SUEZ Recycling and Recovery Pty Ltd

Development Application No. DA 19/0470
Elizabeth Drive Landfill Expansion Technical Advice

February 2020

Executive summary

SUEZ owns and operates the Elizabeth Drive Landfill at Kemps Creek, NSW. SUEZ proposes to increase the capacity of the existing Elizabeth Drive Landfill by raising the currently approved finished maximum height by 15 metres, from RL 80 to RL 95.

The NSW EPA reviewed the Environmental Impact Statement prepared by AECOM (2019) in support of the expansion and issued an additional information and clarification letter request.

This report provides technical assessment and advice in response to the EPA letter (ref DOC 19/1012793 dated 20 November 2019) request for additional information.

In particular, this report provides further clarification and assessment regarding:

- A revised pipe loading assessment in accordance with the AS2566.1 Buried Flexible Pipelines. Based on the outcomes of the AS2566.1 analysis , critical pipework was further assessed utilising 2D PLAXIS to take into account arching within the waste and support from the bedding gravel
- A revised final capping design
- A slope stability analysis, assessing the stability of the side slopes of the proposed new landform.

The principal conclusions from the pipe strength, slope stability and final cap design assessments are summarised below.

Pipe strength assessment

The leachate pipe strength integrity assessment (undertaken in accordance with AS 2566.1 and PLAXIS 2D modelling) concluded that installed and proposed leachate collection pipes in general and restricted waste cells should not be affected by the proposed additional waste fill load associated with the proposed final landform elevation (maximum RL 95m). The strength of the leachate pipes is considered suitable to maintain performance of the leachate collection system (LCS). The modelling results support the required design safety factors would be achieved for two pipe failure modes, buckling and deflection, in accordance with the Environmental Guidelines Solid waste landfills, Second edition, 2016.

The drainage triaxial geocomposite and protection geotextile already instated (existing cells) and proposed to be installed (new cells) is considered to be suitable to accommodate the additional waste load.

The leachate pipe strength assessment as detailed in section 3 indicates that the leachate collection system at the Site is expected to satisfy long term performance criteria under the proposed additional waste fill load as part of the proposed expansion.

Final cap design

The revised final cap design will be in full conformance with the *NSW EPA Environmental Guidelines: Solid Waste Landfill, Second edition 2016*. The revised final cap is substantially thicker (>1.9m) than previously proposed and includes a subsurface drainage layer (on batters).

An adjustment to the EPL (licence) will be required, to reflect these changes to the cap design.

Slope stability assessment

The results of the slope stability analyses indicate that the proposed final landfill landform batter slopes are stable for the anticipated landfill extension. The proposed revised final cap design includes a subsurface drainage layer that improves the veneer stability of the landform batters and is predicted to be stable.

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1. Introduction

1.1 Overview

SUEZ Recycling and Recovery Pty Ltd (SUEZ) owns and operates the Elizabeth Drive Landfill at Kemps Creek, NSW (the Site). The Site includes both the active landfill operation, the SUEZ Advanced Waste Treatment (SAWT) facility and a landfill gas to energy system, which operate concurrently and independently of the landfill.

SUEZ proposes to increase the capacity of the existing Elizabeth Drive Landfill by raising the currently approved finished maximum cap height by 15 metres, from RL 80 to RL 95 (the Project). The approved and proposed landform contours are provided in Appendix A.

The Project would provide an additional landfill airspace capacity of approximately 4.8 million cubic metres and extend the life of the landfill by approximately five and a half years at the proposed disposal rate of 950,000 tonnes per annum (tpa).

An Environmental Impact Statement (EIS) was prepared by AECOM (2019) in support of the expansion of Elizabeth Drive Landfill.

The NSW EPA issued a request for additional information and clarification for the Elizabeth Drive Landfill expansion (ref. DOC 19/703132 dated 19 Aug 2019). GHD (2019) prepared a technical letter on behalf of SUEZ to address the EPA's comments with respect to:

- Leachate pipe strength
- Slope stability
- Final cap design

The EPA reviewed the GHD technical letter (GHD 2019) and has sought additional information detailed in a second letter (ref DOC 19/1012793 dated 20 November 2019) on the above three issues.

1.2 The project area

The project area currently operates as a regional landfill accepting non-putrescible general solid waste and restricted solid waste. The Project would provide an additional landfill airspace capacity for non-putrescible general solid waste of approximately 4.8 million cubic metres and extend the life of the landfill by approximately 5.5 years, based on an increased filling rate of 950,000 tonnes per annum.

It is envisaged that the rate of filling would increase slightly to take into account changes in the volume of waste being generated and disposed of in NSW and the industry capacity to receive the waste. Under the Project approximately 950,000 tpa of non-putrescible general solid waste and restricted solid waste is expected to be received during the remaining life of the landfill.

Landfilling operations would generally be undertaken in a manner consistent with the current practices and as outlined in the existing Elizabeth Drive Landfill Environmental Management Plan (SUEZ, 2018) for the Site. Waste would continue to be deposited, spread and compacted in layers. At the end of each working day, exposed waste surfaces would be covered with tarps and/or virgin excavated natural material (VENM) or other EPA approved material to reduce environmental impacts such as litter, odour etc, in compliance with the Environment Protection Licence (EPL) for the landfill operations.

The landfill cap would be progressively constructed and revegetated as soon as practicable after reaching final landform levels. It is anticipated that capping material would be predominantly sourced from material stockpiled during historic quarrying activities within the site, or imported from suitable external sources.

1.3 Purpose

This report provides technical assessment and advice in response to the EPA's second letter (ref DOC 19/1012793 dated 20 November 2019) request for additional information.

This report provides further clarification and assessment of all general and restricted waste cells as requested by the EPA's Letter. In particular, it provides further clarification and assessment based on the additional as-built details provided by SUEZ in relation to the 'D' and 'E' series cells regarding:

- The revised pipe loading assessment using 2D Plaxis modelling, assessing extra loading on the leachate pipes from the proposed additional waste;
- A slope stability analysis, assessing the stability of the side slopes of the proposed new landform based on the revised final capping design; and
- The revised final capping design in accordance with the NSW Landfill Guidelines.

1.4 Reliance

This document was prepared with reliance to the following documentation:

- AECOM 2019, Concept Design Technical Report Proposed Final Landform – Concept Design Report;
- AECOM 2019, Environmental Impact Statement Elizabeth Drive Landfill Expansion;
- AS/NZ 2566.1 Supplement 1:1998 Buried Flexible Pipelines Part 1: Structural Design – Commentary;
- AS/NZ 2566.1 Supplement 1:1998 Buried Flexible Pipelines Part 1: Structural Design – Commentary;
- AS/NZ 2566.1:1998 Buried Flexible Pipelines Part 1: Structural Design;
- AS/NZ 2566.1:1998 Buried Flexible Pipelines Part 1: Structural Design;
- Bentley S P (1996) Engineering Geology of Waste Disposal, The Geological Society, London;
- Dixon N & Jones V (2005) Engineering properties of municipal solid waste, *Geotextiles & Geomembranes,* 25:3, pp 205-33;
- ERM, 2018 Elizabeth Drive Landfill Annual Environmental Monitoring Report (AEMR);
- GHD 2019, Elizabeth Drive Landfill support services EDL Expansion Technical Advice;
- GHD, 2007, Elizabeth Drive Landfill Leachate Pipe Strength Calculations;
- Golder 2012, Environmental Assessment – Whytes Gully New Landfill Cell;
- Maunsell and AECOM 2007, Industrial Cell A4 – Leachate collection and conveyance system, drawing number: 20021405.01-CI-1005;
- Maunsell and AECOM 2007, SITA Industrial Waste Cell A5 Design Report;
- NSW EPA, 19 August 2019, Development Application No. DA19/0470 – Stop the Clock Letter Request for additional information and clarification;
- NSW EPA, 20 November 2019, Development Application No. DA19/0470 – Stop the Clock Letter Request for additional information;

- NSW EPA, 2016, Environmental Guidelines Solid waste landfills, Second edition, 2016;
- Oweis I S & Khera R P (1998) Geotechnology of Waste Management, Second Edition, PWS Publishing Company, Boston;
- Parsons Brinckerhoff, Elizabeth Drive Landfill Cell A3 - Bulk excavation plan and set out;
- Poliplex Polyethylene pipe Design Textbook (James Hardie Pipelines 1997);
- Qian X, Koerner R M & Gray D H (2002) Geotechnical Aspects of Landfill Design and Construction, Prentice-Hall Inc., New Jersey;
- R.B.J Brinkgreve, L.M. Zampich and N. Ragi Manoj 2019, PLAXIS 2D CONNECT Reference Manual;
- Rowe R K, Quigley R M, Brachman R W I, Booker J R, (2004) Barrier Systems for Waste Disposal Facilities, E & FN Spon, London;
- Rowe, 2001, *Geotechnical and Geoenvironmental Engineering Handbook;*
- SLR, 2012, SITA Australia Pty Ltd – Elizabeth Drive Landfill Solid Waste Cell E4 drawings;
- SUEZ, 2016, Environmental Management Plan - Elizabeth Drive Landfill (LEMP), 22 January 2016;
- SUEZ, 2018, Elizabeth Drive Landfill Environmental Management Plan; and
- VanGulck J F, Rowe R K (2004) Evolution of clog formation with time in columns permeated with synthetic landfill leachate*, Journal of Contaminant Hydrology* 75, pp 115–139.

1.5 Limitations

This report has been prepared by GHD for SUEZ Recycling and Recovery Pty Ltd and *may only be used and relied on by SUEZ Recycling and Recovery Pty Ltd for the purpose agreed between GHD and the SUEZ Recycling and Recovery Pty Ltd as set out in section 1.2 of this report.*

GHD otherwise disclaims responsibility to any person other than SUEZ Recycling and Recovery Pty Ltd arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by SUEZ Recycling and Recovery Pty Ltd and others who provided information to GHD (including Government authorities)], which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

2. Site overview

2.1 General

The Site and surrounds are generally flat with a slight fall in elevation from south to north. Badgerys Creek is located adjacent to the western Site boundary. The creek is an ephemeral watercourse, which flows from south to north following periods of sufficient rainfall.

The landfill is bound on all sides by access roads. The landfill currently includes over 34 cells, sub cells and mono-cells, all of which either currently or historically have accepted waste material. The southern portion of the landfill is capped, with an active general waste landfill area and active quarrying areas in the central and northern portion of the Site, and active restricted solid waste landfill cells in the central-eastern portion of the Site.

The Site is licensed under the Protection of the Environment Operations Act 1007 (EPL No. 4068) to accept the following waste streams into the landfill cells:

- General solid waste (non-putrescible);
- Asbestos waste;
- Waste tyres; and
- Restricted solid waste.

2.2 Project location

The Site is located at 1725 Elizabeth Drive in the suburb of Kemps Creek, approximately 41 kilometres west of the Sydney Central Business District (CBD), within the Penrith Local Government Area (LGA).

2.3 Local climate

Daily average maximum temperatures range from 30.1ºC in January to 17.4ºC in July at Badgerys Creek. Daily average minimum temperature range from 17.1ºC in January/February to 4.1ºC in July at Badgerys Creek. Annual average rainfall is 680.9 mm at Badgerys Creek (067108) and 765.0 mm at Horsley Park (067119).

2.4 Geology

The Elizabeth Drive Landfill is situated within the Cumberland Plain, which is generally a undulating shale landscape composed of the Triassic Wianamatta Group which is composed of, in ascending stratigraphic order, Ashfield Shale, Minchinbury Sandstone and Bringelly Shale. It should be noted that the Minchinbury Sandstone is thin to absent in parts of the Basin.

The local geology comprises the Bringelly Shale Formation of the Wianamatta Group of the Sydney Basin. The formation consists of relatively impervious, naturally occurring clays and shales, comprised of sub-horizontal carbonaceous claystone, siltstone, sandstone and laminite. The uppermost 5 to 6 metres of shale is highly weathered to plastic clay, varying in colour from mottled red to yellow and white. Geological long wall mapping of the sidewalls of excavations has indicated the rock mass is generally undisturbed with minor discontinuous vertical jointing.

Joints and fractures are frequently found to be in-filled with weathering products or deposition of secondary minerals.

2.5 Hydrogeology

Groundwater in the region is intercepted within the bedding planes/geological discontinuities of the shale at approximately 47 m AHD to the south/south-east, and at approximately 39 m AHD in the low-lying flood plain areas to the west/north-west regions near Badgerys Creek. Groundwater is primarily located within a relatively shallow transition zone between upper-lying weathered and underlying fresh shale, with the storage and transmission of groundwater below this zone decreasing with depth. Regional water quality within this groundwater system (which can be regarded as an aquitard) is brackish to saline, with elevated levels of iron and ammonia also known to occur (Golder 1991).

Regional groundwater is thought to generally flow in a west to north-westerly direction, but locally, the excavation works and landfill cells with their controlled leachate levels would influence the direction of groundwater movement at the Site.

A detailed review of the landfill's performance with respect to groundwater is provided in the AEMR. This covers the monitoring period for the 2018 calendar year and takes into account the entire historical record of groundwater monitoring at the Site.

A summary of the findings of reviewing the Site's groundwater quality follows:

- Groundwater monitoring has been undertaken at seven locations at the Site in accordance with the requirements of the EPL. Ammonia is an indicator for the possible presence of leachate in waters. Concentrations of ammonia in the groundwater collected for the Site have never exceeded the EPL reporting criteria of 15 mg/L;
- A statistical increasing trend in ammonia concentrations is observed in EPL identification points 17 (G3a), 18 (G4a), 20 (G6) and 22 (G9) which are all located along the western site boundary;
- A statistical decreasing trend in ammonia concentration was identified in EPL identification point 21 (G7); and
- Barium was is also observed to be statistically increasing and decreasing or stable in the groundwater.

To assess whether the fluctuating ammonia concentrations may be attributable to leachate, L/N ratios have been assessed. The leachate/native cation ratio (L/N from Mulvey 1996) was applied. The cations of potassium and ammonium rarely occur in the natural environment together but do accumulate in leachate derived from solid waste.

The finding from this L/N analysis is that statistically there is no conclusive evidence that leachate is impacting significantly on groundwater quality monitored at the Site.

With regard to the barium concentrations in the groundwater, they are above the measured concentrations of barium in the leachate and therefore likely to be reflective of natural conditions.

Further details on the groundwater quality at the Site are provided in the AEMR and the Annual Returns provided to the EPA.

2.6 Surface water

The Site is located adjacent to Badgerys Creek, a 16 km long minor tributary of South Creek. South Creek is a tributary of the Hawkesbury River and flows approximately 600 metres to the east of the Site. The South Creek catchment drains approximately 414 km^2 in western Sydney, stretching from Narellan in the south to the confluence with the Hawkesbury River at Windsor in the north.

Surface water drainage within the Project Area predominantly involves diversion drainage around the ridge of each active waste disposal cell to control surface water runoff flowing into the cells. It typically comprises of open channel drains on the outer edge of earthen bunds. Surface water is then collected in drains, swales and ponds before being diverted into one of five sediment dams around the Site boundary, listed in Table 2-1.

Table 2-1 Surface water dams

Name	Identifier	Design Capacity (m3)
Main water supply dam	S19 or F3	24,500
South western dam	S10 of F2	8,500
North western dam	S20 or sedimentation pond	6,160
North eastern dam	S5 or F4	5,700
The wheel wash dam	S9 or F1	1,200

The sediment dams, excluding S5, are inter-connected via pipelines or pump-out drains to transfer stormwater between these dams, to minimise off site overflow/discharge.

Surface water discharge is permitted in accordance with the water quality limits set out in the EPL. The site discharged surface water via the licensed discharge points on two occasions during 2018. Ammonia discharge limits were not exceeded on either occasion. On neither occasion were the discharge limits applicable for Total Suspended Solids due to rainfall in the preceding five days being sufficient to meet EPL criteria.

Sampling of surface water in Badgerys Creek both upstream and downstream of the Site is undertaken quarterly, as required by the LEMP and EPL. Results of sampling activities are included in the AEMR and are provided to Penrith City Council. The 2018 AEMR identified that surface water within Badgerys Creek did not indicate the presence of leachate impact. Some surface water locations were noted to be dry during the year and were not able to be sampled. These have been reported to the NSW EPA as data non-conformances by SUEZ.

2.7 Leachate management

Leachate is generated within the landfill cells through breakdown of waste, surface water infiltration and groundwater infiltration.

The Site is designed to maintain an inward groundwater hydraulic gradient, with groundwater contributing to the total leachate volume. Perimeter drainage control has been adopted to prevent surface water from adding to leachate reservoirs.

The waste cells are designed for leachate to percolate through the waste, until it reaches the landfill liner and drains to the leachate sump.

Leachate is collected via a grid of trapezoidal shaped drains incorporated in the bottom on the liner. These drains are filled with porous material and slope to header lines leading to a collection sump within each cell.

Leachate from the general solid waste (GSW) cells is then removed from the sump and transferred to 4 x 20 kL on-site storage tanks. Leachate from the restricted solid waste (RSW) cells is kept separate from the GSW leachate and stored in 8 x 20 kL tanks. From the storage tanks the leachate is then recirculated in the landfill. Some of the leachate is lost to evaporation and the remainder is retained within the solid waste. Any excess leachate is currently transported off site to a licensed facility for treatment.

2.8 Landfill gas

The primary function of the landfill gas management system is to control odorous emissions from the landfill. Landfill gas is collected via a series of wells and pipes, and transported to the gas engines adjacent to the SAWT. Here the gas is combusted to generate electricity, with excess gas being flared.

Gas infrastructure is maintained and monitored by the landfill gas contractors to ensure that landfill gases are being effectively managed. Landfill gas monitoring includes surface gas monitoring, subsurface gas monitoring and gas accumulation monitoring of buildings and structures (e.g. service pits and weighbridge hatches). It was concluded in the 2018 AEMR that the gas extraction system is effective at managing landfill gas at the Site.

The gas infrastructure and collection system consists of gas extraction wells, the associated header pipe, a knock-out pot, blower flare station, and two landfill gas to power generation engines (1.5 MWh each).

The gas extraction system complements the engineered containment system as it provides advective pressure relief, reducing the risk of a breach in the containment system and reducing upward migration of landfill gas prior to the construction of final capping. The active extraction coupled to a flare and electricity generators (x2) allows the effective destruction (in excess of 98% of NMOC and methane) and provides the added benefit of renewable energy.

2.9 Waste density

2.9.1 Published waste densities

Solid waste is a multiphase, heterogeneous material and as such, the in-situ unit weight varies widely both between various landfills and within differing depths and locations of a single landfill. Numerous factors are responsible for the variability including (Oweis & Khera 1998):

- Waste composition;
- State of decomposition;
- Degree of control during placement (thickness of daily cover etc);
- Compaction;
- Moisture content;
- Depth; and
- Settlement.

Qian et al (2002) conducted a literature review of published average unit weights noting that the data ranged from 3.1 to 13.2 kN/m^3 (note: the upper bound includes cover soil). Further to this they noted that several studies have been conducted using waste samples gathered from landfills and compacted in specialised test cells to study the effects on compression, stiffness and moisture content. A summary of the unit weight values gathered in these and other literature is included in Table 2-2 below.

Table 2-2 Typical unit weight of waste

Source	Waste Placement Conditions	Unit Weight (kN/m3)
NSWMA (1985 in Qian et al 2002)	Fresh	6.9 - 7.7
	After decomposition and settlement	9.9 – 11
Landva & Clark (1986 in Qian et al 2002)	Cover soil ratio of 2:1 to 10:1	9 - 13.2
EMCON Associates (1989 in Qian et al 2002)	Cover soil ratio of 6:1	7.2
Zorberg et al (1999 in Qian et al 2002)	(Including cover material) - Depths 8 - 50 m	10 – 15
Kavazanjian (2001 in Dixon & Jones 2005)	Initial placement	6 – 7

In addition Kavanzanjian et al. (1995) developed a profile to show the relationship between the unit weight of the waste and landfill depth. The profile clearly demonstrates the increasing waste density with increased waste depth and incorporates data from several landfills. The following section provides for calculations to determine the design waste density for the Elizabeth Drive Landfill using the technique developed by Kavazanjian et al 1995 and literature waste data as well as site specific data supplied by SUEZ.

2.9.2 Existing waste density

Existing general waste density

Volumetric surveys, undertaken by SUEZ, indicate that the average global waste density within the general waste cells (filled to 30m) varied between 0.85 t/m^3 and 0.86 t/m^3 (8.7 kN m^{-3}). To be conservative, calculations of the design unit weight were undertaken taking into account daily cover (the majority daily cover is stripped prior to placement of new waste lift) final capping soils, leachate recirculation, waste decomposition and the additional waste to be placed as part of the expansion.

Existing restricted waste density

Volumetric surveys, undertaken by SUEZ in 2019, indicate that the average global waste density within the restricted waste cells (filled to 30m) varied between 0.98 t/m^3 and 1.18 t/m^3 (9.6 kN m^{-3} to 11.6 kN m^{-3}). To be conservative, calculations of the design unit weight were undertaken taking into account operational cover, final capping soils, waste decomposition and the additional waste to be placed as part of the expansion.

2.9.3 Design unit weight

General waste cells

Table 2-3 below illustrates the changing waste density with landfill depth to a maximum waste depth of 76 m (the maximum pre-settled general waste depth at the Site). The design unit weight density was calculated using a refuse density of 6 kN/m^3 directly below the cap to 12 KN m^{-3} at the base of the landfill providing an average unit waste density of 10.8 KN/m^3 (1.15 t/m^3). This equates to an average unit waste density of **11.3 KN/m^3** for general waste cells including capping [(1)].

[1] Capping and various cover materials where assumed to compose approximately 9% of the total landfill volume which is comprised of mostly compacted onsite shale material.

The figures are conservative when correlated with measured site data supplied by SUEZ with the average unit weight of waste modelled for the first 30m waste equal to 9 KN/m^3 being slightly above the measured 8.7 KN/m^3 value determined by SUEZ.

Table 2-3 Design unit weight – general waste

Material	Depth (m)	Design Unit Weight		% of Volume
Soil material	Variable	16	kN/m3	9%
Waste (2)	0-10	6	kN/m3	-
	10-20	9.8		
	20-30	11.3		
	30-40	11.8		
	40-50	12		
	50-60	12		
	60-70	12		
	70-76	12		
Waste unit weight (general waste only)		10.8	kN/m3	91%
Design unit weight (incl. soil)		**11.3**	kN/m3	100%

Restricted waste cells

Table 2-4 below illustrates the changing waste density with landfill depth to a maximum waste depth of 58m (the maximum pre-settled restricted waste depth at the Site) as would be applicable to the restricted waste cells. The design unit weight density was calculated using a refuse density of 8 kN/m^3 (representing the more soil like nature of the waste) directly below the cap to 16 KN m^{-3} at the base of the landfill providing an average unit waste density of **13.2 KN/m^3** (1.35 t/m^3) for restricted waste cells including capping [3].

Table 2-4 Design unit weight – restricted waste

Material	Depth (m)	Design Unit Weight		% of Volume
Soil material	Variable	16	kN/m3	16%
Waste	0-10	8	kN/m3	-
	10-20	11		
	20-30	13		
	30-40	14		
	40-50	15		
	50-58	16		
Waste unit weight (restricted waste only)		12.7	kN/m3	84%
Design unit weight (incl. soil)		**13.2**	kN/m3	100%

2.10 Existing leachate collection pipework

The landfill currently includes over 29 cells, sub cells and mono-cells, with four cells (general waste cells F5 and F6, restricted waste cells A9 and A10) remaining to be constructed. The approximate cell locations and their leachate collection system layouts (where known) are illustrated in Figure 2-5.

[2] Waste density assumed to asymptote at 50m depth as per Qian et al (2002). It is noted that due to degradation the waste density may increase post filling but this would be proportional to ongoing settlement and loss of mass due to gas and leachate production, as such the surcharge would not increase.

[3] Capping and various cover materials where assumed to compose approximately 16% of the total landfill volume which is comprised of mostly compacted onsite shale material.

Leachate collection pipe and riser specifications for each Cell are provided in Table 2-5. In addition, Table 2-5 provides the maximum depth of waste over the leachate collection pipes in each cell calculated based on the pre settled landform height.

Cell levels and pipework information has been gathered from various detailed design drawings and specifications, as built survey and CQA reports.

Cell A1 – A3 pipework

Leachate collection pipework in Cell A3 (shown in Figure 2-1) is understood to have been installed above a flat base liner system, rather than in trench. It is assumed that Cells A1 and A2 utilised a similar arrangement.

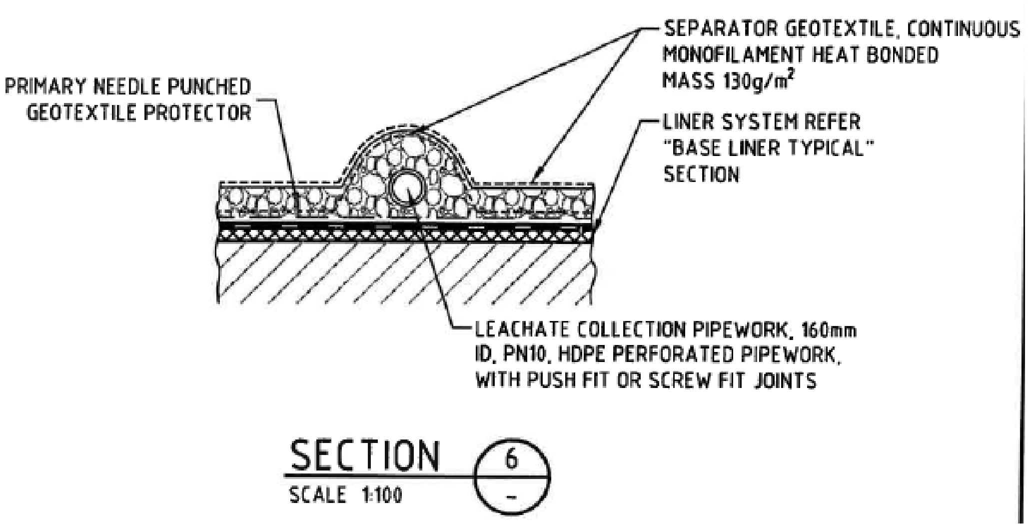


Figure 2-1 Cell A3 leachate collection pipework construction detail

Cell A4 – A8 pipework

The pipework installed in restricted waste cells A4 onwards is perforated DN200 PN16 SDR11 high density polyethylene (HDPE) leachate collection pipes. The pipes within these cells are known to be installed within trenches (shown in Figure 2-2).

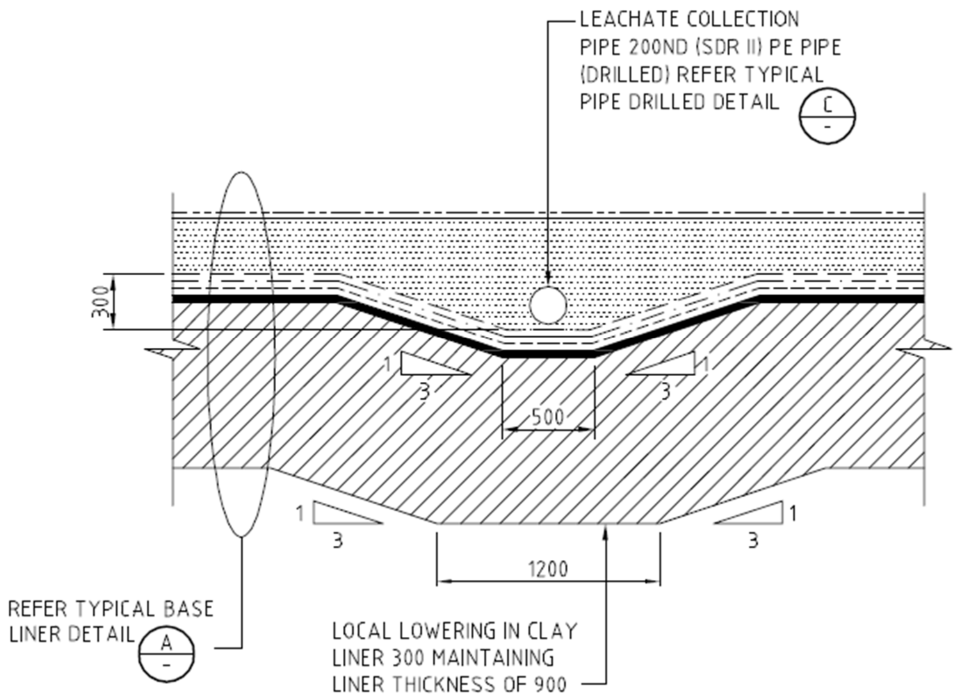


Figure 2-2 Cell A5 leachate collection pipework construction detail

Cells B1-B5, C1-C2 and D1 pipework

The pipework installed in early general solid waste cells were designed with perforated 160 mm diameter PN12.5 SDR11 medium density polyethylene (MDPE) leachate collection pipes laid within trenches and surrounded by filter material (shown in Figure 2-3).

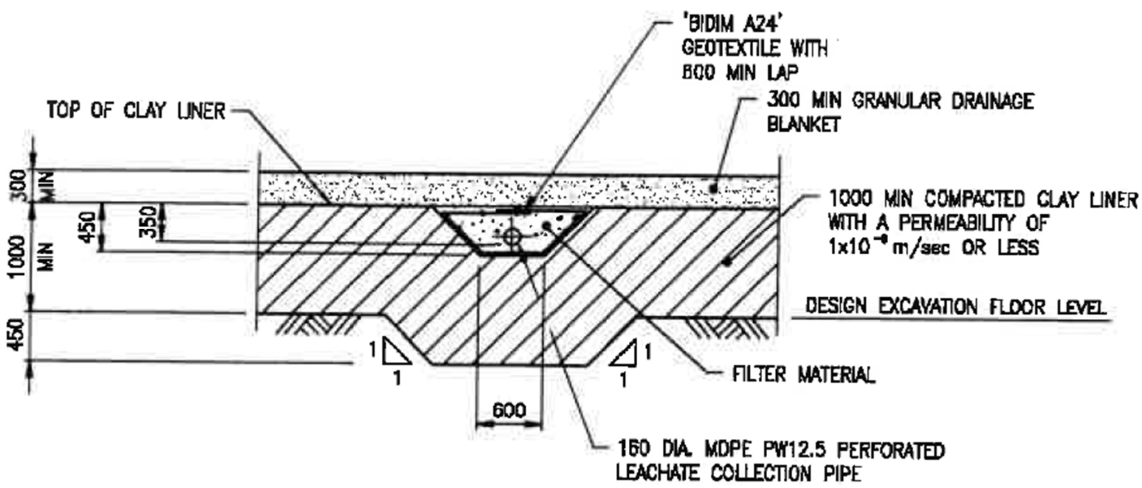


Figure 2-3 Cell B5 leachate collection pipework construction detail

Cell D2, D3, E2-E4, F serries pipework

The pipework installed in general solid waste cells D2, D3, E2-E4 and the F-series cells is perforated DN200 PN16 SDR11 high density polyethylene (HDPE) leachate collection pipes laid within trenches and surrounded by filter material or leachate drainage aggregate (shown in Figure 2-4).

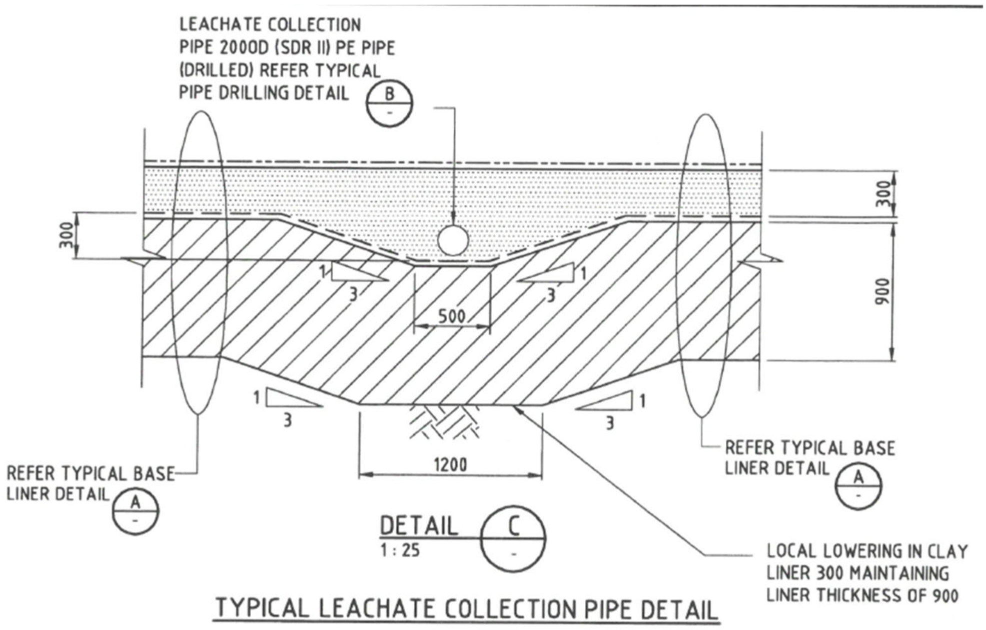


Figure 2-4 Cell D2 leachate collection pipework construction detail

Cell E1 pipework

General solid waste cell E1 utilised DN200 PN10 SDR17 high density polyethylene (HDPE) leachate collection pipes laid within trenches and surrounded by filter material.

Leachate collection layer

A 300 mm of leachate drainage aggregate covers the base of all constructed landfill cells. The leachate collection system has an inbuilt level of redundancy in that, should the leachate collection pipes buckle or become clogged, leachate is still permitted to flow through the continuous gravel drainage blanket and leachate collection trenches.

It is understood that minimal compaction of the gravel aggregate was performed at the time of placement (other than that provided by the construction equipment) though it is considered that due to the heavy landfilling equipment and the overburden pressure of the waste, significant primary and creep settlement has already occurred. It can be assumed that the aggregate surrounding the pipes would now be well-compacted.

0 20 40 60 80 100m
SCALE 1:2000 AT ORIGINAL SIZE

SUEZ RECYCLING & RECOVERY
ELIZABETH DRIVE LANDFILL

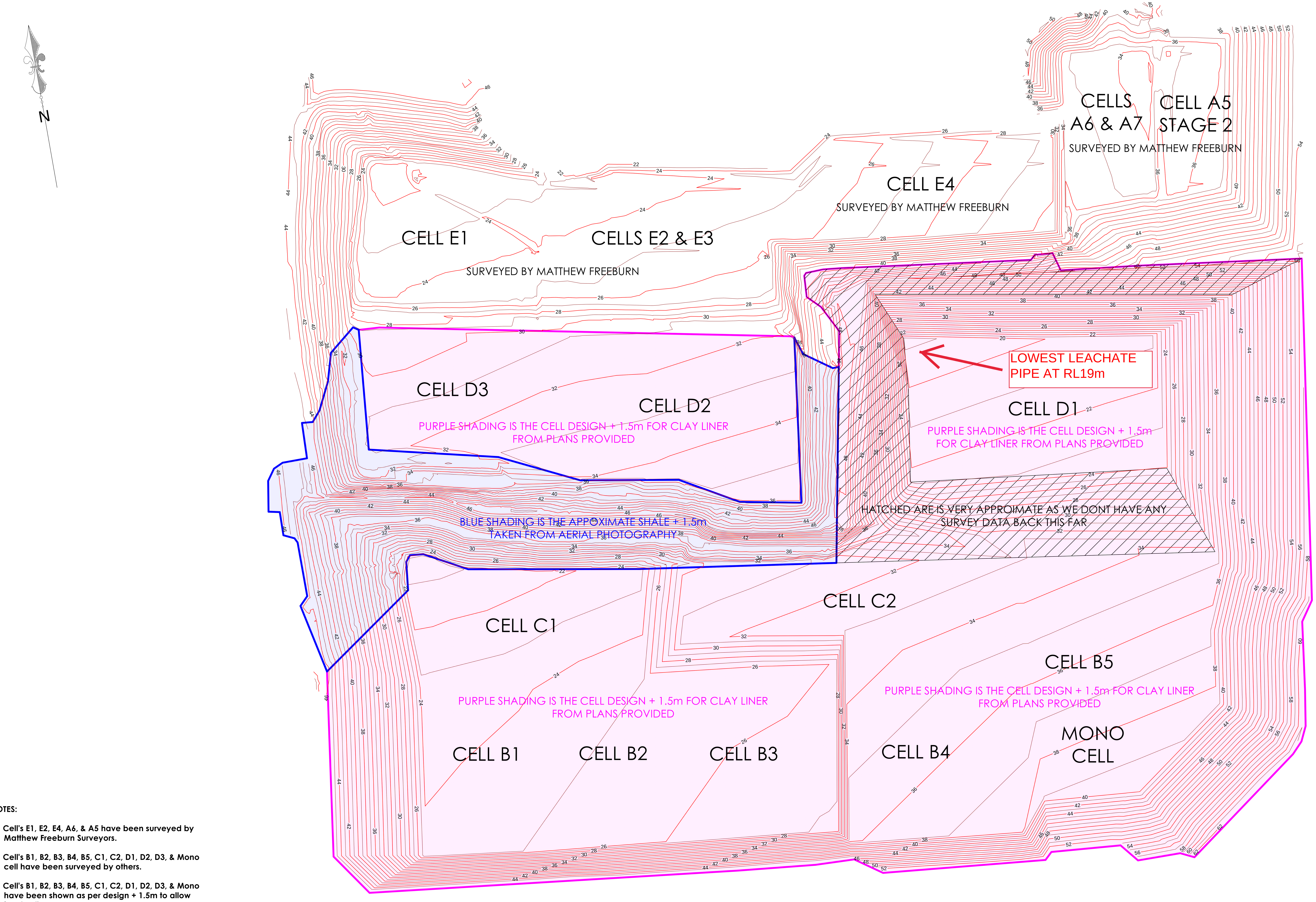
LEACHATE COLLECTION SYSTEM
PIPEWORK LAYOUT

Job Number 2127038
Revision C
Date DEC 2019

Figure 2-5

Level 15, 133 Castlereagh Street, Sydney NSW 2000 Australia T 61 2 9239 7100 F 61 2 9239 7199 E sydmail@ghd.com W www.ghd.com

Plot Date: 20 December 2019 - 3:04 PM Plotted by: Alison Horlyck Cad File No: G:\21\27038\CADD\Drawings\21-27038-FIG7-3.dwg



NOTES:

* Cell's E1, E2, E4, A6, & A5 have been surveyed by Matthew Freeburn Surveyors.
* Cell's B1, B2, B3, B4, B5, C1, C2, D1, D2, D3, & Mono cell have been surveyed by others.
* Cell's B1, B2, B3, B4, B5, C1, C2, D1, D2, D3, & Mono have been shown as per design + 1.5m to allow for clay liner.

Client: SUEZ ENVIRONNEMENT	Project: PLAN SHOWING APPROXIMATE CLAY LINER OVER LOT 1 IN DP 542395 & LOT 740 IN DP 810111 LOCATED AT 1725 ELIZABETH DRIVE KEMPS CREEK.	FREEBURN W E SURVEYING	MATTHEW FREEBURN LAND, ENGINEERING & MINING SURVEYOR SUITE 2, FIRST FLOOR, "SURVEYOR HOUSE" 2 CASTLEREAGH STREET PENRITH 2750	Telephone 02 4721 2289 Fax 02 4721 5646 email matthew@freeburnsurveyors.com	Date: 27/05/2015 Scale 1: 1300 Surveyor: CD Stored: 28727 TOTAL CLAY 15-5-20	Ref: 28727 Datum: AHD Drawn By: CD	Sheet 1 of 1 Contour: 1.0m Checked: MF A1 SHEET

Table 2-5 Leachate pipe details (existing and future cells

Cells			Leachate pipe Type	Min wall thickness (mm)	Min ID (mm)	Max depth over pipe due to proposed landform (m) (4)
Restricted waste cells	Cell A	A1 *	160 ID PN10 HDPE	9.5	140.6	37
		A2 *	160 ID PN10 HDPE	9.5	140.6	37
		A3	160 ID PN10 HDPE	9.5	140.6	37
		A4	200ND (SDR11) PE	18.2	162.4	38
		A5 Stga	HDPE 200 SDR11, PE100	18.2	162.4	36
		A5 Stgb	HDPE 200 SDR11, PE100	18.2	162.4	41
		A6&A7	DN200 (SDR 11) PE Pipe	18.2	162.4	53.5
		A8	DN200 (SDR 11) PE Pipe	18.2	162.4	55
		A9 **	DN200 (SDR 11) PE Pipe	18.2	162.4	58
		A10 ***	DN200 (SDR 11) PE Pipe	18.2	162.4	47
General waste cells	Cell B	B1	160 DIA MDPE PN12.5	14.6	129.9	60
		B2	160 DIA MDPE PN12.5	14.6	129.9	60
		B3	160 DIA MDPE PN12.5	14.6	129.9	60
		B4	160 DIA MDPE PN12.5	14.6	129.9	28
		B5	160 DIA MDPE PN12.5	14.6	129.9	61.6
	Cell C	C1	160 DIA MDPE PN12.5	14.6	129.9	70
		C2	160 DIA MDPE PN12.5	14.6	129.9	63.5
	Cell D	D1	160 DIA MDPE PN12.5	14.6	129.9	75
		D2	200 OD (SDR11) PE Pipe	18.2	162.4	63
		D3	200 OD (SDR11) PE Pipe	18.2	162.4	49
	Cell E	E1	DN200 (PN10) SDR HDPE	11.9	175.7	50
		E2	DN200 (PN16) PE	18.2	162.4	63
		E3	DN200 (PN16) PE	18.2	162.4	66
		E4	DN200 (PN16) PE	18.2	162.4	68.5
	Cell F	F1(A)	HDPE 200 SDR11, PE100	18.2	162.4	52

[4] Includes all waste, capping and cover materials

		F1(B)	HDPE 200 SDR11, PE100	18.2	162.4	36
		F2(A)	HDPE 200 SDR11, PE100	18.2	162.4	66.5
		F2(B)	HDPE 200 SDR11, PE100	18.2	162.4	65.5
		F3(A)	HDPE 200 SDR11, PE100	18.2	162.4	67.5
		F3(B)	HDPE 200 SDR11, PE100	18.2	162.4	67
		F4	DN200 (SDR 11) PE Pipe	18.2	162.4	50
		F5 **	DN200 (SDR 11) PE Pipe	18.2	162.4	65
		F6 **	DN200 (SDR 11) PE Pipe	18.2	162.4	66

* Cell A1-A2 assumed the same as Cell A3

** Design information available - not yet built

*** To be designed. Pipework in accordance with concept design

3. Leachate pipe strength assessment

3.1 General

GHD previously assessed the leachate pipe integrity for the Project Area based on the additional load of the proposed waste with consideration to the as-built cell contours and the proposed final cap of the landform in accordance with the *NSW EPA, 2016, Environmental Guidelines Solid waste landfills* and Australian Standard AS 2566.1- 1998 Buried flexible pipelines – Part 1: Structural design (Standards Australia, Reconfirmed 2018).

The original assessment based on the AS2566.1 determined that the leachate collection pipes may fail due to deflection and buckling though sufficient contingency existed within the leachate gravel drainage blanket to convey the required leachate flows. Based on comments contained within EPA's letter titled *Development Application No. DA19/0470 – Stop the Clock Letter Request for additional information* (dated: 20 November 2019) (henceforth referred to as 'EPA Letter'), the EPA has indicated that a prescriptive interpretation of the requirements in the 2016 Landfill Guidelines (refer section 3.2) is preferred rather than a performance based approach (which is also offered by these Guidelines). With this in mind GHD has reassessed the previous analysis in accordance with the accompanying commentary included with AS2566.1 Supplement 1:1998 Buried Flexible Pipelines Part 1: Structural Design.

SUEZ provided all available as-built details of the leachate pipe network installed at the Site for the purpose of this revised detailed analysis.

3.2 Requirements

The *NSW EPA Environmental Guidelines Solid waste landfills (2016)* specifies that leachate collector pipes should:

- Be flexible pipes (typically high density polyethylene) at least 150 millimetres in internal diameter (water balance and pipe flow calculations should confirm the pipe size needed to convey peak leachate flow rates);
- Be perforated such that the size, frequency and layout of the perforations are sufficient to facilitate leachate inflow and extraction without clogging, prevent entry of drainage gravel, and maintain adequate pipe strength;
- Be strong enough to maintain performance under the maximum loads likely to be imposed in service, complying with the requirements of Australian Standard AS 2566.1- 1998 Buried flexible pipelines – Structural design (Standards Australia, various dates); and
- Be joined by using techniques and materials recommended by the pipe manufacturer.

3.3 Existing leachate collection system

Details of the leachate collection system are provided in Section 2.10.

3.4 Final landform heights and existing stockpile heights

The maximum waste depth occurs at Cell D1 where the proposed final landform (including capping) is 94m RL. The level of the deepest leachate collection pipe within Cell D1, located at the leachate collection sump (PPK drawing 52K038A-6), is at 19m RL. The difference in these heights (75m) represents the maximum static load the leachate collection pipes would be subjected to once landfilling ceases and the cap is installed. At the proposed final landform maximum level of 95m RL, the cell below this location is D2 where the leachate lines and sumps are at 27m RL, with the maximum waste depth of 68m only. Consequently worst case leachate pipe static loading occurs at Cell D1.

3.5 Existing pipe assessment

As part of the assessment an inspection of leachate pipes in cells E1 and D3 via jetting was undertaken by JJ Richards, on behalf of SUEZ. The jetting was able to proceed approximately 400m into each pipe, suggesting that the integrity of existing pipes inspected up to this distance is sound and that they are maintaining their performance.

It is noted that the selected leachate pipes, E1 and D3, currently have soil stockpiles placed as overburden in addition to existing waste. Based on the existing waste and soil stockpile overburden GHD undertook calculations to estimate the current surcharge exerted on the existing leachate pipes. The existing surcharge load was calculated as approximately 565 KN/m^2 and 725 KN/m^2 for pipes E1 and D3, respectively. The value calculated for Cell E1 is equivalent to the to the final landform surcharge value for this pipe (565 KN/m^2) presented in Table 3-2. The value calculated for Cell D3 is slightly higher than the final landform surcharge value (712 KN/m^2) presented in Table 3-2. Although not all pipes could be inspected, those that could be correlated well with the modelling assumptions and results.

3.6 Pipe assessment in accordance with AS2566.1

GHD has reassessed the previous analysis in accordance with the accompanying commentary included with AS2566.1 Supplement 1:1998, relevant parameters based on this guidance have been tabulated in Table 3-1. In particular the commentary in C4.3 notes that the method for calculating the load is conservative as it based on the construction of a prism above the pipe as quoted below (refer Figure 3-1):

“ignores the effects of soil friction within the fill above the pipe, but has been adopted because of its simplicity and because it gives conservative values.”

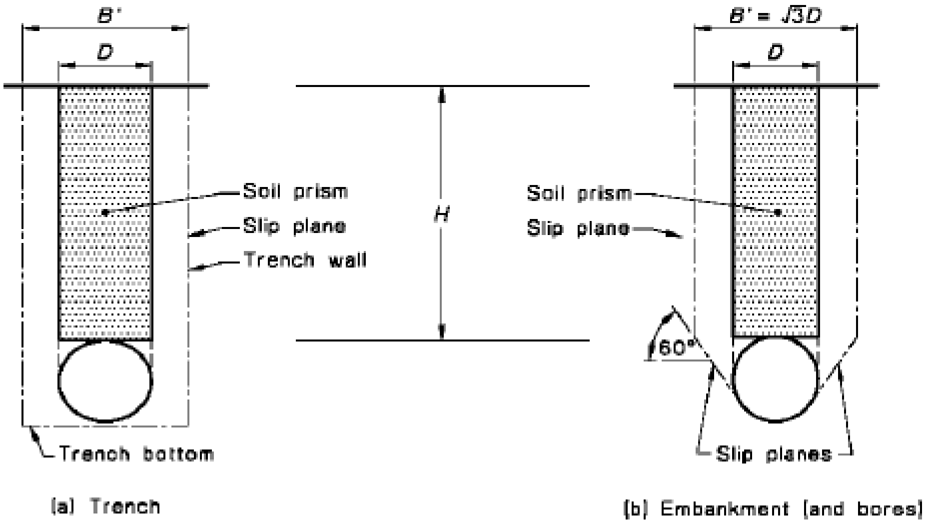


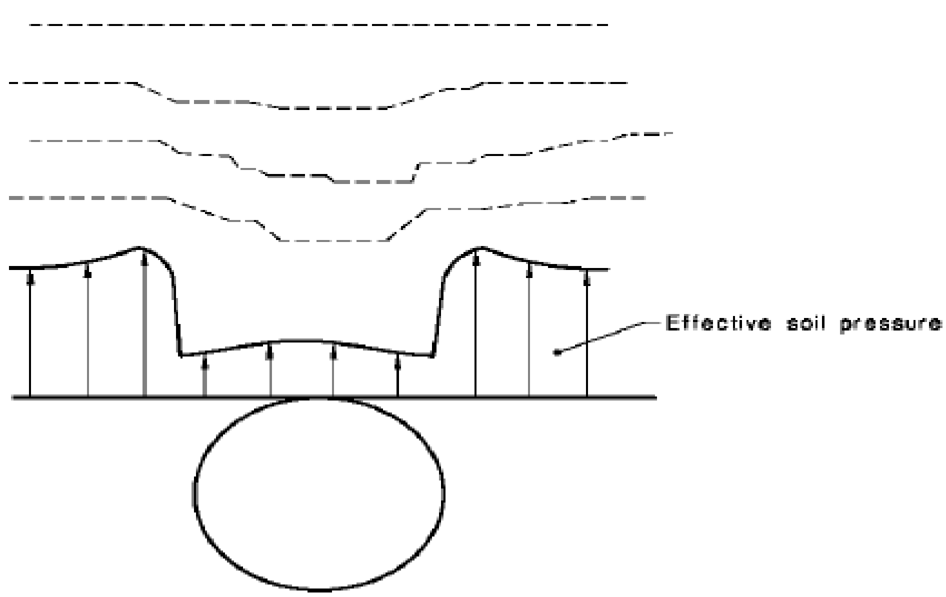
Figure 3-1 Soil prism loading and slip planes

This is particularly relevant to pipework at the base of a landfill which has a significant height of cover that would be subject to soil arching (refer Figure 3-2). With respect to this commentary, AS2566.1 does not account for unusually large cover heights such as at landfills. In such cases, AS2566.1 Section 5.1 notes that independent assessment using other methods should be used for evaluating vertical loading.

> ***"Independent assessment should be made of conditions that fall outside the scope of this Standard, such as non-uninform embedment material and density and three-dimensional effects due to groundwater, settlement or foundation movement, joint requirements, abnormal loadings, and unusually large cover heights such as embankments in tailings dams and waste dumps"***

AS2566.1

Based on this guidance GHD has used the conservative approach in the AS2566.1 as an initial screening tool to identify critical pipework. This critical pipework was then further assessed utilising a 2D Plaxis finite element analysis to provide a more realistic assessment of the forces and strains surrounding the pipework taking into consideration the pipework configuration, location within the landfill and arching within the waste overburden.



NOTE: Arching effect in soil strata above flexible pipe reduces soil pressure on pipe.

FIGURE C4.2 ARCHING EFFECT

Figure 3-2 Arching effect

3.6.1 Influence of pipe perforations

The circular perforations within the leachate collection pipes are not expected to have a significant influence on the integrity of the pipes (buckling etc) due to the compacted drainage aggregate supporting the pipes. Stresses exerted on the pipes from the waste mass are likely to be transferred around the holes in much the same way that stresses are transferred around the rock mass of a tunnel.

3.6.2 Summary of modelling parameters

Table 3-1 summarises the relevant parameters used in the modelling based on this guidance provided in:

- Poliplex Polyethylene pipe Design Textbook (James Hardie Pipelines 1997)
- AS/NZ 2566.1 Supplement 1:1998 Buried Flexible Pipelines Part 1: Structural Design – Commentary
- AS/NZ 2566.1:1998 Buried Flexible Pipelines Part 1: Structural Design

Table 3-1 Summary of modelling parameters in accordance with AS 2566.1

Characteristic	Data Source	Cell A1-A3	Cell A4	Cell A5-A10	Cell B1-B5	Cell C1-C2	Cell D1	Cell D2-D3	Cell E1	Cell E2-E4	Cell F1-F6
Initial (3-minute) ring bending modulus of elasticity (MPa)	Poliplex design book p 7-46 and 3-13 (5)	880	880	880	650	650	650	880	880	880	880
Long-term ring-bending modulus of elasticity (2-year [6]) (MPa)	Poliplex design book p 7-46 and 3-13 (7)	303	303	303	247	247	247	303	303	303	303
Native soil modulus (MPa)	Table 3.2 AS2566.1	15 (8)	10	10	10	10	10	10	10	10	10
Embedment soil modulus (MPa)	Table 3.2 AS2566.1	15	15	15	15	15	15	15	15	15	15
Unit weight of fill (kN/m^3) (9)	Section 2.9.3 of report	13.2	13.2	13.2	11.3	11.3	11.3	11.3	11.3	11.3	11.3
Allowable long-term vertical pipe deflection for non-pressure (%)	Table 2.1 AS2566.1	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
Allowable long-term ring-bending strain (%)	Table 2.1 AS2566.1	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0

[5] Corrected for 25°C (assumed initial temperature within cell)
[6] 2-year value utilised as per guidance in Section 5.1.2 of AS2566.1
[7] Corrected for 28°C (assumed long-term operational temperature within leachate collection system)
[8] Trenching of pipe work was not undertaken in these cells. Therefore the native soil component of the side support (within the zone of influence) is taken to be the leachate drainage blanket material
[9] It is noted that the unit weight modelled is the maximum density for each waste type. Most pipes would not be subjected to the maximum waste density.

Factor of safety for long-term combined external load and internal pressure (combined loading)	Table 2.1 AS2566.1	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Factor of safety for long-term internal pressure	Table 2.1 AS2566.1	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
Factor of safety for long-term ring-bending strain	Table 2.1 AS2566.1	2	2	2	2	2	2	2	2	2	2
Height of water surface above the top of the pipe (m)	Maximum height as per guidelines (o.3m) including a FoS of 2.5	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Design factor for buckling	Section 5.4 of AS2566.1 [10]	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

[10] FOS assumed as 2 as the consequence of localised buckling failure is not critical due to the pipework being contained within a leachate collection blanket

3.6.3 Results of pipework assessment in accordance with AS2566.1

Results of the pipework assessment in accordance with AS2566.1 have been tabulated in Table 3-2. The results show that worst case defection, 7.7%, would occur in Cell D1, which is slightly above the required maximum of 7.5%. As per AS2566.1 (refer Section 3.6) further analysis using finite element analysis modelling of the deflection performance of the pipework in Cell D1 was undertaken (refer Section 3.7).

Table 3-2 Pipe strength calculations results summary

Location	Pipe size	Load depth	Waste density	Overburden load	Pipe class	Strain %	Deflection %	FoS against buckling
Cell	mm	m	kN/m3	kPa		(<4)	(<7.5)	(>2)
A1-A3	160	37.00	13.2	490	PN10	1.4	5.1	2.33
A4	200	38.00	13.2	503	SDR11	1.6	4.4	3.63
A5-A10	200	58.00	13.2	768	SDR11	2.4	6.7	2.39
B1-B5	160	61.60	11.3	696	PN12.5	2.3	6.3	2.46
C1-C2	160	70.00	11.3	791	PN12.5	2.6	7.2	2.17
D1	160	75.00	11.3	859	PN12.5	2.8	7.7 (11)	2.00
D2-D3	200	63.00	11.3	712	PN16	2.2	6.2	2.58
E1	200	50.00	11.3	565	PN10	1.7	5.8	2.02
E2-E4	200	68.50	11.3	774	PN16	2.4	6.8	2.37
F1-F6	200	67.50	11.3	763	PN16	2.4	6.7	2.41

3.7 Pipe assessment utilising 2D Plaxis

PLAXIS 2D is a special purpose finite element package intended for two-dimensional analysis of deformation and stability in geotechnical engineering. PLAXIS is used worldwide by engineering companies and institutions in the civil and geotechnical engineering industry. PLAXIS is equipped with a broad range of advanced features to model a diverse range of geotechnical problems. PLAXIS uses predefined structural elements and loading types in a CAD-like environment.

PLAXIS 2D uses finite element analysis to compute highly non-linear geotechnical problems with greater reliability than the calculation methods achieved in the standards. Finite element analysis can provide comprehensive understanding of failure modes as well as locating the areas of failure which traditional analytical methods cannot achieve

[11] Pipework selected for further analysis in accordance with AS2566.1

GHD note that 2D Plaxis is an industry recognised software and has previously been used in the environmental approval of Whytes Gully Resource Recovery Park landfill expansion as a detailed assessment tool of liner settlement.

3.7.1 PLAXIS modelling approach

The modelling undertaken in Section 3.6 in accordance with AS2566.1 identified the pipework in Cell D1 as requiring further analysis due to a possible excessive deflection. To further assess this pipework two PLAXIS finite element models were developed:

1. Leachate collection pipes located within the centre of the cell
2. Leachate collection pipes located on the toe of the batter

For each scenario, the revised assessment uses 2D PLAXIS to assess forces and strains acting on a 160 MDPE leachate pipe resulting from differential settlement of the existing landfill under the overburden of the proposed new landfill materials. The modelling parameters are summarised in Table 3-1. The modelling is summarised in Appendix B.

3.7.2 Deflection criteria

Figure 3-3 shows the deformed pipe shape under the additional surcharged loads. The revised Plaxis analysis shows the resultant deflections for the leachate pipes in Cell D1 in the most critical case which calculate to a maximum deflection of 3.54%. This complies with the deflection limits set in the standards by a factor of over 2.

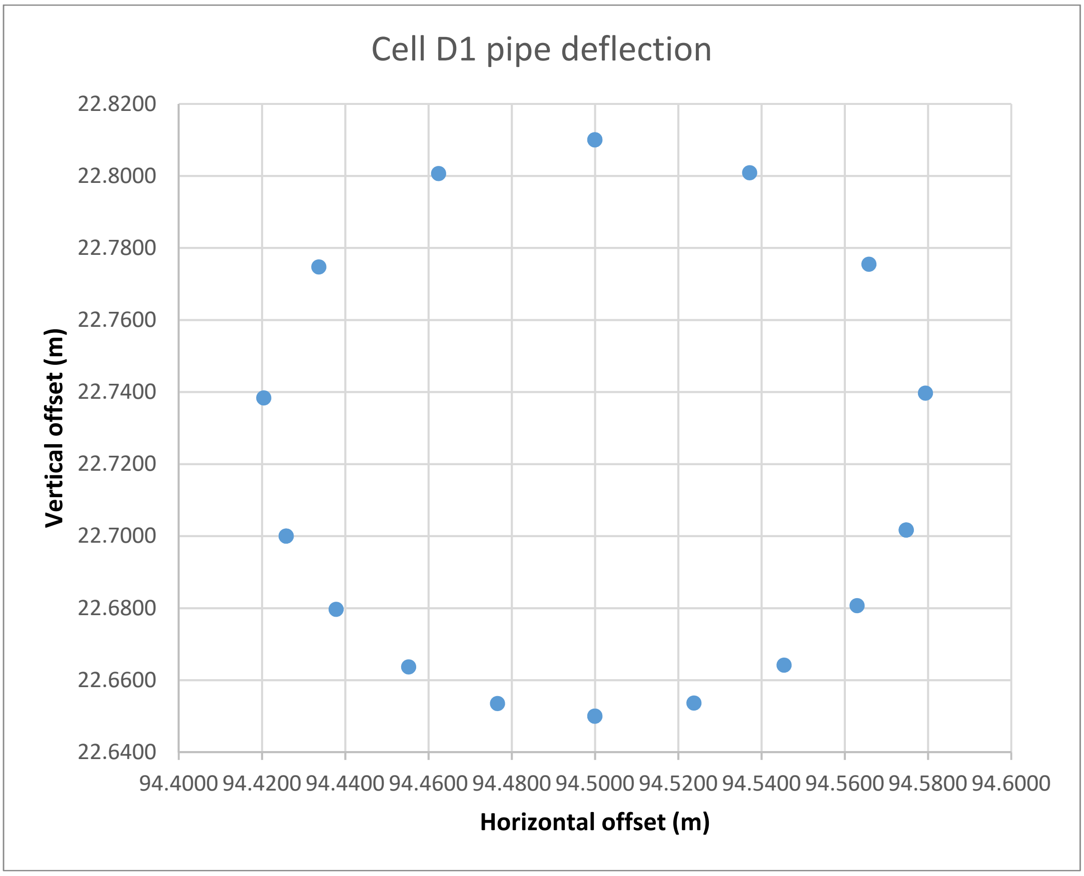


Figure 3-3 Deflected MDPE pipe shape due to vertical stresses

3.8 Leachate pipe strength conclusions

Pipe strength integrity assessments were performed on all pipework in accordance with AS2566.1 (refer Table 3-2). Except for Cell D1 all pipework met the required criteria. The pipework in Cell D1 required further analysis due to a calculated deflection of 7.7% (slightly above the required 7.5%). Due to the conservatism in the AS2566.1 (refer Section 3.6) further analysis using finite element analysis modelling of the deflection performance of the pipework in Cell D1 was undertaken (refer Section 3.7). The results showed that, taking into consideration arching within the waste and the trench configuration, the resultant deflection on the pipework in Cell D1 was only 3.54% which is well within the required pipework criteria (less than 7.5%).

Based on the results of the pipe strength integrity assessment, the LCS should not be adversely affected by the proposed additional fill associated with the Proposed Final Landform Contours.

Further, it is noted that due to the redundancy within the pipe flow volumes, provision of leachate collection trenches, and a continuous leachate collection gravel drainage blanket constructed across the entire floor of each cell, sufficient contingency exists in the LCS to provide a suitable factor of safety should localised failures occur.

3.9 Gas collection pipes

A quantitative integrity assessment of the gas collection pipes has not been undertaken as part of this report. GHD note that gas collection at the site is undertaken by the installation of vertical wells connected to an active gas extraction system. As the installation of vertical gas wells by extending through waste is standard practice at a landfill, retrofitting of additional wells as required would be undertaken as the landfill waste profile reaches final capping height.

Hence the gas extraction at the site would not be adversely affected by the proposed additional waste fill, associated with the proposed final landfill contours (maximum 95m RL).

3.10 Drainage geocomposite

The drainage geocomposite specified for the project did not have a compression requirement however the specification did require that it be constructed with a triaxial geonet.

Triaxial geonets typically have much higher compressive strengths than biaxial and higher than required by the additional waste mass so based on the design as-built information it is likely that the product installed would be suitable for the additional waste mass.

3.11 Protection geotextile

Based on a review of the previous design documentation prepared for the A3-A5 restricted waste cells by PB and Maunsell, a protection geotextile with a minimum mass of 800 g/m^2 was specified. Calculations undertaken to assess the required protection geotextile mass for the additional waste mass in accordance with (refer Appendix C Protection Geotextile Design Procedures):

- Designing with Geosynthetics (5th Edition), Robert M. Koerner
- Barrier Systems for Waste Disposal (2nd Edition) Rowe et al

The calculations resulted in a protection geotextile requirement of a minimum mass of 730 g/m^2.

Cells A6 onwards have a suitable protection geotextile and were subjected to conforming compression testing.

Since the specified geotextile exceeds the calculated minimum mass and compression testing, the protection geotextile is considered suitable to accommodate the addition waste load.

3.12 Leachate Collection system (LCS) conclusions

Based on the leachate pipe strength integrity assessment (undertaken in accordance with AS 2566.1 and PLAXIS 2D modelling) provided in this section, the following conclusions were made:

- The installed and proposed leachate collection pipes in general and restricted waste cells should not be affected by the proposed additional waste fill load associated with the proposed final landform elevation (maximum RL 95m)
- The strength of the installed and proposed leachate collection pipes in general and restricted waste cells is considered suitable to maintain performance of the LCS. The modelling supports that the required design safety factors would be achieved for two pipe failure modes, buckling and deflection, in accordance with 2016 landfill guidelines
- The drainage triaxial geocomposite already instated (existing cells) and proposed to be installed (new cells) would be suitable to accommodate the additional waste load
- Protection geotextile already installed (existing cells) and proposed to be installed (new cells) would suitable to accommodate the additional waste load as the geotextiles exceed the minimum mass criteria

The leachate pipe strength assessment as detailed in this section indicates that the LCS at the Site is expected to satisfy long term performance criteria under the proposed additional waste fill load as part of the proposed expansion.

4. Final cap design (revised)

4.1 Overview

An alternative final cap design was originally submitted to the EPA for consideration. GHD revised the final cap design to comply with recommendations contained within the second EPA letter (ref DOC 19/1012793 dated 20 November 2019).

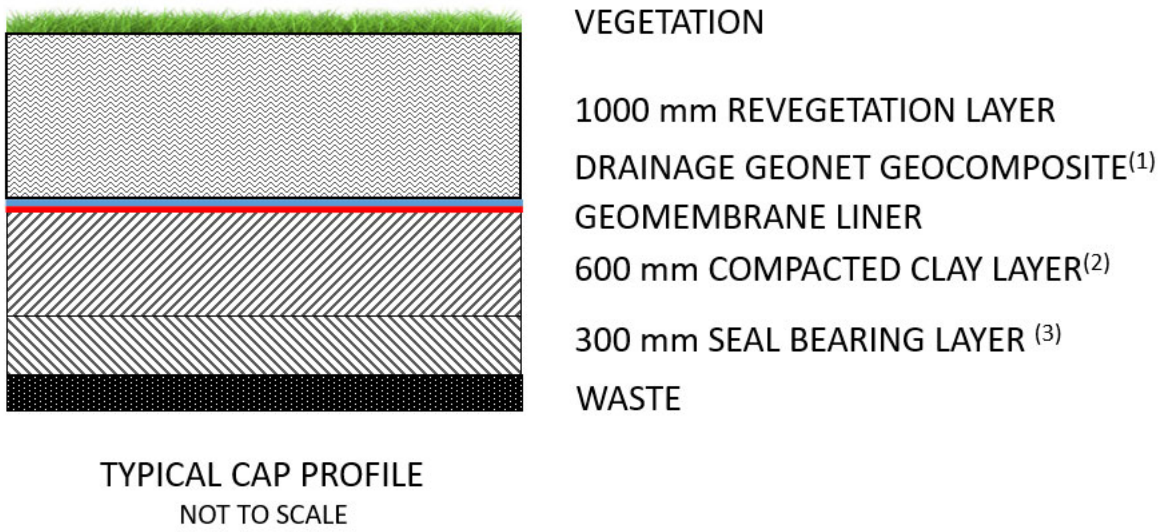
4.2 Revised final cap

The proposed final cap design is in accordance with the *NSW EPA Environmental Guidelines: Solid Waste Landfill, Second edition 201*6 (2016 Guidelines). The proposed final cap comprises the following layers (bottom to top):

- 300 mm thick seal-bearing layer; the material should meet recognised specifications for engineered materials, such as QA Specification 3071: Selected Material for Formation (NSW Roads and Maritime Services, December 2011), as amended time to time
- A 600 mm thick sealing layer, comprising of a compacted clay layer, with an in situ saturated hydraulic conductivity of less than 1 x 10^{-9} m/s
- LLDPE geomembrane liner
- Drainage geonet geocomposite (subsurface drainage layer)
- 1000 mm thick revegetation layer; the upper 200 mm should be a topsoil layer.

The typical cap profile is shown in Figure 4-1.

GHD note that Section 9.3 of the 2016 Guidelines allows for alternative final cap designs to be proposed and provides detailed requirements. Should an alternative cap be proposed as part of detailed design it is recommended that it meet the requirements of Section 9.3 of the 2016 Guidelines and the outcomes of this report.



(1) SUBSURFACE DRAINAGE LAYER WOULD BE INCLUDED IN CERTAIN GENERAL WASTE CELL AREAS (BATTERS) AND ALL RESTRICTED WASTE CELLS. LAYER MAY BE SUBSTITUTED FOR A GRAVEL DRAINAGE LAYER. DETAILS TO BE DETERMINED IN THE DESIGN FOR CONSTRUCTION

(2) COMPACTED CLAY LAYER MAY BE SUBSTITUTED FOR A GEOSYNTHETIC CLAY LAYER SUBJECT TO DETAIL FOR CONSTRUCTION

(3) SITE HAS AN ACTIVE GAS COLLECTION SYSTEM, GAS COLLECTION TRENCHES UNDERNEATH SEAL BEARING LAYER ARE NOT REQUIRED

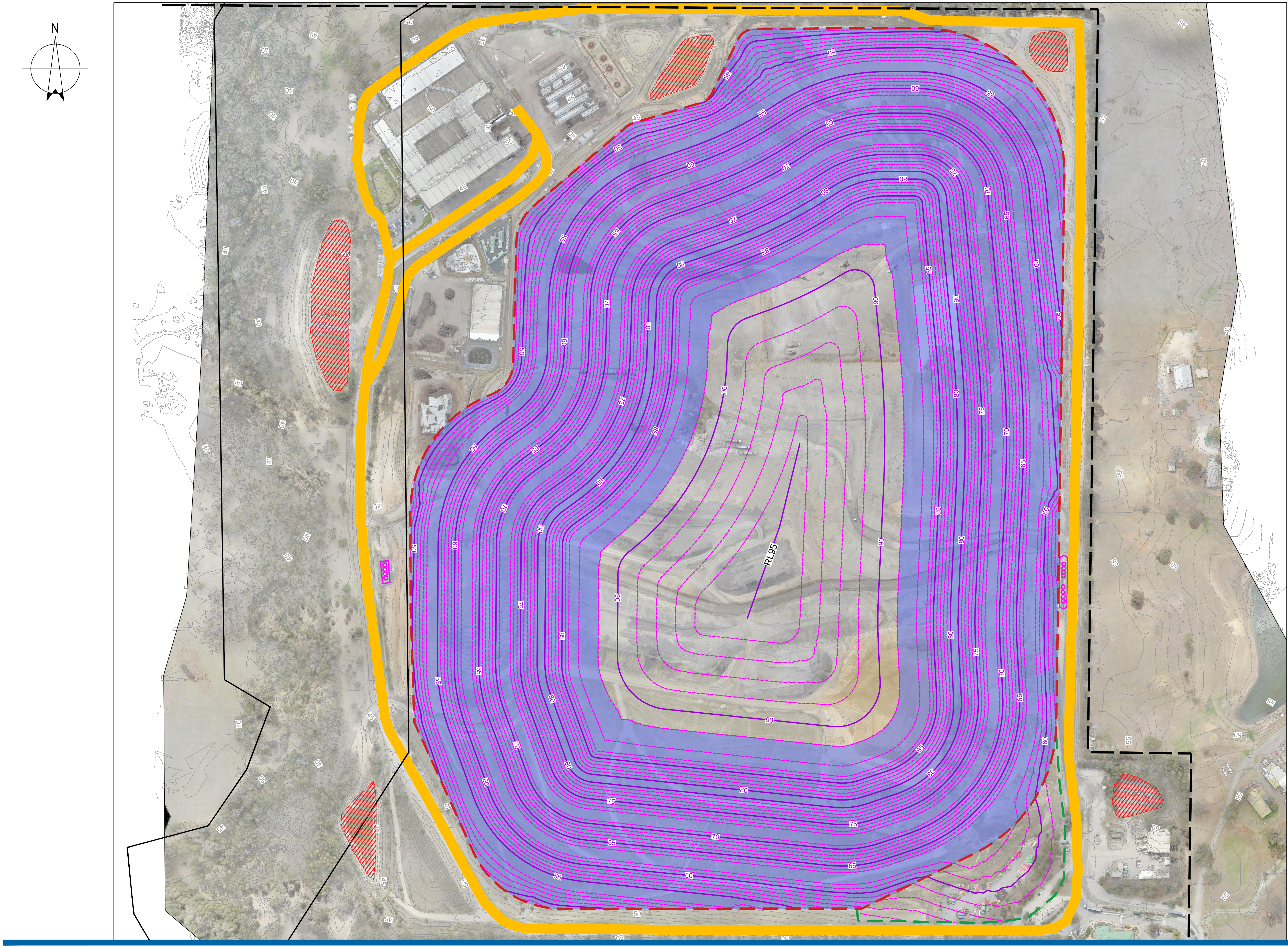
Figure 4-1 Typical Cap profile

An adjustment to the EPL (licence) will be required, to reflect these changes to the cap design.

4.3 Subsurface drainage layer

The original veneer stability assessment identified the case with parallel seepage (caused by the lack of a subsurface drainage system) to be below the target Factor of Safety. Consequently the revised final cap design would utilise a subsurface drainage layer. The subsurface drainage layer (drainage geonet geocomposite) would be included on all batters of the general waste cell and over the entire restricted waste cell. An indicative illustration identifying the likely areas that would include a subsurface drainage layer are shown on Figure 4-2.

The specification of subsurface drainage layer within the general waste cells would be determined during detailed design for the construction stage.



LEGEND:

- 20 — EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- 20 — PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- INDICATIVE EXTENT OF SUBSURFACE DRAINAGE LAYER

0 20 40 60 80 100m
SCALE 1:2000 AT ORIGINAL SIZE

GHD

SUEZ RECYCLING & RECOVERY
ELIZABETH DRIVE LANDFILL

INDICATIVE SUBSURFACE DRAINAGE LAYER EXTENT

Job Number	2127038
Revision	A
Date	DEC 2019

Figure 4-2

Level 15, 133 Castlereagh Street, Sydney NSW 2000 Australia T 61 2 9239 7100 F 61 2 9239 7199 E sydmail@ghd.com W www.ghd.com

Plot Date: 16 December 2019 - 10:17 AM Plotted by: Alison Horlyck Cad File No: G:\21\27038\CADD\Drawings\21-27038-FIG7-2.dwg

5. Slope stability assessment

5.1 Overview

GHD (2019) undertook a slope stability assessment for the previously proposed final cap design (assessed final cap design). Based on the conclusions of the assessment, it was recommended that a thicker final capping design incorporating sub-surface drainage in accordance with the 2016 Landfill Guidelines be considered to increase veneer stability. In conjunction with guidance provided in the second EPA Letter (ref DOC 19/1012793 dated 20 November 2019), a revised final cap design that conforms with the 2016 Landfill Guidelines is proposed (refer Section 4.2).

The global slope stability (refer Section 5.3) assessment undertaken for the assessed final cap design is considered relevant despite the proposed revision to the final cap design as the total depth and mass of the combined waste and cap does not change. Alterations to the assessed final cap design would have negligible impacts on the global slope stability analysis however would affect the veneer slope stability. Therefore, the existing global stability assessment is considered to adequately address the landfill stability.

An updated veneer slope stability assessment (refer Section 5.4) was undertaken for the revised final cap design. The revised capping system is significantly thicker than that previously proposed and consequently the capping systems stability would be an improvement over the original proposed design. The original veneer stability assessment noted the case with parallel seepage (caused by no subsurface drainage system) is below the target Factor of Safety. The primary cause is a loss of cohesion due to saturation at the interface. The revised final cap design would include a subsurface drainage layer across the entire restricted waste cell and certain areas (batters) of the general waste cell (refer Section 4.3 for more detail), with the exact extent to be determined by detailed design for the construction stage in accordance with 2016 Landfill Guidelines.

5.2 Slope stability assessment

GHD has undertaken a slope stability assessment for the proposed final landform to assess if the overfill will create any instability within the new fill, the existing waste, at the interface with the existing waste or within the proposed cap profile.

The proposed batter profile for the final landfill formation is generally uniform on all sides and is to comprise five slopes (typically at 1V:3.5H pre-settlement) separated by 10m wide slope benches. The slope benches are graded inwards to a swale drain at the toe of the higher slope batter thereby controlling stormwater flows to within the drainage system and preventing greater stormwater flows down the steeper slope batters.

Noting the generally flat nature of the surrounding landform and the uniform nature of the proposed slope batters, a single worst case slope batter section was analysed; see Figure 5-1.

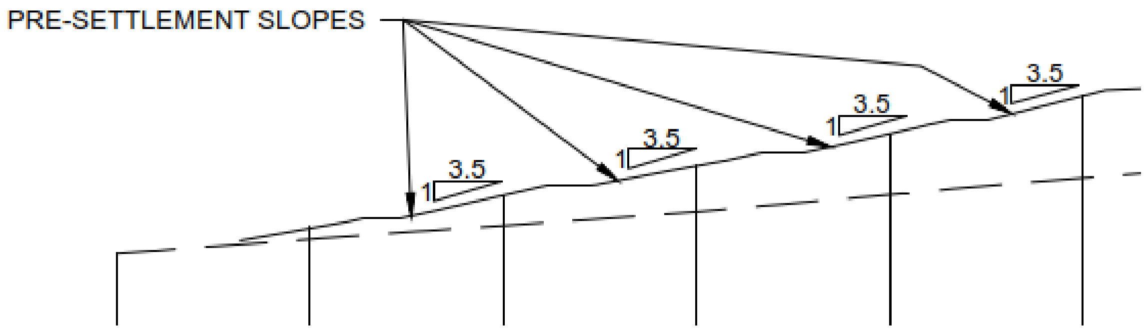


Figure 5-1 Typical side slope design (extracted from Section 3.1 of the Concept Design Report)

5.3 Global slope stability analysis

The slope stability analysis has been carried out using the commercially available two dimensional limit equilibrium software ‘Slope/W’ by Geosolve Limited. This software package is one of a number of industry standard slope stability software packages which have been validated and approved for use by GHD’s geotechnical service line.

5.3.1 Input data for slope stability analysis

The limit equilibrium analysis of soil slopes requires defined ground model geometries (including leachate or groundwater levels / pore pressures) and geotechnical parameters for the landfill and underlying natural or other materials (bulk unit weight and shear strength parameters).

The stability analysis has been carried out using:

- Characteristic values for the proposed clay capping material and underlying Bringely Shales; and
- A range of effective shear strength parameters for the landfill material, in recognition of the inherent variability of this type of material.

With reference to Figure 5-2 for municipal solid waste (“MSW”), there is a broad range of effective stress shear strengths recommended for use in slope stability analysis.

Although it is noted that the waste streams landfilled at the Site comprise general solid waste, the shear strength values adopted by in this analysis are towards the lower bound of the envelope highlighted in Figure 5-2. They are considered to be realistic and moderately conservative with respect to the type of waste streams received at the Site.

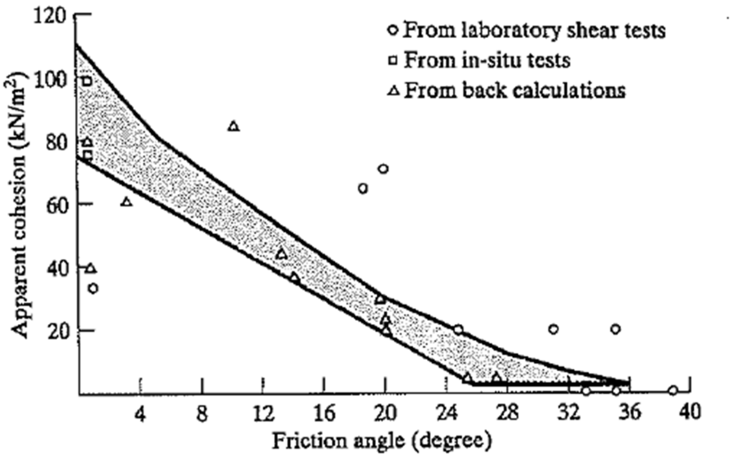


Figure 5-2 Summary of municipal solid waste strength data[12]

The bulk unit weight of waste in landfills is also highly variable due to the range of waste types, moisture content, placement procedures and other environmental and site specific factors. A bulk unit weight value of 11kN/m^3 has been adopted for this assessment, which is consistent with the unit weight characteristic of MSW where the level of compaction is "good".

A summary of the material parameters adopted for this stability analysis are shown in Table 5-1.

Table 5-1 Material properties adopted for slope stability analysis

Material	Unit weight (kN/m3)	Effective friction angle (°)	Effective cohesion (kPa)
New landfill	11.3	25	0
		20	20
		12	40
Existing landfill	11.3	30	0
Residual soil	20	28	5
Capping layer	20	30	8

In relation to the cross sections analysed, groundwater / leachate levels have been assumed to be towards the base of the landfill based upon the assumption that leachate will be captured by the leachate collection system installed at the base of the landfill. It is also anticipated that infiltration and surface runoff following rainfall events will not permeate through the final landform capping in significant volumes, and will be captured and diverted from the landfill by the surface drainage system.

5.3.2 Earthquake and traffic loading

The stability assessment was carried out at each of the five proposed benches initially without a traffic load applied, and then with an assumed nominal uniform 10kPa traffic load which was distributed partially over the respective benches and up-slope from the slope bench being analysed (refer to Appendix D for details of the analyses).

[12] Extracted from Qian, X, Koerner, R & Gray, D. (2001), *Geotechnical Aspects of Landfill Design and Construction*, Prentice Hall, Sydney

A pseudo-seismic earthquake analysis was also conducted with the most critical scenarios (with and without traffic loading) by applying a horizontal seismic acceleration coefficient of 0.05 to model the effects of a seismic event.

5.3.3 Slope stability analysis results

A range of analyses were carried out on the typical cross-section to assess the sensitivity to varying combinations of material input parameters and loading scenarios. A minimum calculated factor of safety (FoS) of 1.5 was deemed appropriate for long term stability based on local geotechnical practice, though this was lowered to 1.2 for the addition of seismic loading due to its transient nature.

The results of the slope stability analysis carried out are summarised in Table 5-2**.**

Table 5-2 Summary of slope stability analysis results for different scenarios

Case	Traffic load (kPa)	New landfill parameters		Critical FoS	Critical FoS (Seismic)
		Effective friction angle (°)	Effective cohesion (kPa)		
1	0	25	0	2.23	1.84
2		20	20	3.61	-
3		12	40	4.11	-
4	10	25	0	2.16	1.83
5		20	20	3.21	-
6		12	40	3.71	-

The results of the slope stability analyses indicates that the proposed final landfill landform batter slopes analysed are stable under the revised final landfill cap construction, for all cases including the cases with plant surcharge load applied.

In order to ensure safe working practises, it is recommended that no large stockpiles of capping (or other) materials be placed within 30m of the slope crests.

5.4 Veneer stability of the capping system

The veneer stability of the final capping system was developed to provide a preliminary evaluation of interface strength of the capping layer materials. The veneer stability assessment was undertaken with consideration to the revised final cap design as summarised in Figure 4-1 and discussed in Section 4.2. The revised final cap design offers improved veneer stability due to its increased thickness compared with the originally proposed final cap design.

The veneer stability assessment assesses a range of failure mechanisms and destabilising forces and provide basis for proposing suitable layers in the capping system. Particularly, it is used to confirm the need of a subsurface drainage layer for rainwater infiltration, which will be overlying the sealing layer.

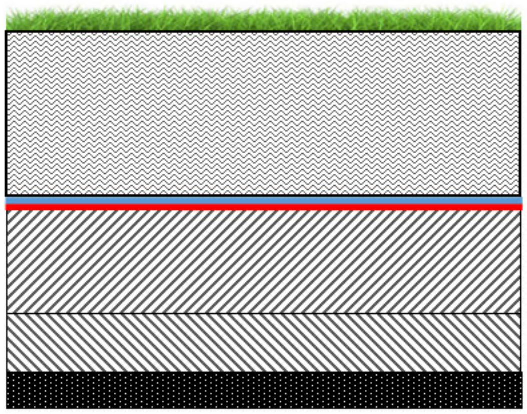
Detailed assessments are attached in Appendix E.

5.4.1 Selection of parameters

The assumptions and parameters outlined in this Section were adopted for analysis of the veneer stability at the site. They are considered to be reflective of the expected properties of the materials to be used in the capping works

Revised final cap system

The proposed revised final cover system consists of the layers and material thicknesses as summarised in Figure 5-3 below.



TYPICAL CAP PROFILE
NOT TO SCALE

(1) SUBSURFACE DRAINAGE LAYER WOULD BE INCLUDED IN CERTAIN GENERAL WASTE CELL AREAS (BATTERS) AND ALL RESTRICTED WASTE CELLS. LAYER MAY BE SUBSTITUTED FOR A GRAVEL DRAINAGE LAYER. DETAILS TO BE DETERMINED IN THE DESIGN FOR CONSTRUCTION

(2) COMPACTED CLAY LAYER MAY BE SUBSTITUTED FOR A GEOSYNTHETIC CLAY LAYER SUBJECT TO DETAIL FOR CONSTRUCTION

(3) SITE HAS AN ACTIVE GAS COLLECTION SYSTEM, GAS COLLECTION TRENCHES UNDERNEATH SEAL BEARING LAYER ARE NOT REQUIRED

Figure 5-3 Typical Cap profile

Critical interfaces

Based on experience and assessment of relevant literature, the most critical interface within the final cap system is the drainage geonet geocomposite / revegetation layer interface.

Surface slope

The surface slope of the proposed final landform surface is to be 1V :3.5H (~ 15.3°) pre-settlement.

Veneer stability variables

Table 5-3 Veneer stability adopted variables

Veneer stability variable	Adopted value	Comment
Unit weight of revegetation soil	18 kN/m3	
Peak friction angle of revegetation soil	32°	Based upon shear testing of soil samples
Residual friction angle of revegetation soil	24°	Based upon an interface friction angle efficiency of 80%
Residual cohesion of revegetation soil	0 kPa	

5.4.2 Analysis method

The calculations below are based on the methods outlined by Qian et al (2002). Three conditions were assessed with respect to the veneer slope stability, as follows:

- Veneer stability with no seepage
- Veneer stability with horizontal seepage
- Veneer stability with parallel seepage.

5.4.3 Analysis of veneer stability results

The veneer stability assessment identified the case with no subsurface drainage system on the batters as below the target Factor of Safety. The primary cause is a loss of cohesion due to saturation at the interface.

Based on the conclusions of the veneer stability assessment, the revised final cap design has been updated to include a subsurface drainage layer on the batters underlying the revegetation layer to collect the infiltrated rainwater. The revised analysis, including a sub-surface drainage layer, resulted in a suitable factor of safety. Further details of the subsurface drainage layer are discussed in Section 4.3.

6. Conclusions

6.1 General

This report provides technical assessment and advice in response to the EPA’s second letter (ref DOC 19/1012793 dated 20 November 2019) request for additional information.

In particular, this report provides further clarification and assessment regarding:

- A revised pipe loading assessment in accordance with the AS2566.1 Buried Flexible Pipelines. Based on the outcomes of the AS2566.1 analysis , critical pipework was further assessed utilising 2D PLAXIS to take into account arching within the waste and support from the bedding gravel
- A revised final capping design
- A slope stability analysis, assessing the stability of the side slopes of the proposed new landform.

The principal conclusions from the pipe strength, slope stability and final cap design assessments are summarised below.

6.2 Conclusions

Pipe strength assessment

The leachate pipe strength integrity assessment (undertaken in accordance with AS 2566.1 and PLAXIS 2D modelling) concluded that installed and proposed leachate collection pipes in general and restricted waste cells should not be affected by the proposed additional waste fill load associated with the proposed final landform elevation (maximum RL 95m). The strength of the leachate pipes is considered suitable to maintain performance of the leachate collection system (LCS). The modelling results support the required design safety factors would be achieved for two pipe failure modes, buckling and deflection, in accordance with the Environmental Guidelines Solid waste landfills, Second edition, 2016.

The drainage triaxial geocomposite and protection geotextile already instated (existing cells) and proposed to be installed (new cells) is considered to be suitable to accommodate the additional waste load.

The leachate pipe strength assessment as detailed in section 3 indicates that the leachate collection system at the Site is expected to satisfy long term performance criteria under the proposed additional waste fill load as part of the proposed expansion.

Final cap design

The revised final cap design will be in full conformance with the *NSW EPA Environmental Guidelines: Solid Waste Landfill, Second edition 2016*. The revised final cap is substantially thicker (>1.9m) than previously proposed and includes a subsurface drainage layer on batters.

An adjustment to the EPL (licence) will be required, to reflect these changes to the cap design.

Slope stability assessment

The results of the slope stability analyses indicate that the proposed final landfill landform batter slopes are stable for the anticipated landfill extension. The proposed revised final cap design includes a subsurface drainage layer that improves the veneer stability of the landform batters and is predicted to be stable.

7. References

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- Maunsell and AECOM 2007, Industrial Cell A4 – Leachate collection and conveyance system, drawing number: 20021405.01-CI-1005
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- SUEZ, 2016, Environmental Management Plan - Elizabeth Drive Landfill (LEMP), 22 January 2016
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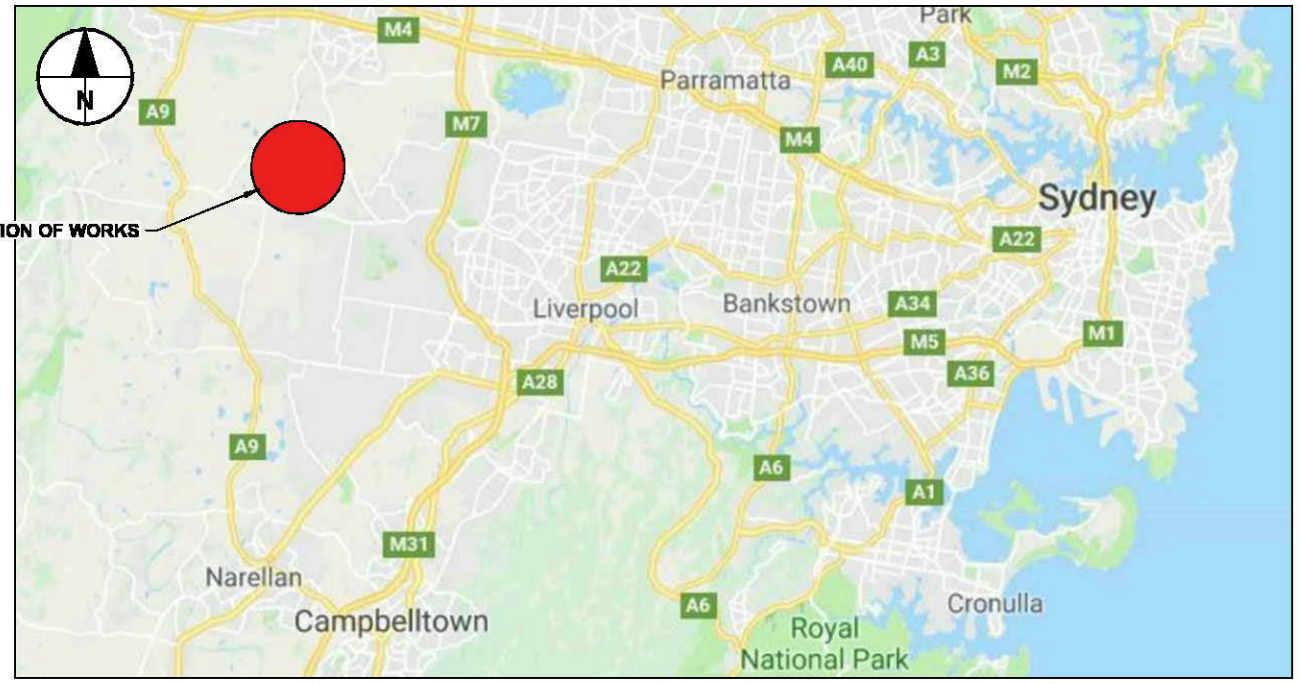
Appendices

Appendix A – Design drawings

ISO A1 594mm x 841mm

BADGERYS CREEK LANDFILL EXPANSION PROJECT

PROJECT NO. 60571292



LOCALITY PLAN - OVERALL

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DRAWING C00005 TO BE UPDATED

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60571292-SHT-CI-00002	CURRENT FILL AND EXISTING SITE LAYOUT
60571292-SHT-CI-00003	APPROVED FINAL LANDFORM
60571292-SHT-CI-00004	PROPOSED FINAL LANDFORM PRE-SETTLEMENT CONTOURS
60571292-SHT-CI-00005	PRE-SETTLEMENT FINAL LANDFORM SECTIONS AND APPROVED CAP SECTIONS
60571292-SHT-CI-00006	PRE-SETTLEMENT FINAL LANDFORM SECTIONS
60571292-SHT-CI-00007	INDICATIVE STAGE 1 FILL - LAYOUT
60571292-SHT-CI-00008	INDICATIVE STAGE 2 FILL - LAYOUT
60571292-SHT-CI-00009	INDICATIVE STAGE 3 FILL - LAYOUT
60571292-SHT-CI-00010	INDICATIVE STAGE 1 FILL - CATCHMENTS AND STROM WATER DRAINAGE PLAN
60571292-SHT-CI-00011	INDICATIVE STAGE 2 FILL - CATCHMENTS AND STROM WATER DRAINAGE PLAN
60571292-SHT-CI-00012	INDICATIVE STAGE 3 FILL - CATCHMENTS AND STROM WATER DRAINAGE PLAN
60571292-SHT-CI-00013	PROPOSED FINAL ACCESS ROAD ON PROPOSED FINAL LANDFORM
60571292-SHT-CI-00014	PROPOSED FINAL ACCESS ROAD - LONG SECTION
60571292-SHT-CI-00015	POST-SETTLEMENT FINAL LANDFORM
60571292-SHT-CI-00016	POST-SETTLEMENT FINAL LANDFORM - SECTION SHEET 01 OF 02
60571292-SHT-CI-00017	POST-SETTLEMENT FINAL LANDFORM - SECTION SHEET 02 OF 02

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LOCALITY PLAN - SITE

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PROJECT

SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

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PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

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A	01.11.2018	FINAL DRAFT
I/R	DATE	DESCRIPTION

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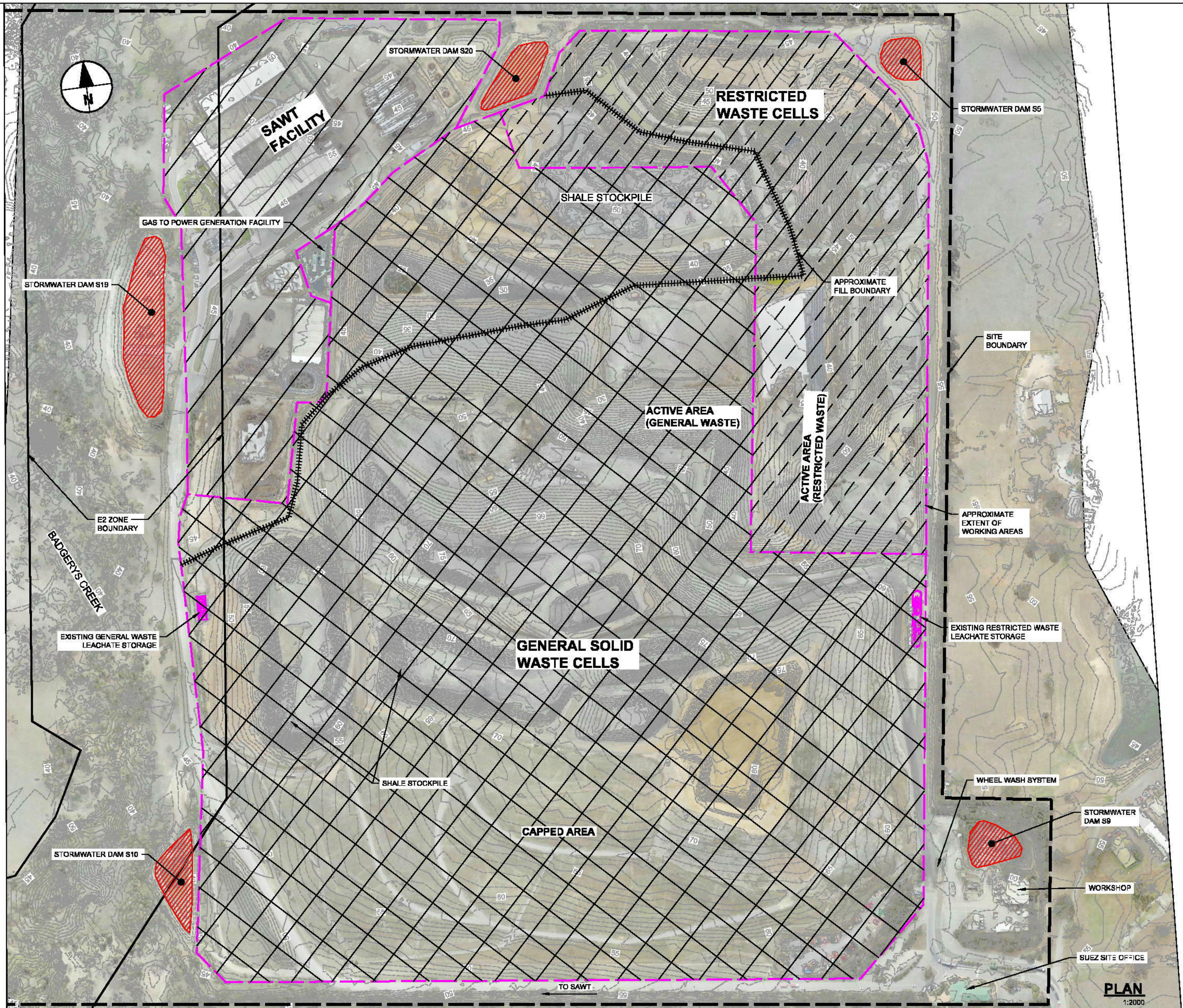

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- EXISTING GROUND CONTOURS MINOR (1m)
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- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN

NOTE:

EXISTING GROUND SURVEY IS SEPT 2018 SURVEY.

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A	01.11.2018	FINAL DRAFT

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SHEET TITLE
BADGERYS CREEK LANDFILL CURRENT FILL & EXISTING SITE LAYOUT

SHEET NUMBER
60571292-SHT-CI-00002

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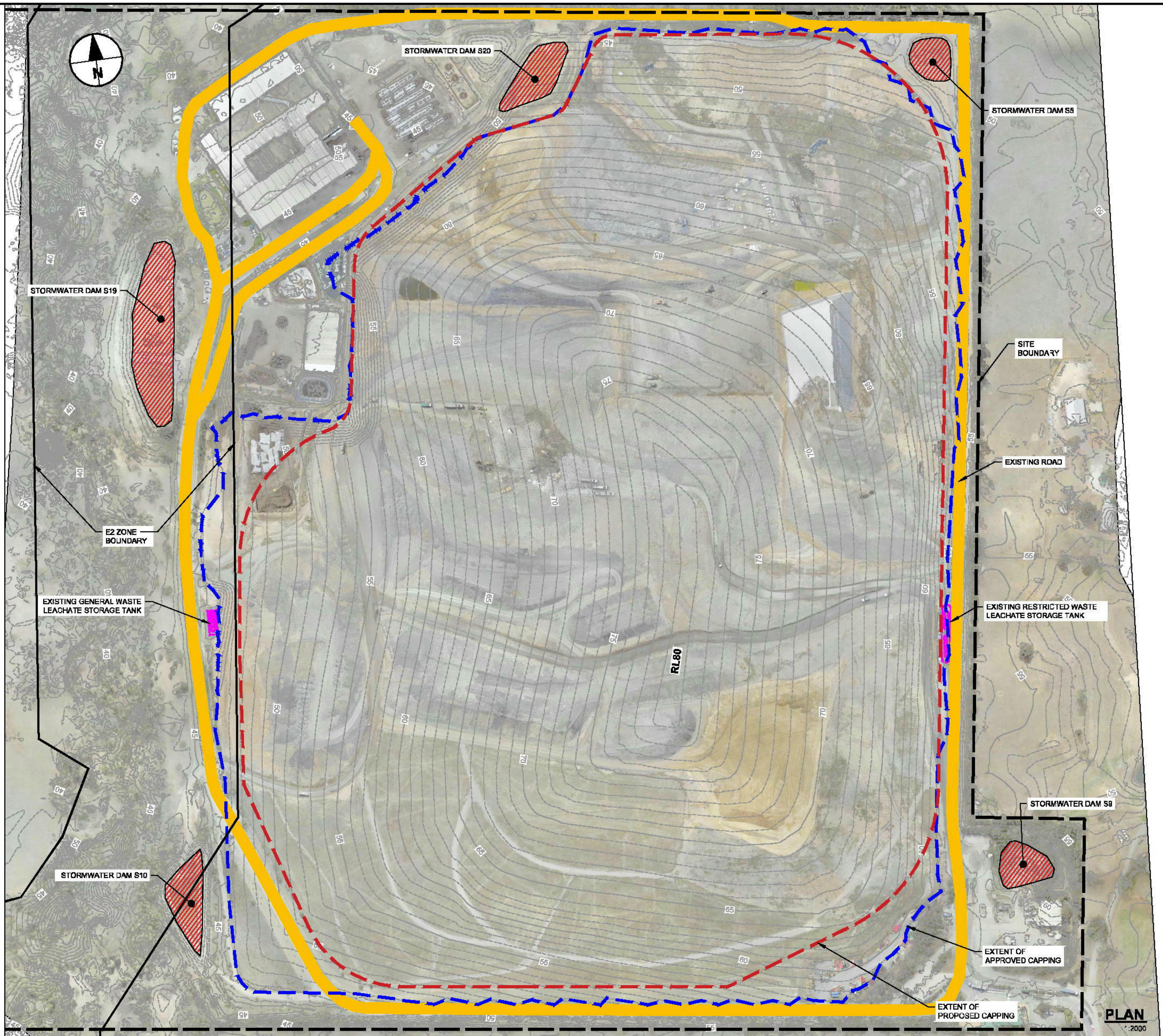

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- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN

NOTE:

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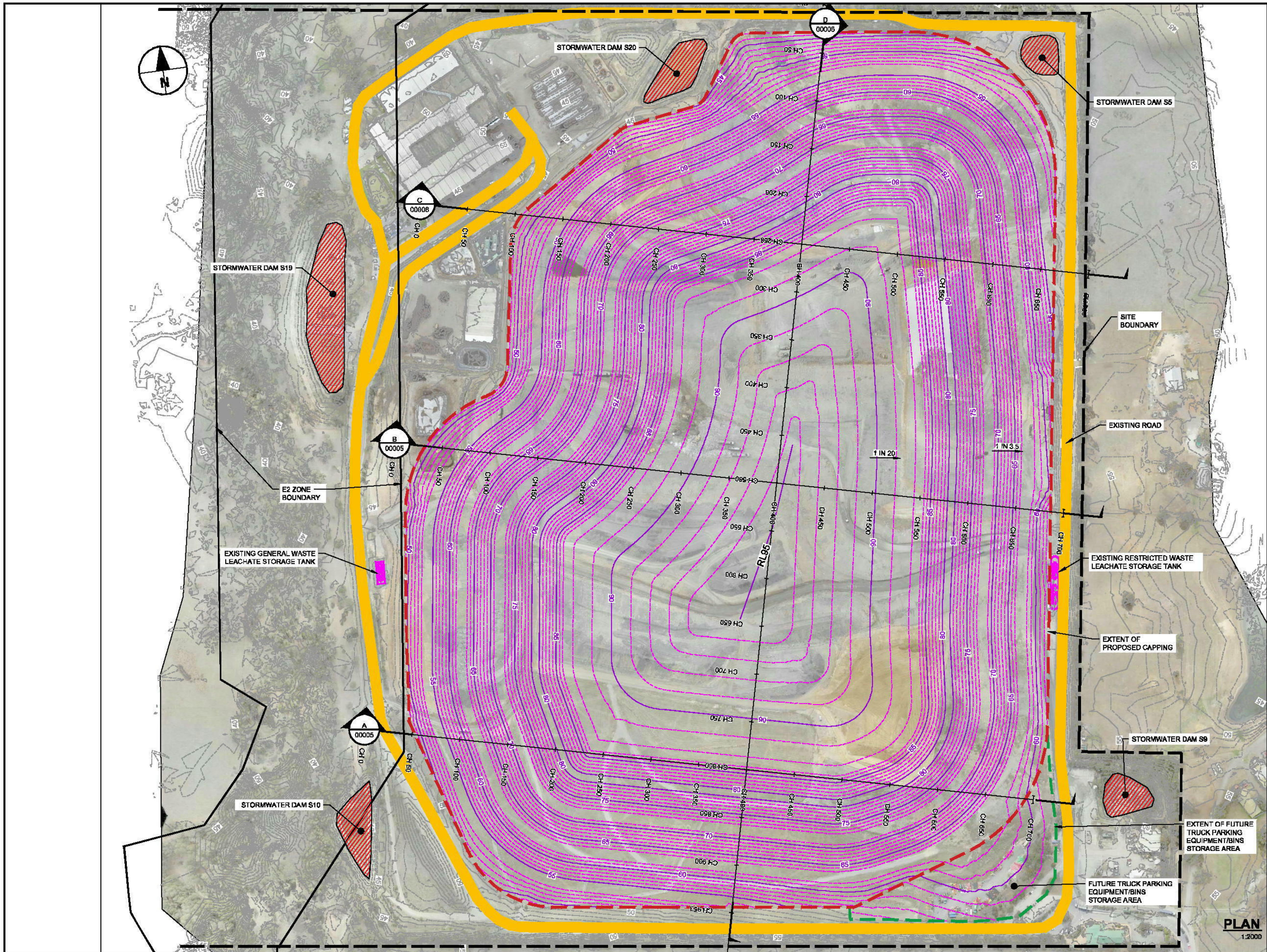

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- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
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- SLOPE / SWALE DRAIN

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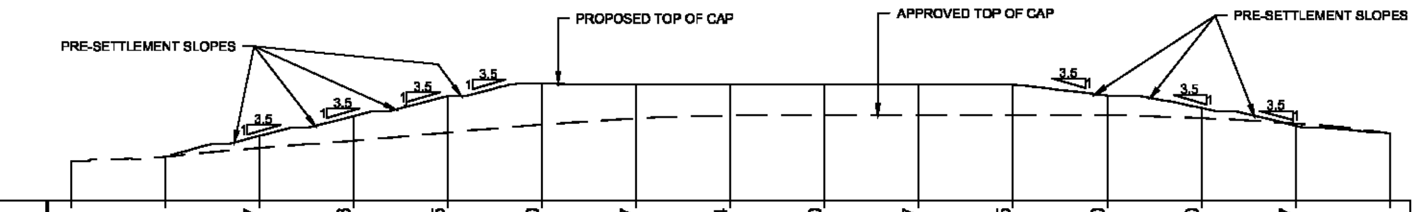

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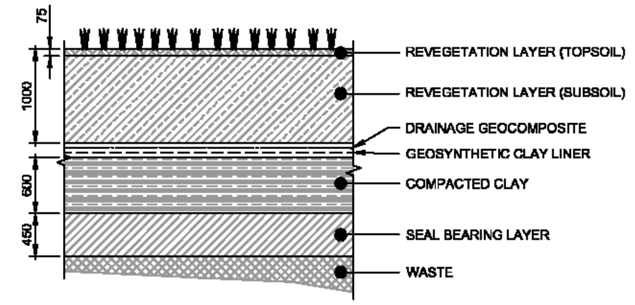

APPROVED RESTRICTED SOLID WASTE CELL CAP
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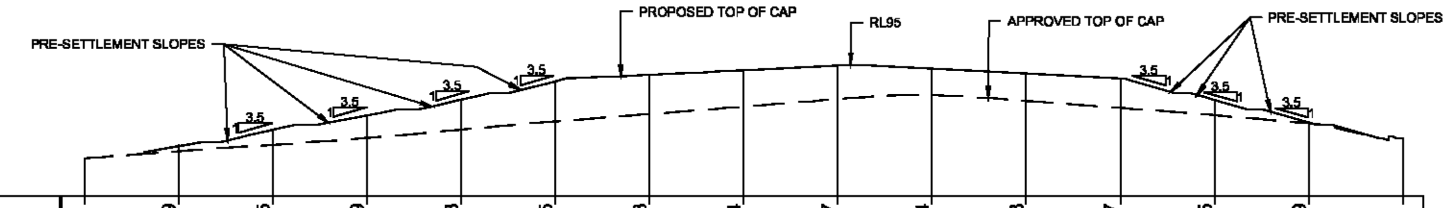

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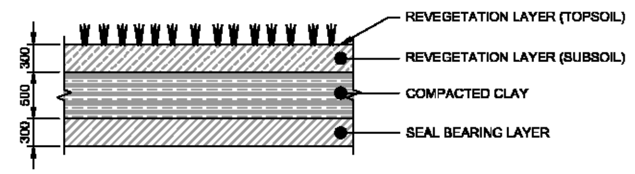

APPROVED GENERAL SOLID WASTE CELL CAP
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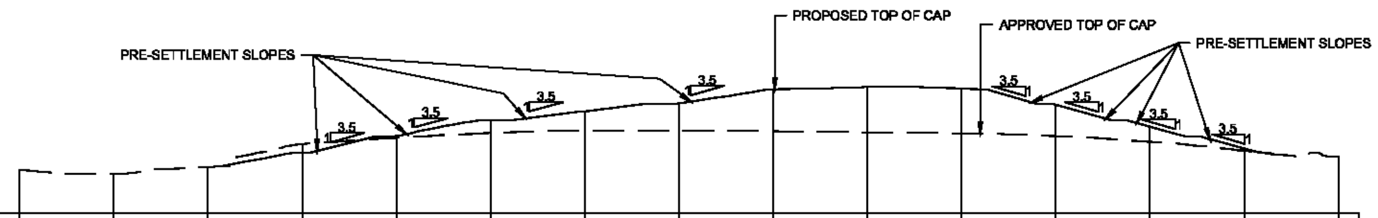

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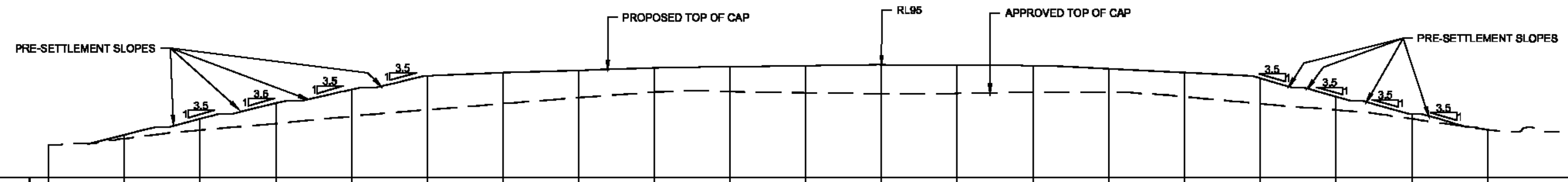
CHAINAGES	0	50	100	150	200	250	300	350	400	450	500	550	600	650	700

SECTION C
SCALE 1:2000 @A1



DATUM 21.0

PROPOSED TOP OF CAP		49.005	59.563	69.736	78.816	87.669	89.742	91.331	92.891	93.731	94.427	94.990	94.717	94.494	92.333	89.835	86.190	74.164	62.800			
APPROVED TOP OF CAP	42.286	46.404	52.108	56.349	61.142	65.500	69.402	73.312	76.974	77.436	76.655	76.231	76.304	76.921	77.205	73.967	69.363	65.331	59.442	52.376		
CHAINAGES	0	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000	1019

SECTION D
SCALE 1:2000 @A1

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PROJECT
SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT
SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

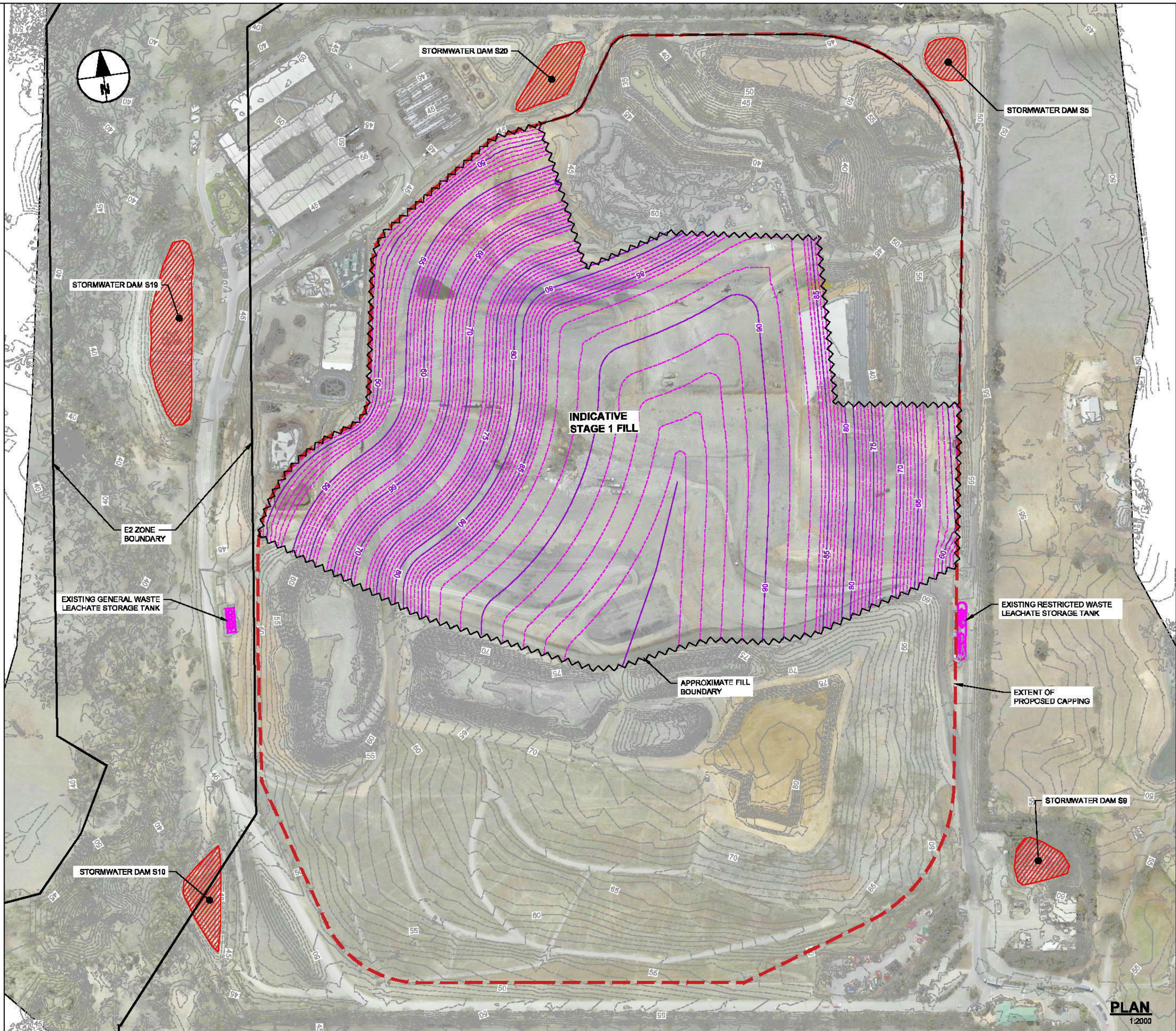
PROJECT NUMBER
60571292

SHEET TITLE
BADGERYS CREEK LANDFILL
PRE-SETTLEMENT FINAL LANDFORM SECTIONS
SECTIONS

SHEET NUMBER
60571292-SHT-CI-00006

Document Set ID: 8777016
Version: 1, Version Date: 18/07/2019

ISO A1 594mm x 841mm



LEGEND:

- EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN

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PROJECT

SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT

SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

PROJECT NUMBER

60571292

SHEET TITLE

BADGERYS CREEK LANDFILL INDICATIVE STAGE 1 FILL LAYOUT

SHEET NUMBER

60571292-SHT-CI-00007

ISO A1 594mm x 841mm

LEGEND:

- EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN

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PROJECT

SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT

SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

PROJECT NUMBER

60571292

SHEET TITLE

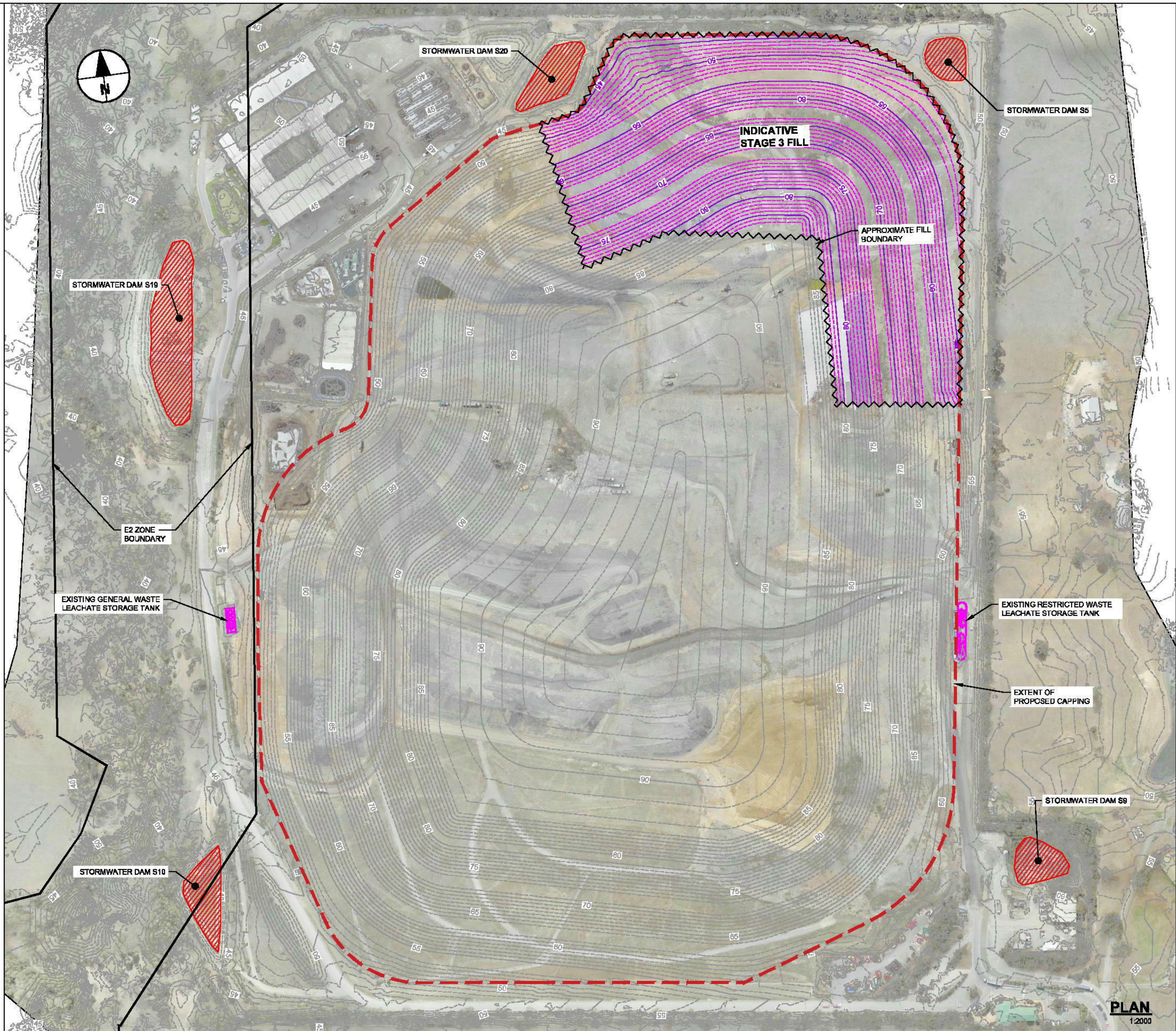
BADGERYS CREEK LANDFILL INDICATIVE STAGE 2 FILL LAYOUT

SHEET NUMBER

60571292-SHT-CI-00008

Document Set ID: 8777016
Version: 1, Version Date: 18/07/2019

ISO A1 594mm x 841mm



LEGEND:

- EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN

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PROJECT

SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT

SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

PROJECT NUMBER

60571292

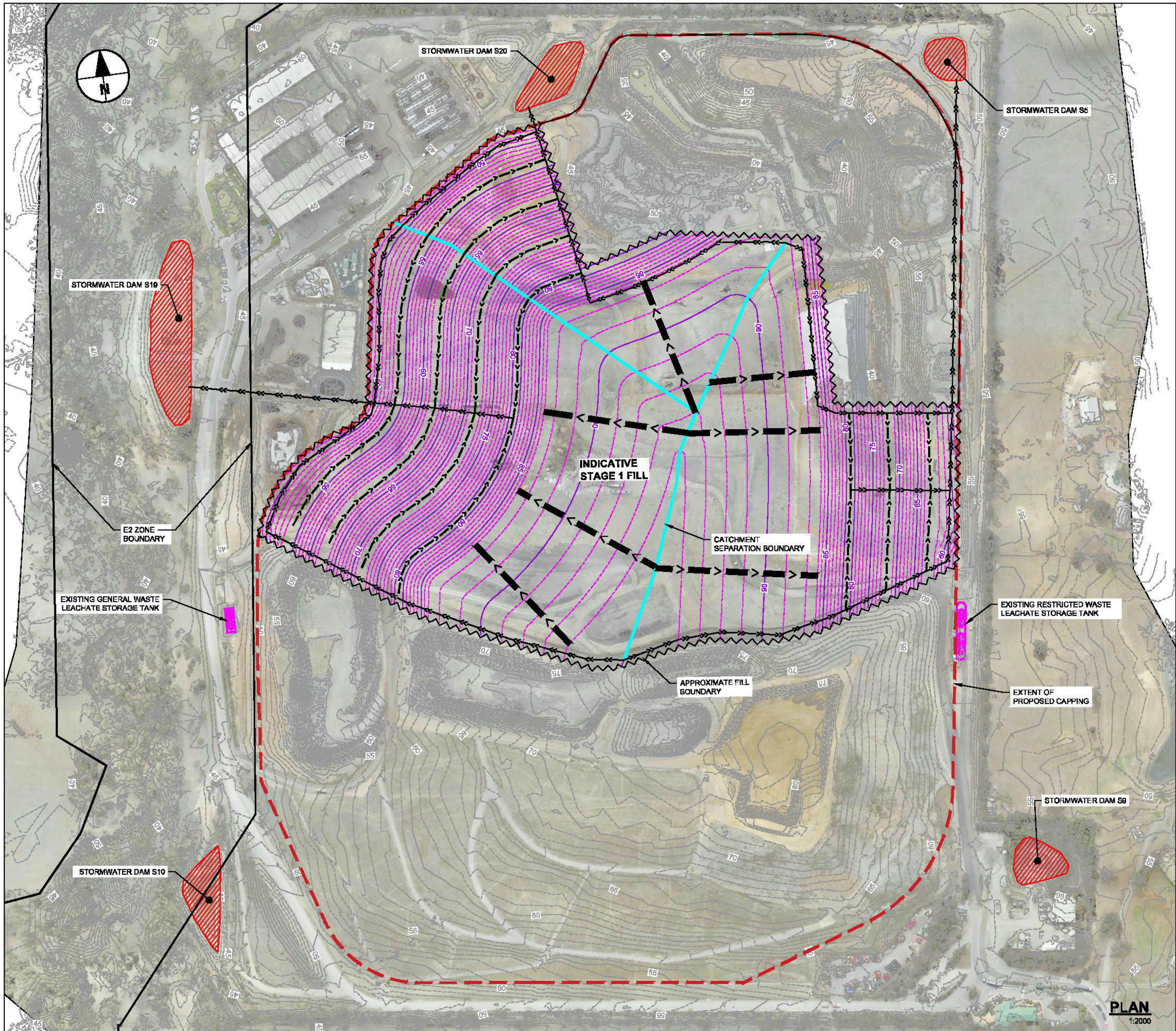
SHEET TITLE

BADGERYS CREEK LANDFILL INDICATIVE STAGE 3 FILL LAYOUT

SHEET NUMBER

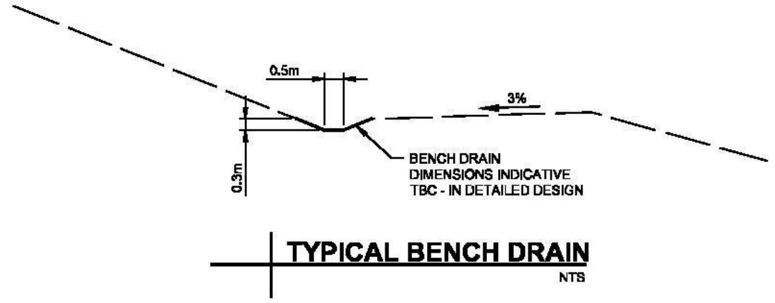
60571292-SHT-CI-00009

ISO A1 594mm x 841mm

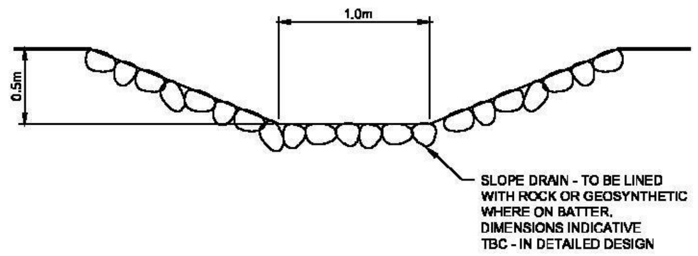


LEGEND:

- 20 — EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- 20 — PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN



TYPICAL BENCH DRAIN
NTS



TYPICAL SLOPE / SWALE DRAIN
NTS

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PROJECT
SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT
SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

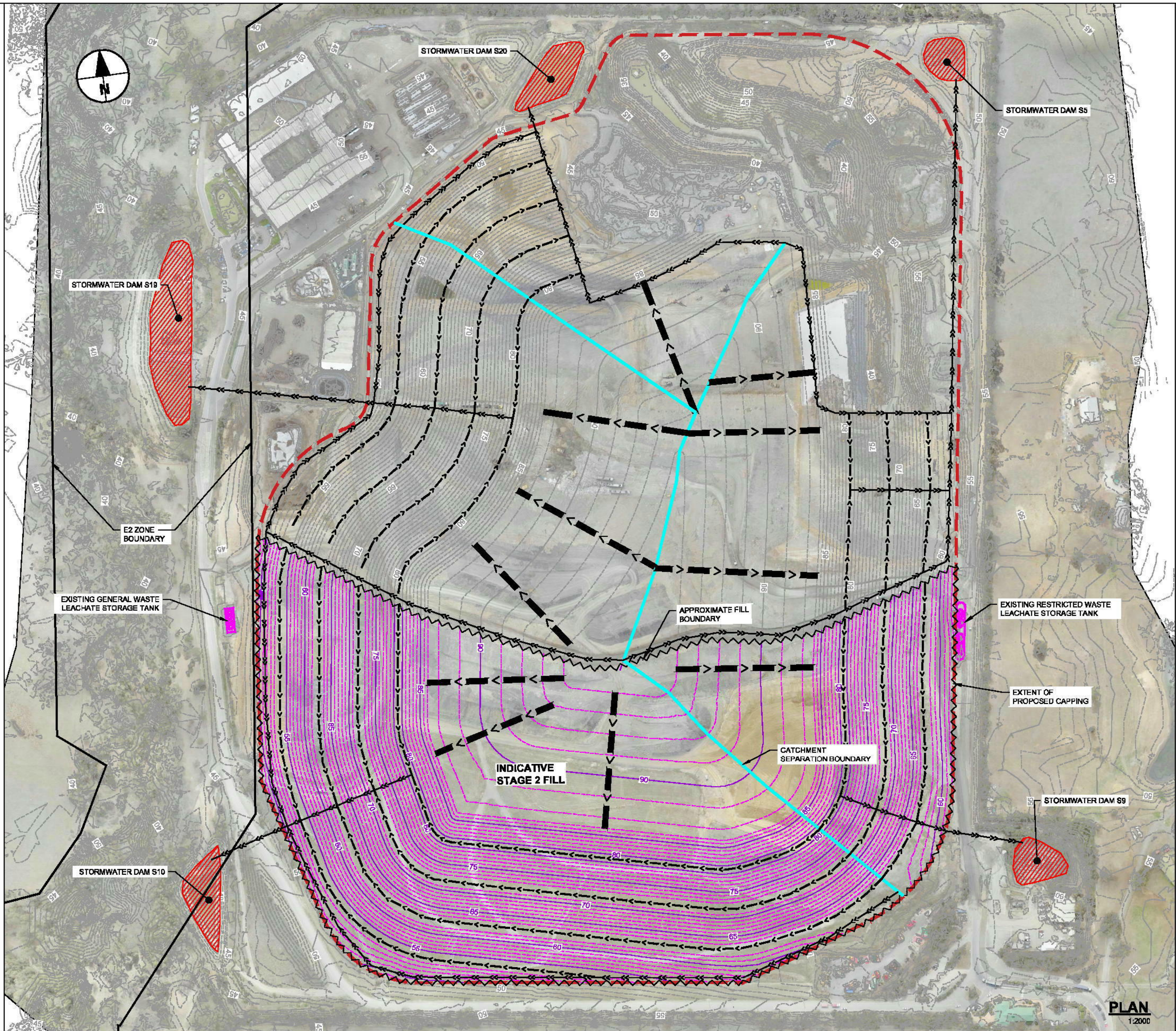
ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2018	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

PROJECT NUMBER
60571292

SHEET TITLE
BADGERYS CREEK LANDFILL
INDICATIVE STAGE 1 FILL
CATCHMENTS AND STORMWATER DRAINAGE PLAN

SHEET NUMBER
60571292-SHT-CI-00010



LEGEND:

- EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN

ISO A1 594mm x 841mm

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PROJECT
SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT
SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

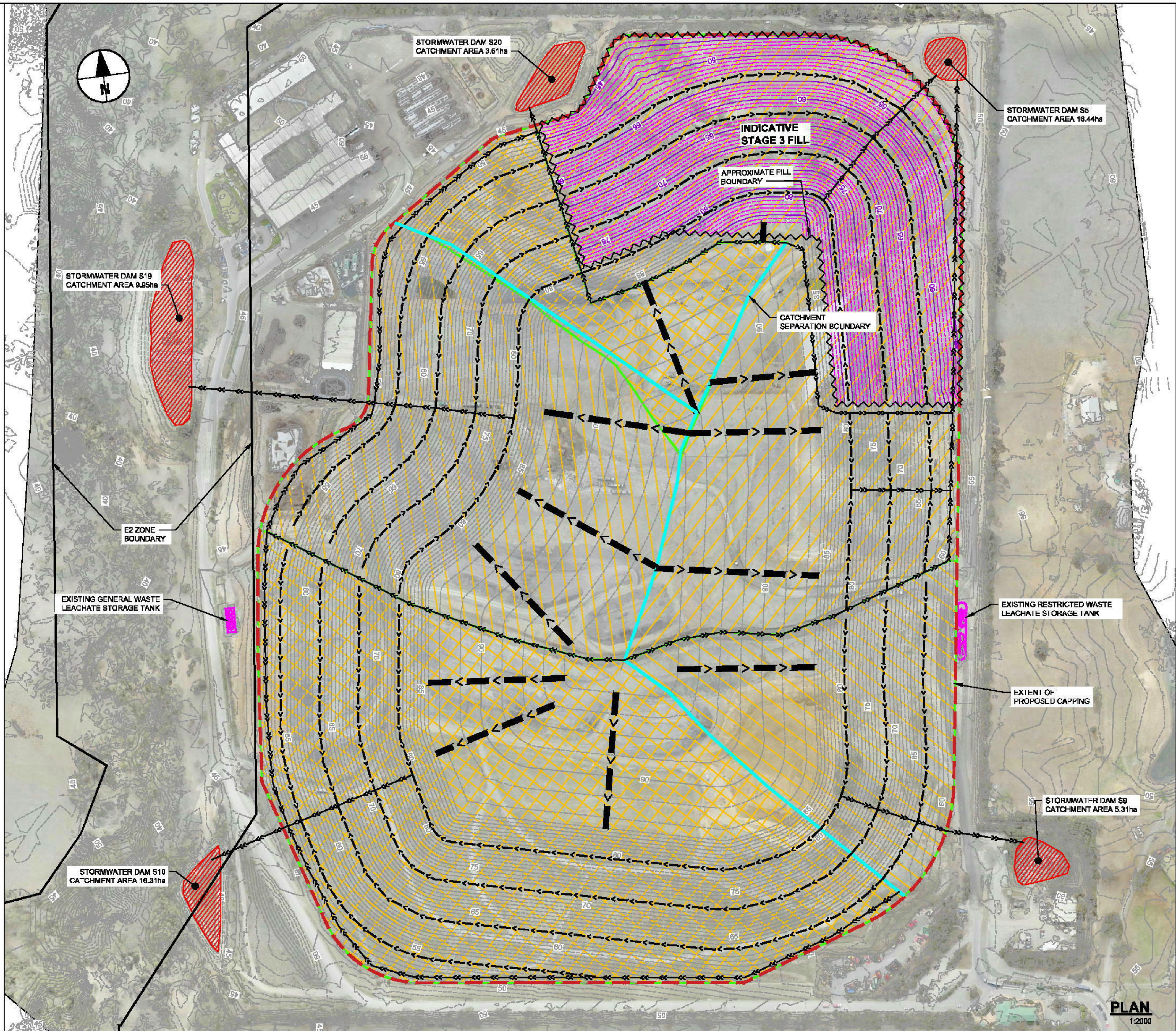
I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

PROJECT NUMBER
60571292

SHEET TITLE
BADGERYS CREEK LANDFILL
INDICATIVE STAGE 2 FILL
CATCHMENTS AND STORMWATER DRAINAGE PLAN

SHEET NUMBER
60571292-SHT-CI-00011

ISO A1 594mm x 841mm



LEGEND:

- EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN

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PROJECT

SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT

SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

PROJECT NUMBER

60571292

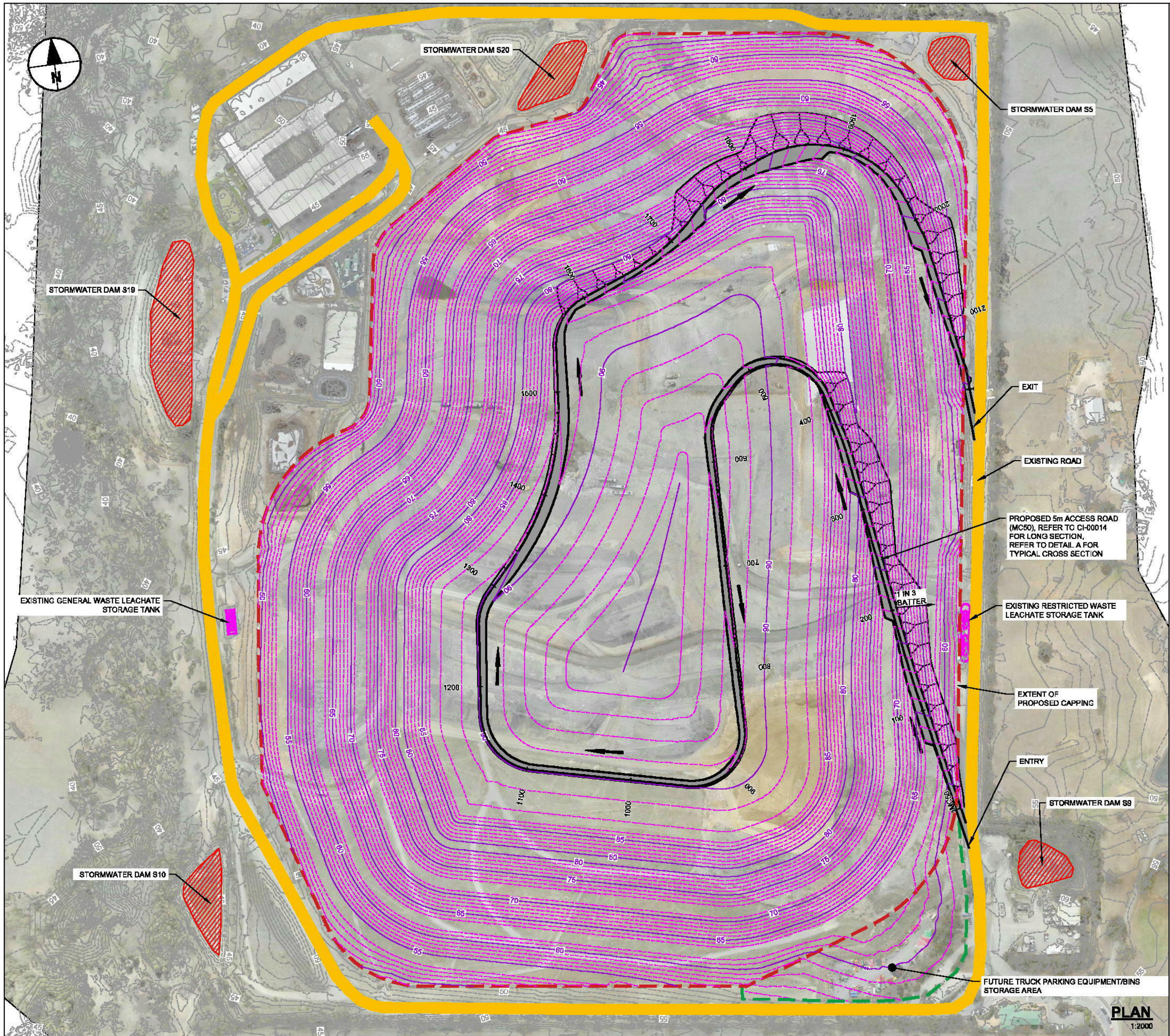
SHEET TITLE

BADGERYS CREEK LANDFILL
INDICATIVE STAGE 3 FILL
CATCHMENTS AND STORMWATER DRAINAGE PLAN

SHEET NUMBER

60571292-SHT-CI-00012

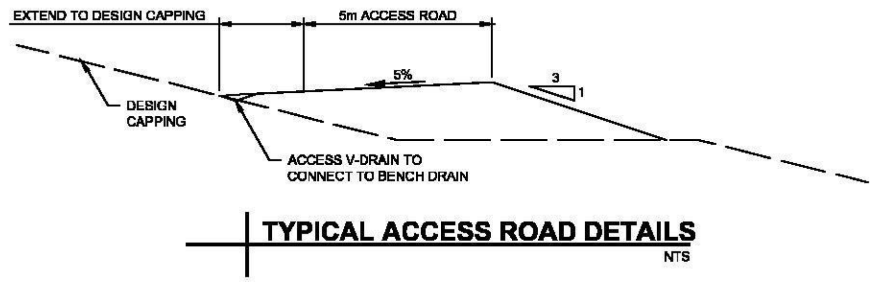
Document Set ID: 8777016
Version: 1, Version Date: 18/07/2019



PLAN
1:2000

LEGEND:

- EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN



TYPICAL ACCESS ROAD DETAILS
NTS

ISO A1 594mm x 841mm

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PROJECT
SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT
SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

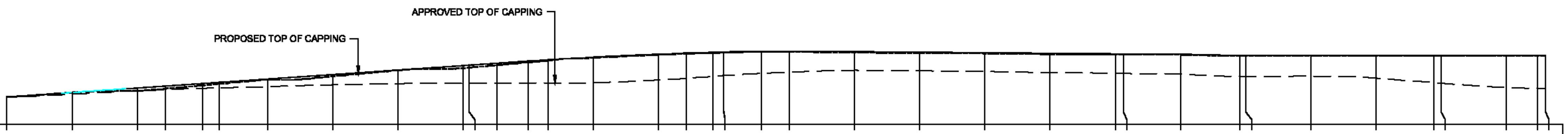
I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

PROJECT NUMBER
60571292

SHEET TITLE
BADGERYS CREEK LANDFILL PROPOSED FINAL ACCESS ROAD ON PROPOSED FINAL LANDFORM

SHEET NUMBER
60571292-SHT-CI-00013

ISO A1 594mm x 841mm

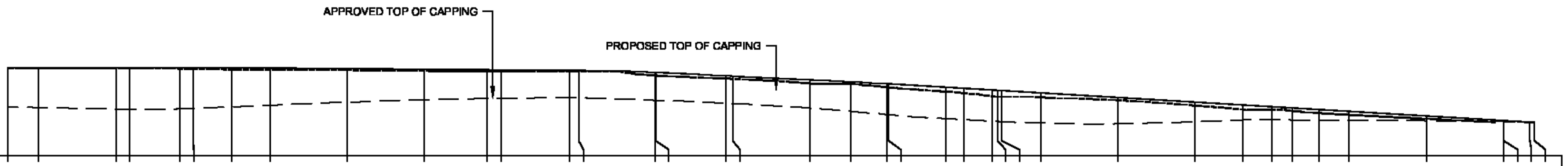


VERTICAL DETAILS: 7.493% · R=7994.338 · 6.867% · R=115438.548 · 6.911% · R=2293.309 · 4.731% · R=2557.754 · 3.167% · R=1785.861 · -0.753% · R=2816.462 · R=1650.000 · -1.818% · 0.002%

HORIZONTAL DETAILS: L=121.501 · R=1000.000 · L=191.844 · R=-200.000 · L=38.815 · R=-50.000 · L=20.366 · R=-50.000 · L=277.703 · R=50.000 · L=148.645 · R=55.000 · L=85.367

CHAINAGES	ACCESS ROAD	PROPOSED TOP OF CAPPING	APPROVED TOP OF CAPPING
0.000	57.991		58.354
50.000	61.734	61.640	60.446
100.000	65.279	62.983	63.036
121.501	66.756	64.672	63.984
150.000	68.713	67.153	64.942
162.815	69.593	68.340	65.325
200.000	72.147	71.400	66.252
250.000	75.580	74.123	67.638
300.000	79.015	78.711	68.477
350.000	82.463	80.489	68.791
354.658	82.785	80.836	68.784
376.021	84.262	82.656	68.844
400.000	85.919	84.905	68.883
414.836	86.905	86.219	68.899
450.000	88.861	88.382	69.308
500.000	91.207	90.727	71.268
521.258	92.042	91.528	72.614
541.624	92.689	92.162	74.089
550.000	92.904	92.396	74.696
578.563	93.340	92.782	76.412
600.000	93.367	92.752	77.384
650.000	92.998	92.447	78.689
700.000	92.621	92.047	78.450
750.000	92.245	91.645	77.747
800.000	91.868	91.256	77.109
850.000	91.492	90.987	76.460
856.266	91.444	90.941	76.372
900.000	91.085	90.619	75.816
945.686	90.506	90.000	74.171
950.000	90.500	90.000	74.184
1000.000	90.500	90.000	74.294
1050.000	90.500	90.000	72.727
1094.330	90.500	89.998	69.346
1100.000	90.500	89.990	68.913
1150.000	90.500	89.939	65.615
1173.825	90.500	89.928	64.874
1180.000	90.500	89.922	64.784

ACCESS ROAD MC50 - LONGITUDINAL SECTION (CH 0 TO CH 1180)
SCALE 1:2000 @A1



VERTICAL DETAILS: 0.002% · R=8318.182 · -0.721% · R=1144.270 · -5.091% · R=5968.085 · -4.586% · R=3660.341 · -5.954%

HORIZONTAL DETAILS: L=85.367 · R=50.000 · L=33.920 · R=-250.000 · L=53.586 · R=50.000 · L=46.201 · R=-200.000 · L=24.479 · R=100.000 · L=29.946.343 · R=50.000 · R=100.000 · L=32.160 · R=100.000 · L=119.962 · R=100.000 · L=3.061

CHAINAGES	ACCESS ROAD	PROPOSED TOP OF CAPPING	APPROVED TOP OF CAPPING
1180.000	90.500	89.922	64.784
1200.000	90.500	89.893	64.419
1250.000	90.498	89.822	63.395
1259.193	90.488	89.809	63.220
1291.412	90.371	89.563	63.502
1300.000	90.318	89.449	63.849
1325.332	90.137	89.181	64.890
1350.000	89.959	88.985	65.896
1400.000	89.598	88.597	67.875
1450.000	89.238	88.244	69.430
1490.598	88.945	87.988	70.239
1500.000	88.877	87.941	70.383
1544.183	88.558	87.774	70.548
1550.000	88.516	87.766	70.543
1599.567	87.291	85.223	69.166
1600.000	87.271	85.202	69.152
1645.767	85.007	83.224	67.250
1650.000	84.812	83.073	67.049
1700.000	82.514	80.109	64.127
1726.907	81.144	79.919	61.989
1750.000	79.814	77.632	59.944
1751.386	79.731	77.498	59.822
1788.107	77.545	74.723	57.193
1800.000	76.837	73.768	56.522
1818.051	75.762	71.791	55.545
1821.934	75.531	71.400	55.355
1824.277	75.391	71.400	55.246
1850.000	73.860	70.945	54.392
1900.000	70.882	69.124	53.955
1950.000	67.905	65.715	55.221
1980.839	66.069	63.193	56.317
2000.000	64.928	62.800	56.547
2012.999	64.154	62.216	56.275
2030.192	63.130	60.820	56.053
2050.000	61.951	59.707	56.037
2100.000	58.974	57.122	55.919
2150.000	55.997		55.310
2150.154	55.988		55.304
2167.039	54.982		54.871
2170.100	54.800		54.834

ACCESS ROAD MC50 - LONGITUDINAL SECTION (CH1180 TO END)
SCALE 1:2000 @A1

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Filename: C:\PROJECTS\6605\60571292\20-SHEETS\60571292-SHT-CI-00014.DWG

AECOM

CONSULTANT
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PROJECT
SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT
SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

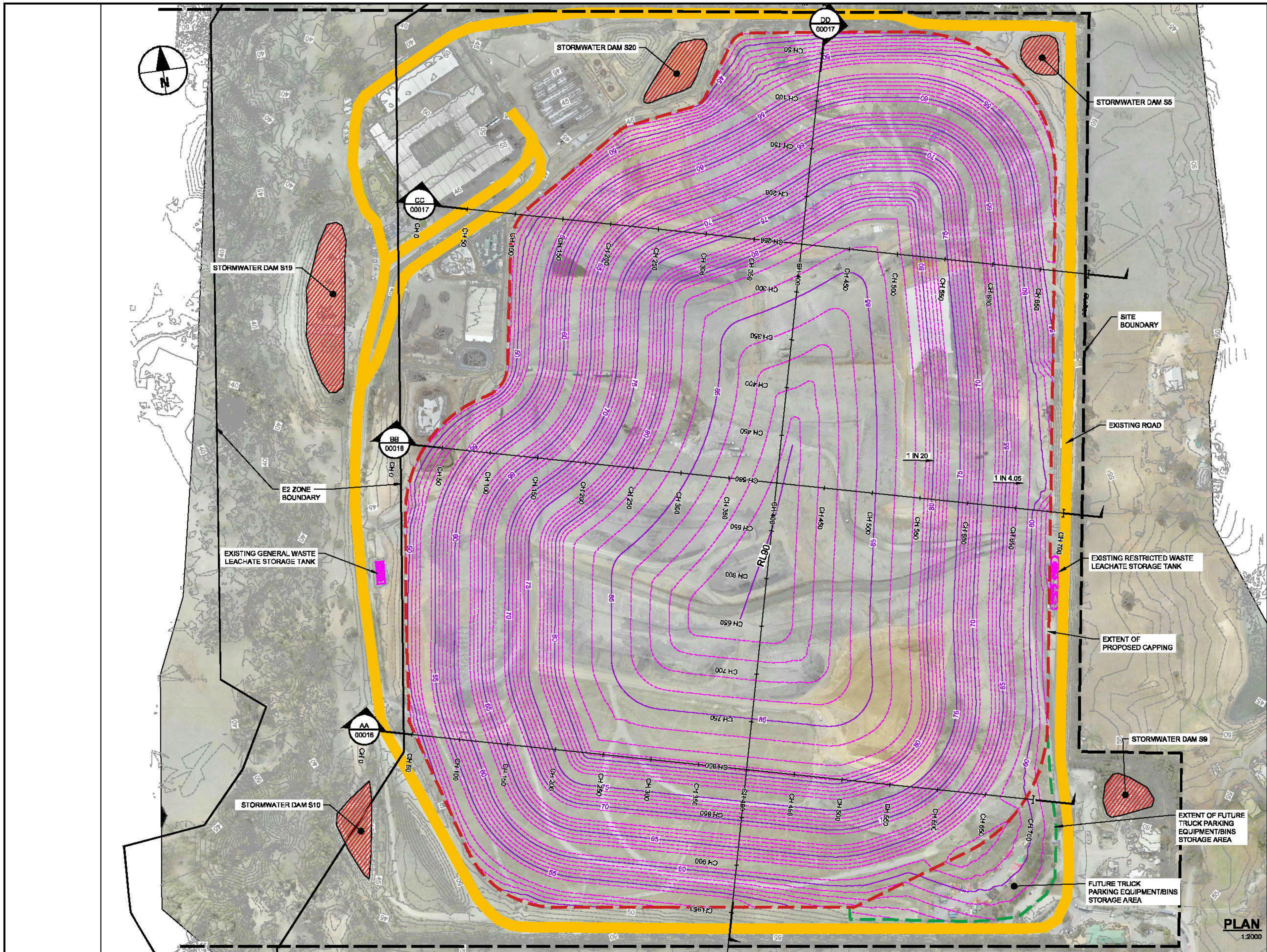
ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	FINAL DRAFT

PROJECT NUMBER
60571292

SHEET TITLE
BADGERYS CREEK LANDFILL PROPOSED FINAL ACCESS ROAD LONG SECTION

SHEET NUMBER
60571292-SHT-CI-00014



LEGEND:

- EXISTING GROUND CONTOURS MAJOR (5m)
- EXISTING GROUND CONTOURS MINOR (1m)
- PROPOSED DESIGN CAP CONTOURS MAJOR (5m)
- PROPOSED DESIGN CAP CONTOURS MINOR (1m)
- SITE BOUNDARY
- EXTENT OF APPROVED CAPPING
- EXISTING ROAD
- EXISTING STORMWATER POND
- EXISTING LEACHATE STORAGE TANKS
- EXTENT OF PROPOSED NEW CAPPING
- EXTENT OF FUTURE PARKING/STORAGE AREA
- EXTENT OF APPROXIMATE CATCHMENT
- EXTENT OF STAGE FILLING BOUNDARY
- PROPOSED ROAD
- DIRECTION OF SHEET FLOW
- BENCH DRAIN
- SLOPE / SWALE DRAIN

AECOM

CONSULTANT

AECOM Australia Pty Ltd
A.B.N 20 093 846 925
www.aecom.com

PROJECT

SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT

SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	DRAFT

PROJECT NUMBER

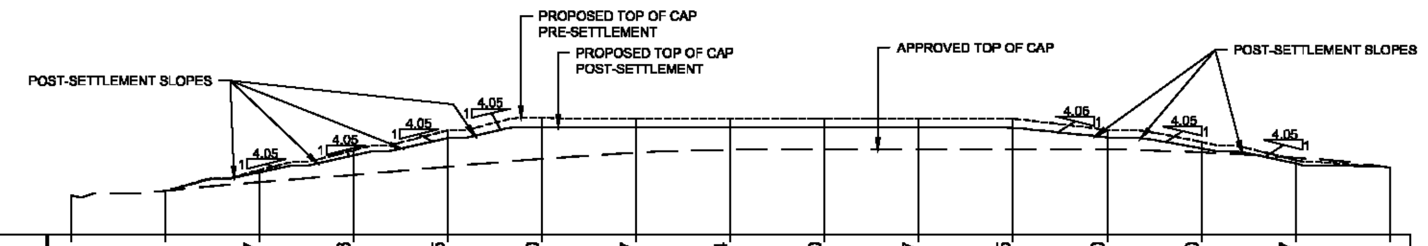
60571292

SHEET TITLE

BADGERYS CREEK LANDFILL PROPOSED FINAL LANDFORM POST-SETTLEMENT CONTOURS

SHEET NUMBER

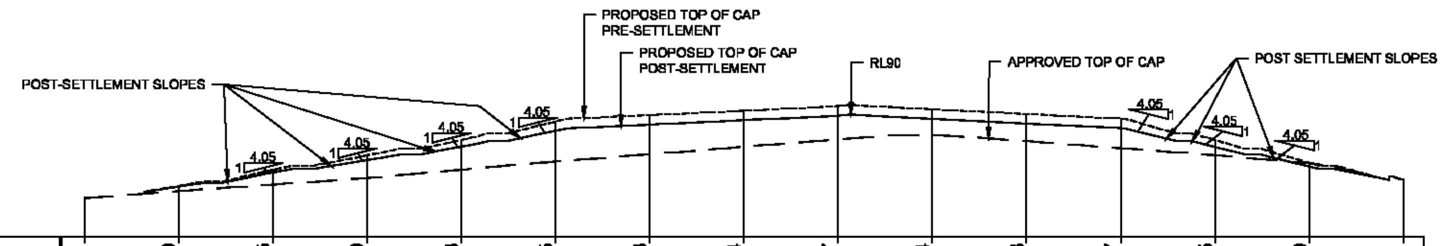
60571292-SHT-CI-00015



DATUM 24.0

PROPOSED TOP OF CAP PRE-SETTLEMENT			58.467	69.056	79.645	86.230	86.217	86.204	86.190	86.177	85.705	80.000	73.260	63.837	
PROPOSED TOP OF CAP POST-SETTLEMENT			57.327	66.449	75.613	81.394	81.382	81.371	81.359	81.347	80.932	75.920	70.089	61.929	
APPROVED TOP OF CAP	44.947	47.464	52.065	56.083	60.365	64.486	67.802	69.245	69.363	69.554	69.642	69.343	67.944	65.065	59.939
CHAINAGES	0	50	100	150	200	250	300	350	400	450	500	550	600	650	700

SECTION AA
SCALE 1:2000 @A1



DATUM 25.0

PROPOSED TOP OF CAP PRE-SETTLEMENT		51.799	60.375	68.109	76.333	86.115	89.578	91.904	94.427	92.674	90.178	87.687	76.195	64.179	
PROPOSED TOP OF CAP POST-SETTLEMENT		51.502	58.957	65.629	72.748	81.293	84.578	86.904	89.427	87.674	85.178	82.687	72.628	62.224	
APPROVED TOP OF CAP	45.319	49.013	52.535	56.087	60.568	64.857	68.768	72.701	76.655	79.000	75.685	71.973	68.175	63.972	55.647
CHAINAGES	0	50	100	150	200	250	300	350	400	450	500	550	600	650	700

SECTION BB
SCALE 1:2000 @A1

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PROJECT
SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT
SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

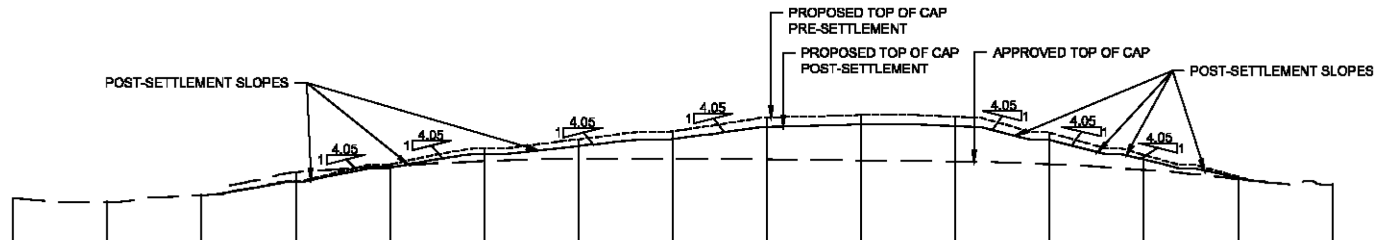
ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	DRAFT

PROJECT NUMBER
60571292

SHEET TITLE
BADGERYS CREEK LANDFILL
POST-SETTLEMENT FINAL LANDFORM SECTIONS
SECTION SHEET 01 OF 02

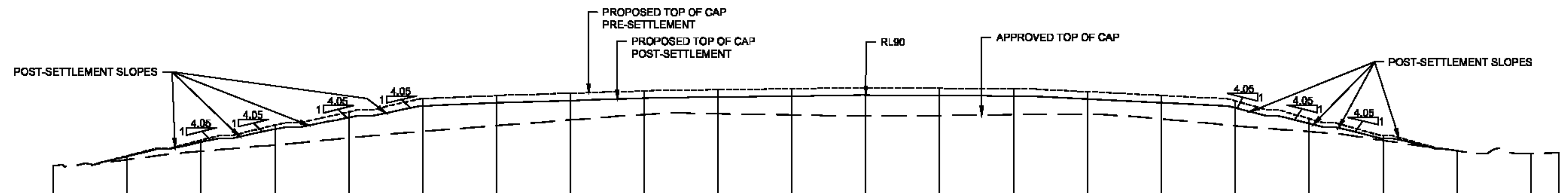
SHEET NUMBER
60571292-SHT-CI-00016



DATUM 22.0

PROPOSED TOP OF CAP PRE-SETTLEMENT				54.200	62.899	71.400	76.176	80.473	87.669	88.976	88.172	79.662	67.905	56.342	
PROPOSED TOP OF CAP POST-SETTLEMENT				53.680	61.116	68.480	72.612	76.336	82.669	83.976	83.172	75.627	65.452	55.676	
APPROVED TOP OF CAP	45.000	43.178	47.023	58.829	62.030	64.564	65.881	65.937	65.500	65.031	64.367	63.016	60.051	55.131	52.371
CHAINAGES	0	50	100	150	200	250	300	350	400	450	500	550	600	650	700

SECTION CC
SCALE 1:2000 @A1



DATUM 21.0

PROPOSED TOP OF CAP PRE-SETTLEMENT		49.005	59.563	69.736	78.818	87.669	89.742	91.331	92.891	93.731	94.427	94.990	94.717	94.494	92.333	89.835	86.190	74.164	62.800			
PROPOSED TOP OF CAP POST-SETTLEMENT		48.734	58.264	67.039	74.898	82.669	84.742	86.331	87.891	88.731	89.427	89.980	89.717	89.494	87.333	84.835	81.359	70.871	61.030			
APPROVED TOP OF CAP	42.660	46.404	52.108	56.349	61.142	65.500	69.402	73.312	76.974	77.436	76.655	76.231	76.304	76.921	77.205	73.967	69.363	65.331	59.442	52.376	50.791	50.992
CHAINAGES	0	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000	1019

SECTION DD
SCALE 1:2000 @A1

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PROJECT
SUEZ BADGERYS CREEK LANDFILL EXPANSION PROJECT

CLIENT
SUEZ

0 50 100 m
1:2000

PROJECT MANAGEMENT INITIALS

QL		
DESIGNER	CHECKED	APPROVED

PROJECT DATA

DATUM	AHD	SURVEY	MGA 56

ISSUE/REVISION

I/R	DATE	DESCRIPTION
B	04.02.2019	FINAL DRAFT REVISED
A	01.11.2018	DRAFT

PROJECT NUMBER
60571292

SHEET TITLE
BADGERYS CREEK LANDFILL
POST-SETTLEMENT FINAL LANDFORM SECTIONS
SECTION SHEET 02 OF 02

SHEET NUMBER
60571292-SHT-CI-00017

Existing access road

Existing endemic vegetation cover along Badgerys Creek corridor

Buffer zone to be retained west of site and incorporate the rehabilitation of existing bushland. Degraded areas to be rehabilitated and provide a robust community of endemic plants for screening and habitat opportunities.

The incorporation of well integrated screen planting along the site boundary to minimise the extent of the final landform when viewed from surrounding locations.

Supplementary screen planting to site boundary. The reinstatement of endemic vegetation with characteristics of local plant communities to provide a consistent landscape character. Supplementary screen planting will consist of endemic trees and a bush understorey including shrubs, grasses and groundcovers.

Final cap surface to be stabilised using a mix of grass species and maintained via slashing/ mowing. The specific mix of grass species to be determined based upon ongoing discussions with Western Sydney Airport with view to discouraging congregation of birds.

KEY

- Supplementary screen planting
- Existing vegetation cover
- Buffer zone
- Landfill pasture grass cover
- Stormwater dams
- Site boundary
- Extent of proposed new capping

GENERAL NOTES

- Plant species to be sourced from local suppliers and to be of local provenance.
- Provide rabbit guards to tree and shrub species only.
- Existing endemic vegetation and tree cover to be retained and protected
- Bush regeneration work to be undertaken by members of the Australian Association of Bush Regenerators.

0 200m

AECOM

Client: SUEZ
Date: 24/06/19

Project Name: Badgerys Creek Landfill Expansion Project
Landscape Plan

Landscape Plan **L01**

Appendix B – Flexible pipe loading assessment

Client:	SUEZ	**Job Number:**	21-27038	**Revision:**	4
Project:	Elizabeth Drive Landfill	**Calcs by:**	A Roberts	**Date:**	28/01/2020
Subject:	Leachte Collection Pipe Calculations	**Checked by:**		**Date:**	

Statement of design procedure

This spreadsheet provides design calculations for the structural capacity of leachate collection pipe

References

AS2566.1 Buried flexible pipelines - part 1: structural design & 2D PLAXIS modelling

Item	Description	Symbol	1	2	2	3	4	4	5	5	5	5	Unit	Notes
			Cell A1-A3 HDPE DN160 PN10 PE100	**Cell A4 HDPE DN200 SDR11 PE100**	**Cell A5-A10 HDPE DN200 SDR11 PE100**	**Cell B1-B5 MDPE DN160 PN12.5 PE80**	**Cell C1-C2 MDPE DN160 PN12.5 PE80**	**Cell D1 MDPE DN160 PN12.5 PE80**	**Cell D2-D3 HDPE DN200 PN16 PE100**	**Cell E1 HDPE DN200 PN10 PE100**	**Cell E2-E4 HDPE DN200 PN16 PE100**	**Cell F1-F6 HDPE DN200 PN16 PE100**		
1	**Ring-bending stiffness**													
1.1	DN		160	200	200	160	160	160	200	200	200	200	mm	
1.2	External diameter	D_e	0.1600	0.2000	0.2000	0.1600	0.1600	0.1600	0.2000	0.2000	0.2000	0.2000	m	
1.2 (a)	Mean internal diameter	D_i	0.1406	0.1624	0.1624	0.1299	0.1299	0.1299	0.1624	0.1757	0.1624	0.1624		
1.3	Wall thickness	t	0.0097	0.0188	0.0188	0.0151	0.0151	0.0151	0.0188	0.0122	0.0188	0.0188	m	Poliplex design book p 6-3
1.4	Initial (3-minute) ring bending modulus of elasticity	E_b	880	880	880	650	650	650	880	880	880	880	MPa	Poliplex design book p 7-46 and 3-13 (corrected for 25°C)
1.5	Long-term ring-bending modulus of elasticity	E_{bL}	303	303	303	245	245	245	303	303	303	303	MPa	Poliplex design book p 7-46 and 3-13 (corrected for 28°C)
1.6	Diameter of neutral axis	D	0.1503	0.1812	0.1812	0.1450	0.1450	0.1450	0.1812	0.1879	0.1812	0.1812	m	
1.7	Moment of inertia for ring bending	I_{xx}	7.6056E-08	5.5372E-07	5.5372E-07	2.8407E-07	2.8407E-07	2.8407E-07	5.5372E-07	1.4947E-07	5.5372E-07	5.5372E-07	m^4/m	Equation 2.2.1.2
1.8	Initial (3-minute) ring-bending stiffness	S_{DI}	19712	81903	81903	60630	60630	60630	81903	19843	81903	81903	N/m/m	Equation 2.2.1.1(1)
1.9	Long-term ring-bending stiffness	S_{DL}	6794	28229	28229	22827	22827	22827	28229	6839	28229	28229	N/m/m	Equation 2.2.1.1(2)
2	**Soil moduli**													
2.1	Width of trench or embedment measured at the spring line	B	0.60	1.10	1.10	1.12	1.12	1.12	1.70	1.70	1.70	1.70	m	
2.2	Native soil modulus	E'_n	15	10	10	10	10	10	10	10	10	10	MPa	Given
2.3	Embedment soil modulus	E'_e	15	15	15	15	15	15	15	15	15	15	MPa	Given
2.4		Δ_f	1.16	1.43	1.43	1.57	1.57	1.57	1.67	1.67	1.67	1.67	-	Equation 3.4.3(3)
2.5	Leonhardt Correction Factor	ξ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	Equation 3.4.3(2)
2.6	Effective combined soil modulus	E'	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	MPa	Equation 3.4.3(1)
3	**Design loads due to trench fill and embankment fill**													
3.1	Cover, vertical distance between top of the pipe and the finished surface	H	37.00	38.00	58.00	61.60	70.00	75.00	63.00	49.00	68.50	67.50	m	Given (1 and 3). Estimated (2)
3.2	Assessed unit weight of trench fill or embankment fill	γ	13.2	13.2	13.2	11.3	11.3	11.3	11.3	11.3	11.3	11.3	kN/m^3	Assumed
3.3	Vertical design load (pressure at top of pipe) due to soil dead load	w_g	490	503	768	693	788	844	709	551	771	760	kPa	Equation 4.3
4	**Deflection**													
4.1	Bedding constant	K	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	-	Assumed value
4.2	Vertical design load due to surface-applied dead load	w_{gs}	0	0	0	0	0	0	0	0	0	0	kPa	Estimation
4.3	Wheel load (ΣP is the sum of the individual wheel loads)	P	510	510	510	510	510	510	510	510	510	510	kN	Largest likely plant
4.4	Length of wheel or track load contact area	a	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	m	Assumed
4.5	Width of wheel or track load contact area	b	4.4	4.4	4.4	4.4	4.4	4.4	4.4	4.4	4.4	4.4	m	Assumed
4.6	Allowable long-term vertical pipe deflection for non-pressure	Δ_{yall}/D	7.5%	7.5%	7.5%	7.5%	7.5%	7.5%	7.5%	7.5%	7.5%	7.5%	%	Poliplex design book p 7-52
4.7	Length of the base of the live load distribution, resulting from wheel or track loads, measured perpendicular to the direction of travel at the top of the pipe	L_1	58.05	59.50	88.50	93.72	105.90	113.15	95.75	75.45	103.73	102.28	m	Figure 4.2
4.8	Length of the base of the live load distribution, resulting from wheel or track loads, measured parallel to the direction of travel of the vehicle at the top of the pipe	L_2	54.05	55.50	84.50	89.72	101.90	109.15	91.75	71.45	99.73	98.28	m	Figure 4.2
4.9	Live load impact factor	α	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	-	Equation 4.7.2(2)
4.10	Vertical design load due to surface-applied live load	w_q	0.2	0.2	0.1	0.1	0.1	0.0	0.1	0.1	0.1	0.1	kPa	Equation 4.7.2(1)
4.11	Predicted long-term vertical deflection	Δ_y/D	5.1%	4.4%	6.7%	6.3%	7.2%	7.7%	6.2%	5.7%	6.8%	6.7%	%	Equation 5.2(2)
4.12	**Is $\Delta_y/D \le \Delta_{yall}/D$?**		**YES**	**YES**	**YES**	**YES**	**YES**	**PROCEED TO FINITE ELEMENT ANALYSIS**	**YES**	**YES**	**YES**	**YES**		
5	**External loadings**													
5.1	Allowable long-term ring-bending strain	ε_{ball}	4.0%	4.0%	4.0%	4.0%	4.0%	4.0%	4.0%	4.0%	4.0%	4.0%	%	Poliplex design book p 7-52
5.2	Effective wall thickness	t_{es}	0.0097	0.0188	0.0188	0.01505	0.01505	0.01505	0.0188	0.01215	0.0188	0.0188	m	Poliplex design book p 6-3
5.3	Shape factor	D_f	4.39	3.40	3.40	3.49	3.49	3.49	3.40	4.38	3.40	3.40	-	Equation 5.3.1(3)
5.4	Predicted long-term ring-bending strain	ε_b	1.4%	1.6%	2.4%	2.3%	2.6%	2.8%	2.2%	1.6%	2.4%	2.4%	%	Equation 5.3.1(2)
5.5	**Is $\varepsilon_b \le \varepsilon_{ball}$?**		**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**		
6	**Internal pressure**													
6.1	Internal working pressure	P_w	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	MPa	Given
6.2	Allowable long-term internal pressure	P_{all}	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	MPa	Poliplex design book p 3-9 using SDR=13.6 and safety factor of 1.25
6.3	**Is $P_w \le P_{all}$?**		**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**		
7	**Combined loading**													
7.1	Factor of safety for long-term combined external load and internal pressure (combined loading)	η	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	-	Poliplex design book p 7-54
7.2	Factor of safety for log-term internal pressure	η_p	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	-	Poliplex design book p 3-4 (lower typical value)

Client:	SUEZ	**Job Number:**	21-27038	**Revision:**	4
Project:	Elizabeth Drive Landfill	**Calcs by:**	A Roberts	**Date:**	28/01/2020
Subject:	Leachte Collection Pipe Calculations	**Checked by:**		**Date:**	

Statement of design procedure

This spreadsheet provides design calculations for the structural capacity of leachate collection pipe

References

AS2566.1 Buried flexible pipelines - part 1: structural design & 2D PLAXIS modelling

Item	Description	Symbol	1	2	2	3	4	4	5	5	5	5	Unit	Notes
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7.3	Factor of safety for long-term ring-bending strain	η_b	2	2	2	2	2	2	2	2	2	2	-	Poliplex design book p 7-57
7.4	Re-rounding coefficient	r_c	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.997	-	Section 5.3.3
7.5	$P_w/\eta_p P_{all} + r_c \varepsilon_b/\eta_b \varepsilon_{ball}$		0.1833	0.1992	0.3013	0.2903	0.3292	0.3524	0.2785	0.2057	0.3024	0.2980	-	Equation 5.3.3
7.6	$1/\eta$		0.667	0.667	0.667	0.667	0.667	0.667	0.667	0.667	0.667	0.667	-	Equation 5.3.3
7.7	**Is $P_w/\eta_p P_{all} + r_c \varepsilon_b/\eta_b \varepsilon_{ball} \leq 1/\eta$?**		**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**		
8	**Buckling**													
8.1	Height of water surface above the top of the pipe	H_w	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	m	Given
8.2	Assessed unit weight of liquid external to the pipe	γ_L	10	10	10	10	10	10	10	10	10	10	kN/m^3	Assumed
8.3	Internal vacuum	q_v	0	0	0	0	0	0	0	0	0	0	kPa	Assumed
8.4	Design factor for buckling	F_s	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	-	Poliplex design book p 7-57
8.5	Poisson's ratio	v	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	-	Poliplex design book p 3-24
8.6	Specific gravity of soil particle	ρ_s	2.65	2.65	2.65	2.65	2.65	2.65	2.65	2.65	2.65	2.65	kN/m^3	Assumed (from p 25 of standard)
8.7	Allowable buckling pressure, based on pipe alone	q_{all1}	97	403	403	326	326	326	403	98	403	403	kPa	Equation 5.4(4)
8.8	Allowable buckling pressure, based on pipe/embedment interaction	q_{all2}	576	926	926	863	863	863	926	577	926	926	kPa	Equation 5.4(5)
	Buckling pressure used to calculate factor of safety		495	510	774	700	794	862	719	572	781	770		
8.9	Allowable buckling pressure for material	q_{all}	576	926	926	863	863	863	926	577	926	926	kPa	Max of q_{all1} and q_{all2}
8.10	Submerged unit weight of trench fill or embankment fill	γ_{sub}	8.24	8.24	8.24	7.01	7.01	7.01	7.01	7.01	7.01	7.01	kN/m^3	Equation 5.4(2)
	Calculated factor of safety		2.3	3.6	2.4	2.5	2.2	2.0	2.6	2.0	2.4	2.4		
8.11	$\gamma(H-H_w)+(\gamma_L+\gamma_{sub})(D_e/2+H_w)+w_{gs}+w_q+q_v$		496	509	774	699	794	850	715	558	777	766	kPa	Equation 5.4(1)
8.12	**Is $\gamma(H-H_w)+(\gamma_L+\gamma_{sub})(D_e/2+H_w)+w_{gs}+w_q+q_v \leq q_{all}$?**		**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**	**YES**		

PLAXIS Finite Element Analysis

1. Introduction

GHD Pty Ltd (GHD) has carried out geotechnical analysis to assess the deformation and related ground stresses induced on the MDPE leachate pipe by overlying waste materials which are to be placed as a result of landfill activity.

2. Numerical model

2.1 Subsurface profile and geotechnical model

The original subsurface profile was assessed by considering the following:

- In the absence of geotechnical investigation data, the original subsurface profile prior to the cell excavation was assessed by considering the following:
    - Penrith 1:100,000 geological map (Series xxxx) indicates that the overall landfill site is located in the vicinity of the geological boundary between Quaternary Alluvium and Bringelly Shale of the Wianammatta Formation
    - The relevant bore details (Bore ID 072774.1.1) available in the public domain (Australian Groundwater Explorer by Bureau of Meteorology) indicates that the site is underlain by about 5-m thick clayey soil over shale bedrock
- Cell D1 shown in the supplied survey plan of the approximate clay liner (ref. 28727 TOTAL CLAY 15-5-20 dated 27 May 2015) has been adopted in our analysis
- Total waste fill thickness is 76 m measured above the top of gravel layer

The design parameters were obtained by considering the following:

- Initial unit weight of waste materials was assumed to be 11.3 kN/m3. This value is typical average given in Zekkos et al. (2006) and consistent with average value reported in GHD letter correspondence (ref. 21/17412/142306 dated 18 July 2008) for the application to modify consent notice no. 451/89
- With regards to the waste properties, it is assumed that the waste will comprise typically Municipal Solid Waste with some cover layers. As a result, the parameters were adopted as per the following:
    - Compressibility parameters was adopted by using correlation given in Bareither et al. (2012) with a Waste Compressibility Index value of about 0.1.
    - Strength parameters was adopted by using correlation given in Bareither et al (2012)
    - A relatively small Over Consolidation Ratio (OCR) value of 2 was assumed to represent initial placement of nominally compacted waste layer.

Summary of subsurface profile and adopted design parameters are shown in Table 1 below.

Table 1 – Summary of geotechnical model

Layer	Unit weight (bulk) kN/m^3	Strength Parameters		Stiffness Parameters		Initial OCR
		Peak friction angle (deg.)	Effective cohesion (kPa)	Compression Index ($c_c/1+\ e_0$)	Equivalent E' (MPa) – approx.	
Waste materials	11.3	35 (Note 2)	5 (Note 2)	0.2 (Note 1)	2 (Note 1)	2 (Note 1)
Bedding Gravel	19	40	0	N/A	15	N/A
Compacted Clay	18	26	6	N/A	10	N/A
Note:						

Layer	Unit weight (bulk) kN/m³	Strength Parameters		Stiffness Parameters		Initial OCR
		Peak friction angle (deg.)	Effective cohesion (kPa)	Compression Index ($c_c/1 + e_0$)	Equivalent E' (MPa) – approx.	

1. Associated with initial condition (i.e. placement of waste – before decomposition and self-weight settlement take place)
2. See below graphs (Bareither et al. 2012)

2.2 MDPE leachate pipe

The configuration of compacted clay liner and gravel layer on which the MDPE pipe is to be placed is shown in Figure 1 below.

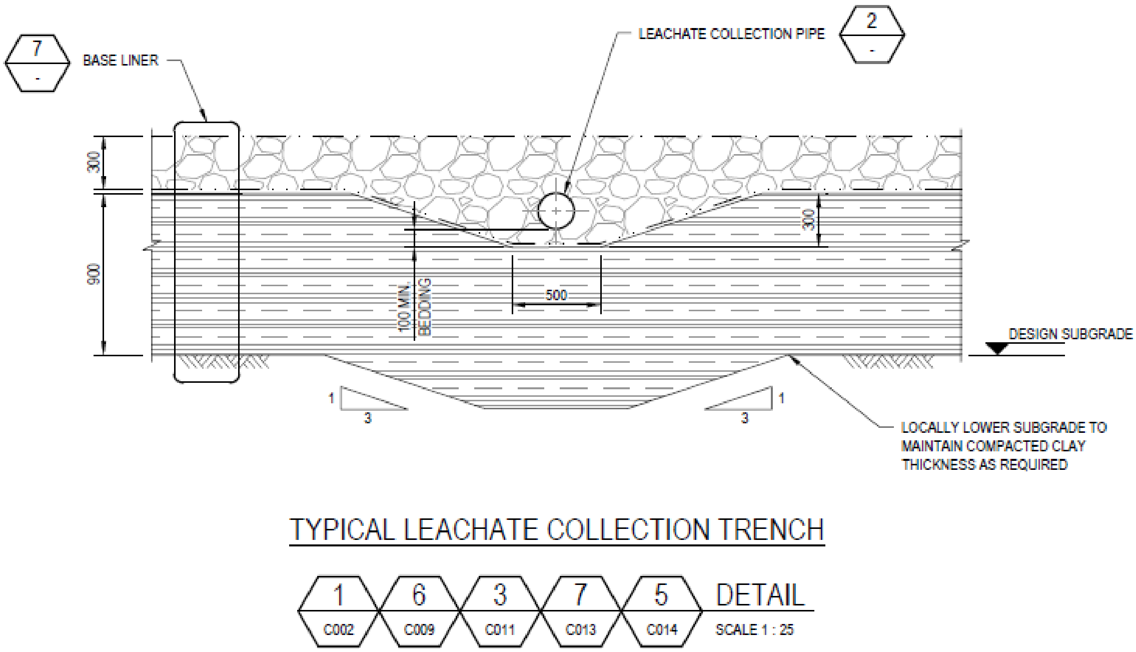

Figure 1 – Configuration of compacted clay liner and gravel layer with MDPE pipe

In our analysis, the following properties have been adopted:

- Pipe external diameter of 160 mm and thickness of 14.6 mm
- Young's Modulus of 247 MPa and 650 MPa

3. Assessment Methodology

GHD has undertaken a Finite Element Analysis (FEA) by using a commercially available program PLAXIS 2D to assess the impact of waste placement to the proposed MDPE leachate pipe. The following points are noted:

- Pipe was represented by a circular plate element with appropriate stiffness and thickness.
- Waste materials were modelled by using soft soil model to allow time-dependent change in stress state but the secondary (creep) settlement was excluded.
- Other materials were represented by Mohr Coulomb model.

With respect to the position of the proposed MDPE pipe, we have adopted 2 possible governing scenario where the stress conditions vary due to difference in the confinement. These scenario comprise the pipe located at the toe and pipe located at the centre of cell. As a result, 4 analysis have been conducted:

- Pipe with E value of 247 MPa located at the toe
- Pipe with E value of 247 MPa located at the centre
- Pipe with E value of 650 MPa located at the toe
- Pipe with E value of 650 MPa located at the centre

It is assumed that the waste materials were placed in 9 successive layers within a period of 1.5 years. Following a placement of final layer, additional time of 2.5 years was allowed to allow for the completion of remaining primary consolidation (i.e. reduction in void ratio).

4. Assessment Results

Summary of assessed change in pipe diameter is given in Table 2 below.

Table 2 – Assessed change in MDPE pipe diameter

Case	Description	Change in diameter measured vertically between crown and invert point (mm)	Change in diameter measured horizontally between leftmost and rightmost point (mm)
1	160-mm dia. Pipe with E = 247 MPa at the centre	5.7	0.6
2	160-mm dia. Pipe with E = 650 MPa at the centre	3.6	0.6
3	160-mm dia. Pipe with E = 247 MPa at the toe	3.4	0.7
4	160-mm dia. Pipe with E = 650 MPa at the toe	2.3	0.6

From the FEA, the orthogonal ground stresses (σ'_{xx} and σ'_{yy}) adjacent to the MDPE pipe were obtained for 4 analysis. These stresses were plotted below.

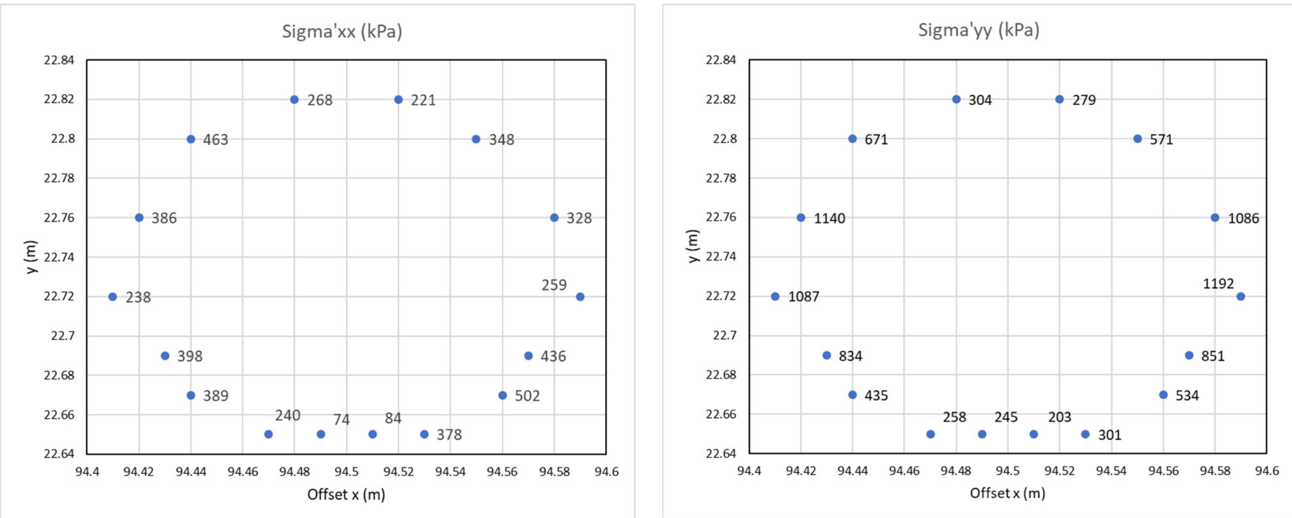

Figure 2a and 2b – Horizontal and vertical orthogonal ground stresses for 160-mm pipe with E = 247 MPa located at the centre

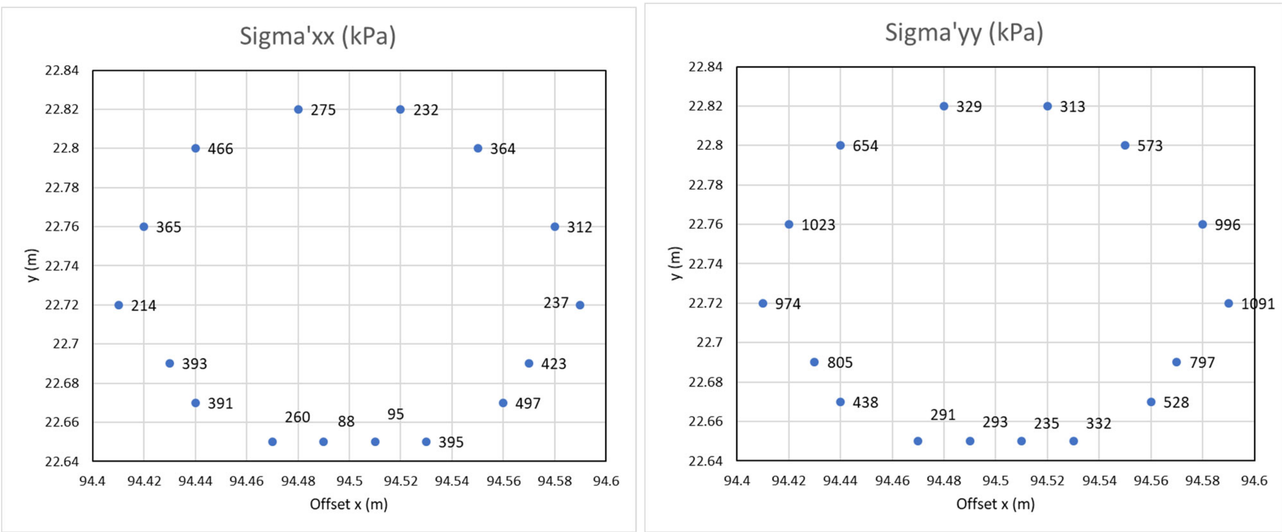

Figure 3a and 3b – Horizontal and vertical orthogonal ground stresses for 160-mm pipe with E = 650 MPa located at the centre

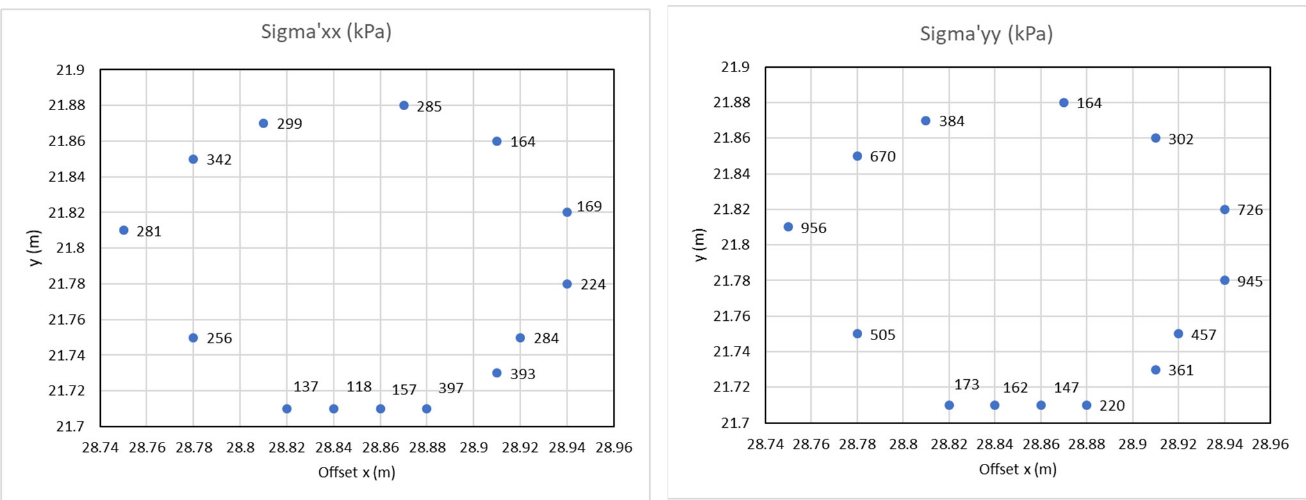

Figure 4a and 4b – Horizontal and vertical orthogonal ground stresses for 160-mm pipe with E = 247 MPa located at the toe

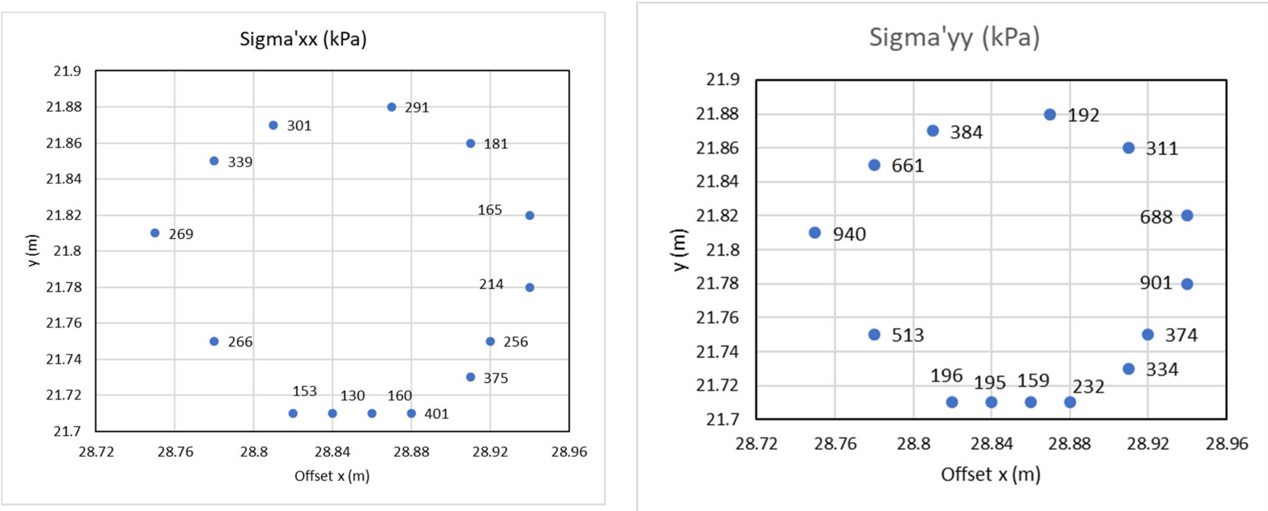

Figure 5a and 5b – Horizontal and vertical orthogonal ground stresses for 160-mm pipe with E = 650 MPa located at the toe

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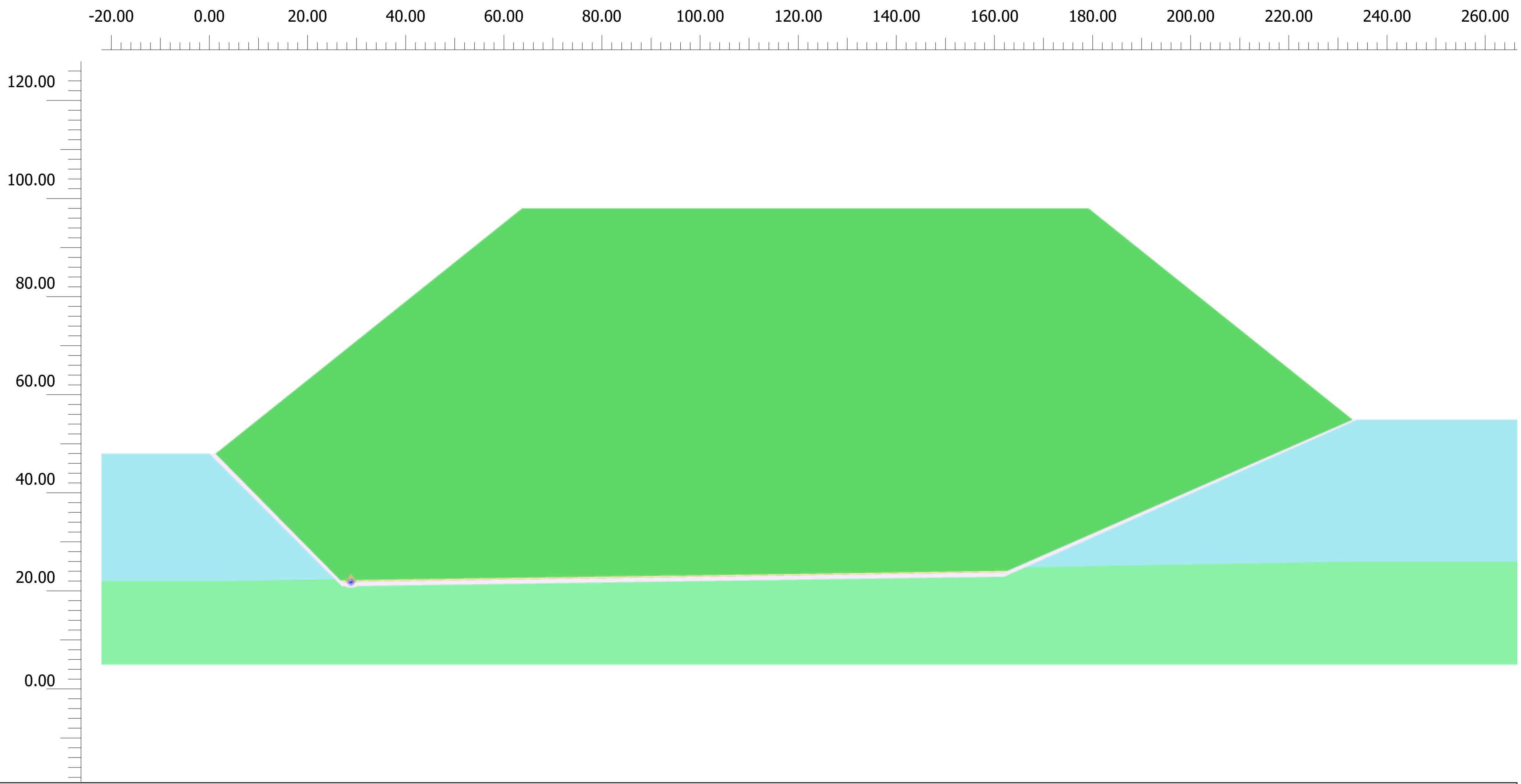

Connectivity plot

Project description	Date
Elizabeth Drive - Model overview for Pipe at cell toe	1/9/2020

Project filename	Step	Company
A4c2	102	

Appendix C – Protection geotextile Design Procedures

Client:	SUEZ	**Job Number:**	21/27038		
Project:	EDL EIS	**Calcs by:**	A Roberts	**Date:**	3/10/2019
Subject:	Protection geotextile	**Checked by:**		**Date:**	

Statement of design procedure

This spreadsheet provides design calculations for the protection geotextile thickness

References

Designing with Geosynthetics (5th Edition), Robert M. Koerner

Barrier Systems for Waste Disposal (2nd Edition) Rowe et al

Design cases / assumptions

1. Cell base membrane protection - restricted waste cells
2. Not used
3. Not used
4. Not used

Item	**Description**	**Symbol**	**Case**				**Unit**	**Notes**
			1	**2**	**3**	**4**		
1	Factor of safety	FOS	3					Refer table 13.3 pg 412 (Rowe, 2004)
2	Height of fill	h	42				m	From design
3	Density of waste	γ	10.9				kN/m^3	Calculated
4	Slope of batter	θ	1.7				degrees	From design
5	Pressure allow	p_{app}	457.8				kN/m2	Refer table 13.3 pg 412 (Rowe, 2004)
6	Protrusion height	Hp	0.019				m	Half the particle size
7	Protrusion shape		Subrounded					Assumed
8	Modification factor for protrusion shape	MF_S	0.5					Refer table 13.3 pg 412 (Rowe, 2004)
9	Packing density		Dense, 38 mm					Assumed
10	Modification factor for packing density	MF_{PD}	0.83					Refer Table 5.18, pg 548 (Koerner, 2005)
11	Arching in solids		Geostatic, shallow					Assumed
12	Modification factor for arching in solids	MF_A	0.75					Refer table 13.3 pg 412 (Rowe, 2004)
13	Factor for creep	RFcr	1.5					Refer Table 5.18, pg 548 (Koerner, 2005)
14	Leachate strength		Harsh Leachate					Assumed
15	Factor for degradation	RFcbd	1.5					
16	**Minimum mass of geotextile neccesary**		**730**				g/m2	

Appendix D – Global slope stability analysis

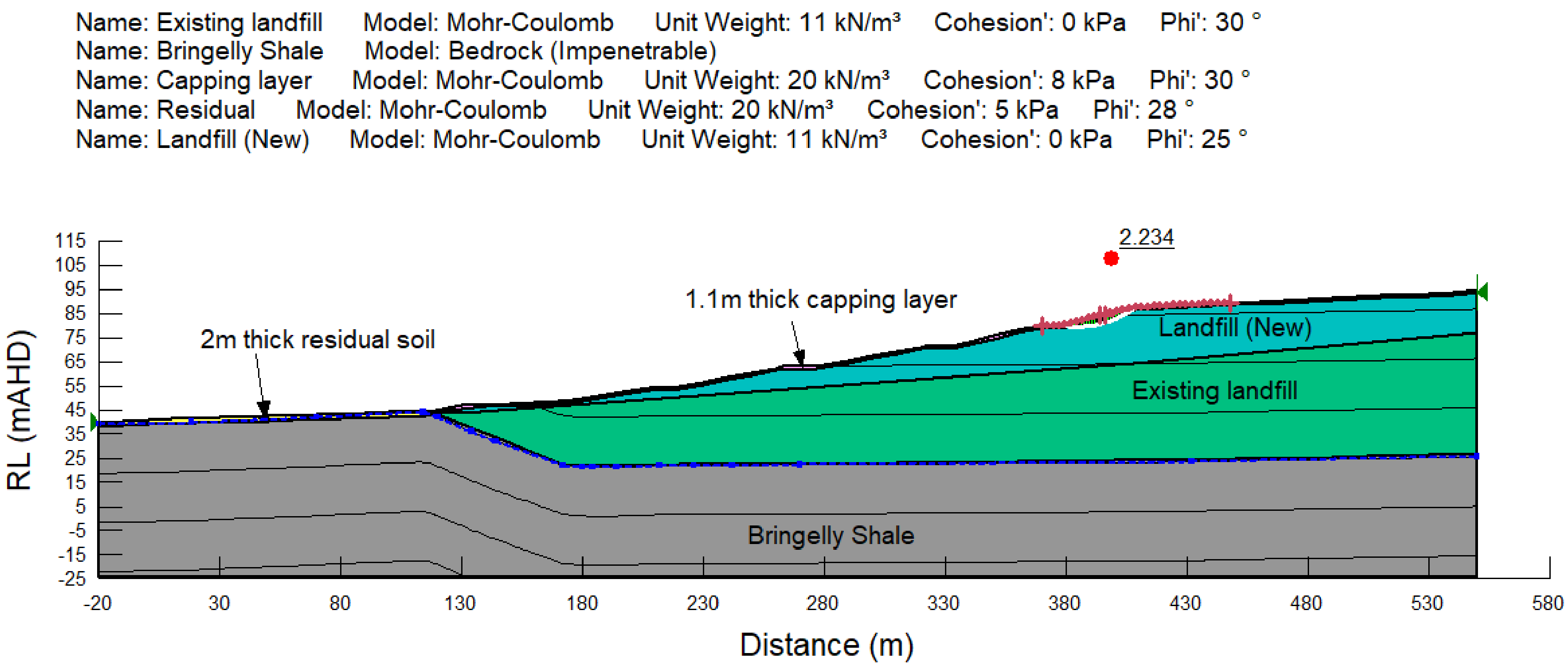

Figure A 1 Case 1 slope stability assessment at the top bench

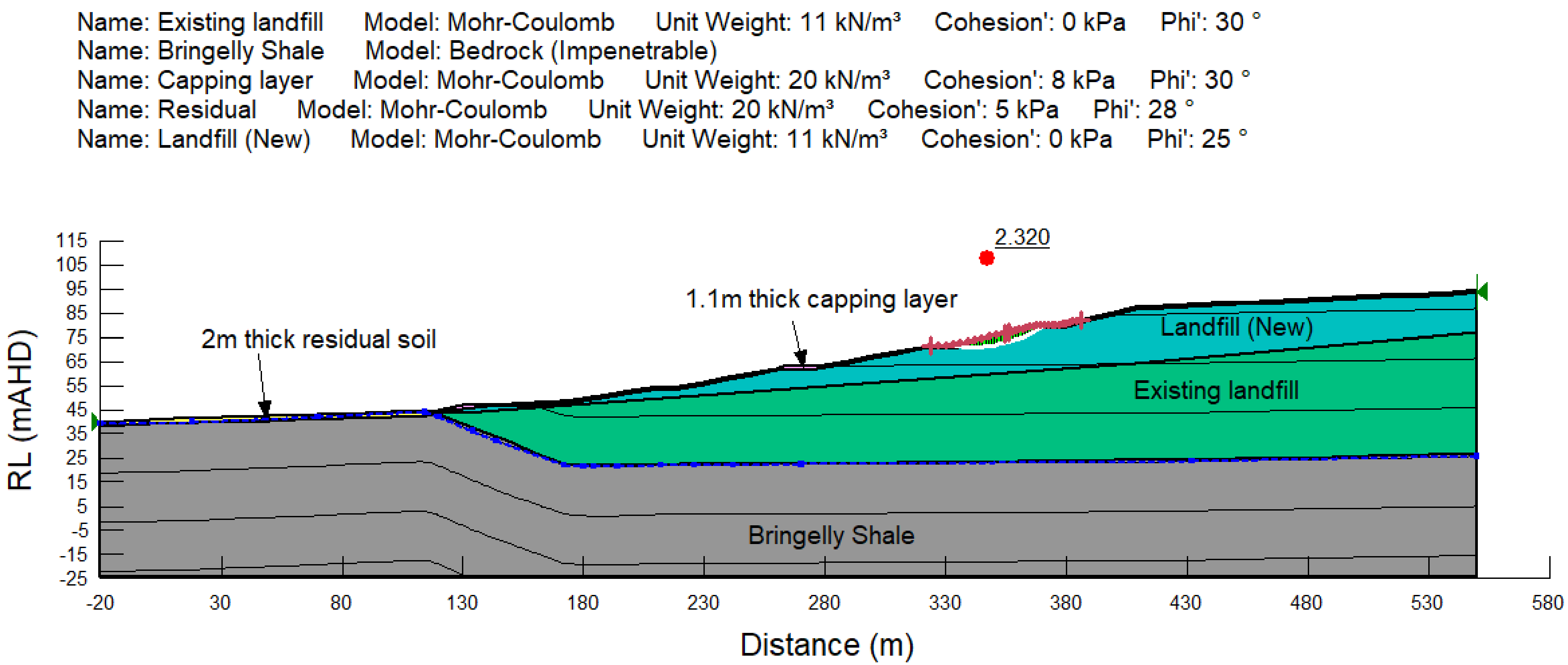

Figure A 2 Case 1 slope stability assessment at the 2nd bench

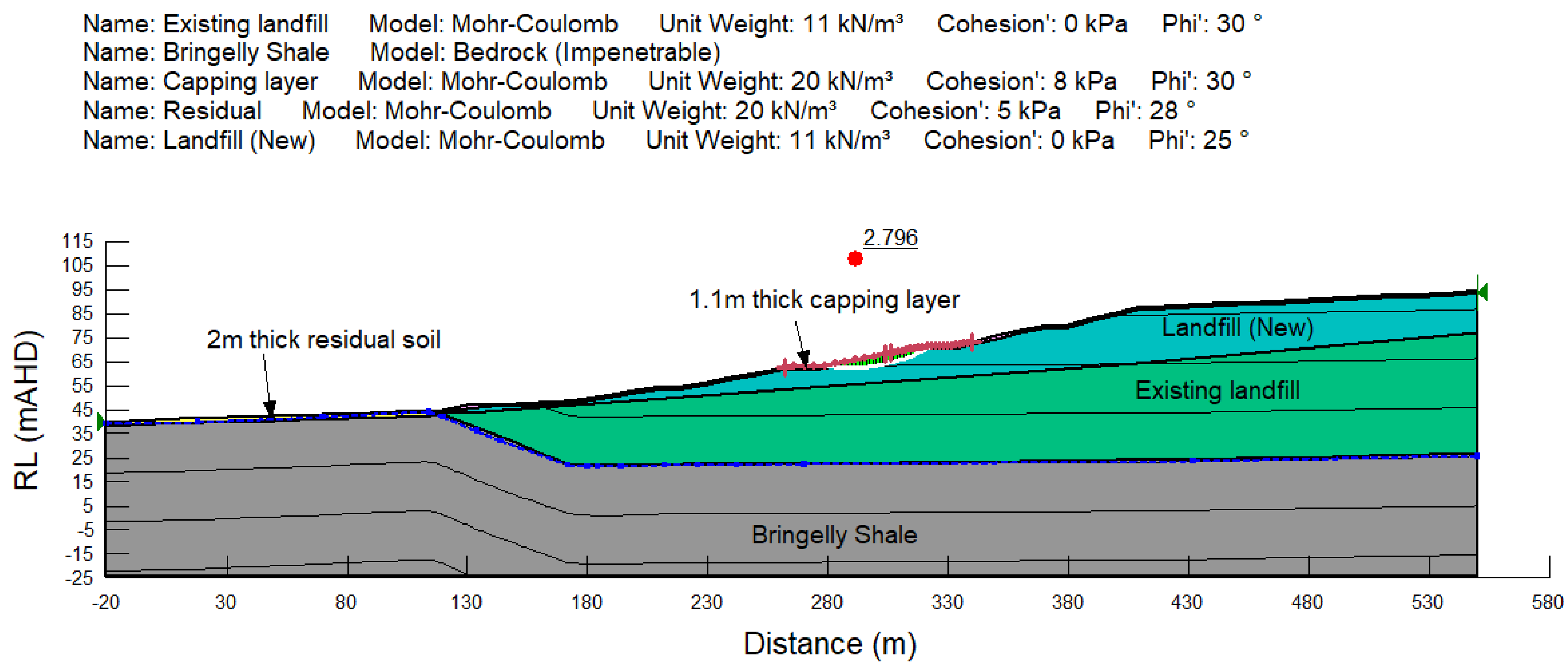

Figure A 3 Case 1 slope stability assessment at the 3rd bench

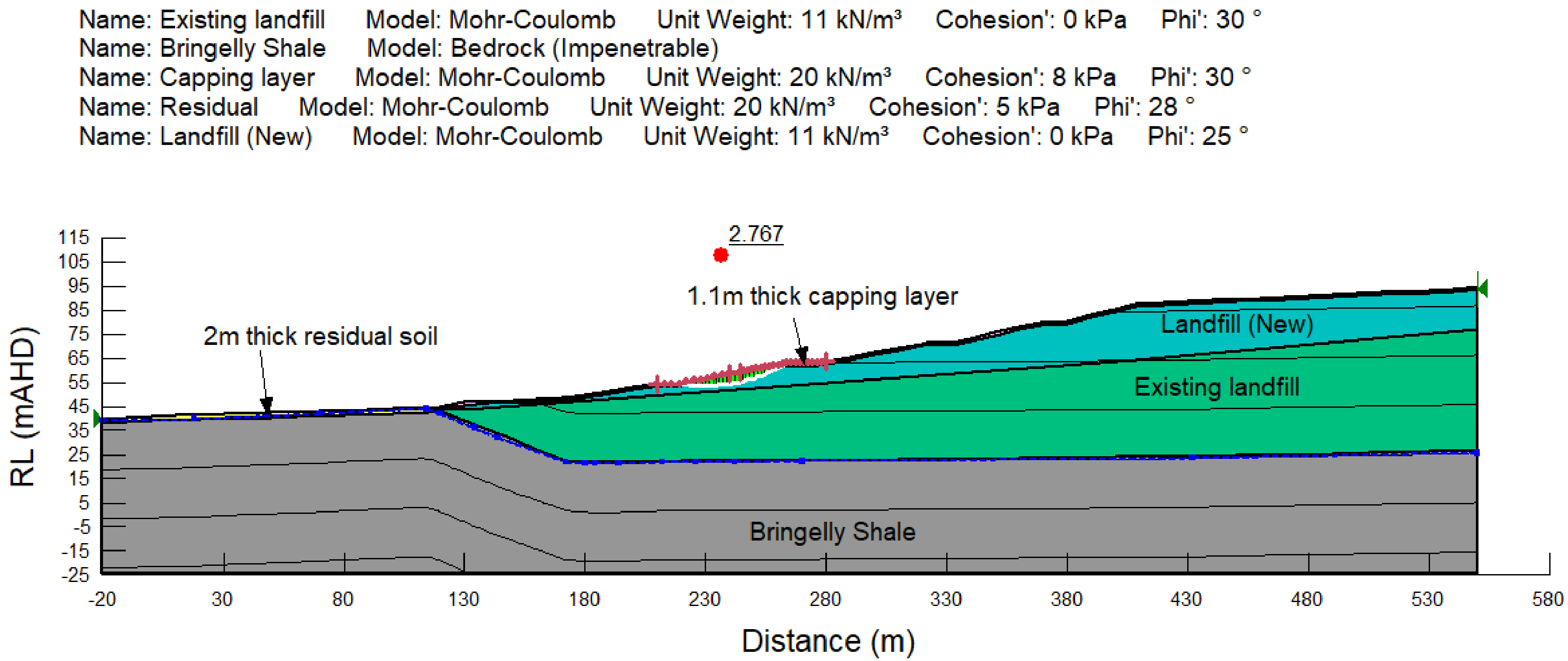

Figure A 4 Case 1 slope stability assessment at the 4th bench

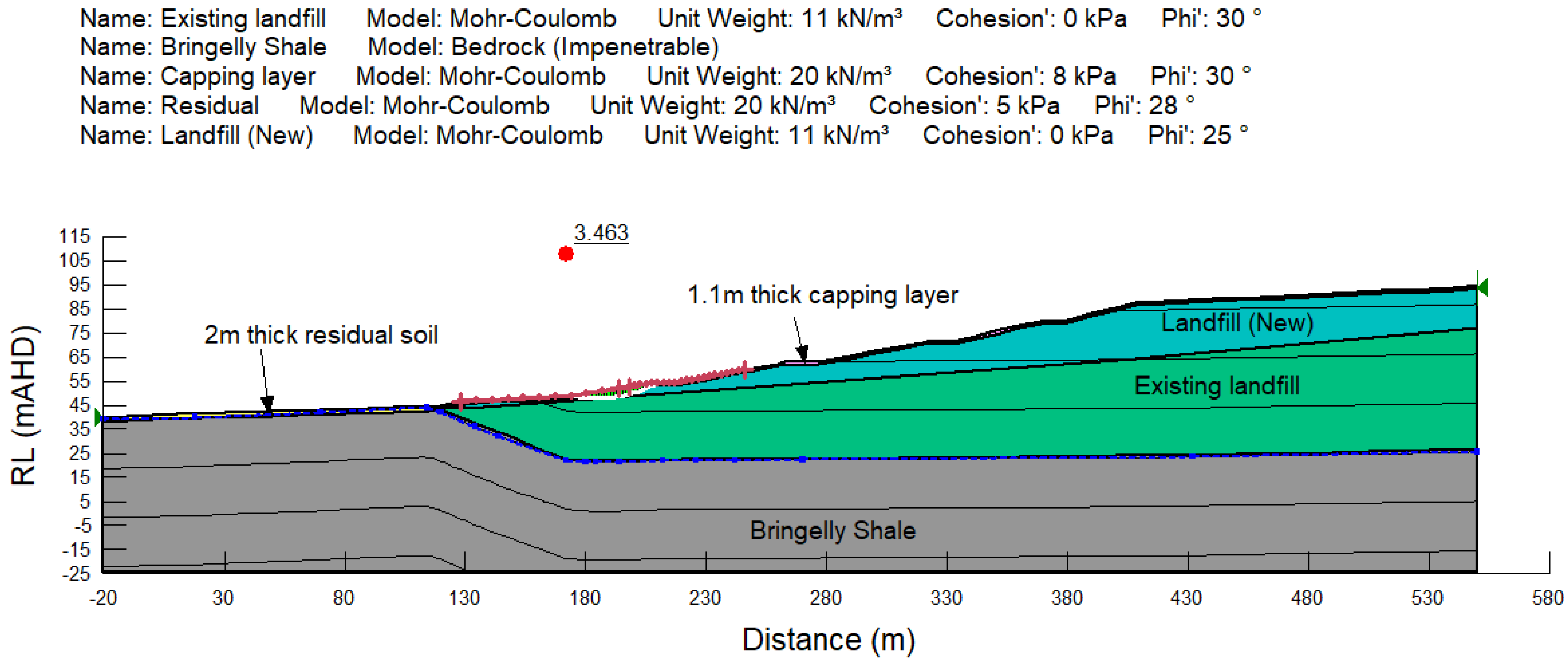

Figure A 5 Case 1 slope stability assessment at the 5th bench

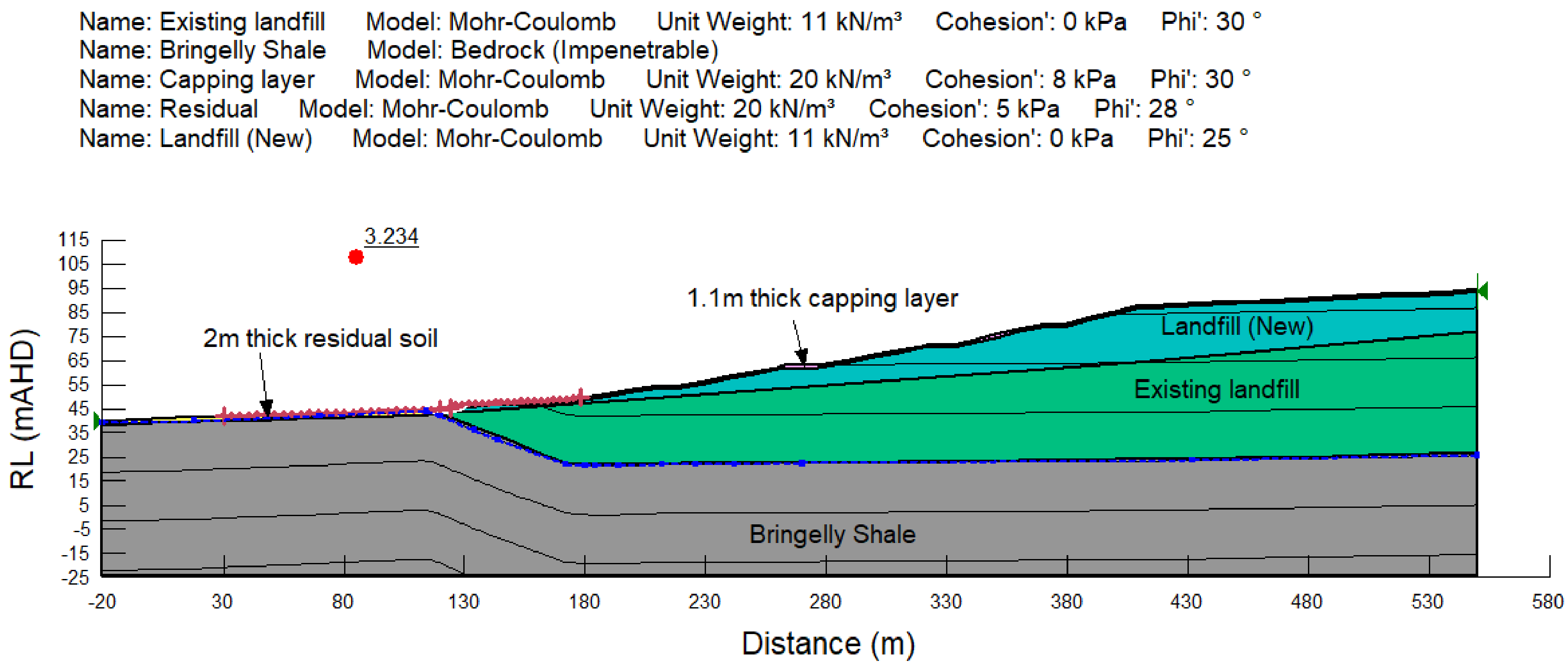

Figure A 6 Case 1 slope stability assessment at the bottom bench

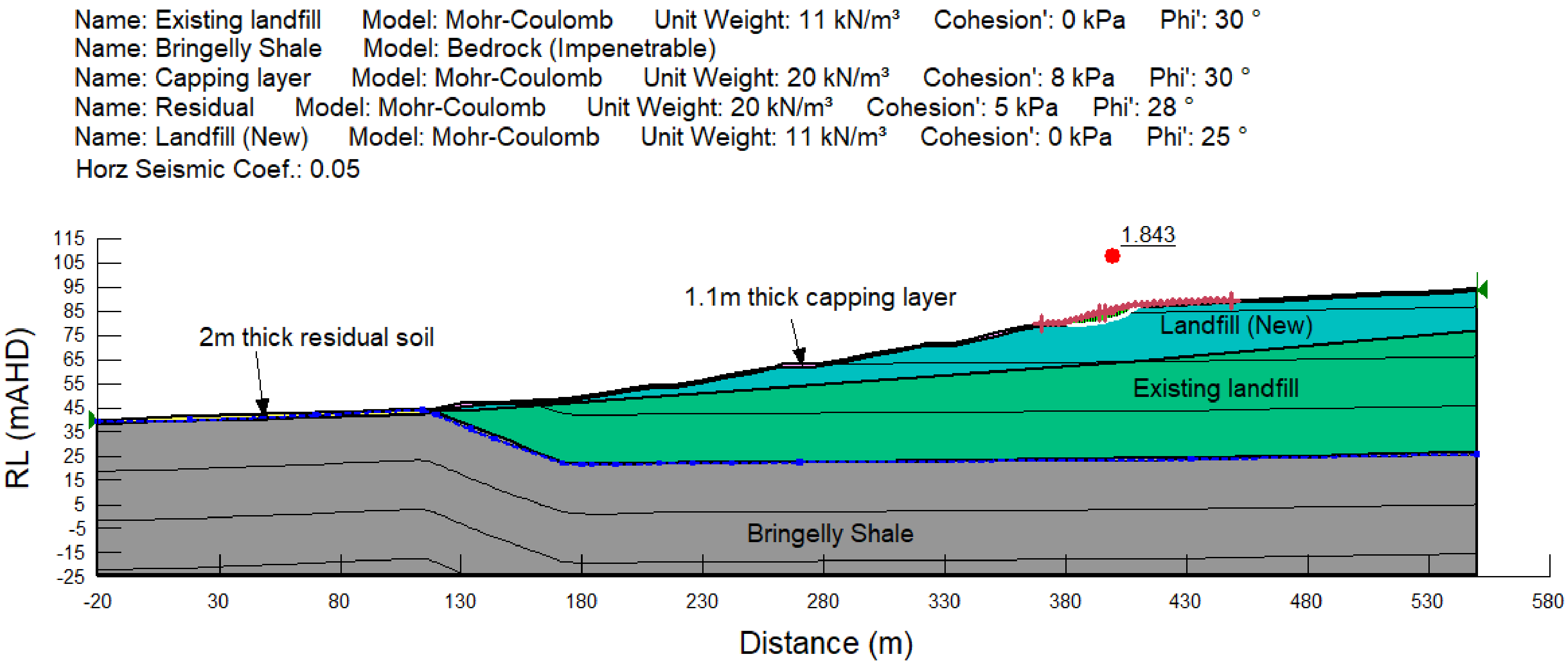

Figure A 7 Case 1 slope stability assessment for earthquake scenario at the most critical failure surface

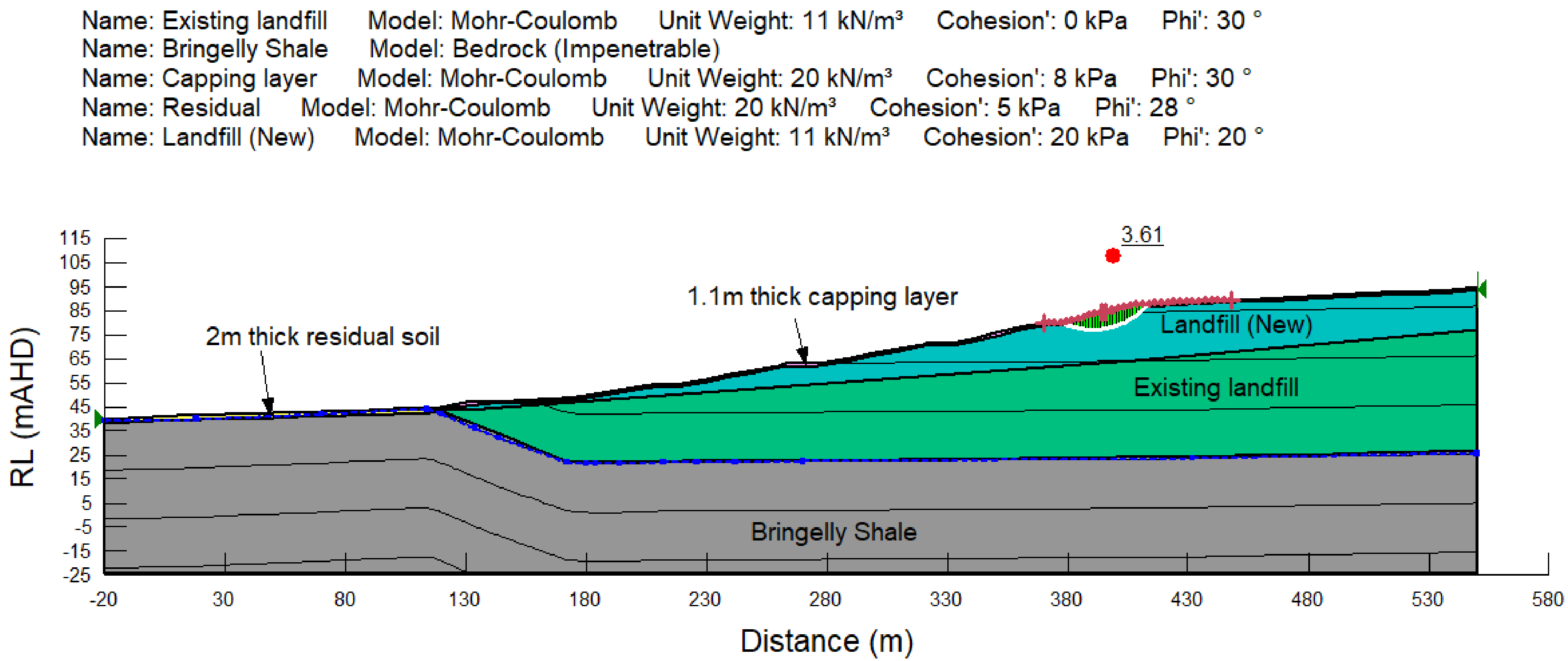

Figure A 8 Case 2 slope stability assessment at the top bench

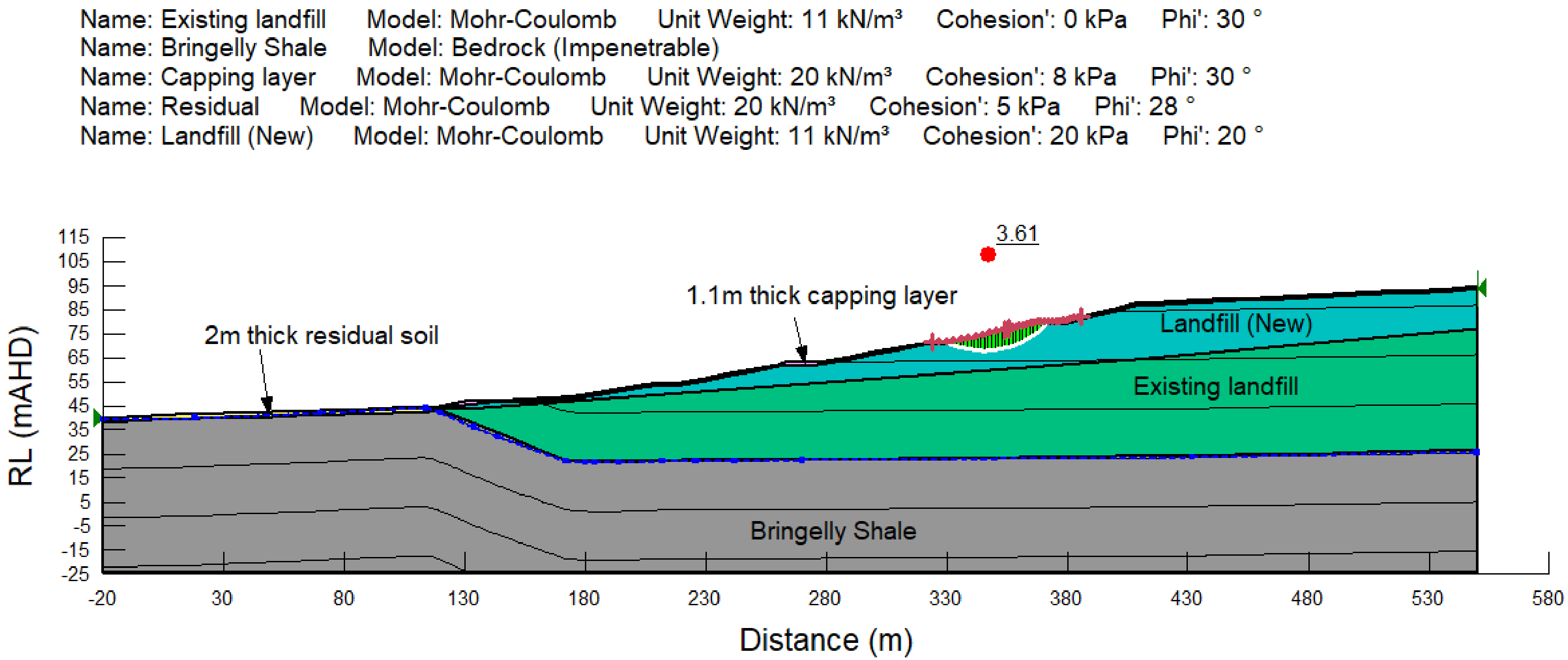

Figure A 9 Case 2 slope stability assessment at the 2nd bench

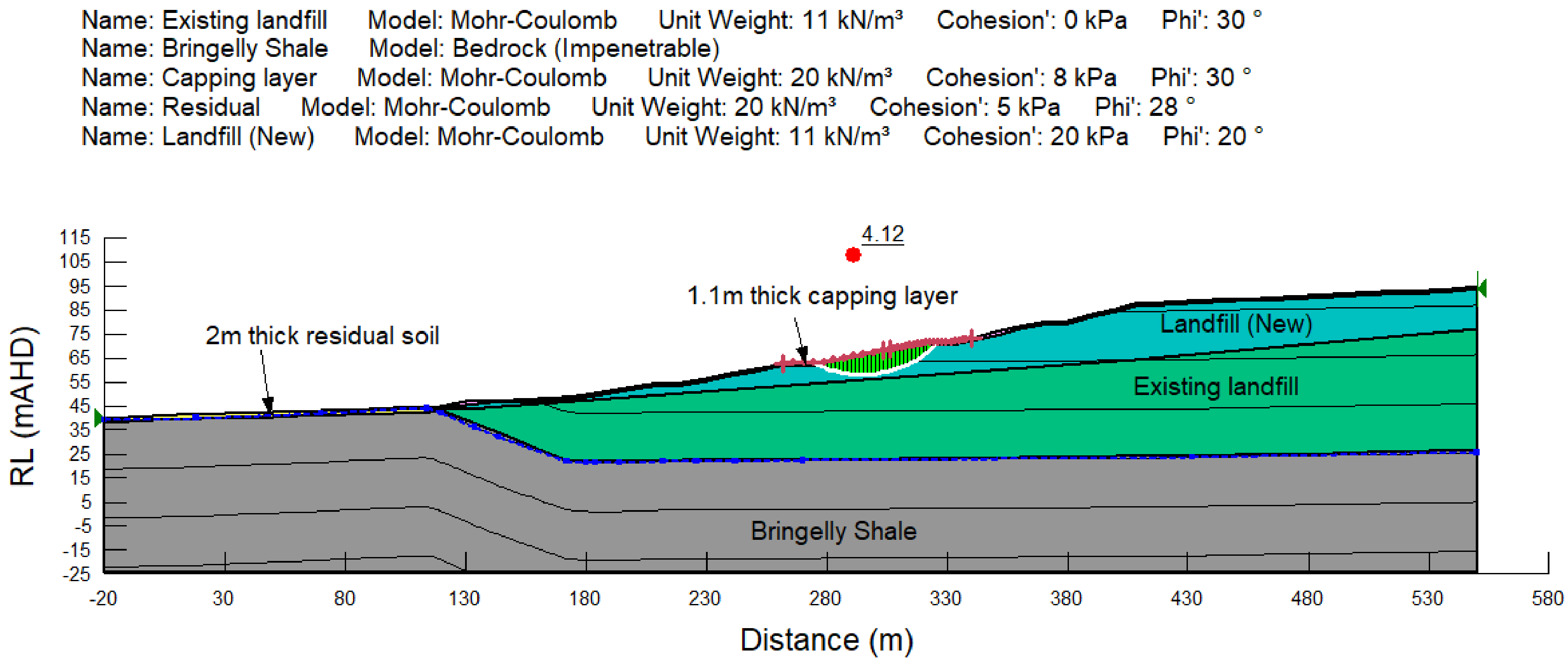

Figure A 10 Case 2 slope stability assessment at the 3rd bench

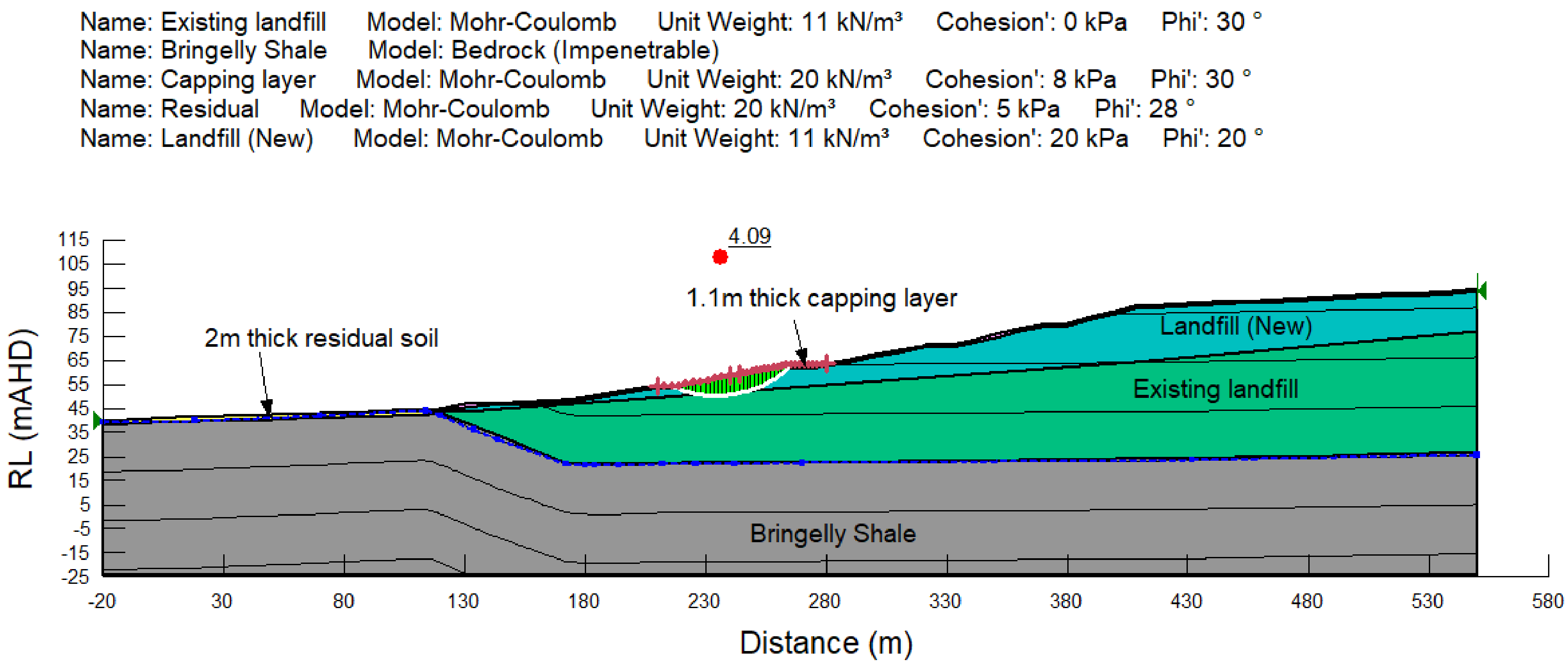

Figure A 11 Case 2 slope stability assessment at the 4th bench

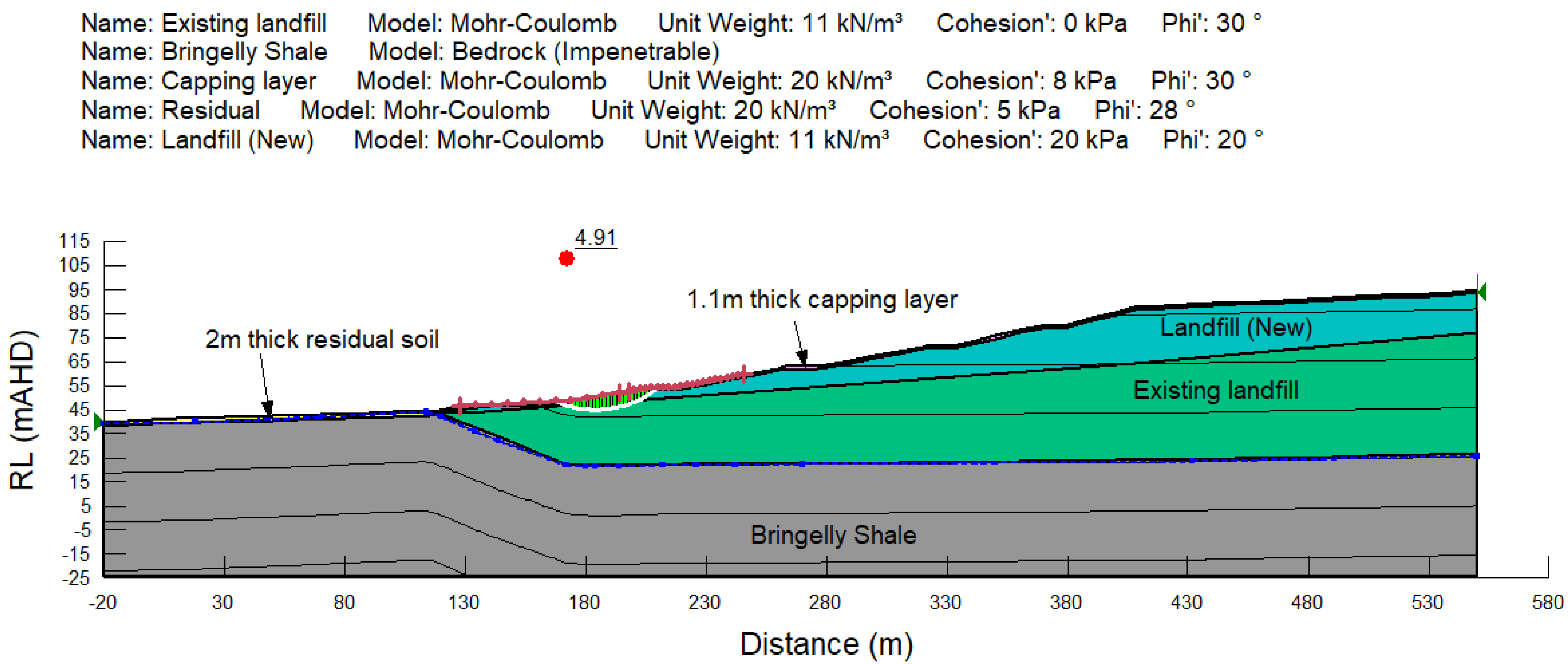

Figure A 12 **Case 2 slope stability assessment at the 5th bench**

Name: Existing landfill Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bringelly Shale Model: Bedrock (Impenetrable)
Name: Capping layer Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 8 kPa Phi': 30 °
Name: Residual Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 5 kPa Phi': 28 °
Name: Landfill (New) Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 20 kPa Phi': 20 °

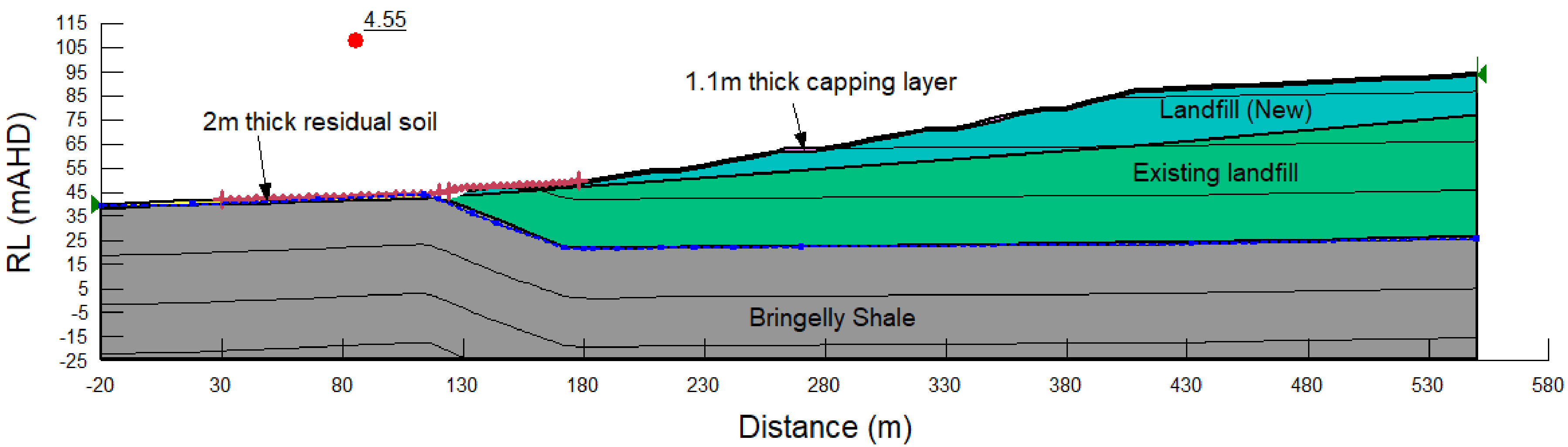

Figure A 13 **Case 2 slope stability assessment at the bottom bench**

Name: Existing landfill Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bringelly Shale Model: Bedrock (Impenetrable)
Name: Capping layer Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 8 kPa Phi': 30 °
Name: Residual Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 5 kPa Phi': 28 °
Name: Landfill (New) Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 40 kPa Phi': 12 °

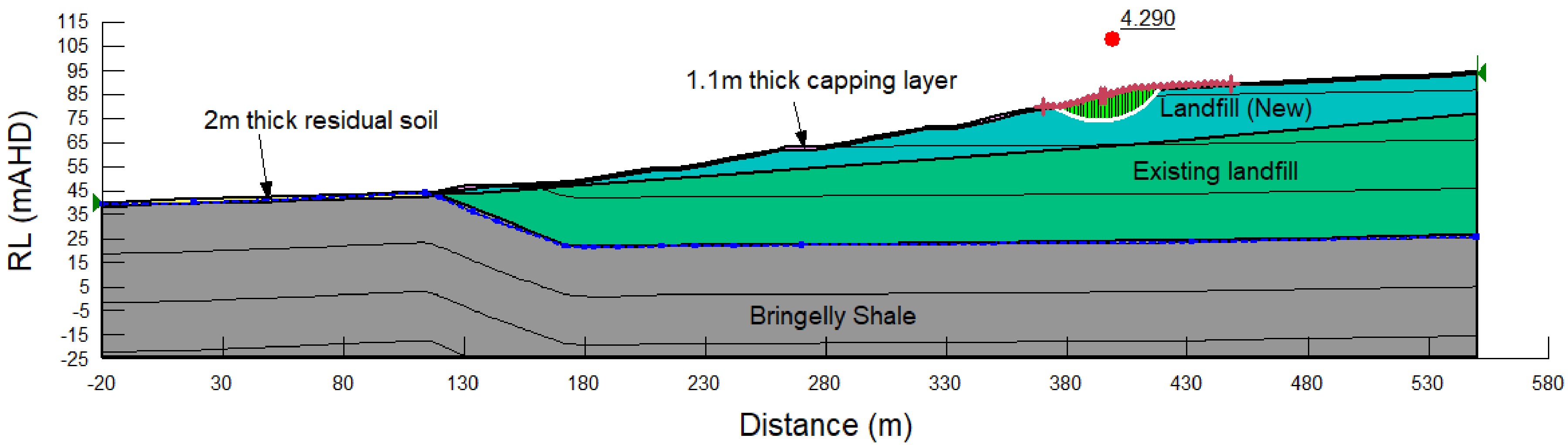

Figure A 14 **Case 3 slope stability assessment at the top bench**

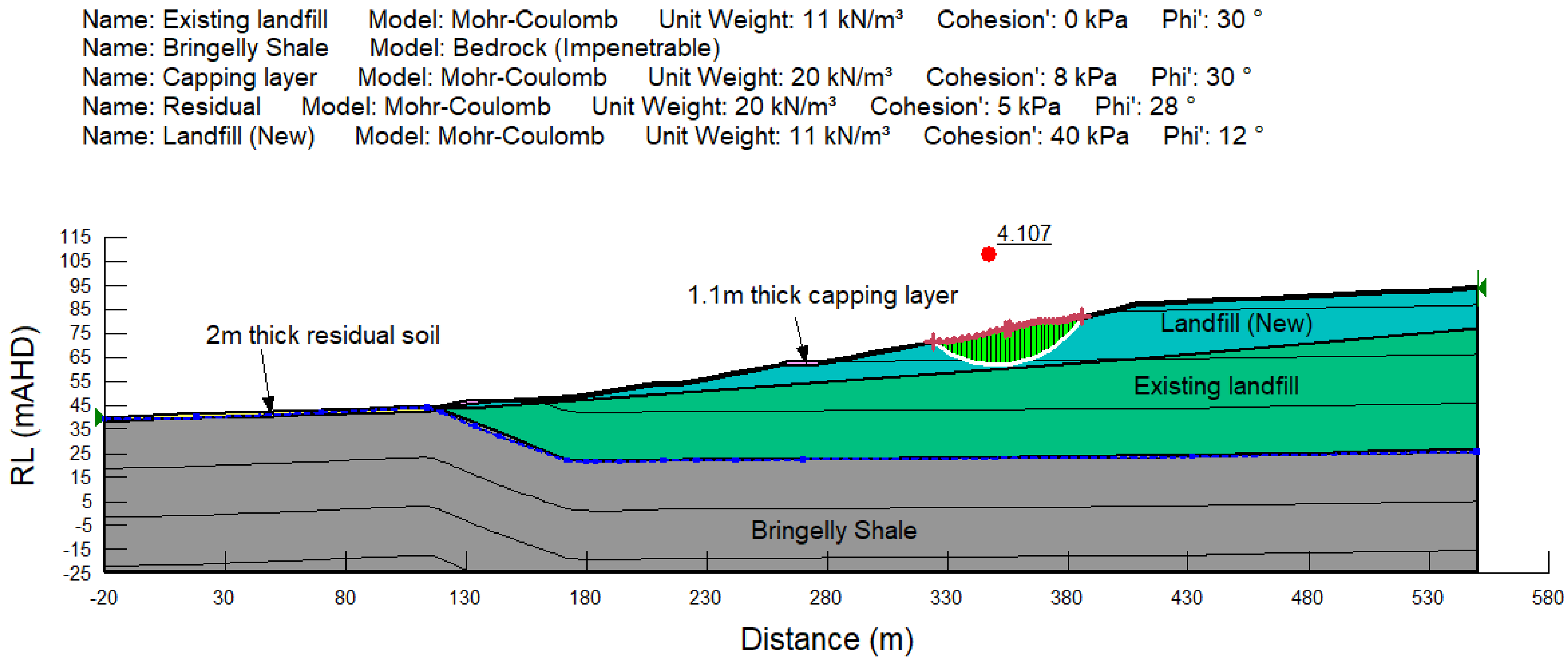

Figure A 15 **Case 3 slope stability assessment at the 2nd bench**

Name: Existing landfill Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bringelly Shale Model: Bedrock (Impenetrable)
Name: Capping layer Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 8 kPa Phi': 30 °
Name: Residual Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 5 kPa Phi': 28 °
Name: Landfill (New) Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 40 kPa Phi': 12 °

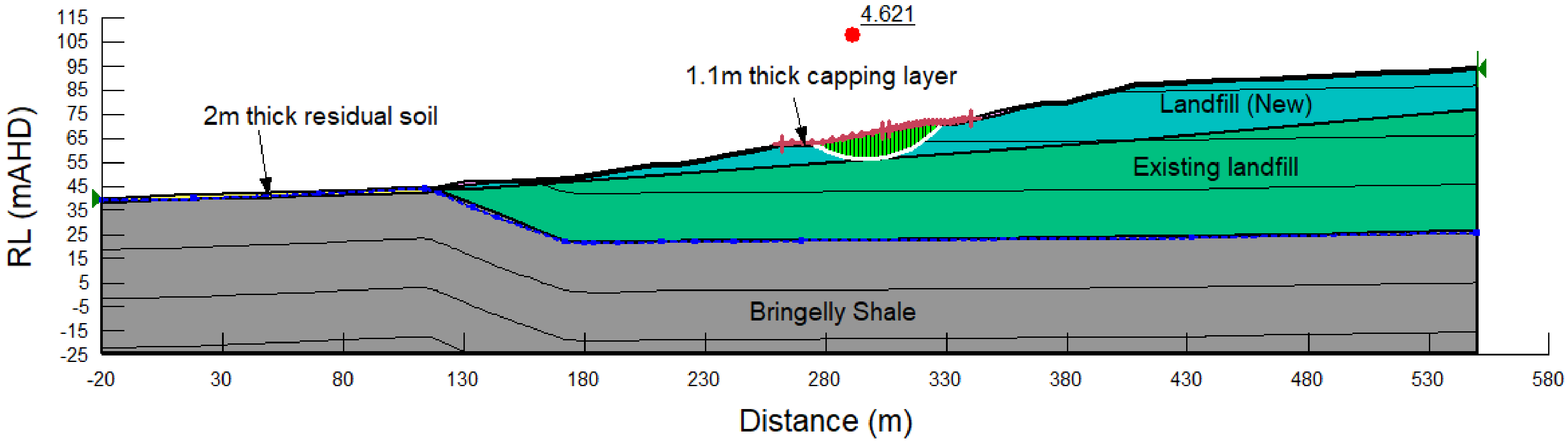

Figure A 16 **Case 3 slope stability assessment at the 3rd bench**

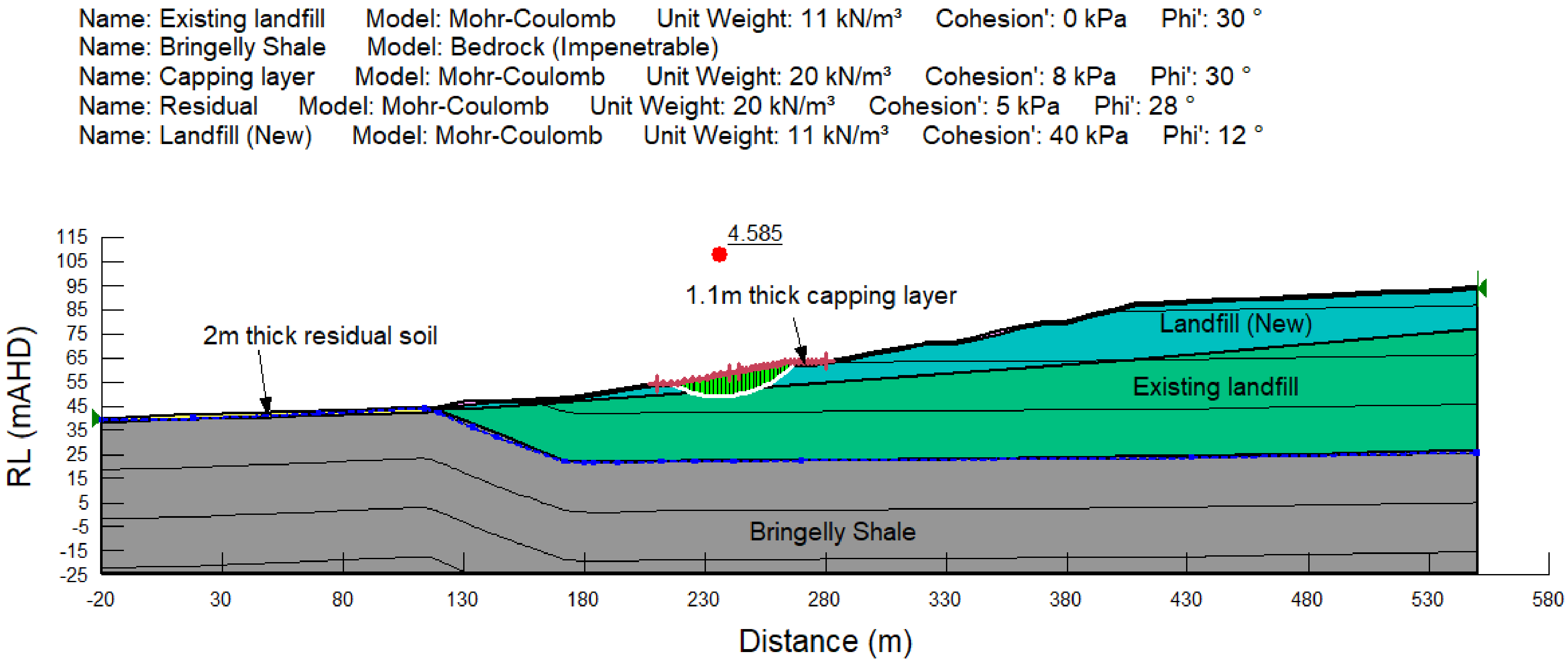

Figure A 17 **Case 3 slope stability assessment at the 4th bench**

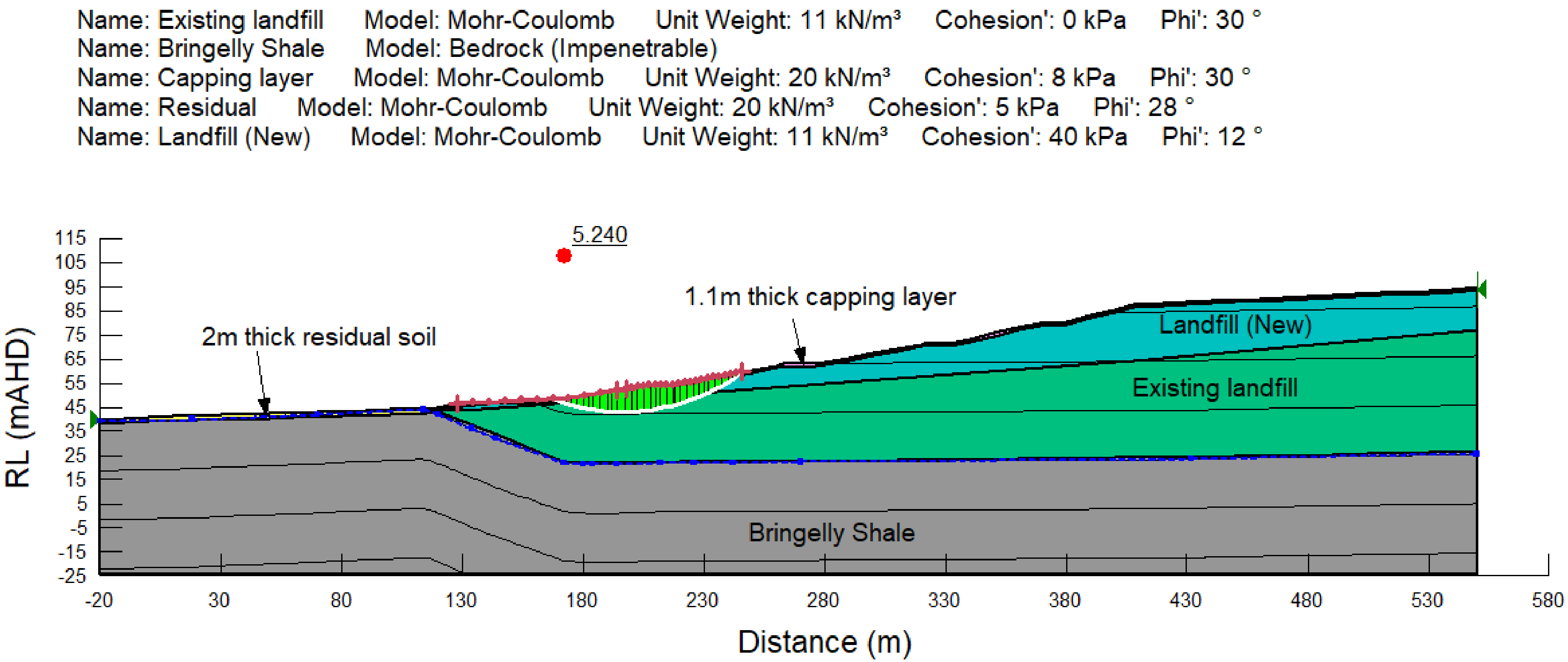

Figure A 18 **Case 3 slope stability assessment at the 5th bench**

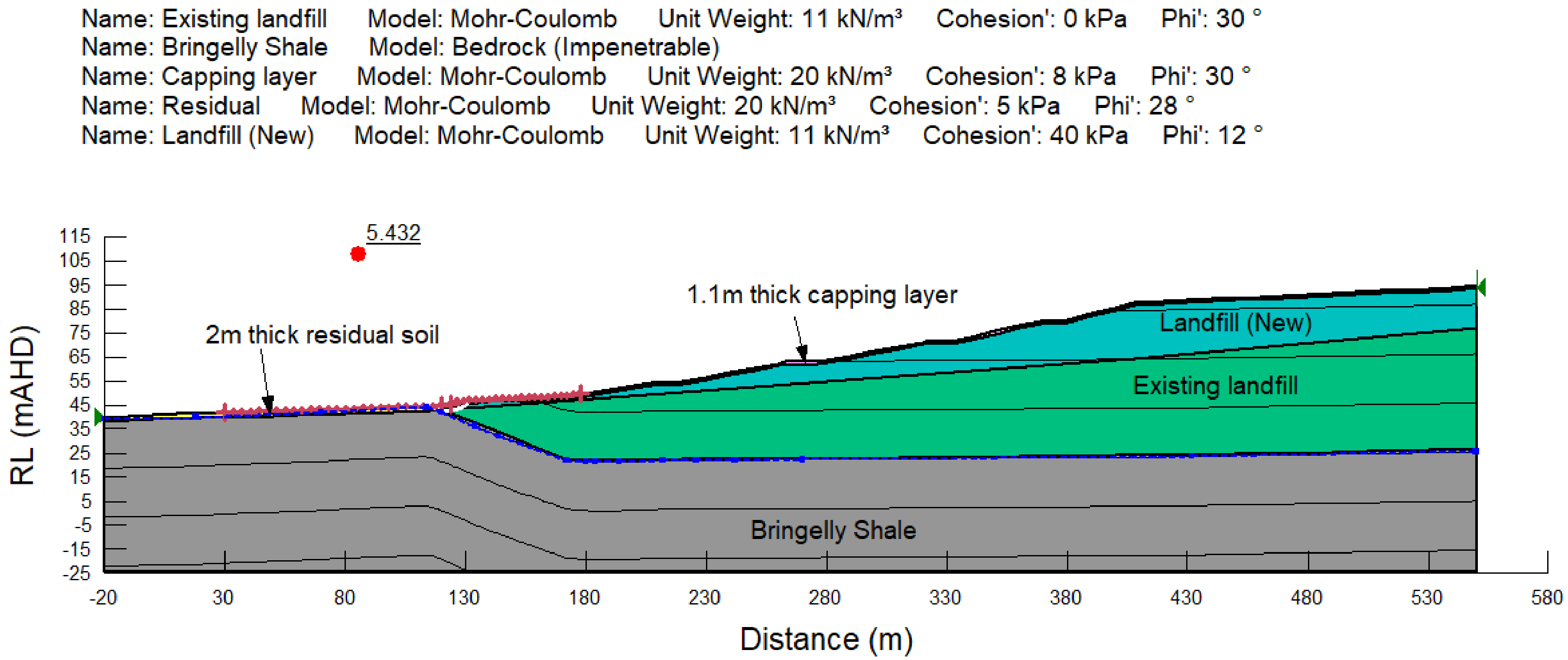

Figure A 19 **Case 3 slope stability assessment at the bottom bench**

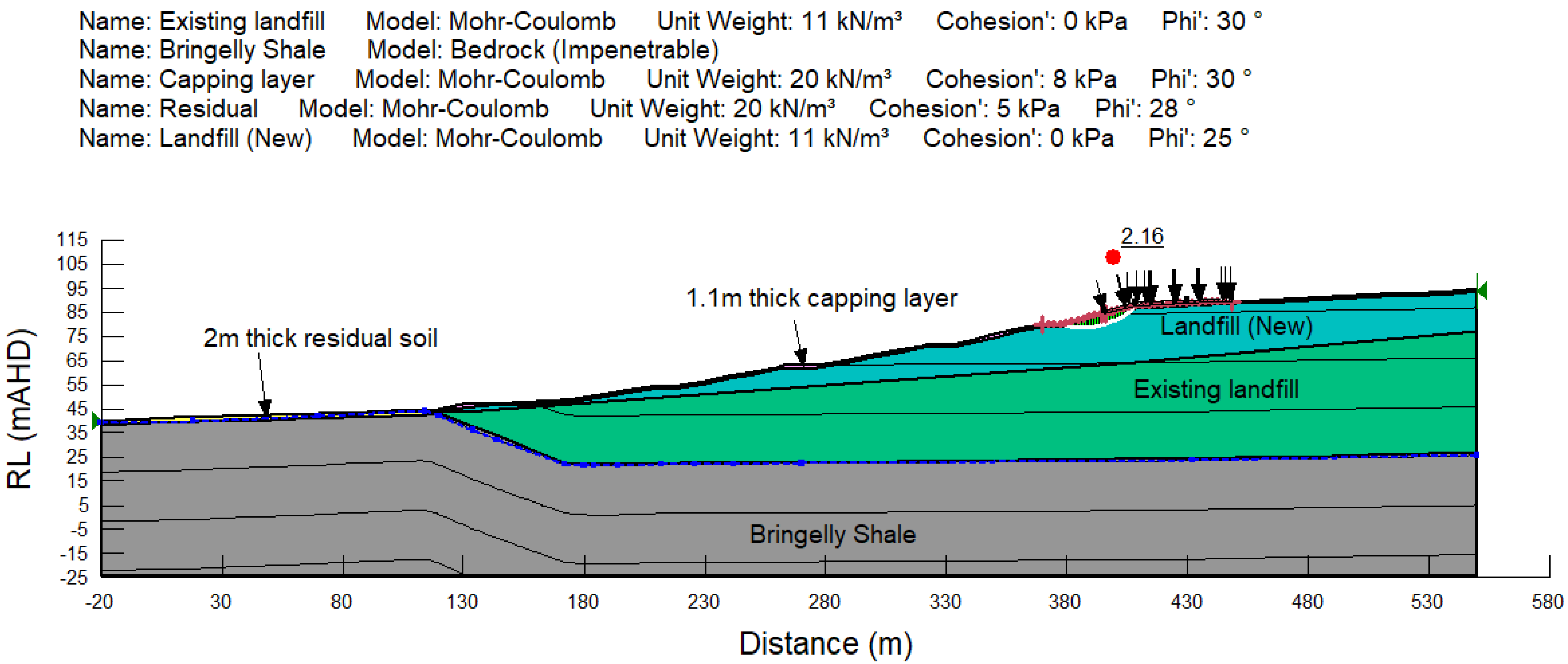

Figure A 20 Case 4 slope stability assessment at the top bench

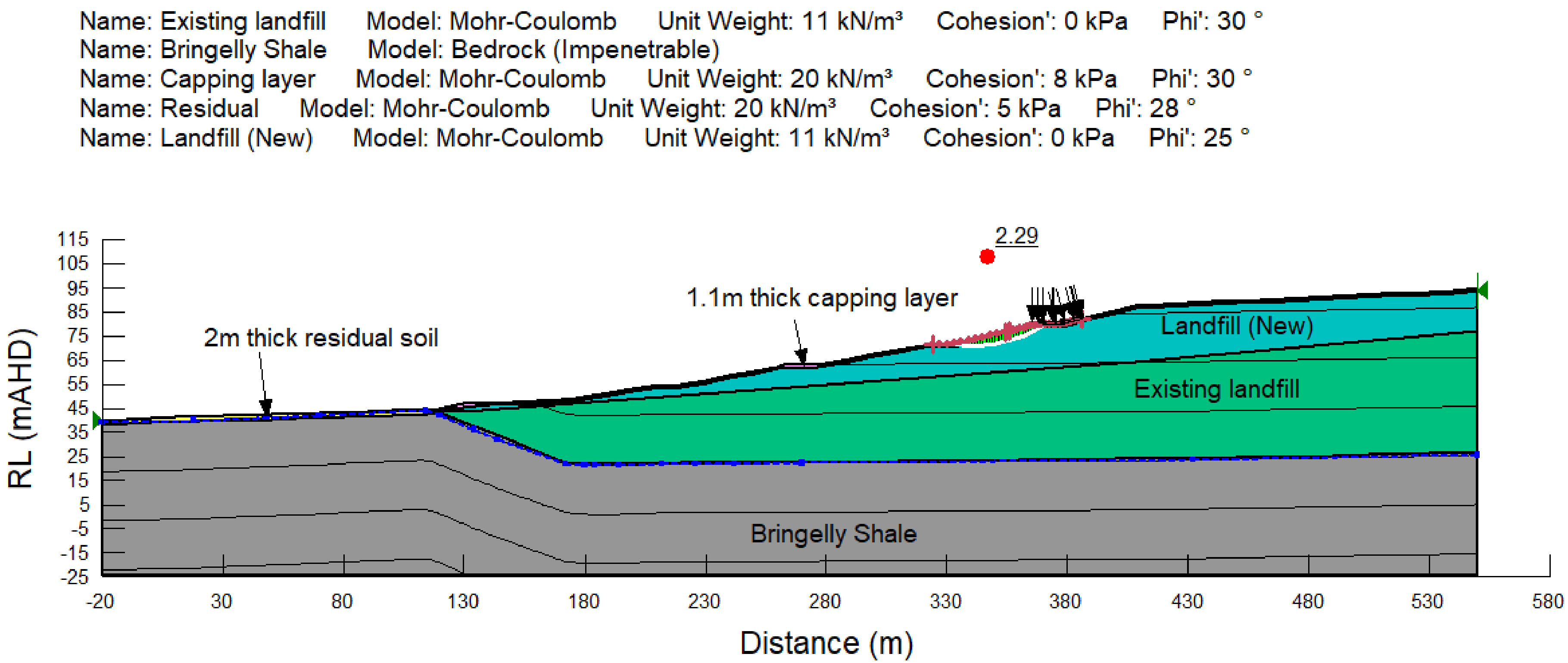

Figure A 21 **Case 4 slope stability assessment at the 2nd bench**

Name: Existing landfill Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bringelly Shale Model: Bedrock (Impenetrable)
Name: Capping layer Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 8 kPa Phi': 30 °
Name: Residual Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 5 kPa Phi': 28 °
Name: Landfill (New) Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 25 °

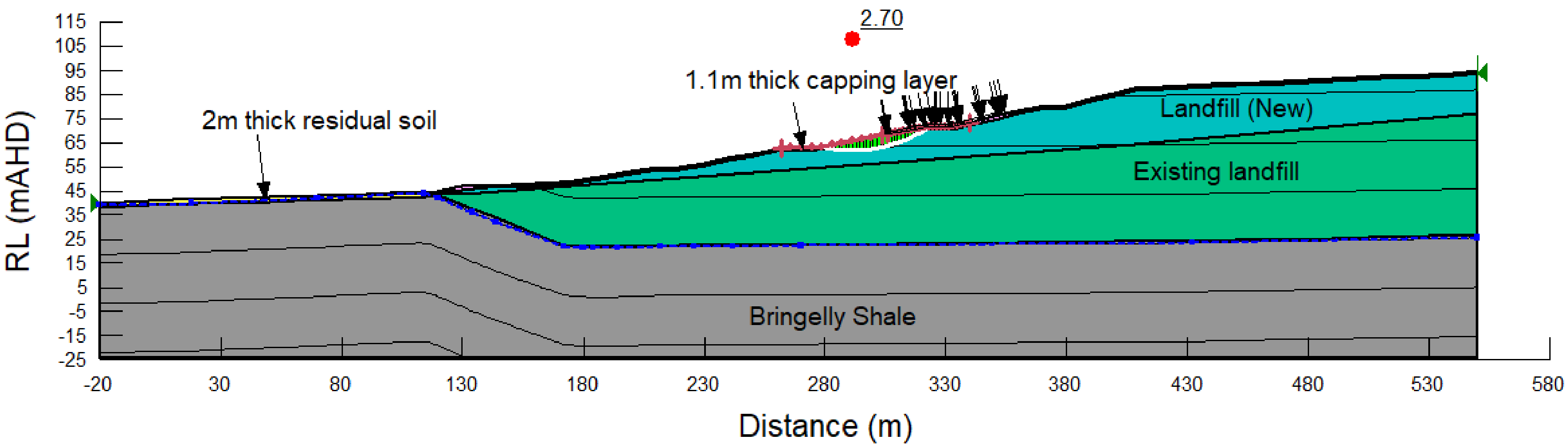

Figure A 22 **Case 4 slope stability assessment at the 3rd bench**

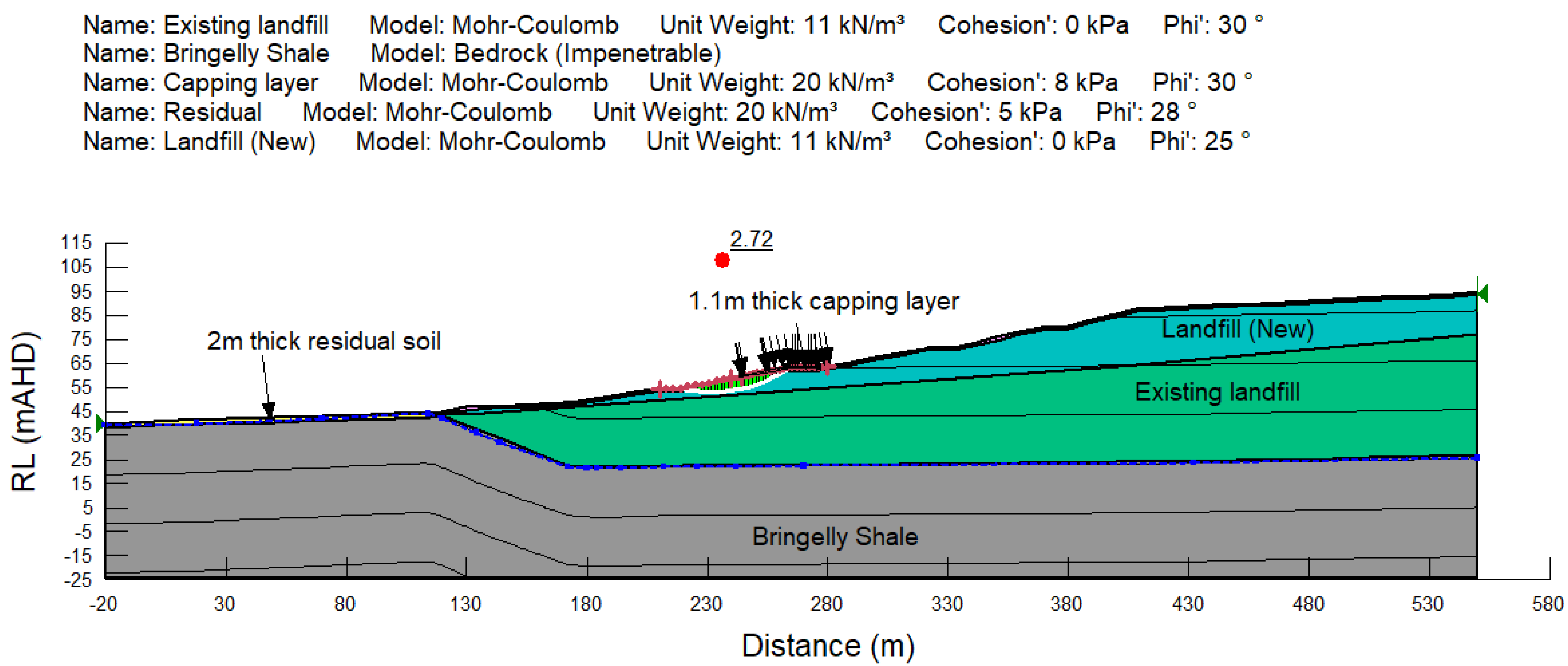

Figure A 23 **Case 4 slope stability assessment at the 4th bench**

Name: Existing landfill Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bringelly Shale Model: Bedrock (Impenetrable)
Name: Capping layer Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 8 kPa Phi': 30 °
Name: Residual Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 5 kPa Phi': 28 °
Name: Landfill (New) Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 25 °

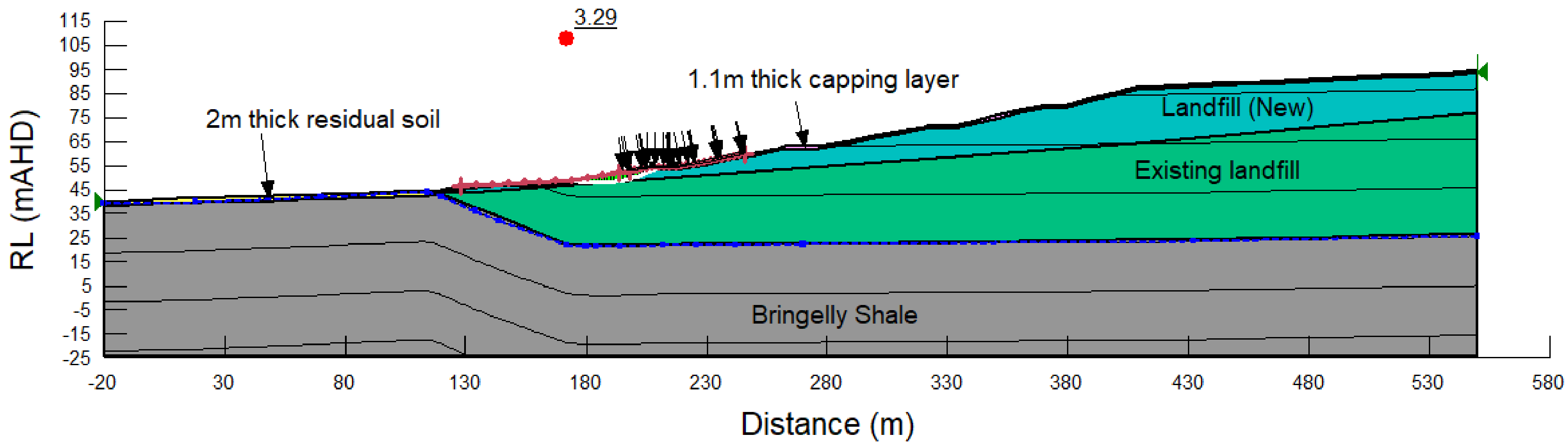

Figure A 24 **Case 4 slope stability assessment at the 5th bench**

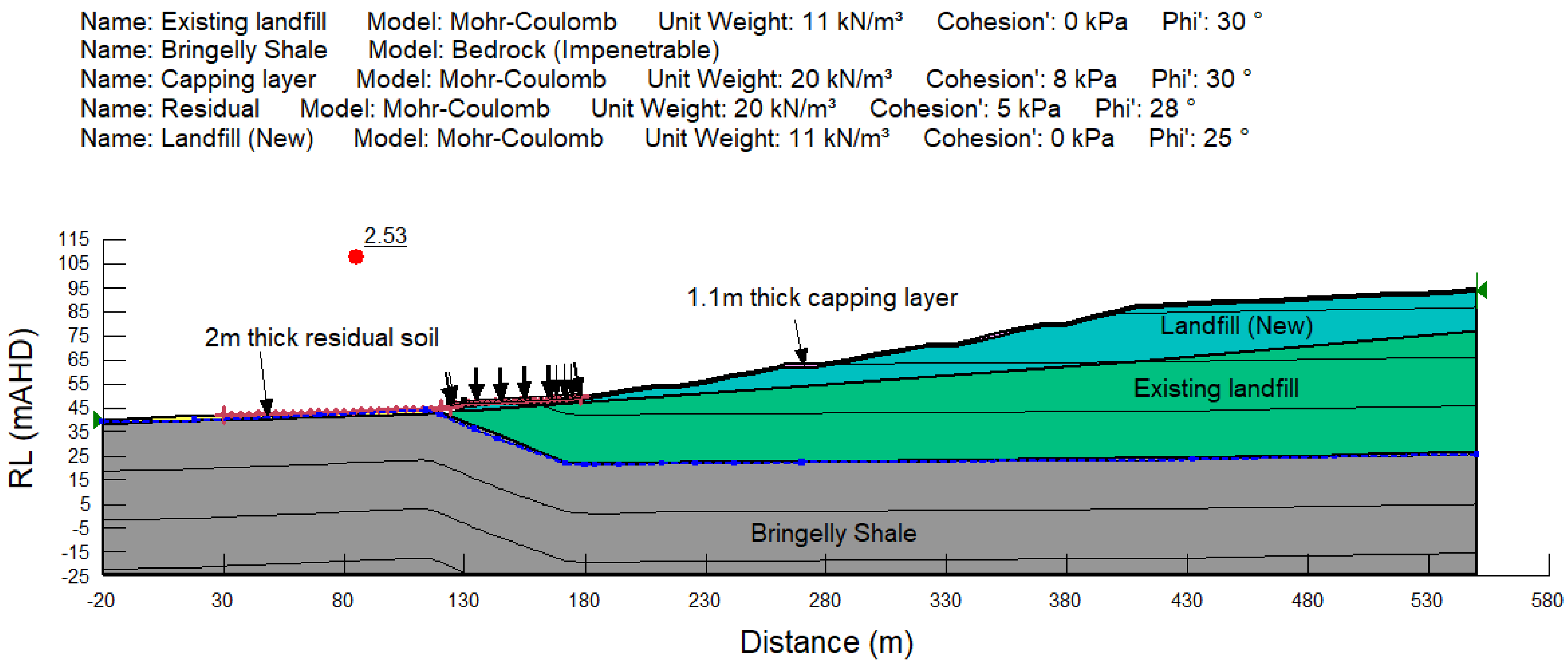

Figure A 25 **Case 4 slope stability assessment at the bottom bench**

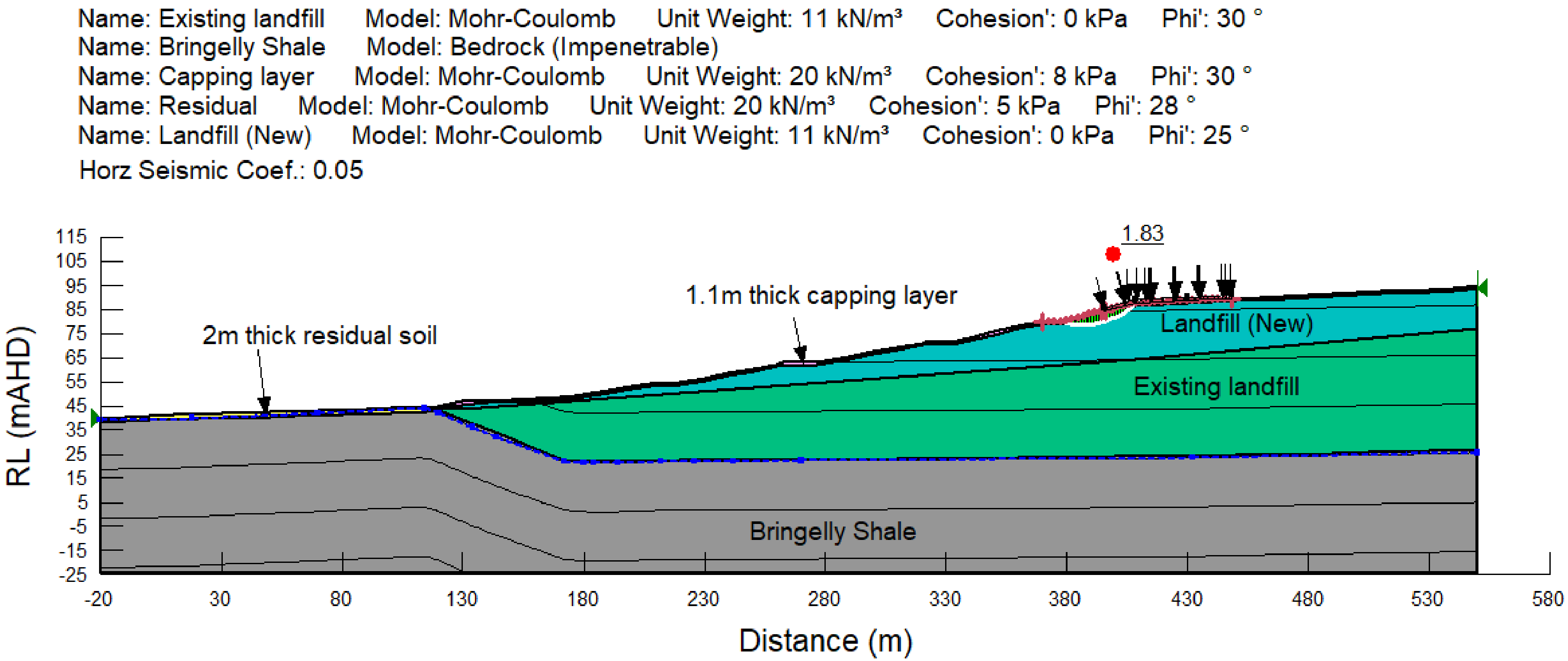

Figure A 26 **Case 4 slope stability assessment for earthquake scenario at the most critical failure surface**

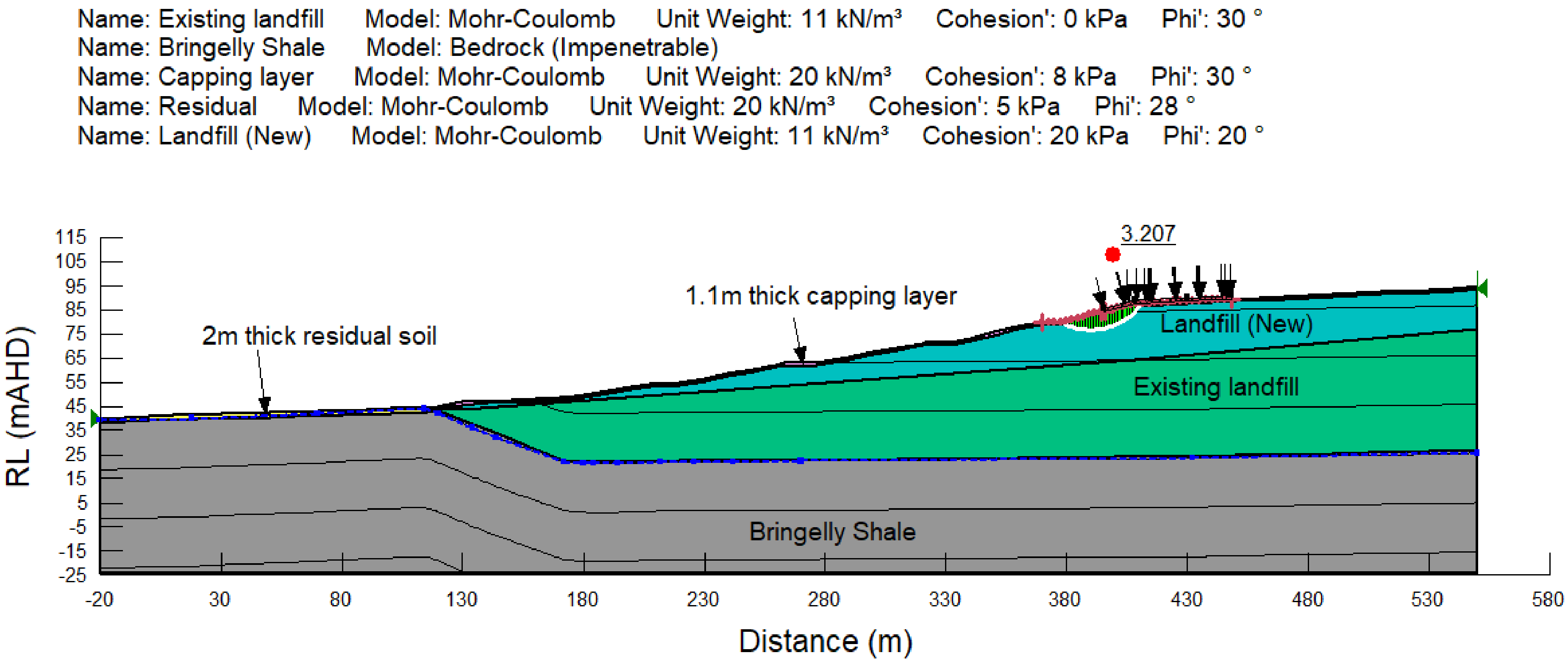

Figure A 27 **Case 5 slope stability assessment at the top bench**

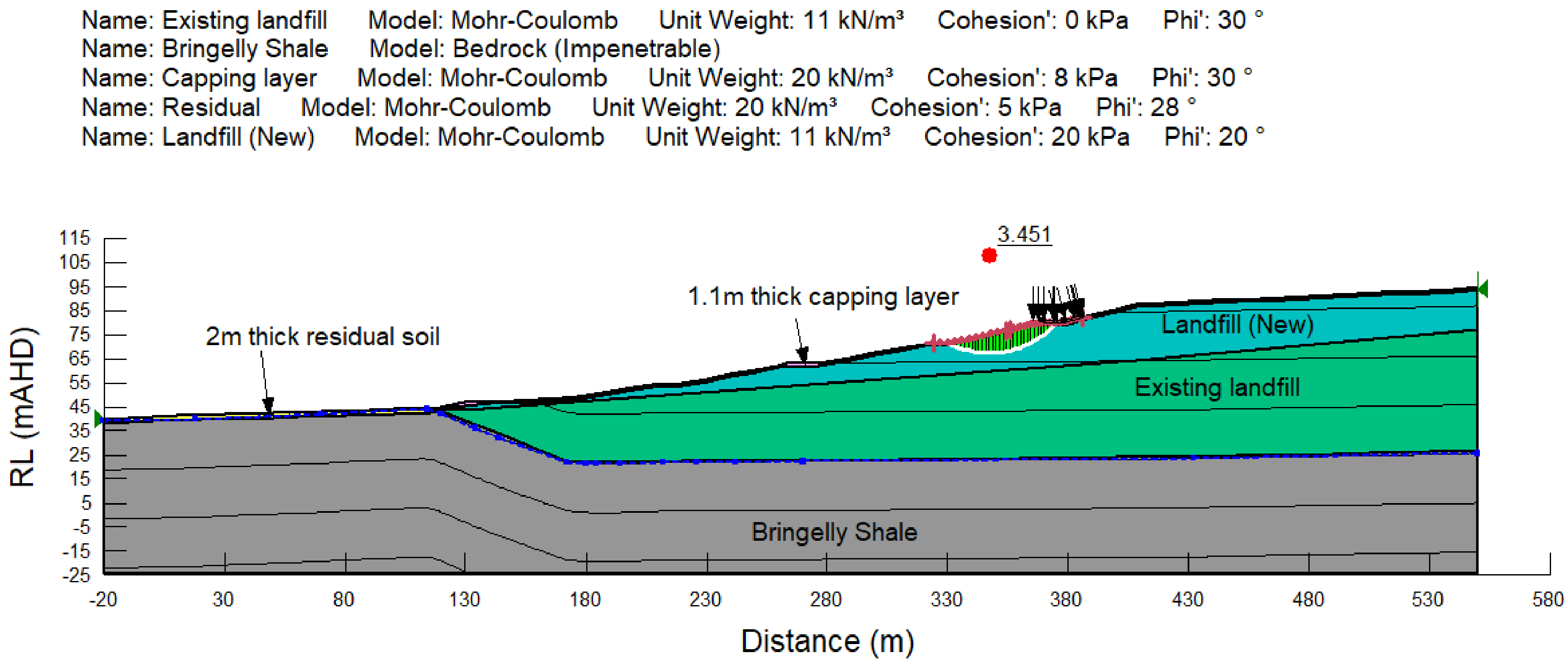

Figure A 28 Case 5 slope stability assessment at the 2nd bench

Name: Existing landfill Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bringelly Shale Model: Bedrock (Impenetrable)
Name: Capping layer Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 8 kPa Phi': 30 °
Name: Residual Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 5 kPa Phi': 28 °
Name: Landfill (New) Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 20 kPa Phi': 20 °

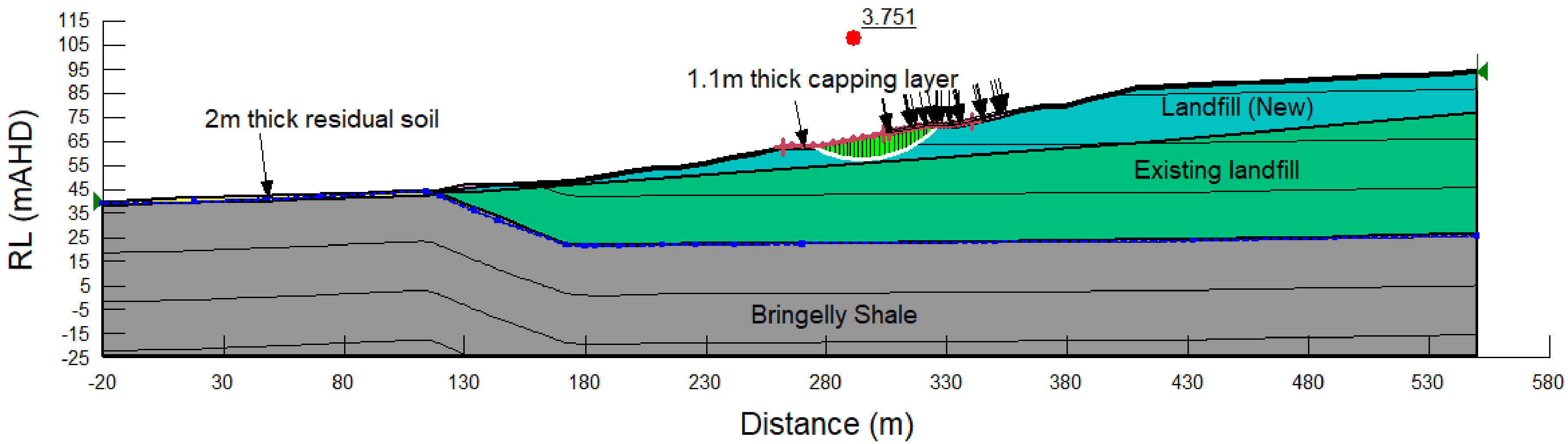

Figure A 29 **Case 5 slope stability assessment at the 3rd bench**

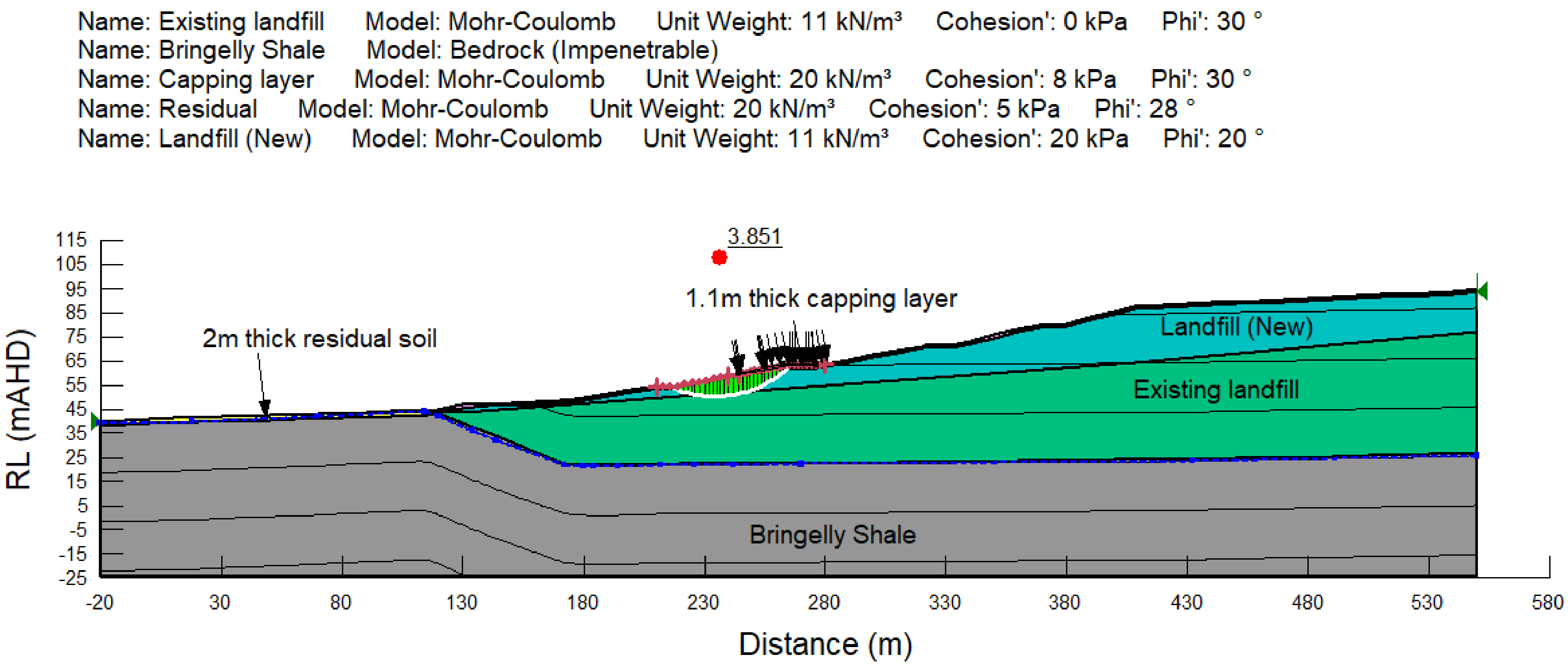

Figure A 30 **Case 5 slope stability assessment at the 4th bench**

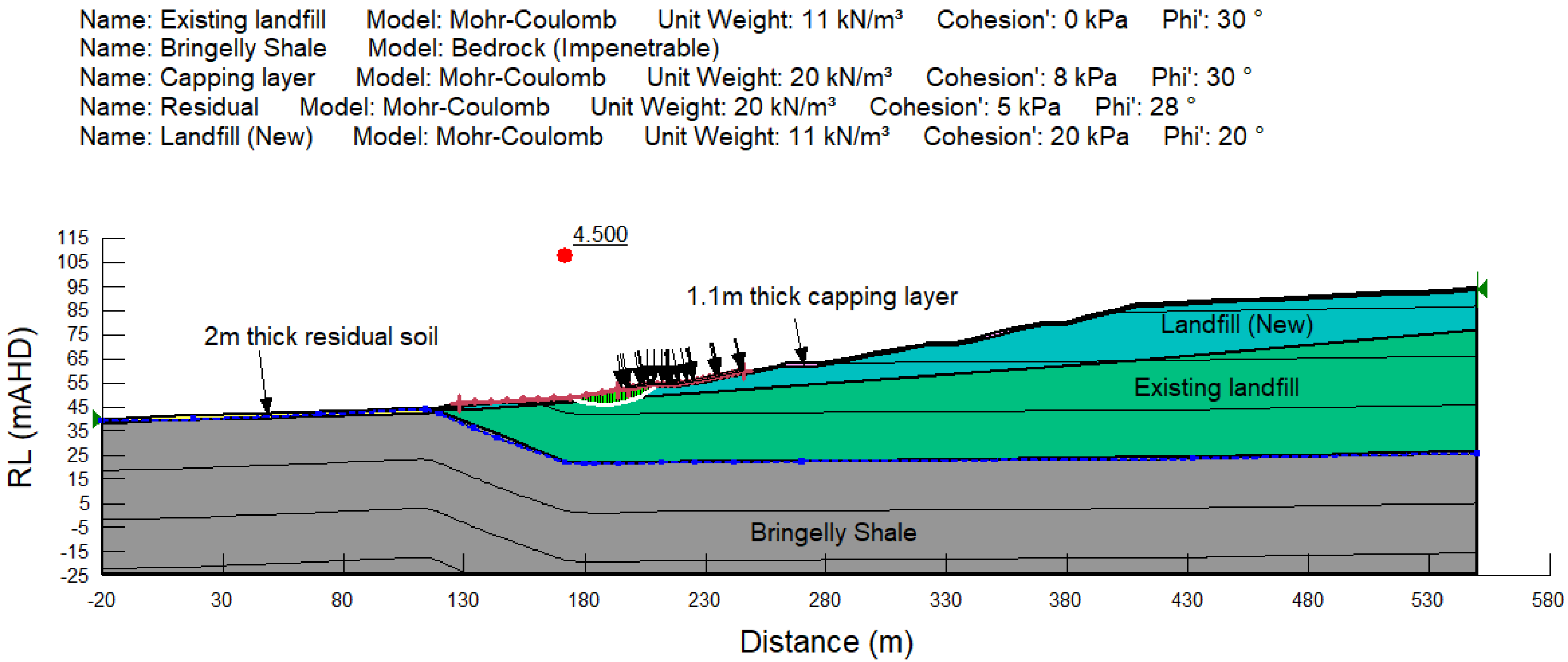

Figure A 31 **Case 5 slope stability assessment at the 5th bench**

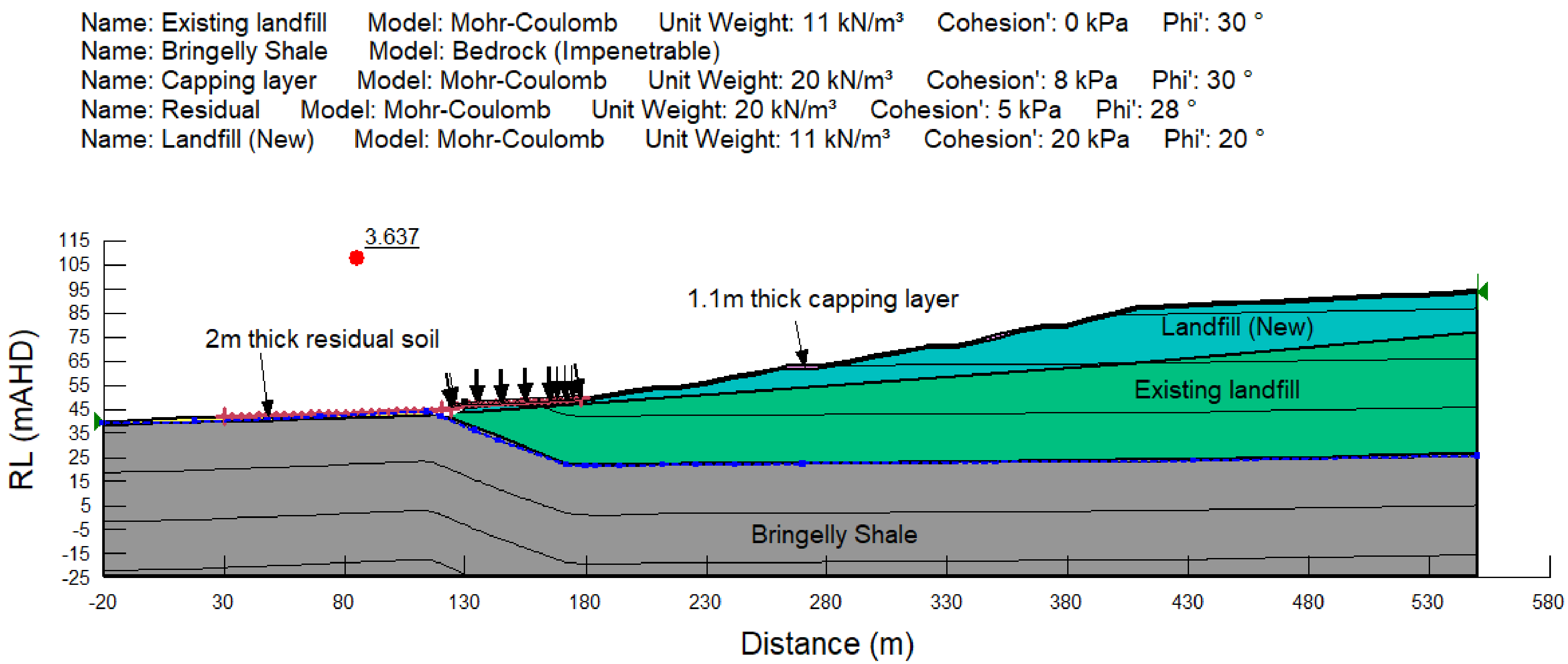

Figure A 32 **Case 5 slope stability assessment at the bottom bench**

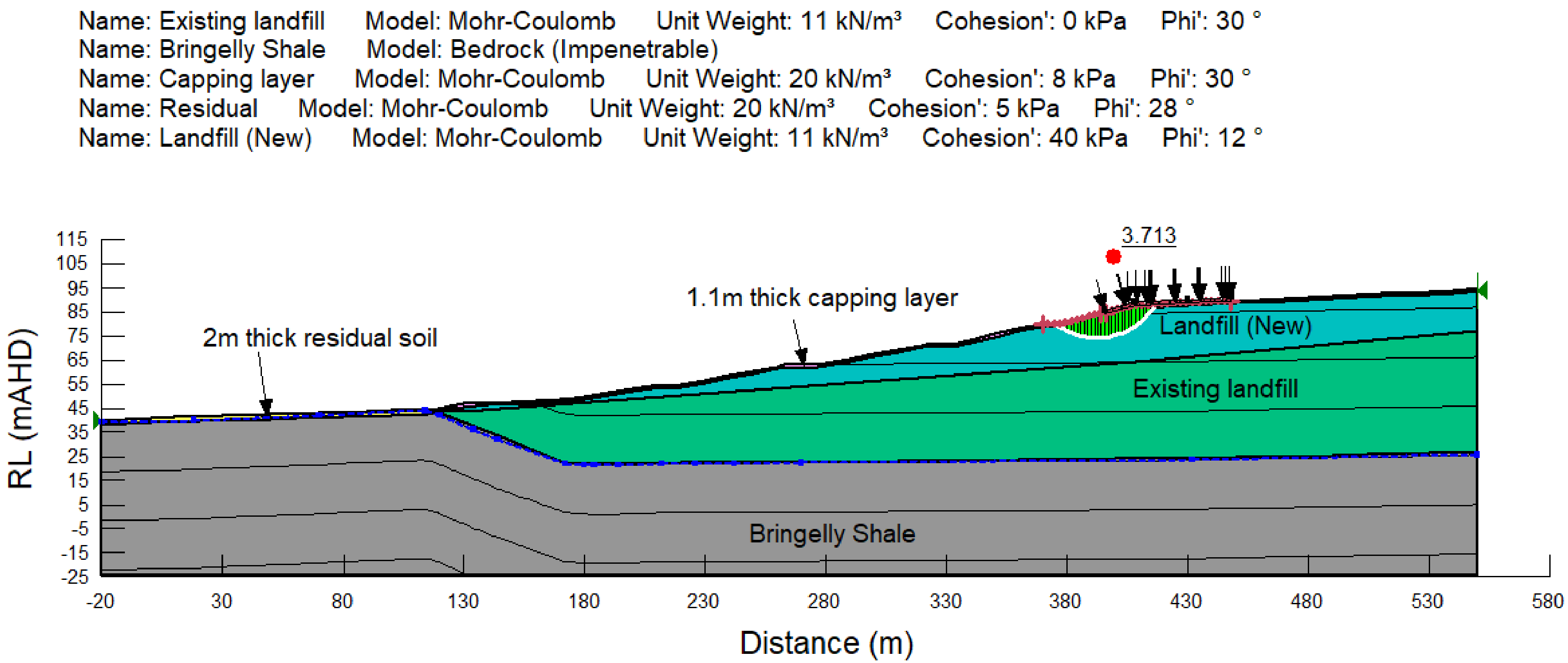

Figure A 33 **Case 6 slope stability assessment at the top bench**

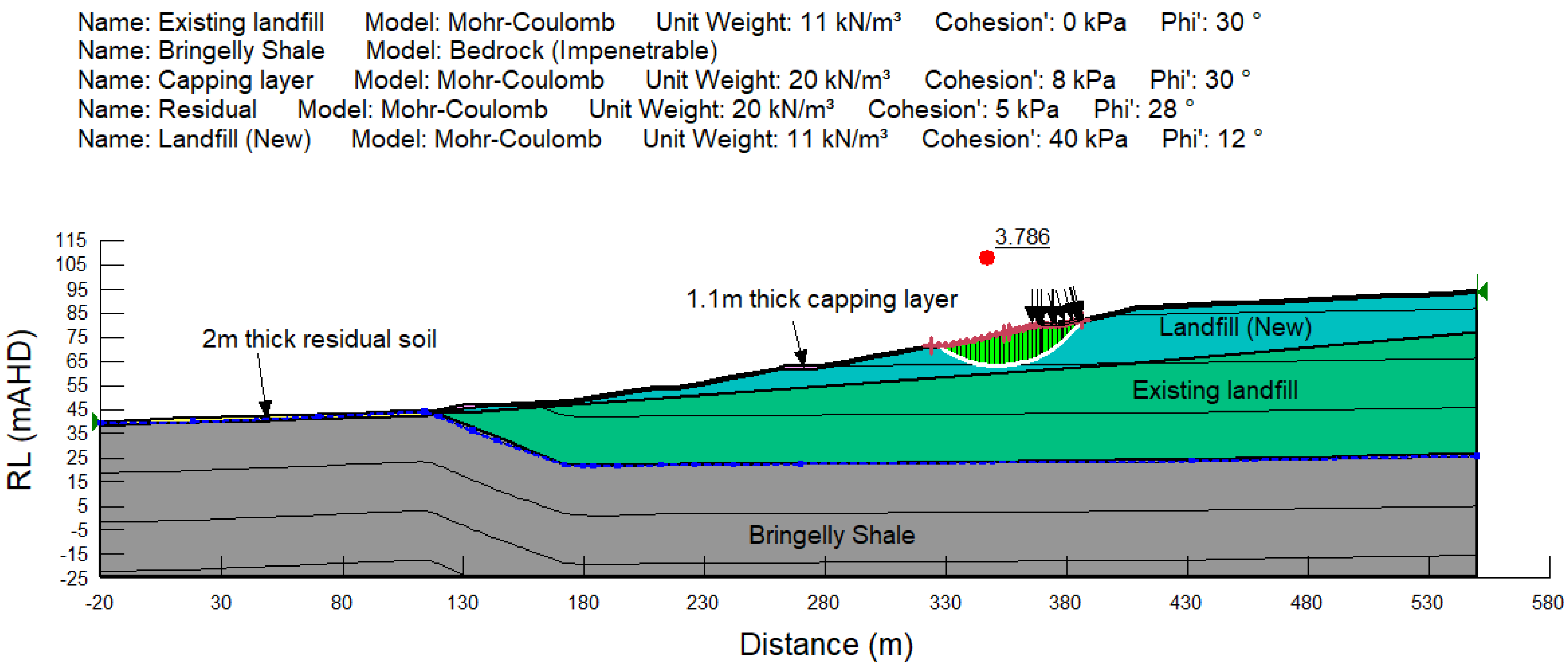

Figure A 34 **Case 6 slope stability assessment at the 2nd bench**

Name: Existing landfill Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bringelly Shale Model: Bedrock (Impenetrable)
Name: Capping layer Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 8 kPa Phi': 30 °
Name: Residual Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 5 kPa Phi': 28 °
Name: Landfill (New) Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 40 kPa Phi': 12 °

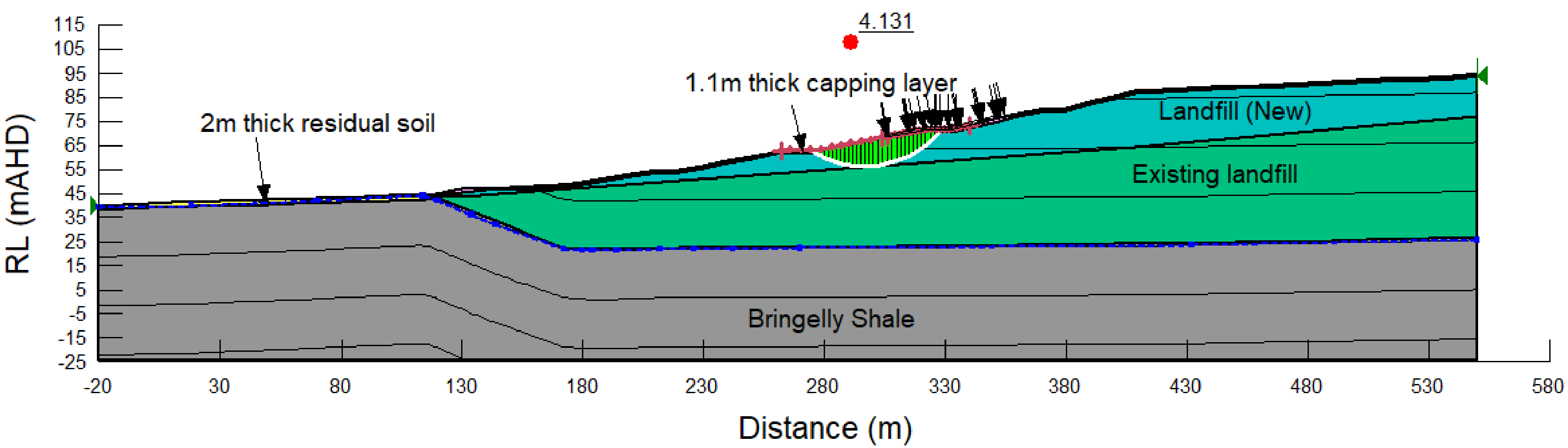

Figure A 35 **Case 6 slope stability assessment at the 3rd bench**

Name: Existing landfill Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bringelly Shale Model: Bedrock (Impenetrable)
Name: Capping layer Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 8 kPa Phi': 30 °
Name: Residual Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 5 kPa Phi': 28 °
Name: Landfill (New) Model: Mohr-Coulomb Unit Weight: 11 kN/m³ Cohesion': 40 kPa Phi': 12 °

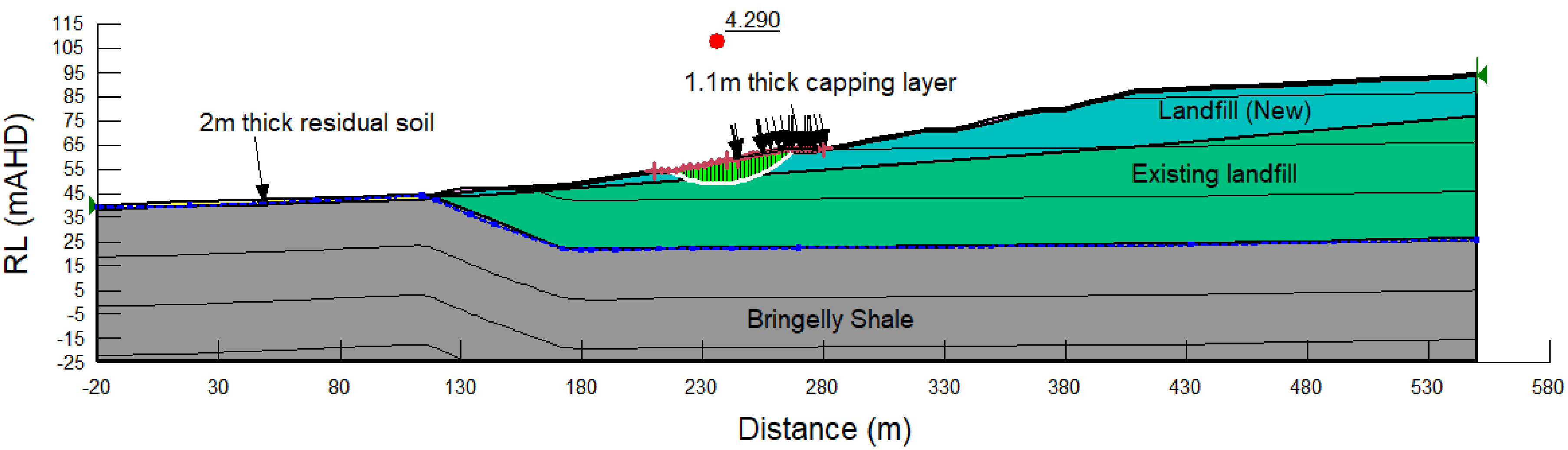

Figure A 36 **Case 6 slope stability assessment at the 4th bench**

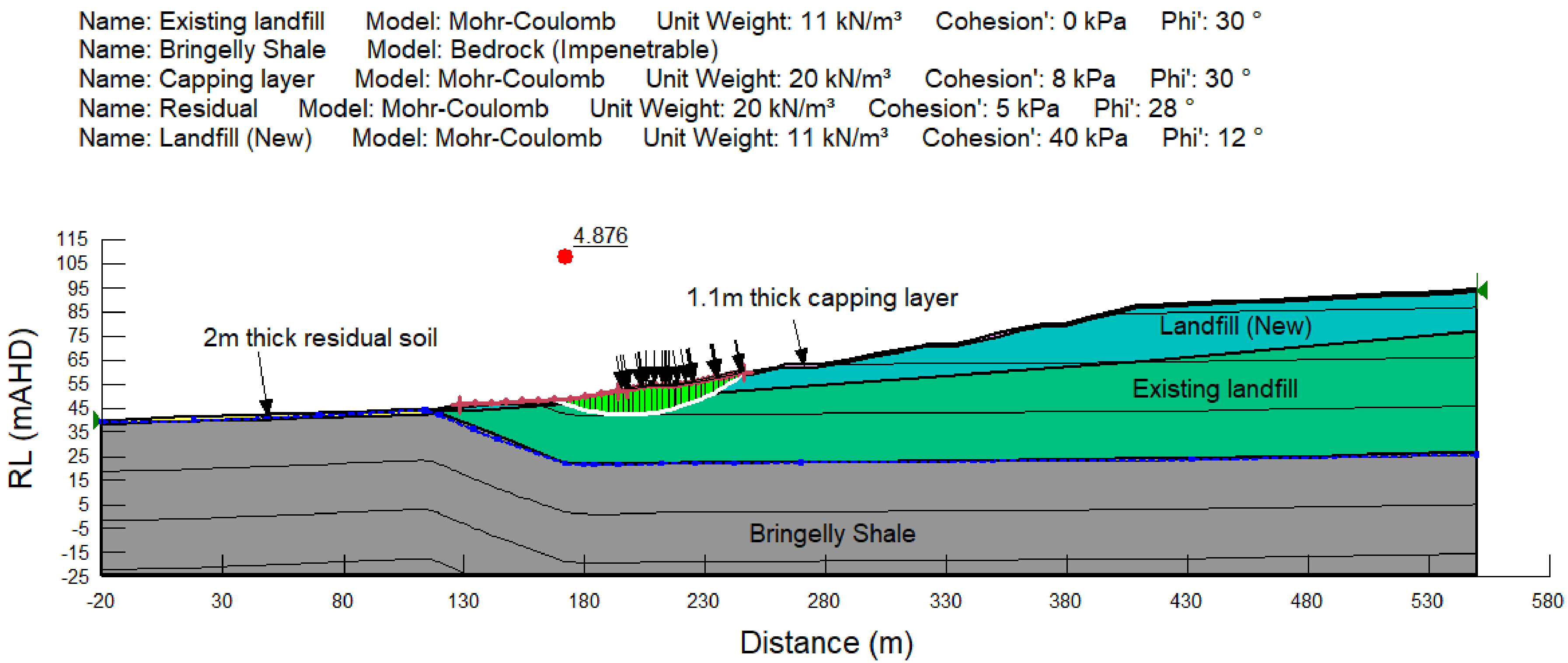

Figure A 37 Case 6 slope stability assessment at the 5th bench

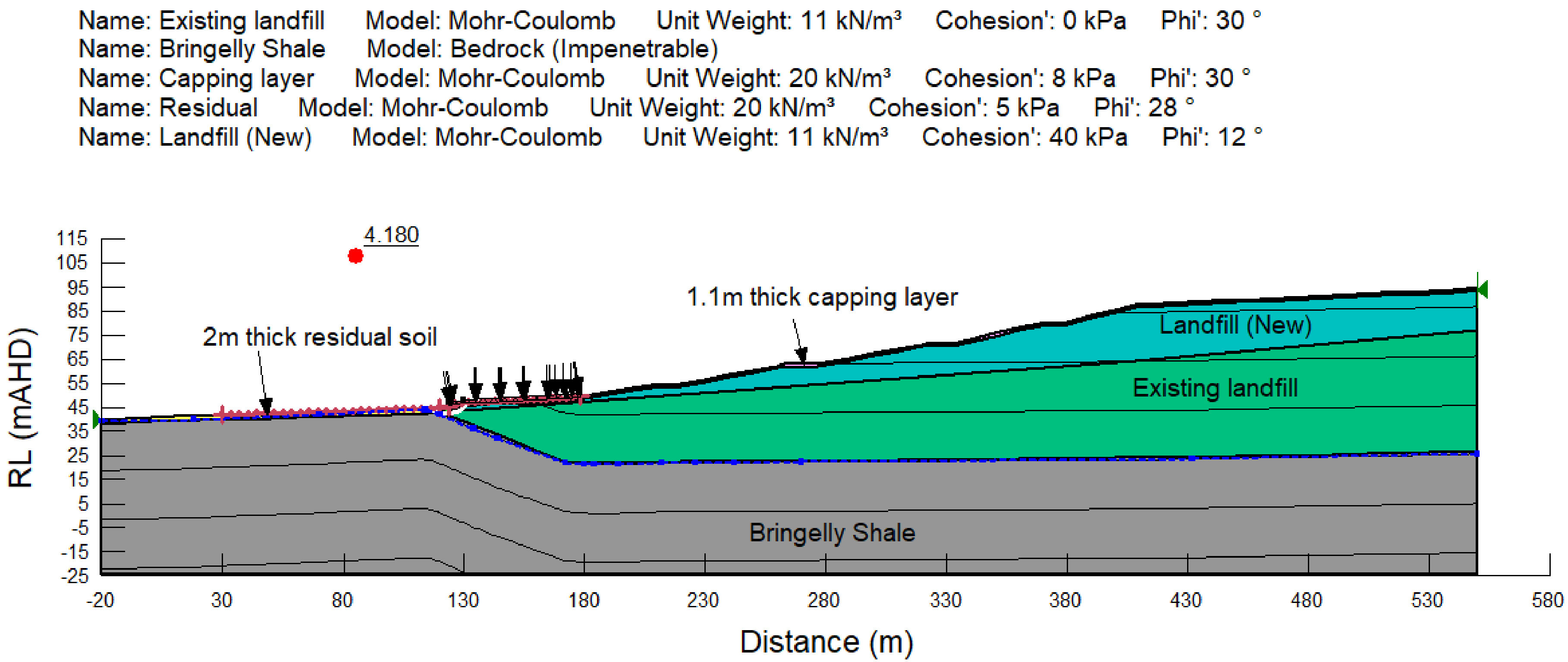

Figure A 38 **Case 6 slope stability assessment at the bottom bench**

Appendix E – Veneer stability analysis

Client:	SUEZ	**Job Number:**	2127038	**Revision:**	
Project:	EDL EIS	**Calcs by:**	R Huynh	**Date:**	13-Dec-19
Subject:	Cap veneer with no seepage	**Checked by:**	A Roberts	**Date:**	

Statement of design procedure

References

Design cases / assumptions

1
2
3
4

This spreadsheet calculates the case of no pore pressure buildup (due to inclusion of geosynthetic drainage layer or similar) based on Qian *et al* (2002)

Item	Description	Symbol	Case				Unit	Notes
			1	**2**	**3**	**4**		
1	Unit weight of cover soil	γ	18.0				kN/m^3	
2	Thickness of cover soil	h	1.00				m	Pependicular to slope
3	Grade of slope		3.5				1 : ?	Vertical : horizontal
4	Vertical height of the slope measured from toe	H	10.0				m	
5	Angle of slope	β	15.9				degree	
6	Length of slope	L	36.4				m	
7	Weight of active wedge	W_A	587.1				kN/m	Equation 13.4
8	Weight of passive wedge	W_P	34.1				kN/m	Equation 13.8
9	Friction angle of cover soil	ϕ'_{soil}	30				degree	
10	Cohesion of cover soil	c'_{soil}	0				kN/m^2	
11	Interface friction angle of critical interface	$\phi'_{critical}$	30.0				degree	
12	Cohesion of critical interface	$c'_{critical}$	1				kN/m^2	
13	Normal force acting on bottom of active wedge	N_A	564.5				kN/m	Equation 13.5
14	Adhesive force acting on bottom of active wedge	C_a	33				kN/m	Equation 13.6
15	Cohesive force along the failure plane of the passive wedge	C	0.00				kN/m	Equation 13.8
16	FoS quadratic equation parameter	a	42.6				kN/m	Equation 13.9
17	FoS quadratic equation parameter	b	-107.2				kN/m	Equation 13.9
18	FoS quadratic equation parameter	c	14.2				kN/m	Equation 13.9
19	Factor of safety for stability of the cover soil mass	FoS	**2.37**					Equation 13.9

Client: SUEZ
Project: EDL EIS
Subject: Cap veneer parrallel seepage

Job Number: 2127038
Calcs by: R Huynh **Date:** 13-Dec-19
Checked by: A Roberts **Date:**

Statement of design procedure

References

Design cases / assumptions

1 with geocomposite drainage net
2 without geocomposite drainage net

This spreadsheet calculates the case of parallel to slope seepage buildup based on Qian *et al* (2002)

Item	Description	Symbol	Case 1	Case 2	Case 3	Case 4	Unit	Notes
1	Saturated unit weight of cover soil	γ_{sat}	18	18			kN/m^3	
2	Unit weight of cover soil	γ	17.3	17.3			kN/m^3	
3	Unit weight of water	γ_w	9.81	9.81			kN/m^3	
4	Thickness of cover soil	h	1.00	1.00			m	Pependicular to slope
5	Vertical height of the slope measured from toe	H	10.00	10.00			m	
6	Grade of slope		3.50	3.50			1 : ?	Vertical : horizontal
7	Angle of slope	β	15.95	15.95			degree	
8	Depth of seepage water in the soil layer (perpendicular to the slope)	h_w	0.5	1			m	
9	Weight of active wedge	W_A	609.39	621.14			kN/m	Equation 13.40
10	Weight of passive wedge	W_P	33.08	34.07			kN/m	Equation 13.41
11	Friction angle of cover soil	ϕ_{soil}	32	32			degree	
12	Interface friction angle between cover soil and geosynthetic	ϕ_{geo}	26	26			degree	
13	Resultant of the pore pressures acting on the interwedge surfaces	U_{AN}	167.55	326.84			kN/m	Equation 13.37
14	Resultant of the pore pressures acting perpendicular to the slope	U_H	1.23	4.91			kN/m	Equation 13.38
15	Resultant of the vertical pore pressures acting on the passive wedge	U_{PN}	4.29	17.17			kN/m	Equation 13.39
16	FoS quadratic equation parameter	a	161.1	164.4			kN/m	Equation 13.36
17	FoS quadratic equation parameter	b	-239.5	-164.6			kN/m	Equation 13.36

Client:	SUEZ	**Job Number:**	2127038		
Project:	EDL EIS	**Calcs by:**	R Huynh	**Date:**	13-Dec-19
Subject:	Cap veneer parrallel seepage	**Checked by:**	A Roberts	**Date:**	

Statement of design procedure

References

Design cases / assumptions

1 with geocomposite drainage net

2 without geocomposite drainage net

This spreadsheet calculates the case of parallel to slope seepage buildup based on Qian *et al* (2002)

Item	Description	Symbol	Case				Unit	Notes
			1	**2**	**3**	**4**		
18	FoS quadratic equation parameter	c	34.4	22.4			kN/m	Equation 13.36
19	Factor of safety for stability of the cover soil mass	FoS	**1.33**	**0.84**				Equation 13.36

Client:	SUEZ	**Job Number:**	2127038		
Project:	EDL EIS	**Calcs by:**	R Huynh	**Date:**	13-Dec-19
Subject:	Cap veneer parrallel seepage	**Checked by:**	A Roberts	**Date:**	

Statement of design procedure

References

Design cases / assumptions

1 Not used
2 Not used
3 Not used
4 Not used

This spreadsheet calculates the case of horizontal seepage buildup (due to a blocked toe drain or similar) based on Qian *et al* (2002)

Item	Description	Symbol	Case 1	Case 2	Case 3	Case 4	Unit	Notes
1	Saturated unit weight of cover soil	γ_{sat}	18				kN/m^3	
2	Unit weight of cover soil	γ	17.3				kN/m^3	
3	Unit weight of water	γ_w	9.81				kN/m^3	
4	Thickness of cover soil	h	1.00				m	Pependicular to slope
5	Vertical height of the slope measured from toe	H	10.00				m	
6	Grade of slope		3.5				1 : ?	Vertical : horizontal
7	Angle of slope	β	15.95				degree	
8	Vertical height of free water surface measured from toe	H_w	0				m	
9	Weight of active wedge	W_A	595.66				kN/m	Equation 13.14
10	Weight of passive wedge	W_P	34.07				kN/m	Equation 13.18
11	Friction angle of cover soil	ϕ	30				degree	
12	Interface friction angle between cover soil and geosynthetic	ϕ_{geo}	30				degree	
13	Normal force acting on bottom of active wedge	N_A	591.94				kN/m	Equation 13.17
14	Resultant of the pore pressures acting on the interwedge surfaces	U_h	4.91				kN/m	Equation 13.16
15	Resultant of the pore pressures acting perpendicular to the slope	U_n	-17.85				kN/m	Equation 13.15

Client:	SUEZ	**Job Number:**	2127038		
Project:	EDL EIS	**Calcs by:**	R Huynh	**Date:**	13-Dec-19
Subject:	Cap veneer parrallel seepage	**Checked by:**	A Roberts	**Date:**	

Statement of design procedure

References

Design cases / assumptions

1	Not used
2	Not used
3	Not used
4	Not used

This spreadsheet calculates the case of horizontal seepage buildup (due to a blocked toe drain or similar) based on Qian *et al* (2002)

Item	Description	Symbol	Case				Unit	Notes
			1	**2**	**3**	**4**		
16	Resultant of the vertical pore pressures acting on the passive wedge	U_v	1.33				kN/m	Equation 13.19
17	FoS quadratic equation parameter	a	157.7				kN/m	Equation 13.20
18	FoS quadratic equation parameter	b	-372.7				kN/m	Equation 13.20
19	FoS quadratic equation parameter	c	54.2				kN/m	Equation 13.20
20	Factor of safety for stability of the cover soil mass	FoS	**2.21**					Equation 13.20

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2127038-80484/https://projects.ghd.com/oc/Sydney/elizabethdrivelandfi3/Delivery/Documents/11 EIS support/2127038-REP_EDL Expansion Technical Advice.docx

Document Status

Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
1	T Shu A Horlyck N Spurrett	A Dixon	a. Dixon	A Roberts		10/02/2020