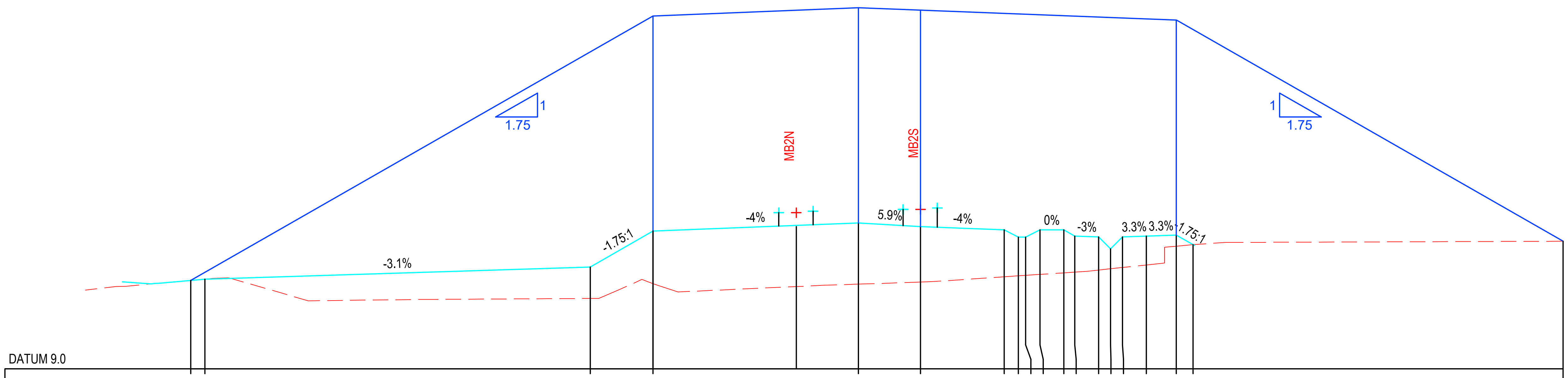
2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40540m
drawing no:	N01031-GRW-DRG-GEO-0019-09



DESIGN OFFSETS	CUT STRINGS	SURCHARGE LEVELS	DESIGN LEVELS	EXISTING GROUND
-30.557		12.701		
-29.967	IY02L		12.754	12.753
-13.828	IA02L		13.261	11.944
-11.200	ES02L	23.763	14.763	12.565
-2.600	COGL	24.107	15.107	12.543
0.000	[illegible]	24.007	14.953	12.631
3.500	ESR		14.813	12.85
4.100	ID21R		14.513	12.89
4.400	IE21R		14.513	12.91
5.000	IC21R		14.813	12.95
6.000	IB01R		14.813	13.016
6.456	EH1R		14.552	13.047
7.456	DGR		14.522	13.133
7.956	DMR		14.022	13.186
8.456	DJR		14.522	13.24
9.456	[illegible]		14.556	13.347
10.706	EVR	23.598	14.597	14.132
11.409	IAR		14.195	14.195
26.909		14.349		

40560

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

2.5 0 2.5 5 7.5 10

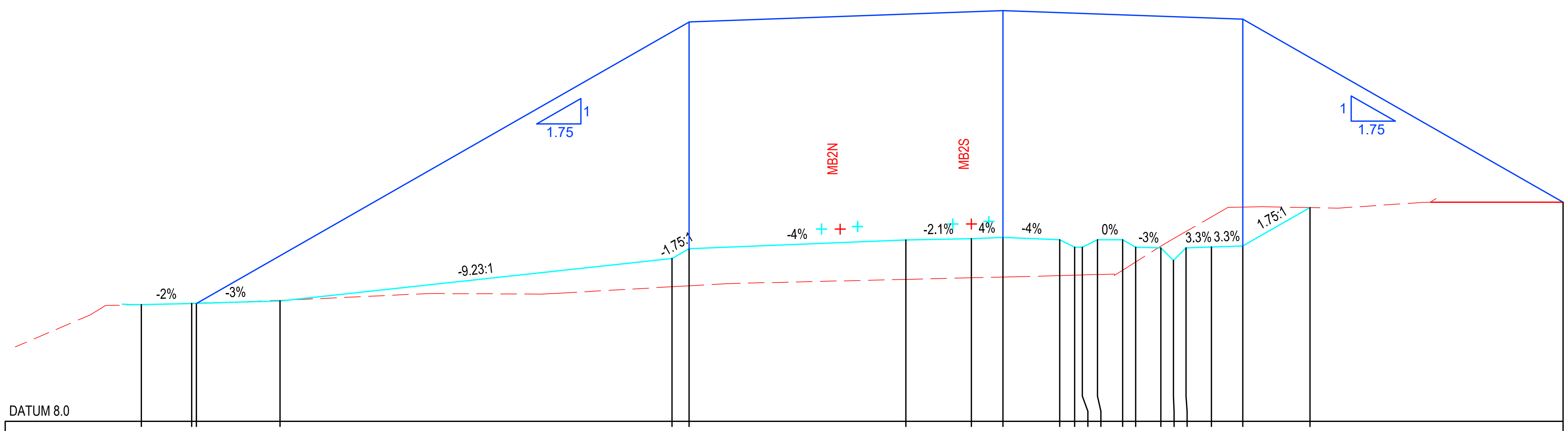
Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40560m
drawing no:	N01031-GRW-DRG-GEO-0020-09

PLOT DATE: 10/07/2018 3:19:33 PM DWG FILE: \\TTS79FS1.TT.LOCAL\DATA\2. TECHNICAL\GEOTECHNICS\1.PROJECTS\GEOTLCOV240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG



CUT STRINGS	SURCHARGE LEVELS	DESIGN LEVELS	EXISTING GROUND	DESIGN OFFSETS
IA09		12.631	12.631	-32.940
EH09		12.671	12.686	-30.940
	13.671			-30.753
EH09		12.774	12.803	-27.436
IA02L		14.459	13.339	-11.883
ES02L	23.850	14.849	13.379	-11.200
COGL		15.193	13.632	-2.600
MB2S		15.247	13.693	0.000
C1R	24.297	15.297	13.722	1.250
ESR		15.207	13.784	3.500
ID21R		14.907	13.8	4.100
IE21R		14.907	13.809	4.400
IC21R		15.207	13.825	5.000
IB01R		15.207	13.992	6.000
EH1R		14.911	14.32	6.519
DGR		14.881	14.95	7.519
DMR		14.381	15.245	8.019
DJR		14.881	15.532	8.519
[illegible]		14.914	16.106	9.519
EVR	23.956	14.956	16.499	10.769
IAR		16.479	16.479	13.435
	16.694			23.477

40580

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

2.5 0 2.5 5 7.5 10

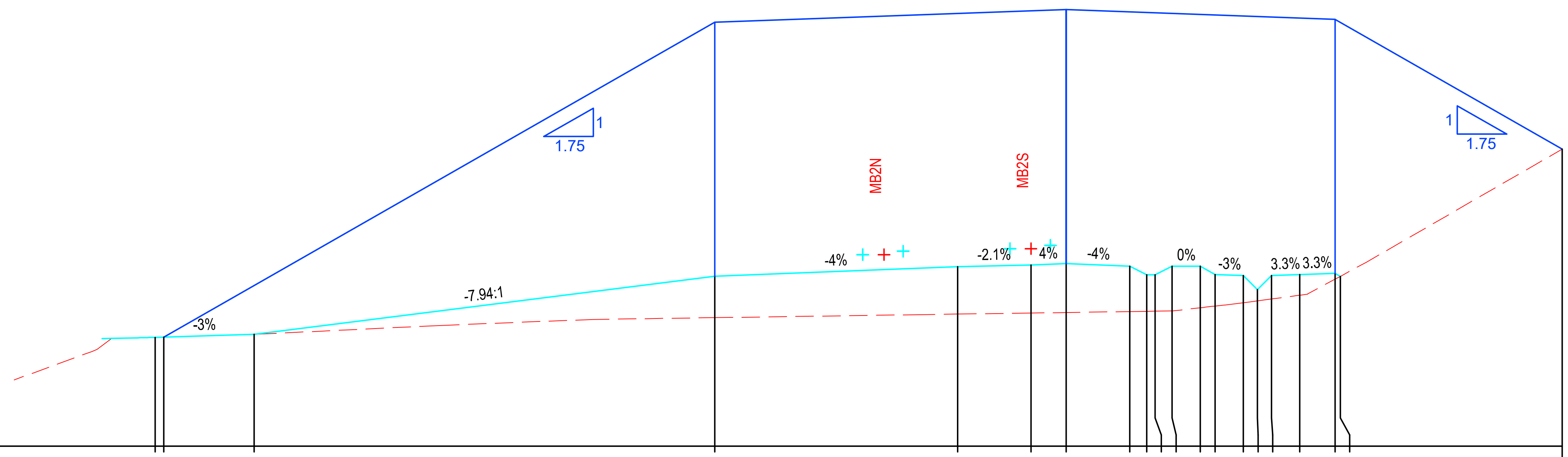
Scale (metres) 1:200

drawn	[redacted]
approved	[redacted]
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40580m
drawing no:	N01031-GRW-DRG-GEO-0021-09

PLOT DATE: 10/07/2018 3:19:45 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA\2. TECHNICAL\GEOTECHNICS\1.PROJECTS\GEOTLCOV\240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG



CUT STRINGS	SURCHARGE LEVELS	DESIGN LEVELS	EXISTING GROUND	DESIGN OFFSETS
EH09		12.854	12.855	-31.018
[illegible]	12.863			-30.716
EH09		12.960	12.96	-27.515
ES02L	24.015	15.015	13.551	-11.200
COGL		15.359	13.677	-2.600
MB2S		15.413	13.716	0.000
C1R	24.463	15.463	13.735	1.250
ESR		15.373	13.768	3.500
ID21R		15.073	13.777	4.100
IE21R		15.073	13.782	4.400
IC21R		15.373	13.791	5.000
IB01R		15.373	13.901	6.000
EH1R		15.075	13.962	6.522
DGR		15.045	14.081	7.522
DMR		14.545	14.148	8.022
DJR		15.045	14.215	8.522
[illegible]		15.079	14.348	9.522
EVR	24.120	15.120	14.914	10.772
AR		15.014	15.014	10.957
	19.523			18.817

40600

FOR COMMENT

PLOT DATE: 10/07/2018 3:19:57 PM DWG FILE: \\TTS779FS1.TT.LOCAL\DATA\2. TECHNICAL\GEOTECHNICS\1.PROJECTS\GEOTLCOV\240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG

revision	no.	description	drawn	approved	date
	[redacted]				

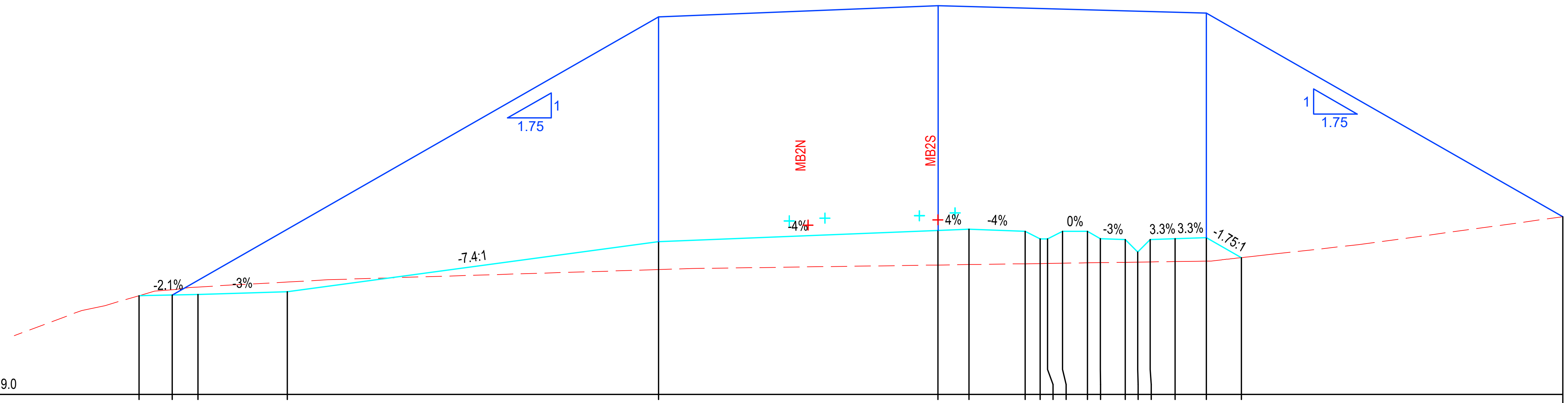
2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40600m
drawing no:	N01031-GRW-DRG-GEO-0022-09



CUT STRINGS	SURCHARGE LEVELS	DESIGN LEVELS	EXISTING GROUND	DESIGN OFFSETS
IA09		12.955	12.955	-32.027
	12.983			-30.696
EH09		13.004	13.304	-29.668
EH09		13.113	13.505	-26.082
ES02L	24.123	15.123	14.001	-11.200
MB2S	24.572	15.571	14.185	0.000
C1R		15.621	14.203	1.250
ESR		15.531	14.235	3.500
ID21R		15.231	14.244	4.100
IE21R		15.231	14.248	4.400
IC21R		15.531	14.257	5.000
IB01R		15.531	14.271	6.000
EH1R		15.238	14.278	6.513
DGR		15.208	14.293	7.513
DMR		14.708	14.3	8.013
DJR		15.208	14.307	8.513
[illegible]		15.241	14.322	9.513
EVR	24.283	15.283	14.341	10.763
IAR		14.480	14.48	12.169
	16.117			25.054

40620

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40620m
drawing no:	N01031-GRW-DRG-GEO-0023-09

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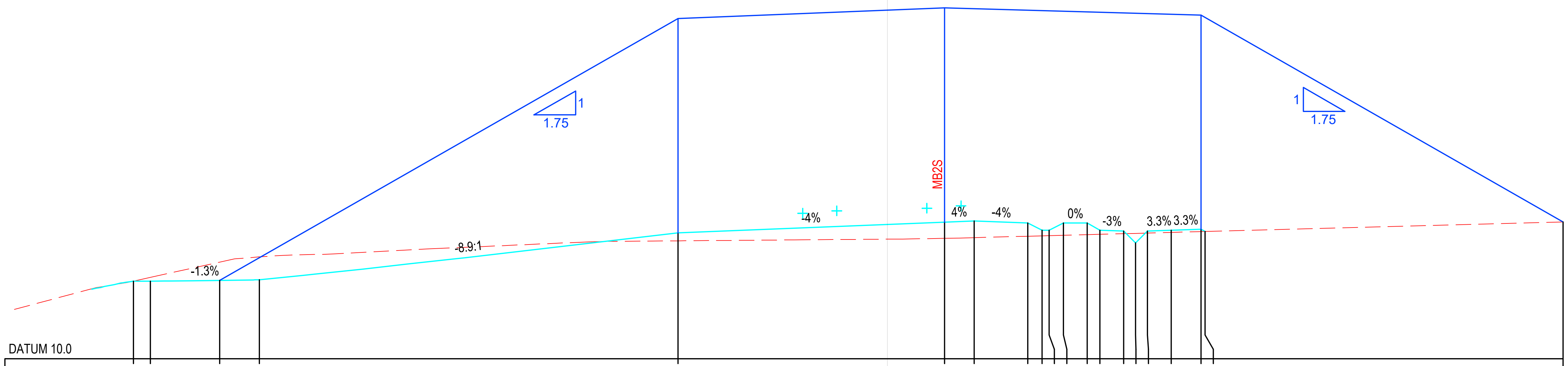

CUT STRINGS	SURCHARGE LEVELS	DESIGN LEVELS	EXISTING GROUND	DESIGN OFFSETS
IA09		13.261	13.261	-34.087
EH09		13.259	13.419	-33.370
	13.291			-30.456
EH09		13.318	14.281	-28.792
ES02L	24.294	15.294	14.955	-11.200
MB2S	24.742	15.742	15.061	0.000
C1R		15.792	15.088	1.250
ESR		15.702	15.148	3.500
ID21R		15.402	15.164	4.100
IE21R		15.402	15.172	4.400
IC21R		15.702	15.188	5.000
IB01R		15.702	15.214	6.000
EH1R		15.400	15.229	6.529
DGR		15.370	15.255	7.529
DMR		14.870	15.269	8.029
DJR		15.370	15.282	8.529
[illegible]		15.403	15.309	9.529
EVR	24.445	15.445	15.342	10.779
FAR		15.346	15.346	10.951
	15.748			25.999

40640

FOR COMMENT

revision				
no.	description	drawn	approved	date
[redacted]				

2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40640m
drawing no:	N01031-GRW-DRG-GEO-0024-09

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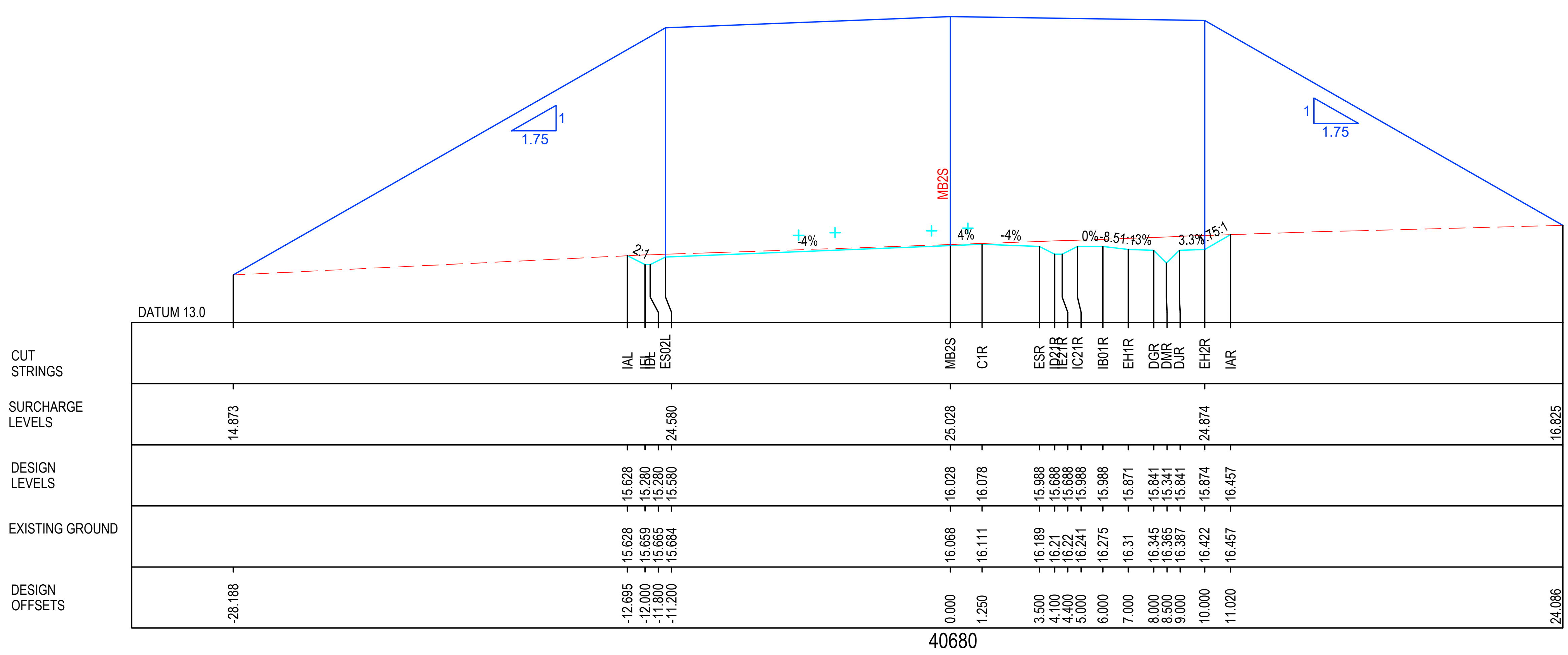

FOR COMMENT

revision	no.	description	drawn	approved	date
	[redacted]				

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Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40680m
drawing no:	N01031-GRW-DRG-GEO-0025-09

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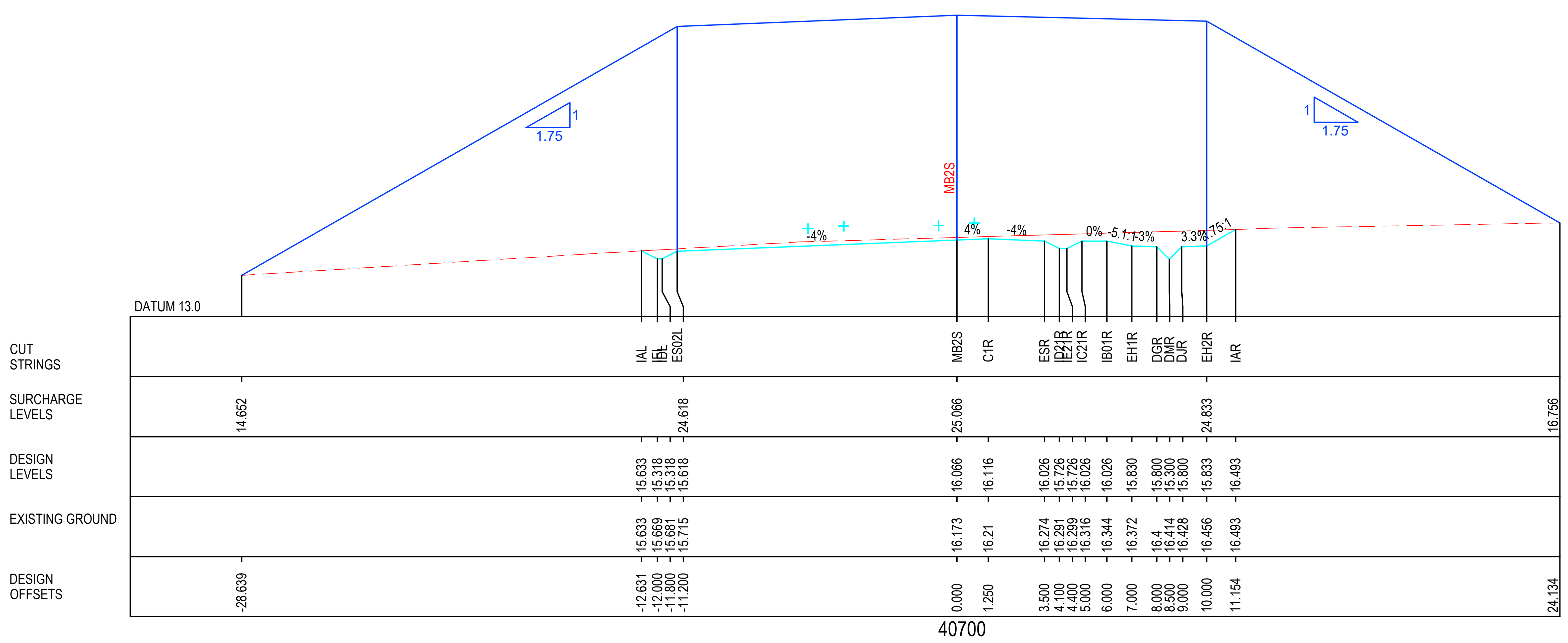


FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

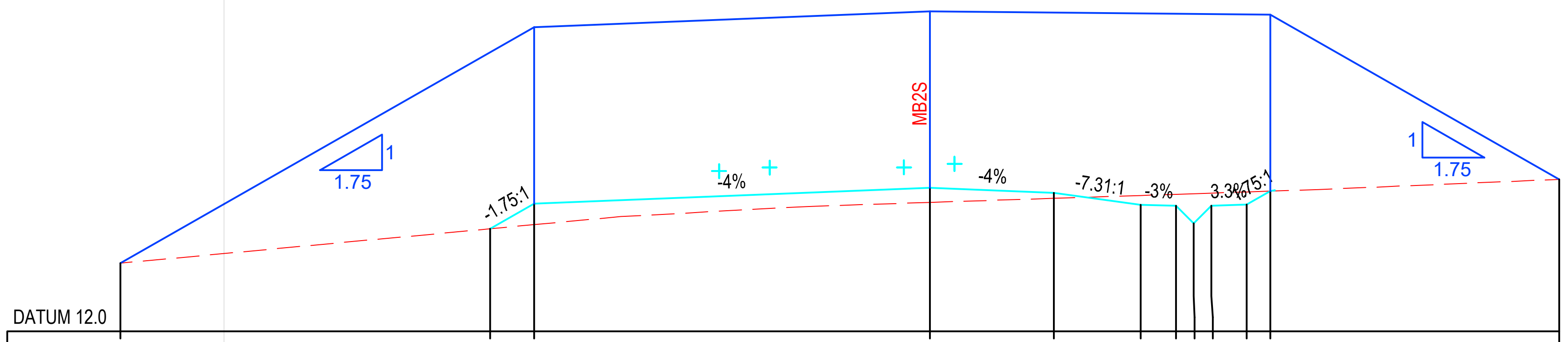
2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40700m
drawing no:	N01031-GRW-DRG-GEO-0026-09



CUT STRINGS		IAL	ES02L	MB2S	ESR	EH1R	DGR	DMR	DJR	EH2R	IAR	
SURCHARGE LEVELS	13.933		20.717	21.165							21.073	16.306
DESIGN LEVELS		14.907	15.617	16.065	15.925	15.589	15.559	15.059	15.559	15.592	15.972	
EXISTING GROUND		14.907	15.023	15.654	15.77	15.851	15.884	15.901	15.917	15.95	15.972	
DESIGN OFFSETS	-22.896	-12.442	-11.200	0.000	3.500	5.958	6.958	7.458	7.958	8.958	9.624	17.789

40720

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40720m
drawing no:	N01031-GRW-DRG-GEO-0027-09

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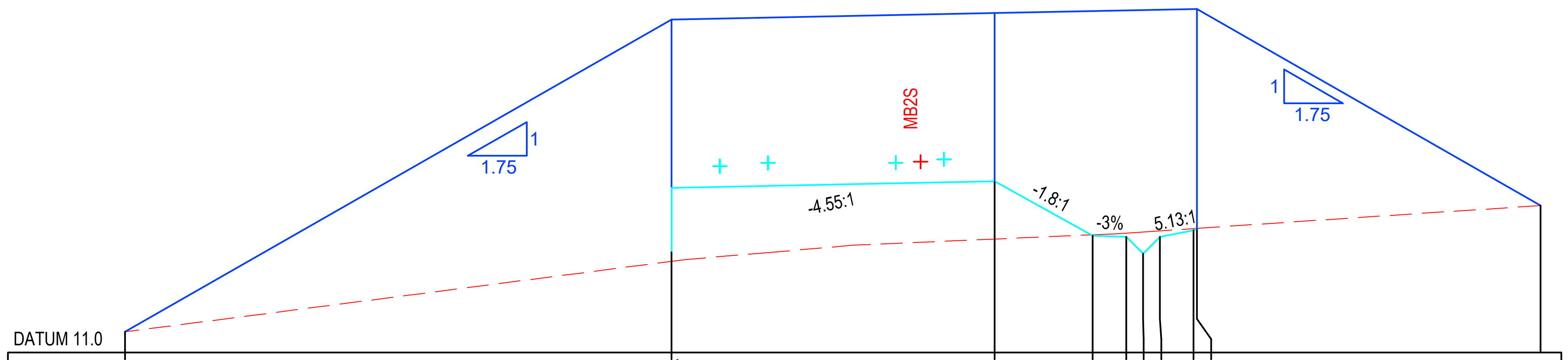


CUT STRINGS		[illegible]	ESR	EH1R	DGR	DMR	DJR	EH2R	AR	
SURCHARGE LEVELS	11.619	20.901	21.093						21.213	15.375
DESIGN LEVELS		13.982	16.093	14.474	14.444	13.944	14.444	14.639	14.693	
EXISTING GROUND		13.715	14.377	14.504	14.553	14.587	14.62	14.687	14.693	
DESIGN OFFSETS	-23.643	-7.400	2.200		6.114	6.614	7.114	8.114	8.209	8.209

40740

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

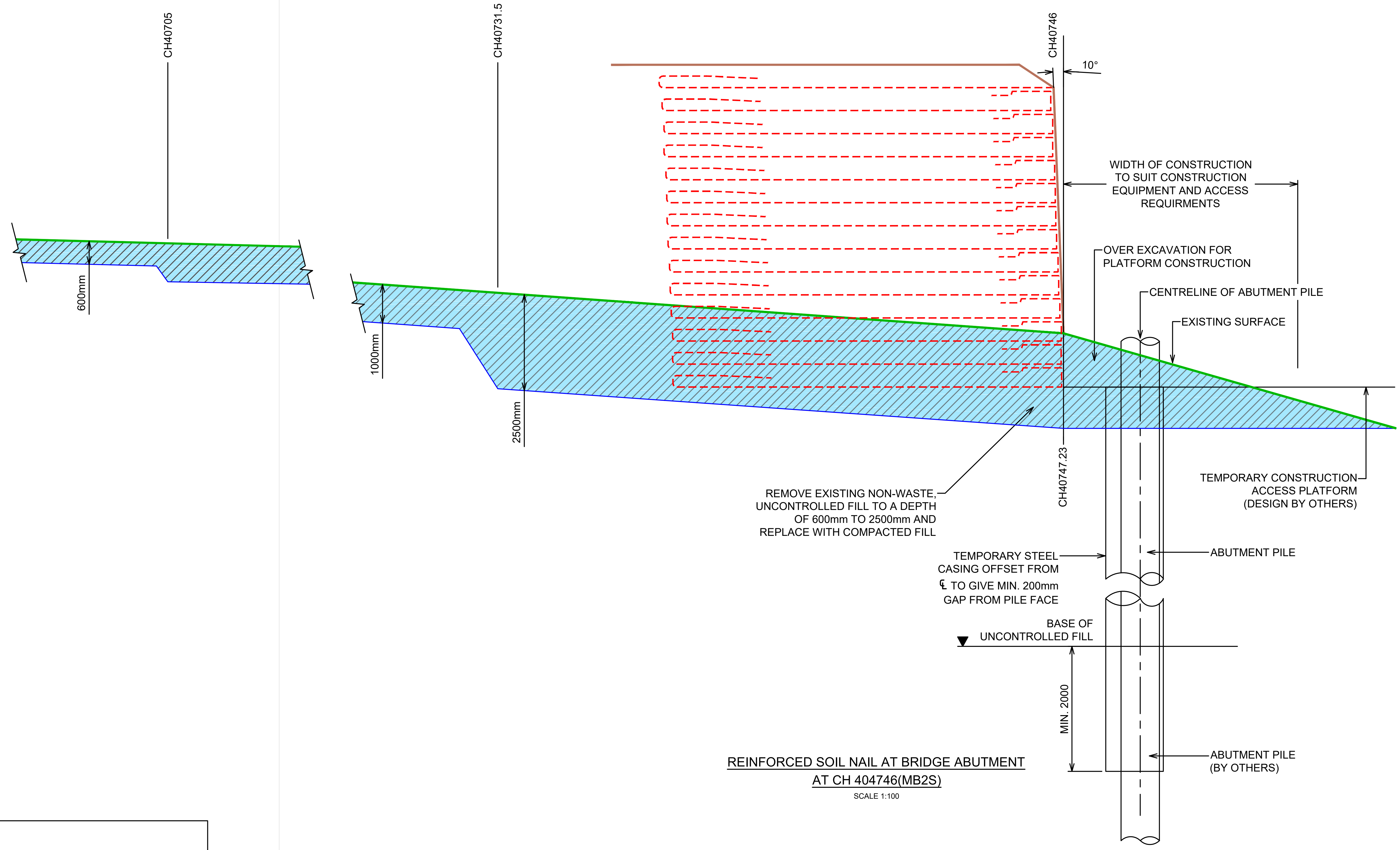
2.5 0 2.5 5 7.5 10

Scale (metres) 1:200

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	MB2S - CHAINAGE - 40740m
drawing no:	N01031-GRW-DRG-GEO-0028-09



REINFORCED SOIL NAIL AT BRIDGE ABUTMENT
AT CH 404746(MB2S)
SCALE 1:100

LEGEND

GEOGRID LAYER
- ULTIMATE TENSILE STRENGTH = 100kN/m
- DESIGN STRENGTH = 50KN/M

FOR COMMENT

no.	description	drawn	approved	date
[redacted]				

revision

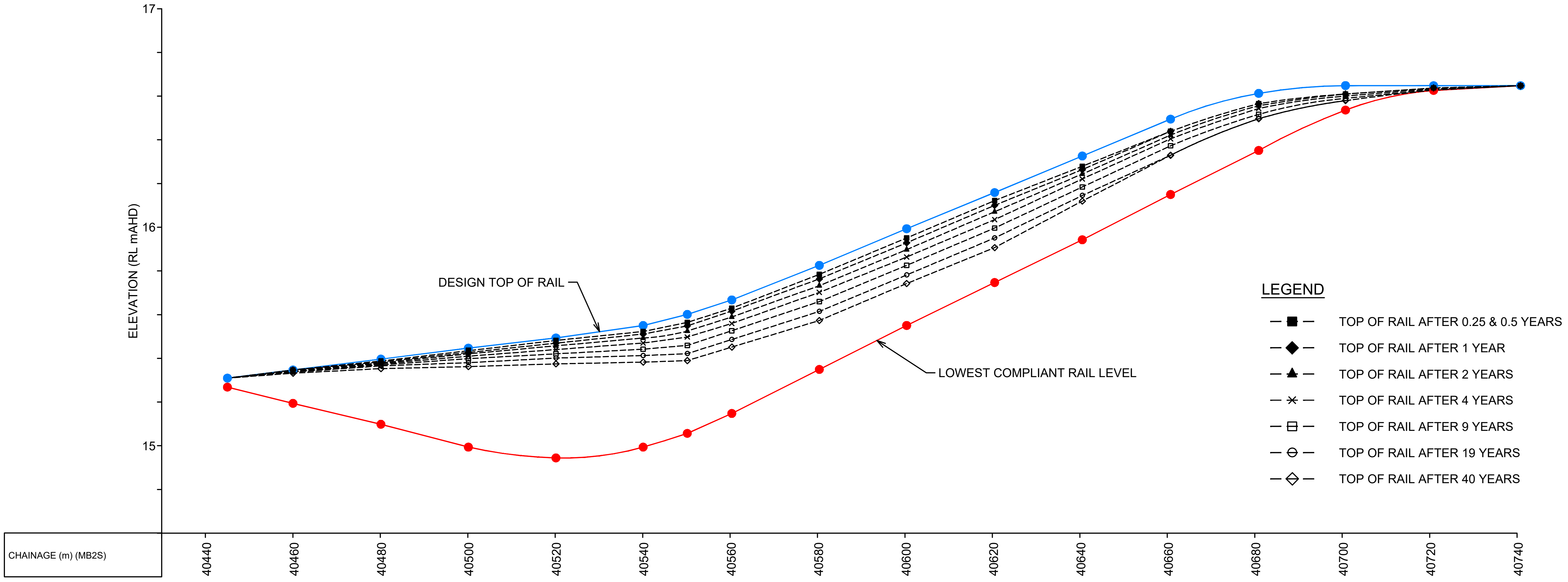
1 0 1 3 5
Horizontal Scale (metres) 1:100

1 0 1 3 5
Vertical Scale (metres) 1:100

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	TYPICAL CROSS SECTIONS OF REINFORCED EMBANKMENT
drawing no:	N01031-GRW-DRG-GEO-0029-09



LEVELS OF TOP OF RAILS AT INTERVENTIONS (MB2S) AFTER READJUSTMENT

Table 5 - Minimum Reduced Levels of Rail for Interventions and Trigger (MB2S)

Chainage (MB2S)	Existing Ground Level (RL, mAHD)	Design Top of Rail Level (RL, mAHD)	Lowest Compliant Rail Level (RL, mAHD)	Minimum Reduced Levels (RL, mAHD) of Rail for Interventions[(1)] and Trigger before Readjustment													
				Intervention 0.25 year	Adjusted top of Rail after 0.25 years	Intervention 0.5 year	Adjusted top of Rail after 0.5 years	Intervention 1 year	Adjusted top of Rail after 1 year	Intervention 2 years	Adjusted top of Rail after 2 years	Intervention 4 years	Adjusted top of Rail after 4 years	Intervention 9 years	Adjusted top of Rail after 9 years	Intervention 19 years	Adjusted top of Rail after 19 years	Intervention 40 years	Adjusted top of Rail after 40years
40460	4.268	15.344	15.194	15.328	15.344	15.328	15.344	15.334	15.344	15.330	15.343	15.328	15.342	15.323	15.341	15.319	15.337	15.310	15.332
40480	5.945	15.394	15.094	15.368	15.390	15.368	15.390	15.374	15.388	15.365	15.384	15.359	15.380	15.349	15.374	15.340	15.366	15.328	15.353
40500	8.142	15.444	14.994	15.408	15.432	15.408	15.432	15.410	15.427	15.396	15.418	15.384	15.408	15.366	15.395	15.347	15.379	15.331	15.362
40520	10.086	15.494	14.945	15.451	15.478	15.451	15.478	15.452	15.469	15.432	15.455	15.414	15.440	15.391	15.421	15.369	15.400	15.345	15.375
40540	11.323	15.554	14.996	15.497	15.523	15.497	15.523	15.486	15.511	15.462	15.492	15.438	15.470	15.405	15.442	15.374	15.412	15.341	15.383
40560	12.78	15.667	15.147	15.609	15.629	15.609	15.629	15.594	15.613	15.564	15.588	15.534	15.559	15.494	15.523	15.454	15.485	15.412	15.452
40580	13.684	15.827	15.347	15.772	15.783	15.772	15.783	15.749	15.764	15.717	15.735	15.683	15.702	15.639	15.660	15.594	15.616	15.547	15.573
40600	13.661	15.993	15.547	15.941	15.948	15.941	15.948	15.916	15.930	15.885	15.901	15.851	15.868	15.808	15.826	15.763	15.781	15.715	15.742
40620	13.992	16.159	15.747	16.108	16.120	16.108	16.120	16.089	16.100	16.056	16.070	16.022	16.036	15.978	15.995	15.934	15.952	15.887	15.907
40640	14.904	16.325	15.947	16.269	16.280	16.269	16.280	16.246	16.269	16.221	16.247	16.194	16.220	16.157	16.186	16.119	16.148	16.078	16.120
40660	15.779	16.491	16.147	16.431	16.439	16.431	16.439	16.403	16.436	16.385	16.420	16.364	16.400	16.332	16.370	16.300	16.330	16.266	16.330
40680	16.205	16.609	16.347	16.549	16.554	16.549	16.554	16.517	16.558	16.507	16.550	16.493	16.536	16.468	16.512	16.454	16.496	16.441	16.496
40700	16.245	16.645	16.534	16.584	16.600	16.584	16.600	16.562	16.608	16.556	16.605	16.536	16.599	16.534	16.534	16.534	16.534	16.534	16.534
40720	15.767	16.645	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633	16.633

Notes:

[(1)] Assumed a preloading period of 3 months and a rail construction period of 4 months from commencement of surcharge remmoval or end of dynamic compaction to practical completion.

FOR COMMENT

revision	no.	description	drawn	approved	date
	[redacted]				

10 0 10 30 50

Horizontal Scale (metres) 1:1000

0.25 0 0.25 0.5 0.75 1

Vertical Scale (metres) 1:20

drawn	[redacted]
approved	
date	10 / 07 / 18
scale	AS SHOWN
original size	A3

coffey
A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	PROPOSED READJUSTED VERTICAL ALIGNMENT AT INTERVENTIONS
drawing no:	N01031-GRW-DRG-GEO-0031-09

PLOT DATE: 10/07/2018 4:23:13 PM DWG FILE: \\TTS79FS1.TT.LOCAL\DATA2_TECHNICAL\GEOTECHNICS\1 PROJECTS\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-BA-N01031-09.DWG

Appendix B – Responses to Comments of DCD and FD Design Review

DESIGN REVIEW RECORD (DRR)

ARCADIS

CLIENT
Sydney Intermodal Terminal Alliance (SIMTA)

CONTRACTOR DESIGN PACKAGE NUMBER
RALP No.1

CONTRACTOR DESIGN PACKAGE TITLE
GWS Landfill Ground Treatment

Design Report: N01031-GRW-DRP-GEO-0001[02]
Design Specification: N01031-GRW-GEO-SPE-0001[02]
Drawing List: Refer to Appendix A of Design Report (N01031-GRW-DRP-GEO-0001-02)

COMPLIANCE STATUS
O Observation / Comment
D From info currently provided not able to determine whether design / proposal is compliant.
N Non-Compliant
M Minor non-compliance

RESPONSE STATUS
O Open
C Closed
CA Closed against this package but subject to action in another package
CS Closed SUBJECT TO additional action / information

No.	Rev	Reviewer	Initial Comment Date	Discipline	Document Reference	Reviewer Comment	Contract / Standard Requirement Reference	Compliance Status	D&C Contractor Response	Initial Response Date	Response Status	Reviewer Comment on Response	Date Comment Closed	Additional D&C Contractor Response	Additional Reviewer Comment	Additional D&C Contractor Response
1	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Please clarify what is the treatment for the embankment immediately behind the piled bridge abutment. Is there any allowance for a transfer slab to limit any differential settlement.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5	N	As the abutment is away from the landfill area anticipated settlement just behind the abutment between interventions is in the order of 5-10mm. Approach will be designed according to ARTC requirements. These details will be cited in our Final Design submission.	24-Jun-2016	C		31-Oct-2016		*Comment not closed. Refer to 100% Final Design comments*	Refer 100% Final Design comments
2	A		08-Jun-16	Geotech	GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Second paragraph of Section 3 of the memo indicates that 7 months will be required for the construction of the embankment and surcharging. Please clarify if this time of period required would have any impact on the project program.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	7 Months included in CPB Final Offer Program	24-Jun-2016	C	Response covered in Arcadis GI Memo [11/01/17] Refer to Arcadis Letter to client.	11-Jan-2017	7 Months included in CPB construction program	*It is not clear what the DC program is based on the revised CPB design.*	The CPB construction program is in alignment with the GWS landfill ground treatment package
3	A		08-Jun-16	Geotech	GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Section 3 of the Memo refers a post-construction settlement of 100mm or differential settlement of 0.25% as an intervention threshold. Please provide supporting document that demonstrate these values are suitable for live railway tracks.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5	M	Intervention threshold are proposed based on Coffey experience in similar rail project. Coffey have previously designed haul rail tracks with a predicted post construction settlement of over 500 mm (Example: Hexham Relief Roads Project - Client ARTC). The intervention period was developed based on 0.25% differential settlement grade for this project. These thresholds to be discussed with SIMTA for agreement.	24-Jun-2016	C	As discussed CPB to provide evidence of ARTC's acceptance of the 500mm designed settlement. Are CPB to continue design development to further reduce likely settlement by integrating other improvements in additional to Dynamic Replacement? [11/01/17] Refer to Arcadis' comments on AFC.	11-Jan-2017	Refer Appendix H of Final design (FD) report for details on a project (Hexham Relief road) where predicted post construction settlement is over 500 mm. CPB's D&C Contract is based on Dynamic Replacement only.	*The case was not natural ground which is relatively more uniform. Comment not closed. Refer to 100% Final Design comments*	Original comment and our responses are on adopted settlement criteria. Performance of rail track depends on the settlement of tracks irrespective of type of ground. The PPR requires under section 1.1.3(e)(ii) "Expected long term post construction settlements of top of the surface (in areas of fill or virgin material) are equal or less than 1:400 over 30 years". SIMTA are to advice any requirement beyond the PPR Refer 100% Final Design comments
4	A		08-Jun-16	Geotech	GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Please clarify why the assessed secondary consolidation settlement 0.5 year after track commissioning shows such a large range. Please clarify if any creep settlement of the newly placed engineer fill has been considered or not.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Variation in the transverse direction is due to the variability in landfill thickness and surcharge thickness. Variation in longitudinal direction is due to variation in surcharge thickness. Surcharge thickness was varied to create sufficient primary consolidation so that secondary consolidation will be within intervention thresholds. Creep of engineered fill has been considered in the post construction settlement.	24-Jun-2016	C	Response covered in Arcadis GI Memo	31-Oct-2016	Variation in the transverse direction is due to the variability in landfill thickness and surcharge thickness. Variation in longitudinal direction is due to variation in surcharge thickness. Surcharge thickness was varied to create sufficient primary consolidation so that secondary consolidation will be within intervention thresholds. Creep of engineered fill has been considered in the post construction settlement assessed in FD stage (100%) and reported in Section 4.4 of FD Report.	*Closed refer to similar comments on 100% Final Design*	Refer 100% Final Design comments
5	A		08-Jun-16	Geotech	GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	The predicted settlements presented in Section 3 of the Memo also expect to have interventions at 0.5, 1.3,4.4, 19 and 40 years. From CPB memo wording this implies that an on-going settlement is likely to be 500mm post track construction?? This can induce significant operating risks for the client and at the interface between bridge and the approach embankment. Please clarify. Assessed Secondary consolidation is to be provided for 40 year case.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	Proposed interventions period are to maintain intervention threshold (i.e. 100mm settlement or 0.25% differential settlement). Approach has been designed (i.e. transition) to maintain the proposed intervention threshold. Post construction settlement at approach is limited to 5 to 10 mm within given intervention period. Hence, total PCS is in the range of 50 to 100mm. Away from the approach the total PCS can be in the order of 400 to 500mm while maintaining the intervention threshold. Assessed post construction settlement for 40 year will be included in detail in our Final Design submission.	24-Jun-2016	C	1. Design is not compliant. CPB to demonstrate no compliant design solution is achievable at this location. 2. Demonstrate a viable drainage solution is achievable following these significant settlement. Design does not make any reference to how capping requirements of the site will be achieved following the proposed settlement. [11/01/17] Refer to Arcadis' comments on AFC.	11-Jan-2017	1. CPB's D&C Contract is based on the current design solution at this location. 2. Details are included in the final design report (refer Section 6.4 of Final Design Report) regarding the impact on drainage due to the post construction settlement of about 400 mm over 40 years period	*Comment not closed. Refer to 100% Final Design comments*	Refer 100% Final Design comments
6	A		08-Jun-16	Geotech	GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	The land fill thickness shown on cross-section in Attachment A appears different from that shown on the long section in Attachment C.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	It is noted that there is a slight difference in the landfill thickness within the new landfill area due to the alignment change. This will be updated accordingly in the final design based on the most up-to-date alignment.	24-Jun-2016	C		29-Jun-2016			
7	A		08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	Please clarify what is the treatment for the embankment foundation immediately behind the piled viaduct abutment. Is there any allowance for a transition slab to limit differential settlement?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	As the abutment is away from the landfill area anticipated settlement just behind the abutment between interventions is in the order of 5-10mm. Approach will be designed according to ARTC requirements. These details will be cited in our Final Design submission.	24-Jun-2016	C		29-Jun-2016			
8	A		08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	Second paragraph of Section 4 of the memo indicates that 7 months (6months plus one month for filling) will be required for the construction of the embankment and surcharging. Please clarify if this time of period required would have any impact on the project program.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	7 Months included in CPB Final Offer Program.	24-Jun-2016	C		29-Jun-2016		*CPB cost allowance does not infer that SIMTA will accept a non-compliant design solution.*	
9	A		08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	Section 4 of the Memo refers to a differential settlement of 0.25% as an intervention threshold. Please provide supporting document that demonstrate these values are suitable for live railway tracks. CPB to provide a summary table of design criteria	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Intervention threshold are proposed based on Coffey experience in similar rail project. Coffey have previously designed a haul rail tracks with a predicted post construction settlement of over 500 mm (Example: Hexham Relief Roads Project - Client ARTC). The intervention period was developed based on 0.25% differential settlement grade for this project. These thresholds to be discussed with SIMTA for agreement.	24-Jun-2016	C	As discussed CPB to provide evidence of ARTC's acceptance of the 500mm designed settlement. Are CPB to continue design development to further reduce likely settlement by integrating other improvements in additional to Dynamic Replacement? [11/01/17] Refer to Arcadis' letter to client.	11-Jan-2017	Refer Appendix H of Final design (FD) report for details on a project (Hexham Relief road) where predicted post construction settlement is over 500 mm. CPB's D&C Contract is based on Dynamic Replacement only.	*CPB cost allowance does not infer that SIMTA will accept a non-compliant design solution.*	CPB offer is based on Dynamic Compaction with a maximum post construction settlement of 500mm
10	A		08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	The predicted settlements presented in Table 5 of the Memo also expect to have interventions at 0.5, 1.1, 2.4, 5, 11, 24 and 40 years, with the predicted settlement of the order of 500mm post track construction. This can induce significant operating risks for the client overall and at the interface between bridge and the approach embankment. Please clarify. As an example for a similar previous project - the maximum allowable vertical displacement over 40 years was 150mm.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	Coffey have previously designed a haul rail tracks with a predicted post construction settlement of over 500 mm (Example: Hexham Relief Roads Project - Client ARTC). The intervention period was developed based on 0.25% differential settlement grade for this project. These thresholds are to be discussed with client and agreed on. In addition, refer our response to Item 5 above.	24-Jun-2016	C	1. Design is not compliant. CPB to demonstrate no compliant design solution is achievable at this location. 2. Demonstrate a viable drainage solution is achievable following this significant settlement.	21-Oct-2016	1. CPB's D&C Contract is based on the current design solution at this location. 2. Details are included in the final design report (refer Section 6.4 of Final Design Report) regarding the impact on drainage due to the post construction settlement of about 400 mm over 40 years period	*Closed. Transferred it to similar comments in Item 5.*	
11	A		08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	Use of dynamic compaction and a trial embankment has been proposed in the memo. Please clarify how much time is required to undertake this trial and to build confidence for the final design. There is no discussion in the report on what additional actions would be undertaken if settlement encountered during the trial is greater than expected.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	DC trial is expected to take about 2 weeks and allow one week to assess the performance. DC trial is to assess the energy requirement to achieve improvement depth of about 10 m. If landfill is more compressible, depth of improvement is likely to be more (Van Impe and Bouazza, 1996), hence the overall effect of DC is expected to be similar. In addition, compressibility parameters assessed from Coffey DC trial in 1984 within Glenfield tip are lower than the compressibility parameters adopted in our design. The slightly higher compressibility have been adopted considering likely variability in landfill material. As detailed in Section 5, DC production data will also be compared with DC trial data to assess the variability in ground condition and hence the settlement performance be refined and hence revise the interventions.	24-Jun-2016	C	CPB to provide actual data from previous works in GWS to support design assumption of achieving 10m of ground improvement. CPB response states that interventions will be revised but makes no statement regarding revising the design following the trial. During discussions CPB stated that other ground improvement options such as 'Rigid Inclusions" were discounted during the tender phase as the scope was required to be a lump sum submission and they didn't have a sufficient level of confidence that significant obstructions would not exist that would increase considerable the number of rigid inclusions required. considering this how has this impacted the assessment that 10m of ground improvement will be achieved by dynamic replacement. [11/01/17] Subject to field trial and refer to Arcadis' letter to client.	11-Jan-2017	The summary of the DC trial carried out in 1984 is included as Appendix H of the FD Report. As proposed in Section 5 and Appendix C of the FD Report, we proposed to carry out refinement to settlement assessments after the DC production if some variability in ground condition is encountered. Once the monitoring data is available (during waiting period), we will assess the post construction settlement and refine intervention period. The process control diagrams are also provided in the drawings (No. GEW-DRG-GEO-0009-C) of geotechnical ground treatment design package. Obstruction of rigid inclusion can occur due to localised hard objects within the land fill as the drilling and concreting is carried out into the landfill. DC work involve pounding the granular platform surface and hence transferring energy trough landfill to densify the landfill. Presence of localised hard object will not inhibit in transferring the energy in to landfill below the localised hard object, in fact it may assist the energy transfer.	*Evidence of 1984 project using the dynamic compaction by Coffey was for natural ground improvement which is more uniform and it was completed in 2014, ie 2 years after construction. The long term settlement behaviour under train loading has not been realised yet. A such this is not reliable from a long term performance perspective.*	**Reviewers original comment** Use of dynamic compaction and a trial embankment has been proposed in the memo. Please clarify how much time is required to undertake this trial and to build confidence for the final design. There is no discussion in the report on what additional actions would be undertaken if settlement encountered during the trial is greater than expected. **Our Response** DC trial is expected to take about 2 weeks and allow one week to assess the performance. DC trial is to assess the energy requirement to achieve improvement depth of10 m. If DC improvement depth is less than adopted 10 m, surcharging will be proposed as a contingency measure.
12	A		08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	Figure 1 in Attachment A shows a zone with FILL of possible sandstone and boulders. Please clarify if adequate geotechnical investigation has been undertaken to confirm the material within this zone.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Shallow trial pits and Geophysical investigations has been carried out. Performance will be observed during DC production and assess the compressibility of the zone. In addition, observational approach (i.e. monitoring) during construction will be carried out to further refine the post construction settlement performance.	24-Jun-2016	C	What monitoring is proposed to ensure optimised performance can be demonstrated. [11/01/17] Subject to field trial and refer to Arcadis' letter to client.	11-Jan-2017	Refer to Section 5 of FD Report and the ground treatment drawings (GRW-DRG-GEO-0010-C). Settlement plates and survey monuments will be installed and monitored as part of the observational approach.	*Please clarify at what time line the instruments will be installed and monitored. Details should be provided.*	Timeline of instrument installation is provided in the construction staging (Ref: drawing GRW-DRG-GEO-007-C1). Drawing will be amended to add clarity on installation timing. Instrumentation and Monitoring specification will be prepared separately and issued during IFC submission.
13	A		08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Please confirm differential settlement at abutments of GWS Viaduct. There appears to be no consideration of design integration between the Viaduct structure and GWS landfill ground treatment design packages. It should be noted that Viaduct drawing N01031-PWD-DRG-BRD-0010 states "TRANSITION DETAIL FROM VIADUCT TO LANDFILL TO BE CO-ORDINATED WITH COFFEY'S GROUND TREATMENT PACKAGE IN THIS AREA"	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 1.1.3, 2.1, Appendix 8 Cl. 2.5	M	As the abutment is away from the landfill area, anticipated settlement just behind the abutment between interventions is in the order of 5-10mm. Transition zone has been designed to maintain intervention thresholds. Design integration will be addressed in our Final Design submission.	24-Jun-2016	C		01-Jul-2016			
14	A		08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	Provide summary of preload volume and how material will be managed to avoid disposal.		O	Summary of preload/surcharge volume will be included in the Final Design submission.	24-Jun-2016	C		01-Jul-2016			

DESIGN REVIEW RECORD (DRR)

ARCADIS

CLIENT
Sydney Intermodal Terminal Alliance (SIMTA)
CONTRACTOR DESIGN PACKAGE NUMBER
RALP No.1
CONTRACTOR DESIGN PACKAGE TITLE
GWS Landfill Ground Treatment

Design Report:	N01031-GRW-DRP-GEO-0001[02]
Design Specification:	N01031-GRW-GEO-SPE-0001[02]
Drawing List:	Refer to Appendix A of Design Report (N01031-GRW-DRP-GEO-0001-02)

COMPLIANCE STATUS
O Observation / Comment
D From info currently provided not able to determine whether design / proposal is compliant.
N Non-Compliant
M Minor non-compliance

RESPONSE STATUS
O Open
C Closed
CA Closed against this package but subject to action in another package
CS Closed SUBJECT TO additional action / information

No.	Rev	Reviewer	Initial Comment Date	Discipline	Document Reference	Reviewer Comment	Contract / Standard Requirement Reference	Compliance Status	D&C Contractor Response	Initial Response Date	Response Status	Reviewer Comment on Response	Date Comment Closed	Additional D&C Contractor Response	Additional Reviewer Comment	Additional D&C Contractor Response
15	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	CPB are required to justify assumed 10m improvement depth for Dynamic Replacement. Has a 10m improvement depth been achieved on landfill previously? Report should also to include discussion of consequences if 10m improvement cannot be realised. Does this increase trial duration or are additional treatment options required?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	10m improvement depths has been achieved for similar landfill material (Van Impe and Bouazza 1996 and Dimitrios et al 2013). Preliminary discussion with sub-contractors confirmed that depth of about 10m improvement has been achieved previously. If 10 m improvement depth cannot be realised, likely consequence would be the increase in number of intervention.	24-Jun-2016	C	If 10m isn't achieved, is it not likely the initial construction settlement will be greater and take longer to realise, extending the preload period. CPB claim direct experience of carrying out this method of improvement within GWS previously. Request actual data from this works demonstrating depth of improvement achieved.	24-Mar-2017	The summary of 1984 DC trial and case studies from local contractors and literatures (describing that the improvement depth of up to 10 m has been achieved) are given in Appendix H of the Final Design Report. Previous experience in adopting DC method of improvement in GWS landfill involve less energy and hence depths lower than 10 m. As indicated previously above experience has been cited to justify the creep strain parameters of the DC improved landfill.	*Arcadis continues to have concerns regarding the suitability of this design solution* *Given the cosideration of surcharging as a fall back option together with in-situ trial of dynamic compaction Arcadis has not further comments.*	10m improvement depths has been achieved for similar landfill material (Van Impel and Bouazza 1996 and Dimitrios et al 2013). Preliminary discussion with sub-contractors confirmed that depth of about 10m improvement has been achieved previously. Case studies from local contractors and literatures (describing that the improvement depth of up to 10 m has been achieved) are given in Appendix H of the Final Design Report (GRW_RPT_GEO-0001-C1: GEOTLCOV24072AF-BA dated 17 October 2016). If DC trial indicates that the improvement depth is less than 10m and measured modulus is less than adopted in the current design, surcharging solution will be adopted so that post-construction settlement over 40 years will be limited to 500mm as stipulated in the "Head Contract Clarification No 20". As our current design is based on reasonable assumptions and surcharging requirement is not envisaged. We have carried out sensitivity assessment assuming 30% reduction in DC improvement depth (i.e. up to 7m) and hence assess the surcharging requirement to satisfy post-construction settlement criteria (refer Section 5.6.1 Scenario 7). Further refinement to surcharging design will be carried out once DC trial performance are observed. As the primary consolidation of landfill is completed within about one month period, available waiting period of 6 months is expected to be sufficient to complete the surcharging (if required) and meet post-construction settlement requirement.
16	A	[redacted]	08-Jun-16	Environment	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Consideration should be given to Final Compilation of Mitigation Measures (FCMM) 6A, Recommended Condition of Approval (MCOA) C5 (particularly (h) & (i)), and C6. Consultation with the EPA should be demonstrated. CPB to demonstrate consideration with suitable evidence. Related to the above; CPB are also required to show compliance of this design with their GWS Project Specific Procedure.	SIMTA-CPB Contract, PPR Annexure K, Cl. 1.1.5	N	CPB have prepared an assessment report of the proposed impacts of construction within GWS and a project specific procedure for works within GWS. Design considers the detail within each report. Consultation with the EPA will take place upon final project approval.	24-Jun-2016	C	the assessment report should also influence design	30-Mar-2017	The proposed design has taken into consideration the GWS Facility Assessment Report (prepared by Coffey) and GWS Project Specific Procedure (prepared by CPB). Refer Sections 7 and 8 of the Final Design Report.	*NONE - Closed*	
17	A	[redacted]	08-Jun-16	Environment	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	Memo fails to demonstrate how CPB would prevent infiltration of groundwater in landfill during dynamic compaction.	SIMTA-CPB Contract, PPR Annexure K, Cl. 1.1.5	D	Anticipated groundwater level is at RL 3 mAHD or below. Hence, DC work will be confined to a zone above groundwater level. If this comment is related to rain water infiltration into the landfill, there is no change in such condition as rain water currently infiltrate into the landfill. With the DC, thickness of soil cover will be increased and hence retard the infiltration. Construction staging will be included in our Final Design submission so that DC will be carried out in stages/zones so that ironing pass will be completed and provide sufficient grade to facilitate surface runoff and hence reduce the infiltration.	24-Jun-2016	C	This should be noted in the design report	01-Jul-2016	Construction staging are included in the FD report. Refer Section 5.5 and Drawings GRW-DRG-GEO-0003-C and GRW-DRG-GEO-0007-C. In addition, commentary on existing land cover is provided in Section 7 (Environmental Consideration).	*NONE - Closed*	
18	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	This 35% Developed Design package for Protection of Utilities near 'GWS Landfill Ground Treatment' has been submitted containing only two design memorandum. This package does not include a formal Developed Design report, which CPB/Aurecon has submitted for all other 35% Developed Design packages. As such this design package fails to demonstrate design consideration of: - Environmental and Approvals Considerations - Design Criteria - Design Integration (especially with Rail, Structures and Earthworks packages) - Safety in design - Constructability - Operation and Maintenance, etc. It would also be expected that a 35% Developed Design submission would include formal drawings which adequately display ground improvement zones. Hand sketches (plans and cross sections) and other diagrams provided in both design memorandums are hard to follow.	SIMTA-CPB Contract, Annexure K (PPR) Cl. 1.1.3	N	35% Developed design has been prepared to demonstrate the proposed ground treatment design and observational approach to satisfy performance of the rail formation. Critical element here is the post construction settlement (total and differential settlement). A design report will be included in our 100% Final Design submission.	24-Jun-2016	C	Response covered in Arcadis GI Memo	20-Jan-2017	1. Design integration (structures, utilities, drainage, capping layer, etc.) is addressed in Section 6 of the Final Design Report. 2. Design criteria is described in Section 3 of the Final Design Report. 3. Environmental Considerations including the Environmental Impact Statement are included in Section 7 of the Final Design Report.	*Design report still does not have all sections of information that Aurecon's reports have ie design integration, constructability, environmental considerations etc. CPB should ensure that all its design reports are presented to SIMTA in a similar format.* *Comment remains open*	FD report included sections relevant to "Geotechnical Ground Treatment Design" including: Design Integration; Environmental Considerations; Consideration on Project Specific Procedures; Operation and Maintenance; and Safety in Design An additional section address "Constructability" in the revised FD report.
19	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Over what chord length has differential settlement been calculated?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Differential settlement of 0.25% change in grade is considered here. i.e. 0.25% over given length. i.e. say 25 mm over 10 m. The proposed thresholds to be discussed with client and agreed on.	24-Jun-2016	C	Response covered in Arcadis GI Memo	31-Oct-2016	Settlement assessment is carried out for sections at 20m intervals. Hence, differential settlement is assessed for cord length/distance of 20m. Refer to Sections 3 and 5.4 of the FD Report.	*Closed refer to 100% Final Design comments*	
20	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016	Section 8, states that excavation of landfill may be required. Please confirm on plans the assumed extent of landfill which is required to be removed	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	It has been marked in long section (see Figure 1). Will be included on the plan as well. Anticipated 600 mm excavation may not extend to landfill itself as the capping layer thickness is likely to be more than 600mm.	24-Jun-2016	O	No details of the capping requirements of the site are included for areas of excavation, or any reference to the overall Landfill Closure Plan for the GWS construction footprint. The Ministers Conditions of Approval C5e states: "the methodology proposed to ensure that the landfill barrier system disturbed in the removal process is replaced/ repaired to ensure its ongoing performance."		This is addressed in Section 7 of the Final Design report. It is noted that the excavation of 600 mm is likely done in the existing cover layer.	*Provide EPA acceptance of CPB design regarding landfill capping* *Hydraulic conductivity testing has now been completed by CPB/Coffey. This design should be updated to reflect the results of the testing undertaken as the results may influence the final design if the hydraulic conductivity is not sufficient for capping. Further, as the GWS EPL must be resinded for to enable CPB construction EPL to be granted, the capping must be compliant woth the NSW Landfill Closure Guidelines - KP 31/3/17.* *Comment to remain open until EPA agreement is confirmed - JN 16/6/17.*	CPB requires access to GWS landfill to conduct permeability testing of the existing GWS cover. CPB submitted GWS landfill ground treatment design documentation in accordance with conditions of approval, Item C6 to the EPA. EPA have responded that they would not be providing any comment. Considering performance base requirement for the capping (i.e. "infiltration from the base of the final cap should be less than 5% of the annual rain fall"), an infiltration assessment has been carried out. A critical embankment section has been considered as dtailed in Section 7.2 and Appeidix M of IFC submision. Assessment indicated that the anticipated infiltration in to the landfill is neglegible. Hence, CPB have initiated engagement with the EPA to obtain acceptance of the above landfill capping performance. **10 May 2017** In-situ testing and laboratory testing demonstrated that the hydraulic conductivity of the existing cover layer is lower (better) than that assumed in the IFC first issue. The design is updated to include results of hydraulic conductivity tests and revised analysis. CPB will engage with the EPA to confirm acceptance of the landfill capping proposal following SIMTA confirmation that it is acceptable to engage with the EPA. **27 Sep 2017** Please refer to Section 7.2, last paragraph and QUBE mail number Qube PMS-GCOR-001214 to chnage Riviewer response status to "CS"
21	A	[redacted]	08-Jun-16	Environment	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Include a table to show how various conditions of approval have been met. This has been done in previous 35% designs.	SIMTA-CPB Contract, PPR Annexure K, Cl. 1.1.5, Appendix 8, Cl. 2.1	N	Noted. Will be included	24-Jun-2016	C	Consideration of environmental approvals for GWS ground improvement has not been witnessed	30-Mar-2017	The Final Design report (Section 7) addresses how the relevant condition in EIS are met.	*Agreed, addresses EIS and references the project Specific Procedure in Section 8. However, it is noted that it would be beneficial to state how the design has met the recommendations and findings of the project Specific Procedure.*	Design considered the construction within GWS landfill area. Which include, identification of landfill extent and tip batter within the footprint of the proposed embankment. Design considered the reduction/eliminate any impact on existing environment. Selected ground treatment eliminates excavation of waste material. Existing cover will be in place or where it removed partially will be covered with working platform material. As noted above, location of rail embankment has been identified in relation to landfill cells and hence landfill thickness and its variability in thickness have ben considered in the design. As noted in Section 8, no ground treatment work extended beyond Project Works area into the No-go zones. Construction material requirement is as per the relevant standards. **CLOSEd - KP 30/3/17**
22	A	[redacted]	08-Jun-16	Geotech	All drawings and reports	No design deliverables have been provided in electronic format and no design models or calculations have been provided. It should be noted that both memorandums contain only relatively few drawings and sketches - many of them are by hand.	SIMTA-CPB Contract, PPR Annexure K, Cl. 1.1	N	Adopted design parameters, ground models (layer thicknesses) and calculations are presented in Appendix B. Development of design parameters is presented in the report. A more detailed set of drawings will be prepared for the Final Design.	24-Jun-2016	C		01-Jul-2016			

DESIGN REVIEW RECORD (DRR)

CLIENT
Sydney Intermodal Terminal Alliance (SIMTA)

CONTRACTOR DESIGN PACKAGE NUMBER
RALP No.1

CONTRACTOR DESIGN PACKAGE TITLE
GWS Landfill Ground Treatment

Design Report: N01031-GRW-DRP-GEO-0001[02]
Design Specification: N01031-GRW-GEO-SPE-0001[02]
Drawing List: Refer to Appendix A of Design Report (N01031-GRW-DRP-GEO-0001-02)

COMPLIANCE STATUS
O Observation / Comment
D From info currently provided not able to determine whether design / proposal is compliant.
N Non-Compliant
M Minor non-compliance

RESPONSE STATUS
O Open
C Closed
CA Closed against this package but subject to action in another package
CS Closed SUBJECT TO additional action / information

No.	Rev	Reviewer	Initial Comment Date	Discipline	Document Reference	Reviewer Comment	Contract / Standard Requirement Reference	Compliance Status	D&C Contractor Response	Initial Response Date	Response Status	Reviewer Comment on Response	Date Comment Closed	Additional D&C Contractor Response	Additional Reviewer Comment	Additional D&C Contractor Response
23	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	It appears that no PLAXIS modelling has been conducted to support settlement calculation. CPB to confirm if PLAXIS modelling has been undertaken?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	O	No PLAXIS modelling has been carried out. We consider that 1-D analysis is reasonable in assessing the primary consolidation and post construction settlement. Although, the horizontal movement at rail level is likely to be minimal, 2D analysis using PLAXIS will be carried out in Final Design stage to assess the lateral movement of the embankment.	24-Jun-2016	C	What percentage of settlement has therefore been assumed to be lateral? What impacts are assumed on the landfill liner from both vertical and horizontal settlement and how are these to be modelled and confirmed.	24-Mar-2017	The assessed settlements and lateral movements are presented in sections 5.4 and 5.5 of the FD Report. As the compressible material is placed above the liner (i.e. at the base of the pit) and settlement and any lateral movement occur within the compressible material, there will be no impact on the liner. In addition, 2D PLAXIS analysis indicates minimal movement of compressible material adjacent to the liner. Furthermore, creep will occur within the landfill pit irrespective of the placement of embankment.	*Comment not closed. Refer to 100% Final Design comments* *Noted updated IFC report*	The assessed settlements and lateral movements are presented in sections 5.4 and 5.5 of the IFC Report. As the compressible material is placed above the liner (i.e. at the base of the pit). Settlement and lateral movement of landfill material at the liner area (i.e. at the stiff base) are minimal as anticipated and as indicated by the 2D PLAXIS analysis. Section 7.1 is amended to include above. Furthermore, creep will occur within the landfill pit irrespective of the placement of embankment.
24	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Both design memorandums contain no details of post construction monitoring details/proposed setup. Type, locations and number of monitoring arrays should be considered at this Developed Design stage.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	Monitoring details are discussed in Section 6.2 and 6.3 in GEOTLCOV24072AF-AR Rev 1. Further details and instrumentation drawings will be included in Final Design submission.	24-Jun-2016	C	Response covered in Arcadis GI Memo	31-Mar-2017	Settlement monitoring requirements are detailed in Section 5.3.1.2 of FD report and drawing GRW-DRG-GEO-0010-C	*Comment not closed. Refer to 100% Final Design comments*	Settlement monitoring requirements are detailed in Section 5.3.1.2 of IFC report and drawing GRW-DRG-GEO-0010-01. In addition, Instrumentation & Monitoring specification: N01031-GRW-GEO-SPE-0001-01 is submitted with IFC documentation.
25	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	CPB to demonstrate consideration of requirements/design criteria setout in PPR Annexure K Cl. 1.2.1e.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 1.2.1e	N	Cl 1.2.1.e refers to benching work within the rail corridor. In subject area the formation is mainly consist of rail formation (as per ARTC requirement) over general embankment. General embankments satisfy items (i), (iii) and (iv). Due to the long term creep settlement of landfill, differential settlement of 1:400 (0.25% change in grade) is considered within intervention period. Since the formation adjustment (re-ballasting) is carried out at each intervention period based on differential settlement criteria or typical maintenance interval for operation (whichever occurs first), differential settlement limit of 1:400 can be maintained over the design life.	24-Jun-2016	C		01-Jul-2016			
26	A	[redacted]	08-Jun-16	Geotech	GEOTLCV24072AF-AR Rev 1 by Coffey dated 20/05/2016 & GEOTLCV24072AF-AN Rev 1 by Coffey dated 20/05/2016	Coffey to confirm vertical and horizontal maximum allowable post-construction settlement assumed.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Assumed settlement criteria here is: 100mm post construction settlement within any intervention period and differential settlement of 0.25% (change in grade) within any intervention period. No limit has been imposed on total maximum PCS over 40 years period. 2D analysis using PLAXIS will be carried out in Final Design stage to assess the lateral movement of the embankment. The assessed lateral movement will be provided for the assessment of track performance.	24-Jun-2016	C	Response covered in Arcadis GI Memo	31-Mar-2017	Refer to Section 5.4 of Final Design Report	*Comment not closed. Refer to 100% Final Design comments*	Refer 100% Final Design comments
100% Final Desig[redacted]s																
27	Rev0	[redacted]	27-Oct-16	Geotech	GRW-RPT-GEO-0001-C1 Table 11, Design Report	It is noted that some of predicted settlement post construction as presented in Table 11 of Design Report please explain why there is no other means of improvement has been considered to limit the settlement such that the Project Performance specification will be satisfied.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	The design is based on the dynamic compaction with a maximum post construction settlement of 500 mm.	22-Dec-2016	C		11-Jan-2017			
28	Rev0	[redacted]	27-Oct-16	Geotech	GRW-RPT-GEO-0001-C1 Table 5 of Design Report	We notes that the vertical Young's Modulus has been presented in Table 5, noting Ev'=2MPa for landfill and Ev'=3 MPa for treated landfill. Please clarify how these values were used for design purpose.	General	O	These values are equivalent stiffness values based on adopted "Compression Ratio" of 0.15 and 0.075. However, these are not used in the settlement calculation. Therefore, we have removed them from the Table 5 of the amended FD report.	22-Dec-2016	C	Noted	11-Jan-2017			
29	Rev0	[redacted]	27-Oct-16	Geotech	GRW-RPT-GEO-0001-C1 Section 5.3.1.1 of Design Report	The design requires an improvement depth of up to 10m. Please clarify how the improvement depth will be confirmed post DC compaction and the potential of non-uniformity of the landfill beneath the improved depth will be assessed. What will be the contingency plan should the improvement is not as effective as designer predicted.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Refer Item SI2 in General notes, Drawing No.: GRW-DRG-GEO-003-C. Pressuremeter testing will be carried out before and after DC (both during trial and production) to assess the modulus. Considering the thick raft (embankment and improved landfill), it is unlikely that any localised non uniformity of landfill beneath the improved depth will have any impact on differential settlement at rail levels. If DC trial indicates that the improvement depth is less than 10m and/or measured modulus is less than adopted in the current design, surcharging solution will be adopted so that post-construction settlement over 40 years will be limited to 500mm as stipulated in the "Head Contract Clarification No 20". As the primary consolidation of landfill is completed within about one to two months period, available waiting period of 6 months will be sufficient to complete the surcharging (if required) and meet post-construction settlement requirement.	22-Dec-2016	C	Please provide the frequency of pressuremeter testing (spacing) for trial and mass production.	24-Mar-2017	Details are provided in Section 5.3.1.2 of IFC documentation	*Noted updated IFC report*	
31	Rev0	[redacted]	27-Oct-16	Geotech	GRW-RPT-GEO-0001-C1 Section 5.3.3 of Design Report	The numerical model adopted for dynamic settlement was uniform and the degradation rate is expected to be uniform due to dynamic effects. Please clarify what will be the potential impact on the predicted effect when non-uniform ballast and the underlying materials are considered.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Modelling is to compare performance of thicker ballast (i.e. 650 mm) against 500 mm thick ballast. The results indicate that the difference in performance is negligible. Considering the DC treatment, upper layer of treated material is likely to have similar properties. Dynamic effect is likely to be within the upper 5 m or so (depth of influence). Therefore, the effect of subgrade variation on the dynamic deflection is expected to be within reasonable.	22-Dec-2016	C	The ballast failure is progressive while he numerical modelling is based on uniform ballast with lower stiffness. This is not true representation of the actual failure mechanism of ballast.	24-Mar-2017	Modelling is to **compare performance of thicker ballast (i.e. 650 mm) against 500 mm thick ballast**. The results indicate that the difference in performance is negligible. **Degradation of ballast is maily due to the particle breakage**. Research finding indicated that ballast breakage will not reduce the stiffness, in fact tend to increase the stiffness. Indraratna et. al. (2008, Geotechnique, Vol 59, No. 7, pp 643-646) presented that resilient modulus increases with particle breakage. In FD submission, reduced ballast stiffness has been considered in modeling any **ballast contamination**. Contamination of the ballast is mainly due to intrusion of fine particles caused by mud pumping etc. A properly constructed rail formation with capping layer and area where groundwater level is well below the formation, ballaste contamination is less likely. Hence, assessment on ballast comtamination is not included in the IFC submission.	*Noted - This item could be managed after contruction and during the intervention process based on the new information in App I of IFC Report*	
32	Rev0	[redacted]	27-Oct-16	Geotech	GRW-RPT-GEO-0001-C1 Table 9, Design Report	We noted the predicted secondary settlements in Table 9 are different from those presented at 35% design report, e.g. between Ch. 40,560 and Ch40,740. Please clarify what are the input parameters that caused the difference.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Analysis has been refined to consider landfill thickness, formation of granular layer due to DC settlement and formation of granular columns within the upper two meters of the landfill.	22-Dec-2016	C	Noted	11-Jan-2017			
33	Rev0	[redacted]	27-Oct-16	Geotech	GRW-RPT-GEO-0001-C1 Section 5.4.2 of Design Report	We note that the intervention strategy is virtually the same as at 35% design from the number of interventions although the predicted post construction settlement values are slightly less than those at 35% design. Please explain what is the economic ramifications / consequences resulting from the intervention from a operator perspective.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	Intervention requirement depends on the change in grade limit of 0.25%. Hence, slight change in total settlement may not change the number of interventions. Currently proposed interventions are at 0.5, 1, 2.1, 4.2, 9, 19 and 40 year. In addition, design is carried out to maintain the maximum post construction settlement of 500 mm. We understand that routine maintenance (re-levelling and re-tamping) is likely to occur in 2 to 3 years period. Therefore, the use of routine maintained as intervention can be considered by the Principal in their assessment of any economic ramifications after 2 years (or maximum of up to 4 years) from the commissioning.	22-Dec-2016	CS	Subject to field trial	11-Jan-2017			
34	Rev0	[redacted]	27-Oct-16	Geotech	GRW-RPT-GEO-0001-C1 Section 5.6 of Design Report	Please clarify if the sensitivity range is adequate to cover the worst case scenario, and what will be the total ballast thickness for the sensitivity assessment case.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Considering the upper bound secondary compression ratio adopted for land fill, we consider that the sensitivity range is adequate to cover the worst case scenario. Alternative 2 (i.e. additional surcharge) has been developed to maintain the design settlement profile and hence the proposed intervention periods. Requirement of maximum 500mm ballast thickness is maintained by adjusting the rail vertical alignment (refer Section 5.3.3 and Section 9/Appendix I of FD report). Ballast thickness, is likely to increase marginally at isolated locations and hence proposed to use geoweb/geogrid to contain the lower 100mm to 150mm of ballast (Report will be updated accordingly)	22-Dec-2016	CS	Subject to field trial and reforecast of long term settlement after trial.	11-Jan-2017			
35	Rev0	[redacted]	27-Oct-16	Geotech	GRW-RPT-GEO-0001-C1 Section 6.4 of Design Report	The calculated gradient change appears to be less than 0.05%. Please clarify what is the cord length considered in calculating the gradient change.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	In transverse direction, settlement has been assessed at offsets of -11.2, -8.7, -5.2, -2.6, 0, 3.5 from MB2S line. Hence, cord length is approximately 2.5m to 3.5m	22-Dec-2016	C	Closed by John Nicholson.	20-Jan-2017			
36	Rev0	[redacted]	27-Oct-16	Geotech	Drgs 0008 and 0009	Please clarify what is the acceptable settlement criteria for defining the intervention as shown on Drg 0008 and for hold point release on Drg 0009.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Predicted maximum change in grade of 0.25% at proposed intervention periods. Total predicted settlement of 500mm in 40 years.	22-Dec-2016	C	Comment closed - refer to comment 44	20-Jan-2017			

DESIGN REVIEW RECORD (DRR)

CLIENT

Sydney Intermodal Terminal Alliance (SIMTA)

CONTRACTOR DESIGN PACKAGE NUMBER

RALP No.1

CONTRACTOR DESIGN PACKAGE TITLE

GWS Landfill Ground Treatment

Design Report: N01031-GRW-DRP-GEO-0001[02]

Design Specification: N01031-GRW-GEO-SPE-0001[02]

Drawing List: Refer to Appendix A of Design Report (N01031-GRW-DRP-GEO-0001-02)

COMPLIANCE STATUS
O Observation / Comment
D From info currently provided not able to determine whether design / proposal is compliant.
N Non-Compliant
M Minor non-compliance

RESPONSE STATUS
O Open
C Closed
CA Closed against this package but subject to action in another package
CS Closed SUBJECT TO additional action / information

No.	Rev	Reviewer	Initial Comment Date	Discipline	Document Reference	Reviewer Comment	Contract / Standard Requirement Reference	Compliance Status	D&C Contractor Response	Initial Response Date	Response Status	Reviewer Comment on Response	Date Comment Closed	Additional D&C Contractor Response	Additional Reviewer Comment	Additional D&C Contractor Response
37	Rev0	[redacted]	27-Oct-16	Geotech	Drgs 0010	Please clarify if the SPL and SM spacing of 20m c/c along the traffic direction is adequate to reflect the potential deflections of the rail track and the true landfill responses within which heterogeneous materials / lumps may be present.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	We consider 20m c/c spacing of SPL and SM are sufficient. As mentioned above improved landfill, granular platform created by DC settlement will ion out non uniform settlement of lower landfill layer. In addition, HPGs have been proposed for post construction monitoring purposes. Inclinometers have been proposed at the toe of the steep embankment batter (i.e. 0.6H:1V)	22-Dec-2016	C	This comment has been transferred to Item #35.	11-Jan-2017			
38	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 3.1	Second paragraph confirm should be 40 not 30 year design life?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	O	PPR specify "Expected long term post construction settlement of top of the surface (in areas of fill or virgin material) are equal to or less than 1:400 over 30 years (Cl 1.2.1 (d) (ii)). Statement will added to indicate that the design has been carried out to assess the settlement up to 40 year period.	22-Dec-2016	C		20-Jan-2017			
39	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 3.3	First dot point should be PPR	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	It is as given in the "Performance Specification" document. It should in fact read as "Specific provisions of the Performance Specification". Will amend accordingly.	22-Dec-2016	C		20-Jan-2017			
40	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 5.3.1.1	Provide detail of material; to be used for back filling of craters created during pounding	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Refer Item MR1 and MR2 under "Material Requirement" in General notes drawing : GRW-DWG-GEO-003-C1	22-Dec-2016	C		20-Jan-2017			
41	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 5.3.1.2	Needs to include detail for ongoing operational monitoring to assess performance. How will total settlement be recorded once rail construction begins? Are settlement plates to be maintained? If so how will these be protected? Close to real time monitoring is likely to be required. Design report is to include more information on operational and maintenance issues resulting from CPB proposed design solution.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	As discussed in Section 5.3.1.2 of GRW_RPT_GEO-0001-C1, back analysis of secondary compression parameters will be carried out based on the monitored settlement and hence predict the post-construction settlement. As noted in Drawing No.: GRW-DRG-GEO-0009-C1: back analysis data and predicted post-construction settlement will be provided to SIMTA for their review and approval. Drawing will be amended to include surcharge as contingency measure to maintain the total post-construction settlement of less than 500mm in 40 years. Settlement monitoring requirements are detailed in Section 5.3.1.2 of FD report and drawing GRW-DRG-GEO-0010-C. Settlement plates have been proposed at the embankment crest, and away from the ballast. Hence, these settlement plates can be used for post construction monitoring. We understand that the monitoring of the track settlement after the project liability period is carried out by the Principal. Similar approach has been adopted in previous project (Hexham Relief Road Project) where settlement plates have been installed within the embankment (between rail tracks) and monitored continuously (even after tamping works) Based on the predicted settlement, it is unlikely that real time monitoring is required. Expected, settlement in initial stage (up to 0.5 years) is about 2mm per week (which is within the survey accuracy of +/-2mm). Subsequent stages, expected settlement is <1mm per week. In addition, HPGs have been proposed for post construction monitoring purposes. Inclinometers have been proposed at the toe of the steep embankment batter (i.e. 0.6H:1V). Operational and maintenance issues are provided in Section 9.	22-Dec-2016	C	Will be closed once Monitoring Specification is provided with the AFC submission	13-Jun-2017	Noted. Instrumentation & Monitoring specification: N01031-GRW-GEO-SPE-0001-01 is submitted with IFC documentation.	*(Refer to pages 20 and 19 of N01031-GRW-DRP-GEO-SPE-0001-01) Given the dynamic compaction improvement, the compacted surface is expected to be have much higher CBR value for the plate load testing. The limit value of 1% and 3% is not realistic CBRs . Please provide realistic value on the flow charts, reports, Drgs and specifications. Similarly on page 19 for CBR of 3%, given the influcen zone is well within the existing capping layer and platform and subsequent backfill.*	Drawing is amended and minimum CBR of 3% is adopted. We consider that CBR of DC compacted subgrade can be higher than 3%. However, general embankment fill will be placed over DC treated ground before constructing/placing the structural fill (rail formation). Hence, it is reasonable to adopt minimum CBR of 3% as per the requirement stipulated in the ARTC Standard ETM-08-01.
42	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 5.3.1.2	Why have HPG's not been considered?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	HPGs have been proposed for post construction monitoring purposes. Inclinometers have been proposed at the toe of the steep embankment batter (i.e. 0.6H:1V)	22-Dec-2016	C		20-Jan-2017			
43	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 5.3.1.2	Will raw settlement data be provided to the superintendent during the waiting period?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Settlement data can be provided to the superintendent during the waiting period. We consider that monitoring report will be submitted to SIMTA as part of "Hold Point" release process.	22-Dec-2016	C		20-Jan-2017			
44	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 5.3.2.1	What is the criteria for extending or reducing the settlement period?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Predicted maximum change in grade of 0.25% at proposed intervention periods. Total predicted settlement of 500mm in 40 years.	22-Dec-2016	C	As per Appendix X - Clarifications	02-Feb-2017			
45	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 5.3.3	Does the DEM modelling consider only new ballast characteristics?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Modelling here is to compare performance of thicker ballast (more than 500mm) against 500mm thick ballast. However, breakage of Ballast has been considered in the DEM analysis.	22-Dec-2016	C		20-Jan-2017			
46	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 5.5	Has the performance of a 0.15m thick capping layer been considered against the proposed settlement	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	As noted in Section 6.4 maximum change in grade of capping layer is less than 0.05%. Hence, this minor variation in grade will not compromise the performance of well compacted granular capping layer.	22-Dec-2016	C		20-Jan-2017			
47	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 6.1	Update to include proposed structure at interface to viaduct.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	We consider that the structure referred here is 06H:1V batter. Report has been revised to include assessment on reinforced embankment batter	22-Dec-2016	C		20-Jan-2017			
48	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 9	Confirm Superintendents acceptance of non-standard design?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	The head contract is based on dynamic compaction with a maximum post construction settlement of 500 mm.	22-Dec-2016	C	As per Appendix X - Clarifications	02-Feb-2017			
49	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Section 9	More information required for post construction monitoring inc, by who? (qualifications), regular period and when additional monitoring maybe required (i.e. adverse weather events)	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	CPB will carry out the settlement monitoring during the defect liability period of 1 year and hence carry out the tamping as required (Two tamping as per the design prediction). Subsequent monitoring will be carry out by the Principal. CPB/Coffey is happy to prepare a post-construction monitoring program and carry out monitoring if required at additional cost of Principal. An instrumentation and monitoring specification will be developed to outlining the monitoring requirements during construction and post construction (will be issued during IFC submission)	22-Dec-2016	C	Subject to adequate AFC submission	24-Mar-2017	Noted. Instrumentation & Monitoring specification: N01031-GRW-GEO-SPE-0001-01 is submitted with IFC documentation.	*Noted IM Sepcification -N01031-GRW-GEO-SPE-0001-0*	
50	Rev0	[redacted]	28-Oct-16	General	General	What are the contingencies if settlement is significantly greater than predicted? Is there a contingency for localised differential settlement occurs?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Based on the available information, the adopted compressibility parameters are likely to be towards the upper bound. Therefore we do not anticipate significantly greater settlement than predicted. If monitored settlement within 1 to 2 months period is higher, surcharging option can be adopted. As the primary consolidation of landfill is completed within about one to two months period, available waiting period of 6 months will be sufficient to complete the surcharging (if required) and meet post-construction settlement requirement.	22-Dec-2016	C		20-Jan-2017			
51	Rev0	[redacted]	28-Oct-16	General	GRW-RPT-GEO-0001-C1 Appendix I	Appendix I indicates expected settlement along the track. There is a passive level crossing at approximately 40km 310. Has the designer considered the impact of settlement of road vehicles across this level crossing? Is there any additional maintenance requirements/inspection regime as a result of the expected settlement at this location? What are there parameters for intervention when settlement occurs?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	This will be addressed in the earthworks package IFC submission.	22-Dec-2016	CA	Subject to matter being address adequately in the Earthworks AFC submission.	20-Jan-2017			
52	Rev0	[redacted]	28-Oct-16	General	Design Report GRW-RPT-GEO-0001-C1	No verification records (or evidence) have been submitted in this 100% Final Design report. CPB's Design Management Plan states *"The verification process will include representatives from Aurecon, Coffey and Siemens. Verification records will be maintained and included in the Final Design Reports."* Arcadis notes that Ashok Peiris is listed as the Author, Reviewer and Signatory.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9 CPB DMP Element 3.4	N	[redacted]	22-Dec-2016	C	Formal verification records are required to be provided as per CPB design management plan.	24-Mar-2017	Noted. Verification form is included in the IFC submission	*Noted IM Sepcification -N01031-GRW-GEO-SPE-0001-01*	
53	Rev0	[redacted]	28-Oct-16	General	Design Report GRW-RPT-GEO-0001-C1	Appendix G - Does modelling of ballast consider any impacts caused by contamination of ballast by fines material? What impact is this likely to have.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Contamination of ballast has been considered by assuming reduced stiffness. Report is amended accordingly (refer Section 5.4.2 and Appendix G	22-Dec-2016	C		20-Jan-2017			

DESIGN REVIEW RECORD (DRR)

CLIENT
Sydney Intermodal Terminal Alliance (SIMTA)

CONTRACTOR DESIGN PACKAGE NUMBER
RALP No.1

CONTRACTOR DESIGN PACKAGE TITLE
GWS Landfill Ground Treatment

Design Report: N01031-GRW-DRP-GEO-0001[02]
Design Specification: N01031-GRW-GEO-SPE-0001[02]
Drawing List: Refer to Appendix A of Design Report (N01031-GRW-DRP-GEO-0001-02)

COMPLIANCE STATUS
O Observation / Comment
D From info currently provided not able to determine whether design / proposal is compliant.
N Non-Compliant
M Minor non-compliance

RESPONSE STATUS
O Open
C Closed
CA Closed against this package but subject to action in another package
CS Closed SUBJECT TO additional action / information

No.	Rev	Reviewer	Initial Comment Date	Discipline	Document Reference	Reviewer Comment	Contract / Standard Requirement Reference	Compliance Status	D&C Contractor Response	Initial Response Date	Response Status	Reviewer Comment on Response	Date Comment Closed	Additional D&C Contractor Response	Additional Reviewer Comment	Additional D&C Contractor Response
54	Rev0	[redacted]	28-Oct-16	General	Section 6 and All Drawings	Report to be updated for either RE wall and 0.6:1 batter. Impacts of settlement on these structures are required to be considered. Interdisciplinary input and discussion is required to be demonstrated.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	This has been addressed in the updated FD report.	22-Dec-2016	C		20-Jan-2017			
55	Rev0	[redacted]	28-Oct-16	General	General	While the design makes continued references to differential settlement of 0.25% it remains a significant concern of Arcadis of the potential for short track twists to occur.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	As per the "Specification provisions of the Performance Specification" (and considering the order of precedence), intervention has been considered base on changing grade limit of 0.25% (1:400) and not the total settlement. Referred standard (i.e. TfNSW TN 004 2015: SPC 207: **Track monitoring requirement for undertrack excavation**, cited normal limits for track monitoring where localised excavation is likely to create localised settlement. As noted earlier, thick raft (embankment and improved landfill), is unlikely to create localised settlement. Considering the assessed settlement which change gradually along the longitudinal direction and across the transverse direction, twist limits are likely to be satisfied. (assessed by Aurecon rail designers).	22-Dec-2016	C	Not closed. Arcadis concerns have not been addressed.	24-Mar-2017	As discussed with Project Reviewers, visual observation schedule has been included in the Instrumentation & Monitoring Specification to mitigate risks associated with rail twist. Proposed observation to be carried out: 1. after first two trains. 2. twice weekly for 2 weeks. 3. once per week for next 2 weeks. 4. followed by once a month up to practical completion. Observation after practical completion is to be carried out by the Principal.	*Noted in Table 1 of IM Sepcification -N01031-GRW-GEO-SPE-0001-01*	
56	Rev0	[redacted]	28-Oct-16	General	General	There continues to be no meaningful justification for intervention thresholds. Arcadis note the only available track settlement standard applicable(TfNSW TN 004 2015), references intervention must occur much earlier than 100mm of settlement (as referenced in Arcadis GI Memo) and/or regular inspections/monitoring must occur or speeds of the track reduced.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	As per the "Specification provisions of the Performance Specification" (and considering the order of precedence), intervention has been considered base on changing grade limit of 0.25% (1:400) and not the total settlement. Referred standard (i.e. TfNSW TN 004 2015: SPC 207: **Track monitoring requirement for undertrack excavation**, cited normal limits for track monitoring where localised excavation is likely to create localised settlement. As noted earlier, thick raft (embankment and improved landfill), is unlikely to create localised settlement. Considering the assessed settlement which change gradually along the longitudinal direction and across the transverse direction, twist limits are likely to be satisfied. (assessed by Aurecon rail designers).	22-Dec-2016	C	Not closed. Arcadis concerns have not been addressed.	13-Jun-2017	As discussed with Project Reviewers, settlement trigger levels are included in the IFC submission. Settlement trigger levels is the settlement after each intervention at which next re-levelling/intervention is to be carried out.	*Noted in Appendix I of IFC Report. Please add these on Drgs for ease of reference.* *Noted in drawing N01031-GRW-DRG-GEO-0030[02] - 13/06/17.*	New drawing N01031-GRW-DRG-GEO-0030-01 has been added. In addition, as noted in our report (Page 41: Second paragraph under Section 9.2) these details should be included in the Operation and Maintenance Manual.
57	Rev0	[redacted]	28-Oct-16	General	General	If a liner is present no reference is made to how CPB plan to ensure it isn't damaged either by DC or following the proposed settlement?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 1.1.5, 2.1, Appendix 8 Cl. 2.5, 2.9	D	Base of the tip is known reasonably well based on available data. Proposed DC work is to improve existing landfill leaving about 3m to 4m untreated bottom land fill layer. Hence, minimal impact on the landfill liner is anticipated. Landfill is placed on competent material (shall rock bed), hence settlement or relative movement at liner is minimal. Any compressible layer, maximum settlement occur at the top surface where load is applied and virtually no settlement is occurred at the bottom.	22-Dec-2016	C		20-Jan-2017			
58	Rev0	[redacted]	28-Oct-16	General	GRW-DRG-GEO-0008[C1]	It appears that if CPB ground improvement is unsuccessfully CPB/Coffey simply re-issue a revised design report with an increased number of interventions. Is this the case? This is not acceptable to SIMTA. SIMTA will not accept an increased number of interventions. A revised design solution would be required. There appears to be no difference in CPB/Coffey response if settlements are less or greater than expected. In either case the design report is just updated. This is not acceptable to SIMTA.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	If DC trial indicates that the improvement depth is less than 10m and measured modulus is less than adopted in the current design, surcharging solution will be adopted so that post-construction settlement over 40 years will be limited to 500mm as stipulated in the "Head Contract Clarification No 20". As our current design is based on reasonable assumptions and surcharging requirement is not envisaged. Design of surcharging, can only be carried out once DC trial performance are observed. As the primary consolidation of landfill is completed within about one to two months period, available waiting period of 6 months will be sufficient to complete the surcharging (if required) and meet post-construction settlement requirement	22-Dec-2016	O	Follow chart to be updated to show that Superintendent's approval is required to proceed		Noted. Process control diagrams (N01031-GRW-DRG-GEO-0008-01 and N01031-GRW-DRG-GEO-0009-01) have been amended.	*Flow Charts on pages 19 and 20 should reflect the settlement monitoring report after 6 months of waiting time to client prior to track construction (Hold Point for Client).* *As previously stated, this represents a Hold Point for Client - JN 13/06/17.*	Flow charts revised (ref. drawings N01031-GRW-DRG-GEO-0009-02 and N01031-GRW-DRG-GEO-0010-02) to reflect the submission of settlement monitoring report to superintendent prior to track construction. Witness point for superintendent. **28 Sep 2017** Design documentation revised to include Hold Point for the Superintendent. Refer drawings N01031-GRW-DRG-GEO-0008-03 and N01031-GRW-DRG-GEO-0009-03
59	Rev0	[redacted]	28-Oct-16	General	GRW-DRG-GEO-0009[C1]	It appears that if CPB ground improvement is unsuccessfully CPB/Coffey simply re-issue a revised design report with an increased number of interventions. Is this the case? This is not acceptable to SIMTA. SIMTA will not accept an increased number of interventions. A revised design solution would be required. There appears to be no difference in CPB/Coffey response if settlements are less or greater than expected. In either case the design report is just updated. This is not acceptable to SIMTA.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	Refer to Item 58	22-Dec-2016	C		20-Jan-2017			
60	Rev0	[redacted]	28-Oct-16	General	GRW-DRG-GEO-0009[C1]	GRW-DRG-GEO-0009-C1 - Are the assessment post construction settlement greater than the predicted settlement (COF)? if yes Notify SIMTA also.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	Intermediate monitoring reports will be provided to SIMTA as well.	22-Dec-2016	C		20-Jan-2017			
61	Rev0	[redacted]	28-Oct-16	General	Design Report GRW-RPT-GEO-0001-C1 Appendix I Figure & Table 11	CPB to confirm signoff on revised vertical rail alignment by a qualified rail engineer.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	Revised vertical alignment has been checked by a qualified rail engineer (Aurecon). Report is amended accordingly (refer Section 5.4.2, Section 9 and Appendix I)	22-Dec-2016	C	Revised vertical alignments are to be produced and submitted to SIMTA for DLP tamping based on actual settlement results. Further vertical alignments are then required to be produced at the end of the DLP period for future tamping over the design life.	13-Jun-2017	Section 5.3.1.2 addresses the back analysis based on monitoring data and hence, predict the post-construction settlement performance. As noted in Section 5.3.1.2 and Process control diagrams (N01031-GRW-DRG-GEO-0008-01 and N01031-GRW-DRG-GEO-0009-01), revised design report including revised settlement predictions, revised vertical alignment etc. will be submitted for review and Principal's consent. Further vertical alignments for post defect liability period for Future interventions should be finalised just prior to each intervention by the Principal. Hence, this is not part of CPB's scope.	*Noted in Appendix I of IFC Report. Please add these on Drgs for ease of reference.*	New drawing N01031-GRW-DRG-GEO-0030-01 has been added. In addition, as noted in our report (Page 41: Second paragraph under Section 9.2) these details should be included in the Operation and Maintenance Manual.
62	Rev0	[redacted]	28-Oct-16	General	Design Report GRW-RPT-GEO-0001-C1	CPB to confirm location of DC trial	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	DC trial will be carried out in the vicinity of borehole BH8. Trial area has been marked on the drawing.	22-Dec-2016	C		20-Jan-2017			
63	Rev0	[redacted]	10-Nov-16	General	All drawings	Chainages inconsistent with signalling design	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	The track geometry package details civil design chainages and signal design chainages.	22-Dec-2016	C		20-Jan-2017			
64	Rev0	[redacted]	10-Nov-16	General	All drawings	Document control - inconsistent format from all other CPB drawings submitted. Different project number on drawings.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	O	Project number and drawing numbers are amended accordingly	22-Dec-2016	C		20-Jan-2017			
65	Rev0	[redacted]	10-Nov-16	General	GRW-DRG-GEO-0005[C1]	Drafting errors within GRW-DRG-CEO-0005-CI (arrows are not aligned for "surcharge fill thickness" comment)	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	O	Noted. Has been amended	22-Dec-2016	C		20-Jan-2017			
66	Rev0	[redacted]	10-Nov-16	General	GRW-DRG-GEO-0008[C1]	Immediate notification of SIMTA is required if settlements are greater than predicted.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M	Noted. Intermediate monitoring reports will be provided to SIMTA	22-Dec-2016	C	This is not shown on the flow chart or referenced in the design report. Report and Drawing to be updated to reflect comment response.	24-Mar-2017	Process control diagrams (N01031-GRW-DRG-GEO-0008-01 and N01031-GRW-DRG-GEO-0009-01) indicate that the intermediate assessment is carried out (after 3 months). If assessed settlements are greater than predicted, revised design report is submitted for for review and Principal's consent.	*Noted in IM Sepcification -N01031-GRW-GEO-SPE-0001-01*	
67	Rev0	[redacted]	10-Nov-16	General	Design Report GRW-RPT-GEO-0001-C1	Interventions do not appear to consider long/short twist or greater than 500mm thickness capping. Other track parameters?	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	Considering the gradual change in the settlement profile and maintaining 0.25% change in grade, Long/Short twist is unlikely to be a governing criteria. Assessment of Long/Short twist has been carried out by a qualified rail engineer (Aurecon) based on the predicted settlements and found to be within the limits specified in TfNSW TN 004 2015 Rail vertical alignment adjustment (Refer Item 61) is proposed to maintain maximum ballast thickness of 500mm. Ballast thickness, is likely to increase marginally at isolated locations and hence proposed to use geoweb/geogrid to contain the lower 100mm to 150mm of ballast	22-Dec-2016	C		20-Jan-2017			
68	Rev0	[redacted]	10-Nov-16	General	General	No geotechnical design models have been provided. Including - Slope W - PEM model - Plazis 2D model/other models - FE Model	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N	Geotechnical ground models, design parameters and outputs are provided in design report.	22-Dec-2016	C	Electronic native formats are required to be provided. Drawings are also required to be provided in CAD format.	24-Mar-2017	Provided in IFC submission	*Noted submission og relevant files*	

DESIGN REVIEW RECORD (DRR)

CLIENT
Sydney Intermodal Terminal Alliance (SIMTA)

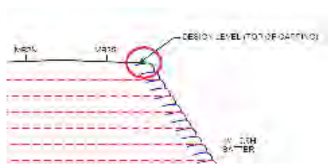
CONTRACTOR DESIGN PACKAGE NUMBER
RALP No.1

CONTRACTOR DESIGN PACKAGE TITLE
GWS Landfill Ground Treatment

Design Report:	N01031-GRW-DRP-GEO-0001[02]
Design Specification:	N01031-GRW-GEO-SPE-0001[02]
Drawing List:	Refer to Appendix A of Design Report (N01031-GRW-DRP-GEO-0001-02)

COMPLIANCE STATUS
O Observation / Comment
D From info currently provided not able to determine whether design / proposal is compliant.
N Non-Compliant
M Minor non-compliance

RESPONSE STATUS
O Open
C Closed
CA Closed against this package but subject to action in another package
CS Closed SUBJECT TO additional action / information

No.	Rev	Reviewer	Initial Comment Date	Discipline	Document Reference	Reviewer Comment	Contract / Standard Requirement Reference	Compliance Status	D&C Contractor Response	Initial Response Date	Response Status	Reviewer Comment on Response	Date Comment Closed	Additional D&C Contractor Response	Additional Reviewer Comment	Additional D&C Contractor Response
69	Rev0	[redacted]	10-Nov-16	General	GRW-DRG-GRO-0007[C1]	Item CSD1- Is top soil left in place? seems like this would adversely effect settlement	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	D	Since the area will be subjected heavy tamping during DC works, thin layer of top soil will not have any impact on the settlement. However, we will amend the note (CSD1) as the top soil will be removed and stockpile for later use.	22-Dec-2016	C		20-Jan-2017			
70	Rev0	[redacted]	10-Nov-16	General	GRW-DRG-GRO-0007[C1]	Items CSD4.2 and 4.4 - Reassess measurements against predictions and remodel if required.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	O	Item CSD4 provided the steps required in DC works. Survey measurements are to assess the settlement due to DC works. This settlement together with performance of DC works as assessed by "pressuremeter testing" and "cone penetrating testing" will be compared with the design intent.	22-Dec-2016	C		20-Jan-2017			
AFC																
71	01	[redacted]	30-Mar-17	Environment	Report - Section 7	Hydraulic conductivity testing has now been completed by CPB/Coffey. This design should be updated to reflect the results of the testing undertaken as the results may influence the final design if the hydraulic conductivity is not sufficient for capping. Further, as the GWS EPL must be resinded for to enable CPB construction EPL to be granted, the capping must be compliant woth the NSW Landfill Closure Guidelines.	SIMTA-CPB Contract, PPR Annexure K, Cl. 1.1.5, Appendix 8, Cl. 2.1	M	In-situ testing and laboratory testing demonstrated that the hydraulic conductivity of the existing cover layer is lower (better) than that assumed in the IFC first issue. The design is updated to include results of hydraulic conductivity tests and revised analysis. CPB will engage with the EPA to confirm acceptance of the landfill capping proposal following SIMTA confirmation that it is acceptable to engage with the EPA.		O	Comment to remain open until EPA agreement is confirmed - JN 16/6/17.		Please refer to Section 7.2, last paragraph and QUBE mail number Qube PMS-GCOR-001214 to change Riviewer response status to "CS"		
72	01	[redacted]	05-Apr-17	General	N01031-GRW-DRP-GEO-0001-01	ITP for this design package to be submitted (including relevant Hold / Witness Points for Superintendent's review).	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5	O	ITP will be provided in accordance with CPB's Inspection and Test Plan schedule. Witness and hold points are included in Appendix L of the design report.		CS	Subject to submission of ITP. Superintendent will require additional HP/WP's which shall be stipulated during ITP review process.	13-Jun-17			
73	01	[redacted]	05-Apr-17	General	N01031-GRW-DRP-GEO-0001-01	CPB to specify maintenance requirements for the soil wall fabric.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5	O	Terramesh or equivalent flexible facing is proposed. Flexible facing system and connection of geogrid to facing should be as per the manufactureres specifications. The mesh forming the Terramesh or equivalent unit is provided with polymer-coated galvanized alloy steel having working life of about 120 years.. Section 9.2 has been amended to include details of Terramesh or equivalent facing. Appendix O (cited in Section 9.2) is added to included data sheet of facing element and typical details. Hence, no ongoing maintenance is required during operational life		CS	Details to be included in the O&M Manual.	13-Jun-17			
74	01	[redacted]	05-Apr-17	General	N01031-GRW-DRG-GEO-0020-01	Provide additional details of the geotech design between the capping and soil wall. 	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5	M	Typical details are included in revised drawing N01031-GRW-DRG-0020-02.		C		13-Jun-17			
75	01	[redacted]	05-Apr-17	General	N01031-GRW-GEO-SPE-000-01	"Monitoring" to be done by CPB during construction and unitl the end of DLP. CPB to amend report accordingly (e.g items 1.1, 3.6.1, etc.)	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5	N	Instrumentation and monitoring specification is amended accordingly		C		13-Jun-17			
76	01	[redacted]	05-Apr-17	General	N01031-GRW-GEO-SPE-000-01; N01031-GRW-DRG-GEO-0010-01	Hydrostatic Profille Gauges - HPG1, HPG2 AND HPG3 appear to be outside the project boundary (even within the riparian corridor). CPB to clarify this and provide more details about the location and final design of these HPG's.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5	N	HPGs shown in the drawing N01031-GRW-DRG-GEO-0010-02 are indicative chainages. Typical section of the HPG is provided in N01031-GRW-DRG-GEO-0011-02. As the extend of HPG is about 2 m to 3 m from the toe of the embankment. So as access to the western side (GWS side of the embankment) is not required, read-out end of the HPG will be on the eastern side of the embankment with provision to have drawcord pull out form the eastern side. This wil also facilitate monitoring of HPG's even if additonal fill is placed on the GWS side. . Drawings N01031-GRW-DRG-GEO-0010-02 and N01031-GRW-DRG-GEO-0011-02 have been revised accordingly		C		13-Jun-17			
77	02	[redacted]	13-Jun-17	Geotech	General	Arcadis notes that the standard plate load testing is based on 300mm square plate. This would not be adequate to confirm the CBR values for the Dynamic Compaction treated zone where a strong crust will be formed. Please clarify what is the plate size and the assessed influenced zone.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	M				Drawing amended to include minimum plate size of 0.75 m for plate load testing. Refer drawing N01031-GRW-DRG-GEO-0009-03				
78	02	[redacted]	13-Jun-17	Geotech	N01031-GRW-DRG-GEO-0003-02	Item DC3 - CPB to indicate that 2 interventions are predicted but more interventions have been allowed for if required, and based on the monitoring.	SIMTA-CPB Contract, , Annexure K (PPR) Cl. 2.1, Appendix 8 Cl. 2.5, 2.9	N				Design Criteria and Assumptions Note DC3 (drawing N01031-GRW-DRG-GEO-0003-02) is consistent with Head Contract clarification No 20 "The Contract Sum includes an allowance for 2 follow up tamps before the end of the defects liability period. The Contractor's design is based on a maximum post construction settlement of 500mm. Ongoing tamping requirements become the responsibility of the Principal."				

Appendix C – Summary of Monitoring and Review Stages

Appendix C - Summary of various stages for monitoring and review

	Stage					
	Design	Construction				
	FDD to AFC	DC Trial	DC Production	Filling up to design level - 1 month	Monitoring - 6 month	
Description	Based on the developed design parameters from available data	Carry out DC trial in a relatively known landfill area (i.e. vicinity of BH08)	Production of DC other than trial area	Construction of embankment up to design level (up to surchage level if required)	End of 3 moths after filling completed	End of 6 moths after filling completed
Monitoring/Review process	-	Settlement due to DC trial under various compacting energy monitored (Depth of treatment based on verification tests carried out by subcontractor)	Settlement due to DC production monitored and compared with that of trial (Depth of treatment based on verification tests carried out by DC subcontractor)	Settlement plates installed before the placement of the fill and monitor primary settlement	1. Review monitoring data weekly 2. Compare monitoring data against the prediction at the end of 3 months	1. Review monitoring data weekly. 2. Compare monitoring data against the prediction at the end of 6 months
Actions	-	-	Assess the relative variation of the ground condition if any and refinement of settlement profile prediction along the alignment and Identify/refine settment plate installation locations. If the DC improved depth is shallower than design assumption, adopt the surchaging option and hence, assess the surchage requirement	-	1. Predict long term settlement performance. If predicted settlements are more than design, adopt surcharging option. Results are provided to CPB for their review. 2. If monitoring data within the first three months indicates that the observed settlement is more than the design prediction, we will carry out intermittent settlement predictions based on the observed settlement and provide the results including predicted intervention period to CPB for their review. If the intermediate back analysis results indicate that the long term settlement performance are better than predicted, advantage of reduction in waiting period can be considered 3. Review by CPB and the client (allow two weeks)	Update the settlement prediction. Revised design report

Appendix D – Design Calculation Sheets

Client: CBP Contractors Pty Ltd GEOTLCOV24072AF
Project: Moorebank Intermodal Rail Link PKW/VN
Location: Glenfield Waste Tip Section 7/10/2018

Foundation designinvolving surcharge preloading along landfill area (Ch40,440 - Ch 40,740 of MB2S) for Developed Concept Design (35%)

1. INPUT

(a) Existing material parameters

Surcharge Fill & Soil Cover	Gams	20	kN/m3
	Msoil	15	MPa
	CAEsoil	0	
Landfill	Mlandfill	2	MPa
	CAE	0.02	Initial creep strain rate (per log time cycle)
	CAE_min	0.015	Minimum creep strain rate after treatment
	β	0.07	Reduction factor such that CAE=CAE_0 - $\beta.\varepsilon_v$ from treatment, but not less than CAE_min)
	CAE*	0.015	Creep strain rate for landfill from CH 40,550 - 40,700
	CR	0.15	Compression Ratio for unimproved landfill
	CR*	0.15	Compresstion Ratio for CH 40,550 - CH 40,700 landfill prior to preloading
	Glfill	13.8	kN/m3 of unit weight
Design Life		40	years

(b) New fill parameters

Gamfill	21	kN/m3
Mfill	15	MPa
Caefill	0.0015	Creep strain rate (per log time cycle)

(c) Waiting Time for Preload & Time after construction

t_0	0.250	year	
t_1	0.583	year	0.333

Chainage (m)	Crest of MB2N Side (i.e. East Side)						Centre of MB2S (i.e. West Side)						Creep at t_1 @ Cae_min		Lateral δl (%)	
	H_D (m)	H_T (m)	D_R (m)	D_L (m)	D_S (m)	Di (m)	H_D (m)	H_T (m)	D_R (m)	D_L (m)	D_S (m)	Di (m)	MB2N	MB2S	width (m)	δl (%)
40440	9.50	9.60	0.50	0.00	0.50	0.50	14.36	14.51	1.00	0.50	0.50	0.50	0	3	16.2	0.02%
40445	9.70	9.80	0.50	0.00	0.50	0.50	13.80	14.00	1.00	0.50	0.50	0.50	0	3	16.2	0.02%
40460	9.50	9.60	0.50	0.00	0.50	0.50	10.60	12.60	6.26	5.76	0.50	0.50	0	32	16.2	0.20%
40480	10.10	10.40	1.50	1.00	0.50	0.50	8.90	11.90	9.67	9.17	0.50	0.50	6	51	16.2	0.28%
40500	9.10	9.60	3.55	3.05	0.50	0.50	6.80	11.30	11.79	11.29	0.50	0.50	17	62	16.2	0.28%
40520	7.20	8.20	4.60	4.10	0.50	0.50	4.80	10.80	13.10	12.60	0.50	0.50	23	70	16.2	0.29%
40540	3.70	7.40	8.10	7.60	0.50	0.50	3.60	11.10	15.66	15.16	0.50	0.50	42	84	16.2	0.26%
40550	3.20	7.50	8.78	8.28	0.50	0.50	3.20	11.50	16.16	15.66	0.50	0.50	46	86	16.2	0.25%
40560	2.50	10.00	12.50	12.00	0.50	0.50	2.00	11.00	14.81	14.31	0.50	0.50	66	79	16.2	0.08%
40580	1.00	10.00	13.93	13.43	0.50	0.50	1.00	10.00	14.99	14.49	0.50	0.50	74	80	16.2	0.04%
40600	1.50	10.50	13.98	13.48	0.50	0.50	1.60	10.60	14.36	13.86	0.50	0.50	74	76	16.2	0.01%
40620	1.00	10.00	13.46	12.96	0.50	0.50	1.20	10.20	13.87	13.37	0.50	0.50	71	74	16.2	0.01%
40640	0.50	9.50	13.98	13.48	0.50	0.50	0.65	9.65	14.74	14.24	0.50	0.50	74	79	16.2	0.03%
40660	0.10	9.10	15.67	15.17	0.50	0.50	0.10	9.10	15.64	15.14	0.50	0.50	84	84	16.2	0.00%
40680	0.10	9.10	15.71	15.21	0.50	0.50	0.10	9.10	15.79	15.29	0.50	0.50	84	84	16.2	0.00%
40700	0.20	9.20	14.12	13.62	0.50	0.50	0.20	9.20	15.82	15.32	0.50	0.50	75	85	16.2	0.06%
40720	0.40	9.40	7.13	6.63	0.50	0.50	0.40	9.40	9.77	9.27	0.50	0.50	37	51	16.2	0.09%
40740	2.00	7.00	0.50	0.00	0.50	0.50	1.60	6.60	0.50	0.00	0.50	0.50	0	0	16.2	0.00%

H_D = Design fill thickness; H_T = Total Fill thickness; DL = Landfill thickness; Ds = Soil thickness; D_R = Total overburden; D_i = Depth of improvement by DR prior to preloading

2. INPUT surcharge and OUTPUT following Surcharge

Hs 2N = surcharge thickness over design height H_D at crest of MB2N side
Hs 2S = surcharge thickness over design height H_D at crest of MB2S side

Sp 2N & Sc 2N = assessed primary and secondary settlement at crest of MB2N side
Sp 2S & Sc 2S = assessed primary and secondary settlement at crest of MB2S side

Chainage (m)	**Crest of MB2N (i.e. East Side)**						**Centre of MB2S**						Longitudinal δl (%)		Lateral δl (%)	
	H_S 2N (m)	Δp2N(kPa)	S_p 2N(mm)	CAE2Sm	CAE2Smf	S_s 2Sm(mm)	H_S 2S (m)	Δp2S(kPa)	S_p 2S(mm)	CAE2Sc	CAE2Scf	S_s 2Sc(mm)	MB2N	MB2S	width (m)	δl (%)
40440	0.1	202	7	0.0100	0.0200	5	0.15	304.5	113	0.0100	0.0150	10			16.2	0.03%
40445	0.1	206	7	0.0100	0.0200	5	0.2	293.8	112	0.0100	0.0150	10	0.00%	-0.01%	16.2	0.03%
40460	0.1	202	7	0.0100	0.0200	5	2	262.6	698	0.0100	0.0150	34	0.00%	0.16%	16.2	0.18%
40480	0.3	218	179	0.0100	0.0150	10	3	246.9	889	0.0100	0.0150	51	0.02%	0.08%	16.2	0.25%
40500	0.5	201	406	0.0100	0.0150	20	4.5	232.8	960	0.0100	0.0150	61	0.05%	0.05%	16.2	0.25%
40520	1	171	460	0.0100	0.0150	24	6	220.8	982	0.0100	0.0150	67	0.02%	0.03%	16.2	0.26%
40540	3.7	152	615	0.0100	0.0150	41	7.5	225.6	1082	0.0100	0.0150	80	0.08%	0.07%	16.2	0.24%
40550	4.3	153	646	0.0100	0.0150	44	8.3	233.2	1120	0.0100	0.0150	82	0.03%	0.02%	16.2	0.24%
40560	7.5	203	912	0.0100	0.0150	63	9	222.0	1044	0.0100	0.0150	74	0.19%	-0.08%	16.2	0.07%
40580	9	201	955	0.0100	0.0151	70	9	201.0	988	0.0100	0.0153	76	0.04%	0.01%	16.2	0.04%
40600	9	212	987	0.0100	0.0150	70	9	213.6	1006	0.0100	0.0150	72	0.00%	-0.02%	16.2	0.01%
40620	9	201	940	0.0100	0.0150	67	9	205.2	966	0.0100	0.0150	69	-0.01%	-0.01%	16.2	0.01%
40640	9	191	926	0.0100	0.0152	71	9	193.7	958	0.0100	0.0153	75	0.02%	0.03%	16.2	0.03%
40660	9	182	946	0.0100	0.0157	82	9	182.1	945	0.0100	0.0157	82	0.06%	0.03%	16.2	0.00%
40680	9	182	947	0.0100	0.0157	82	9	182.1	949	0.0100	0.0157	83	0.00%	0.00%	16.2	0.00%
40700	9	184	911	0.0100	0.0154	72	9	184.2	957	0.0100	0.0157	83	-0.05%	0.00%	16.2	0.07%
40720	9	188	644	0.0100	0.0150	33	9	188.4	771	0.0100	0.0150	47	-0.19%	-0.18%	16.2	0.09%
40740	5	142	5	0.0100	0.0200	1	5	133.6	4	0.0100	0.0200	1	-0.16%	-0.23%	16.2	0.00%

3a. Post-construction Settlement Estimate with Time (at CH 40,440)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	
0.333	5	10	
0.25	5	9	6/10
0.75	5	12	6/13
1.25	5	13	6/14
2.1	5	15	6/15
4.2	5	17	6/17
9	5	18	6/19
19	5	20	6/21
40	5	22	6/23

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	5	10	5	10	5	0.03%	
0.833	5	15	0	4	4	0.03%	6/15
1.333	6	20	0	6	6	0.04%	6/21
1.833	6	24	0	4	4	0.02%	6/25
2.683	6	30	0	5	5	0.03%	7/30
4.783	7	38	1	8	7	0.04%	7/38
9.583	9	49	2	11	9	0.06%	9/49
19.583	12	63	4	14	10	0.06%	13/63
40.583	20	83	8	20	13	0.08%	21/84

Foundation designinvolving surcharge preloading along landfill area (Ch40,440 - Ch 40,740 of MB2S) for Developed Concept Design (35%)

3b. Post-construction Settlement Estimate with Time (at CH 40,445)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	5	10
0.25	5	9
0.75	5	12
1.25	5	13
2.1	5	14
4.2	5	16
9	5	18
19	5	20
40	5	22

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	5	10	5	10	4	0.03%	
0.833	5	14	0	4	4	0.03%	6/15
1.333	6	20	0	6	6	0.03%	6/20
1.833	6	24	0	4	4	0.02%	6/24
2.683	6	29	0	5	5	0.03%	7/29
4.783	7	36	1	8	7	0.04%	7/37
9.583	9	47	2	11	9	0.05%	9/48
19.583	13	61	4	14	10	0.06%	13/61
40.583	21	80	8	20	12	0.07%	21/81

3c. Post-construction Settlement Estimate with Time (at CH 40,460)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	5	34
0.25	5	24
0.75	5	61
1.25	5	77
2.1	5	95
4.2	5	117
9	5	143
19	5	144
40	5	146

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	5	34	5	34	29	0.18%	
0.833	5	48	0	14	14	0.09%	6/49
1.333	6	67	0	19	19	0.12%	6/68
1.833	6	80	0	13	13	0.08%	6/81
2.683	6	96	0	16	15	0.09%	7/96
4.783	7	120	1	24	23	0.14%	7/120
9.583	9	149	2	30	28	0.17%	9/150
19.583	12	182	4	33	29	0.18%	13/183
40.583	20	220	8	38	30	0.18%	21/220

3d. Post-construction Settlement Estimate with Time (at CH 40,480)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	10	51
0.25	8	35
0.75	15	94
1.25	18	122
2.1	21	150
4.2	25	187
9	29	228
19	29	230
40	29	232

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	10	51	10	51	40	0.25%	
0.833	15	72	4	21	17	0.11%	15/73
1.333	20	100	6	28	22	0.14%	21/101
1.833	24	120	4	19	15	0.09%	25/120
2.683	29	143	5	23	18	0.11%	30/143
4.783	37	178	8	35	28	0.17%	38/178
9.583	47	221	10	43	33	0.20%	48/222
19.583	60	267	13	46	34	0.21%	60/268
40.583	77	318	17	51	34	0.21%	77/319

3e. Post-construction Settlement Estimate with Time (at CH 40,500)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	20	61
0.25	15	41
0.75	34	115
1.25	43	150
2.1	52	185
4.2	64	231
9	78	283
19	78	284
40	78	286

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	20	61	20	61	41	0.25%	
0.833	28	86	8	26	17	0.11%	29/87
1.333	39	120	11	34	23	0.14%	40/121
1.833	47	143	8	23	15	0.10%	47/144
2.683	56	171	9	28	18	0.11%	56/171
4.783	70	213	14	42	28	0.17%	70/213
9.583	88	264	18	51	33	0.21%	88/265
19.583	108	318	20	54	34	0.21%	108/319
40.583	132	376	24	58	34	0.21%	133/377

3f. Post-construction Settlement Estimate with Time (at CH 40,520)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	24	67
0.25	17	45
0.75	44	128
1.25	56	167
2.1	68	206
4.2	85	259
9	103	316
19	103	318
40	103	320

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	24	67	24	67	43	0.26%	
0.833	34	95	10	28	18	0.11%	35/95
1.333	48	132	14	37	24	0.15%	48/133
1.833	57	157	9	25	16	0.10%	58/158
2.683	68	188	11	30	19	0.12%	69/188
4.783	85	234	17	46	29	0.18%	86/234
9.583	106	289	21	56	35	0.21%	107/290
19.583	130	348	23	58	35	0.22%	130/348
40.583	156	409	27	62	35	0.22%	157/410

Foundation designinvolving surcharge preloading along landfill area (Ch40,440 - Ch 40,740 of MB2S) for Developed Concept Design (35%)

3g. Post-construction Settlement Estimate with Time (at CH 40,540)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	41	80
0.25	27	54
0.75	78	155
1.25	101	202
2.1	125	249
4.2	157	313
9	191	383
19	191	385
40	191	387

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	41	80	41	80	39	0.24%	
0.833	58	114	17	34	17	0.10%	58/114
1.333	80	158	23	44	22	0.13%	81/159
1.833	96	188	15	30	15	0.09%	96/189
2.683	114	224	18	36	18	0.11%	115/225
4.783	142	279	28	55	27	0.17%	143/280
9.583	176	345	34	66	32	0.20%	177/346
19.583	212	414	36	69	33	0.20%	213/415
40.583	250	486	38	72	34	0.21%	251/487

3h. Post-construction Settlement Estimate with Time (at CH 40,550)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	44	82
0.25	30	55
0.75	84	159
1.25	110	208
2.1	136	257
4.2	171	323
9	209	395
19	209	397
40	209	399

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	44	82	44	82	38	0.24%	
0.833	62	117	19	35	16	0.10%	63/117
1.333	87	163	24	46	21	0.13%	87/163
1.833	103	194	17	31	14	0.09%	104/194
2.683	123	231	20	37	17	0.11%	124/231
4.783	154	287	30	56	26	0.16%	154/288
9.583	190	355	37	68	31	0.19%	191/356
19.583	229	426	38	71	32	0.20%	229/426
40.583	269	500	41	74	33	0.20%	270/500

3i. Post-construction Settlement Estimate with Time (at CH 40,560)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	63	74
0.25	42	50
0.75	83	108
1.25	102	136
2.1	122	164
4.2	148	201
9	177	242
19	205	282
40	233	321

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	63	74	63	74	12	0.07%	
0.833	81	109	18	35	16	0.10%	81/109
1.333	105	155	24	46	22	0.13%	105/155
1.833	121	186	16	31	15	0.09%	122/186
2.683	141	223	20	37	18	0.11%	141/223
4.783	171	279	30	56	27	0.16%	171/280
9.583	207	347	36	68	32	0.20%	207/348
19.583	245	418	38	71	33	0.20%	245/419
40.583	285	492	40	74	33	0.21%	286/492

3j. Post-construction Settlement Estimate with Time (at CH 40,580)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	70	76
0.25	46	51
0.75	136	149
1.25	178	195
2.1	220	241
4.2	277	303
9	339	371
19	400	438
40	461	505

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	70	76	70	76	7	0.04%	
0.833	88	112	18	35	17	0.11%	89/112
1.333	112	158	24	47	22	0.14%	113/159
1.833	129	190	16	32	15	0.09%	129/191
2.683	148	228	20	38	18	0.11%	149/229
4.783	178	286	30	58	28	0.17%	179/286
9.583	215	355	36	70	33	0.21%	215/356
19.583	253	428	38	72	34	0.21%	253/428
40.583	293	503	40	75	35	0.22%	294/503

3k. Post-construction Settlement Estimate with Time (at CH 40,600)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	70	72
0.25	46	47
0.75	136	143
1.25	177	188
2.1	219	234
4.2	276	294
9	338	361
19	399	427
40	459	492

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	70	72	70	72	2	0.01%	
0.833	88	107	18	35	16	0.10%	89/107
1.333	112	152	24	46	22	0.13%	113/153
1.833	128	183	16	31	15	0.09%	129/184
2.683	148	221	20	37	18	0.11%	148/221
4.783	178	277	30	57	27	0.17%	178/278
9.583	214	345	36	68	32	0.20%	214/346
19.583	252	416	38	71	33	0.20%	252/417
40.583	292	490	40	74	34	0.21%	292/491

Foundation designinvolving surcharge preloading along landfill area (Ch40,440 - Ch 40,740 of MB2S) for Developed Concept Design (35%)

3l. Post-construction Settlement Estimate with Time (at CH 40,620)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	67	69
0.25	44	44
0.75	133	141
1.25	175	186
2.1	217	231
4.2	274	292
9	336	360
19	397	425
40	458	491

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	67	69	67	69	2	0.01%	
0.833	85	104	18	35	17	0.10%	86/104
1.333	109	150	24	46	22	0.13%	110/150
1.833	126	181	16	31	15	0.09%	126/181
2.683	145	218	20	37	18	0.11%	146/219
4.783	175	275	30	57	27	0.17%	176/275
9.583	211	343	36	68	32	0.20%	212/344
19.583	249	414	38	71	33	0.20%	250/415
40.583	289	488	40	74	34	0.21%	290/489

3m. Post-construction Settlement Estimate with Time (at CH 40,640)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	71	75
0.25	47	49
0.75	138	148
1.25	180	194
2.1	223	241
4.2	281	303
9	344	372
19	406	439
40	468	506

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	71	75	71	75	5	0.03%	
0.833	89	111	19	36	17	0.11%	90/111
1.333	114	158	25	47	22	0.14%	114/158
1.833	130	190	17	32	15	0.09%	131/190
2.683	150	228	20	38	18	0.11%	151/228
4.783	181	286	30	58	28	0.17%	181/286
9.583	218	356	37	70	33	0.20%	218/356
19.583	256	428	39	73	34	0.21%	257/429
40.583	297	504	41	76	35	0.21%	297/504

3n. Post-construction Settlement Estimate with Time (at CH 40,660)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	82	82
0.25	58	55
0.75	151	157
1.25	195	204
2.1	239	251
4.2	298	315
9	363	386
19	426	454
40	490	523

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	82	82	82	82	0	0.00%	
0.833	101	118	19	36	17	0.11%	102/119
1.333	126	166	25	48	23	0.14%	127/167
1.833	144	199	17	33	15	0.10%	144/199
2.683	164	238	21	39	18	0.11%	165/239
4.783	195	297	31	59	28	0.17%	196/298
9.583	233	369	38	72	34	0.21%	234/370
19.583	273	443	40	74	35	0.21%	274/444
40.583	315	521	42	77	35	0.22%	315/521

3p. Post-construction Settlement Estimate with Time (at CH 40,680)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	82	83
0.25	58	56
0.75	151	158
1.25	195	205
2.1	239	253
4.2	298	317
9	363	387
19	427	456
40	490	524

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement		
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)	
0.583	82	83	82	83	1	0.00%	
0.833	101	119	19	37	17	0.11%	102/120
1.333	127	168	25	48	23	0.14%	127/168
1.833	144	200	17	33	16	0.10%	144/201
2.683	164	239	21	39	19	0.11%	165/240
4.783	196	299	31	59	28	0.17%	196/299
9.583	234	370	38	72	34	0.21%	234/371
19.583	273	445	40	74	35	0.21%	274/445
40.583	315	522	42	77	35	0.22%	316/523

3q. Post-construction Settlement Estimate with Time (at CH 40,700)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	72	83
0.25	48	56
0.75	140	158
1.25	183	205
2.1	226	252
4.2	284	316
9	348	386
19	410	455
40	473	523

(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement	
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)

3r. Post-construction Settlement Estimate with Time (at CH 40,720)

(a) Assessed settlement vs Time

Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)
0.333	33	47
0.25	9	22
0.75	101	120
1.25	144	165
2.1	187	212
4.2	245	274
9	308	342

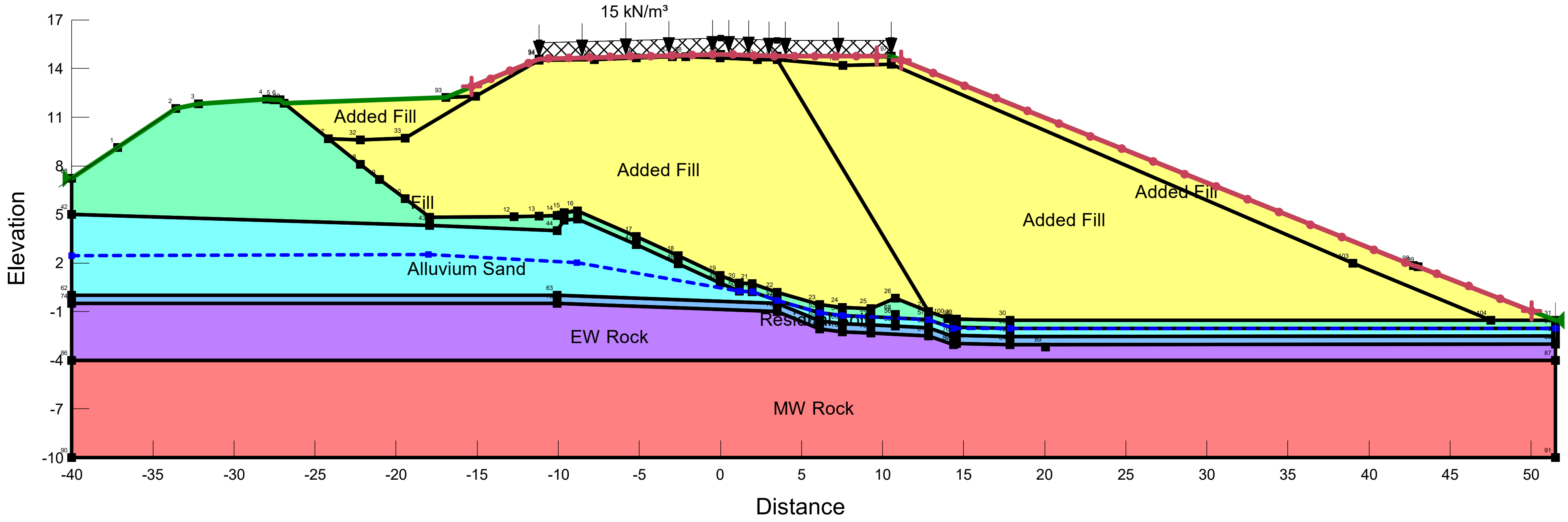
(b) Intervention when total settlement exceeds 100 mm or dl exceeds 0.25% since last intervention

No intervention			Change since last intervention		Lateral Differential Settlement	
Time (yrs)	S_s 2Sm (mm)	S_s 2Sc (mm)	ΔS_s 2N (mm)	ΔS_s 2S (mm)	Diff (mm)	δl (%)

Appendix E – Output of Slope/W analyses

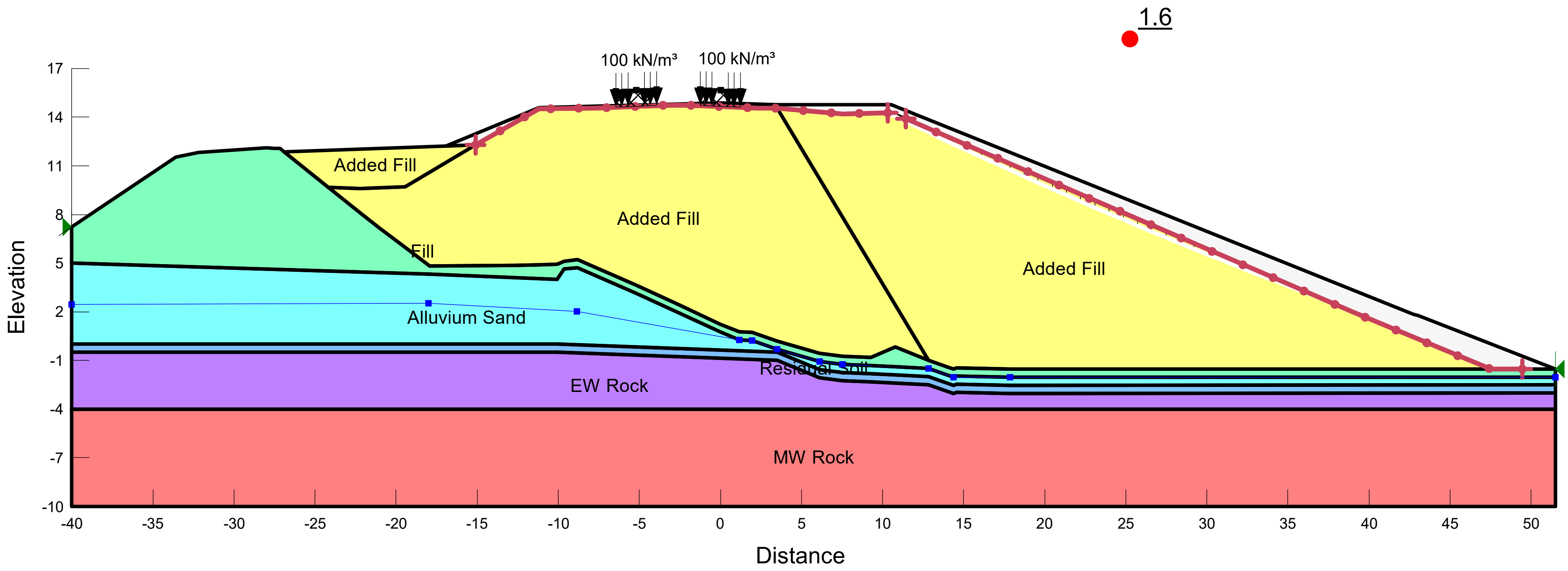
Moorebank Intermodal Rail Link
Description: Chainage 40440 (MB2S)
Case: Short Term
Load: Construction Loading

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Added Fill	Mohr-Coulomb	21	0	34	1
	Fill	Mohr-Coulomb	20	5	30	1
	Alluvium Sand	Mohr-Coulomb	20	0	33	1
	Residual Soil	Mohr-Coulomb	20	5	26	1
	EW Rock	Mohr-Coulomb	20	10	30	1
	MW Rock	Mohr-Coulomb	20	200	30	1



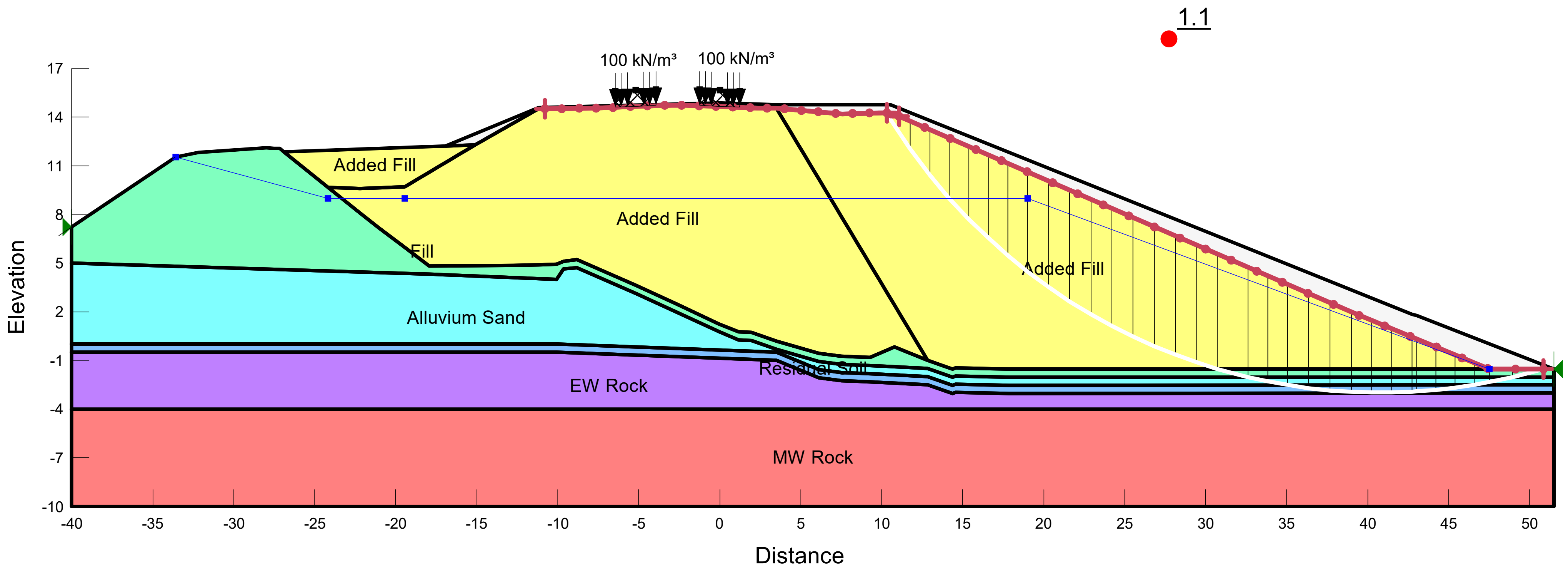
Moorebank Intermodal Rail Link
Description: Chainage 40445 (MB2S)
Case: Long Term
Load: Transient Track Load

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Added Fill	Mohr-Coulomb	21	0	34	1
	Fill	Mohr-Coulomb	20	5	30	1
	Alluvium Sand	Mohr-Coulomb	20	0	33	1
	Residual Soil	Mohr-Coulomb	20	5	26	1
	EW Rock	Mohr-Coulomb	20	10	30	1
	MW Rock	Mohr-Coulomb	20	200	30	1



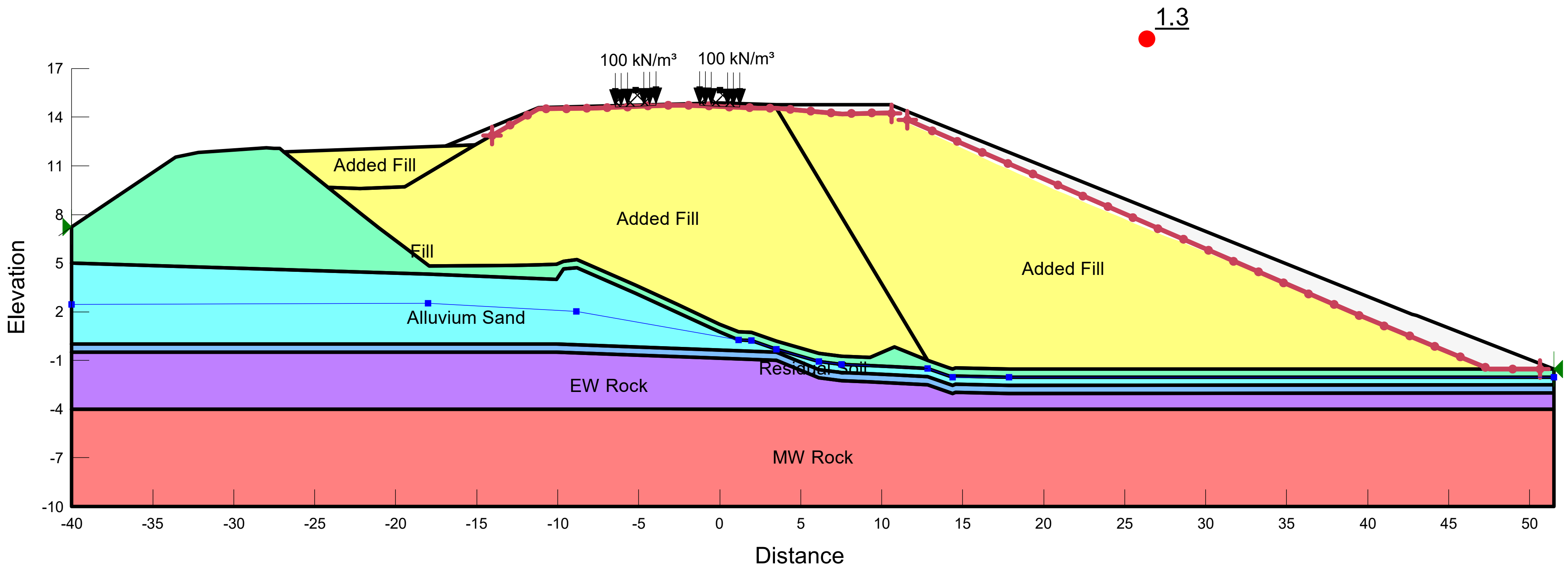
Moorebank Intermodal Rail Link
Description: Chainage 40445 (MB2S)
Case: Rapid Drawdown during Long Term
Load: Transient Track Load

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Added Fill	Mohr-Coulomb	21	0	34	1
	Fill	Mohr-Coulomb	20	5	30	1
	Alluvium Sand	Mohr-Coulomb	20	0	33	1
	Residual Soil	Mohr-Coulomb	20	5	26	1
	EW Rock	Mohr-Coulomb	20	10	30	1
	MW Rock	Mohr-Coulomb	20	200	30	1



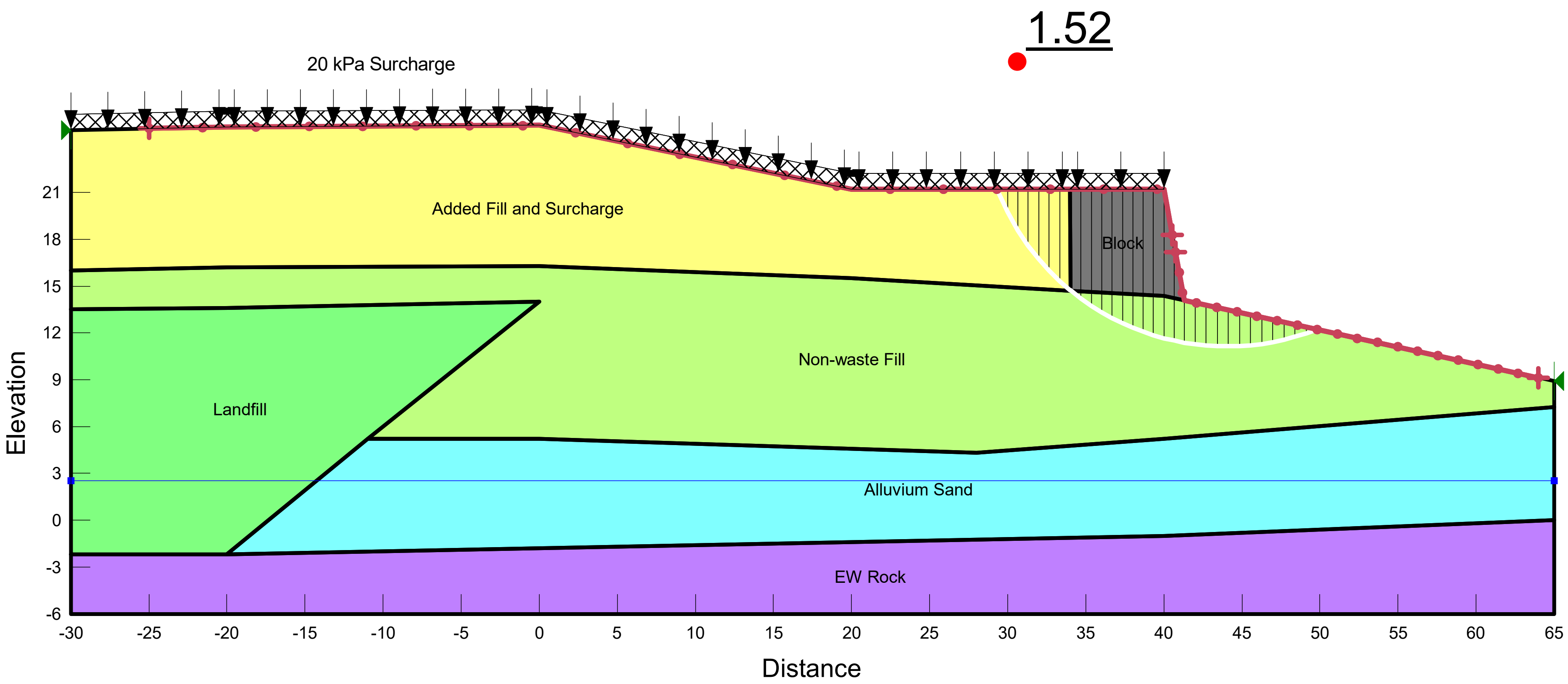
Moorebank Intermodal Rail Link
Description: Chainage 40445 (MB2S)
Case: Seismic during Long Term
Load: Transient Track Load (Seismic Coefficient of 0.08)

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Added Fill	Mohr-Coulomb	21	0	34	1
	Fill	Mohr-Coulomb	20	5	30	1
	Alluvium Sand	Mohr-Coulomb	20	0	33	1
	Residual Soil	Mohr-Coulomb	20	5	26	1
	EW Rock	Mohr-Coulomb	20	10	30	1
	MW Rock	Mohr-Coulomb	20	200	30	1



Stability Analysis (Ch 40740 - MB2S)
Long Section

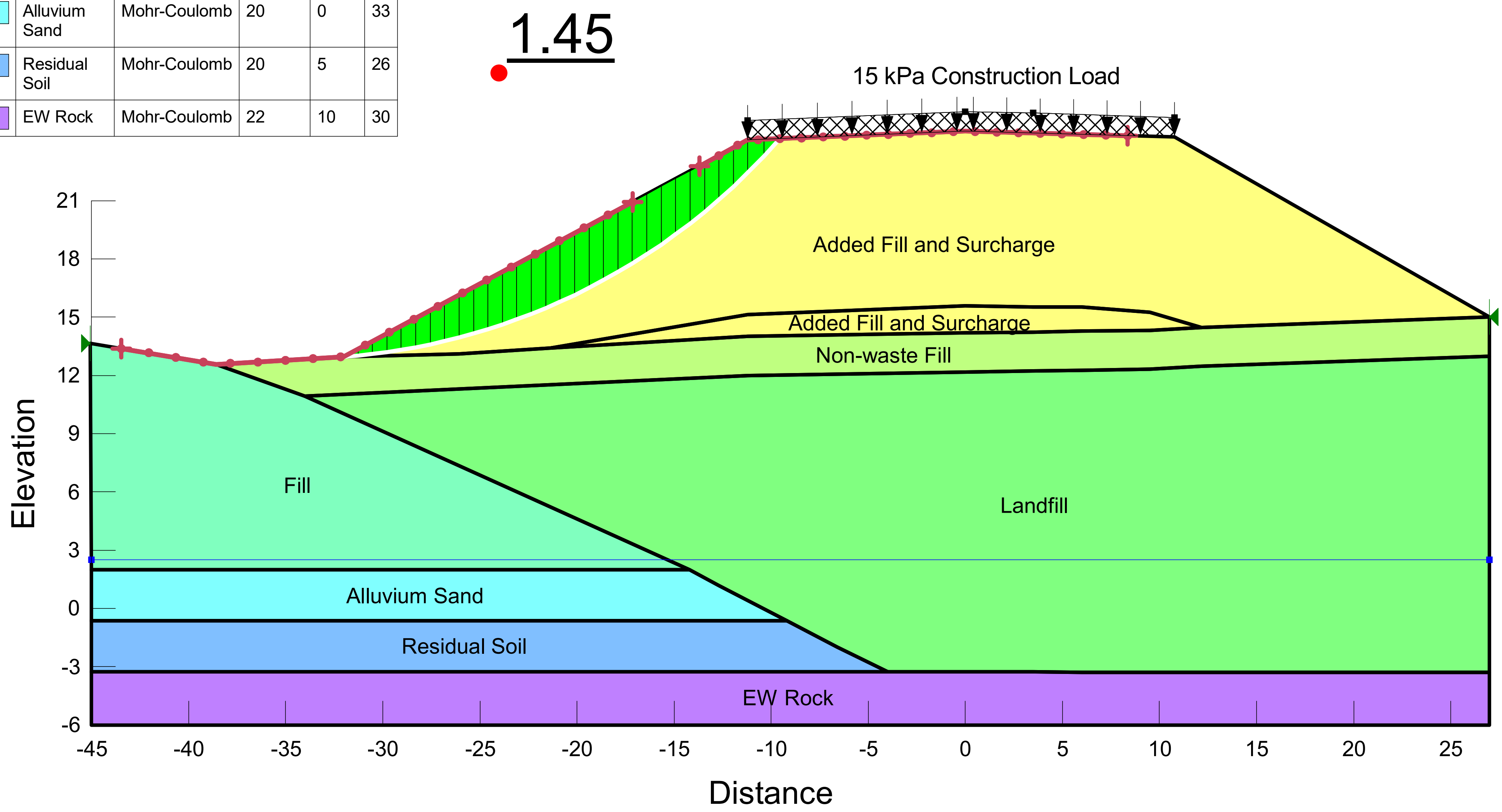
			(kN/m³)	(kPa)	(°)
	Added Fill and Surcharge	Mohr-Coulomb	21	5	30
	Non-waste Fill	Mohr-Coulomb	20	2	33
	Landfill	Mohr-Coulomb	13.6	0	30
	Alluvium Sand	Mohr-Coulomb	20	0	33
	EW Rock	Mohr-Coulomb	22	10	30
	Block	Mohr-Coulomb	24	200	0



			(kN/m³)	(kPa)	(°)
	Added Fill and Surcharge	Mohr-Coulomb	21	5	30
	Non-waste Fill	Mohr-Coulomb	20	2	33
	Landfill	Mohr-Coulomb	13.6	0	30
	Fill	Mohr-Coulomb	20	5	30
	Alluvium Sand	Mohr-Coulomb	20	0	33
	Residual Soil	Mohr-Coulomb	20	5	26
	EW Rock	Mohr-Coulomb	22	10	30

Stability Analysis (Ch 40620 - MB2S)

Short Term Analysis with Surcharge Load



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Agrément Certificate
03/4065
Product Sheet 1

LINEAR COMPOSITES' SOIL REINFORCEMENT PRODUCTS

PARALINK GEOCOMPOSITES

PRODUCT SCOPE AND SUMMARY OF CERTIFICATE

This Certificate relates to Paralink Geocomposites, for use as basal reinforcement in embankment foundations.

AGRÉMENT CERTIFICATION INCLUDES:
- factors relating to compliance with Building Regulations where applicable
- factors relating to additional non-regulatory information where applicable
- independently verified technical specification
- assessment criteria and technical investigations
- design considerations
- installation guidance
- regular surveillance of production
- formal three-yearly review.

KEY FACTORS ASSESSED

Mechanical properties — short-term and long-term tensile strength and strain properties of the geocomposites have been assessed (see section 6).

Partial material factors — partial material factors for manufacture (f_{m11}), extrapolation of test data (f_{m12}), installation damage (f_{m21}) and environmental effects (f_{m22}) have been established (see section 7).

Soil/geocomposite interaction — interaction coefficients relating to direct sliding and pull-out resistance have been evaluated (see section 8).

Durability — the geocomposites have good resistance to chemical degradation, biological degradation, temperature and weathering used in fills normally encountered in civil engineering practice (see section 10).

The BBA has awarded this Agrément Certificate to the company named above for the products described herein. These products have been assessed by the BBA as being fit for their intended use provided they are installed, used and maintained as set out in this Certificate.

On behalf of the British Board of Agrément [redacted]

Date of First issue: 21 July 2010

Originally certificated on 3 December 2003

Certificate amended 18 November 2011 to replace Figures [redacted]

The BBA is a UKAS accredited certification body — Number 113. The schedule of the current scope of accreditation for product certification is available in pdf format via the UKAS link on the BBA website at www.bbacerts.co.uk

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Regulations

The Building Regulations 2000 (as amended) (England and Wales)

In the opinion of the BBA, Paralink Geocomposites for use as basal reinforcements are not subject to these Regulations.

The Building (Scotland) Regulations 2004 (as amended)

In the opinion of the BBA, Paralink Geocomposites for use as basal reinforcements are not controlled under these Regulations.

The Building Regulations (Northern Ireland) 2000 (as amended)

In the opinion of the BBA, Paralink Geocomposites for use as basal reinforcements are not controlled under these Regulations.

Construction (Design and Management) Regulations 2007

Construction (Design and Management) Regulations (Northern Ireland) 2007

Information in this Certificate may assist the client, CDM co-ordinator, designer and contractors to address their obligations under these Regulations.

See sections: 2 *Delivery and site handling* (2.1 and 2.4) and 11 *General* of this Certificate.

Non-regulatory Information

NHBC Standards 2010

In the opinion of the BBA, the use of Paralink Geocomposites, in relation to this Certificate, is not subject to the requirements of these Standards.

General

This Certificate relates to Paralink Geocomposites, for use as basal reinforcement under embankments where the following foundation conditions exist:

- soft foundation soils
- piled foundations
- areas prone to subsidence.

Paralink Geocomposites are planar structures consisting of a regular array of composite geosynthetic straps, nominally interconnected laterally to form soil reinforcement materials with high unidirectional strength.

The design and construction of embankments must be in accordance with the conditions set out in the *Design Considerations* and *Installation* parts of this Certificate.

Technical Specification

1 Description

1.1 Paralink Geocomposites are planar structures consisting of a regular array of composite geosynthetic straps, nominally interconnected laterally to form soil reinforcement materials with high unidirectional strength.

1.2 The straps comprise polyester tendons encased in a polyethylene sheath. The composite is passed through rollers to give a knurled finish on the sheath. They are cooled and cut to length. The products are formed by heat-bonding widely spaced composites of nominal strength across an array of the straps to produce a nominal 4.5 metre wide planar structure.

1.3 The products are identified on site by clear marking of the product type and grade, along the length of the roll. The range of specification of the geocomposites assessed by the BBA is given in Tables 1 and 2. A typical Paralink Geocomposite is shown in Figure 1.

Table 1 General specification

Grade[1]	Mass[2] (±5.0%) (g·m^{-2})	Grid size[3] warp/weft A x B (mm)	Aperture size[2] warp/weft C x D (mm)	Standard roll length (m) (+1/–0%)	Roll weight (kg) (±5%)
100	425	180 x 1000	98 x 940	200	440
150	515	180 x 1000	95 x 940	200	520
200	590	180 x 1000	95 x 940	200	590
250	697	180 x 1000	95 x 940	200	690
300	789	180 x 1000	92 x 940	200	770
350	890	180 x 1000	91 x 940	150	660
400	1014	180 x 1000	90 x 940	150	750
450	1124	180 x 1000	90 x 940	130	720
500	1219	180 x 1000	90 x 940	130	780
550	1410	180 x 1000	90 x 940	100	700
600	1507	180 x 1000	90 x 940	100	750
650	1681	180 x 1000	89 x 940	100	830
700	1835	180 x 1000	89 x 940	50	480
750	1970	150 x 1000	59 x 940	50	510
800	2135	150 x 1000	59 x 940	50	550
850	2221	125 x 1000	34 x 940	50	570
900	2351	125 x 1000	34 x 940	50	600
950	2543	125 x 1000	34 x 940	50	640
1000	2616	125 x 1000	34 x 940	50	660
1050	2695	100 x 1000	9 x 940	50	680
1100	2829	100 x 1000	9 x 940	50	710
1150	3018	100 x 1000	9 x 940	50	750
1200	3171	100 x 1000	9 x 940	50	790
1250	3254	100 x 1000	9 x 940	50	800
1300	3475	100 x 1000	9 x 940	50	860
1350	3674	100 x 1000	9 x 940	50	900

(1) Intermediate grades are available on request and are covered by this Certificate.

(2) Mass/unit area measured in accordance with BS EN ISO 9864 : 2005.

(3) Mean measured dimensions (see Figure 1 for reference).

Table 2 Performance characteristics

Grade	Short-term tensile strength[1] in warp direction T_{ult} (T_{char}) (kN·m^{-1} width)	α_s[2]	Ratio of bearing[3] surface to plan area $\alpha_b \times B/2S$	Strain at maximum tensile strength[4] (%)
100	103 (–2.4)	0.49	0.00022	10.5 ± 1
150	154 (–3.2)	0.50	0.00021	10.5 ± 1
200	206 (–4.9)	0.50	0.00021	10.5 ± 1
250	257 (–5.6)	0.50	0.00021	10.5 ± 1
300	309 (–7.4)	0.52	0.00020	10.5 ± 1
350	360 (–8.1)	0.52	0.00020	10.5 ± 1
400	412 (–9.8)	0.53	0.00020	10.5 ± 1
450	463 (–10.5)	0.53	0.00020	10.5 ± 1
500	515 (–12.3)	0.53	0.00020	10.5 ± 1
550	566 (–13)	0.53	0.00020	10.5 ± 1
600	612 (–8.8)	0.53	0.00020	10.5 ± 1
650	669 (–15.5)	0.54	0.00020	10.5 ± 1
700	721 (–17.2)	0.54	0.00020	10.5 ± 1
750	772 (–18)	0.63	0.00016	10.5 ± 1
800	826 (–21.7)	0.63	0.00016	10.5 ± 1
850	875 (–20.5)	0.74	0.00011	10.5 ± 1
900	927 (–22.1)	0.74	0.00011	10.5 ± 1
950	980 (–23.4)	0.74	0.00011	10.5 ± 1
1000	1038 (–24.8)	0.74	0.00011	10.5 ± 1
1050	1081 (–25.4)	0.92	0.00004	10.5 ± 1
1100	1133 (–27.1)	0.92	0.00004	10.5 ± 1
1150	1184 (–27.8)	0.92	0.00004	10.5 ± 1
1200	1236 (–29.5)	0.92	0.00004	10.5 ± 1
1250	1287 (–30.0)	0.92	0.00004	10.5 ± 1
1300	1339 (–32.0)	0.92	0.00004	10.5 ± 1
1350	1390 (–32.8)	0.92	0.00004	10.5 ± 1

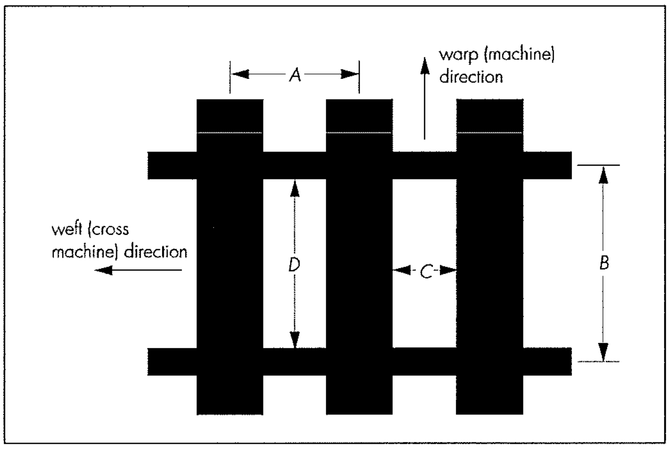
1) Short-term tests in accordance with BS EN ISO 10319 : 2008; the values given are mean values of ultimate strength (T_{ult}) and tolerance (–) values correspond to the 95% confidence level to establish the characteristic short-term tensile strength (T_{char}) in accordance with BS EN 13251 : 2001

(2) α_s is the proportion of the plane sliding area that is solid and is required for the calculation of the bond coefficient f_b and the direct sliding coefficient f_{ds} (see sections 8.1 and 8.3)

(3) The ratio is required to calculate the bond coefficient f_b in accordance with CIRIA SP123 : 1996 *Soil Reinforcement with Geotextiles*, Jewell R.A (see section 8.4) where:
- α_b is the proportion of the width available for bearing
- B is the thickness of a transverse member taking bearing
- S is the spacing between transverse members taking bearing (equivalent to B in Figure 1)

(4) Tests in accordance with BS EN ISO 10319 : 2008; the values given are the mean and tolerance values (±) of strain in accordance with BS EN 13251 : 2001.

Figure 1 Paralink



1.4 Product quality is maintained by statistical process control at the point of manufacture.

2 Delivery and site handling

2.1 Paralink Geocomposites are delivered to site in rolls nominally 4.5 m wide, edge to edge of roll, and approximately 4.6 m wide end to end of the central lifting tube. The roll length is normally 50 m, 100 m, 130 m, 150 m or 200 m depending upon the grade, although non-standard lengths can be produced on request. Roll diameters and weights vary, as indicated in Tables 1 and 2. Each roll is wrapped in black polyethylene for transit and site protection. Each package is labelled in accordance with BS EN ISO 10320 : 1999. Packaging should not be removed until immediately prior to installation. Each roll has the product grade marked at regular intervals for identification.

2.2 Rolls should be stored in clean, dry conditions. The rolls should be protected from mechanical or chemical damage and extreme temperatures. Toxic fumes are given off if the geogrids catch fire and, therefore, the necessary precautions should be taken following the instructions of the material safety data sheet for the product.

2.3 To prevent damage, care should be taken in the handling and lifting of the rolls. The weight of the rolls is such that mechanical lifting arrangements are necessary.

2.4 Rolls should be stacked not more than three rolls high. Other loads should not be stored on top of the stack.

Assessment and Technical Investigations

The following is a summary of the assessment and technical investigations carried out on Paralink Geocomposites.

Design Considerations

3 General

3.1 Design of basal reinforcements should be in accordance with the recommendations of BS 8006 : 1995.

3.2 Prior to, during and after installation, particular care should be taken to ensure:

- site preparation and foundation construction is as detailed in sections 11 to 13
- fill properties satisfy the design specification
- drainage is adequate at all stages of construction, as required by the contract documents
- the geocomposites are protected against damage from site traffic and installation equipment
- the stability of existing structures is not affected.

4 Practicability of installation

The products are easily installed by trained ground engineering contractors in accordance with the specifications and construction drawings (see the *Installation* part of this Certificate).

5 Design considerations

5.1 The design should be carried out by a suitably qualified engineer, taking into account all requisite partial material factors (f_m) described in section 7 and applying all other appropriate load factors, soil material factors and soil/reinforcement interaction factors in accordance with BS 8006 : 1995.

5.2 The ultimate limit state design strength of the reinforcement (T_D), should be taken as T_{CR}/f_m, where:

- T_{CR} = the characteristic tensile creep strength of the reinforcement, at the appropriate times and design temperature (see section 6.4)
- f_m = the partial material factor for the reinforcement (see section 7).

5.3 The serviceability limit state design strength of the reinforcement (T_D), should be taken as T_{CS}/f_m, where:

- T_{CS} = the maximum tensile load in the reinforcement which does not cause the prescribed serviceability limit state strain (ϵ_{max}) to be exceeded during the design life (see section 6.6)
- f_m = the partial material factor for the reinforcement (see section 7).

5.4 Guidance on soil/geocomposite interaction coefficients applied to calculate direct sliding and pull-out resistance can be found in section 8.

5.5 Working drawings should show the correct orientation of the geocomposites.

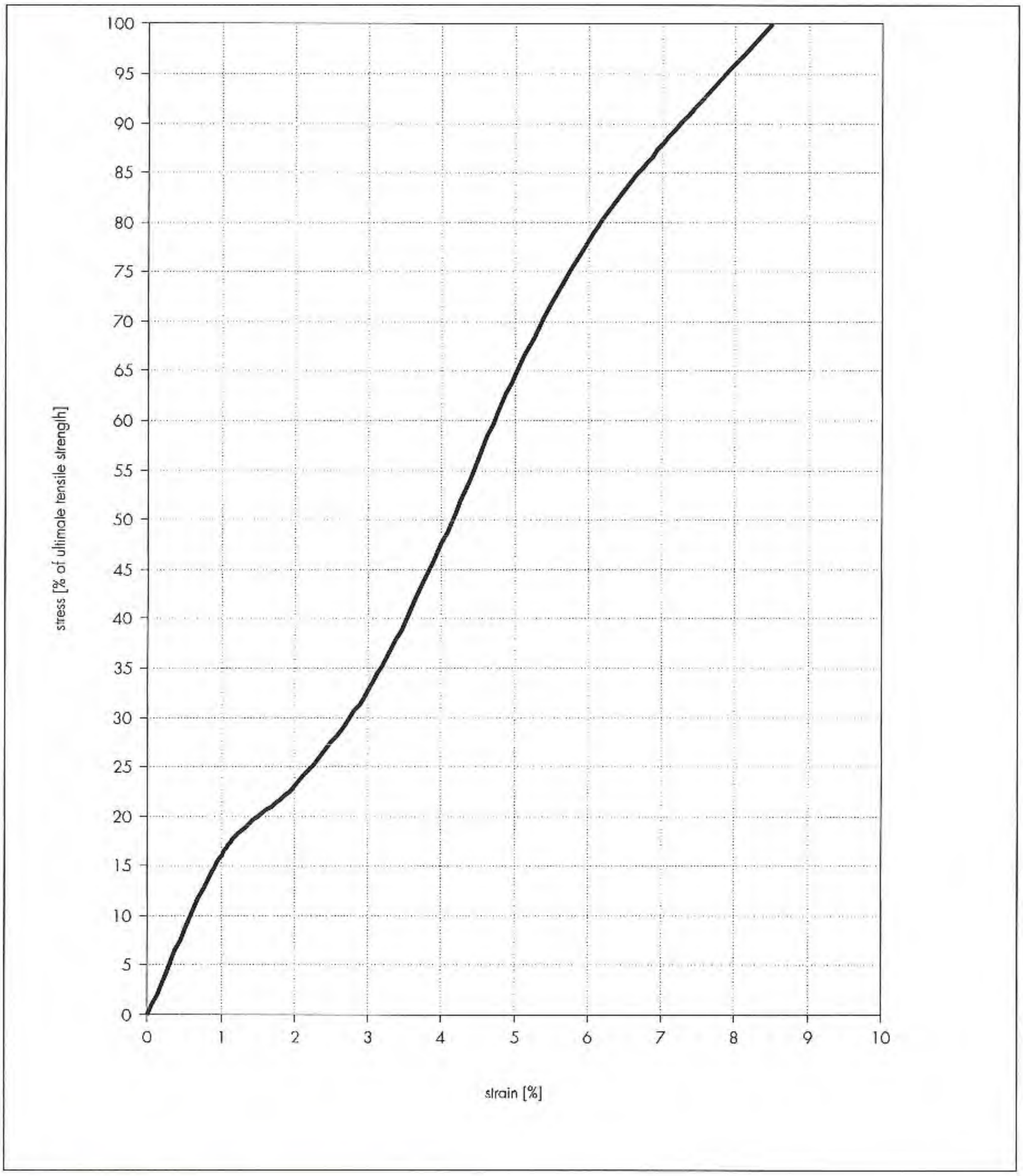
5.6 The designer should specify the relevant properties of the fill material for the foundation deemed acceptable for the purposes of the design. Acceptable materials should meet the requirements of the *Manual of Contract Documents for Highway Works* (MCHW), Volume 1.

6 Mechanical properties

Tensile strength and strain — short-term

6.1 The short-term values of tensile strength and strain for the geocomposites are given in Table 2. A typical short-term stress/strain curve is shown in Figure 2.

Figure 2 Typical short-term stress/strain curve



Tensile strength — long-term

6.2 Long-term creep strain and rupture testing, generally in accordance with the principles of BS EN ISO 13431 : 1999, has been carried out for periods in excess of 10 years and at varying test temperatures, to cover the range of Paralink detailed in this Certificate.

6.3 Real time data has been extrapolated by <1.0 log cycles to allow the characteristic long-term strength (T_{CR}) for design lives of up to 120 years to be determined. Therefor creep factor is < 2.0

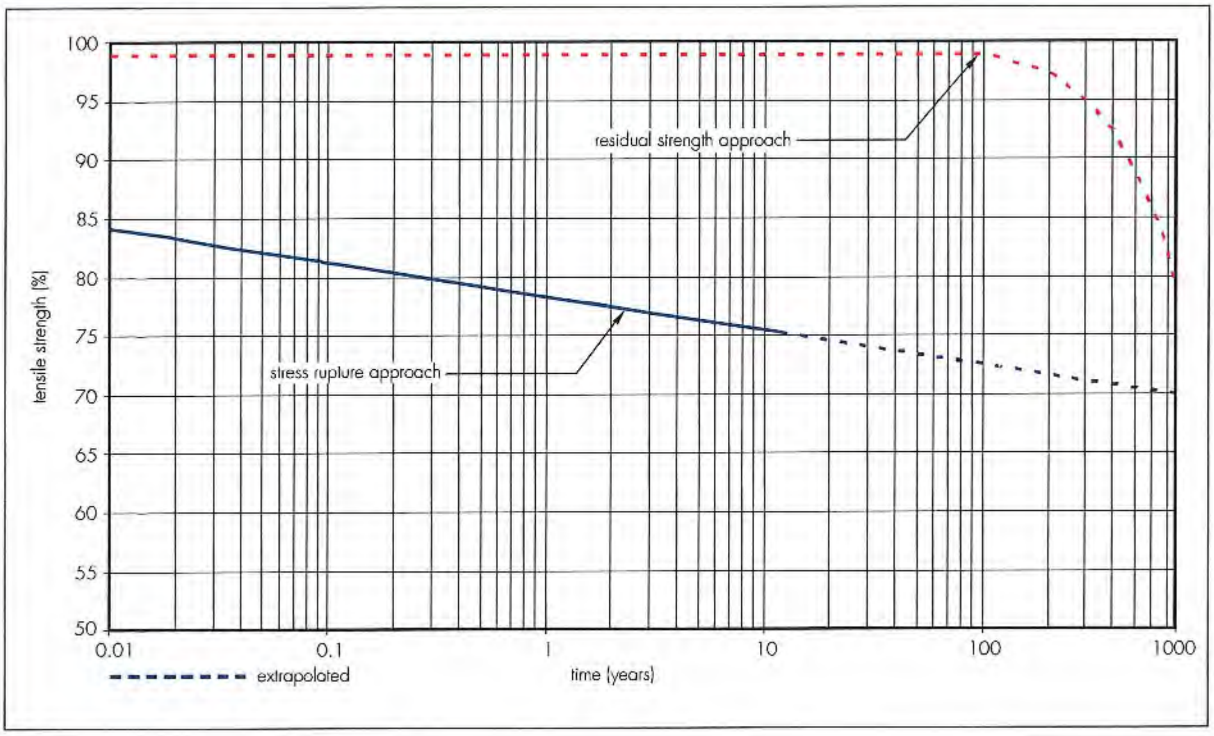
6.4 For ultimate limit state, the value of T_{CR} is a percentage of the characteristic short term tensile strength (T_{char}) (see Figure 3) at various design temperatures and design life as shown in Table 3. The characteristic short-term tensile strength values (T_{char}) are given in Table 2.

Table 3 Percentages of T_{char} to determine T_{CR} at various temperatures and design life

Design temperature (°C)	Percentage of T_{char}		
	2-year design life	60-year design life	120-year design life
20	77	73	72
25	76	72	71
30	74	70	69

6.5 An alternative approach to determine the long-term strength is one of residual strength (see Figure 3), particularly in respect of the strength available during seismic events. Such an approach is outside the scope of this Certificate and would require separate evaluation and justification of the partial safety factor components.

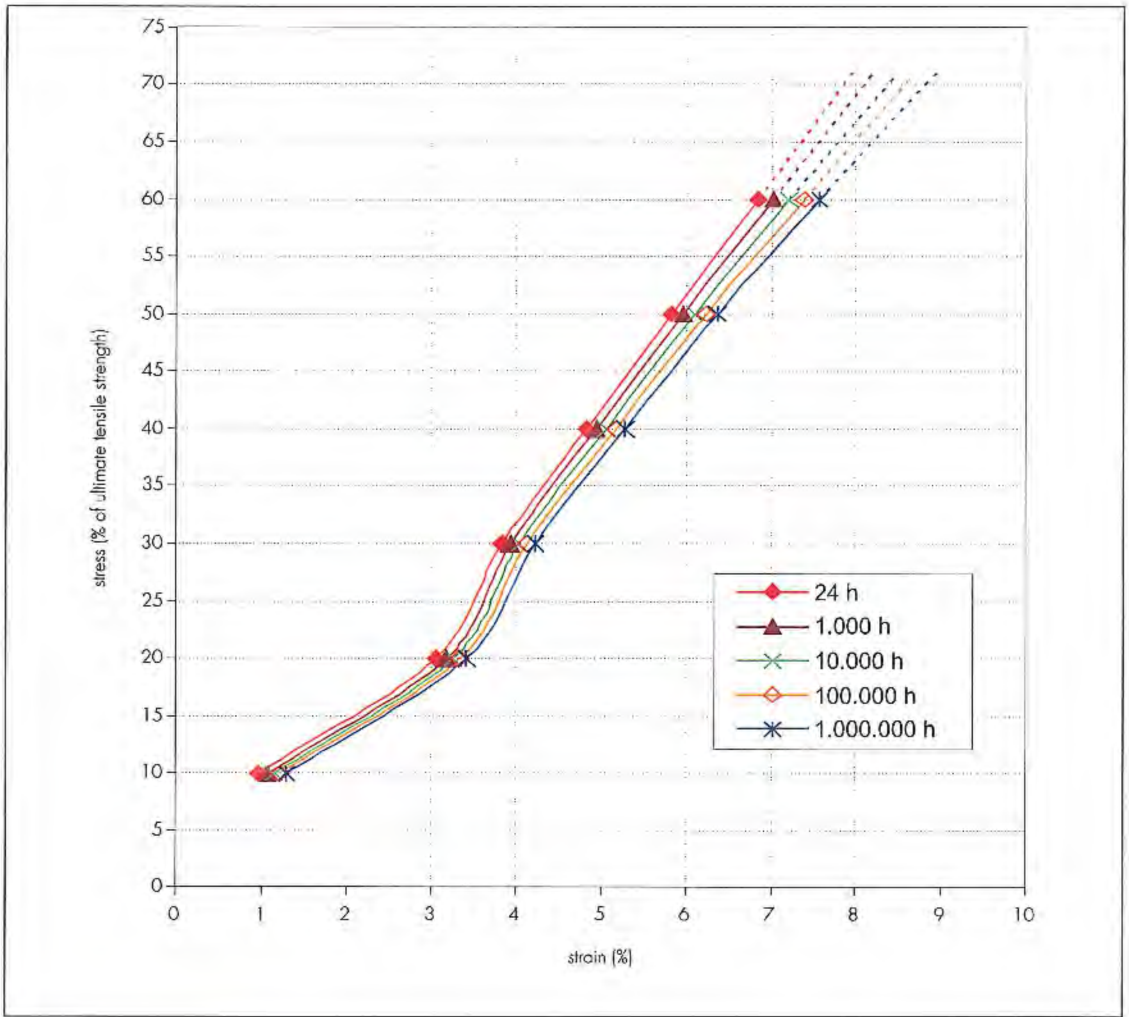
Figure 3 Regression line for life expectancy at constant stress defined by percentage of characteristic short-term strength at 20°C



Creep

6.6 The isochronous curves for Paralink are given in Figure 4 and can be used to predict strain under load over the design life of the structure. If strain is limiting, the critical load can be established for a given design life. As a general guide, the maximum strain ϵ_{max} in the basal reinforcement used for soft foundation soil should not exceed 5% for short-term applications and 5% to 10% for long-term conditions. For piled foundations the practical upper limit of short-term tensile strain is 6% and the allowed long-term strain due to creep should not exceed 2% over the initial strain. For areas prone to subsidence, the maximum allowable reinforcement strain should be calculated in accordance with BS 8006 : 1995, section 8.4.4.4.

Figure 4 Stress/strain isochronous curves



7 Partial material factors

7.1 In establishing the design tensile strength of Paralink Geocomposites and ensuring that during the life of the reinforced soil structure the geocomposite will not fail in tension, the BBA recommends that in line with BS 8006 : 1995, a set of partial material safety factors for both the ultimate (ULS) and serviceability (SLS) limit states should be applied to T_{CR} and T_{CS}. Conditions of use outside the scope for which partial safety factors are defined (see also sections 7.3 to 7.10) are not covered by this Certificate and advice should be sought from the manufacturer.

7.2 The total material factor (f_m), is given by $f_m = f_{m11} \times f_{m12} \times f_{m21} \times f_{m22}$, where:

- f_{m11} is a material factor relating to manufacture
- f_{m12} is a material factor relating to extrapolation of test data
- f_{m21} is a material factor relating to susceptibility of installation damage
- f_{m22} is a material factor relating to environmental effects.

Manufacture — partial material factor (f_{m11})

7.3 For Paralink Geocomposites a characteristic base strength is specified and the partial material factor (f_{m11}) can be taken as 1.0 for both ULS and SLS.

Extrapolation of test data — partial material factor (f_{m12})

7.4 To account for extrapolation of data the values for the partial material factor (f_{m12}) can be taken as 1.0 for both ULS and SLS for a 2-year, 60-year or 120-year design life.

Installation damage — partial material factor (f_{m21})

7.5 To allow for loss of strength due to mechanical damage that may be sustained during installation, the appropriate value for f_{m21} for ultimate limit state (ULS) may be selected from Table 4. These partial material factors were established from full-scale installation damage tests using a range of materials whose gradings can be seen in Figure 5. For soils not covered by Table 4, appropriate values of f_{m21} may be determined from site-specific trials.

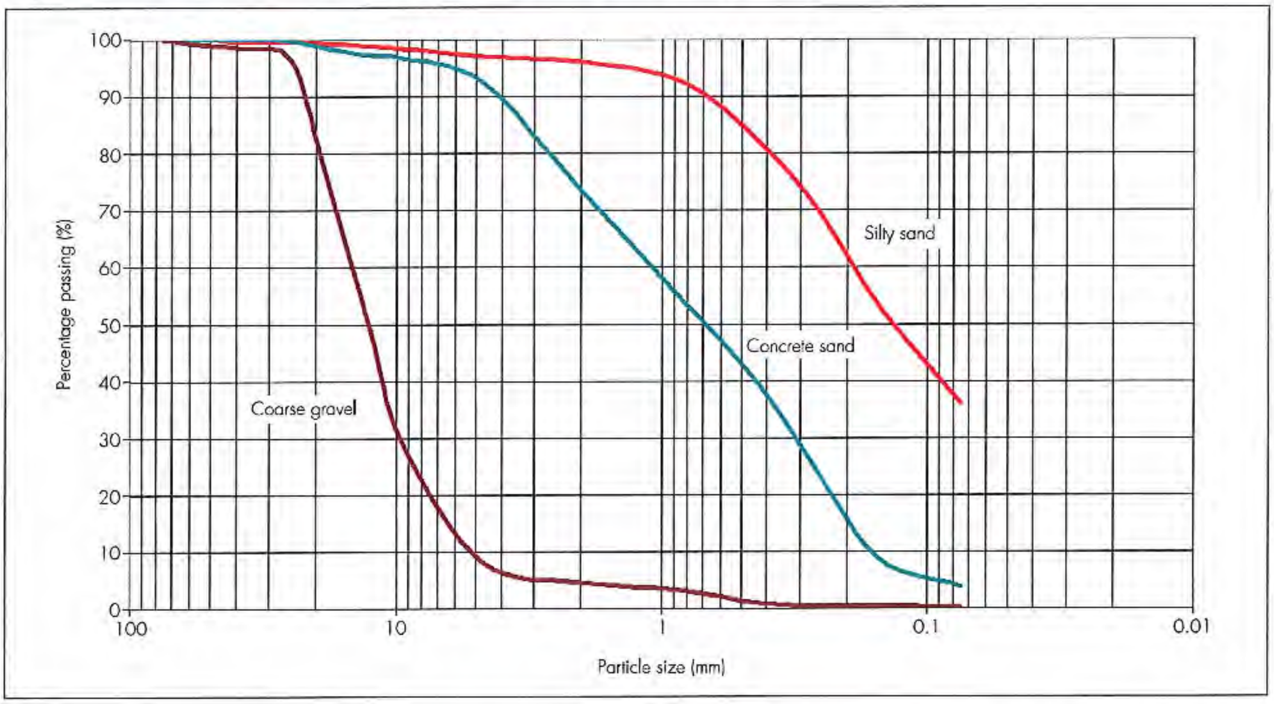
Table 4 Partial material factor — installation damage (f_{m21})

Soil type	D_{50} particle size[1] (mm)	D_{90} particle size[1] (mm)	Paralink range	Partial material factor (f_{m21})
Silty sand[2]	0.15	0.70	300 – 450	1.01
			500 – 650	1.01
			700 – 1350	1.00
Concrete sand[2]	1.0	4.0	300 – 450	1.02
			500 – 650	1.02
			700 – 1350	1.01
Coarse gravel[2]	13	23	300 – 450	1.05
			500 – 650	1.03
			700 – 1350	1.02

(1) Detailed particle size distributions are shown in Figure 5

(2) Depth of soil layer before compacting: 200 mm
Weight of vibrating roll: 1600 kg·m^{-1}
Number of passes: 8.

Figure 5 Particle size distributions of soils used in installation damage testing



7.6 For Paralink range 100 to 250, a cautionary value of at least 1.10 should be applied in the absence of test data.

7.7 For the serviceability limit state (SLS), the value of f_{m21} may be taken as 1.0.

Environmental effects — partial material factor (f_{m22})

7.8 The polyethylene sheath used on Paralink acts as a chemical barrier which, if not broken or damaged, will reduce the risk of chemical attack on the polyester fibres. It should be noted that the most aggressive fills are usually of fine particle sizes which cause little or no damage to the polyethylene sheath. Compaction can reduce the high pH level of a fill. Tests have shown that, 48 hours after the compaction stage, the pH level of a soil–lime mix reduces from 12.5 to 11. Where appropriate, site- and soil-specific testing should be carried out to verify the reduction.

7.9 To account for environmental conditions, the appropriate value for f_{m22} for ultimate limit state (ULS) should be selected from Table 5.

Table 5 Partial material factor — environmental effects (f_{m22})

Design temperature (°C)	Partial safety factor (f_{m22})					
	2-year design life		60-year design life		120-year design life	
	Soil pH level 4.0 – 9.5	Soil pH level 9.6 – 11.0	Soil pH level 4.0 – 9.5	Soil pH level 9.6 – 11.0	Soil pH level 4.0 – 9.5	Soil pH level 9.6 – 11.0
20	1.00	1.01	1.05	1.09	1.10	1.17
25	1.00	1.01	1.07	1.11	1.14	1.21
30	1.01	1.02	1.12	1.16	1.23	1.31

7.10 For the serviceability limit state (SLS), the value of f_{m22} may be taken as 1.0.

8 Soil/geocomposite interaction

Direct sliding

8.1 The theoretical expression for direct sliding recommended for design is:

$f_{ds} \times \tan \phi'$ where: f_{ds} is the direct sliding coefficient.

$f_{ds} = \alpha_s \times (\tan \delta/\tan \phi') + (1 - \alpha_s)$

where: $(\tan \delta/\tan \phi')$ is the coefficient of skin friction (f_{sf}) [synonymous with the term 'friction coefficient (α')' defined in BS 8006 : 1995], and

α_s is the proportion of plane sliding area that is solid (see Table 2).

8.2 When calculating f_{ds}, the coefficient of skin friction (f_{sf}) for the product may be assumed, for routine design purposes, to be 0.7 and 0.4 for compacted frictional fill ($\phi' = 30°$) and compacted cohesive fill ($\phi' = 15°$) respectively. This is a conservative value. Where more precise values are required, for use in design, suitable soil and geocomposite specific shear box testing may be carried out.

Pull-out resistance (bond strength)

8.3 The theoretical expression for bond is:

$f_b \times \tan \phi'$ where: f_b is the bond coefficient.

8.4 For routine design purposes, values may be estimated using the calculation method of Jewell (CIRIA SP123, 1996 *Soil Reinforcement with Geotextiles*, section 4.6). When calculating f_b, the coefficient of skin friction [$f_{sf} = (\tan \delta/\tan \phi')$ — synonymous with the term 'friction coefficient (α')' defined in BS 8006 : 1995] for the product may be assumed conservatively, for routine design purposes, to be 0.7 and 0.4 for compacted frictional fill ($\phi' = 30°$) and compacted cohesive fill ($\phi' = 15°$) respectively, and the ratio of bearing surface to plane area can be taken from Table 2. Significantly enhanced values of f_b can be justified in design by carrying out site- and soil-specific pull-out tests in accordance with BS EN 13738 : 2004. Values of $f_{sf} > 1.0$ have been reported based on site- and soil-specific testing.

Formulae notation

δ = angle of friction between soil and plane reinforcement surface

ϕ' = effective angle of friction of soil.

9 Maintenance

As the product is confined within the soil and it has suitable durability (see section 10), maintenance is not required.

10 Durability

10.1 Paralink Geocomposites may be used in fills normally encountered in civil engineering practice (see section 5.6).

10.2 Evidence from tests shows that the products have good resistance to chemical degradation, biological degradation, temperature and weathering (see sections 10.3 to 10.8).

Chemical degradation

10.3 Within a soil environment where pH ranges from 4.0 to 9.5 and temperatures are typical of those normally found in embankments in the United Kingdom, the strength of the geocomposites is not adversely affected by hydrolysis. Should pH values exceed 9.5, suitable safety factors can be found in Table 5.

Biological degradation

10.4 The geocomposites are highly resistant to microbial attack.

Effects of temperature

10.5 The long-term creep performance of the geocomposites is not adversely affected by the range of soil temperatures typical to the UK.

10.6 The long-term creep performance for a range of soil temperatures is shown in Table 3. Where the geocomposites may be exposed to temperatures greater than 30°C or lower than −20°C for significant periods, consideration should be given to temperature levels, range of temperature, period of exposure and stress levels at the location in question.

10.7 The long-term environmental effects factor for a range of soil temperatures is shown in Table 5. Sustained temperatures of greater than 30°C increase the rate of hydrolysis and further reduction factors may be required.

Resistance to weathering

10.8 The geocomposites have a high resistance to ultraviolet light. The product may be exposed to light for up to one month on site. Exposure of up to four months may be acceptable depending upon the season and location.

Installation

11 General

11.1 In general, the execution of the reinforced soil structures should be carried out in accordance with BS EN 14475 : 2006.

11.2 Care should be exercised to ensure geocomposites are laid with the longitudinal direction parallel to the direction of principal stress. Design drawings should indicate the orientation of the geocomposite.

11.3 Rolls should be placed on the formation in the position where the length of Paralink is required to start and with the roll as closely as possible at right angles to the line of the run. Accurate alignment at the start is essential to ensure a satisfactory positioning of the laid material.

12 Preparation

To ease the laying and proper performance of the run, the formation on which it is to be laid should be flat without ruts and sharp undulations.

13 Procedure

13.1 The roll should be unwound a small amount by pushing the roll in the direction of the run. The loose end of the Paralink now exposed should be secured by weighting or pinning it to the formation. The roll should be unwound carefully, avoiding slack or undulations wherever possible — laying must not continue until corrections are made. When the roll is completely unwound, the free end of the Paralink should be hand tensioned and secured by weighting or pinning.

13.2 The run of Paralink should be straight and all strip elements flat and untwisted. Undulations should not be evident.

13.3 Where Paralink is to be used in two layers at right angles to each other, the edge joints will normally be simple butt joints. The drawings should be consulted to verify this as certain circumstances may dictate otherwise.

13.4 Where a number of rolls are to be laid at one time, rolls should be arranged to be in a slightly staggered formation to avoid the lifting tubes interfering with one another.

13.5 Fill material in immediate contact with the Paralink should be placed and spread in the longitudinal direction only. If this results in some undulations of the Paralink, the secured end should be released and the undulations removed by pulling the free end.

13.6 Site vehicles should not be allowed to traffic over the laid, unprotected Paralink.

13.7 Paralink is a structural material and, where joints are necessary in its longitudinal direction, they should be full structural joints capable of carrying the full design tensile force. This will normally be shown as a full anchorage bond length on the drawings. The anchorage bond length depends on the depth of cover and type and characteristics of the fill in which Paralink is being used. Where pile caps are spanned, this length is unlikely to be less than the distance across three pile caps. Where the products are being used to span subsidence voids it will depend upon the size of the void anticipated by the designer.

Technical Investigations

14 Investigations

14.1 The manufacturing process of the geocomposite materials was examined, including the methods adopted for quality control, and details were obtained of the quality and composition of the materials used.

14.2 An examination was made of data relating to:

- evaluation of long- and short-term tensile properties
- chemical resistance including hydrolysis
- resistance to biological attack
- resistance to weathering
- effects of temperature
- site damage trials and resistance to mechanical damage
- soil/geocomposites interaction.

14.3 Calculations were made to establish the plane sliding area that is solid and the ratio of bearing surface to plane area.

14.4 The practicability of installation and ease of handling were assessed.

Additional Information

The management systems of Linear Composites Limited have been assessed and registered as meeting the requirements of BS EN ISO 9001 : 2008 by Lloyds Register Quality Assurance, Approval Certificate No LRQ 0902157.

Bibliography

BS 8006 : 1995 *Code of practice for strengthened/reinforced soils and other fills*

BS EN 13251 : 2001 *Geotextiles and geotextile-related products — Characteristics required for use in earthworks, foundations and retaining structures*

BS EN 13738 : 2004 *Geotextiles and geotextile-related products — Determination of pullout resistance in soil*

BS EN 14475 : 2006 *Execution of special geotechnical works — Reinforced fill*

BS EN ISO 9001 : 2008 *Quality management systems — Requirements*

BS EN ISO 9864 : 2005 *Geosynthetics — Test method for the determination of mass per unit area of geotextiles and geotextile-related products*

BS EN ISO 10319 : 2008 *Geotextiles — Wide-width tensile test*

BS EN ISO 10320 : 1999 *Geotextiles and geotextile-related products — Identification on site*

BS EN ISO 13431 : 1999 *Geotextiles and geotextile-related products — Determination of tensile creep and creep rupture behaviour*

Manual of Contract Documents for Highway Works, Volume 1 *Specification for Highway Works*, August 1998 (as amended)

Conditions of Certification

15 Conditions

15.1 This Certificate:

- relates only to the product/system that is named and described on the front page
- is granted only to the company, firm or person named on the front page — no other company, firm or person may hold or claim any entitlement to this Certificate
- is valid only within the UK
- has to be read, considered and used as a whole document — it may be misleading and will be incomplete to be selective
- is copyright of the BBA
- is subject to English law.

15.2 Publications and documents referred to in this Certificate are those that the BBA deems to be relevant at the date of issue or re-issue of this Certificate and include any: Act of Parliament; Statutory Instrument; Directive; Regulation; British, European or International Standard; Code of Practice; manufacturers' instructions; or any other publication or document similar or related to the aforementioned.

15.3 This Certificate will remain valid for an unlimited period provided that the product/system and the manufacture and/or fabrication including all related and relevant processes thereof:

- are maintained at or above the levels which have been assessed and found to be satisfactory by the BBA
- continue to be checked as and when deemed appropriate by the BBA under arrangements that it will determine
- are reviewed by the BBA as and when it considers appropriate.

15.4 In granting this Certificate, the BBA is not responsible for:

- the presence or absence of any patent, intellectual property or similar rights subsisting in the product/system or any other product/system
- the right of the Certificate holder to manufacture, supply, install, maintain or market the product/system
- individual installations of the product/system, including the nature, design, methods and workmanship of or related to the installation
- the actual works in which the product/system is installed, used and maintained, including the nature, design, methods and workmanship of such works.

15.5 Any information relating to the manufacture, supply, installation, use and maintenance of this product/system which is contained or referred to in this Certificate is the minimum required to be met when the product/system is manufactured, supplied, installed, used and maintained. It does not purport in any way to restate the requirements of the Health & Safety at Work etc Act 1974, or of any other statutory, common law or other duty which may exist at the date of this Certificate; nor is conformity with such information to be taken as satisfying the requirements of the 1974 Act or of any statutory, common law or other duty of care. In granting this Certificate, the BBA does not accept responsibility to any person or body for any loss or damage, including personal injury, arising as a direct or indirect result of the manufacture, supply, installation, use and maintenance of this product/system.

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Appendix F – Simulation Note on PLAXIS 2D Analyses of Case Study in Flood Plain Bridge No. 1

PLAXIS 2D Simulation Note on Back Analyses and Post Construction Creep Settlement of Embankment over Flood Plain Bridge 1 (Ballina Bypass NSW Australia)

1. Project Description

Flood Plain Bridge (FPB1) is a part of the newly constructed Ballina Bypass in which a bridge was built over one of the floodplains located to the west of Ballina township. This area was underlain by a 13-m thick very soft to soft marine clay deposited during the Holocene age. Surcharge and wick drain (SWD) ground improvement technique has been adopted for the northern and southern approaches to FPB1 to accelerate the primary consolidation under surcharge loading and reduce the post construction settlement in 40 years under design height of the embankment.

A snapshot showing the ground treatment plan and instrumentation locations is given in Figure F1 below.

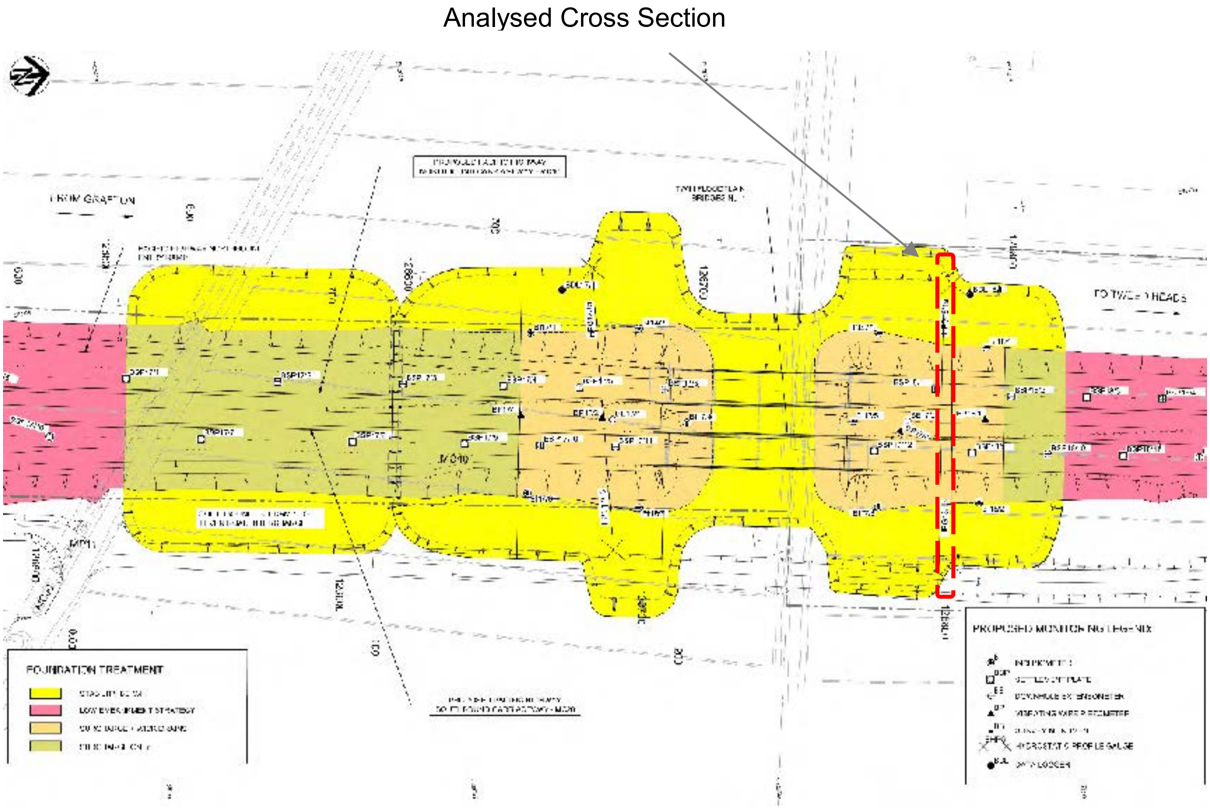


Figure F1 – Ground Treatment Plan and Instrumentation Locations for subject area in FPB1

This simulation note summarises the analyses carried out on the northern abutment of FPB1.

2. Monitoring Data

The settlement data across the abutment measured using Hydrostatic Profile Gauge HPB18-1 is shown in Figure F2 below, while the lateral movements measured in inclinometers BI17-3 and BI18-2 are given in Figure F3 below.

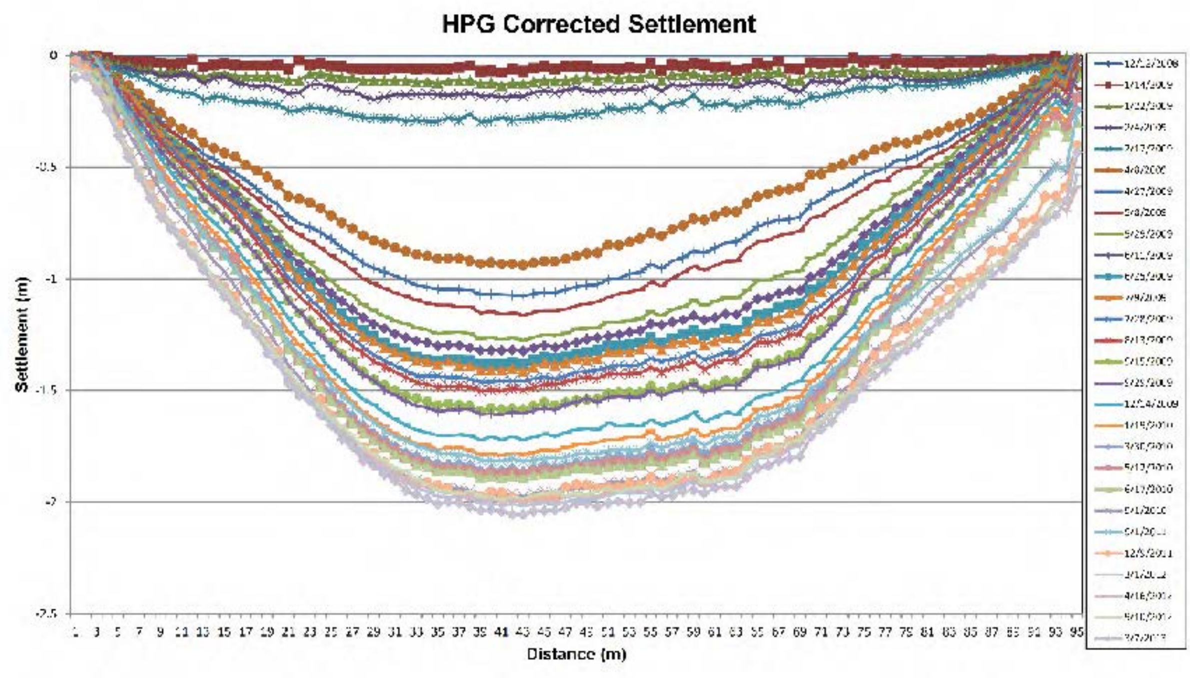


Figure F2 – Hydrostatic Profile Gauge (HPG) Monitoring Data of the subject area

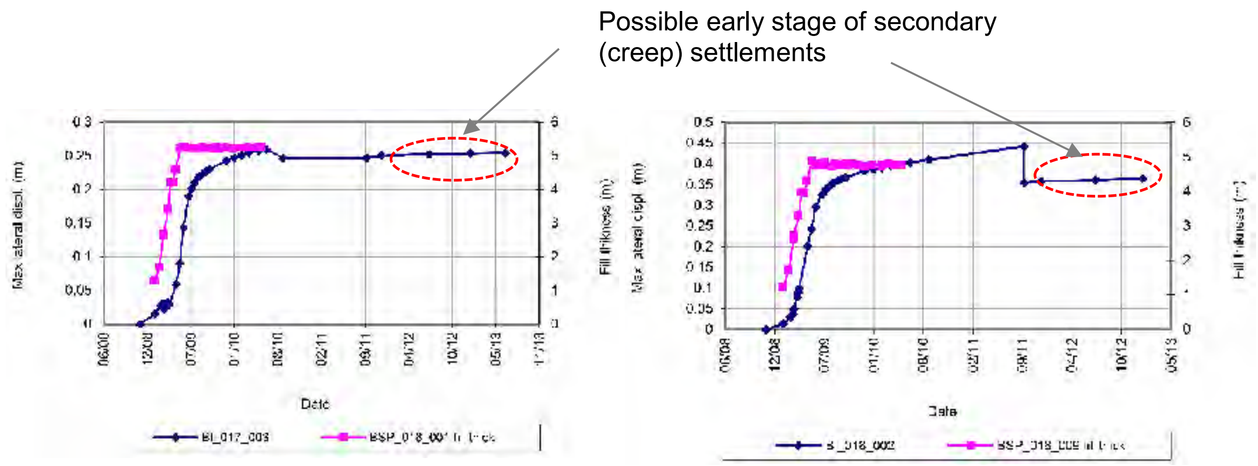


Figure F3 – Inclinometer Monitoring Data of BI17-3 (left) and BI18-2 (right)

3. Brief Methodology

The back analyses were carried out using Finite Element method coded in a commercially available program PLAXIS 2D. The Soft Soil Creep model was used.

For the purpose of back-analyses, the compressibility parameters (CR and CRR) were adjusted to match the settlement profiles shown in Figure F2. Similarly, the creep strain rate ($C_{\alpha\varepsilon}$) was adjusted to reasonably match the settlement reading for the last 1.5 years of monitoring period (see Figure F3) where creep settlement as likely occurred. Following steps were adopted in PLAXIS 2D:

Phase 1. Initial phase

Phase 2. Construction of Fill to the surcharge level including the stability berm

Phase 3. Waiting period for approximately 6 months

Phase 4. Stripping to the design level including the removal of stability berm

Phase 5. Waiting period (until the end of construction)

Phase 6. Waiting period (from the end of construction to the end of monitoring period)

4. Analyses Results and Brief Discussions

The contour of settlement using back-analysed parameters from the beginning of the surcharge construction up to the end of construction is shown in Figure F4 below. In general, the magnitude and settlement profile are reasonably consistent with the measured magnitude and profile shown in Figure F2. The opening of the road occurs sometimes in the middle of 2011, while the monitoring of HPG and inclinometers were continued up to March 2013. Figures F5 and F6 show the settlement and corresponding lateral deformation which were likely caused by creep over a periof of 1.5 years up to the middle of 2011. The maximum settlement (Figure F5) under the centre of embankment was assessed to be about 49 mm while the maximum lateral deformation (Figure F6) under the embankment toe was assessed to be 9 mm.

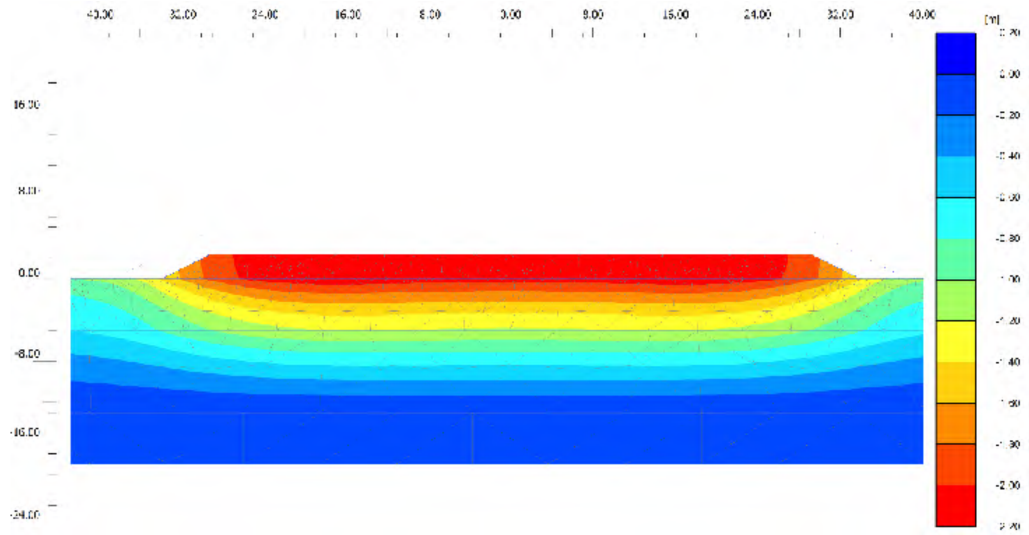


Figure F4 – Plot of Settlements occurring throughout the construction (up to mid 2011)

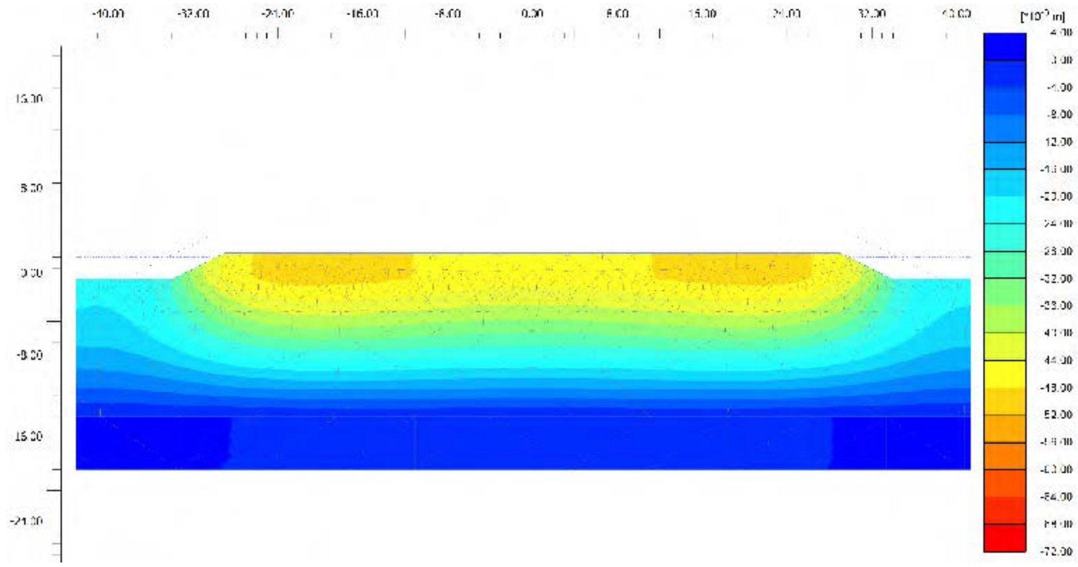


Figure F5 – Plot of Secondary Settlements occurring over the period of 1.5 year up to the final measurement in March 2013

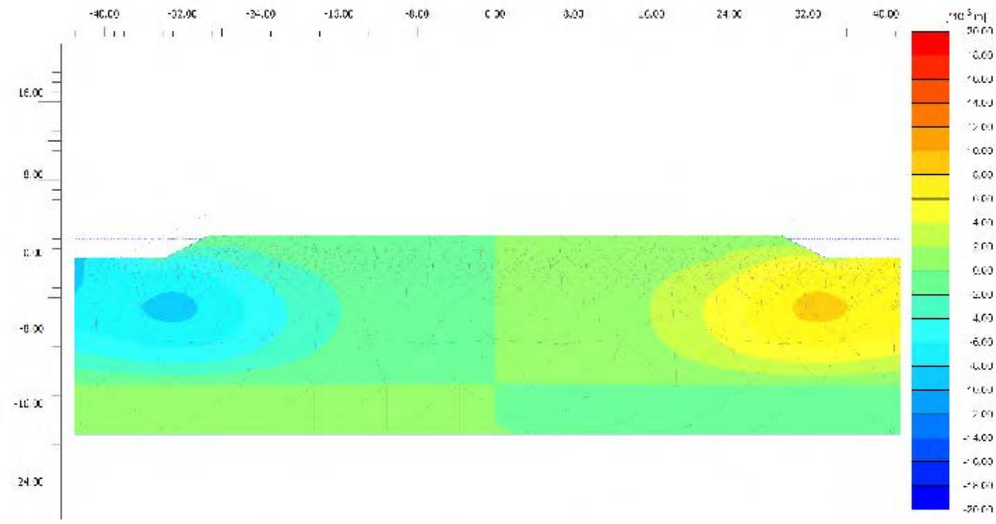


Figure F6 – Plot of Lateral Deformation occurring over the period of 1.5 year up to the final measurement in March 2013

Appendix G – Outputs of Discrete Element Modelling and Dynamic Deflection (Finite Element) Analyses

G1. Discrete Element Modelling of Ballast

Brief Description

The Discrete Element Method (DEM) was carried to assess deformation of ballast assembly. In DEM, each particle/wall is assigned appropriate micromechanical parameters. It is noted that micromechanical parameters are intrinsic parameters which are unique to each particle/wall and govern how the particle/wall interact with other particles/walls.

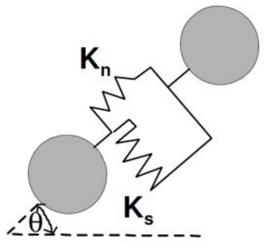


Figure G1 – Illustration of particles interaction in DEM

For the modelling of ballast two simulations were carried out as shown below:

Case A Ballast Thickness of 500 mm to model maximum standard thickness as per ARTC Heavy Haul Guideline

Case B Ballast Thickness of 675 mm to model anticipated maximum ballast thickness

A typical cross section shown below (Figure G1) is considered.

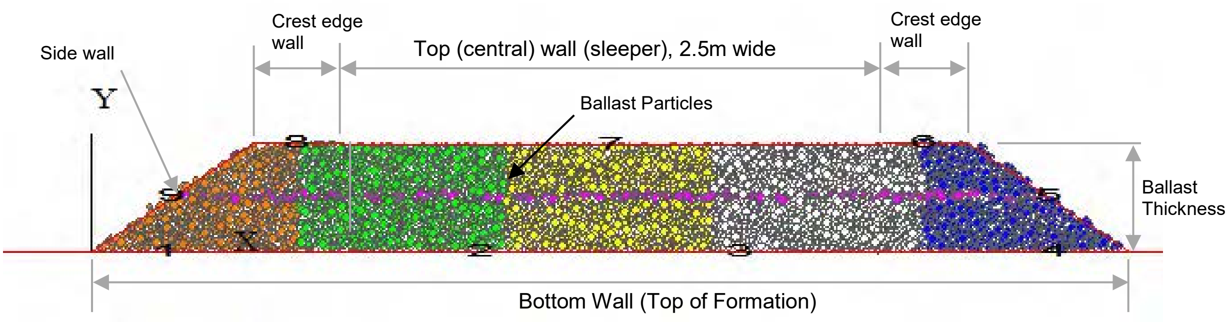


Figure G1 – Model used in DEM Simulation

As ballast particles comprise granular materials with negligible cohesion, a linear contact model has been used with three micromechanical parameters namely friction coefficient (μ), normal stiffness (k_n) and shear stiffness (k_s). These parameters were selected to be consistent with a range of parameters typically adopted for the modelling of ballast particles (Lu and McDowell 2007, Lu and McDowell 2010, O'Sullivan 2011)

The adopted micromechanical parameters and flexibility descriptions of walls are summarised in Table G1 below:

Table G1 – Adopted Micromechanical Parameters

DEM Element	Stiffness (N/m)		Frictional Coefficient (μ)	Remarks
	Normal (k_n)	Shear (k_s)		
Base Walls	5×10^7	5×10^7	0.33	Flexible Wall[(a)]
Side Walls and Crest Edge Walls	1×10^5	1×10^5	0.01	Minimal confinement[(b)]
Top (central) platen	1×10^{10}	1×10^{10}	0.5	Rigid Wall (rail sleeper)
Ballast Particles	1×10^8	1×10^8	0.5	Angular particles with high particle strength

Note:

a) The parameters of base walls were adopted to take into account an overall stiffness of foundation below the ballast assembly
b) The minimal confinement simulates only small forces due to particle interlocking effect. However these walls are not expected to provide any additional confinement

The simulation stages are summarised below:

Stage 1. Randomly generate walls and ballast particles as per the Particle Size Distributions (as per ARTC ETA04-01 – Ballast Specification) and angularity

Stage 2. Equilibrium Stepping required to bring the particles into near equilibrium (i.e. mean unbalanced forces substantially smaller than the mean contact forces) during which no movement can occur

Stage 3. Light tamping of ballast simulated by moving the top platen upwards and downwards

Stage 4. Equilibrium Stepping (as per Stage 2)

Stage 5. Apply track loading in a number of quick successions

The train loading assessed based on AS5100.2 has been considered in the modelling. This loading was calculated using the method described in Section 5.3.4 of the main body of this report. The load was applied on the 2.5 m wide top wall (i.e. sleeper) and hence the vertical pressure distribution was not considered in the load calculation. The train loading was applied in a number of quick successions where each succession comprises an application of a single pass of a train loading idealised as a static load followed by no train loading (i.e four times longer that the duration of static load). It is noted that the computational time-step adopted in analysis is proportional to the actual time.

Throughout the simulation, the movements of walls and selected boundary particles (i.e. particles near walls) were monitored. Additionally, thirty four (34) measurement circles are positioned as shown in Figure G2 to monitor the average particles responses in various locations.

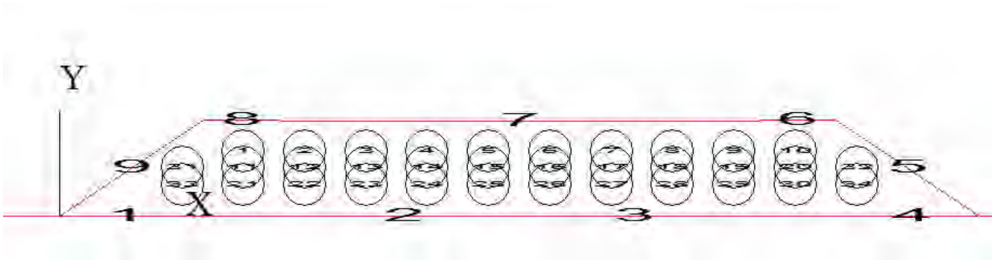


Figure G2 – Locations of Measurement Circles

Analyses Results

The results of Stages 1 to 4 for Case A are shown below in Figure G3 below.

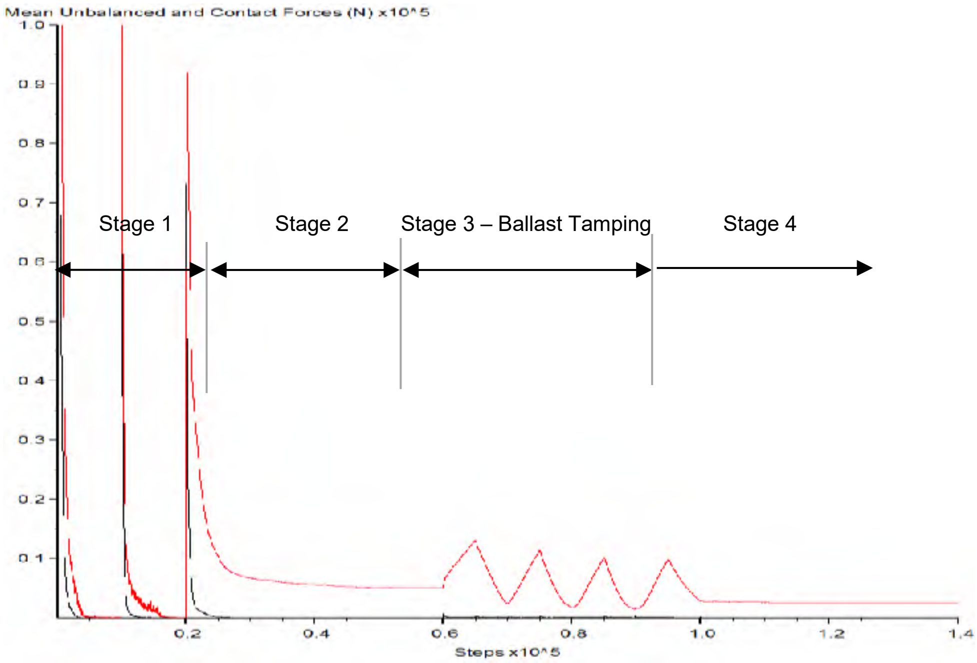
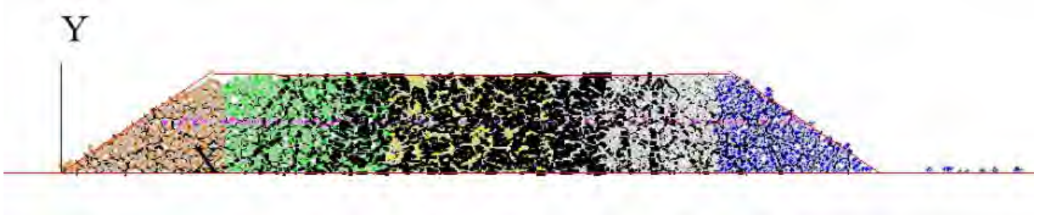


Figure G3 – Illustration of Results from Preliminary Stages of Case A

The simulation process of train loading via sleeper is shown in Figure G4 below. The black lines and their thicknesses in the figure indicate contact stresses and corresponding relative magnitudes of contact stresses, respectively.

(a) Case A (500-mm thick ballast)



(b) Case B (650-mm thick ballast)

Figure G4(a) and (b) – Simulation of Train Loading and Relative Contact Stresses of Case A

The plots showing settlements of top wall (i.e. sleeper) due to the train loading in Cases A, B and C are shown in Figure G5 below.

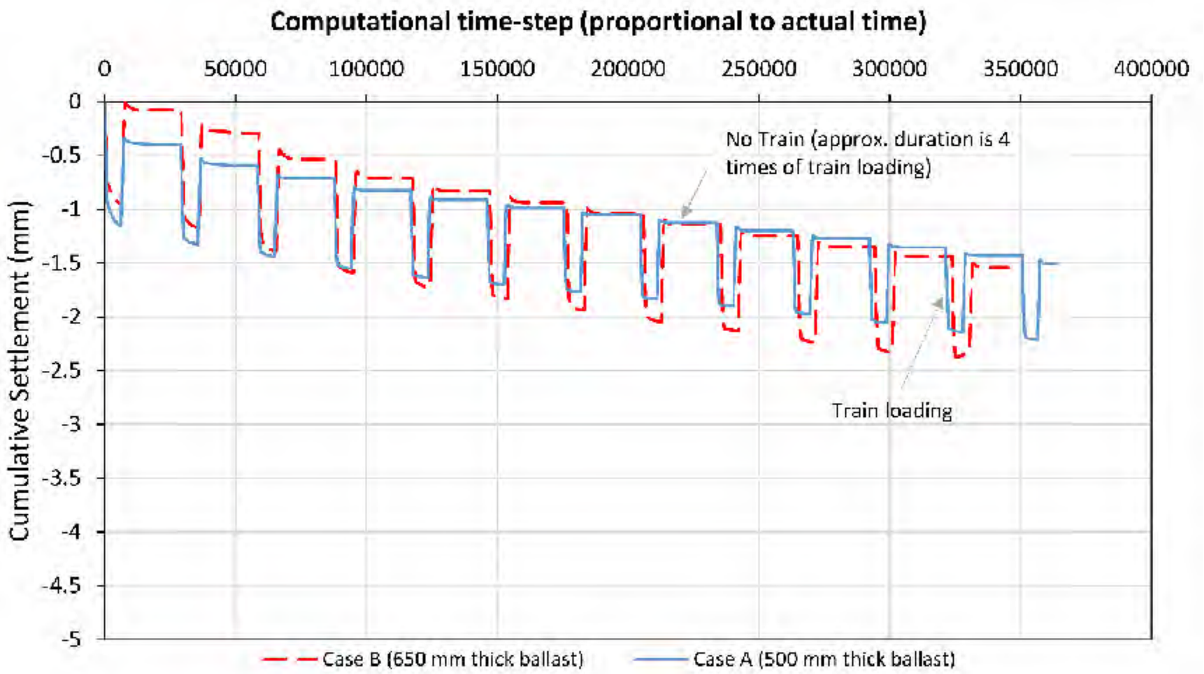
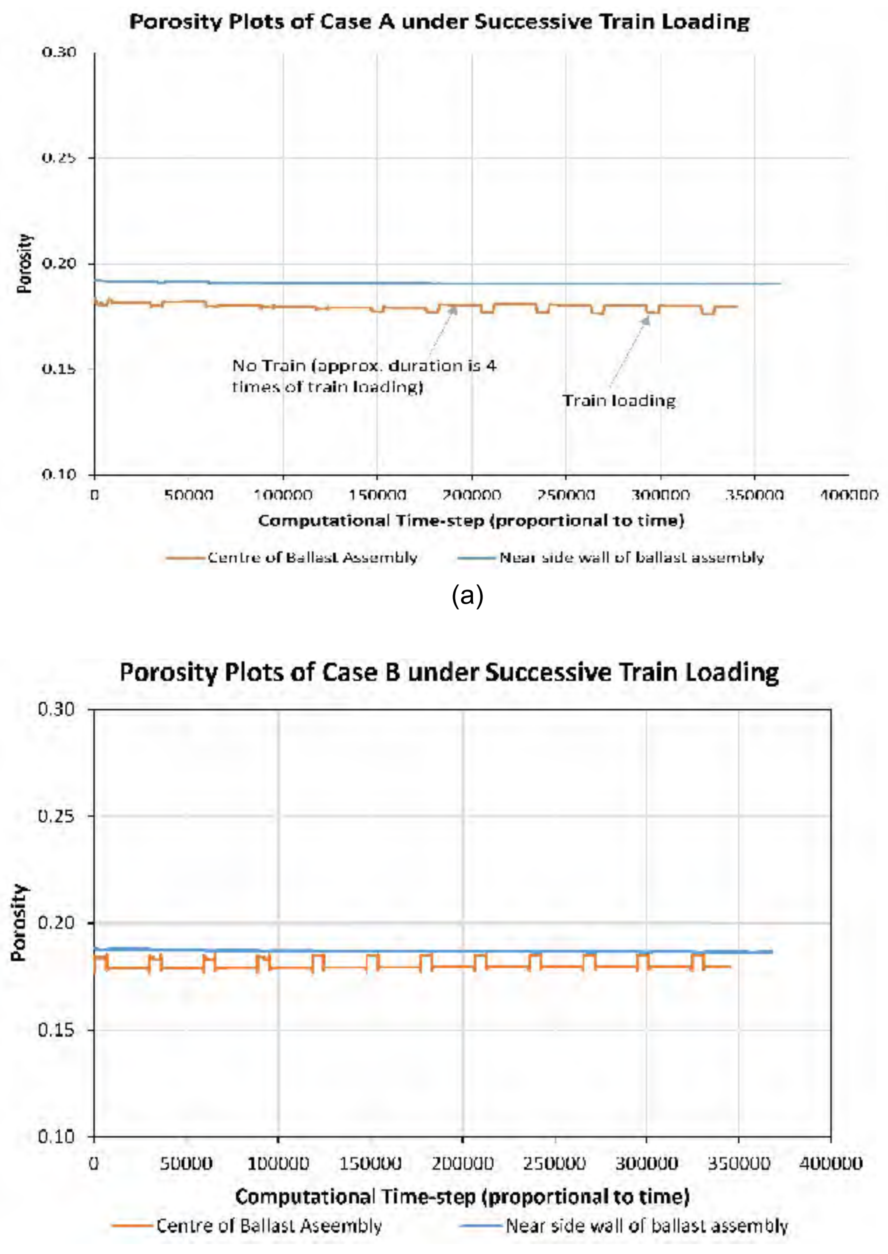


Figure G5 – Assessed Sleeper Settlements under successive train loading for Cases A and B

Plots of changes in porosity near side walls 5 and 9 (see Fig. G1) and below the centre of ballast assembly (see Fig. G1) as measured using the measurement circles are shown in Figures G6(a) and (b) below for Cases A and B, respectively.

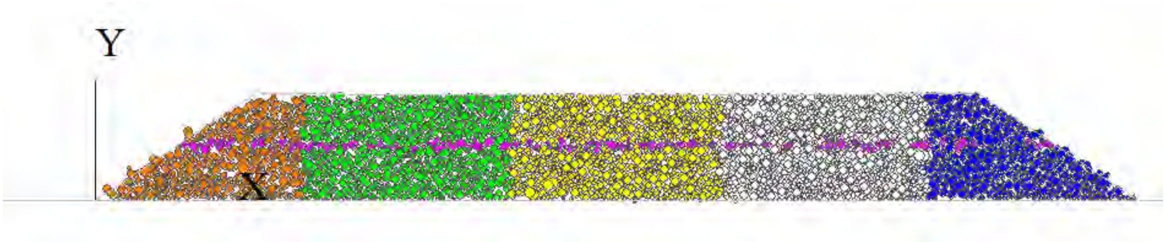


(a)

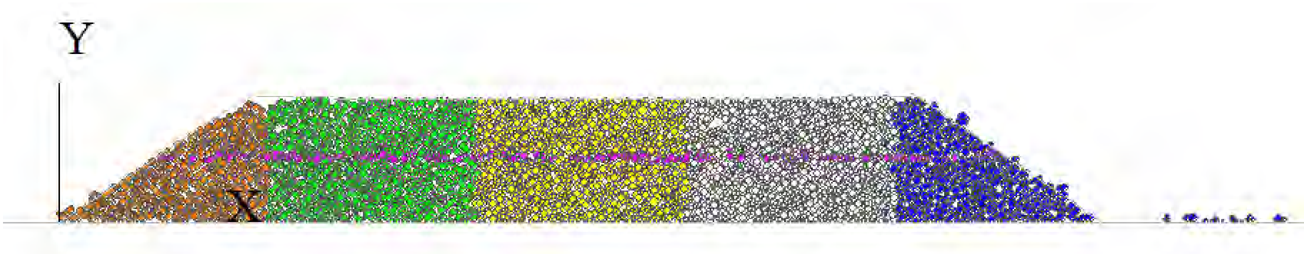
(b)

Figure G6(a) and (b) – Plots showing porosity evolution throughout the loading

The final profiles of ballast assemblies from Analyses Cases A and B are shown in Figures G7(a) and (b) for Cases A and B, respectively, for overall visual comparison.



(a) Case A (500 mm thick ballast



(b) Case B (675 mm thick ballast)

Figure G7(a) and (b) – Final Profile of Ballast Assemblies in Cases A and B

<u>Discussion and Conclusions</u>

Based on the aforementioned analyses, the settlement plot (Figure G5) indicates the typical settlement pattern under successive train loading. In general, the settlement is trending downward and stabilises with the increase of train passes. The maximum settlement for thicker ballast (675 mm thick) in Case B is expected to be in order of 2.5 mm. The difference in settlement magnitude of sleepers between Cases A and B is anticipated to be about 0.5 mm.

The porosity plots (Figures G6(a) and (b)) indicate the following:

- In each case the porosity in the centre of ballast assembly is lower than the porosity near the side wall due to higher intensity of loading beneath the sleeper
- In each case, the porosity value is generally stable and does not indicate any significant change in void ratio. The regular changes in porosity in the centre of ballast assembly throughout loading stage can be associated to the loading pattern.

The overall final profile shown in Figures G7 indicate the following:

- No adverse change in ballast profiles in comparison against initial profile
- No difference in resulting ballast profiles between 500 mm thick ballast and 675 mm thick ballast assemblies.

G2. Finite Element Modelling (Dynamic Deflection) of Ballast

Brief Description

The Finite Element (FE) Modelling of Ballast is carried out using a commercially available software PLAXIS 2D. The FE generally simulates the ballast behaviour in continuum condition while the abovementioned DEM simulates the ballast behaviour in discrete condition (i.e. particle to particle interaction). Therefore, DEM is mostly reliable to model the ballast responses while a direct DEM modelling of subgrade materials can be computationally expensive and time-consuming. Therefore FE modelling is done to incorporate the subgrade responses.

The specific objective of FE modelling with various ballast thickness is to obtain the difference in settlement exclusively due to the difference in ballast thickness and contamination effect under train loading calculated in accordance to AS5100.2. Two cases were considered in the Dynamic Deflection modelling:

Case C — Ballast thickness of 500 mm over the capping layer followed by structural fill and subgrade materials

Case D — Ballast thickness of 650 mm over the capping layer followed by structural fill and subgrade materials

Case E — Contaminated ballast thickness of 500 mm over the capping layer followed by structural fill and subgrade materials

Case F — Contaminated ballast thickness of 650 mm over the capping layer followed by structural fill and subgrade materials

In Cases D and F, the 650 mm thick ballast is considered to model the ballast thickness of greater than 650 mm thick anticipated in some localised area due to intervention strategy in 40 years (see Section 5.4.2).

Brief Methodology and Results

The loading stages considered in the FE simulation are:

Stage 1. Initial Stage

Stage 2. Build Embankment Fill to the Design Level following the DC ground treatment

Stage 3. Reset Displacement to Zero. Apply Train Loading

The simulation results for Cases C to F are shown in Figures G8 to G11 below.

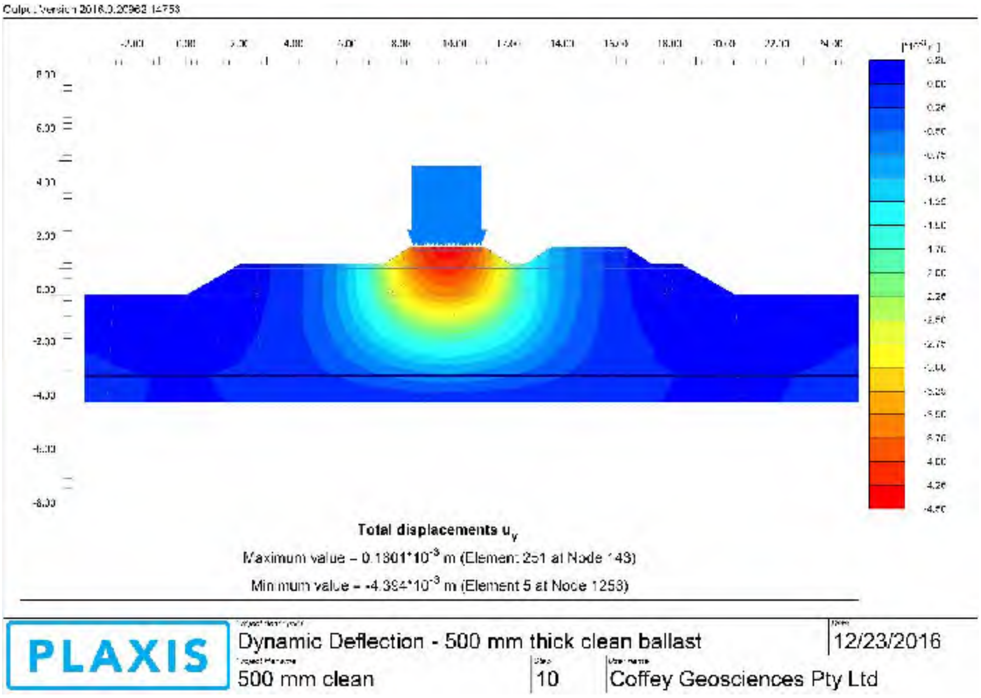


Figure G8 – Vertical settlement under train loading with 500 mm thick clean ballast (Case C)

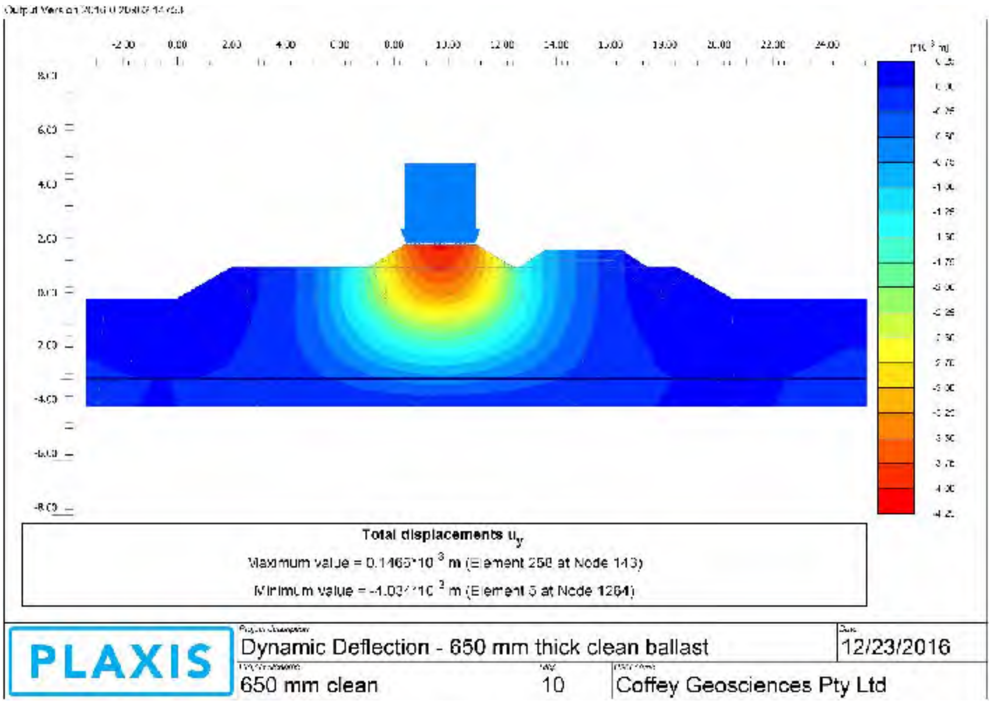


Figure G9 – Vertical settlement under train loading with 650 mm thick clean ballast (Case D)

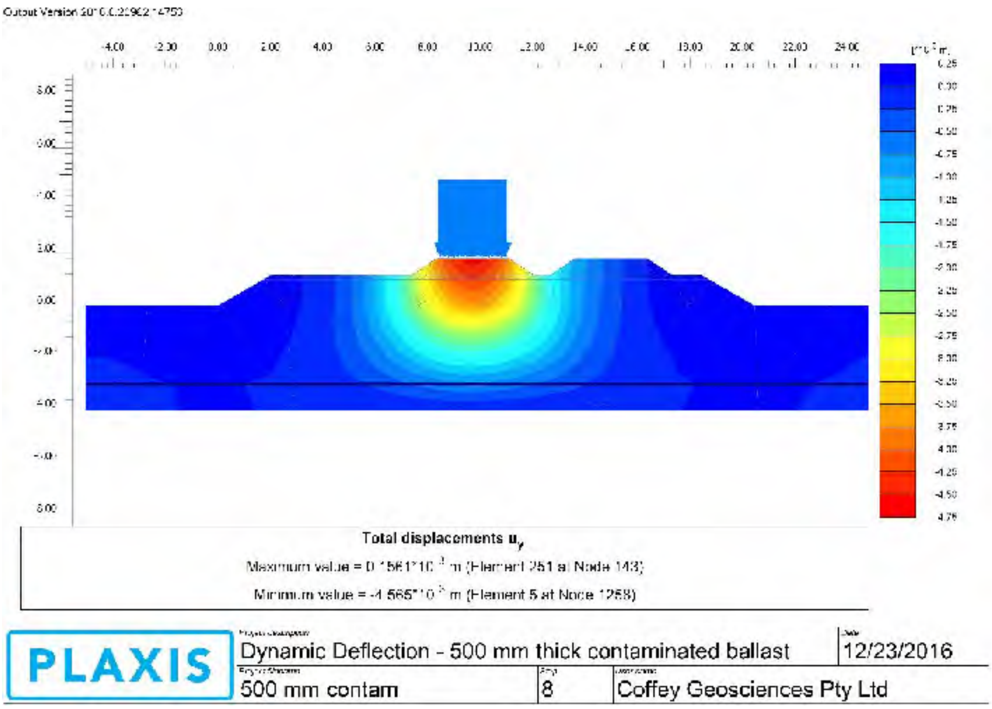


Figure G10 – Vertical settlement under train loading with 500 mm thick contaminated ballast (Case E)

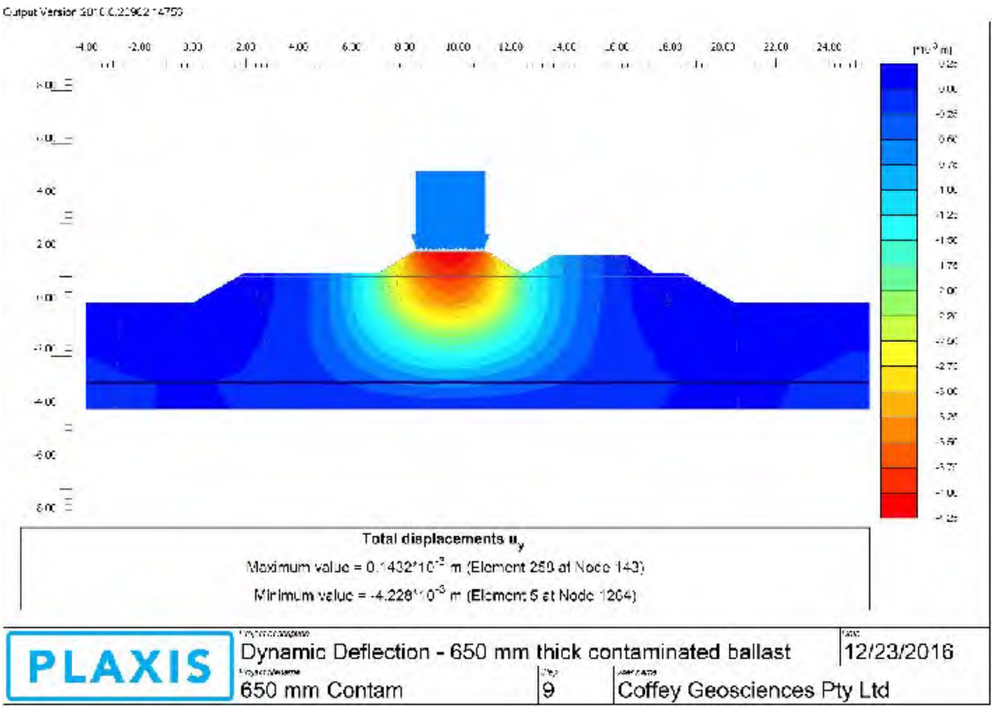


Figure G11 – Vertical settlement under train loading with 650 mm thick contaminated ballast (Case F)

Appendix H – Summary of Coffey Past Experience (Hexham Project and 1984 Dynamic Compaction Trial) and Other Case Studies

1. 1984 Dynamic Compaction Trial (Coffey's experience)

In 1984, Coffey undertook a DC trial on a section of the then proposed East Hill Railway Line as part of the geotechnical investigation for Gutteridge Haskins & Davey Pty Ltd. The trial was carried out within the GWS landfill area. Back in 1984, this part of the landfill was considered relatively younger in age. The landfill thickness of 10 m was assumed during the trial and the landfill materials comprised wood, paper and metal which are comparable to that observed during investigation conducted for MIRC. We note that the 1984 trial location was situated about 100 – 120 m to the south of the proposed MIRC alignment within the GWS landfill.

The trial was carried out in 2 areas:

- Area 1 on triangular grids with square prints spacing of 2.5 m centre to centre (c/c); and
- Area 2 on square grids with prints spacing of 2.5 m c/c

A 8.6 tonne pounder with base dimensions of 1 m x 1 m was dropped from a height of 10 m. Ten drops were completed on each print during the primary and secondary passes. Crater filling and levelling were carried out upon the completion of each pass. Additionally, two drops of lighter and larger diameter mass were performed over the whole test area in an attempt to provide compaction between the craters generated during the primary and secondary passes. A number of 3 m high test embankments with circular shape in plan were constructed on the DC treated and unimproved areas, and surface settlements were measured over a period of 50 days.

The results of settlement monitoring have been back analysed by assessing the end of primary consolidation and calculating the slope of secondary settlement curve to obtain the creep strain rate for secondary consolidation ($C_{\alpha\varepsilon}$). A value of $C_{\alpha\varepsilon}^{*}$ of 0.0065 was obtained for DC treated area in the 1984 trial.

To take into account uncertainty and variability of landfill, we have adopted a creep strain rate ($C_{\alpha\varepsilon}^{*}$) of 0.01 for DC treated area consisting of older landfill in our Final Design (FD) of the Moorebank Intermodal Rail Link (MIRL) project, which is higher than the back analysed value of 0.0065 for the DC treated area comprising then newer landfill (i.e. 1984 trial) with similar material composition (i.e. GWS landfill). Generally, young landfill has higher $C_{\alpha\varepsilon}$ than old landfill. Hence, adopted parameter in MIRC is considered reasonable.

2. Hexham Relief Roads Project (Coffey's experience)

The Hexham Relief Road (HRR) project, commissioned by the Australian Rail Track Corporation (ARTC), involves the construction of five tracks over a length of approximately 2.6 km. The design was being undertaken by the Upper Hunter Valley Alliance (UHVA) comprising Leighton Contractors, Coffey Geotechnics, Parsons and Brinckerhoff (PB), KMH, ASCAA and ARTC. The proposed tracks were underlain by soft to firm estuarine clay with a thickness of more than 25 m in the southern end and 12 m in the northern end of the project.

For the design of rail formation over soft compressible soil, the Alliance did not adopt a ground-intrusive and expensive ground treatments. Instead, only conventional treatment in the form of surcharge and preloading has been considered for the generic area of the project. The waiting period of 6 months was adopted. The dynamic deflection analysis has been carried out to design the non-standard formation over the soft to firm clay. The designed formations comprised rockfill materials and generally varied from 0.8 m to 1.8 m in thicknesses.

Although initially there was no criteria set for the post construction differential settlement, the criteria of 1:400 for the plain tracks have been agreed upon and adopted for the purpose of settlement assessment. The total post construction settlement was assessed to be in the order of 500 mm in 40 years, and the differential settlements have been checked against the criteria by considering the typical intervention periods. At the completion of design, as requested Coffey only presented the values of total settlement with time up to 40 years, which would be used by ARTC to decide and modify the intervention periods as required.

The design has been accepted by ARTC and the following statement is quoted from the design report:

> "*It is expected that the rail tracks will settle with time due to primary consolidation and long term creep settlement of soft ground. The magnitude of predicted settlement is much greater than the allowable track deformations stated in Hunter 200+ guidelines. We understand that ARTC agrees that the HRR formation is a non-standard design and appropriate maintenance such as re-levelling and re-tamping of ballast will be carried out at regular intervals by ARTC to meet their operational criteria including differential settlement during the design life of the rail tracks. Predicted post construction settlement provided in this report can be used for the development of tamping intervals. We understand that ARTC will revise these intervals through the future settlement monitoring.*"

The subsequent back analyses of settlement data have also been carried out by comparing the predicted values of consolidation settlement against the settlement reading measured using settlement plates. It was considered that predicted settlement values were reasonably comparable with the measured settlements. The HRR project was successfully completed and the tracks have been in operation from 2014.

3. Case Studies (by Others)

We have undertaken discussions with a number of ground improvement contractors to study the application of DC specifically for the improvement of landfill. An extensive study has also been carried out to assess the worldwide application of DC for the landfill improvement. Selected local and international case studies are summarised in Table H1 below. These information are available in public domain in the form of project sheets.

Table H1 – Selected Case Studies on the Application of DC for the Landfill Improvement

No.	Project Name	Location	Brief Comment on DC Application	Source
1	Nan Tien Temple(1)	Wollongong NSW Australia	Improvement of up to 10 m depth of uncontrolled landfill	Contractor's project sheet
2	Perth Stadium(1)	Perth WA Australia	Improvement of a 8 m deep landfill overlying estuarine sediments	Contractor's project sheet
3	Gateway Arterial Roads Project(1)	Brisbane QLD Australia	Static cone penetrometer testing indicated that the compaction was accompanied by a doubling of the static cone end resistance	Literature (Hausmann et al 1993) (2)
4	Maldegem Landfill	Maldegem, Belgium	The improvement depth is distinctly shown by Surface Analysis of Surface Waves (SASW) test up to the depth of 4 to 5 m in a 4.5 to 8 m thick landfill. The estimated applied energy was 15 to 65 tonne.m/m^2	Literature (Van Impe and Bouazza, 1996)

No.	Project Name	Location	Brief Comment on DC Application	Source
5	Highway embankment in New Jersey and North Albany Demolition Landfill in New York	New Jersey and New York, USA	The authors indicate that the Dynamic Compaction is the most effective ground treatment method for long-term settlement reduction of landfill	Literature (Sharma and De, 2012)
6	Database of a number of sites[(3)]	Numerous (worldwide)	Improvement depths vary from 4 m to 9 m for applied energy range of 150 to 300 tonne.m/m^2. The ranges for pounder weights and drop heights are 10 to 20 tonnes and 10 to 30 m, respectively	Literature (Zekkos et al, 2013)

Note:

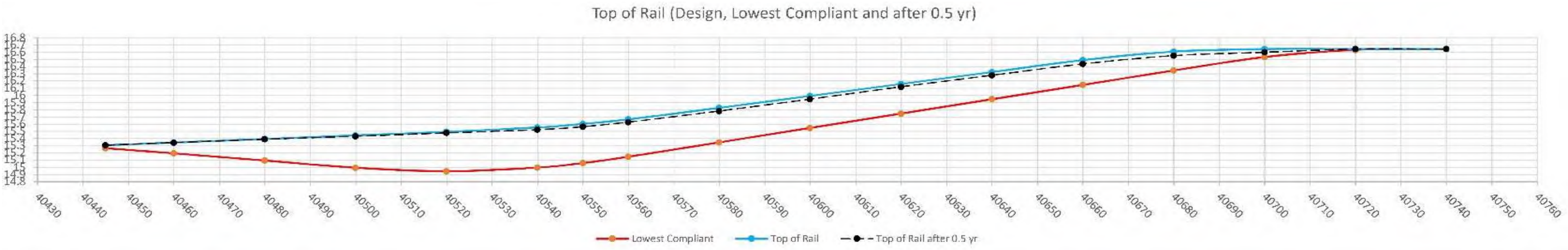
(1) Local experience

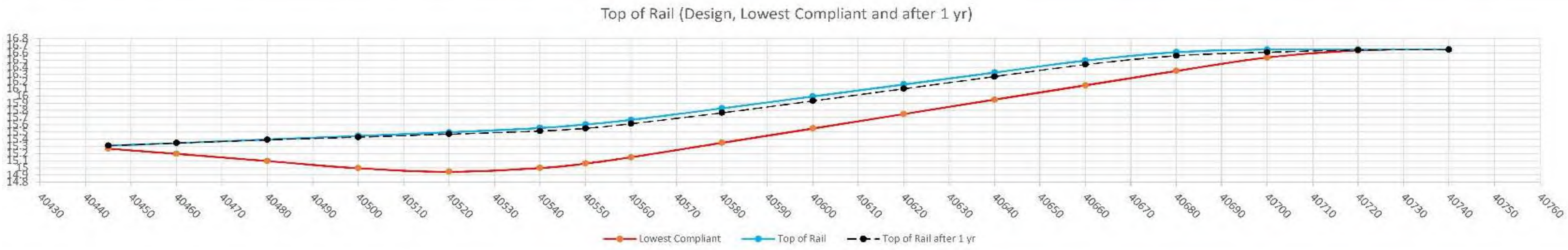
(2) Hausmann et al (1993) also outlines that some increase in density has been achieved in local landfills (Merrylands NSW, Thornleigh NSW and Lucas Heights NSW) due to the application of waste compactors with mass ranging from 10 tonnes to 40 tonnes.

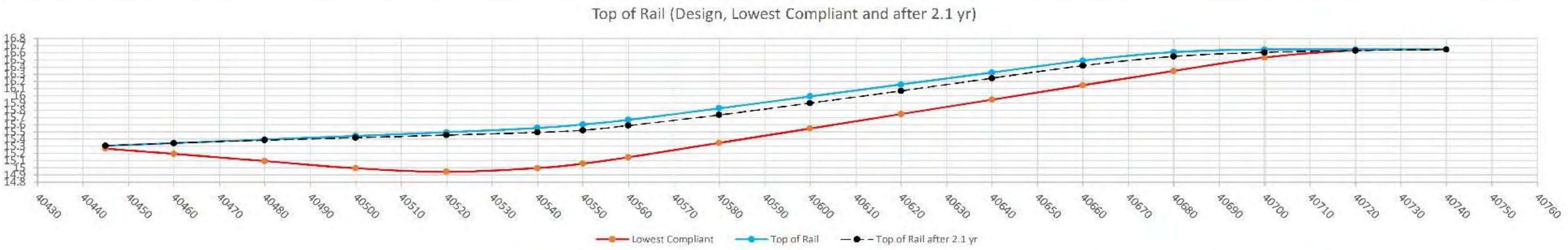
(3) This literature provides an extensive database on the application of DC for the landfill improvement as well as the effect of improvement (i.e. depth of influence and reduction in settlement).

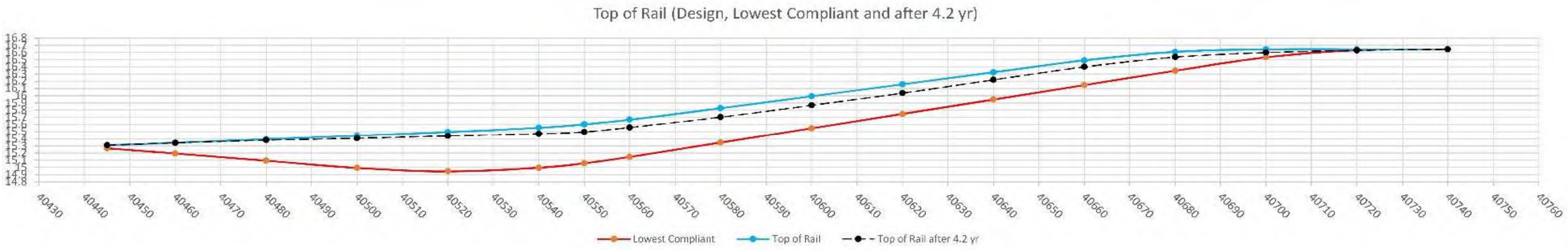
Appendix I – Summary of Assessed Top of Ballast and Cumulative Ballast Thicknesses at Interventions

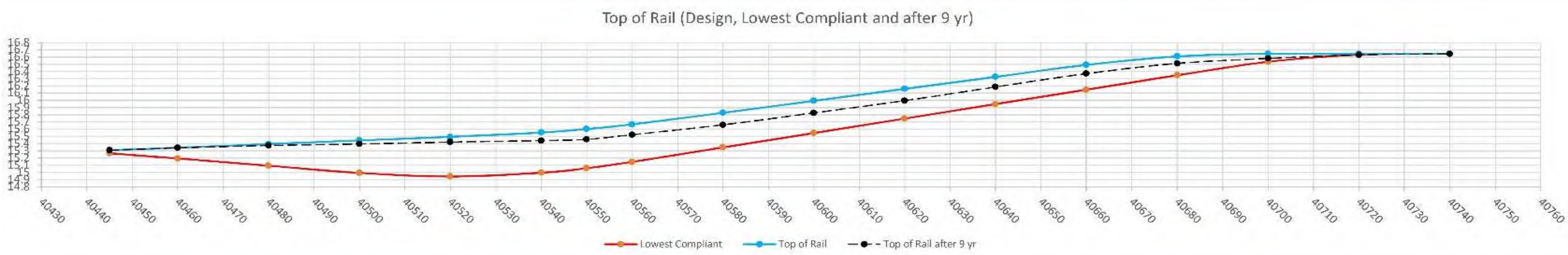
CH M2BS	Rail Level m AHD		Max allowed settement (mm)	Settlement after 40 yrs (mm)		Allowed Settlement (mm)		Initial Ballast thickness (mm)		Max. Ballast thickness at 40 yrs (mm)	
	Design Rail Level	Rail level at 40 yrs		M2BN	M2BS	M2BN	M2BS	M2BN	M2BS	M2BN	M2BS
40460	15.344	15.310	34	21	220	-13	186	356	251	343	437
40480	15.394	15.328	66	77	319	11	253	320	251	331	504
40500	15.444	15.331	113	133	377	20	264	287	251	307	515
40520	15.494	15.345	149	157	410	8	261	250	251	258	512
40540	15.554	15.341	213	251	497	38	284	258	287	296	571
40560	15.667	15.412	255	282	492	27	237	253	331	280	568
40580	15.827	15.547	280	292	503	12	223	259	253	271	476
40600	15.993	15.715	278	291	491	13	213	251	252	264	465
40620	16.159	15.887	272	289	489	17	217	250	252	267	469
40640	16.325	16.078	247	296	504	49	257	254	255	303	512
40660	16.491	16.266	225	314	521	89	296	250	250	339	546
40680	16.609	16.441	168	315	523	147	355	291	253	438	608
40700	16.645	16.633	12	50	50	38	38	341	251	379	289
40720	16.645	16.633	12	50	50	38	38	355	253	393	291

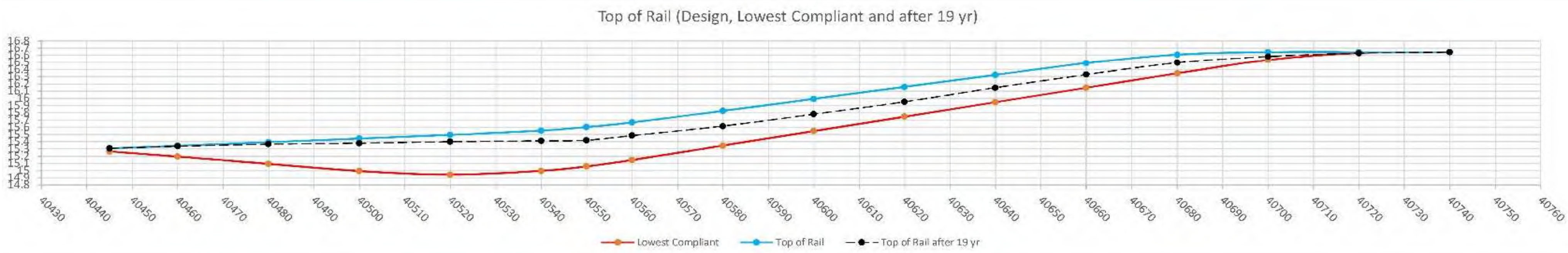












drawn	[redacted]	coffey	client:	**CPB CONTRACTORS PTY. LTD.**
approved			project:	**MOOREBANK INTERMODAL RAIL LINK**
date	**10/17/2016**			
scale	**NTS**		title:	Required Levels of Top of Rails (of MB2S) at Interventions
original size	**A3**		project no: **GEOTLCOV24072AF**	figure no: **I1**

Peiris, Ashok

From: [redacted]
Sent:
To:
Cc:
Subject: RE: Rail settlements to assess Twist
Attachments: 161219 - DCZ Settlement Alignments.pdf; 161219 - DCZ Settlement Alignments.dwg

[redacted]

Please see attached PDF & .dwg showing the current design (green) and the "lowest acceptable alignment" shown in black.
These alignments have been checked & verified.

Also, I have completed the twist analysis based on the parameters shown in TMC 203 and SPC 207 and found no exceedance of the standards in regards to short or long twist.
As mentioned in the email trail below, any level differences shown in the "rail-twist" excel tables provided are very gradual and no limits are exceeded based on the 60km/h speed in this section of track.

[redacted]

DISCLAIMER

[redacted]

[redacted]

Hope this email finds you doing well and fine.

I tried to contact you few times. You must be busy or away.
Just to touch base on progress of our assessment on twist and vertical elevation (may be Top???) as requested by the reviewers.

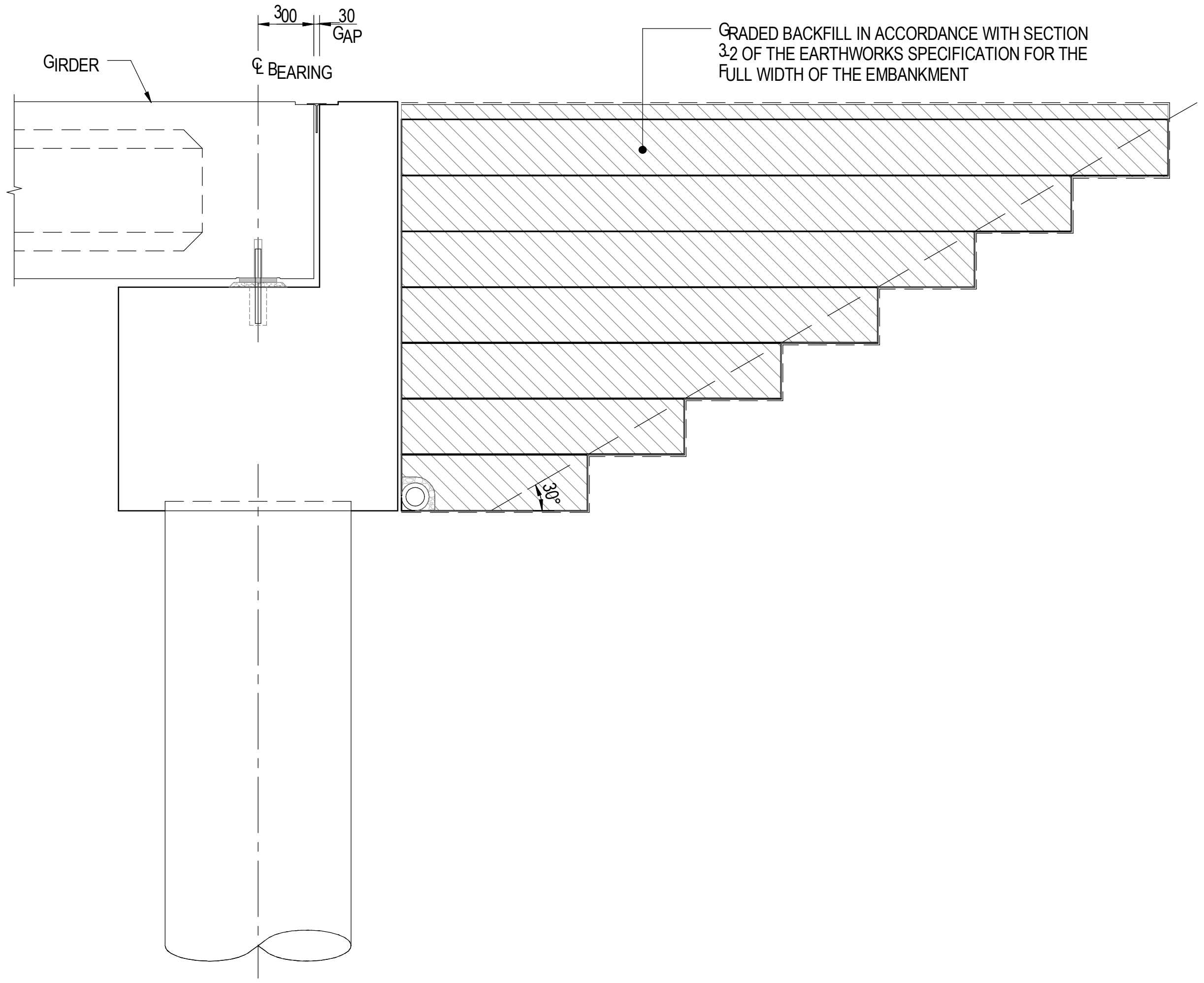
Thanks in advance.

[redacted]

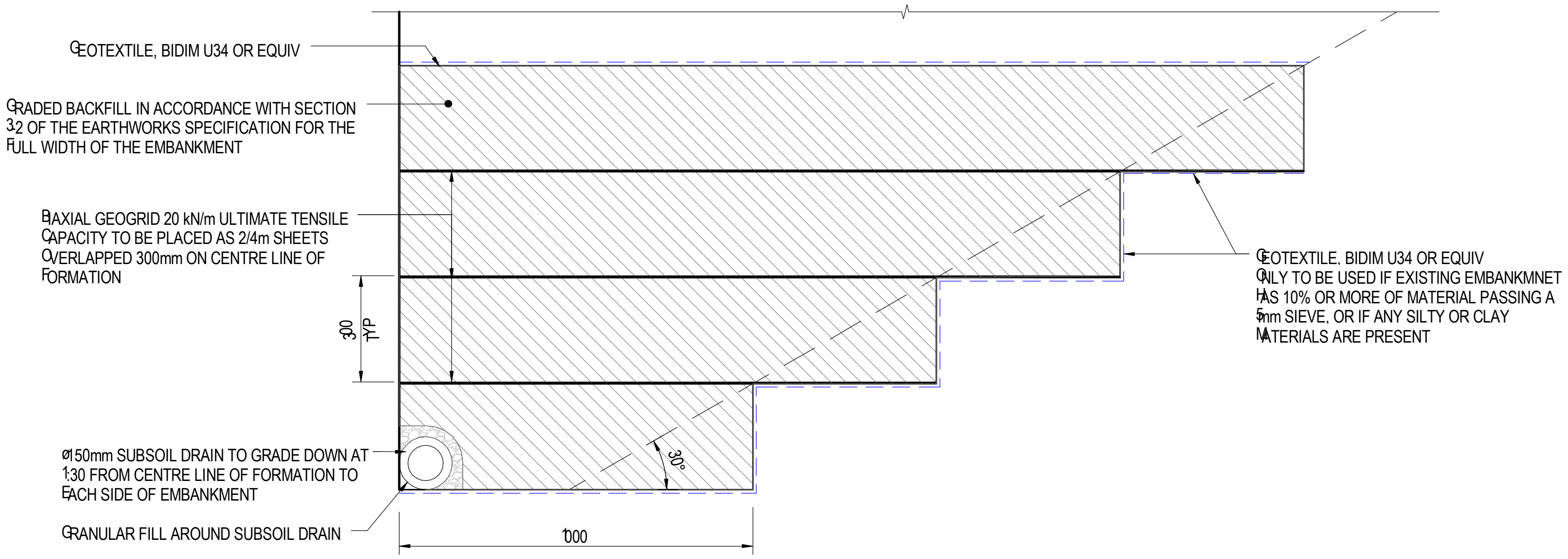
[redacted]

Appendix J – Cross Sections of Embankments showing the Change of Cross Fall due to Long Term Settlement in 40 years

Appendix K – Western Bridge Approach Details



TYPICAL ABUTMENT APPROACH DETAIL
1 : 20



TYPICAL BACKFILL DETAIL
1 : 10

GENERAL NOTES

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE PROJECT EARTHWORKS SPECIFICATION (DOCUMENT NUMBER N01031-PWD-SPE-EWK-0001)

0 100 200 300 400 500mm
SCALE 1:10

0 0.2 0.4 0.6 0.8 1m
SCALE 1:20

[redacted]

aurecon
www.aurecongroup.com

CLIENT

CPB CONTRACTORS

SIMTA SYDNEY INTERMODAL TERMINAL ALLIANCE

REV	DATE	REVISION DETAILS	[redacted]
[redacted]			

[redacted]

Appendix L – Witness and Hold Points

<u>Hold/Witness Point</u>

Item	Element of work	Description	H/W Point release by
1	Installation of settlement plates, survey monuments, HPGs and inclinometers	Relevant hold points below has been included in the project “Instrumentation and Monitoring Specification (ref. N01031-GRW-GEO-SPE-0001-02)” Hold Point 1: • Documentation for the proposed instrumentation; • Experienced Geotechnical Instrumentation Contractor under the supervision and direction of an experienced Geotechnical Engineer (minimum 5 years relevant experience) to meet the requirements of the Specification and the instrument manufacturer’s recommendations. **Process held:** Commencement of the instrumentation works on site. Hold Point 2: Review the monitoring data before placing each layer (say 300 mm lift and frequency to be reviewed based on monitoring data), prior to the release of the Hold Point. **Process held:** Placement of fill and geogrid. Hold Point 3: CPB to reinstate damaged instruments and provide details to COF for review. **Process held:** Continuation of embankment construction ceased due to damage to instruments. Hold Point 4: Review the monitoring data before placing embankment fill, prior to the release of the Hold Point. **Process held:** Placement of embankment fill (deferred during filling or any other operation).	CPB/COF
2	Geogrid and facing element	Witness point: Witnessing certification of compliance of geogrid compared to design requirements. Geogrid layers to be connected to facing elements, as per manufacturer’s requirement and method specification. Process held: Placement of geogrid and facing elements.	CPB

Item	Element of work	Description	H/W Point release by
3	Carry out CBR test on the embankment fill (i.e. subgrade)	Hold Point: Tests must be carried out by qualified professional. Review test results before placing formation. **Process Held:** Construction of formation	CPB/COF
4	Monitoring settlement and lateral movement. Back-analysis and prediction of post construction settlement.	Hold Point: Review of monitoring data before releasing embankment for construction of rail formation/ballast/sleeper **Process held:** Release for construction of rail formation, ballast, sleepers and rails	CPB Witness point : SIMTA

Abbreviations:

CPB: CPB Contractors

COF: Coffey Services Pty. Ltd

SIMTA: Sydney Intermodal Terminal Alliance

Appendix M – Rail Embankment infiltration performance assessment – Landfill within GWS

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Electronic Transmission

To	[redacted]	From	[redacted]
Email address	[redacted]	Date	13 April 2017
Company	CPB Contractors	Reference	GEOTLCOV24072AF-BN Rev 1
cc	[Cc]	Pages	1 of 5
Subject	Infiltration Performance Assessment - Rail Embankment within GWS Landfill area		

[redacted],

1. Introduction

The existing landfill cover layer will be disturbed during Dynamic Compaction (DC) works. Prior to the DC, a minimum 500 mm thick working platform will be added on the existing cover layer from Ch 40,560 to Ch 40,740 (MB2S). The anticipated settlement during DC work is about 1 m. Hence, the minimum overall thickness (approximately at Ch 40,680) of soil layer over the landfill after the DC work and rail embankment construction will be over 3 m. This soil layer will include rail capping layer, rail embankment, working platform and existing cover layer.

In accordance with the EPA guidelines, the sealing layer over landfill should be at least 600 mm thick, with an in-situ hydraulic conductivity of not more than 1×10^{-9} m/sec. The sealing layer is recommended to achieve “infiltration from the base of the final cap to be less than 5% of the annual rainfall".

An infiltration analyses has been carried out using commercially available software SEEP/W, at a critical rail embankment section (i.e. at Ch 40,680) to assess the infiltration performance within the rail embankment footprint and hence, compare with the EPA performance requirement as detailed above.

2. Model

Rail embankment at Ch 40,680 consists of following soil layers immediately above landfill and as shown in Figure 1:

- 150mm thick rail capping layer;
- 500mm thick formation layer (Structural fill);
- 500mm thick general embankment layer;
- 500mm thick working platform layer; and
- 1.5m thick existing cover layer.

The existing cover layer and working platform will be subjected to DC tamping and hence the level of compaction will be increased.

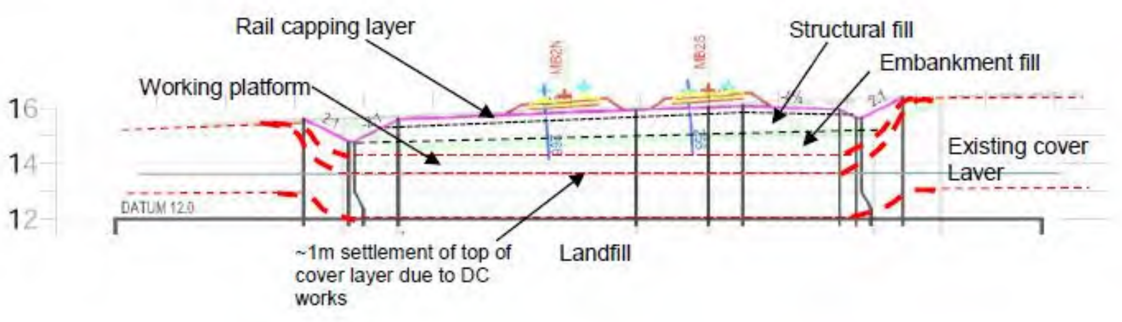

Figure 1: Rail embankment and subsurface layers at Ch 40,680

Our simplified SEEP/W model representing these conditions is presented in Figure 2 below:

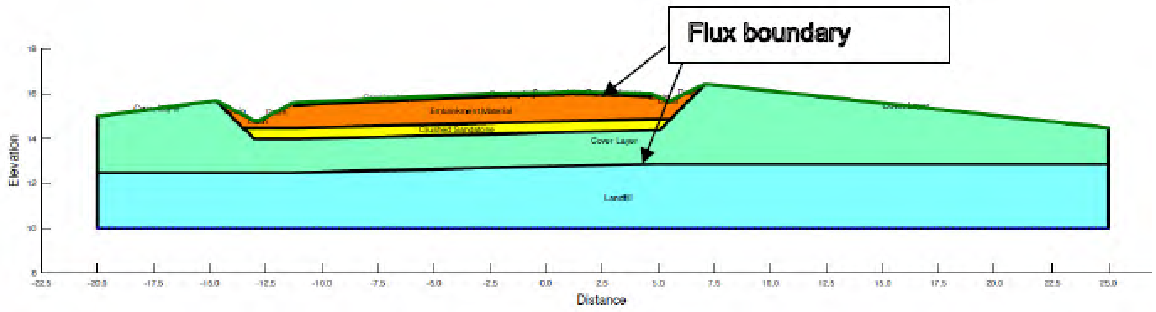

Figure 2: SEEP/W model at Ch 40,680

3. Infiltration parameters and rain events

3.1. Infiltration parameters

In-situ infiltration tests and laboratory permeability tests have been conducted on the existing cover layer material. As reported in memo GEOTLCOV24072AF-BP (refer Attachment 1), the assessed in-situ hydraulic conductivity of existing cover layer is in the order of 10^{-7} m/sec. Laboratory permeability test carried out on soil samples from existing cover layer provided saturated hydraulic conductivity in the order of 10^{-9} m/sec. As the existing cover layer is subjected to DC tamping including an ironing pass to address shallow soil disturbance, the anticipated in-situ hydraulic conductivity of the existing cover layer after DC works would be lower than 10^{-7} m/sec

However, in this analysis, in-situ hydraulic conductivity of 10^{-7} m/sec was adopted for existing cover layer.

Infiltration parameters adopted for the other soil layers are presented in Table 1

Table 1: Infiltration parameters

Layer	Layer thickness, (m)	Saturated permeability (m/s)	Volumetric water content(1)	Typical unsaturated soil characteristic curve(2)
Rail capping layer	0.15	10^{-9}		Clay
Structural fill/Embankment fill	1	10^{-5}	0.4	Gravel
Working platform	0.5	10^{-4}		Gravel
Existing Cover layer	1.5(3)	10^{-7}		Gravel

Note:

1) Typical value for well compacted gravel has been adopted.

2) Unsaturated soil characteristic curves have been assumed based on anticipated behaviour of the layers. Conservatively assumed gravely behaviour for layers other than the capping layer.

3) Although expected thickness of existing landfill cover layer is about 2m, a lower thickness is assumed.

3.2. Rainfall data

Annual rainfall data relevant to GWS landfill area has been assessed from two weather stations namely "Bankstown airport" and "Ingleburn". Considering the higher average annual rain fall and number of rainy days, data from Bankstown airport weather station has been used for this analysis. Table 2 below summarise the rainfall data for year 2016 and average over years 1968 to 2016:

Table 2: Summary of annual rainfall data

	Year 2016	Average
Annual rainfall (mm)	973	895
Number of rain days/year	107	116

In addition to above annual rainfall events, two isolated rain events were considered in the analysis:

Table 3: Isolated rainfall events

Event	Description
Rain Event 1: Long duration rainfall event	Cumulative rainfall of 152mm over 17 days. With rain occurs in 4 consecutive days
Rain Event 2: High intensity rainfall event	Cumulative rainfall of 293mm over 3 days.

4. Analysis methodology and results

Steady state flow analysis was carried out based on following assumptions:

- Considering that the rail formation has been constructed with appropriate cross falls and longitudinal drainage is provided at the toe of the embankment, no water ponding is anticipated. During periods of rainfall saturated conditions at the surface of the rail capping layer are assumed for the full day for each day of rainfall. Unsaturated conditions with no water ingress are assumed to occur on days with no rainfall; and

- Evapotranspiration is not modelled. However, adopting only rainy days to assess the average annual flow is considered reasonable.

Results of steady state seepage analysis carried out using SEEP/W are summarised in Table 4 below:

Table 4: Results of steady state seepage flow analysis

Steady state flow	**Through capping layer**	**Through existing cover layer below rail embankment foot print**
m^3/day/m	0.009 over width of about 16m[(1)]	0.017 over width of about 45m[(1)]
mm/day (rain day only)	0.56	0.38
Annual rainfall event		
mm/year during anticipated rainy days	60	41
% as annual rain fall	6.2%	4.2%
Isolated rainfall events	~1.5%[(2)]	<1%[(2)]

[1] Refer Attachment 2: SEEP/W output plot for flow through the formation
[2] % as cumulative rainfall during the isolated rain event

5. Conclusion

Based on above assessment the assessed infiltration through existing cover layer in to landfill is about 4.2% of the average annual rainfall. Considering the conservative infiltration parameters adopted and evapotranspiration is not modelled exclusively, assessed infiltration of 4.2% of the average rainfall is considered conservative.

As the infiltration percentage in to the landfill is less than 5% of the average annual rainfall, no additional sealing layer is required within the embankment footprint.

Should you have any queries please do not hesitate to contact the undersigned.

For and on behalf of Coffey

[redacted]

Attachments:

Attachment 1: GEOTLCOV24072AF-BP – Factual test results on infiltration rate and hydraulic conductivity of existing cover layer

Attachment 2: SEEP/W output plot for flow through the formation

Attachment 1: GEOTLCOV24072AF-BP – Factual test results on infiltration rate and hydraulic conductivity of existing cover layer

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Electronic Transmission

To	[redacted]	From	[redacted]
Email address	[redacted]	Date	10 April 2017
Company	CPB Contractors Pty Ltd	Reference	GEOTLCOV24072AF-BP
cc		Pages	3 + 3 Attachments
Subject	Factual Test Results on Infiltration Rate and Hydraulic Conductivity for existing cover layer of GWS landfill		

1. Introduction

As requested by CPB Contractors (CPB), Coffey has carried out a fieldwork on 23 March 2017 within the Glenfield Waste Service (GWS) facility as part of the Moorebank Intermodal Rail Link (MIRL) project. The fieldwork was carried out to undertake a number of in-situ tests and collect soil samples for the laboratory tests for the measurement of infiltration rate and hydraulic conductivity of the existing cover layer of the landfill. The works were commissioned by CPB Contractors Pty Ltd (CPB) in order to characterise the cover layers and refine the assessment of infiltration performance of these layers within the landfill area treated by Dynamic Compaction (DC) from Ch 40,560 to Ch 40,740 (MB2S).

This correspondence summarises the factual results of the in-situ and laboratory tests. The interpretation provided was undertaken to process the raw data for the assessment of parameters in accordance to the relevant standards and published literature.

2. Fieldwork and Laboratory Testing

The in-situ tests undertaken during the aforementioned fieldwork comprise the following:

- Two Double Ring Infiltration (DRI) Tests in accordance to the ASTMD3385-03 (Standard Test Method for Infiltration Rate of Soils in Field using Double- Ring Infiltrometer) to measure the incremental infiltration rate.
- Four Inversed Auger Hole (IAH) Tests or “Porchet Method” to measure saturated hydraulic conductivity (K).
- Two Field Density Testing (FDT) using the nuclear moisture-density gauge in accordance to AS1289.5.8.1 – 2007 to measure the field dry density and field moisture content values.

Locations of the abovementioned tests are shown in Figure 1 in the attachment. The FDT tests were carried out next to the locations of DRI tests. Two bulk samples were collected from the same locations as those of the FDT tests and transported to our NATA-accredited laboratory. Those samples were tested for the following:

- Two compaction tests using standard compaction to measure the Standard Maximum Dry Density (SMDD) and Standard Optimum Moisture Content (SOMC) in accordance to AS1289.5.1.1 – 2003, AS1289.2.1.1 – 2005 and AS1289.5.4.1 – 2007.

- Two Falling Head Permeability (FHP) tests to measure hydraulic conductivity of samples compacted to SMDD in accordance to AS1289.6.7.2 – 2001.

The test locations and materials observed at these test locations are summarised in Table 1 below.

Table 1 – Test locations and descriptions of observed materials

Test Location	Easting (m)	Northing (m)	Corresponding Chainage (MB2S)[a]	Material Description [b]
LC1	307065	6239996	40,585	Gravelly SAND with some clay
HA1	307067	6239993	40,588	Gravelly SAND with some clay
HA2	307071	6239978	40,610	A mix of gravel, sand and clay
HA3	307077	6239944	40,650	Gravelly SAND with some clay
HA4	307083	6239917	40,672	Gravelly CLAY with some sand
LC2	307088	6239913	40,675	Gravelly CLAY with some sand

Note:

a. The corresponding chainage is approximate only based on projection of coordinates
b. Based on observations of materials near the surface

The results of DRI tests at locations LC1 and LC2 are presented as Figures 2 to 3 in Appendix A. The results of IAH tests at locations HA1 to HA4 are presented as Figures 4 to 7 in Appendix B. The results of FDT, FHP and other laboratory tests are presented in Appendix C.

3. Conclusions and Limitations

The factual results of in-situ and laboratory tests are summarised in Table 2 below.

Table 2 – Summary of in-situ and laboratory testing results

Test ID/Test Locations	Infiltration Rate (cm/hr)		Saturated Hydraulic Conductivity (m/s)	Field/Laboratory Dry Density (t/m^3)
	Peak	End of Test		
In-situ Testing				
DRI – LC1[b]	0.26	0.024	Approx. 2 x 10^{-7} to 3 x 10^{-7} (Note a)	2.09
DRI – LC2[b]	0.21	0.07		1.80
IAH – HA1[b]	Not measured		3.8 x 10^{-7}	2.09
IAH – HA2	Not measured		3.7 x 10^{-7}	Not measured
IAH – HA3	Not measured		2.8 x 10^{-7}	Not measured
IAH – HA4[b]	Not measured		2.6 x 10^{-7}	1.80

Test ID/Test Locations	Infiltration Rate (cm/hr)		Saturated Hydraulic Conductivity (m/s)	Field/Laboratory Dry Density (t/m^3)
	Peak	End of Test		
Laboratory Testing				
FHP – LC1	Not measured		6.6×10^{-9}	2.09[c]
FHP – LC2	Not measured		4.5×10^{-10}	1.96[c]

Note:

a. Adopted hydraulic conductivity values provide typical depths of saturation profile. Hence, adopted hydraulic conductivity values are considered reasonable in comparison to the values of nearby IAH tests.
b. Locations LC1 and LC2 are situated within a close proximity to the locations HA1 and HA4, respectively
c. Denotes laboratory measured dry density

The following limitations should be noted:

- The permeability of the soil is also influenced by other properties of the soil including but not limited to the in-situ void ratio and dry density of the soil layers; and
- Subsurface soil conditions may vary over a distance; and
- Subsurface soil conditions may vary over the depth including any localised presence of soil layers with varying permeability.

The attached document entitled "Important Information about Your Coffey Report" presents additional information on the uses and limitations of this report. Should you have any queries, please do not hesitate to contact the undersigned or [redacted].

For and on behalf of Coffey

[redacted]

Attachments

Appendix A – Results of Double Ring Infiltration Tests
Appendix B – Results of Inversed Auger Hole Testing
Appendix C – Results of Laboratory Testing

Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Important information about your **Coffey** Report

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way. Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

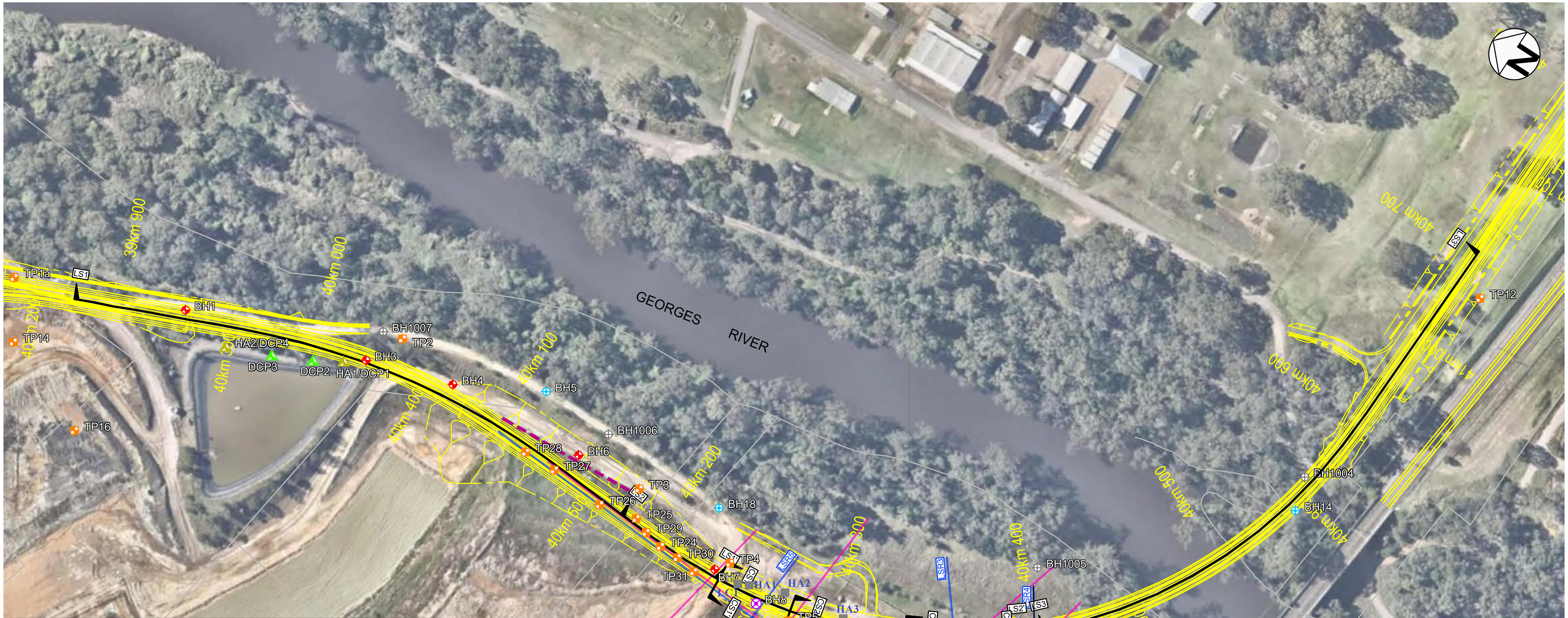
Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction,
speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.



LEGEND

- Locations of in-situ Tests for Infiltration Rate and Hydraulic Conductivity
- BOREHOLE LOCATION
- BOREHOLE WITH GROUNDWATER WELL LOCATION
- GEOTECHNICAL TEST PITS LOCATION
- ENVIRONMENTAL TEST PITS LOCATION
- BOREHOLE WITH GAS WELL LOCATION
- HAND AUGER WITH DYNAMIC CONE PENETRATION LOCATION
- DYNAMIC CONE PENETRATION LOCATION
- BOREHOLES DONE BY GOLDERS ASSOCIATES (2015)
- LSR - GEOPHYSICS - SEISMIC REFRACTION
- ERI - GEOPHYSICS - ELECTRICAL RESISTIVITY
- EXPOSED EDGE OF BLACK MEMBRANE
- LS/CS - GEOTECHNICAL LONG/CROSS SECTION

PLOT DATE: 27/06/2016 10:55:48 AM DWG FILE: F:\GEOTECHNICAL\S1 PROJECTS\GEOTLCOV240\GEOTLCOV24072AF EARLY SERVICES MOOREBANK RAIL LINK\CAD\GEOTLCOV24072AF-AM.DWG

revision

no.	description	drawn	approved	date
A	ORIGINAL ISSUE	[redacted]		
B				
C	NEW ALIGNMENT			

20 0 20 40 60 80 100

Scale (metres) 1:2000

drawn	[redacted]
approved	
date	27 / 06 / 16
scale	AS SHOWN
original size	A3

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A TETRA TECH COMPANY

client:	CPB CONTRACTORS
project:	MOOREBANK INTERMODAL RAIL CONNECTION MOOREBANK, NSW
title:	GEOTECHNICAL INVESTIGATION LOCATION PLAN SHEET 2 OF 4
project no:	GEOTLCOV24072AF-AM
figure no:	FIGURE 1
rev:	C

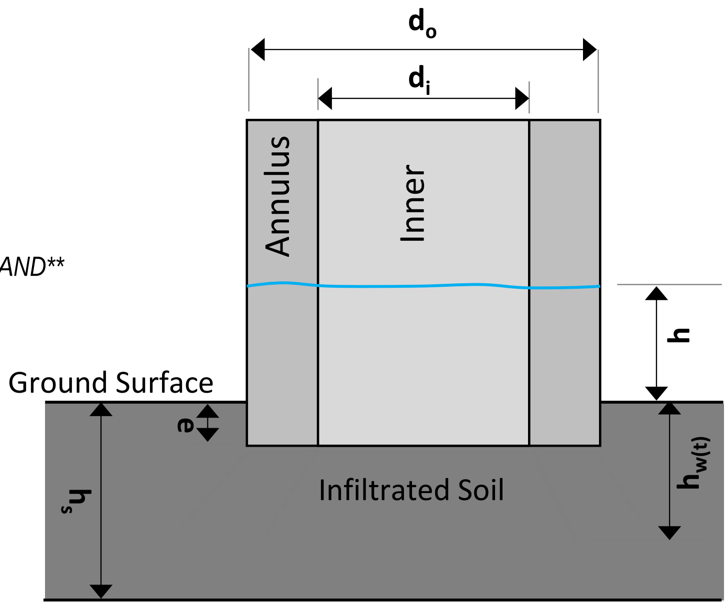
Appendix A – Results of Double Ring Infiltration Testing

DOUBLE RING INFILTRATION TEST (ASTM D3385-03)

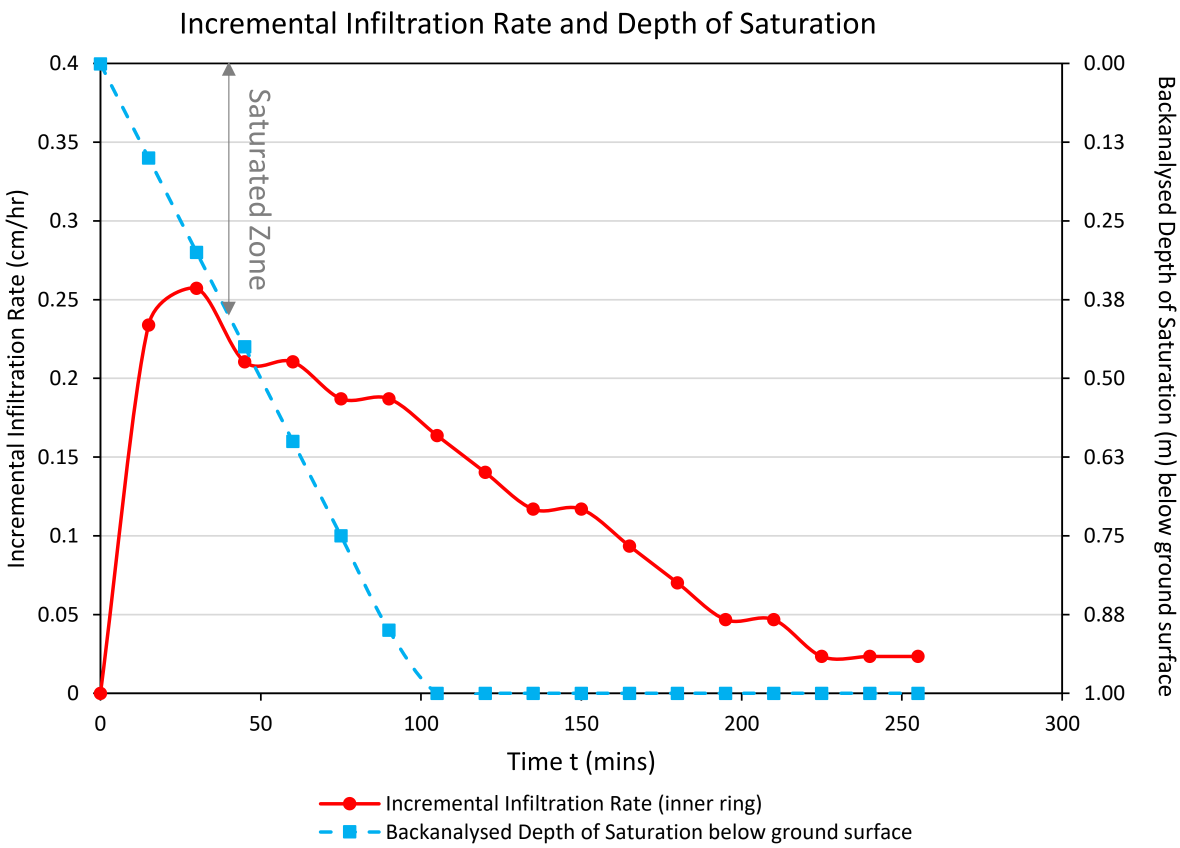
TEST ID = *LC1*
Location = *Ch 40,585 (near centreline)*
TEST Date = *23-Mar-17*
Inner diameter of inner ring, d_i (mm) = *330*
Inner diameter of outer ring, d_o (mm) = *650*
Depth of embedment into ground, e (mm) = *150*
Water height above Ground Surface, h (mm) = *290*
Thickness of soil being infiltrated, h_s (m) = *1**
Description of soil being infiltrated = *Gravelly SAND***
Water Temperature at the start of test (C) = *17*
Weather during the test = Cloudy

$h_{w(t)}$ = Depth of saturated soil at time t

** Based on nearby borehole (see Fig. 1)*
*** Based on near surface observation*



Hydraulic Conductivity (K) (m/sec) = *3.80E-07* (assumed based on Inversed Auger Hole Test)



Note: Depth of Saturation below the ground surface was backanalysed by matching the value of Hydraulic Conductivity adopted from laboratory testing or other in-situ testing

drawn	[redacted]	client: **CPB Contractors Pty Lrd**
approved		project: **Moorebank Intermodal Rail Link**
date	4/10/2017	title: Double Ring Infiltration Test Result - LC1
scale	NTS	project no: **GEOTLCOV24072AF**
original size	A4	figure no: **2**

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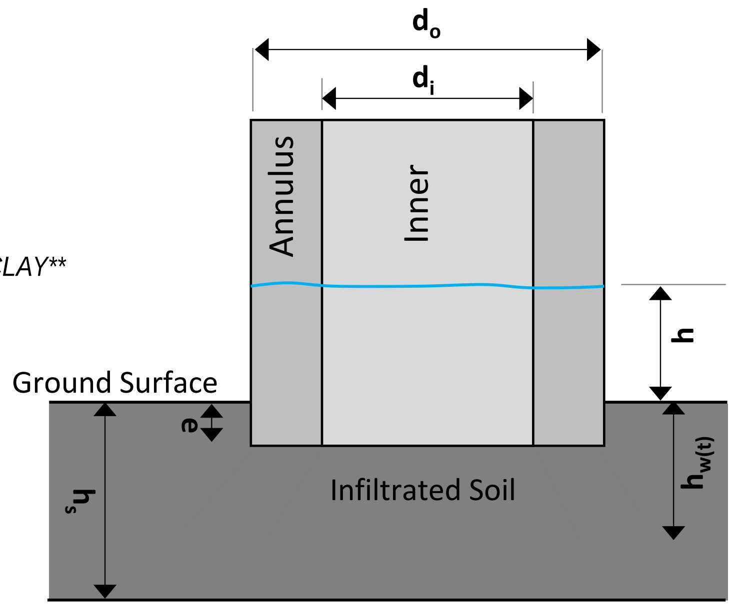
DOUBLE RING INFILTRATION TEST (ASTM D3385-03)

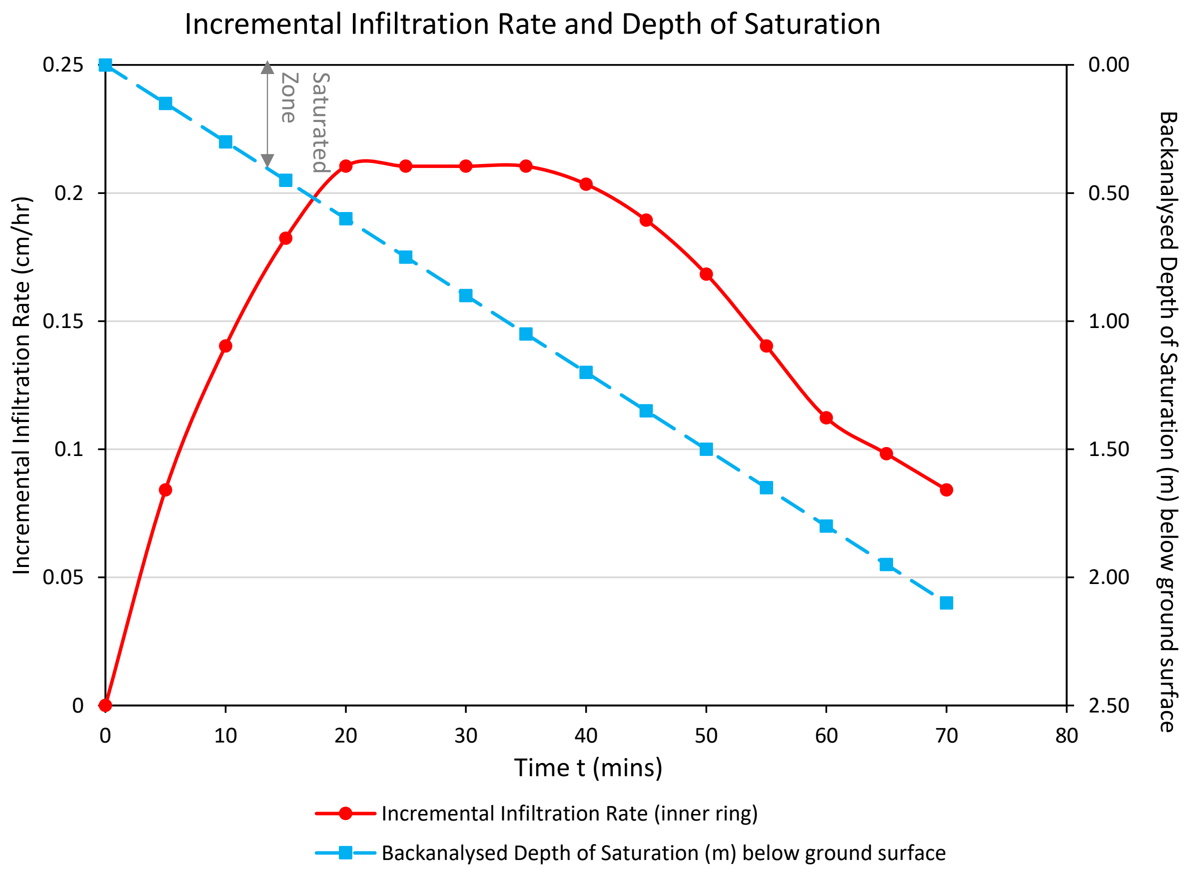
TEST ID = *LC2*
Location = *Ch 40,675 - MB2S (near centreline)*
TEST Date = *23-Mar-17*
Inner diameter of inner ring, d_i (mm) = *330*
Inner diameter of outer ring, d_o (mm) = *650*
Depth of embedment into ground, e (mm) = *150*
Water height above Ground Surface, h (mm) = *270*
Thickness of soil being infiltrated, h_s (m) = *2.5**
Description of soil being infiltrated = *Gravelly CLAY***
Water Temperature at the start of test (C) = *17*
Weather during the test = Cloudy

$h_{w(t)}$ = Depth of saturated soil at time t

** Based on nearby borehole (see Fig. 1)*
*** Based on near surface observation*



Hydraulic Conductivity (K) (m/sec) = *2.60E-07* (assumed based on Inversed Auger Hole Test)



Note: Depth of Saturation below the ground surface was backanalysed by matching the value of Hydraulic Conductivity adopted from laboratory testing or other in-situ testing

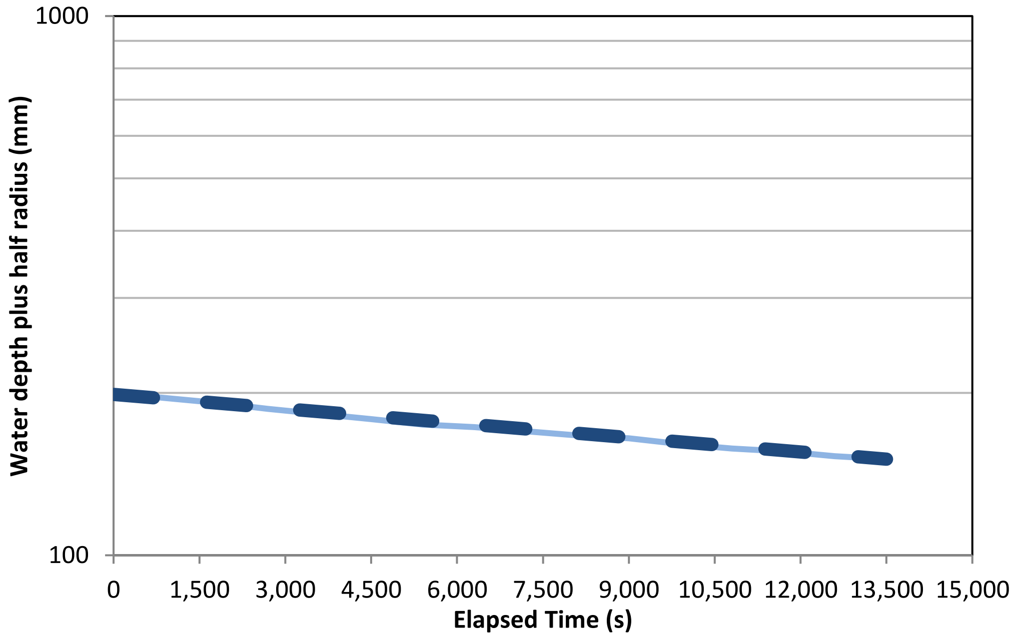
drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	**CPB Contractors Pty Ltd**
approved			project:	**Moorebank Intermodal Rail Link**
date	4/10/2017			
scale	NTS		title:	Double Ring Infiltration Test Result - LC2
original size	A4		project no: **GEOTLCOV24072AF**	figure no: **3**

Appendix B – Results of Inversed Auger Hole Testing

HOLE DIMENSIONS	
Diameter (mm)	75
Depth (mm)	180

INVERSED AUGER TEST

MEASUREMENTS	
Time (hr:min:secs)	Depth (mm)
0:00:00	0
0:01:00	0
0:03:00	0
0:07:00	1
0:15:00	3
0:30:00	7
0:45:00	12
1:00:00	16
1:15:00	20
1:30:00	24
1:45:00	26
2:00:00	29
2:15:00	32
2:30:00	34
2:45:00	38
3:00:00	41
3:15:00	43
3:30:00	46
3:45:00	48



Match interval	
Initial time	0:00:00
Final time	3:45:00

Intepreted Permeability
3.8E-07 m/s
3.3E-02 m/d

Note: Method described by: RJ Oosterbaan and HJ Nijland in *Determining the saturated hydraulic conductivity*

See chapter 12 of *Drainage Principals and Applications* ILRI Publication 16 2nd Edition 1994

drawn	[redacted]	client:	CPB Contractors Pty Ltd
approved		project:	Moorebank Intermodal Rail Link Moorebank
date	10-Apr-2017		
scale	AS SHOWN	title:	Inversed Auger Testing - HA1
original size	A4	project no:	GEOTLCOV24072AF
		figure no:	4

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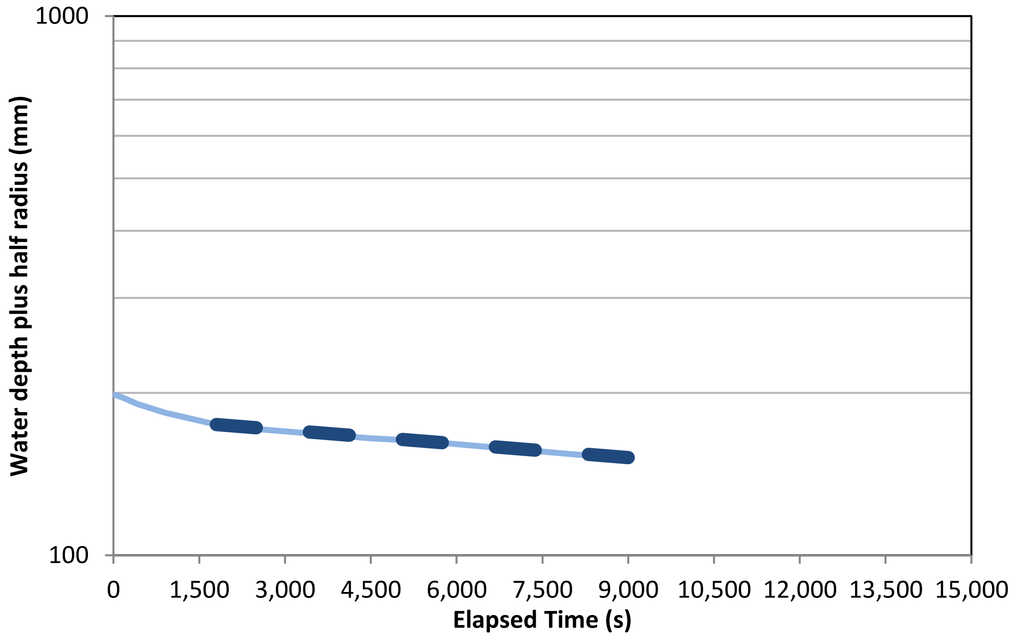
HOLE DIMENSIONS	
Diameter (mm)	75
Depth (mm)	180

MEASUREMENTS	
Time (hr:min:secs)	Depth (mm)
0:00:00	0
0:01:00	1
0:03:00	3
0:07:00	8
0:15:00	15
0:30:00	24
0:45:00	28
1:00:00	31
1:15:00	34
1:30:00	36
1:45:00	39
2:00:00	42
2:15:00	45
2:30:00	47

INVERSED AUGER TEST



Match interval	
Initial time	0:30:00
Final time	2:30:00

Intepreted Permeability
3.7E-07 m/s
3.2E-02 m/d

Note: Method described by: RJ Oosterbaan and HJ Nijland in *Determining the saturated hydraulic conductivity*

See chapter 12 of *Drainage Principals and Applications* ILRI Publication 16 2nd Edition 1994

drawn	[redacted]	client:	CPB Contractors Pty Ltd
approved		project:	Moorebank Intermodal Rail Link Moorebank
date	10-Apr-2017		
scale	AS SHOWN	title:	Inversed Auger Testing - HA2
original size	A4	project no: **GEOTLCOV24072AF**	figure no: **5**

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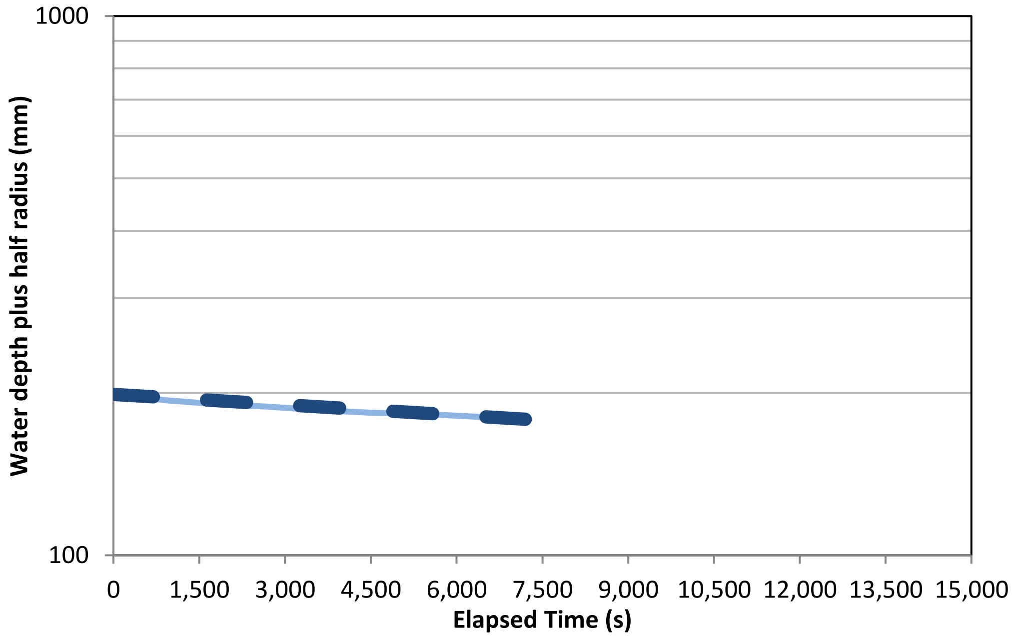
HOLE DIMENSIONS	
Diameter (mm)	75
Depth (mm)	180

INVERSED AUGER TEST

MEASUREMENTS	
Time (hr:min:secs	Depth (mm)
0:00:00	0
0:01:00	0
0:03:00	1
0:07:00	2
0:15:00	5
0:30:00	8
0:45:00	10
1:00:00	13
1:15:00	15
1:30:00	16
1:45:00	18
2:00:00	20



Match interval	
Initial time	0:00:00
Final time	2:00:00

Intepreted Permeability	
2.8E-07	m/s
2.4E-02	m/d

Note: Method described by: RJ Oosterbaan and HJ Nijland in *Determining the saturated hydraulic conductivity*

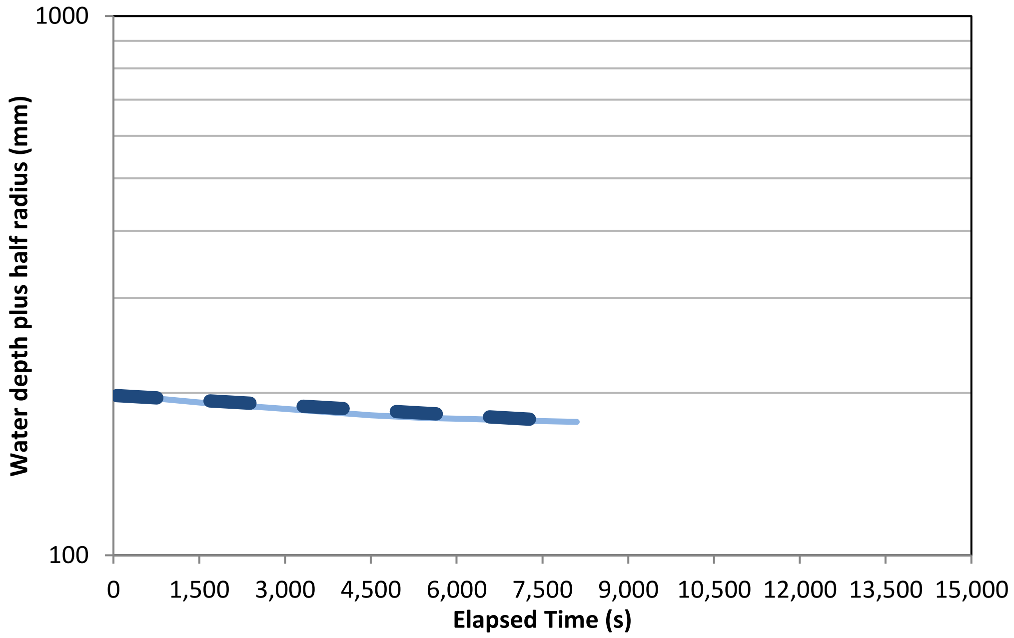
See chapter 12 of *Drainage Principals and Applications* ILRI Publication 16 2nd Edition 1994

drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	CPB Contractors Pty Ltd
approved			project:	**Moorebank Intermodal Rail Link** **Moorebank**
date	10-Apr-2017		title:	Inversed Auger Testing - HA3
scale	AS SHOWN		project no:	**GEOTLCOV24072AF** figure no: **6**
original size	A4			

HOLE DIMENSIONS	
Diameter (mm)	75
Depth (mm)	180

INVERSED AUGER TEST

MEASUREMENTS	
Time (hr:min:secs	Depth (mm)
0:00:00	0
0:01:00	1
0:03:00	1
0:07:00	2
0:15:00	4
0:30:00	8
0:45:00	11
1:00:00	14
1:15:00	17
1:30:00	19
1:45:00	20
2:00:00	21
2:15:00	22



Match interval	
Initial time	0:01:00
Final time	2:15:00

Intepreted Permeability
2.6E-07 m/s
2.3E-02 m/d

Note: Method described by: RJ Oosterbaan and HJ Nijland in *Determining the saturated hydraulic conductivity*

See chapter 12 of *Drainage Principals and Applications* ILRI Publication 16 2nd Edition 1994

drawn	[redacted]	client: CPB Contractors Pty Ltd
approved		project: Moorebank Intermodal Rail Link Moorebank
date	10-Apr-2017	
scale	AS SHOWN	title: Inversed Auger Testing - HA4
original size	A4	project no: GEOTLCOV24072AF / figure no: 7

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Appendix C – Results of Laboratory Testing

Dry Density Ratio Report

Sydney Laboratory

Coffey Services Australia Pty Ltd
31 Hope Street
Melrose Park NSW 2114
ABN 55 139 460 521
Phone: +61 (2) 9352 5000

Report No: DDR:SYDN17W00961
Issue No: 1

Client: Coffey Services Australia Pty Ltd (Chatswood)
Level 19, 799 Pacific Highway
Chatswood NSW 2067

Principal:
Project No.: 754-SYDN00058AA
Project Name: 754-GEOTLCOV24072AF - 754 EARLY SERVICES- MOOREBANK
Lot No.: **TRN:**

Accredited for compliance with ISO/IEC 17025 - Testing.

The results of the tests, calibrations and/or measurements included in this document are traceable [redacted]

NATA Accredited Laboratory Number:431
Date of Issue: 6/04/2017

Sample Details

Location:	Glenfield Tip
Client Request ID:	
Specification Requirements:	
Field Test Procedures:	AS 1289.5.8.1
Laboratory Test Procedures:	AS 1289.5.1.1, AS 1289.2.1.1, AS 1289.5.4.1
Sampling Method:	AS1289.1.2.1 Clause 6.4 (b)
Source:	Ex. Site
Material:	Fill

Sample Data

Sample ID	SYDN17S-02181	SYDN17S-02182
Field Sample ID	00001	00002
Date Tested	26/03/2017	26/03/2017
Time Tested	09:00	09:20
Location	L.C.1	L.C.2
Easting	0307064.8	0307088.7
Northing	6239995.9	6239913.4
RL	13.49	16.20
Soil Description	Gravelly SAND	Gravelly CLAY

Field and Laboratory Data

Depth of Test (mm)	300	300
Depth of Layer (mm)	300	300
Compactive Effort	Standard	Standard
AS Sieve Size (mm)	19.0	19.0
Oversize Wet (%)	0	0
Oversize Dry (%)	0	0
Field Moisture Content (%)	8.8	15.6
Field Wet Density (t/m³)	2.28	2.08
Field Dry Density (t/m³)	2.09	1.80
Lab Result from Test No.	SYDN17S-02181	SYDN17S-02182
Maximum Dry Density* (t/m³)	2.09	1.81
Optimum Moisture Content* (%)	10.0	14.5
Moisture Ratio (%)	87.0	108.5
Moisture Variation	1.5 dry	1.0 wet
Density Ratio (%)	**100.0**	**99.5**
legend * adjusted for oversize material		

Comments

Artarmon, Sydney Laboratory

Coffey Services Australia Pty Ltd
ABN 55 139 460 521
31 Hope Street
Melrose Park NSW 2114

Phone: +61 (2) 9352 5000

A TETRA TECH COMPANY

Maximum Dry Density Report

Report No: MDD:ARTA17S-00207
Issue No: 1

Client:	Coffey Services Australia Pty Ltd (Chatswood) Level 19, 799 Pacific Highway Chatswood NSW 2067
Principal:	CPB Contractors
Project No.:	754-ARTA00035AF
Project Name:	754-GEOTLCOV24072AF - 754 Sydney Lab External Proj
Lot No.:	**TRN:**

NATA
WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.

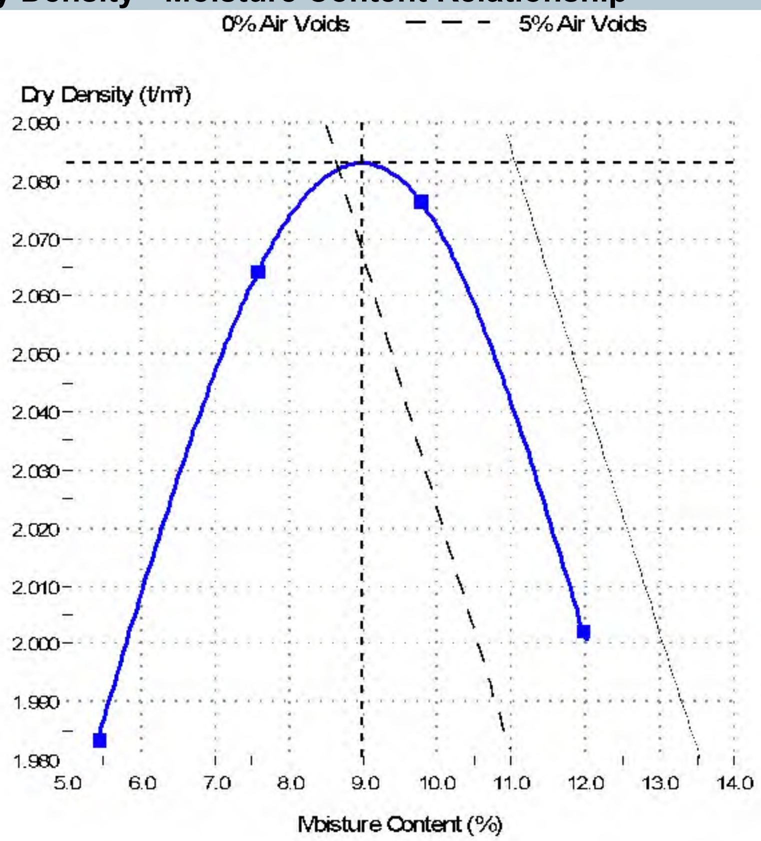
[redacted]

(Specialised Testing Manager)
NATA Accredited Laboratory Number:431
Date of Issue: 7/04/2017

Sample Details

Sample ID:	ARTA17S-00207	**Sampling Method:**	Submitted by client
Date Sampled:	23/03/2017	**Material:**	Subgrade
Date Submitted:		**Source:**	Ex Job Site
Date Tested:	3/04/2017	**Specification:**	No Specification
Project Location:	Moorebank, NSW		
Sample Location:	LC1 (0.05 to 0.30 m)		

Dry Density - Moisture Content Relationship



Test Results

AS 1289.5.1.1

Standard MDD (t/m³):	**2.08**
Standard OMC (%):	**9.0**
Retained Sieve 19.0mm (%):	0

Comments

Coffey Corporate Services Pty Ltd
31 Hope Street, Melrose Park, NSW 2114
ph: (61 2) 9352 5000
ABN: 92 114 364 046

test results - falling head permeability report

client:	*Coffey Services Australia Pty Ltd (GEOTLCOV24072AF)*	job no:	*754-ARTA00034AA*
principal:	*CPB Contractors*	laboratory:	*Melrose Park*
project:	*Moorebank Intermodal Rail Connection*	report date:	*7th April, 2017*
location:	*Morrebank, NSW*	test report:	*IOLT 9730*
test procedure:	*AS 1289.6.7.2*	test date:	*03/04/17 to 07/04/17*

Sample Identification	REMOULDED DRY DENSITY	REMOULDED MOISTURE CONTENT	REMOULDED FALLING HEAD PERMEABILITY	REMOULDED FALLING HEAD PERMEABILITY
	t/m^3	(%)	cm/sec	m/sec
LC1 ***(0.05 to 0.30 m)***	*2.08*	*9.0*	6.6×10^{-7}	6.6×10^{-9}
Artarmon Sample Number: ARTA17S-00207				

Notes:
1. *Specimen recompacted to 100% of Standard Maximum Dry Density and at Standard Optimum Moisture Content.*
2. *Specimen tested with Distilled Water.*
3. *0.0 % material retained on the 19mm sieve*
4. *0 kPa pressure was applied to the specimen.*
5. *Sample received from Client*

Remarks:

F:\2. TECHNICAL\INFO-TESTING\01. Laboratory_754-QEXT-SYDN-FY17\754-ARTA00035AF - Moorebank, NSW\[LC1_0.05-0.30_FHP.xls]Sheet1

NATA Accredited Laboratory
No 431

Date: 7th April, 2017

Approve[redacted]
Signator[redacted]

Artarmon, Sydney Laboratory

Coffey Services Australia Pty Ltd
ABN 55 139 460 521
31 Hope Street
Melrose Park NSW 2114

Phone: +61 (2) 9352 5000

coffey
A TETRA TECH COMPANY

Maximum Dry Density Report

Report No: MDD:ARTA17S-00208
Issue No: 1

Client:	Coffey Services Australia Pty Ltd (Chatswood) Level 19, 799 Pacific Highway Chatswood NSW 2067
Principal:	CPB Contractors
Project No.:	754-ARTA00035AF
Project Name:	754-GEOTLCOV24072AF - 754 Sydney Lab External Proj
Lot No.:	**TRN:**

NATA
WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing.

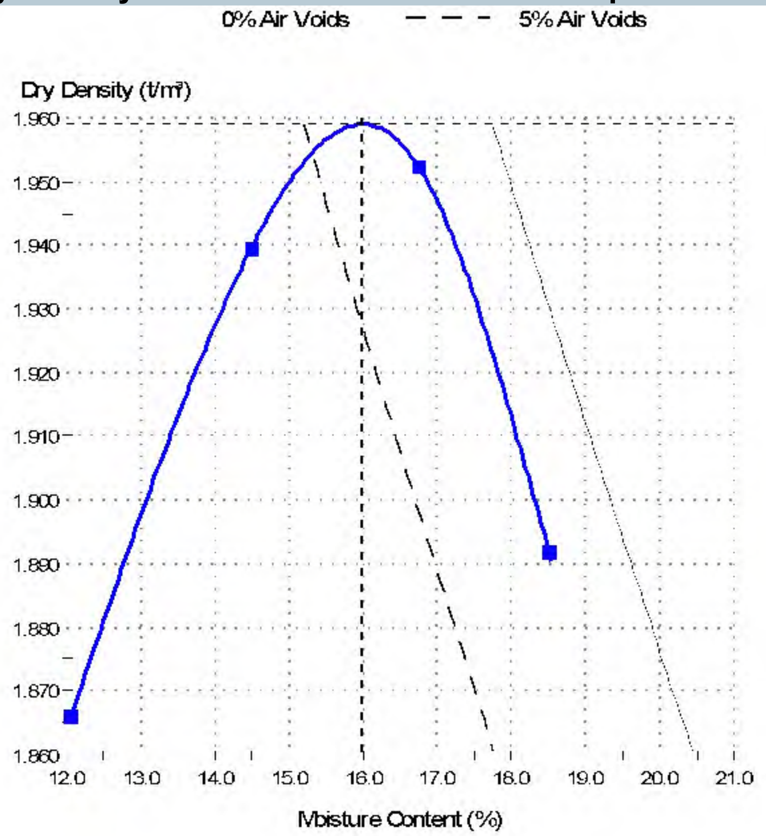
The results of the tests, calibrations and/or measurements included in this document are traceable [redacted]

(Specialised Testing Manager)
NATA Accredited Laboratory Number:431
Date of Issue: 7/04/2017

Sample Details

Sample ID:	ARTA17S-00208	**Sampling Method:**	Submitted by client
Date Sampled:	23/03/2017	**Material:**	Subgrade
Date Submitted:		**Source:**	Ex Job Site
Date Tested:	3/04/2017	**Specification:**	No Specification
Project Location:	Moorebank, NSW		
Sample Location:	LC2 (0.05 to 0.30 m)		

Dry Density - Moisture Content Relationship



Test Results

AS 1289.5.1.1

Standard MDD (t/m³): **1.96**
Standard OMC (%): **16.0**
Retained Sieve 19.0mm (%): 0

Comments

Coffey Corporate Services Pty Ltd
31 Hope Street, Melrose Park, NSW 2114
ph: (61 2) 9352 5000
ABN: 92 114 364 046

test results - falling head permeability report

client:	*Coffey Services Australia Pty Ltd (GEOTLCOV24072AF)*	job no:	*754-ARTA00034AA*
principal:	*CPB Contractors*	laboratory:	*Melrose Park*
project:	*Moorebank Intermodal Rail Connection*	report date:	*7th April, 2017*
location:	*Morrebank, NSW*	test report:	*IOLT 9731*
test procedure:	*AS 1289.6.7.2*	test date:	*03/04/17 to 07/04/17*

Sample Identification	REMOULDED DRY DENSITY t/m^3	REMOULDED MOISTURE CONTENT (%)	REMOULDED FALLING HEAD PERMEABILITY cm/sec	REMOULDED FALLING HEAD PERMEABILITY m/sec
LC2 ***(0.05 to 0.30 m)*** *Artarmon Sample Number: ARTA17S-00208*	*1.96*	*16.0*	4.5×10^{-8}	4.5×10^{-10}

Notes:
1. *Specimen recompacted to 100% of Standard Maximum Dry Density and at Standard Optimum Moisture Content.*
2. *Specimen tested with Distilled Water.*
3. *0.0 % material retained on the 19mm sieve*
4. *0 kPa pressure was applied to the specimen.*
5. *Sample received from Client*

Remarks:

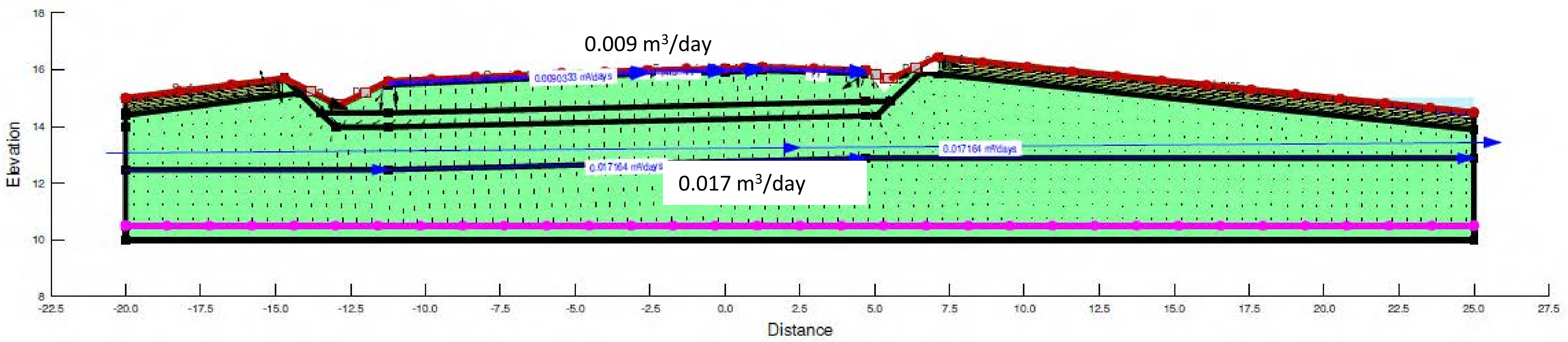
NATA Accredited Laboratory

No 431

Date: 7th April, 2017

Approved Signatory: [redacted]

Attachment 2: SEEP/W output plot for flow through the formation



drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	CPB Contractors Pty Ltd
approved			project:	Moorebank Intermodal Rail Link Infiltration assessment
date	11/04/2017			
scale	NTS		title:	SEEP/W output for flow through the rail formation
original size	A4		project no: GEOTLCOV24072AF	figure no: A2-1

F:\2. TECHNICAL\GEOTECHNICS\1.PROJECTS\GEOTLCOV240\GEOTLCOV24072AF Early Services Moorebank Rail Link\4. Correspondence\[Figure.xls]A4 Landscape Figure

Appendix N – Verification Records

Review and Verification

Project: Moorebank Intermodal Terminal Development – Package 1 – RALP No. 1

<table>
<tr><td colspan="4">Work Verification Record</td></tr>
<tr><td>Design Lot No:
N01031-GRW-DRP-GEO-0001</td><td>Issue (Rev):
05 (Final)</td><td colspan="2">Description: Ground Treatment Design between Ch 40,440 and Ch 40,740 (MB2S)</td></tr>
<tr><td colspan="4">Comments/Instructions for document control:</td></tr>
<tr><td colspan="4">☐Drawings ☐Report ☐Calculations
☐Specification ☐Other (please specify)</td></tr>
<tr><td colspan="2">Reason for issue
☐ Fifth Issue</td><td colspan="2">☐Minor Modification to Approved Design
☐Significant Modification to Approved Design</td></tr>
<tr><td colspan="4">Description of Modification:
Significant Modification to Design Calculation, Report, and Drawings</td></tr>
<tr><td colspan="2">Designer [redacted]</td><td>Signed [redacted]</td><td>Date 09/07/2018</td></tr>
<tr><td colspan="4">Design Verification</td></tr>
<tr><td colspan="2">Pre-requisite:
☐Engineering/Construction Manager Acceptance</td><td colspan="2">☐Principal's review comments addressed</td></tr>
<tr><td colspan="4">Result of Independent Design Verification (Action/Comments as a result of Review)</td></tr>
<tr><td colspan="4">Comments:</td></tr>
<tr><td colspan="2">Verifier/Project Directo[redacted]</td><td>Signed [redacted]</td><td>Date 09/07/2018</td></tr>
<tr><td colspan="4">Comments Addressed and Review Accepted</td></tr>
<tr><td colspan="2">Geotechnical Design Manager: [redacted]</td><td>Signed [redacted]</td><td>Date 09/07/2018</td></tr>
</table>

Appendix O – Reinforced embankment batter facing details at Ch 40,740 at bridge approach

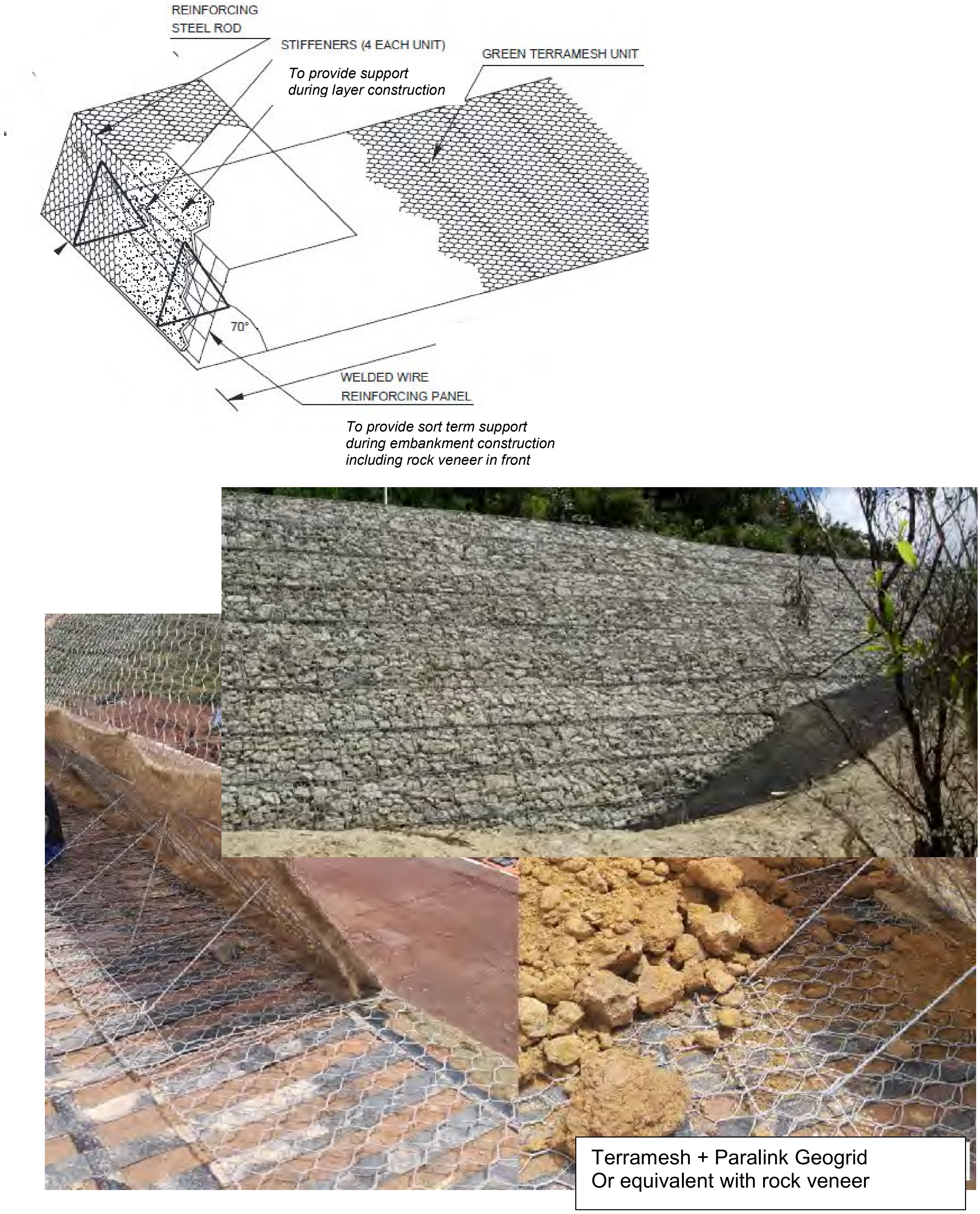


Figure O1: Typical details of Terramesh or equivalent facing with rock veneer

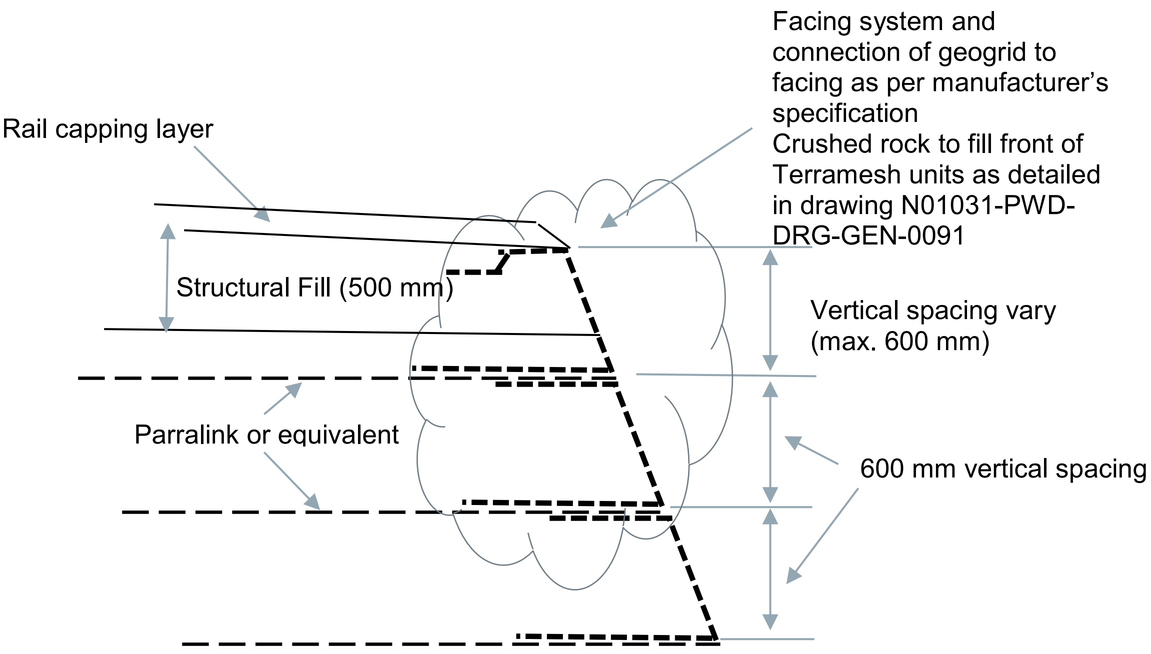
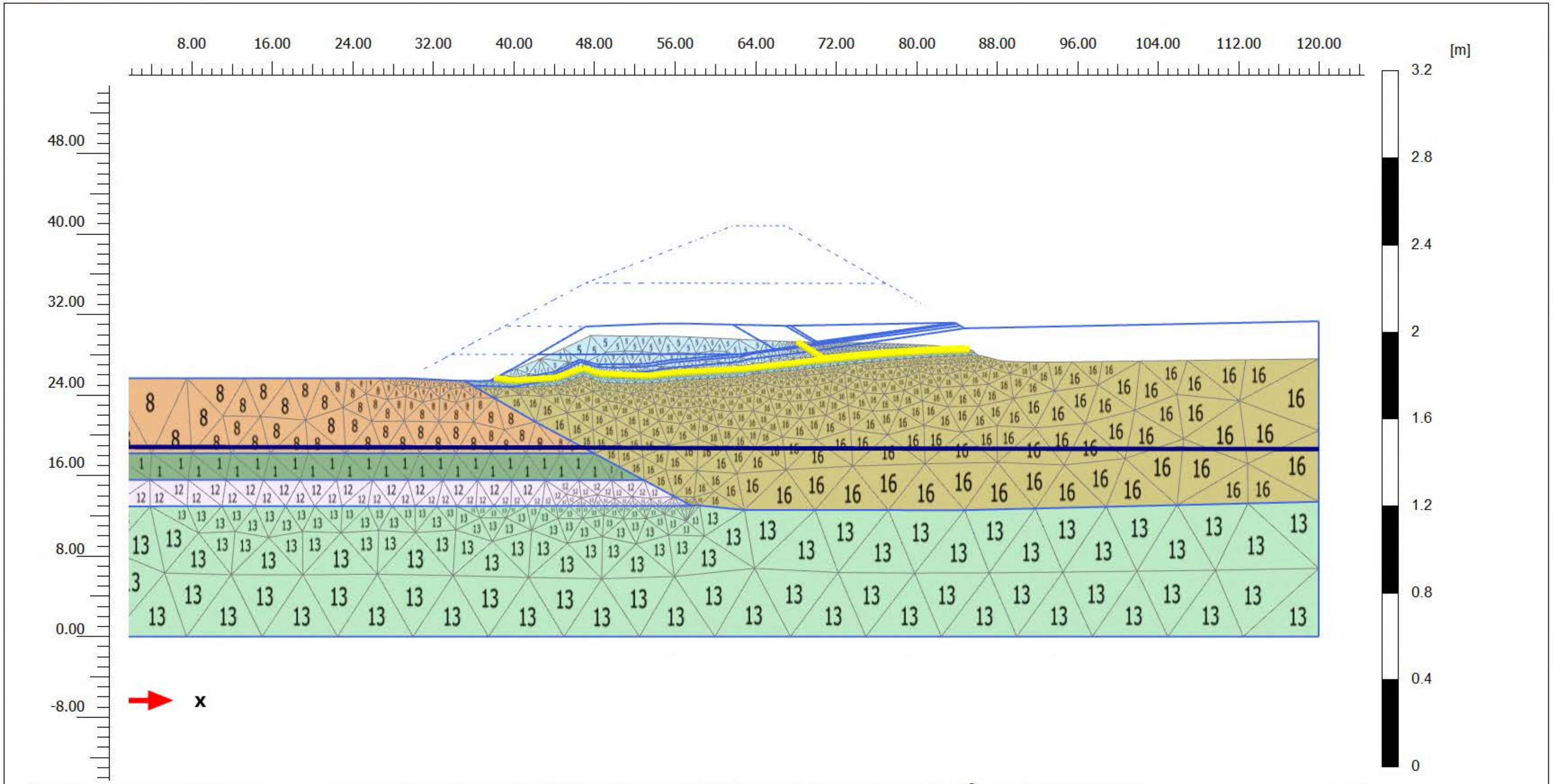


Figure O2: Details of Terramesh or equivalent facing at rail capping layer

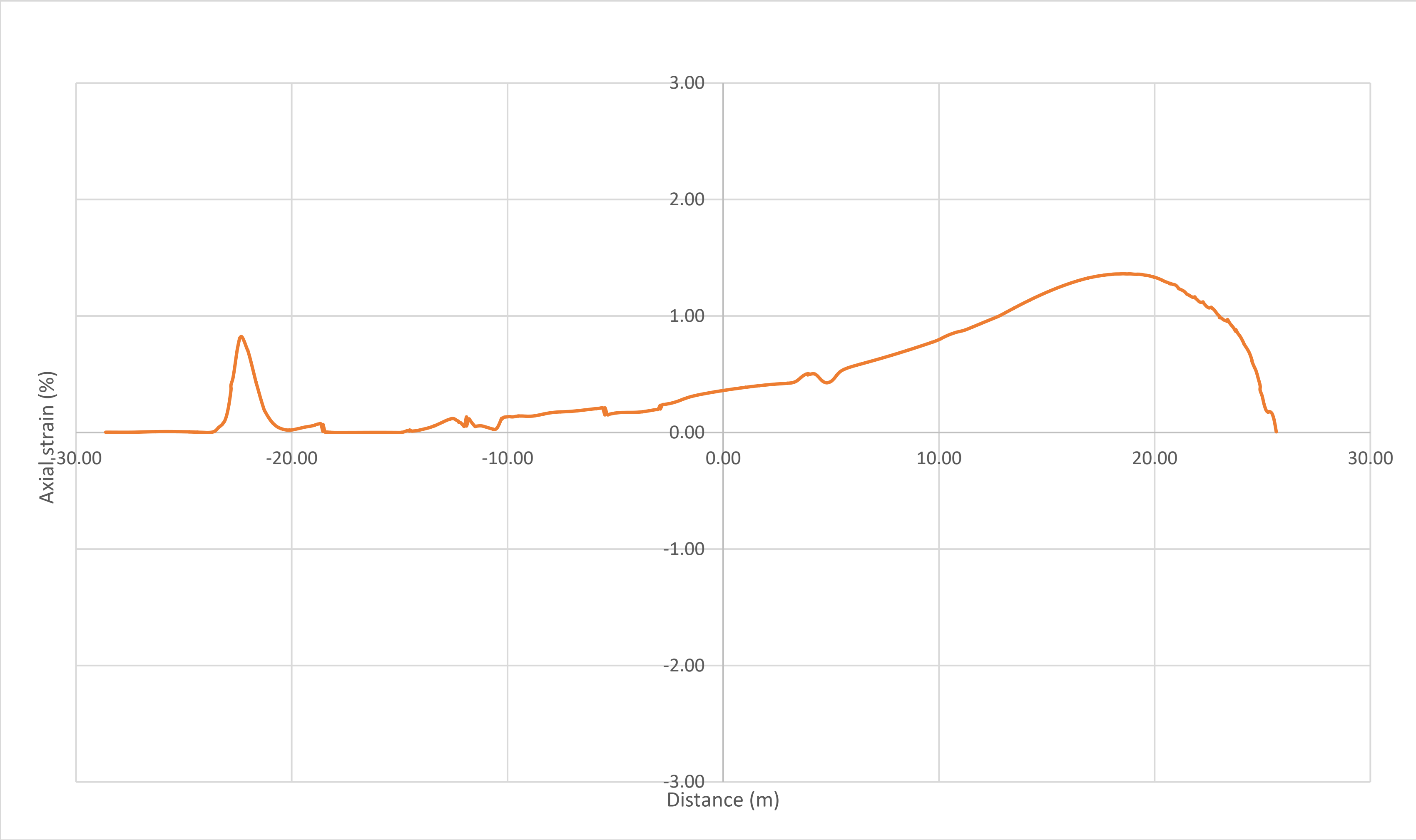
Appendix P – Results of Settlement Impact Assessment on Landfill Liner

Output Version 2017.1.0.0

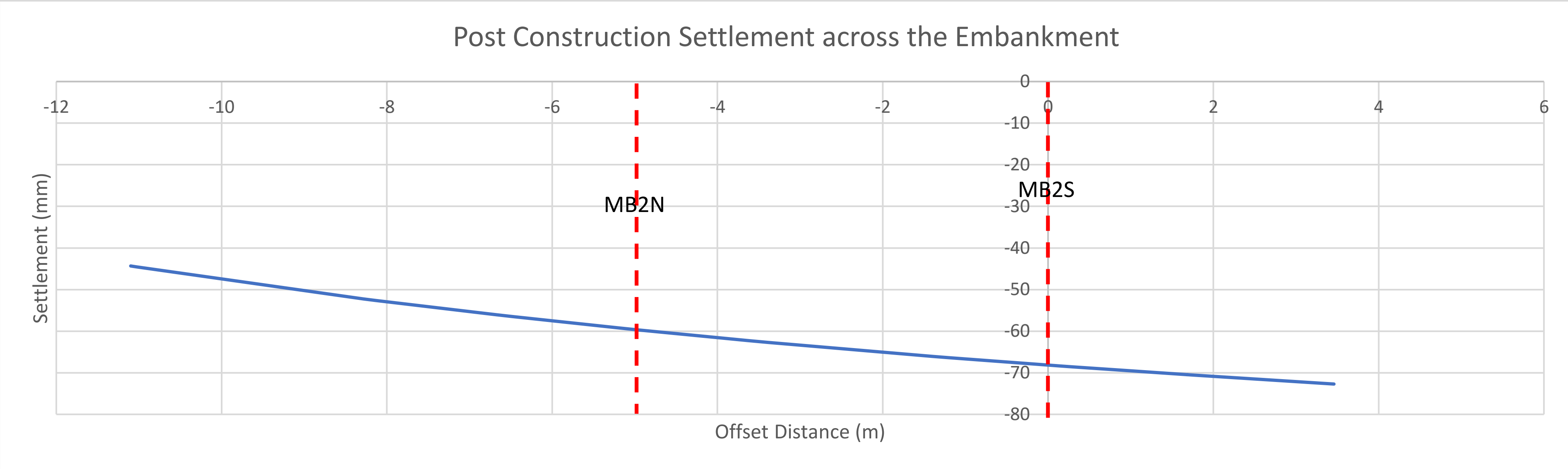


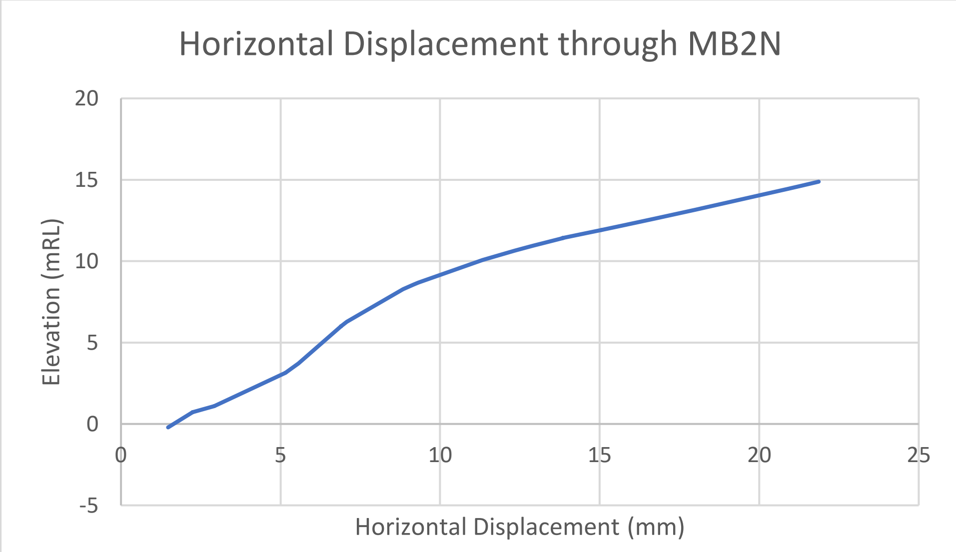
Deformed mesh |u| (scaled up 20.0 times) (Time $18.35*10^3$ day)

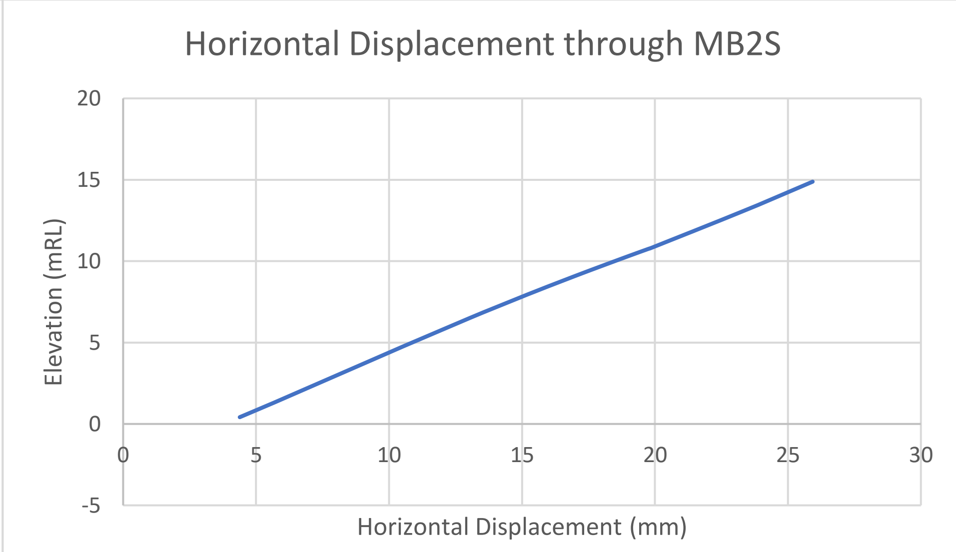
drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	**CPB Contractors**
approved			project:	**Moorebank Intermodal Rail Link**
date	**06/07/2018**			
scale	**NTS**		title:	**Ch. 40+540 – Deformed Mesh**
original size	**A4**		project no: GEOTLCOV24072AG	figure no: **P1**



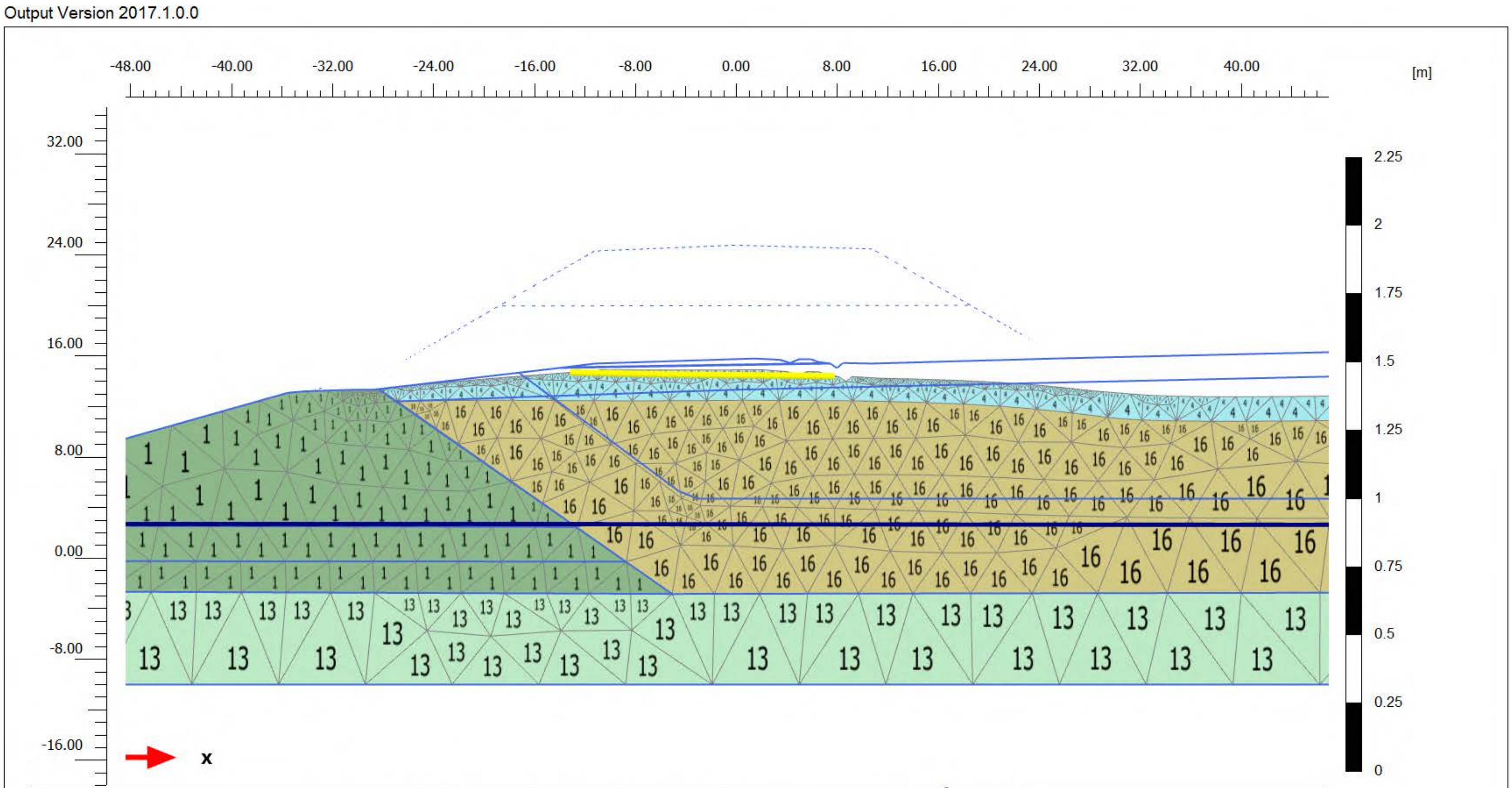
drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	CPB Contractors
approved	[redacted]		project:	Moorebank Intermodal Rail Link
date	06/07/2018		title:	Ch. 40+540 – Axial Strain on GCL
scale	NTS			
original size	A4		project no: GEOTLCOV24072AG	figure no: P2







drawn	[redacted]	coffey A TETRA TECH COMPANY	client:	**CPB Contractors**
approved			project:	**Moorebank Intermodal Rail Link**
date	**06/07/2018**		title:	**Ch. 40+540 – Post construction settlement and horizontal movement**
scale	**NTS**		project no: GEOTLCOV24072AG	figure no: **P3**
original size	**A4**			



Deformed mesh |u| (scaled up 20.0 times) (Time 18.35*10^3 day)

drawn	[redacted]	client:	**CPB Contractors**
approved		project:	**Moorebank Intermodal Rail Link**
date	**06/07/2018**	title:	**Ch. 40+640 – Deformed Mesh**
scale	**NTS**		
original size	**A4**	project no: GEOTLCOV24072AG	figure no: **P4**

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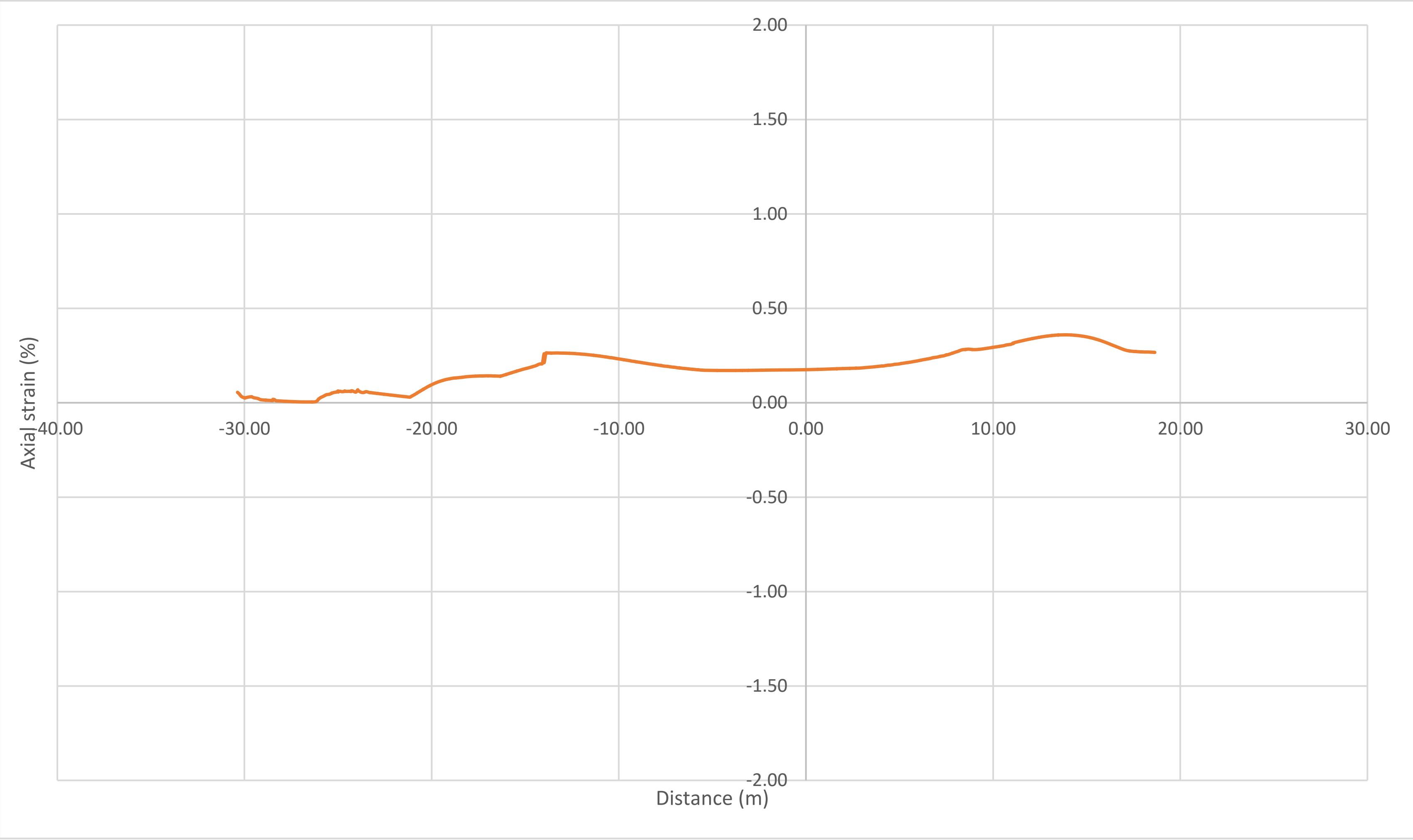

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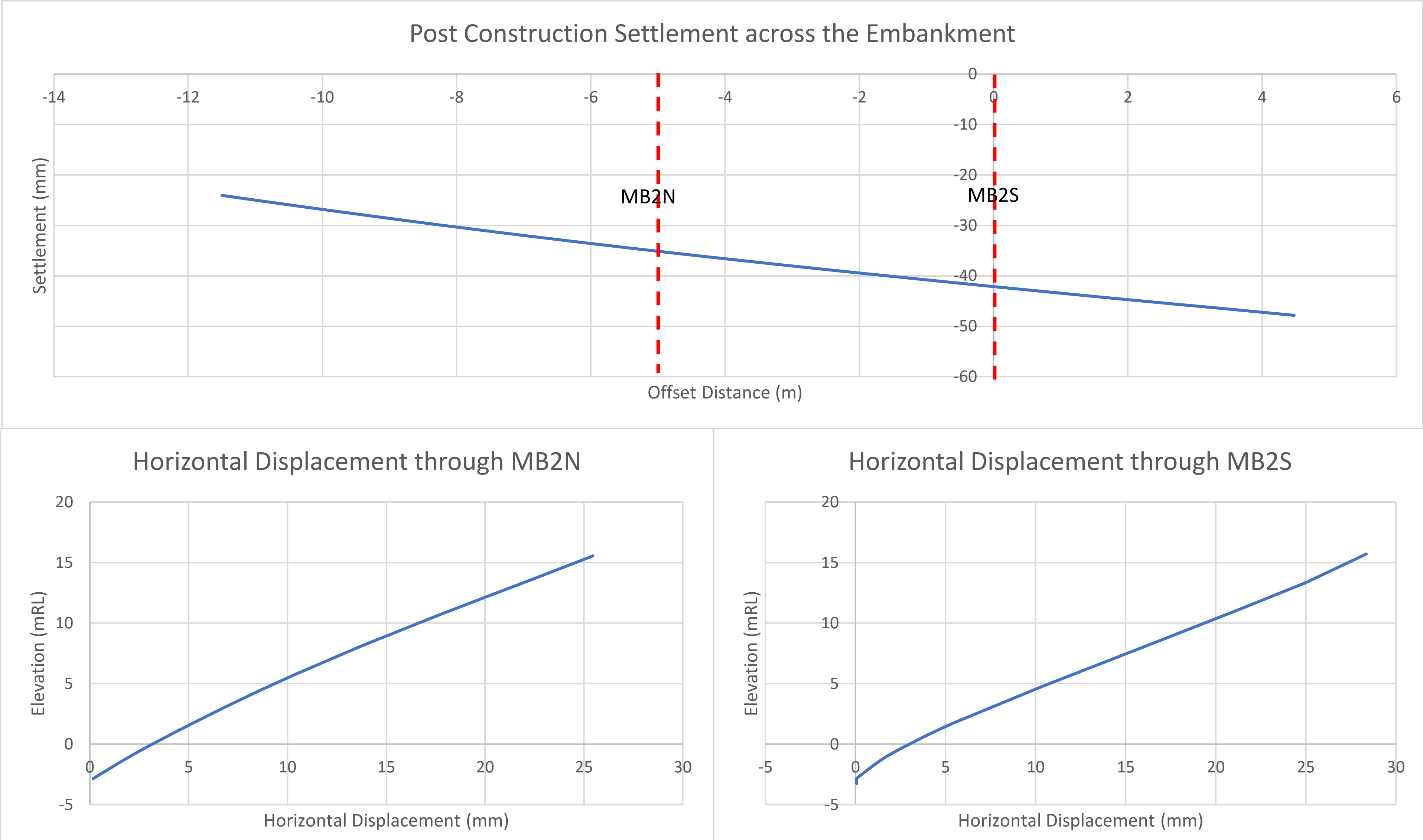
approved			project:	**Moorebank Intermodal Rail Link**
date	**06/07/2018**			
scale	**NTS**		title:	**Ch. 40+640 – Axial Strain on GCL**
original size	**A4**		project no: GEOTLCOV24072AG	figure no: **P5**



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approved			project:	Moorebank Intermodal Rail Link
date	**06/07/2018**		title:	**Ch. 40+640 – Post construction settlement and horizontal movement**
scale	**NTS**			
original size	**A4**		project no: GEOTLCOV24072AG	figure no: **P6**