Appendix E – Sub-catchment Integrated Catchment Management Plan

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TE RAPA RACECOURSE

SUB-CATCHMENT INTEGRATED CATCHMENT MANAGEMENT PLAN

TE RAPA ROAD, HAMILTON

WAIKATO RACING BOARD LTD

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EXECUTIVE SUMMARY

This Sub-catchment ICMP forms a key part of the Plan Change application for the Te Rapa Racecourse Re-Development Area owned by the Waikato Racing Board. It outlines the proposed strategies and solutions for Three Waters demand management within the development areas. The objective of the ICMP is to present feasible Three waters management options to confirm that the site is suitable for residential development without resulting in adverse effects on the downstream HCC networks and ultimate natural receiving environment.

It is proposed to re-zone approximately 6.48ha of the Te Rapa Racecourse site currently zoned 'Major Facilities Zone' into Medium Density Residential Zone for subsequent medium density residential development. The area is proposed to be developed with a variety of dwelling styles as well as associated road and reserve areas.

Receiving Environment

Stormwater management is identified as the key Three Waters constraint to site development and hence comprises the focus of this ICMP document.

The Racecourse Re-development area is well serviced by existing HCC stormwater reticulation. A 1050mm stormwater main extends through the middle of the site, draining north west to south east, and connects to a 1650mm pipe within Garnett Avenue. Flows are then conveyed to the east toward the Waikato River, terminating with an 1800mm pipe discharging into the river approximately 1.5km from the subject site.

The site is also identified as encompassing an area of existing extreme peak event flood storage/conveyance.

Policies, Objectives and BPO's

Strategic objectives have been developed for three waters infrastructure for the sub-catchment. The objectives form the basis on which Best Practicable Options (BPOs) were selected for infrastructure design recommendations.

Key operational objectives are presented below:

1. Maintain or enhance stream water quality
2. Minimise alterations to the natural flow regime
3. Maintain or enhance existing ecological values
4. Protect cultural values
5. Public safety
6. Minimise water consumption
7. Minimise wastewater discharges
8. Have due regard for economic affordability

STORMWATER MANAGEMENT - Means of Compliance

Based on the proposed primary discharge to existing HCC stormwater infrastructure and presence of flooding within the subject site and wider catchment, peak flow attenuation is considered a key requirement for the sub-catchment and future developments.

The key performance criteria (in accordance with HCC's and WRC's standards) are:

- Water quality treatment in accordance with RITS requirements.
- Flow Attenuation - Match pre-development flow rates for the 2,10 and 80% of 100 year ARI events through controlled attenuation and multi stage discharge outlets.
- Flood volume balancing and maintenance of extreme flood flow conveyance through the development site to avoid adverse flooding effects upon areas proposed for residential development and upstream and downstream properties.

The recommended methods for achieving these key performance criteria comprise provision for a sub-catchment stormwater management wetland within the development site located within a central reserve corridor and aligning with the identified extreme flood event flow path through the site.

Wastewater

The existing reticulation within the Racecourse Redevelopment Area is at a sufficient depth for gravity connections to be provided to all proposed development areas within the sub-catchment.

An assessment of the network downstream of the sub-catchment has been undertaken and confirms that the HCC wastewater pipe network downstream of the sub-catchment has sufficient capacity for the entire developed sub-catchment.

__Water Supply__

It is proposed to service the developments from the existing reticulation surrounding the area. Modelling of the existing network was undertaken to determine the effects of the additional demand from the development on the network. Modelling results showed that sufficient levels of service can be achieved within the proposed development, including firefighting supply. Pressures are predicted to drop in parts of the remaining network by up to 1.3m as a result of the proposed development, prior to the Pukete Zone closure. Post Pukete Zone closure pressures are expected to be above 30m throughout.

1 INTRODUCTION

This Sub-Catchment Integrated Management Plan (ICMP) forms a key part of the Plan Change application for the Te Rapa Racecourse Residential Re-Development Area owned by the Waikato Racing Club. It outlines identifies existing constraints and proposed solutions for Three Waters Demand Management within the development, assesses any effects occurring as a result of the proposed development, and provides mitigation options where possible.

It is notable that the proposal comprises the development of an existing brownfields site located centrally within the Hamilton City urban development area with all Three Waters servicing proposed via connection to the existing HCC infrastructure network.

1.1 Project Overview

It is proposed to re-zone approximately 6.48Ha of the Te Rapa Racecourse site currently zoned 'Major Facilities Zone' into 'Medium Density Residential Zone' for subsequent medium density residential development. The area is proposed to be developed with a variety of dwelling styles as well as associated road and open space areas.

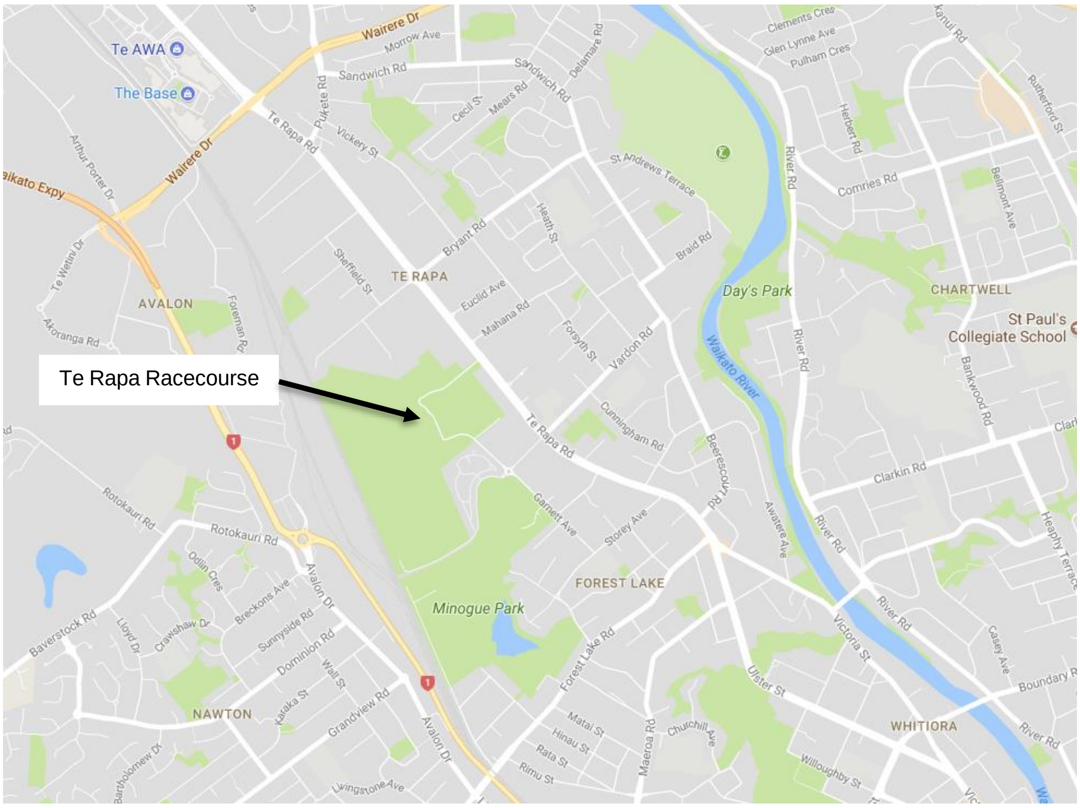

Figure 1: Site Location (Source: Google maps)

Figure 2: Site Extents (Source Google Earth)

The site is located within the wider St Andrews catchment, a completely urban catchment located centrally within the Hamilton City boundaries. The St Andrews catchment generally drains in an easterly direction, discharging to the Waikato River via a number of minor tributaries, most of which have been piped across their upper reaches. Several sub-catchments, including the subject sub-catchment discharge directly to the River via a piped network.

No ICMP currently exists for the St Andrews catchment.

HCC holds consents from Waikato Regional Council (WRC) for the discharge of stormwater, to take water from the Waikato River and to discharge wastewater to the Waikato River for Hamilton City as follows.

- A Comprehensive Stormwater Discharge Consent (CSDC) for urban Hamilton authorises the discharge of stormwater from existing built up areas. This ICMP is required to provide guidance on how to manage the stormwater diversion and discharge effects associated with the proposed racecourse development.

- HCC's water take consent includes a stepped increase in water take for growth but requires water demand management to be implemented.

- HCC's wastewater discharge consent requires network management to avoid events such as wastewater overflows.

1.2 Purpose of this ICMP

It is a key aspect of this ICMP document that it does not intend to present a detailed/finalised design for the proposed residential development area. Rather, the intent of the document is to outline viable Three Waters management options which can realistically be implemented at the site to enable the intended land use while avoiding any potential adverse effects upon surrounding land, the existing HCC 3 Waters network infrastructure and the downstream receiving environment.

In this respect, the key purpose of this ICMP includes the following items:

- To present sufficient information to confirm that best practice Three Waters management can be achieved at the proposed residential development site to avoid any adverse network/environmental effects and enable the planned land use.
- To determine an integrated catchment management approach which is based upon the best practicable option(s) to avoid as far as practicable and otherwise minimise, the cumulative adverse effects of all new stormwater diversion and discharge activities as a result of the proposed rezoning and residential development.
- To provide guidance on how water, wastewater and stormwater can be managed in an integrated way and in accordance with proposed new land uses that occur with development.
- To ensure that the Three Waters networks within the development and beyond can accommodate growth while avoiding, remedying, or mitigating adverse effects that can occur from land use change. This includes effects of flooding and erosion, ad-hoc stormwater discharges and unreasonable increase in water demand and wastewater generation.
- To ensure that existing Three Waters networks are not compromised and any future networks to accommodate growth comply with RMA requirements, and HCC's Level of Service, HCC's CSDC and water conservation and demand management objectives.

1.3 Levels of Service

Refer to the following documents for level of service information:

- Operative District Plan 2017
- Regional Infrastructure Technical Specifications (RITS)
- HCC Stormwater Modelling methodology
- HCC Standard Assessment Methodology – Water, and
- HCC Standard Assessment Methodology – Wastewater.

HCC's established hierarchy for the management of the Three Waters is as follows.

1. Minimise demand
2. Reuse
3. Treat & dispose to ground
4. Treatment & detention
5. Reticulation

Requirements of the DP for addressing matters in a WIA must also be met.

2 STRATEGIC CONTEXT

In order to strategically plan the management of Three Waters across the St Andrews Catchment, an integrated approach is needed. This is achieved through comprehensive catchment planning – considering the current land use and future development across the whole catchment and planning for the anticipated services that will be required.

Development within the sub-catchment must be consistent with statutory central and regional government policies, plans and resource consents, and HCC policies and plans. Non-statutory policy and planning documents that may influence catchment management and development must also be considered.

2.1 Resource Management Act 1991 (RMA)

The Te Rapa Racecourse sub-catchment ICMP is a document that informs the Regional and District Council with regards to how the catchments resources will be managed in a sustainable way. Section 5 of the RMA defines 'sustainable management' as follows:

> *In this Act, "sustainable management" means managing the use, development, and protection of natural and physical resources in a way, or at a rate, which enables people and communities to provide for their social, economic, and cultural wellbeing and for their health and safety while —*

a) *Sustaining the potential of natural and physical resources (excluding minerals) to meet the reasonably foreseeable needs of future generations; and*

b) *Safeguarding the life-supporting capacity of air, water, soil, and ecosystems; and*

c) *Avoiding, remedying, or mitigating any adverse effects of activities on the environment.*

To meet this end, the starting point of the proposed development is to avoid as far as possible any adverse effects on the environment. This ICMP lays out strategies which will ensure the development occurs within the limitations of the site and catchment, by way of managing the Three Waters in a way which ensures the life supporting capacity of the receiving environment is not degraded, as well as ensuring the development avoids any adverse flooding effects. This will be balanced with providing for the communities social, economic, and cultural wellbeing as well – ensuring design concepts include community areas which encourage recreation and neighbourly interaction.

2.2 National Policy Statement for Freshwater Management 2020 (NPS: FW)

The Freshwater Management NPS has policies and objectives that direct local government to manage water in an integrated and sustainable way while providing for economic growth within specified water quality and quantity limits. The NPS requires councils to develop Regional Policy Statements and standards to safeguard the life supporting capacity of water bodies, with the objective that water quality will be maintained or improved. Indeed, the NPS: FW was recently updated, with a new version that came into force on 3 September 2020. The main changes being an increased focus on the protection of wetlands and other freshwater systems. The fundamental concept is "Te Mana o te Wai", which is explained as:

"…the fundamental importance of water and recognises that protecting the health of freshwater protects the health and well-being of the wider environment. It protects the mauri of the wai. Te Mana o te Wai is about restoring and preserving the balance between the water, the wider environment, and the community." Specifically, the NPS: FW has one Objective and associated Policies, as per below.

> *Objective 2.1*
>
> *The objective of this National Policy Statement is to ensure that natural and physical resources are managed in a way that prioritises:*

a) *first, the health and well-being of water bodies and freshwater ecosystems*

b) *second, the health needs of people (such as drinking water)*

c) *third, the ability of people and communities to provide for their social, economic, and cultural well-being, now and in the future.*

Supporting this objective are 15 Policies, of which the below are considered relevant to the urban setting that this sub-catchment ICMP relates to:

Policy 1: Freshwater is managed in a way that gives effect to Te Mana o te Wai.

Policy 2: Tangata whenua are actively involved in freshwater management (including decision making processes), and Māori freshwater values are identified and provided for.

Policy 3: Freshwater is managed in an integrated way that considers the effects of the use and development of land on a whole-of-catchment basis, including the effects on receiving environments.

Policy 4: Freshwater is managed as part of New Zealand's integrated response to climate change.

Policy 5: Freshwater is managed through a National Objectives Framework to ensure that the health and well-being of degraded water bodies and freshwater ecosystems is improved, and the health and well-being of all other water bodies and freshwater ecosystems is maintained and (if communities choose) improved.

Policy 11: Freshwater is allocated and used efficiently, all existing over-allocation is phased out, and future over-allocation is avoided.

Policy 12: The national target (as set out in Appendix 3) for water quality improvement is achieved.

Policy 15: Communities are enabled to provide for their social, economic, and cultural wellbeing in a way that is consistent with this National Policy Statement.

The above objective and policies mandate Regional Councils to consider development and associated activities in a holistic, whole of catchment manner – and to plan accordingly. Managing development in this way ensures that cumulative effects are identified at the planning stage and accounted for, with resources protected and used in a manner which ensures degradation is avoided and environmental improvement is the overall outcome. In accordance with this Objective, the Te Rapa Racecourse sub-catchment ICMP seeks to fulfil this directive through ensuring development and services are not designed in isolation of the wider catchment context. At this point, no specific water quality standards are considered to have been established for these purposes within the catchment, however, when considering development proposals/consent applications, councils must have regard for any effects (actual or cumulative) that contaminants contained in the discharge from developments may have on freshwater and freshwater ecology. The principle of adopting best practicable options in order to minimise effects is included in the decision-making process under this policy. Given the urban context of the site and reticulated network that forms the immediate receiving environment, there are no existing waterways which will be directly affected by the proposed activities or which require protection/enhancement. It is nevertheless important that development and associated runoff is managed in a way that ensures the water discharged from the site is of an acceptable quality so as not to compromise downstream environments.

2.3 Waikato Regional Policy Statement

The Vision and Strategy for the Waikato River (discussed below in section 2.6) is the primary direction-setting document for the Waikato and Waipa River catchments, and the Vision and Strategy is deemed to be part of the Regional Policy Statement. There are objectives laid out in the Waikato Regional Policy Statement that relate to how resources are managed.

Objective 3.1 addresses "Integrated Management" of natural and physical resources, recognising the inter-relationship between water body catchments, riparian areas, wetlands, the coastal environment, the Hauraki Gulf and Waikato River. The objective highlights that resources need to be managed in a way that recognises natural processes and interactions, while taking into account the needs of future generations as well.

Objective 3.2 allows for resource use and development to occur in a way which ensures the natural environment (inclusive of soils, water and ecosystems) is maintained and where appropriate enhanced, which ties in with Objective 3.10 which states that resources will be used in a sustainable and efficient way.

Policy 4.1.2 relates to land use change and intensification, and states the following:

> *Waikato Regional Council will work with territorial authorities to identify and manage the adverse effects of large-scale land use change or intensification, by taking account of:*

a) the potential to adversely affect the range of natural and physical resources, including effects occurring off site;

b) the potential cumulative effects;

c) opportunities to manage adverse effects in collaboration with territorial authorities, <u>tāngata whenua</u>, industry, landowners and other stakeholders; and

d) options for managing adverse effects including:

  i. regulatory and non-regulatory methods;

  ii. education and advocacy; and

iii. use of economic instruments.

Furthermore, Policy 6.1 calls for subdivision, use and development to occur in a planned and co-ordinated manner, again taking into account possible cumulative effects and being sufficiently informed to allow consideration of potential long-term effects. As the proposed development is occurring as a brownfields/infill development within an existing urban area, it is considered that it meets this Policy through undertaking development in area of similar landuse.

Policy 6.3 relates to co-ordinating growth and infrastructure to ensure timing of new development is co-ordinated with implementation and operation of transport and other infrastructure in order to:

a) optimise the efficient and affordable provision of both the development and the infrastructure;

b) maintain or enhance the operational effectiveness, viability and safety of existing and planned infrastructure;

c) protect investment in existing infrastructure; and

d) ensure new development does not occur until provision for appropriate infrastructure necessary to service the development is in place;

More specifically, Policy 6.3 (e) states:

e) that where new infrastructure is provided by the private sector, it does not compromise the function of existing, or the planned provision of, infrastructure provided by central, regional, and local government agencies

The proposed use and development, as well as proposed supporting infrastructure in context of the existing catchment is something this ICMP seeks to address and plan for.

2.4 Waikato Regional Plan

Under Section 30 of the RMA, the Waikato Regional Council is charged with certain functions regarding water, river and lake beds, land and soil, and geothermal resources. It is anticipated that many activities relating to these resources will either individually or cumulatively have adverse environmental effects and therefore require managing through permitted activity rules or resource consent. The Waikato Regional Plan has Objectives and Policies which guide resource use across the region. Policy 3.4.3 relates to the management of water use and states the following:

Manage, through permitted activities and resource consents, the use of water, any associated discharge of water onto or into land in a manner that ensures that:

a) The overarching purpose of the Vision and Strategy to restore and protect the health and wellbeing of the Waikato River for present and future generations is given effect to

b) The further degradation of water quality is avoided

c) Any adverse changes to natural flow regimes are avoided as far as practicable and otherwise mitigated

d) Adverse effects on the relationship tangata whenua as Kaitiaki have with water are avoided, remedied or mitigated

e) Adverse effects on in-stream ecological values are avoided, remedied or mitigated

f) Adverse effects on wetlands that are habitats for significant indigenous vegetation and significant habitats for indigenous fauna are avoided, remedied, or mitigated

g) Adverse effects on groundwater quality are avoided as far as practicable and otherwise mitigated

h) Does not result in an adverse effect relating to the objectives in Chapter 5.2 of this plan

i) The benefits to be derived from the efficient take and use of water for reasonably foreseeable future uses, and in particular for domestic or municipal supply, are maintained and/ or enhanced.

In relation to the management of stormwater – the WRC in section (methodologies) 3.5.11.2 commits to work with territorial authorities to ensure the integrated management of stormwater in the Region, and section 3.5.11.3 states that the WRC will work with resource users to:

1. *Find ways to mitigate adverse effects of existing stormwater discharges;*
2. *Promote the development of stormwater management plans which record the way in which the stormwater network is operated, including methods to avoid, remedy or mitigate the adverse effects of stormwater discharge; and*

3. *Promote alternative methods for the treatment and disposal of stormwater from existing and new subdivisions and development.*

These two methodologies promote the need for integrated management of stormwater with territorial authorities, recognising that they own and manage a large proportion of stormwater systems in the Region. In the Te Rapa Development, it is anticipated that the stormwater assets will be vested to the Hamilton City Council, therefore they have been involved in discussions at the design phase.

Also, as mentioned, the development will be planned through this ICMP process so as to ensure the above Policy (3.4.3) is adhered to – with potential adverse effects from the proposed development avoided and appropriately managed to ensure water quality is not adversely affected, and flooding as well as network capacity issues are addressed. Waikato Stormwater Management Guideline.

As previously mentioned, the WRC is charged with managing the regions resources as laid out in the RMA. As part of this responsibility, Waikato Stormwater Management Guidelines have been developed to help ensure urban stormwater is appropriately managed to help protect the regions waterways from further degradation, and where possible to restore and enhance them. The Te Ture Whaimana o Te Awa o Waikato – the Vision and Strategy is the prevailing document and is embedded within the Waikato Regional Policy Statement, covering both the Waipa and Waikato River catchments.

In line with these documents, the Waikato Stormwater Management Guideline document was created by the WRC to provide guidance when designing, constructing, and maintaining stormwater systems for urban areas, encouraging a low impact design approach. The guidelines provide design criteria and standard recommendations by the WRC which are directed by the above discussed documents and policies as guiding principles.

To ensure adequate stormwater management in both urban and rural areas is achieved a catchment approach is required when in the planning phase. Catchment management looks at all of a catchments' waterways and aims to result in an overall enhancement through low impact design philosophies when designing the management of the stormwater network. The Stormwater Management Guidelines direct developers and territorial authorities to consider erosion and sediment control, flood control, water quality, and ecological values when planning stormwater management for a catchment, as all these attributes are closely interrelated with stormwater. Te Ture Whaimana o Te Awa o Waikato.

In 2008 the Vision and Strategy for the Waikato River was published, closely followed by the establishment of the Waikato River Authority in 2010. The Vision and Strategy was a response to four key issues concerning the Waikato River catchment:

1. *The degradation of the Waikato River and its catchment has severely compromised Waikato River iwi in their ability to exercise mana whakahaere or conduct their tikanga and kawa;*
2. *Over time, human activities along the Waikato River and land uses through its catchments have degraded the Waikato River and reduced the relationships and aspirations of communities with the Waikato River;*
3. *The natural processes of the Waikato River have been altered over time by physical intervention, land use and subsurface hydrological changes. The cumulative effects of these uses have degraded the Waikato River; and*
4. *It will take commitment and time to restore and protect the health and wellbeing of the Waikato River.*

   (https://waikatoriver.org.nz/wp-content/uploads/2011/07/Vision-and-Strategy.pdf)

There are 13 objectives that were set out in order to realise the Vision, which need to be given effect to when planning catchment wide development and use of associated resources. Ultimately, waterways need to be protected and enhanced, with adverse cumulative effects avoided. A key objective is the recognition that the Waikato River is degraded and should not experience further degradation as a result of human activities. This ICMP will aim to coordinate development and the services that will support it in a way that does not contribute to the further degradation of the Waikato River (acknowledging that the immediate receiving environment is the reticulated stormwater network).

2.5 Waikato Regional Infrastructure Technical Specifications (RITS)

The Regional Infrastructure Technical Specifications (RITS) was developed as a Regional document to set the specifications for designing and constructing transportation, water supply, wastewater, stormwater and landscaping infrastructure. Prior to the RITS there were differing standards and requirements across the Waikato Region which were determined by each District Council. Consistency with the RITS when planning new infrastructure as part of any subdivision usually ensures compliance with the conditions set by councils as part of the resource consenting process.

This ICMP will explore development options and preferred design concepts for the infrastructure associated with the proposed development of the Te Rapa sub-catchment that are in-line with the RITS standards and specifications.

2.6 Hamilton City District Plan

In terms of managing all manners of "water" associated with development activities, 'Three Waters' is a principle that integrates the management of water supply, wastewater and stormwater for any proposed development in a strategic manner. This management approach is one of the principles adopted by the Hamilton City District Plan, addressed in Section 25.13 of the HCCDP. The below table shows the relevant Three Waters objective and associated policies:

Objective	**Policies**
25.13.2.3 Three Waters infrastructure is provided as part of subdivision and development, and in a way that is: • Integrated • Effective • Efficient • Functional • Safe • Sustainable	**25.13.2.3a** All subdivision and development provides integrated Three Waters infrastructure and services to a level that is appropriate to their location and intended use.
	25.13.2.3b Subdivision and development shall not occur unless the required infrastructure is available to service it.
	25.13.2.3c Three Waters infrastructure is to be designed and constructed in accordance with any existing Structure Plan and relevant Integrated Catchment Management Plan.
	25.13.2.3d Large scale subdivision and development proposals are to prepare an Integrated Catchment Management Plan (where one does not already exist) or a Water Impact Assessment.

Further to the above directives regarding how the Three Waters should be provided for and managed, the HCCDP states that where there is not already a full ICMP, then the following policies also apply

Design **25.13.2.3e** Three Waters infrastructure is designed and constructed to:	
i.	Minimise the effects of urban development on downstream receiving waters and groundwater.
ii.	Ensure that the capacity, efficiency and sustainability of upstream and downstream infrastructure will not be compromised.
iii.	Facilitate access, maintenance and operational requirements.
iv.	Cater for the potential effects of climate change.
v.	Ensure appropriate standards of public health, safety and amenity.

<table>
<tr><td>vi.</td><td>Ensure that surface water runoff is appropriately managed in accordance with the following drainage hierarchy.
1. Retention for reuse.
2. Soakage techniques.
3. Detention and gradual release to a watercourse.
4. Detention and gradual release to stormwater reticulation.</td></tr>
<tr><td colspan="2">Stormwater
25.13.2.3f
Stormwater management techniques are designed and constructed to:</td></tr>
<tr><td>i.</td><td>Maintain or improve the quality of stormwater entering the receiving environment.</td></tr>
<tr><td>ii.</td><td>Avoid or mitigate off-site effects from surface water runoff.</td></tr>
<tr><td>iii.</td><td>Sustainably manage the volume and rate of discharge of stormwater to the receiving environment.</td></tr>
<tr><td colspan="2">Water Supply
25.13.2.3g
Water supply infrastructure is designed and constructed to meet consumption, hygiene, water-sensitive design and firefighting requirements.</td></tr>
<tr><td colspan="2">Wastewater
25.13.2.3h
Wastewater is treated and disposed of in a way that minimises effects on public health, the environment, and cultural values.</td></tr>
</table>

The HCCDP is explicit in terms of the information requirements in relation to Sub-Catchment ICMP's, stipulating that as well as a Water Impact Assessment being required, an assessment of effects (and management of these effects) arising from the following aspects also needs to be included:

- Flood hazards;
- Stormwater disposal;
- Discharges of contaminants, and
- Identified network constraints.

This ICMP aims to address the above details in order to demonstrate a co-ordinated and considered approach to the proposed development which takes into account the existing setting and requirements as per the above objectives/policies and other guidelines and statutory documents.

2.7 Resource consents and designations

Development planning and implementation shall be carried out to comply with all HCC held resource consents, HCC Bylaws, levels of service, designations and easements. Resource consent requirements in relation to development will normally be communicated when a development application is assessed i.e. subdivision consent level. Nonetheless, this ICMP is considered to present appropriate stormwater management options to ensure that stormwater discharges from the planned development area are able to maintain consistency with the existing HCC comprehensive stormwater discharge consent requirements for the existing Hamilton City discharge network.

Developers and designers should seek advice from HCC as to the presence of designations or easements, prior to the initial planning phase. The same should apply to other major service providers i.e. power or gas.

2.8 Key strategic issues

Development within the sub-catchment must be consistent with both statutory and non-statutory central and regional government policies, plans and resource consents, and HCC policies and plans (as explored above). Most policy and rules ultimately flow out of the Regional Policy Statement (RPS) through planning documents such as District Plans. The RPS also reflects iwi aspirations for the region and National Policy Statements.

Relevant chapters in the HCC District Plan that reflect direction coming from the PRPS include Chapter 20 Natural Environments, Chapter 21 Waikato River Corridor and Gully Systems, Chapter 22 Natural Hazards, Chapter 23 Subdivision, Chapter 25.2 Earthworks and Vegetation Removal, Chapter 25.13 Three Waters (as explained above in Section 2.7), Chapter 2 Strategic Framework (see 2.2.7, 2.2.8 and 2.2.9), and Chapter 3 Structure Plans (see 3.3.3 and 3.3.6).

The key issues coming from the strategic planning documents listed in Section 2.1 above, that need to be addressed when developing an approach for integrated management of Three Waters infrastructure and urban development in the ICMP area include the following:

1. Erosion and instability of waterways
2. Water quality
3. Quality of riparian areas
4. Flooding and natural flow regime
5. Drainage in rural areas
6. Mauri of waterways
7. Water sensitivity
8. Adhoc planning
9. Indigenous biodiversity

2.9 Strategic objectives

In summary, the following Key Strategic objectives in the Hamilton City District Plan have been derived from these National and Regional planning policies and set the framework for urban development within sub-catchment (and all Hamilton city growth cells).

Table 1: Strategic Objectives

Objective 1	Where technically possible, development should incorporate a natural environment-based system. Onsite management and disposal of stormwater is preferred. Key elements of this approach include avoiding or minimising impervious surfaces, minimising earthworks during construction, and utilising vegetation to assist in trapping sediments and pollutants. (Rototuna Structure Plan).
Objective 2	Stormwater should as far as practicable be used to sustain groundwater levels in peat soils and base flows in freshwater receiving environments and stormwater infrastructure should result in a hydrological cycle as close to the predevelopment hydrological cycle as possible.
Objective 3	The quality of stormwater (and any wastewater) discharges to the existing stormwater network should not pose a risk to human or ecosystem health and will assist with enhancement of the water quality in the receiving environment
Objective 4	Stormwater is minimised and stormwater discharges managed to avoid adverse effects on channel stability, rural drainage, aquatic life supporting capacity and protect and enhance natural flow regimes in waterways, and maintain and enhance the values of ecologically significant freshwater habitats
Objective 5	Manage stormwater discharges to mitigate the effects of flooding on both existing and new urban areas, while avoiding and remedying the adverse effects of channelization and channel deepening on ecologically significant freshwater habitats.

Objective 6	Restore and protect the health and mauri of the catchment waterways and restore and protect the relationship of tangata whenua as Kaitiaki of waterways.
Objective 7	Wastewater generation is minimised, and wastewater discharges are managed such that no adverse effects are encountered on HCC's existing infrastructure network or natural environment.
Objective 8	Potable water consumption is managed to minimise peak and total demand (Citywide).
Objective 9	Infrastructure needs i.e. pump stations and stormwater treatment and detention devices are minimised.
Objective 10	Water networks accommodate growth in accordance with water conservation and demand management objectives and potable water consumption is managed to minimize peak and total demand.

2.10 Waikato Sub-Regional 3 Waters Strategy/HCC Infrastructure Strategy

2.10.1.1 Waikato Sub-Regional 3 Waters Strategy

The Waikato sub-regional three waters study is being delivered through the Future Proof partnership and is one of the initiatives being delivered as part of the broader Hamilton to Auckland Corridor Plan.

The study focuses on the development, delivery and management of municipal three waters (water, stormwater and wastewater) infrastructure for urban settlement areas surrounding Hamilton City and includes parts of the Waikato and Waipa districts and all of the Hamilton City Council jurisdiction. A key focus of this strategy is how future development infrastructure requirements may be able to be accommodated across the various district boundaries to ensure a coordinated and practical management approach.

Having reviewed this strategy in relation to the proposed development, the strategy comprises a broad scale strategy document to be developed and implemented over extended timeframes and is not considered to present any specific matters of relevance to 3 waters management within the subject development site.

2.10.1.2 HCC Infrastructure Strategy

Similarly, the HCC Infrastructure Strategy also comprises a high level strategy document with the key purpose of to identify significant infrastructure challenges for Hamilton City Council over the next 30 years, and to identify the principal options for managing those challenges and the implications of those options. The Strategy outlines how the Council intends to manage its infrastructure assets, including the need to renew or replace existing assets, respond to growth or decline in demand for services, and provide for the resilience of its assets.

In this instance, the methods proposed through the ICMP to manage the infrastructure requirements for the development including provision for a proposed stormwater treatment/attenuation wetland, consideration of retro-fitting the design for existing up-catchment flows, management of identified flood risks and identification of required infrastructure upgrades are considered to present a proposed infrastructure strategy for the site which is consistent with the high level direction presenting in this strategy.

2.11 Levels of Service

Levels of Service are documented in different levels of detail in various key documents. Refer to the following documents for further level of service and key design standards information:

a) Operative & Proposed District Plan

b) HCC Infrastructure Technical Specifications

c) HCC Stormwater Modelling methodology

d) HCC Standard Assessment Methodology - Water (Model)

e) HCC Standard Assessment Methodology - Wastewater (Model)

f) HCC's 2015-25 10 Year Plan

g) Waikato Regional Council's Long-Term Plan 2012-2022

h) Bylaws

i) HCC Water Master Plan June 2015

j) HCC Draft Wastewater Master Plan May 2015

k) WRC Technical Report 2014/13 –Managing land use change and Council's administered drainage area

3 Existing Site Information

3.1 Land Use

3.1.1.1 Existing Land Use

Existing land use within the Racecourse Re-development area is predominantly for activities associated with the Te Rapa Racecourse. Three large stable buildings occupy the centre of the site, with external horse pens and paddocks to the south of these stables. The remainder of the area is open space used for exercising and grazing horses – refer Figure 3.

Figure 3: Existing Site Landuse

The site is in the Major Facilities Zone as identified within the Operative Hamilton City District Plan – refer Figure 4.

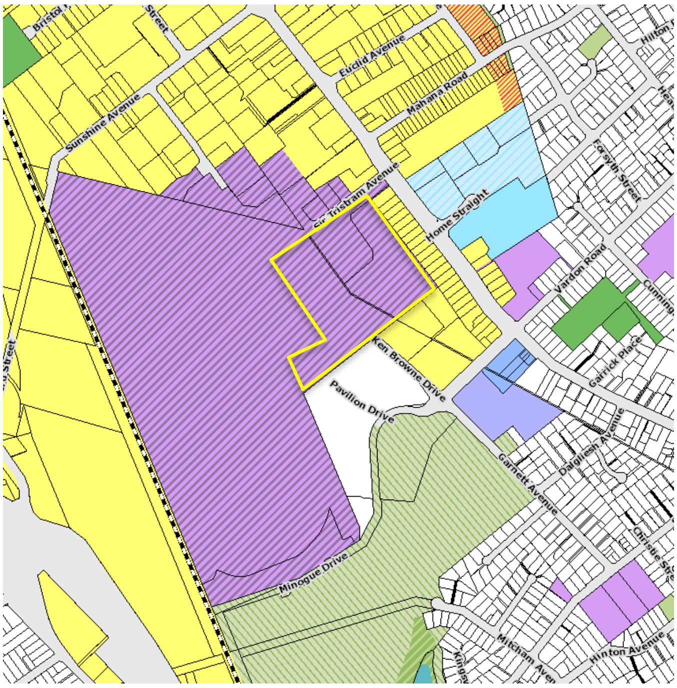

Figure 4: Major Facilities Zone and Thoroughbred Business Park- (Source HCC Online Maps)

Specific rules within the Major Facilities Zone -Te Rapa Racecourse are outlined in Rule 17.6 of the HCC District Plan. Maximum building coverage within the zone is currently 25%.

3.1.1.2 Proposed Land Use

The site will be developed for a variety of standalone, duplex, terraced and apartment living. The living environment has been master planned to provide an environment closely integrated with the Te Rapa Racecourse. In addition to providing an attractive gateway to the racecourse, the residential development will also enable the establishment of some dwellings which will enjoy an outlook over the race track and adjoining park like grounds. It is likely that the higher density residential development will be clustered around these areas.

The residential development area is approximately 6.5ha (not including the central open space area). The concept design indicates that the likely yield is approximately 200 residential dwellings based on a mix of single dwellings, duplexes, terrace houses and apartments. This creates a gross density of approximately 31 dwellings per hectare.

The initial development concept plan prepared for the site is included within Appendix A.

3.1.1.3 Physical Environment

3.1.1.4 Existing Topography

The existing Racecourse Redevelopment Area topography can be generally characterised as a shallow low-lying basin, with ground levels on the eastern and western site boundaries generally falling toward a low point which runs from northwest to southeast through the centre of the site. Ground levels to the south of the site are generally level with the southern part of the site. Ground levels on the northern side of Sir Tristram Ave are generally lower than the site. Within the Re-development Area two small sub- catchments are delineated by a slight high point/divide running south to north through the centre-east of the site.

Ground levels within the proposed Re-development Area range from RL38.0m on the western boundary (to the south of the existing grandstand) to RL32.6m within the low point running through the centre of the site.

3.1.1.5 Catchment Area

The Racecourse Redevelopment Area has a total site catchment area of 8.8ha (including the Central Open Space area and Road areas).

Upstream flows from a localised industrial catchment area are conveyed southward through the site via the existing stormwater network (refer below). Hence, the immediate site catchment is limited to the site itself with the developed area generally forming the catchment boundary, except on the eastern side where a small portion of the external lots grade toward the subject site. On the western side the catchment divide is located in-line with the existing grandstand building with ground levels to the west falling toward the racecourse track. An additional catchment area to the south of the redevelopment area is also identified as discharging overland flow to the subject site in extreme storm events (refer below).

3.1.1.6 Geology and soils

- CMW Geosciences (CMW) undertook a geotechnical assessment of the proposed development site in May 2017. Refer to Appendix E for the Geotechnical Investigation Report. Key extracts from the report in relation to 3 water management are provided below:
- *The geological map of the area indicates that the site is underlain by fluvially reworked soil deposits of the Hinuera Formation. The Hinuera Formation infills the majority of the Waikato Basin and deposits generally comprise interbedded sands, silts and clays with interspersed peats.*
- *The relict Te Rapa Channel extends through the eastern part of the site which is a paleo river channel when the Waikato River was a braided system.*
- *In summary, the Hinuera Formation materials at the site consist of the following:*
  - *The upper 0.8m consists of firm to very stiff silt and sandy silt*
  - *Underlying the surface silt, primarily medium dense to dense sand/silty sand with interbeds of stiff to very stiff silt/sandy silt to depths of around 7.0 to 13.0mbgl*
- *Groundwater was encountered at the test locations within the proposed development area at depths of 1.5m to 2.8m below the current ground level (RL 32m to 33.2m). In the weeks and months prior to the site investigation there were several significant rainfall events which are likely to have contributed to an elevated groundwater table.*
- *Stormwater Disposal - Conventional soakage trenches or soakholes are considered a practical solution for the disposal of stormwater where located within the proposed building development area due to the sands encountered. Groundwater was encountered between 1.5m to 2.8m below the current ground level throughout the building development area. A coefficient of permeability (k) of 1 x 10-5 m/sec to 5 x10^6m/sec should be used for modelling unless further site-specific testing is undertaken.*

3.1.1.7 Soakage Suitability

Two falling head percolation tests were undertaken by CMW to assess the permeability of the near surface soils underlying the eastern part of site. Testing was undertaken within 100mm diameter hand auger boreholes, drilled to depths of between 1.5m to 2.2m and pre-soaked (filled with water) and allowed to drain over approximately 2.5 hours prior to testing.

CMW calculated permeability based on guidelines presented in CIRIA 1133. Coefficient of Permeability rates of between 1 x 10^{-5} m/sec to 5 x 10^{-6} m/sec were observed within the Hinuera sands and silty sands. Lower permeability values were attributed to silt migration from the upper portion of the boreholes causing “caking” of the sides and base during testing. These average soakage rates were calculated in accordance with the NZBC method as 450mm/hr and 300mm/hr in boreholes HA03 and HA05 respectively. The above soakage rates are above the minimum rate of 150mm/hr stipulated in the RITS Section 4.

However, based on the limited soakage testing undertaken to date, elevated water tables and the relatively low soakage rates encountered, soakage is not considered a viable method of stormwater disposal for the primary (10 year) storm events. Further testing is recommended at time of future resource consent process to gain a better understanding on soakage suitability to address for small events such as the ‘Reduced at Source Soakage Measure[1]” to cater for the HCC DP Water Efficiency Measure requirements. The nature of the proposed site development work and earthworks, and associated geotechnical improvements will greatly affect the soakage suitability across the site.

3.2 Receiving Environment

3.2.1.1 Introduction

The Racecourse Re-development area is serviced by existing HCC stormwater reticulation.

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[1] HCC Three Waters Management Practice Note – HCC 03: Soakage, Section 3. Soakage – Reduced at Source Measure

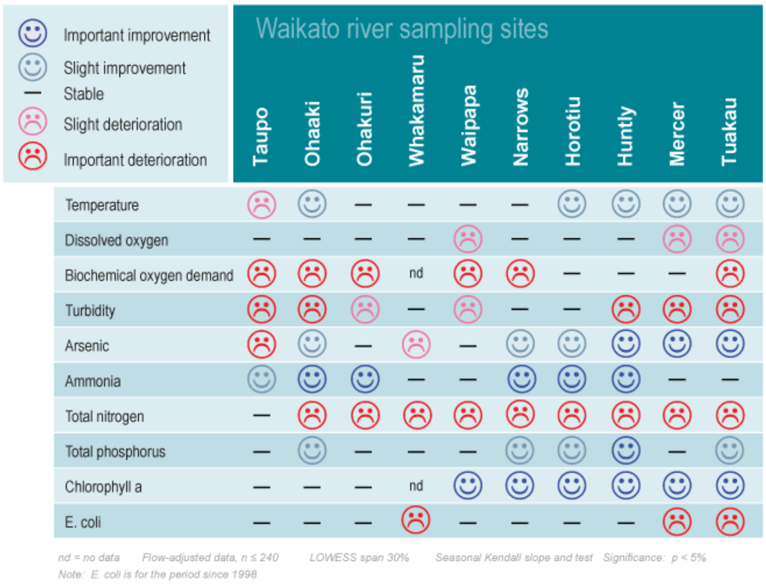
Stormwater from a localised upstream catchment area which includes runoff from various industrial properties, the Fairview Motors car dealership and Sir Tristram Avenue is captured within the HCC reticulation network and enters the site at its northern boundary within a 1050mm stormwater main. This main extends through the middle of the Re-development Area, draining north-west to south-east, and connects to a 1650mm pipe within Garnett Avenue. Flows are then conveyed eastward toward the Waikato River collecting stormwater from numerous industrial/commercial/residential properties as well as the high volume Te Rapa roading network with all inflows from this extensively developed catchment expected to be occurring in an untreated/unattenuated manner. The stormwater catchment network eventuates with an 1800mm pipe discharging into the Waikato River in the vicinity of Minchin Crescent approximately 1.5km from the subject site.

3.2.1.2 Waikato River

At 425 km long, the Waikato River is the regions, and the countries, longest and most significant river.

Water quality in the Waikato River is not always good enough for swimming. It is safe to swim upstream of Hamilton city, but levels of Escherichia coli ('E. coli') bacteria (an indicator of health risk) in the city reaches and downstream were often above the safe level for swimming. Higher bacteria levels in the lower river are the result of the combined discharges from farm and stormwater runoff, farm dairies and sewage treatment plants.

Figure 5 below shows the changes in several water quality measures during the 20-year period between 1995 and 2014. Overall, 15 per cent of water quality measures improved at individual sites, and 24 per cent deteriorated.



- 😀 Important improvement
- 🙂 Slight improvement
- — Stable
- 🙁 Slight deterioration
- 😠 Important deterioration

	Waikato river sampling sites									
	Taupo	Ohaaki	Ohakuri	Whakamaru	Waipapa	Narrows	Horotiu	Huntly	Mercer	Tuakau
Temperature	🙁	🙂	—	—	—	—	🙂	🙂	🙂	🙂
Dissolved oxygen	—	—	—	—	🙁	—	—	—	🙁	🙁
Biochemical oxygen demand	😠	😠	😠	nd	😠	😠	—	—	—	😠
Turbidity	😠	😠	🙁	—	🙁	—	—	😠	😠	😠
Arsenic	😠	🙂	—	🙁	—	🙂	🙂	😀	😀	😀
Ammonia	🙂	😀	😀	—	—	😀	😀	😀	—	—
Total nitrogen	—	😠	😠	😠	😠	😠	😠	😠	😠	😠
Total phosphorus	—	🙂	—	—	—	🙂	🙂	😀	—	🙂
Chlorophyll a	—	—	—	nd	😀	😀	😀	😀	😀	😀
E. coli	—	—	—	😠	—	—	—	—	😠	😠

nd = no data   Flow-adjusted data, n ≤ 240   LOWESS span 30%   Seasonal Kendall slope and test   Significance: p < 5%

Note: E. coli is for the period since 1998

Footnote to trends chart

- 'Stable' means that overall change in average values is likely just to be due to chance (at the 5 per cent level).
- Where a trend is identified, any overall change is unlikely to be due to chance (at the 5 per cent level).
- A slight trend has a rate of change which is less than 1 per cent of the median value per year.
- An important trend has a rate of change which is more than 1 per cent of the median value per year.

Figure 5: Water quality trends in the Waikato River between 1995 and 2014 (based on methods from Waikato Regional Council's Technical Report 2013/20.)

As the region continues to grow and develop, putting pressure on the river's catchment, careful management is needed to maintain and improve the quality of the Waikato River.

3.3 Surface Water Quantity and Flooding

3.3.1.1 Rapid Flood Hazard Modelling

A Rapid Flood Hazard Modelling Assessment (RFHM) was undertaken by AECOM in several areas throughout the City to provide a high level of understanding of areas that may be subject to flooding in significant storm events (100 year ARI event). The results from the RFHM were used to identify areas of priority within the city for Detailed Flood Hazard modelling.

The ‘Flood Hazard Report’ prepared by AECOM Ltd and dated 2012 outlines the RFHM background and limitations of the modelling. The St. Andrews area was modelled as part of the City-Wide Catchment Model with 5m cells adopted over the catchment extents. The 5m RFHM results have the least confidence due to the larger cell size. Pipes smaller than 900mm diameter have been ignored in the model and 100% of the rainfall is modelled as runoff i.e.100% impervious surface throughout the catchment.

Rainfall data was taken from the HCC Development manual allowing for 2.08 ºC for Climate Change. 2008 LIDAR data was used to develop a ground surface profile. This LIDAR data is understood to have an accuracy of +/- 250mm. The range of uncertainty in the water levels from the RFHM results due to the inaccuracies from LIDAR and other factors can be in some cases up to 0.50m.

HCC have provided the RFHM data for the Te Rapa Racecourse Area in the form of screen shots of the subject site and surrounding area. Data included;

- Flood Depth data (m)
- Flood velocity data (m/s) and
- Flood level data (RL m)

Based on the RFHM data received, an assessment was undertaken of the flood hazard and the overland flow paths surrounding the subject site to understand what effects the proposed plan change will have on existing properties downstream of the development area, and to confirm the stormwater quantity management objectives to be adopted within the development site.

The flood maps show that flooding is anticipated within the Racecourse Redevelopment Area during the 100 year ARI event with flood depths ranging from 0.1 -1.14m.

Flood velocity and flood level data indicates that the flood waters are generally slow moving, occurring within a generally confined corridor in a south to north direction, being the opposite direction to the primary stormwater reticulation flows.

The RFHM shows floodwaters from an area to the south of the site, bounded by Garnett Ave and Ken Browne Drive and the residential lots to the east, are conveyed through the to the Racecourse Re-development area. To the north of the site floodwaters pond in a large low-lying area through the centre of the area bordered by Te Rapa Road and Mainstreet Place. This flooded area continues north of Sunshine Avenue becoming concentrated along Sheffield Street. This generally low lying, flood prone area extending south to north through the development site and beyond is understood to be the general location of the now relict Te Rapa Channel, a paleo river channel once forming part of the Waikato River.

Figure 6: Extent of 100 year ARI flooding based on RFHM

It should however be acknowledged that there is a significant impediment to flood waters entering the subject site from the south due to a solid fence and retaining structures. Refer images below showing the neighbouring property.

Figure 7: Southern boundary fence looking south-west from Racecourse side

Figure 8: Southern boundary fence and retaining looking north-west from carpark at No 6k Ken Browne Drive

3.3.1.2 Flood Volumes- Racecourse Re-Development Area

100 year ARI flood levels within the Racecourse Re-Development Area are generally at RL33.44m at the southern end of the site falling to 33.40m in the central -north part of the site. Several small isolated areas of higher flood elevation are shown throughout the site however as the corresponding flood depths are all generally lower than 0.20m deep in these areas they have been ignored in the subsequent flood volume calculations.

An assessment has been undertaken to determine the volume of flood storage across the site based on the HCC RFHM and detailed topographic survey. The analysis shows there is approximately 7,500m^3 of flood storage volume within the Racecourse Re-Development Area.

Refer to DWG WE1733-03-03.2 for flood volume depth and volumes

3.4 Existing Utilities and Network

Service plans have been provided in Appendix D. Descriptions of the existing stormwater, wastewater and water supply reticulation is provided in Sections 5.0, 6.0 and 7.0 respectively.

4 STORMWATER MANAGEMENT

4.1 EXISTING UTILITIES AND SERVICES

The Racecourse Re-Development area is currently serviced with stormwater reticulation. No known stormwater treatment or attenuation measures are present within the subject sites.

4.1.1.1 Racecourse Re-Development Area

As noted, an existing upstream industrial catchment to the north drains into the site via existing 1050mm stormwater main. This main runs through the middle of the site, draining north west to south east forming the main stormwater conveyance pipe within/through the site.

A 525mm line runs parallel to the existing stables (to the west of the 1050mm) discharging to a 750mm main and ultimately the 1050mm pipe via a manhole located near the south eastern end of the site.

A 225mm SW pipe runs along the southern boundary of the site and discharges to the 1050mm main via a manhole in the adjacent property. The 225mm pipe receives runoff from the catchpits at the end of Ken Browne Drive and another catchpit located within the development site

A 450mm SW pipe runs along the south eastern boundary of the development site. The pipe starts within the Service lane adjacent to Te Rapa Road and receives runoff from several industrial/commercial lots to the east of the site prior to discharging to the 1050mm main via a manhole within the development site.

The 1050mm main discharges to a 1650mm main on Garnett Avenue, to the south of the site which drains to the north east. The 1650mm continues east along Vardon Road prior to discharging to a 1800mm pipe near the intersection with Cunningham Road. The 1800mm main continues to the east, discharging to the Waikato River via an outlet near Michin Crescent approximately 1.5km from the site.

Figure 9 below shows the existing stormwater reticulation within the site.

Figure 9: Existing Stormwater Reticulation (Source: HCC 3 waters GIS Viewer – May 2015)

4.1.1.2 Existing Hydrology

The proposed Racecourse Re-Development Area has been designated as one catchment, with stormwater flows currently discharging to the central 1050mm SW pipe via the internal drainage network. Refer to Catchment plans in Appendix A for further details.

The existing site consists of several buildings, pathways, and other impervious areas. Table 2 below presents the existing site catchment breakdown.

Table 2: Existing Site Catchment Breakdown

Existing site Sub-catchment	Area (Ha)	% breakdown
Roof and other impervious areas	2.54	29
Pervious area	6.28	71
Total Area	8.82	100

A TP108 analysis was undertaken to determine rainfall runoff values with CN values adopted for the existing site catchment in accordance with TP108 guidelines. Based on the soils found in the geotechnical assessment soil Type B was adopted for the subsequent TP108 calculations. A weighted CN runoff curve number was calculated based on existing impervious percentages found within the site in accordance with ARC TP108 methodology. A curve number of CN = 61 was adopted for all pervious areas, and a curve number of CN =98 adopted for all impervious/hardstand areas.

Rainfall data was taken from the RITS Section 4 Stormwater with the existing climate rainfall used for the pre-developed scenario.

Table 3 below presents the key results from the pre-development assessment.

Table 3: Pre-development Hydrologic Assessment

ARI Event	Racecourse Re-development Area	
	Peak flows [m^3/s]	Runoff Volume [m^3]
2 Year	0.384	1,993
10 Year	0.815	3,963
100 Year	1.441	6,528

4.1.1.3 HCC Reticulation Capacity

Hydraulic capacity assessment of the reticulated pipe network downstream of the proposed development area has not been undertaken as part of this sub-catchment ICMP process. The pipe networks downstream of both the Re-Development Area is over 1 kilometre long with an estimated developed catchment in the order of 160Ha.

A preliminary capacity assessment has been undertaken on the existing HCC 1050mm SW line extending through the subject site to confirm whether the existing pipe has sufficient capacity to meet the current RITS level of service (10 year ARI without surcharge). Refer summary of calculations undertaken below:

1. Catchment hydrology based on pre-development ‘existing’ land-use, with climate change adjusted rainfall.
2. Analysis extends downstream to SWMH SWO16057 located within Garnett Avenue where the 1050mm pipe turns to the north-east and increase in size to a 1650mm pipe.
3. Hydraulic assessment undertaken using both Manning’s part full pipe assessment and SSA for 2,5,10 and 100 year ARI events.

4. Existing pipes and manhole parameters were obtained from HCC GIS Viewer.
5. The 1050mm pipe extending through the site is at a notably flat grade in the order of 0.1-0.2%.
6. Refer attached calculations within Appendix H

Results of the modelling shows the 1050mm pipe does not have capacity to convey the 2 year ARI flows without surcharge and overflow at the upstream extent of the site (EX-SWN15031). Further surcharging and overflows occur for larger storm events.

Refer Section 4.4.3 below for recommendations for capacity upgrades as part of future development work.

4.2 STORMWATER MANAGEMENT STRATEGY

4.2.1.1 Stormwater Quantity Management

Based on the proposed primary discharge to existing HCC stormwater infrastructure and presence of flooding within the subject site and wider catchment, peak flow attenuation is considered a key requirement for the sub-catchment and future developments.

- In general accordance with the RITS (Table 4-3) the following stormwater quantity design parameters are recommended for the sub-catchment:
- Flow Attenuation - Match pre-development flow rates for the 2 and 10 year ARI events through controlled attenuation and multi stage outlets or devices that reduce volume of runoff
- Flood Control (100 year ARI event) - Detention required, limiting the post development 100 year ARI event flow rates to 80% of the pre-development 100 year ARI event flow rates (required where downstream flooding exists).
- Volume Control and Extended detention is not specifically required for the sub-catchment as flows are conveyed to the Waikato River within pipe reticulation with no risk of channel erosion.
- Protection of overland flow paths and retention of flood storage volumes is also considered an important requirement to ensure no adverse flood effects on properties both upstream and downstream of the proposed developments and to avoid flooding of the residential development area..
- Flood Volume mitigation is also required to ensure the increased runoff volume from the developments do not exacerbate flooding both upstream and downstream of the site.

4.2.1.2 Stormwater Quality Management

As runoff from the site ultimately discharges directly to the Waikato River, a high level of water quality treatment is considered vital.

In general accordance with the RITS (Table 4-3) the following stormwater quality design parameters are recommended for all discharges:

- Total suspended solids (TSS) (75% removal of post development loads taken as being at the discharge point from site).
- Total Metals (copper, zinc) to achieve maximum practical removal possible.
- Temperature (<25°C)
- Nutrients (total nitrogen, total phosphorus and ammoniacal nitrogen) to achieve maximum practical removal rates.
- Hydrocarbons to achieve maximum practical removal rates
- Removal of gross pollutants (litter and commercial waste).

4.2.1.3 Discharge Parameters

A summary of the required discharge parameters for all development with the Te Rapa Racecourse Sub-catchment are presented below:

Table 4: Te Rapa Racecourse Sub-catchment ICMP discharge parameters

	Parameter	**Requirement**
A	Suspended solids (TSS)	75% removal
B	Extended detention	Not required – however may be utilised to reduce constructed wetland WQV requirements.
C	Volume Control	Not required – Discharge to existing HCC pipe reticulation with ultimate discharge to the Waikato River
D	Flow Attenuation	Match pre-development flow rates for the 2 and 10 year ARI events through controlled attenuation and multi stage outlets or devices that reduce the runoff flow.
E	Flood Control (100-year ARI event)	Detention required, limiting the post-development 100 year ARI event flow rates to 80% of the pre-development 100 year ARI event flow rates.
F	Temperature	<25° Celsius at point of discharge
G	Contaminants	Refer to requirements of RITS (current standards are outlined in Section 5.2.2 above)

4.3 STORMWATER MANAGEMENT MEASURES

The overall development shall be advanced based on water sensitive design principles and integration of stormwater management into all design stages of the proposed developments.

Water sensitive urban design practices, such as minimising hard stand areas, clustering development, providing at-source treatment measures, and using a treatment train approach, are all applicable.

When selecting stormwater management solutions, the following HCC hierarchy shall be adopted with regards to disposal.

a) Retention of rainwater/stormwater for reuse

b) Soakage techniques

c) Treatment and detention and gradual release to a watercourse

d) Treatment and detention and gradual release to a piped stormwater system

Proposed methods to achieve these water sensitive design principles for the Re-development area are outlined within the following sections.

4.4 STORMWATER MANAGEMENT RECOMMENDATIONS

The recommended stormwater management strategy for the site comprises the collection of development catchment runoff within a reticulated stormwater network for conveyance to a centralised constructed stormwater wetland located within the central open space area with a controlled discharge outlet to the existing 1050mm outlet pipe draining the site. Further details of the recommended strategy are described as follows and outlined on the plan attached within Appendix B.

In this instance, the collection of catchment runoff within a reticulated stormwater network for conveyance to a centralised stormwater management device is considered to present the most practical option to achieve the identified stormwater management functions for the site. Hence the recommended stormwater concept for the development area comprises provision of a stormwater treatment/attenuation wetland device located within the large central open space area. It is anticipated that this area will be utilised for a centralised wetland device which would be vested to HCC for long term operation and maintenance as part of the municipal stormwater network.

4.4.1.1 Post Development Hydrology

A preliminary hydrologic assessment of the proposed development has been undertaken using HEC-HMS. The post development assessment has adopted a fully developed scenario according to the proposed land uses and have been assigned impervious fractions according to the maximum levels allowable within the Operative District Plan (Medium Density Residential Zone = 80% impervious). The Roads and ROW's within the development have been assigned an impervious percentage of 90%.

The RITS requires that for all catchments where detention storage is required, stormwater modelling shall be undertaken using a 24-hour design storm. The 24-hour temporal rainfall pattern was taken from the WRC Stormwater Modelling Guidelines and adjusted using site specific rainfall from Hirds v4. The post development analysis was determined using the RCP6.0 climate change adjusted rainfall. This was extrapolated out to provide a 100 year design horizon by adjusting the historical rainfall by 2.3 degrees and using the percent change per degree of temperature increase values provided by MfE. The model was also run using the RCP8.5 scenario rainfall to determine water levels within the wetland and any corresponding effects within the proposed subdivision were this climate change scenario to occur.

Runoff calculations were undertaken in accordance with the methodology outlined in the WRC Stormwater Modelling Guidelines 2020. Soil types and CN numbers have been adopted based on soil testing across several sites within the catchment and with supporting information from S-Maps Online. These soil types correspond with soil testing undertaken within the catchment.

Pre-developed soil types have been classified as Soil Type B. Soils within the lower lying areas of the site have been classified as Soil Type D.

Table 5 below presents the key results from the post-development assessment, including a preliminary assessment of the required detention volumes to achieve the discharge parameters.

Table 5: Post-development Runoff and Detention Volumes

ARI Event	**Racecourse Re-development Area**			
	Post-development Peak flows (m^3/s)	**Detention Volumes* [m^3]**	**Discharge Volume [m^3]**	**% Volume increase**
2 Year	0.354	1,856	4,231	+53%
10 Year	0.727	2,903	7,376	+46%
100 Year	1.145**	4,589	12,527	+48%

* Assumes no on-lot soakage systems within the sub-catchment

** 80% of 100 year ARI Greenfields flows

4.4.1.2 Constructed Stormwater Wetland

Constructed stormwater wetlands are systems built to mimic the water cleansing processes of natural wetlands. Wetland environments represent the intersection of aquatic and terrestrial ecologies and support a wide variety of vegetation types. In this way, they can be designed as a landscape feature of significant amenity, with diverse habitat types, and opportunities for passive recreation.

If sited within accessible open space, constructed wetlands or ponds can significantly enhance the built environment and provide a suite of environmental services (e.g. interception of dust, moderation of heat, noise, and light). Wetland environments provide a refuge for local residents and a place of tranquillity. Elements of water and associated lush native vegetation adds significantly to the amenity of a development

Wetlands also provide a destination for passive recreation, with potential viewing areas, pathways, and gathering spaces. Community education is also possible with appropriate information or public art. Constructed wetlands are optimal areas for education as they can demonstrate basic principles of plant succession, food webs, and nutrient cycling.

Detailed design of the constructed stormwater shall be undertaken in accordance with Section 4.2.17 of the RITS.

Key aspects of the proposed wetland device are outlined as follows.

Wetland Footprint/Sizing

The footprint of the proposed wetland device is shown on the preliminary stormwater management plans.

- Top of batter area = 4,500m^2 (5.1% of contributing sub-catchment area)
- Weltand base area = 2,500m^2 (2.8% of contributing sub-catchment area)

Location of the wetland within the central open space area presents an opportunity to adjust the wetland footprint area against the adjacent Racecourse Show/Arena Area as part of the detailed design process to ensure that the finalised wetland design sizing requirements can be achieved.

Wetland Stormwater Treatment

The wetland device should be designed to incorporate the following key features to ensure that stormwater quality treatment objectives are achieved for the development area:

- Provision of an inlet forebay for inflow dissipation, capture of large sediment particles and large catchment contaminants (e.g refuse) and to provide a location for routine maintenance.
- Landscape planting of native aquatic wetland plants across at least 80% of the wetland surface area to maximise settlement, biofiltration of soluble contaminants and shading of stormwater flows to minimise thermal effects – along with enhanced amenity values of the wetland device;
- Provision of a controlled discharge outlet to maintain the required wetland water quality storage volume.
- Provision of these water quality features within the wetland design are considered appropriate to ensure that best practice stormwater treatment will be provided for the development site in accordance with the best practice methods promoted through the RITS and WRC Stormwater Guideline documents to mitigate any adverse water quality effects within the Waikato River receiving environment.

It is notable in this instance, that based upon the brownfields nature of this site the proposed Re-development area will likely comprise the only site within the broader urban development catchment providing water quality treatment of catchment flows prior to entering the Waikato River.

Wetland Stormwater Attenuation

The wetland device can be designed to incorporate the following key features to ensure that stormwater quantity attenuation objectives are achieved for the development area:

- Sizing of the wetland storage volume to provide for attenuation of the post development stormwater flows to pre-development levels for the 2, 10 and 80% of the 100-year events. The proposed storage volume is 7,500m^3. Provision of a staged outlet weir system at the outlet point to the downstream stormwater network to ensure that the above attenuation objectives are achieved.

Provision of these stormwater attenuation features within the wetland design are considered appropriate to ensure that best practice stormwater attenuation will be provided for the development site in accordance with the best practice methods promoted through the RITS and WRC Stormwater Guideline documents to mitigate any potential increase in flows within the downstream stormwater network. Accordingly, the proposed development site will not cause increases in peak flow rates or downstream pipe capacity issues up to and including the extreme 1 in 100 year event.

Wetland Ownership/Operation and Maintenance

The intention is that the centralised stormwater wetland device will be constructed by the site developer and will then be separated from the balance of the centralised open space area within a localised parcel of drainage reserve as part of the site subdivision process. Following a suitable defects liability period, the wetland device can then be vested with HCC for long term operation and maintenance responsibility as part of the broader catchment stormwater network.

A detailed Operation and Maintenance Plan will need to be developed for the wetland to establish appropriate monitoring and maintenance procedures to ensure that the design stormwater management objectives of the wetland are maintained on an ongoing basis. Provision of appropriate maintenance access will be paramount to the ongoing operation and function of the wetland, and will need careful consideration as part of future detailed design.

4.4.1.3 Stormwater Conveyance System

Primary Reticulation

The primary stormwater network will convey runoff from all development within the sub-catchment to the proposed wetland forebay within the central open space. The primary stormwater network could consist of the following elements:

- Kerb and channel conveyance;
- Cesspits for collection;
- Pipeline conveyance;
- Energy dissipation devices at outlets

The primary network for the development can generally follow the public road corridors and shall be sized to provide a 10-year ARI level of service in accordance with Table 4-7 of the RITS. With the proposed receiving wetland device located within the lower lying central open space area, pipes can be graded to gravity drain to the inlet forebay with provision of appropriate flow dissipation/erosion protection at the point of discharge.

The current development layout shows residential lot areas located over the existing stormwater reticulation running through the Racecourse Redevelopment site. As outlined in section 4.1.3 above, the 1050mm pipeline extending through the subject site does not have sufficient capacity for the required 10-year ARI design level of service.

Consideration should be given to diverting the stormwater pipes around the proposed residential lots to limit the number of potential build-overs, particularly for the larger 1050mm and 750mm lines. As noted, the existing stormwater reticulation passing through the site does not have adequate capacity to convey the 2 year catchment flow without surcharge. Hence it is recommended that site development includes upsizing and redirection to meet current RITS standards (i.e. 10 year ARI pipe capacity with pipe alignments following public road corridors).

Upsizing of pipe capacity downstream of the site is however not a prerequisite as attenuation will be provided to mitigate effects of any upstream development. It is however likely that over time the capacity of the downstream network will be increased to meet required levels of service as part of future catchment redevelopment activities or Council initiated upgrades.

Secondary Overland Flow

The secondary network shall be designed to accommodate flows from a 100yr ARI rainfall event. These flows will be safely conveyed from property and within the road corridors and open spaces. OLFP shall be designed to convey flows from all upstream land, allowing for fully developed, un-attenuated runoff.

Where possible, development shall be designed to incorporate OLFP's within the road reserves and green spaces so as not to cause nuisance to the built environment. Freeboard above the top water levels within the OLFP's during 100yr rainfall event shall be provided to habitable floor as per the HCC District Plan Volume 1, Section 22 – Natural Hazards, Rule 22.5.6.

4.5 WATER EFFICIENCY MEASURES AND AT-SOURCE STORMWATER REQUIREMENTS

4.5.1.1 District Plan Requirements

Water quality of the Waikato River has declined over time. Although point-source pollutants have reduced since the 1970s, non-point sources now comprise the majority of nutrient and sediment inputs into the Waikato River and its tributaries. The provision of a water efficiency measure to address stormwater is considered important as an enhancement to all discharges from urban development.

All lots within the development will need to comply with the Operative District Plan rule 25.13.4.5(a):

> *Water Efficiency Measures – "In addition to Low Flow Fixtures, at least one water sensitive technique for each water type shall be incorporated, connected to, achieved or maintained as part of any new development as identified below:*
>
> *Water Sensitive Techniques:*

- *Other equivalent feature i.e. Soakage;*
- *Permeable surfaces protected to achieve at least 20% above the minimum standard of the zone;*
- *Rainwater tank for non-potable reuse system;*
- *Detention of stormwater to 80% of pre-development runoff by an appropriate means.*

4.5.1.2 Low Flow Fixtures

All new buildings containing a kitchen, laundry or bathroom must use Low Flow Fixtures for showers, tap equipment and toilets. Low Flow Fixtures with a minimum 3-star rating are an acceptable means of demonstrating compliance. Approved ICMPs or consent conditions arising out of a WIA may require a higher star rating.

Otherwise, Low Flow Fixtures means the following:

- Showers using not more than 9 litres of water per minute. Being the nominal flow rate measured in accordance with AS/NZS 3662: 2005 Performance of showers for bathing.
- Tap equipment using not more than 9 litres of water per minute. Being the nominal flow rate measured in accordance with AS/NZS 3718: 2005 Water supply - Tap ware (excludes outdoor tap equipment).
- Toilets using not more than 4 litres on average per flush:
- For single-flush cisterns – the discharge flush volume, determined in accordance with AS 1172.2 Water closet (WC) pans of 6/3 L capacity or proven equivalent – Cisterns.
- For dual-flush cisterns – the average flush of one full-flush discharge and four reduced-flush discharge volumes, with the full-flush discharge flush volume and reduced-flush discharge volumes determined in accordance with AS 1172.2 Water closet (WC) pans of 6/3L capacity or proven equivalent – Cisterns.

4.5.1.3 Proposed Means of Compliance

The Hamilton City Council District Plan (Section 25.13.2.3e) requires the management of stormwater runoff is undertaken based on the following hierarchy:

- Priority 1 – Retention for reuse.
- Priority 2 – Soakage (onsite retention).
- Priority 3 – Detention and gradual release to a water course.
- Priority 4 – Detention and gradual release to stormwater reticulation.

The above HCC hierarchy applies to the subject site given its location in the catchment and nature of the piped discharge to the Waikato River. Retention and reuse provides multiple benefits for both the stormwater system and water supply system (reduced demand). Measures that improve either peak flow attenuation or WQ treatment are encouraged.

Retention for reuse (Priority 1) can be provided in the form of rain harvesting tanks for capture of stormwater runoff from individual roof surfaces and re-use for non-potable uses within the development area i.e toilets/laundry, irrigation etc. Implementation of these measures presents benefits both in terms of reduced stormwater discharge volumes and subsequent loading on the downstream stormwater network and reduced water supply demand on the HCC network.

As noted, further geotechnical assessment of soakage capacities throughout the development area should be undertaken to identify areas of the site where discharge of stormwater to ground soakage (Priority 2) may be viable and can be accommodated as part of the detailed stormwater management design. Implementation of these measures presents benefits both in terms of reduced stormwater discharge volumes and provision of recharge to the natural groundwater system.

Provision of the proposed stormwater wetland and the recommended stormwater attenuation measures provides for the detention and gradual release of development stormwater to both the downstream reticulation network and Waikato River watercourse in accordance with the Priority 3 and 4 methods outlined above.

Detailed consideration of the need for the site specific measures (retention for re-use/soakage) has not been included as part of this ICMP on the basis of the centralised stormwater management methods recommended within the following sections. However, the specific need for these measures as part of the overall stormwater management strategy for the site can be considered at the time of detailed design and on the basis of the broader/finalised stormwater management strategy and development structure. In any case, compliance with these district plan water efficiency management requirements is considered to be achievable with the final, recommended measures to be confirmed as part of the detailed design process.

4.5.1.4 At-Source Stormwater Management Measures

At-Source Stormwater Management Measures include the use of multiple stormwater treatment/attenuation devices located throughout the development area (roading and reserve areas) such as raingarden/swale treatment devices and subsurface attenuation tanks to provide at-source management of the potential water quality/quantity effects.

In this instance, the use of at-source devices has been discounted based upon the anticipated need for multiple at-source devices, associated operation and maintenance requirements and an indicated preference from HCC City Waters staff to avoid the use of these types of measures in favour of a centralised stormwater management device such as the proposed wetland.

While the use of multiple at-source stormwater management devices are not recommended to provide the bulk stormwater treatment attenuation objectives for the site, the use of at-source water retention measures including the discharge of building roof runoff to ground soakage (subject to detailed site soakage assessment outcomes) or water re-use tanks can be maintained as viable at-source management options to achieve water efficiency design requirements in accordance with Hamilton City district plan requirements.

4.6 MAXIMUM PROBABLE DEVELOPMENT & UPSTREAM CATCHMENT CONSIDERATIONS

4.6.1.1 Maximum Probable Development

The Racecourse site is located at the very upper extent of the Vardon Rd stormwater sub-catchment area draining to the Waikato River. Urban development within this sub-catchment area commenced in the 1940's with the catchment now encompassing a number of inner-city suburbs including St Andrews, Te Rapa and Forest Lake. Accordingly, the catchment comprises established areas of both residential and industrial development with little to no remaining open space within the catchment which could present further development potential. Further analysis of the development potential of the external catchment areas upstream and downstream of the Re-development site is provided as follows.

Upstream Catchment

The upstream catchment area above the site comprises a 8.0ha area with the majority of this area (around 6ha) having an industrial zoning under the HCC District Plan and having an almost 100% impervious cover associated with existing industrial development activities. The only remaining area of green space within this upstream catchment area comprises a narrow section of open space/park land of around 2ha located within the racecourse site and extending between the race track and the existing industrial land to the west.

This area currently has a Major Facilities zoning under the HCC district plan and is not ear marked for any future development potential although cannot be discounted. In the event that a future plan change was sought to enable development of this land, these activities would be subject to the provisions of the HCC district plan including the requirement that development out-flows are controlled to pre-development levels through implementation of appropriate stormwater attenuation/retention measures within the development design thus mitigating any potential impacts upon the downstream stormwater network. Alternatively, there is a potential that the proposed wetland device within the Racecourse Re-development area could be designed to accommodate upstream developed catchment flows (existing and potential) subject to negotiation between the Waikato Racing Club and HCC.

Downstream Catchment

Below the site, the stormwater network extends through the established suburbs of Forest Lake, Te Rapa and St Andrews. Review of aerial maps of the catchment has been undertaken which has identified the catchment as being subject to extensive residential, commercial and industrial development activities with remaining open spaces limited to a handful of public reserve and school sites including:

- Minogue Park/Water World Public Reserve;

- Forest Lake Primary School Playing Fields;
- Vardon Park;
- Vardon School Playing Fields;
- St Peter Chanel School Playing Fields;
- Heath Park
- Hamilton North School Playing Fields;

Accordingly, these remaining areas of open space are not anticipated to be subject to additional development which could contribute additional stormwater flows to the existing catchment network.

As noted, the remainder of the downstream catchment has been fully developed in-line with the HCC district plan allowances and hence is considered to be representative of maximum probable development within this catchment and hence contribution of any additional significant catchment flows is not anticipated. Again, while further infill development upon some of the existing residential lots cannot be discounted, these activities will again be subject to the requirements of the HCC district plan including provision of appropriate stormwater attenuation/retention measures thus mitigating any potential impacts upon the downstream stormwater network.

Overall, the existing catchment is considered to be more or less representative of the maximum probable development scenario for this network. Where any potential for additional development is present including the 2ha open space within the Racecourse site and infill development potential within existing residential areas, these activities are considered to present a minor potential for additional catchment flows and will be subject to the specific design requirements of the HCC district plan including flow attenuation/retention.

4.6.1.2 Upstream Catchment Consideration

As noted, there is an existing developed catchment area located northward/upstream of the site comprising industrial/commercial land including various existing industrial development premises, the Fairview Motors car dealership along with the Sir Tristram Drive roading catchment area. This catchment has a total area of approximately 8ha with stormwater runoff collected within a reticulated stormwater network which enters the subject Re-development site at its northern boundary via the existing 1050mm stormwater main draining through this area.

The intention is that the stormwater management system for the proposed development area will be developed and function in isolation of this upstream catchment with up-catchment flows bypassing the site via the existing (or diverted and upsized) reticulation and with treated/attenuated flows from the Re-development area discharging into this line as it exits the site. Nonetheless, HCC development engineering staff have requested that consideration also be given to the potential to accommodate retro-fitted treatment of up-catchment flows within the proposed wetland devices including the ability of the site to accommodate an enlarged wetland on this basis.

Preliminary calculations have been undertaken incorporating this upstream catchment to determine the likely design requirements for an enlarged wetland in this respect with the assessment indicating a 57% increase in the wetland footprint from 4,500m^2 up to approximately 7,100m^2. Given the location of the existing wetland within the proposed Central Open Space/reserve areas outlined on the preliminary site layout concept, there is considered to be suitable opportunity to provide for an enlarged wetland catering for retro-fit treatment of the upstream catchment area within this location if so desired by HCC and agreed with the Waikato Racing Club. Refer DWG WE1733-03-10 showing indicative footprint of the proposed wetland showing increased area to cater for treatment of the upstream catchment.

WQ treatment flows would need to be diverted from the 1050mm SW line and connected into the constructed wetland. Design of the wetland would also need to consider the level of the reticulation with the base of the wetland set at or below RL30.49m (invert level of the 1050mm SW line at EX-SWO15008). Upgrade and re-alignment of the existing reticulation as recommended would provide opportunity to enable a direct discharge from the upgraded up-catchment stormwater conveyance system into the wetland device.

It is considered appropriate that the scope of the wetland design in relation to this upstream catchment comprises a matter to be determined directly between the Waikato Racing Club and HCC at the time of resource consenting and detailed engineering design. The detailed engineering design for the wetland based upon the actual development catchment determined at this time will need to include climate change allowance for the post development scenario.

4.7 FLOOD MITIGATION

4.7.1.1 Flood storage/conveyance

Potential network capacity effects of increased rates of runoff associated with the site development activities will be mitigated by the proposed attenuation function of the wetland device including attenuation of peak flows to predevelopment levels for the 2, 10 and 80% of the 100 year events.

As outlined in Section 3.4 above, existing flooding is shown across the low-lying areas of the Racecourse redevelopment area. The RFHM data indicates the existing 100 year flood scenario as comprising a generally confined corridor of flooding which extends south to north through the site and with floodwaters generally comprising shallow, low velocity ponding through this corridor.

Risk of flooding is exacerbated by the low level of service provided by the existing stormwater network.

The preliminary design response to this identified flood corridor comprises development of a preliminary development layout plan which maintains a clear flood corridor through the central part of the site with this area maintained as open/undeveloped space containing the Ken Browne Drive extension road carriageway, a reserve network of green open space, the stormwater management wetland drainage reserve area and the Racing Club central arena and horse float parking area.

Upstream and downstream ground levels at the entry and exit to the flood corridor must be maintained as part of the site development works to ensure that no impediment to these extreme flood flows occurs. In this respect, the preliminary layout includes an indicative open swale extending along the southern boundary to ensure that any laminar flood flows entering the site across the southern boundary are captured and conveyed to the central flood corridor for either storage/attenuation until site floodwaters recede via the drainage network or are conveyed northward towards the low lying ponding area bordered by Te Rapa Road and Mainstreet Place.

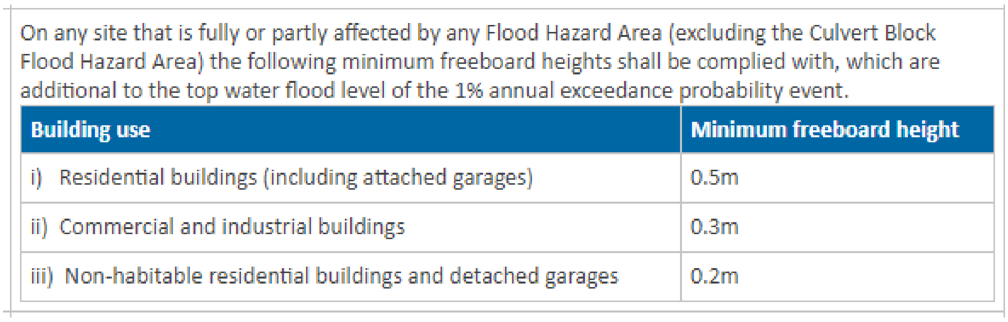
Further to ensure no adverse flooding effects are caused by the development, it will be important that pre-development flood storage volumes are maintained within the site as part of any development works such as recontouring (i.e. no loss of floodplain storage).

Detailed flood modelling will be required as part of future resource consent applications and detailed engineering design for any development or earthworks proposed within the designated low-high flood hazard areas.

4.7.1.2 Minimum Freeboard Requirements

Freeboard above the top water levels for the regional flooding as well as localised OLFP's shall be provided as per the District Plan, Volume 1, Section 22 – Natural Hazards, Rule 22.5.6. Required freeboard levels presented in Table 6 below.

Table 6: Minimum Freeboard Heights in Flood Hazard Area (Source HCC ODP)



On any site that is fully or partly affected by any Flood Hazard Area (excluding the Culvert Block Flood Hazard Area) the following minimum freeboard heights shall be complied with, which are additional to the top water flood level of the 1% annual exceedance probability event.

Building use	**Minimum freeboard height**
i) Residential buildings (including attached garages)	0.5m
ii) Commercial and industrial buildings	0.3m
iii) Non-habitable residential buildings and detached garages	0.2m

The adopted 'regional'/100 year flood level across the Re-Development Area has currently been established at RL33.40m and hence future buildings will need to be designed to accommodate the above freeboard levels above this established flood level or any future updated flood model information.

It is recommended that site/catchment specific detailed flood modelling is undertaken at resource consent stage to establish more accurate flood levels and the appropriate minimum freeboard requirements.

4.8 STORMWATER MANAGEMENT CONCLUSION

While the existing site is considered to comprise a brownfields development area located within an existing urban development catchment, the planned site Re-development will result in changes in stormwater quality and quantity characteristics including increased levels of residential stormwater contaminants and increased levels of stormwater runoff to the downstream reticulation network. The proposed stormwater management strategy comprising collection and conveyance of site runoff to a centralised wetland device designed for treatment and attenuation of post development flows, is considered to present a viable option to ensure that any potential adverse effects upon the downstream network and Waikato River receiving environment are avoided. In addition, the implementation of a specifically design stormwater management system providing water quality treatment and attenuation functions along with the ecological and aesthetic values associated with establishment of the proposed stormwater wetland, is considered to present an improvement from the existing site scenario from which stormwater runoff from the existing racecourse buildings, car parks, roads and yard areas occurs in and uncontrolled manner into the downstream receiving environment and with no existing areas of aquatic/wetland habitat existing within the site. Furthermore, the proposal has also been identified as presenting a potential opportunity for capture and diversion of the currently uncontrolled runoff from the upstream industrial development area for treatment and attenuation within the proposed wetland resulting in retrospective improvements in the existing stormwater quality/quantity effects from this part of the developed catchment. It also appears that once established, the proposed wetland management device will comprise the only specifically designed stormwater management device located within the broader catchment contributing to a higher level of stormwater management in comparison to the broader uncontrolled, existing catchment landuse activities.

The subject site is identified as including an existing central flood storage/conveyance corridor during the extreme 100 year flood event. The preliminary development layout has been configured to maintain the existing corridor as a central area of open space/drainage/road reserve avoiding impediment to flood flows or loss of flood storage and with the planned areas of residential development located outside of the identified flood corridor as shown on plan WE1733-03-400. Accordingly, the development is not considered to result in any adverse effects upon flood levels within the surrounding development areas and is considered to present a viable area for development without any risk of on-site flooding to future development properties.

Overall, viable options are available for stormwater management at the site to enable the planned Re-zoning without presenting a risk for adverse environmental, network or flooding effects.

5 WASTEWATER

Wastewater shall be treated and disposed of in a way that minimises effects on public health, the environment, and cultural values.

The size of infrastructure should be minimized by promoting sustainable water use and where possible, three waters networks are integrated within the catchment prior to discharge to the wider city networks. Future infrastructure upgrades shall be minimised by preventing, identifying and managing inefficiencies such as leakage, inflow and infiltration and unauthorised use.

5.1 Existing utilities and services

The Racecourse site is currently well serviced by wastewater infrastructure. A 600mm/675mm wastewater interceptor runs through the centre of the site, draining southeast to northwest. The 675mm interceptor discharges to a 750mm interceptor via a manhole at the Sunshine Ave/ Sheffield Street intersection approximately 550m northwest of the site. Two existing manholes ((WWO15001 and (WWO15002) are located within the site with depth of approximately 3.1m and 3.0m respectively.

A 150mm service main runs along the north-eastern boundary of the site, draining northwest to southeast. The service main receives flows from the neighbouring properties to the north east and ultimately discharges to the 600mm interceptor main approximately 300m to the south east of the site.

Figure 10 below shows the existing wastewater reticulation layout in the vicinity of the site. Refer to the Topography and services plan in Appendix A for further details.

Figure 10: Existing Wastewater Reticulation (Source: HCC 3 waters GIS Viewer – May 2015)

5.2 Best Practicable Options (BPO) – Wastewater

There are no Best Practicable Options for this catchment that are not standardised city-wide measures as described in RITS and DP.

Wastewater BPO 1 – General Requirements

a) Acceptable means of compliance for the provision, design and construction of wastewater infrastructure is contained within the RITS.

b) Low flow fixtures shall be incorporated in accordance with the PDP requirements.

5.3 Development Loading

Wastewater loadings were calculated in accordance with the RITS and resulting design flows used in the subsequent model assessment.

The proposed Racecourse development includes an area of approximately 6.87Ha, with current development plans indicating approximately198 apartments/dwellings equating to a design population of 535 people.

The development has an estimated peak DWF of 4.26 L/s and an estimated peak WWF of 5.57 L/s.

As the current Thoroughbred Business Park Area is zoned for light industry in the DP, it is expected any additional wastewater demands on the network will only be due to heavy/wet industry developments within the zone. Expected wastewater production from such industries cannot be anticipated at this time and it is recommended each future development within the zone determine its likely wastewater production and anticipated effect on the network.

5.4 Wastewater Capacity Assessment

In partnership with the HCC 3 Water team, AECOM were engaged to undertake wastewater capacity assessment for the proposed Racecourse Re-development.

The objective of this assessment was to determine if the network is likely to have sufficient spare capacity to accommodate the proposed increased discharge. The assessment was undertaken for the 2061 horizon using the existing HCC Wastewater Model.

A summary of the Te Rapa Racecourse development data used in the model is presented below;

- The development has an estimated population density of 78 people per hectare. This is equivalent to 535 people.
- The current population projection for this area in the 2061 horizon is 34 people. The current projection is based on employee numbers within this development block provided in the GIS layer named HCCNonResidentialEmployeesMay2017.
- Development discharge into the existing manhole WWO15001 into the 675mm interceptor.

The following performance measures analysed for both Dry Weather Flow (DWF) and a 10 year ARI overflow event.

- Pipe utilisation i.e. water level within the pipe
- Pipe spare capacity

5.4.1.1 Model results

The key model findings are as follows:

- During dry weather the pipeline is between 41% and 51% full, with an estimated average spare capacity of 188 L/s for the 675mm diameter network, and 275 L/s for the 750mm diameter network.
- During wet weather the pipeline is predicted to be between 65% and 95% full, with an estimated average spare capacity of 51 L/s for the 675mm diameter network, and 93 L/s for the 750mm diameter network.
- No manhole spills are predicted during the 10 year ARI event.

The outcomes of the modelling show the additional demand on the wastewater network from the proposed residential development is not predicted to have adverse effects on the HCC wastewater network.

The complete wastewater capacity assessment report can be found in Appendix F.

5.5 Internal Network Design Recommendations

Design and construction of the wastewater network shall be undertaken in accordance with Section 5 of the RITS.

The network has been designed with reference to the following standards and references:

- Waikato Regional Infrastructure Technical Specification (RITS)
- NZS 4404:2010 Land Development and Subdivision Infrastructure.
- AS/NZS 2566.1:1998 Buried Flexible Pipelines.
- AS/NZS 1260:2009 PVC-U Pipes and Fittings for Drain, Waste and Vent Application.
- New Zealand Building Code, Clause G13, Foul Water – Second Edition.

The existing 675mm interceptor line is at a sufficient depth within the site to enable a gravity network throughout the proposed Racecourse development with pipe inverts at approximately 3.0m deep.

Geotechnical ground investigations indicate the ground water level in the proposed area is relatively high (approx. 2.0m from the surface). This is expected to increase the risk of long term water ingress, construction difficulty and excavation difficulties for future maintenance. In addition to this, the proposed road network consists of relatively narrow local roads (i.e. 18m wide road reserves) which will also contribute to the difficulty of future access and dewatering when excavation for maintenance is required. Accessing deep pipelines in narrow roads is also likely to have a significant impact on local residents and adjacent infrastructure due to the limited space available within the local roads. To mitigate these risks, detailed design of the wastewater network will need to consider the following;

- Provide an optimised wastewater system to minimise overall costs (capital, operational and maintenance costs) by developing an efficient network with minimised pipe lengths.
- Mitigate wastewater system risks (design, construction, health and safety) where possible by minimising the overall depth of pipelines.
- Coordinate the delivery of the wastewater system with the overall development by aligning the size and extent of sub-catchment networks with development staging requirements.

The proposed development layout shows building areas located over the existing wastewater reticulation running through the Racecourse Redevelopment site. Consideration should be given to diverting the wastewater pipes around the proposed buildings to avoid build-overs where possible. Alternatively, the development layout could be altered at detailed design time so that the proposed roads or open space areas are located over the existing wastewater reticulation.

5.6 Wastewater Management Conclusion

Overall, there is existing wastewater reticulation extending through the proposed Re-development area. Anticipated wastewater flows from the planned development have been estimated along with available capacity with the downstream reticulation network. This assessment has determined available capacity within the existing wastewater network to accommodate flows from the planned development activities.

6 WATER SUPPLY

Water supply infrastructure shall be designed and constructed to meet consumption, hygiene, water sensitive design and firefighting requirements.

It is understood that HCC has the following initiatives planned to ensure that water demand is met across the city:

- City wide reticulation upgrades to support infill and intensification;
- Water demand and loss management programme to effectively manage water in the network and reduce loss;
- Continuation of the water model to forecast water demand out to 2061 and beyond;
- Enforcement of Water bylaw which requires water conservation in accordance with trigger levels;
- Education;
- Reduce water demand through universal metering or meet increased growth demand through the construction of additional treatment capacity;
- Continue to work with Waipa and Waikato District Councils to provide a Sub-Regional solution to water as per the Sub-Regional 3 Waters Strategy;
- Implementation of Public Health Risk Management Plan (Water Safety Plan) and Provision in the Proposed District Plan.

The following section provides details of the selected Water BPO measures and how they will achieve the objectives for the proposed Te Rapa Racecourse re-development.

6.1 Existing utilities and services

The proposed Racecourse Development Area is serviced with existing water infrastructure as follows.

An existing 200mm diameter water main is located inside the northern boundary of the site, terminating near the western end of Sir Tristram Avenue. There is an existing hydrant on the 200mm main, within the subject site

A 150mm water main is located on the north-eastern side of Ken Brown Drive and terminates at the south-eastern boundary of the site. A hydrant is located on ken Brown Drive approximately 10m from the site boundary.

A 250mm trunk main and 100mm service main are located on the south western side of Te Rapa Road.

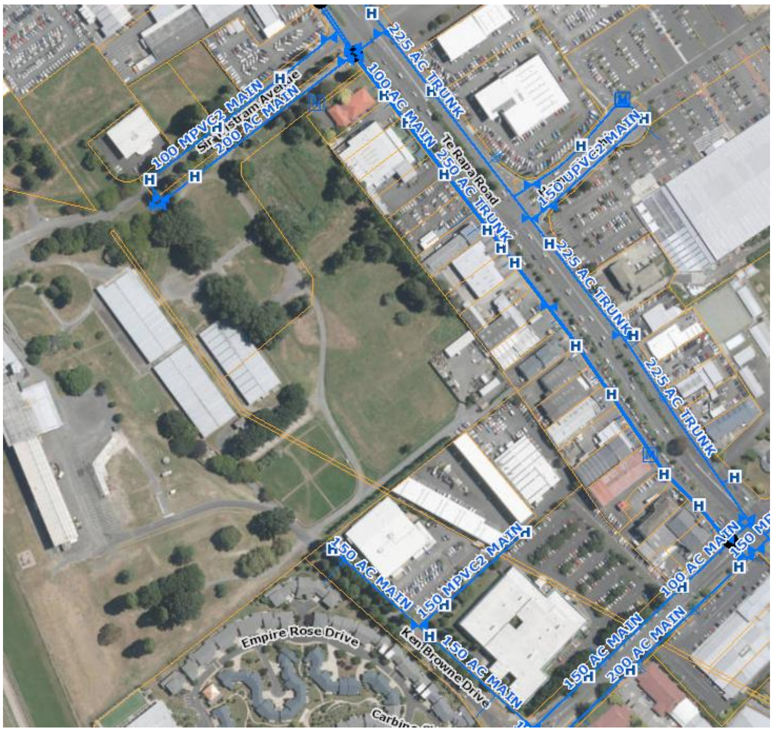


Figure 11: Existing water service infrastructure in the vicinity of the site (Source HCC 3 Waters GIS Viewer)

6.2 Best Practicable Options (BPO)

Best practicable options are standardised citywide measures as described in RITS and DP. Water supply infrastructure shall be designed and constructed to meet consumption, hygiene, water sensitive design and firefighting requirements. Undeveloped areas of the catchment will be serviced by the existing water system. New distribution networks shall be compatible with the existing system in accordance with the RITS.

A list of suitable BPOs for water supply and conservation for the catchment are presented below. The BPOs listed below provide for specific requirements, for items not discussed in this section, refer to the design requirements provided within the RITS.

Water BPO 1 – General Requirements

a) Acceptable means of compliance for the provision, design and construction of water infrastructure is contained within the RITS.

Water BPO 2 – Water Use Reduction

a) Low flow fixtures shall be incorporated into all new buildings in accordance with the PDP

b) If stormwater reuse tanks are installed, the permanent storage shall be used for toilet flushing and laundry, by plumbing the tank into the house. The tank may be plumbed into the mains supplied potable water system via an approved backflow prevention device. See HCC Three Waters Management Practice Note available online[2]

6.3 System Capacity

In partnership with the HCC 3 Waters team, Mott McDonald were engaged to undertake verification modelling of the proposed Racecourse development on the HCC water supply network in the planned Pukete Supply Zone.

The assessment compared the demand for the proposed development with existing and future model demands to confirm whether the proposed development was considered as part of the current and/or future conditions. This preliminary assessment was undertaken to determine whether additional modelling will be required.

The modelling inputs are summarised below;

- 198 dwellings
- 535 people (2.7 persons per dwelling)
- Per capita demand of 260L/day with a Peaking factor of 5
- Total instantaneous peak flow of 8L/s

The existing model showed the instantaneous peak demand in the area was 3.3L/s for the existing peak day scenario and 4.9L/s for the 2061 peak day scenario. Both considerably lower than the proposed development peak demand.

The existing and predicted system performance issues were verified including pressures prior to, and after the Pukete Zone Closure. Results show that before the Pukete Zone closure the increased demand will result in pressures between 20-30m. After the Zone closure pressures are expected to be above 30m (Refer ‘Preliminary Verification’ memo dated 3 October 2017 in Appendix G for full details). The report noted that firefighting capacity is very good in the area (up to FW5 along Te Rapa Rd, on the 225mm pipeline).

Further modelling was subsequently undertaken to assess the impact of the additional development demand prior to the Pukete Zone closure. It was not however considered necessary to verify the system performance post Pukete Zone closure, considering the satisfactory pressure and the limited head losses predicted in the area. The final model report can also be found in Appendix G, titled “Waikato Racing Club- Water Impact Assessment” dated 20 October 2017.

The latest HCC Water Supply model was used to determine the effects of the additional demand from the development on the network. A summary of the model run is summarised below;

- Existing and Peak demands from the proposed development as per the preliminary report
- Proposed development connection via existing 200mm main on Sir Tristram Ave and 150mm main on Ken Browne Drive

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[2] http://www.hamilton.govt.nz/our-council/council-publications/manuals/Pages/Three-Waters-Management-Practice-Notes.aspx

- 2021 Peak Day Scenario investigated with Pukete Zone open, Orange Zone extended, Maeroa, Whitiora and Rototuna Zones closed.

Results from the model show that there is sufficient capacity within the existing network to provide sufficient level of service to the proposed development, including residential firefighting supply.

The analysis of the remaining network including the demands of the proposed development show that the proposed development will have a noticeable impact on the remaining network. Pressures measured at 3 points across the network result in a maximum 1.3m pressure drop, however pressures are generally expected to remain above 20m except along Vercoe Road with pressure of 15.7m modelled. This is however an existing level of service issue related to the operation of the Pukete Reservoir. The report concludes that to maintain pressures above 20m the Pukete reservoir pump station is required to operate during peak demand periods.

Once the Pukete Zone is closed pressure will remain above 30m throughout the network.

6.4 Internal Network Design Recommendations

Water supply is proposed throughout the Racecourse Re-Development Area to provide the appropriate LOS in accordance with the HCC requirements.

Fire Hydrants will be required at distances of no more than 135m from any building within the development area in accordance with the RITS Section 6.

7 CONSULTATION

7.1 HCC Consultation

HCC has been directly consulted as part of the ICMP development process including submittal of an initial draft version of the document for review in 2017. The current ICMP has been developed following receipt of peer review comments/questions and additional meetings with HCC.

7.2 Other Stakeholders

Considering the brownfields nature of the Racecourse site, with all Three Waters management occurring via connection to existing services, there are no other stakeholder parties that are considered to warrant direct consultation as part of the ICMP development process.

Consideration should be given to consultation with the Waikato Regional Council and Tangata Whenua representatives at the time of detailed site design/consenting.

8 INFORMATION GAPS

In terms of this version of this sub-catchment ICMP, further information is not currently deemed to be required on the basis that the key purpose is to outline viable Three Waters management options which can realistically be implemented at the site to enable the intended land use while avoiding any potential adverse effects upon surrounding land, the existing HCC 3 Waters network infrastructure and the downstream receiving environment. New information will however be assessed and incorporated where relevant and will assist in the detailed planning of the proposed developments. Key information gaps are identified throughout this report and are summarised within the recommendations outlined below.

It is considered appropriate that this information can be incorporated at future resource consenting process and associated detailed engineering design phases.

9 RECOMMENDATIONS

The following key recommendations are identified through this ICMP which should be accommodated as part of the future 3 Waters detailed design for the proposed development to ensure that adverse environmental or network effects are avoided, remedied or mitigated.

Stormwater

- Detailed geotechnical assessment of the site should include a detailed assessment of site soakage capacity to assist in identification of areas where discharge to ground soakage may be able to be incorporated as part of the site stormwater management strategy contributing to reduced post development discharge loading on the downstream HCC stormwater network and to achieve the HCC district plan water efficiency design requirements;
- Detailed flood modelling of the post development flood scenario to ensure that pre-development flood storage volumes are maintained and accurate flood floor level freeboards are established to inform future building design;
- Upgrade and realignment of the existing stormwater network within the site to achieve a 10 year ARI design level of service in line with the RITS Standards and to provide a primary reticulation network maintained within the public road corridor thus avoiding potential for build-over conflicts and providing an accessible stormwater system for on-going maintenance;
- Secondary overland flow paths for flood flows up to the 100yr ARI rainfall event contained within the public road corridor and reserve areas;
- Provision of a centralised stormwater wetland device designed for provision of the stormwater management objectives in accordance with the RITS standfards for the development catchment area:
  - Water quality treatment;
  - Peak flow attenuation to pre-development levels for the 2 and 10 year ARI events and to 80% of the pre-development 100 year ARI event flow rates.
- Early engagement with HCC to determine and negotiate the need for the proposed stormwater management wetland to provide retro-fit treatment and attenuation for up-catchment stormwater flows;
- Engagement with WRC and Tangata Whenua to confirm their support to the proposed stormwater management strategy and to determine any potential resource consent requirements for the site development activities under the Waikato Regional Plan.

Wastewater

- Upgrade and realignment of the existing wastewater network within the site to provide a primary reticulation network maintained within the public road corridor thus avoiding potential for build-over conflicts and providing an accessible wastewater system for on-going maintenance;

Water Supply

- Establishment of a water supply network in accordance with the RITS and all HCC engineering design requirements.

APPENDIX A – CONCEPT DEVELOPMENT PLAN

LEGEND

- 🟩 OPEN SPACE
- 🟧 RESIDENTIAL USE
- ▨ NOISE SENSITIVE AREA
- ⬛ TYPICAL STREET
- 🟪 MINOR STREET
- 🟨 REAR LANEWAY
- 🟥 EXISTING NEIGHBOURING VET CARPARK
- 🟦 PROPOSED WETLAND
- 🟦 PROPOSED WETLAND EXTENSION
- ▬ FOOTPATH
- ▬ REAR LANEWAY MEDIUM BARRIER
- ← VIEWSHAFTS
- ⇠ POSSIBLE CONNECTION TO NEIGHBOURING DEVELOPMENT
- ⇦ ACCESS POINT

N

notes:
do not scale from drawings. all data to be verified on site prior to commencement of work.

original scale
0 10 20 30mm

rev	issue	date
B	Landscape Layout Update	18/07/2022

project consultant list:

structural	mechanical
#Structural Engineer	#Mechanical Engineer

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Auckland, New Zealand

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Te Rapa
Waikato Racing Club

WAIKATO RACING CLUB

PROPOSED SITE CONCEPT PLAN
MARCH 2022

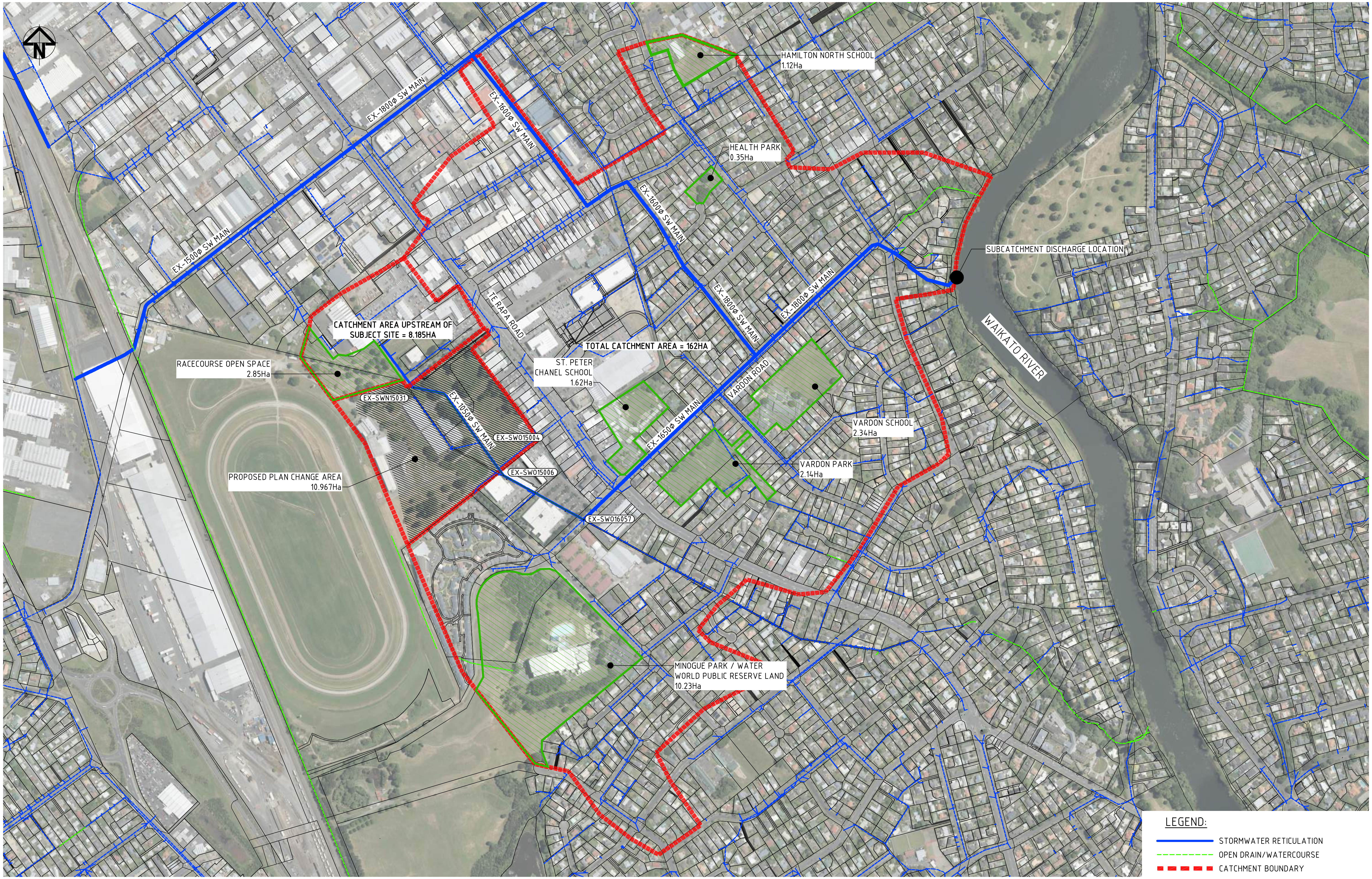
design ChowHill
drawn ChowHill
check
scale 1:2000@A3
approved
C:\Users\joshua.chia\Desktop\16704 Te Rapa Racecourse Plan v22 220713.pln

CONCEPT DESIGN

project no.	sheet	revision
16704	L80.09	2

date printed 18/07/2022

APPENDIX B – WIDER SUB-CATCHMENT PLAN



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No.	BY	DATE	DESCRIPTION	APPD
B	DM	06/04/2022	2022 ICMP UPDATE	
A	DM	04/08/2021	ORIGINAL ISSUE	

SCALE (AT ORIGINAL SHEET SIZE)

SHEET SIZE A1

SCALE 15 0 15 30 1:750

NOTES

DESIGNED	HV	CHECKED	
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APPROVED	HV	DATE	14/07/20

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PROJECT
TE RAPA RACECOURSE REDEVELOPMENT

SUB-CATCHMENT PLAN

Status
PRELIMINARY

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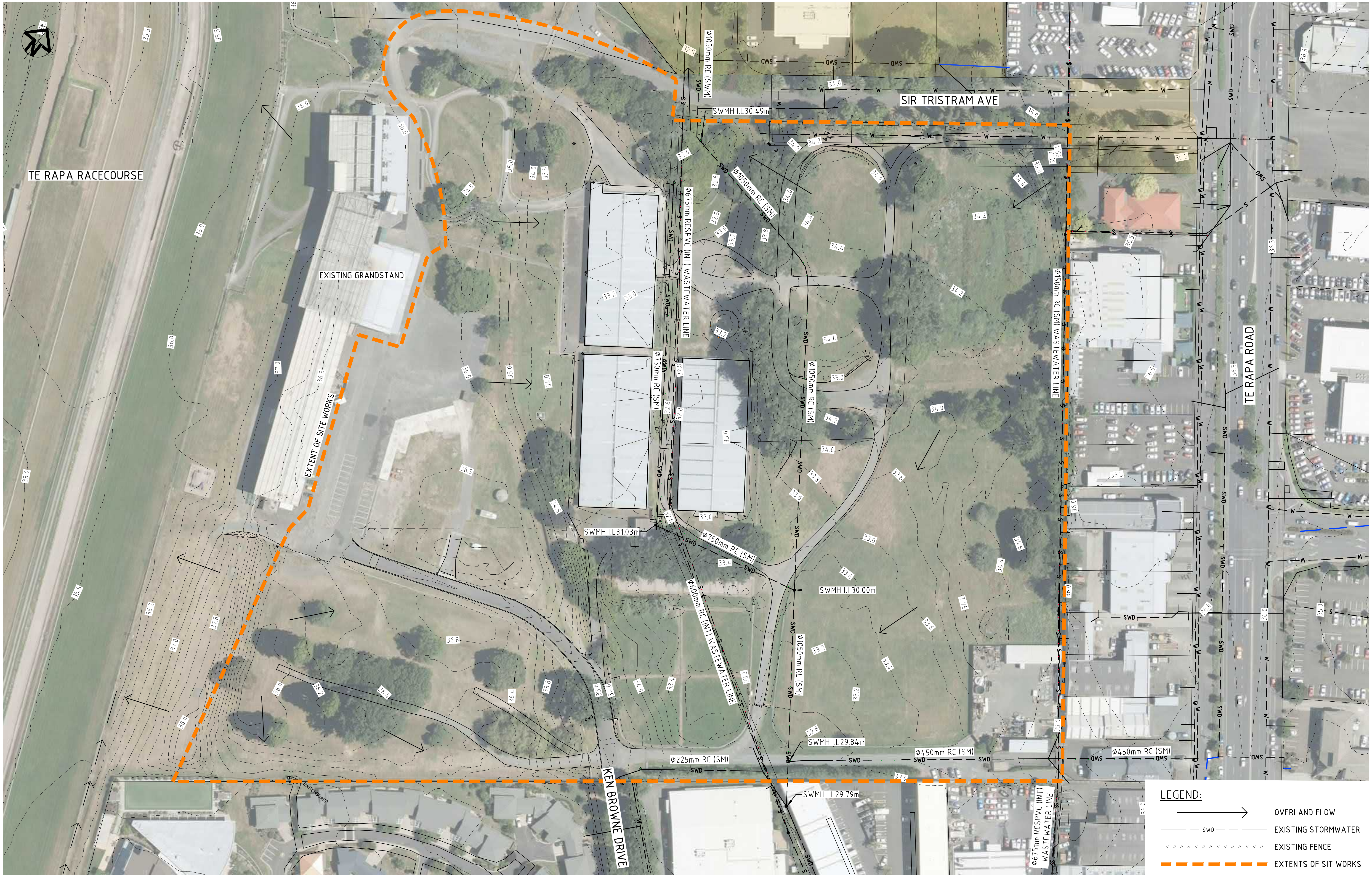
Rev.
B

APPENDIX C – 3 WATERS LAYOUT AND FLOODING PLANS



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A	DM	04/08/2021	ORIGINAL ISSUE	

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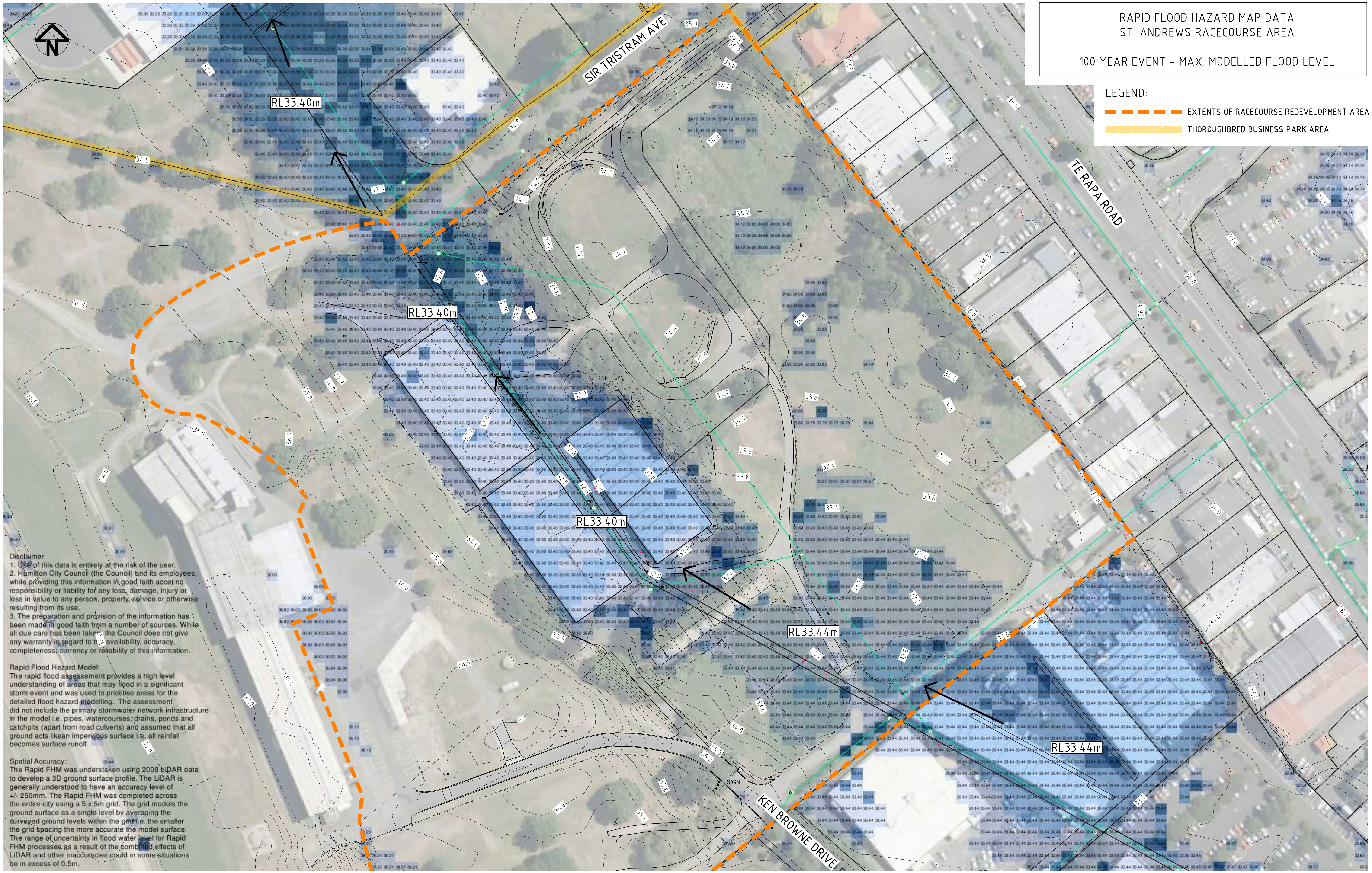
REDEVELOPMENT

PRE-DEVELOPMENT CATCHMENT PLAN

Status
PRELIMINARY

DRAWING NUMBER
WE1733-03-470

Rev.
A



RAPID FLOOD HAZARD MAP DATA
ST. ANDREWS RACECOURSE AREA

100 YEAR EVENT - MAX. MODELLED FLOOD LEVEL

LEGEND:

🟧 EXTENTS OF RACECOURSE REDEVELOPMENT AREA
🟨 THOROUGHBRED BUSINESS PARK AREA

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Rapid Flood Hazard Model:
The rapid flood assessement provides a high level understanding of areas that may flood in a significant storm event and was used to priotitise areas for the detailed flood hazard modelling. The assessment did not include the primary stormwater network infrastructure in the model i.e. pipes, watercourses, drains, ponds and catchpits (apart from road culverts) and assumed that all ground acts likean impervious surface i.e. all rainfall becomes surface runoff.

Spatial Accuracy:
The Rapid FHM was understaken using 2008 LiDAR data to develop a 3D ground surface profile. The LiDAR is generally understood to have an accuracy level of +/- 250mm. The Rapid FHM was completed across the entire city using a 5 x 5m grid. The grid models the ground surface as a single level by averaging the surveyed ground levels within the grid i.e. the smaller the grid spacing the more accurate the model surface. The range of uncertainty in flood water level for Rapid FHM processes as a result of the combined effects of LiDAR and other inaccuracies could in some situations be in excess of 0.5m.

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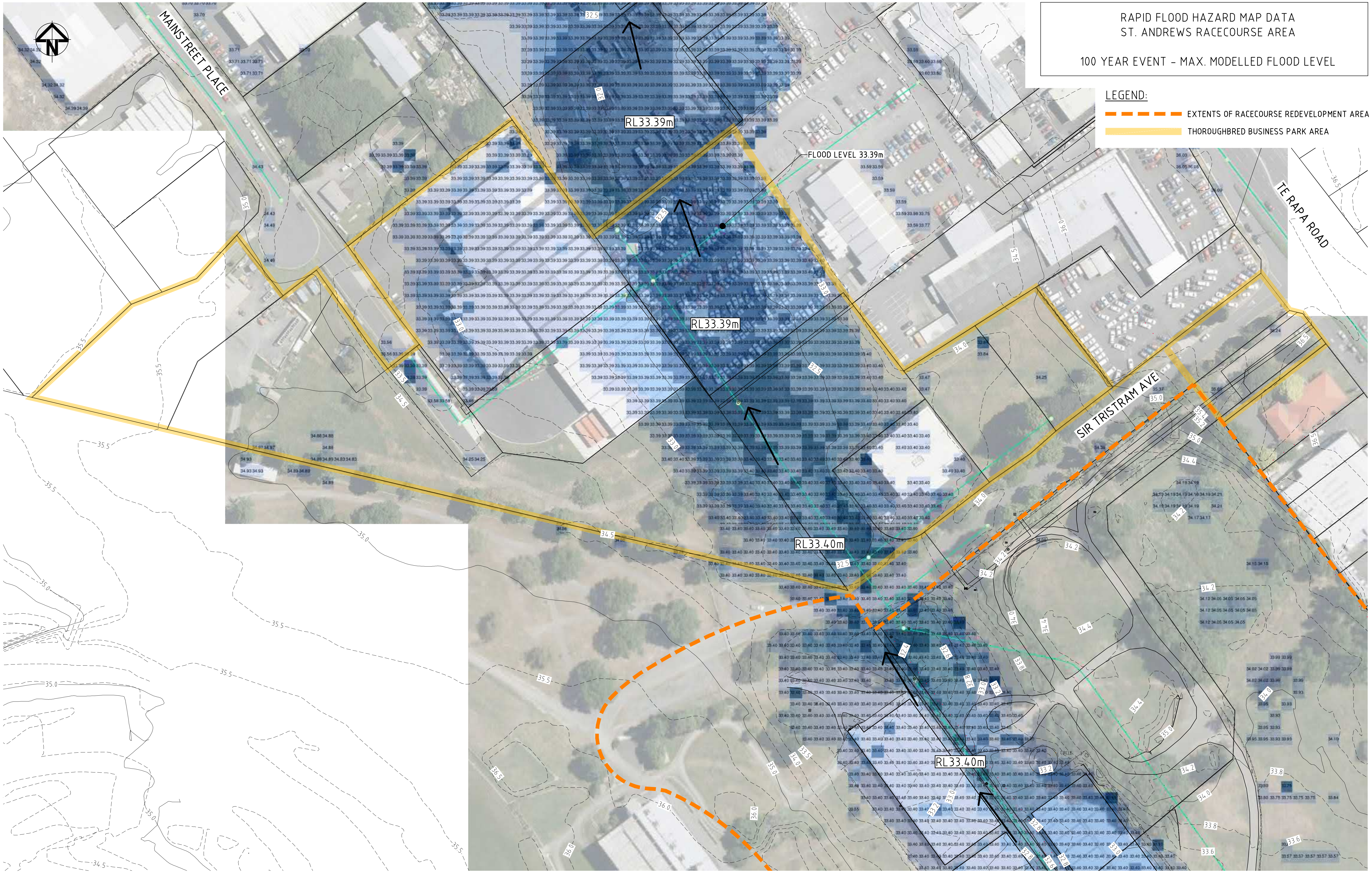
FLOOD LEVELS PLAN (RACECOURSE REDEVELOPMENT AREA)

Status
PRELIMINARY

DRAWING NUMBER
WE1733-03-03.1A

Rev.
A



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A	DM	04/08/2021	ORIGINAL ISSUE	
No.	BY	DATE	DESCRIPTION	APPD

SCALE (AT ORIGINAL SHEET SIZE)

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FLOOD LEVELS PLAN (RACECOURSE REDEVELOPMENT AREA)

Status
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WE1733-03-03.1B

Rev.
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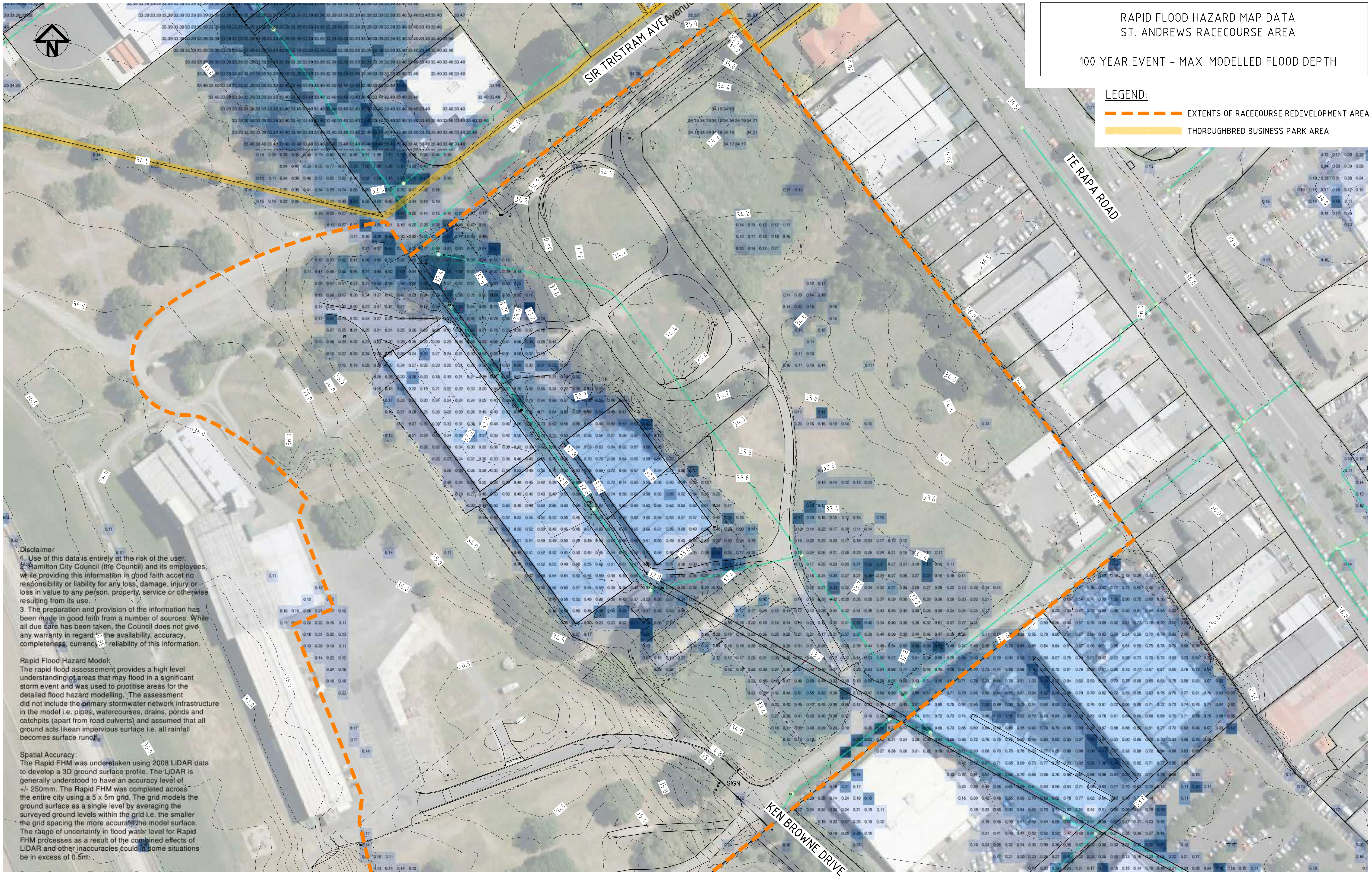
RAPID FLOOD HAZARD MAP DATA
ST. ANDREWS RACECOURSE AREA

100 YEAR EVENT – MAX. MODELLED FLOOD DEPTH



LEGEND:

▬ ▬	EXTENTS OF RACECOURSE REDEVELOPMENT AREA
▬	THOROUGHBRED BUSINESS PARK AREA

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FLOOD DEPTHS PLAN (RACECOURSE REDEVELOPMENT AREA)

Status
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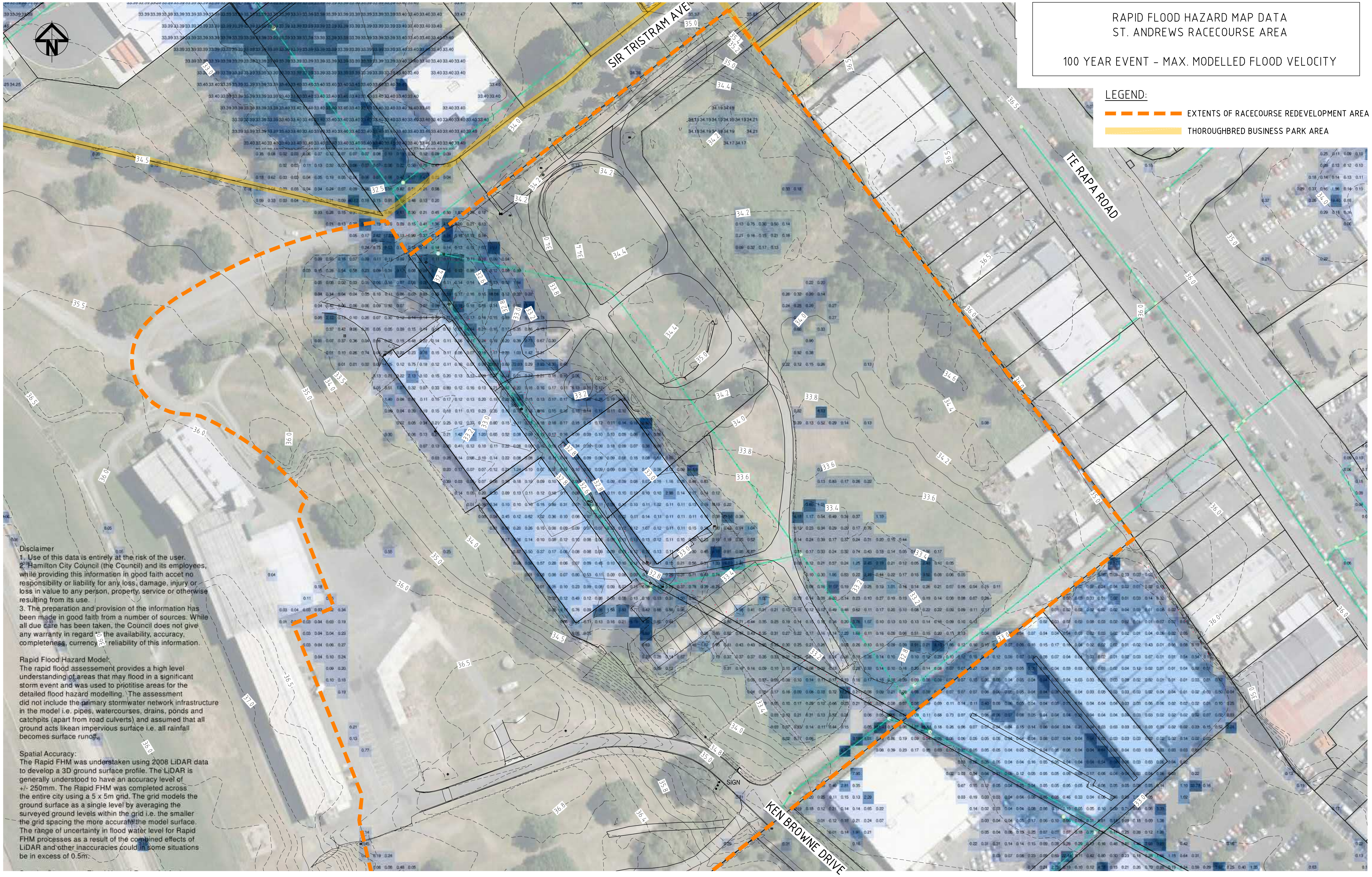
RAPID FLOOD HAZARD MAP DATA
ST. ANDREWS RACECOURSE AREA

100 YEAR EVENT - MAX. MODELLED FLOOD VELOCITY



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▬	THOROUGHBRED BUSINESS PARK AREA

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A	DM	04/08/2021	ORIGINAL ISSUE	

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FLOOD VELOCITY PLAN (RACECOURSE REDEVELOPMENT AREA)

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WE1733-03-03.3

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A

RAPID FLOOD HAZARD MAP DATA
ST. ANDREWS RACECOURSE AREA

100 YEAR EVENT - MAX. MODELLED FLOOD DEPTH

LEGEND:

- ▨ LOW FLOOD HAZARD
- ▨ MEDIUM FLOOD HAZARD
- ▨ HIGH FLOOD HAZARD

EXISTING FLOOD STORAGE VOLUME = 7,500m^3

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FLOOD DEPTHS PLAN (RACECOURSE REDEVELOPMENT AREA)
EXISTING/PRE-DEVELOPMENT

Status
PRELIMINARY

DRAWING NUMBER
WE1733-03-03.2

Rev.
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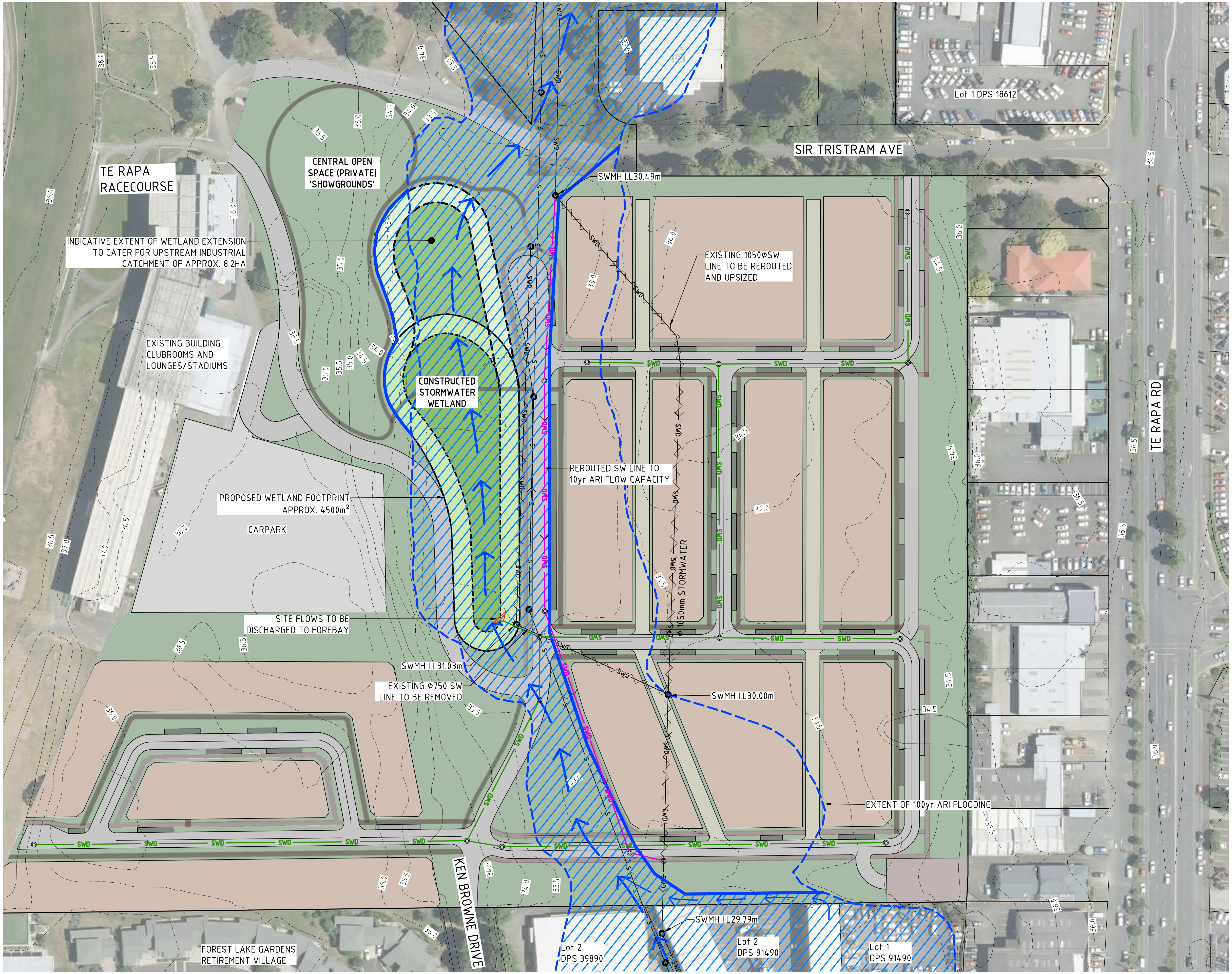

LEGEND:

🟩 LANDSCAPED / GREENSPACE AREAS

🟫 RESIDENTIAL USE

▨ INDICATIVE EXTENT OF POST DEVELOPMENT FLOODING REDIRECTED THROUGH SITE (CONCENTRATED ON GREEN CORRIDOR AND ROADWAY)

⬚ EXTENT OF 100yr ARI FLOODING

—SWD— PROPOSED SW LAYOUT - INTERNAL DRAINAGE DIVERTED TO WETLAND (INDICATIVE ONLY)

—SWD— EXTERNAL STORMWATER LINE TO BE REROUTED THROUGH SITE AND UPSIZED

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No.	BY	DATE	DESCRIPTION	APPD
C	DM	21/07/2022	SCHEME PLAN UPDATE	
B	DM	06/04/2022	2022 ICMP UPDATE	
A	DM	04/08/2021	ORIGINAL ISSUE	

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CONCEPT STORMWATER LAYOUT

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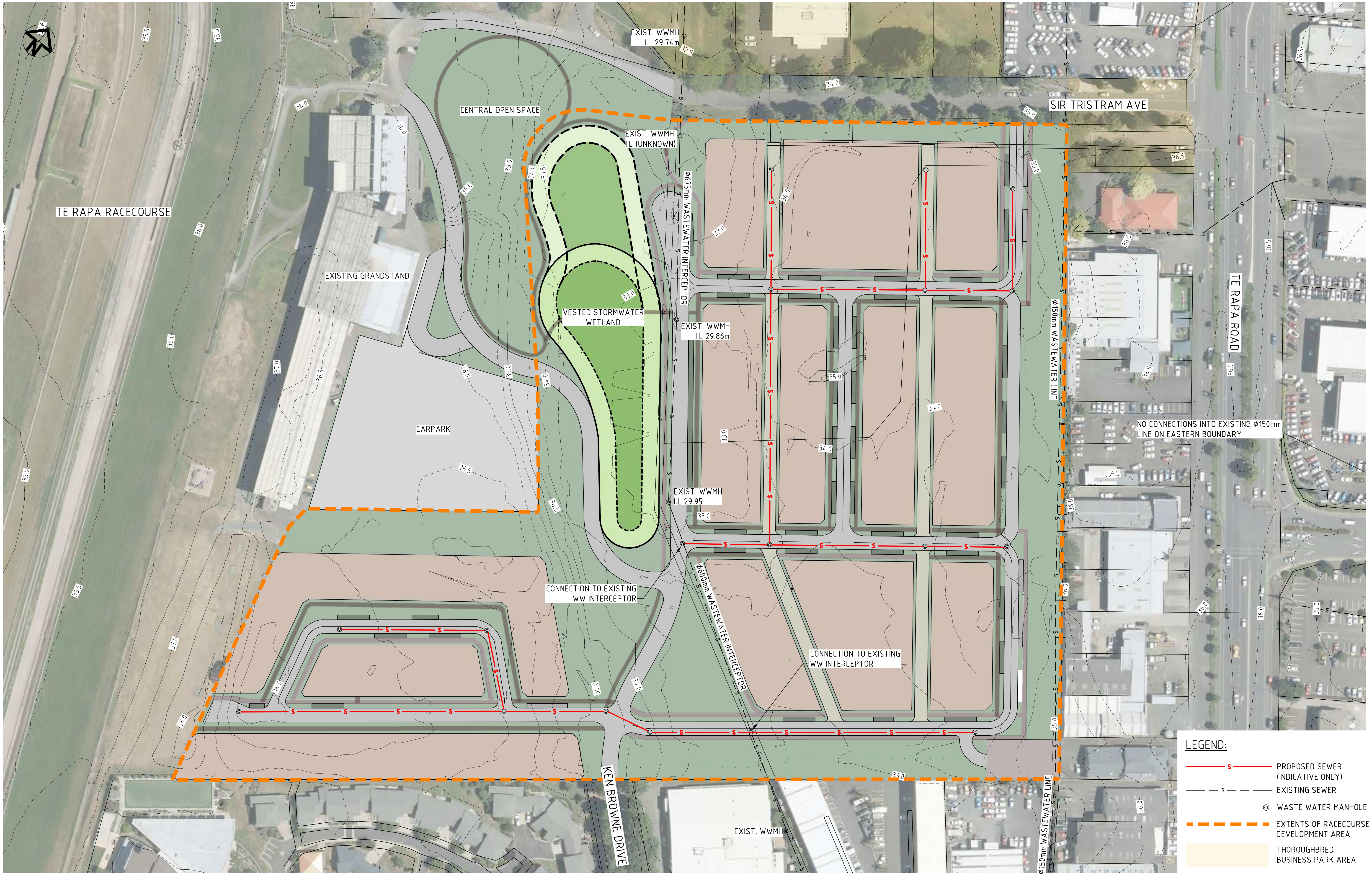

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C	DM	21/07/2022	SCHEME PLAN UPDATE	
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A	DM	04/08/2021	ORIGINAL ISSUE	

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CONCEPT WASTEWATER LAYOUT

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DRAWING NUMBER
WE1733-03-11

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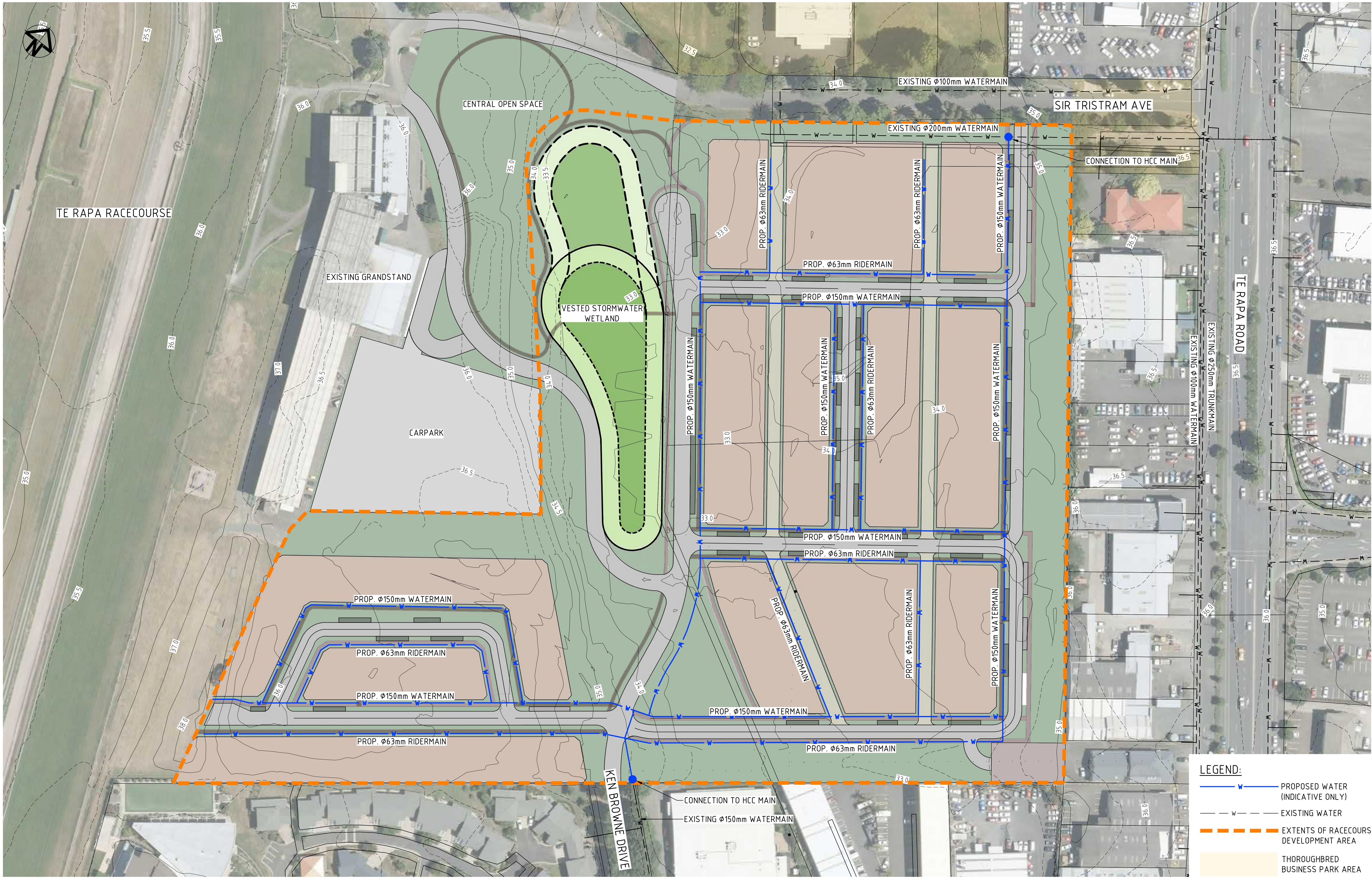
C



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A	DM	04/08/2021	ORIGINAL ISSUE	

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CONCEPT WATER LAYOUT

Status
PRELIMINARY

DRAWING NUMBER
WE1733-03-12

Rev.
C

APPENDIX D – HCC SERVICES MAPS

Water Services

- Closed Water Valve
- Open Water Valve
- Water Hydrant
- Water Meter
- Water Service Line Valve
- Water Main Cross-Junction
- Water Service Line/Connection
- Water Main
- Water Main Abandoned
- Water Storage Unit
- Water Preliminary Plans

Wastewater Services

- WW Pump Station
- WW Manhole
- WW Abandoned Manhole
- WW Node
- WW Service Line
- WW Aerial Main
- WW Rising Main
- WW Main
- WW Abandoned Main
- WW Preliminary Plans

Stormwater Services

- SW Manhole
- SW Abandoned Manhole
- SW Catchpit
- SW Node
- SW Inlet
- SW Outlet
- SW Culvert
- SW Catchpit Lead
- SW Soakage Trench
- SW Channel
- SW Service Line
- SW Main
- SW Abandoned Main
- SW Main Flow Direction
- Planted SW Device
- SW Preliminary Plans

Solid Waste Services

- Solid Waste Vent
- Solid Waste Bore
- Solid Waste Pump Station
- Solid Waste Chamber
- Solid Waste Barrier
- Solid Waste Main
- Solid Waste Building/Structure
- Solid Waste Landfill
- Solid Waste Pond

MAP LEGEND

Hamilton City Council
Te kaunihera o Kirikiriroa

www.hamilton.govt.nz/citywatersviewer

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Print Date 28/10/2017 *Scale* 1 : 1631

N

Water Services

- 🔺 Closed Water Valve
- 🔹 Open Water Valve
- H Water Hydrant
- M Water Meter
- ⊗ Water Service Line Valve
- ● Water Main Cross-Junction
- Water Service Line/Connection
- Water Main
- Water Main Abandoned
- ● Water Storage Unit
- Water Preliminary Plans

Wastewater Services

- P WW Pump Station
- ◎ WW Manhole
- ⊗ WW Abandoned Manhole
- ⊙ WW Node
- WW Service Line
- WW Aerial Main
- WW Rising Main
- WW Main
- WW Abandoned Main
- WW Preliminary Plans

Stormwater Services

- ◎ SW Manhole
- ⊗ SW Abandoned Manhole
- ▤ SW Catchpit
- ⊙ SW Node
- < SW Inlet
- SW Outlet
- SW Culvert
- SW Catchpit Lead
- SW Soakage Trench
- SW Channel
- SW Service Line
- SW Main
- SW Abandoned Main
- ▶ SW Main Flow Direction
- 🟩 Planted SW Device
- SW Preliminary Plans

Solid Waste Services

- ⊙ Solid Waste Vent
- ◎ Solid Waste Bore
- P Solid Waste Pump Station
- ■ Solid Waste Chamber
- Solid Waste Barrier
- Solid Waste Main
- ▭ Solid Waste Building/Structure
- ▭ Solid Waste Landfill
- ▭ Solid Waste Pond

MAP LEGEND

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Te kaunihera o Kirikiriroa

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Contact City Waters Unit, phone (07) 838 6699.

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Print Date 28/10/2017 *Scale* 1 : 1631

N

Water Services
- ▶◀ Closed Water Valve
- ▶◀ Open Water Valve
- H Water Hydrant
- M Water Meter
- ⊗ Water Service Line Valve
- ● Water Main Cross-Junction
- Water Service Line/Connection
- Water Main
- Water Main Abandoned
- ● Water Storage Unit
- Water Preliminary Plans

Wastewater Services
- P WW Pump Station
- ◎ WW Manhole
- ⊗ WW Abandoned Manhole
- ⊙ WW Node
- WW Service Line
- WW Aerial Main
- WW Rising Main
- WW Main
- WW Abandoned Main
- WW Preliminary Plans

Stormwater Services
- ◎ SW Manhole
- ⊗ SW Abandoned Manhole
- ▤ SW Catchpit
- ⊙ SW Node
- SW Inlet
- SW Outlet
- SW Culvert
- SW Catchpit Lead
- SW Soakage Trench
- SW Channel
- SW Service Line
- SW Main
- SW Abandoned Main
- ▶ SW Main Flow Direction
- 🟩 Planted SW Device
- SW Preliminary Plans

Solid Waste Services
- ⊙ Solid Waste Vent
- ◎ Solid Waste Bore
- P Solid Waste Pump Station
- ■ Solid Waste Chamber
- Solid Waste Barrier
- Solid Waste Main
- ▭ Solid Waste Building/Structure
- ▭ Solid Waste Landfill
- ▭ Solid Waste Pond

MAP LEGEND

Hamilton City Council
Te kaunihera o Kirikiriroa

www.hamilton.govt.nz/citywatersviewer

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APPENDIX E – CMW GEOTECHNICAL REPORT

21 JULY 2017

GEOTECHNICAL INVESTIGATION REPORT

TE RAPA RACECOURSE REDEVELOPMENT

KEN BROWNE DRIVE, HAMILTON

Te Rapa Waikato Racing Club
Ref. HAM2016_0109AB Rev. 0

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Tables

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Appendices

1 INTRODUCTION AND SCOPE

CMW Geosciences (CMW) was authorised by Te Rapa Waikato Racing Club to carry out a geotechnical assessment for a proposed residential building development located at the Te Rapa Racecourse, Hamilton.

This authorisation, together with the associated terms and conditions and scope of engagement are detailed in the CMW geotechnical services proposal referenced HAM2016_0109AA, Rev.1 dated 2 May 2017.

This geotechnical investigation report presents the results of a site specific geotechnical investigation to assess the suitability of the land for development. This report is suitable to support a land use plan change application to Council.

2 SITE DESCRIPTION

The site is located at the Te Rapa Racecourse in Hamilton and is relatively flat (RL 32.5m to RL 35m). A horse racing track is located along the western half of the site and to the east of the track is a grandstand and the Te Rapa Waikato Racing Club offices and events centre along with paved car parking. On the eastern portion of the site there are currently three horse stable buildings and large grassed paddocks used for horse grazing.

A soil stockpile up to approximately 2m high is located in the northern corner of the site and a stockpiled bund located adjacent to the racing track in the south-western corner of the site. The Waikato River is the nearest large watercourse and is located approximately 1.2km to the east of the proposed development area. The site is accessed via Ken Browne Drive and Sir Tristram Ave.

3 DEVELOPMENT PROPOSAL

The Chow Hill concept plans (Appendix B) provided by Bloxam Burnett and Olliver Ltd (BBO), indicate that the proposed development is located to the east of the current race track where the existing stables are located. It will consist of mixed-use residential buildings including two storey duplex and detached housing, three storey duplex and terraced housing and three storey apartment blocks together with associated access roads and laneways as depicted on Figure 1.

Based on the relatively level gradients across the site and adjacent land areas being of a similar elevation, it is envisaged that only minor cuts and fills will be carried out as part of the development.

Stormwater disposal is proposed via ground soakage plus there is potential for a large stormwater attenuation pond to be constructed within the centre of the existing racetrack.

4 FIELD INVESTIGATION

Geotechnical field investigations were carried out from 23 May to 30 May 2017 under the direction of CMW. The scope of fieldwork completed was as follows:

- A walkover survey of the site by a CMW Engineering Geologist to assess the general landform and site conditions;
- Eight (8) hand auger boreholes, denoted HA01 to HA08, were drilled using 50mm diameter augers to depths of up to 2.2 metres below existing ground level (mbgl) to allow observation and sampling of the shallow soil profile. In-situ shear vane strength (VSS) measurements were recorded using a hand-held shear vane during the advancement of the hand auger boreholes. Additionally, dynamic cone (Scala) penetrometer (DCP) tests were carried out where coarse-grained soils were present;

- Two (2) falling head permeability tests at hand auger locations HA03 and HA05, were carried out to provide an indication of soil permeability for stormwater retention/drainage. Two additional falling head permeability tests were intended at hand auger locations HA07 and HA08, however due to the high groundwater table these were not able to be undertaken; and
- Four (4) Cone Penetrometer Tests (CPT), denoted CPT01 to CPT04, were advanced to depths of up to 20 metres to provide an understanding of the deeper ground profile and for static settlement and liquefaction assessment purposes.

The approximate locations of the respective investigation locations referred to above are shown on the attached Site Investigation Plan (Figure 01). Investigation results are presented in Appendix B.

5 GROUND MODEL

5.1 Geological Setting

The geological map[1] of the area indicates that the site is underlain by fluvially reworked soil deposits of the Hinuera Formation. The Hinuera Formation infills the majority of the Waikato Basin and deposits generally comprise interbedded sands, silts and clays with interspersed peats.

The relict Te Rapa Channel[2] extends through the eastern part of the site which is a paleo river channel when the Waikato River was a braided system.

5.2 Soil Stratigraphy

A ground model was developed for the site based on the published geology and the results of hand augers and CPT results. Generally, the hand augers and CPTs indicate geological conditions consistent with the published geology described above.

In summary, the Hinuera Formation materials at the site consist of the following:

- The upper 0.8m consists of firm to very stiff silt and sandy silt with peak vane shear strengths of between 56kPa to 152kPa;
- Underlying the surface silt, primarily medium dense to dense sand/silty sand with interbeds of stiff to very stiff silt/sandy silt to depths of around 7.0 to 13.0mbgl were encountered at all hand auger and CPT locations. DCP results in the sand layers typically ranged from 2 to >10 blows/ per 100mm penetration. CPT qc values typically ranged from 4MPa to 10MPa in the sand layers and 0.5MPa to 1MPa in the silt lenses;
- Firm to very stiff clay and silty clay deposits were inferred from the CPT traces from depths of 16 to 20mbgl.

Weak organic material was encountered within the upper 1.5m to 2.0m of CPT02 and CPT04, which may be attributed to the backfilling of the potential Te Rapa Channel running through the site.

The approximate extent of the fill stockpiles described in Section 2 are inferred from surface contours only, is shown on Figure 1. In HA05 silty sand fill and buried topsoil was encountered down to 1m depth adjacent to the northern stock pile.

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[1] Edbrooke, S.W. (compiler) 2001: Geology of the Auckland area. Institute of Geological & Nuclear Sciences 1:250000 geological map 3. I sheet + 74 p. Lower Hutt. New Zealand. Institute of Geological & Nuclear Sciences Limited

[2] McCraw, J. 2011: The Wandering River, Landforms and geological history of the Hamilton Basin. Geoscience Society of New Zealand.

The generalised distribution of the site subsoil units described above are presented on the appended Geological Sections A-A' and B-B' (Figure No. 02).

5.3 Groundwater

Groundwater was encountered at the test locations within the proposed development area at depths of 1.5m to 2.8m below the current ground level (RL 32m to 33.2m).

Within the middle of the race track where the proposed stormwater attenuation pond is to be constructed, groundwater was encountered at the ground surface at HA07 and HA08 (RL 34.5m).

In the weeks and months prior to the site investigation there were several significant rainfall events which are likely to have contributed to an elevated groundwater table. It is expected that there would be groundwater level variations between seasons and relatively high levels following significant rainfall events.

5.4 Permeability Testing

Two falling head percolation tests were undertaken to assess the permeability of the near surface soils underlying the eastern part of site. Testing was undertaken within 100mm diameter hand auger boreholes, drilled to depths of between 1.5m to 2.2m and pre-soaked (filled with water) and allowed to drain over approximately 2.5 hours prior to testing. Permeability was calculated based on guidelines presented in CIRIA 113[3]. Reported results are presented in Appendix E.

Based on the falling head test results, seepage rates of between 1 x 10^{-5} m/sec to 5 x 10^{-6} m/sec were observed within the Hinuera sands and silty sands. Lower permeability values have been attributed to silt migration from the upper portion of the boreholes causing "caking" of the sides and base during testing. Seepage rates could be found to be an order of magnitude higher if more targeted testing is undertaken.

The falling head tests that were proposed to be undertaken within the centre of the racing track were abandoned due to the high groundwater level.

6 ENGINEERING EVALUATION AND RECOMMENDATIONS

6.1 Seismicity

For liquefaction assessment purposes, earthquake loads were calculated in accordance with the NZ Geotechnical Society publication "Earthquake geotechnical engineering practice, Module 1: Overview of the standards", (March 2016) based on the following:

$$a_{max} = C_{0,1000} \times \frac{R}{1.3} \times f \times g$$

Where a_{max} = Peak horizontal ground acceleration (PGA)
$C_{0,1000}$ = unweighted peak ground acceleration coefficient subject to subsoil class
R = return period factor given in NZS1170.5, Table 3.5
f = site response factor subject to subsoil class
g = acceleration from gravity

The Ultimate Limit State (ULS) PGA was calculated based on a 50-year design life in accordance with the New Zealand Building Code and importance level (IL) 2 structures providing an annual

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[3] Appendix 4, Control of Groundwater for Temporary Works (CIRIA Report No.113)

probability of exceedance of 1/500 in accordance with NZS1170.0. The ULS and Serviceability Limit State (SLS) PGA calculation is summarised in Table 1:

Table 1: Earthquake Load for Liquefaction Assessment

Importance Level	Subsoil Classification	Limit States	f	$C_{0,1000}$	R	Earthquake Magnitude	a_{max}
IL2	Class D	SLS	1.0	0.30	0.25	5.75	0.06g
IL2	Class D	ULS	1.0	0.30	1.0	5.75	0.23g

6.2 Liquefaction Analyses

6.2.1 Liquefaction Assessment

The liquefaction potential of the soils beneath the buildings was assessed in accordance with Section 5.2, NZGS Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards, (May 2016), based on the following:

- Ground water level and saturation of the in-situ soils;
- Assessment based on geological age;
- Assessment based on Plasticity Index;
- CPT based liquefaction assessment.

6.2.2 Saturation

Although low water content soils have been reported to liquefy, at least 80% to 85% saturation is generally deemed to be a necessary condition for soil liquefaction. The site investigation information shows that in the proposed building development area the water level varies between 1.5m and 2.8m depth from the existing ground level indicating that the necessary subsoil condition for soil liquefaction is satisfied.

6.2.3 Geological Age

Published geological records indicate that the Hinuera Formation soils beneath the site are of Holocene geological age (> 12,000 years old) and therefore have a moderate susceptibility to liquefaction based on that criterion. No ageing factor was therefore applied during the analyses.

6.2.4 Plasticity Index

A review of the plasticity index (PI) of the soil units was undertaken to assess liquefaction susceptibility in accordance with the recommendations in the NZGS Earthquake Geotechnical Engineering Practice, Module 3. The PI criteria set out in in that publication is summarised below;

PI < 7: Susceptible to Liquefaction

7 ≤ PI ≥ 12: Potentially Susceptible to Liquefaction

PI ≥ 12: Not Susceptible to Liquefaction

No specific laboratory testing of the site soils was carried out, therefore based on the field test results a conservative position was adopted where all materials are considered Non Plastic and therefore susceptible to liquefaction based on the plasticity Index criteria.

6.2.5 Numerical Analyses

A numerical liquefaction assessment of the soils beneath the site was carried out for compliance with Importance Level 2 (IL2) structures in accordance to AS/NZS 1170.0:2002.

The liquefaction susceptibility analysis was carried out using the computer software package CLiq v.1.7.6.49 (Geologismiki, 2006) based on the CPT data in general accordance with the NCEER (2008) method.

Under the SLS design scenario, the liquefaction analyses results show very low liquefaction susceptibility.

The ULS design scenario induces liquefaction within weaker soil layers beneath the water table. The results of our analyses show that liquefaction induced settlements are predicted to range in the order of 30mm to 50mm across the site. The majority of this settlement occurs within the upper 15m and within discrete layers that are up to 0.5m thick. A non-liquefiable crust of between 4m to 7m was encountered at all CPT test locations. The largest differential between two CPTs is 20mm over a distance of 135m.

The CPTs undertaken are considered indicative of site conditions, however further CPT testing and liquefaction assessment will be required at Building Consent stage to more accurately define differential settlements for building design purposes. The predicted settlements are based on free field vertical settlements, however settlements beneath buildings may be larger.

6.3 Earthworks

Based on the relatively gentle relief across the site it is expected that minor bulk cut and fill depths up to nominally 1m will be required during site development. It is anticipated that there will be cuts from the elevated areas and filling of the low-lying areas.

All earthworks should be carried out in accordance with the requirements of NZS4404:2010 (Land Development and Subdivision Infrastructure) and NZS4431:1989 (Code of Practice for Earth Fill for Residential Development), Hamilton City Council Development Manual and under the guidance of a Chartered Professional Geotechnical Engineer.

From initial investigation results the near surface materials likely to be earth worked consist of loose to medium dense sand and silty sand and firm to very stiff sandy silt. These materials may be used for cut to fill earthworks across the site with appropriate conditioning.

In HA07 and HA08 in the centre of the race track, silt with quick sensitivity was encountered in the upper soils. Due to its potential for significant strength reduction when disturbed, it is recommended that if a stormwater pond excavation is required, this material is not used for onsite filling. This material may be used as landscape fill in reserve areas or be removed from site.

In HA05 silty sand fill and buried topsoil was encountered down to 1m depth. It is not recommended to use the material around this area for fill due to the organics located within the topsoil. This test was undertaken next to the soil stockpile in the north of the site. Further investigation on the suitability of the stockpiled material located at the north and south of the site will need to be undertaken however it may be used as landscape fill in reserve areas or removed from site. Other areas of non-engineered fill due to the sites previous history may be present on site and were not identified during this investigation.

CPT02 and CPT04 encountered some weaker silty / organic materials in the upper 1.5m to 2.0m, that is considered unsuitable and should be removed from site or used as landscape fill onsite. The extent of these organic deposits should be confirmed by further hand auger borehole investigation.

Any localised areas of soft/loose material and all existing filling encountered below founding level should be over-excavated and replaced with suitably compacted granular filling (clean well graded sand or hardfill) or foundations extended/deepened to account for a reduced bearing capacity.

6.4 Slope Stability

The proposed development is located on flat to gently sloping (<5°) topography and therefore on this basis the site was assessed qualitatively to have an overall low risk of slope instability for the proposed development. No quantitative stability analysis was completed for the site. Depending on the final design for the development this may need to be undertaken at building consent stage for any localised cut or fill batters.

6.5 Foundation Bearing Capacity

It is anticipated that the proposed buildings will be beyond the scope of NZS3604 and will therefore be subject to specific design by a Structural Engineer.

The design of available foundation bearing pressures for isolated strip and pad footings at this site has been carried out using the Terzaghi bearing capacity equation. Subject to completing the earthworks and foundation preparation recommendations provided herein, shallow strip or pad footings founded within the Hinuera Formation sands and silts may be designed on the basis of the bearing capacities provided in Table 2.

Table 2: Summary of Shallow Footing Design Bearing Pressure

Embedment Depth (m)	Footing Width (m)	Footing Length (m)	Geotechnical Ultimate Bearing Capacity (kPa)
0.5	1.0 strip		240
	2.0	2.0	300
1.0	1.0 strip		300
	2.0	2.0	300

As required by Section B1/VM4 of the New Zealand Building Code Handbook. A strength reduction factor of 0.5 (static) or 0.8 (seismic) must be applied to the recommended geotechnical ultimate soil capacity in conjunction with its use in factored design load cases for static and earthquake overload conditions respectively.

Further field investigations once development plans are confirmed will need to be undertaken and are recommended at Building Consent stage.

6.6 Static Settlement

Foundation settlements were estimated based on allowable pressures derived using a Factor of Safety of 3.0 on the ultimate pressures presented in Table 2 above and the footing dimensions also presented in Table 2.

The settlement predictions were carried out using the Schmertmann method which approximately correlates CPT cone resistance (qc) to Young's Modulus (E'). For this project, the modulus was increased to 5 x q_c to recognise the slightly over consolidated nature of the Hinuera Formation soils present due to natural ageing processes that have occurred since their deposition.

Further, an upper limiting threshold of q_c = 10MPa was adopted to define a soil strength at which settlements are expected to only be relatively minor and may be essentially ignored.

Settlement results are provided in Appendix D and are summarised in Table 3 below.

Table 3: Static Settlement Assessment

Test Location	Foundation Embedment Depth (m)	Footing Width (m)	Footing Length (m)	Estimated Settlement (mm)
CPT01	0.5	1.0 (Strip)		<10
		2.0	2.0	10
	1.0	1.0 (Strip)		<10
		2.0	2.0	<10
CPT02	0.5	1.0 (Strip)		<20
		2.0	2.0	<25
	1.0	1.0 (Strip)		<25
		2.0	2.0	<25
CPT03	0.5	1.0 (Strip)		<20
		2.0	2.0	<25
	1.0	1.0 (Strip)		<20
		2.0	2.0	<20
CPT04	0.5	1.0 (Strip)		<40
		2.0	2.0	<50
	1.0	1.0 (Strip)		<50
		2.0	2.0	<50

Differential settlements between CPT01 and CPT04 are the largest at the site, however these are 250m apart so the angle of distortion across this distance is considered minor. The New Zealand Building Code states that differential settlements across a building platform can be up to 25mm over a 6m length. The largest predicted differential settlements above are 40mm over the 250m length. Further investigation and settlement analysis will need to be undertaken when building locations, layouts and loads are known at building consent stage.

6.7 Stormwater Disposal

It is anticipated that stormwater from the proposed development will discharge into a proposed stormwater attenuation pond to be constructed in the centre of the current racetrack as indicated on Figure 1. We understand from discussions with BBO that the pond is proposed to have a portion that stays wet so it can be used for irrigation during the summer months.

With the groundwater being located at the ground surface at that location during the investigation, the pond concept is not considered feasible because the groundwater level is above the current ground level across the majority of the proposed development area.

Conventional soakage trenches or soakholes are considered a practical solution for the disposal of stormwater where located within the proposed building development area due to the sands encountered. Groundwater was encountered between 1.5m to 2.8m below the current ground level throughout the building development area. A coefficient of permeability (k) of 1 x 10^{-5} m/sec to 5 x10^{-6}m/sec should be used for modelling unless further site specific testing is undertaken.

Detailed assessment of stormwater design volumes, stormwater pond design, soakage trench locations and specific design will be required at the engineering plan approval stage and prior to any building development.

7 SUITABILITY STATEMENT

In summary, it is our opinion that the proposed development as depicted on the appended ChowHill concept plans is geotechnically suitable subject to the recommendations contained herein. A copy of our Statement of Professional Opinion as to the Geotechnical Suitability of Land for Development is appended (Appendix A).

8 PLAN REVIEW AND FURTHER WORK

Given the plans provided to us are still in a concept design stage, we should be given the opportunity for further site investigation prior to earthworks and engineering plan approval stages as only a preliminary geotechnical site investigation and assessment has been completed to support the land use plan change.

Further work will be required at the Resource Consent / Engineering Plan approval stage, this includes and is not limited to the following:

- Site investigation including additional hand auger boreholes or test pits to assess the extent of and non-engineered fill and organic soils and allow installation of standpipe piezometers to measure groundwater level variability over the summer and winter season;
- CPT testing and further liquefaction and settlement assessment;
- Stormwater soakage design; and
- Stormwater attenuation pond design (by others).

9 LIMITATION

This report has been prepared for use by our client Te Rapa Waikato Racing Club, their consultants and Hamilton City Council. Liability for its use is limited to these parties and to the scope of work for which it was prepared as it may not contain sufficient information for other parties or for other purposes.

It should be noted that factual data for this report has been obtained from discrete locations using normal geotechnical investigation techniques. As such investigation methods by their nature only provide information about a relatively small volume of subsoils, there may be special conditions pertaining to this site which have not been disclosed by the investigation and which have not been considered in the report. If variations in the subsoils occur from those described or assumed to exist, then the matter should be referred back to CMW immediately.

10 CLOSURE

Should you require any further information or clarification regarding the information provided in this report, please do not hesitate to contact the undersigned.

For and on behalf of
CMW Geosciences (NZ) Ltd

Prepared by:	Reviewed by:
Jordan Craig	**Dave Morton**
Geotechnical Engineer	Principal Geotechnical Engineer, MIPENZ (Geotechnical), CPEng

FIGURES

1 - Site Investigation Plan

2- Geological Cross Sections

LEGEND:

- HA01 HAND AUGER (HA) LOCATION
- CPT01 CONE PENETROMETER TEST (CPT) LOCATION
- S FALLING HEAD PERMEABILITY (SOAKAGE) TEST LOCATION
- ▨ POTENTIAL NON-ENGINEERED FILL LOCATIONS

NOTES:

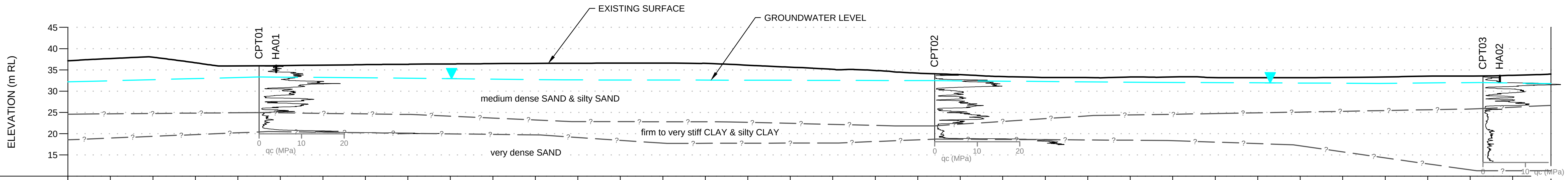
1. IMAGE FROM NEARMAP 21.12.16
2. CONTOUR PLAN SUPPLIED BY BLOXAM BURNETT & OLLIVER

0 25 50 75 100 125 m
1:2500

CMW Geosciences

CLIENT: TE RAPA WAIKATO RACING CLUB
PROJECT: TE RAPA RACECOURSE REDEVELOPMENT
HAMILTON, NZ
TITLE: SITE INVESTIGATION PLAN

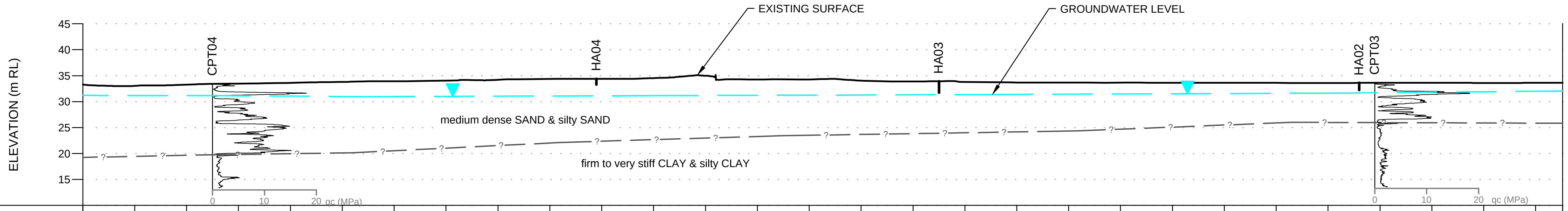
DRAWN:	DE	PROJECT:	HAM2016-0109
CHECKED:	JC	FIGURE:	01
REVISION:	0	SCALE:	1:2500
DATE:	14.06.17	SHEET:	A3 L



EXISTING GROUND (m RL)	37.67	37.99	36.67	35.93	35.99	36.12	36.24	36.32	36.40	36.47	36.54	36.56	36.59	36.60	36.53	36.10	35.65
CHAINAGE (m)	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170

EXISTING GROUND (m RL)	35.15	34.84	34.23	33.89	33.43	33.20	33.19	33.32	33.34	33.17	33.18	33.17	33.24	33.34	33.53	33.55	33.73	34.02
CHAINAGE (m)	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	349

SECTION A
SCALE: H 1:2500,V 1:2500.



EXISTING GROUND (m RL)	33.05	33.27	33.46	33.62	33.78	33.94	34.06	34.22	34.38	34.40	34.52	34.99	34.28	34.25
CHAINAGE (m)	10	20	30	40	50	60	70	80	90	100	110	120	130	140

EXISTING GROUND (m RL)	34.04	33.90	33.79	33.69	33.65	33.65	33.63	33.63	33.62	33.59	33.63	33.62	33.59	33.62	33.63
CHAINAGE (m)	150	160	170	180	190	200	210	220	230	240	250	260	270	280	285

SECTION B
SCALE: H 1:2500,V 1:2500.

0 10 20 30 40 50 m
1:1000

CMW Geosciences

CLIENT: TE RAPA WAIKATO RACING CLUB		DRAWN: DE	PROJECT: HAM2016-0109
PROJECT: TE RAPA RACECOURSE REDEVELOPMENT HAMILTON, NZ		CHECKED: JC	FIGURE: 02
		REVISION: 0	SCALE: 1:1000
TITLE: SECTIONS		DATE: 14.06.17	SHEET: A3 L

APPENDIX A: SITE INVESTIGATION RECORDS

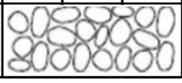
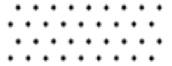
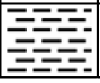
CMW Geosciences - SOIL (Field Logging Guide)

SEQUENCE OF TERMS

Fine: Soil type - Colour - Structure - Strength - Moisture - Bedding - Plasticity - Sensitivity - Origin/Geological Unit - Comments

Coarse: Soil Type - Colour - Structure - Grading - Strength/Relative Density - Moisture - Origin/Geological Unit - Comments

GRAIN SIZE CRITERIA

TYPE	COARSE								FINE		ORGANIC
			Gravel			Sand					Organic Soil
	Boulders	Cobbles	coarse	medium	fine	coarse	medium	fine	Silt	CLAY	
Size Range (mm)	200	60	20	6	2	0.6	0.2	0.06	0.002		
Graphic Symbol											

PROPORTIONAL TERMS DEFINITION (COARSE SOILS)

Fraction	Term	% of Soil Mass	Example
Major	(….) [UPPER CASE]	≥50 [major constituents]	GRAVEL
Subordinate	(….)y [lower case]	20 - 50	Sandy
Minor	with some…	12 - 20	with some sand
	with minor…	5 - 12	with minor sand
	with trace of (or slightly)	< 5	with trace of sand (slightly sandy)

SHADE and COLOUR TERMS

Term	Abbreviation
Light	lt
Dark	dk
pink	pk
red	rd
orange	or
yellow	yl
brown	br
green	grn
blue	blu
white	wh
grey	gr
black	bl

Major divisions			Soil symbol	Soil name
Coarse grained soils more than 65% >0.06mm	gravel >50% of coarse fraction > 2mm	clean gravel <5% smaller 0.075mm	GW	well graded gravel, fine to coarse gravel
			GP	poorly graded gravel
		gravel with >12% fines	GM	silty gravel
			GC	clayey gravel
	sand ≥50% of coarse fraction <2mm	clean sand	SW	well-graded sand, fine to coarse sand
			SP	poorly graded sand
		sand with >12% fines	SM	silty sand
			SC	clayey sand
Fine grained soils 35% or more <0.06mm	silt and clay liquid limit <50	inorganic	ML	silt
			CL	clay of low plasticity
		organic	OL	organic silt
	silt and clay liquid limit ≥50	inorganic	MH	silt of high plasticity
			CH	clay of high plasticity
		organic	OH	organic clay
Highly Organic Soils			Pt	peat

ORGANIC SOILS / DESCRIPTORS

Term	Description
Topsoil	Surficial organic soil layer that may contain living matter. However topsoil may occour at greater depth, having been buried by geological processes or man-made fill, and should be termed a buried topsoil.
Organic clay, silt or sand	Contains finely divided organic matter; may have distinctive smell; may stain; may oxidise rapidly. Desribe as for inorganic soils
Peat	Consists predominantly of plant remains. ***Firm:*** Fibres already compressed together ***Spongy:*** Very compressible and open structure ***Plastic:*** Can be moulded in hand and smears in fingers ***Fibrous:*** Plant remains recognisable and retain some strenght ***Amorphous:*** No recognisable plant remains
Rootlets	Fine, partly decomposed roots, normally found in the uper part of a soil profile or in a redeposited soil (e.g. colluvium of fill)
Carbonaceous	Discrete particles of hardened (carbonised) plant material.

DENSITY INDEX (RELATIVE DENSITY) TERMS

DESCRIPTIVE TERM	Density Index (RD)	SPT "N" value (blows / 300mm)	Dynamic Cone (blows / 100mm)	Abbreviation
Very Dense	> 85	> 50	> 17	VD
Dense	65- 85	30 - 50	7 - 17	D
Medium dense	35 - 65	10 - 30	3 - 7	MD
Loose	15 - 35	4 - 10	1 - 3	L
Very loose	< 15	< 4	0 - 2	VL

Note: No correlation is implied between Standard Penetration Test (SPT) and Dynamic Cone Test values. SPT "N" values are uncorrected. Dynamic Cone Penetrometer (Scala)

CONSISTENCY TERMS FOR COHESIVE SOILS

Descriptive Term	Undrained Shear Strength (kPa)	Diagnostic Features	Abbreviation
Very soft	< 12	Easily exudes between fingers when squeezed	VS
Soft	12 - 25	Easily indented by fingers	S
Firm	25 - 50	Indented by strong finger pressure and can be indented by thumb pressure	F
Stiff	50 - 100	Cannot be indented by thumb pressure	St
Very Stiff	100 - 200	Can be indented by thumb nail	Vst
Hard	200 - 500	Difficult to indent by thumb nail	H

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CMW Geosciences - SOIL (Field Logging Guide)

SEQUENCE OF TERMS

Fine: Soil type - Colour - Structure - Strength - Moisture - Bedding - Plasticity - Sensitivity - Origin/Geological Unit - Comments

Coarse: Soil Type - Colour - Structure - Grading - Strength/Relative Density - Moisture - Origin/Geological Unit - Comments

Moisture Condition

Condition	Look and Feel	Granular Soils	Cohesive Soils	Abbreviation
Dry	Looks and feels dry	Run freely through hands	Hard, powdery or friable	D
Moist	Feels cool, darkened in colour	Tend to cohere	Weakened by moisture, but no free water on hands when remoulding	M
Wet			Weakened by moisture, free water forms on hands when handling	W
Saturated	Feels cool, darkened in colour and free water is present on the sample			S

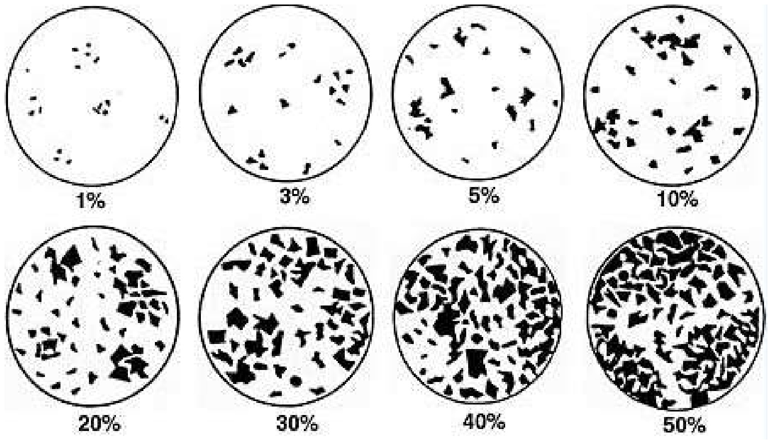
PLASTICITY (CLAYS & SILTS)

Term	Description
High plasticity	Can be mouled or deformed over a wide range of moisture contents without cracking or showing any tendancy to volume change
Low plasticity	When moulded can be crumbled in the fingers; may show quick or dilatant behaviour

GRADING (GRAVELS & SANDS)

Term	Description	
Well Graded	Good representation of all particle size ranges from largest to smallest	
Poorly Graded	Limited representation of grain sizes - further divided into:	
	Uniformly graded	Most particles about the same size
	Gap graded	Absence of one or more intermediate sizes

VISUAL PROPORTION PERCENTAGE



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HAND AUGER BOREHOLE - HA01

Client: Te Rapa Waikato Racing Club
Project: Te Rapa Racecourse Redevelopment
Site Address: Ken Browne Drive, Hamilton
Project No: HAM2016_0109
Date: 30/05/2017
Borehole Location: Refer to site plan

Scale: 1:30

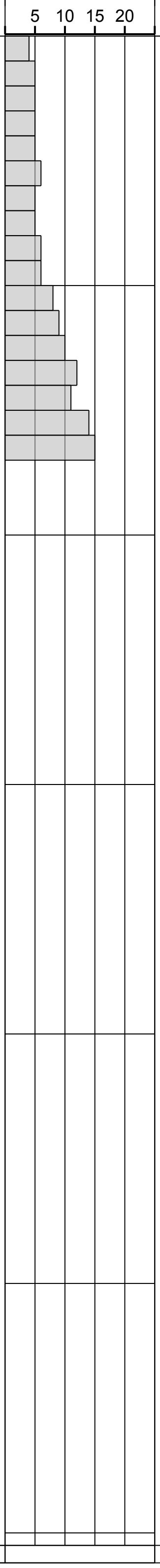
Sheet 1 of 1

Logged by: JC
Checked by: KAL

Position:
Survey Source:

Elevation:
Datum:

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Consistency/ Relative Density	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm)	Comments
TS			0	▨	OL: TOPSOIL - ORGANIC SILT: dark brown, low plasticity	M					
Hinuera Formation				⁘	SP: SAND: brown, fine to coarse, poorly graded, with minor fine subrounded gravel.	M to W	MD				
			1				D				
			1.5		Borehole terminated at 1.500 m						
			2								
			3								
			4								
			5								
			6								



Termination reason: Refusal on gravel

Remarks: Groundwater not encountered

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA02

Client: Te Rapa Waikato Racing Club
Project: Te Rapa Racecourse Redevelopment
Site Address: Ken Browne Drive, Hamilton
Project No: HAM2016_0109
Date: 30/05/2017
Borehole Location: Refer to site plan

Scale: 1:30 Sheet 1 of 1

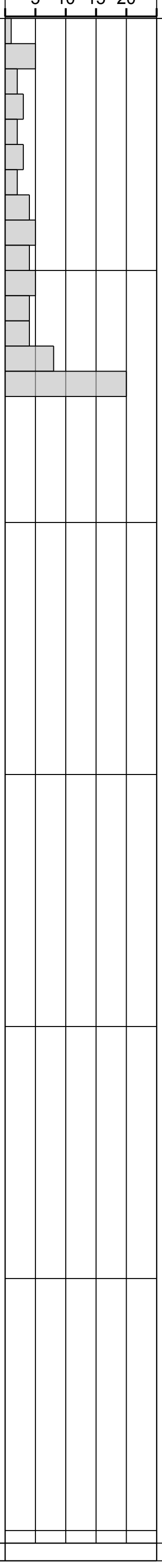
Logged by: AMH
Checked by: KAL

Position:
Survey Source:

Elevation:
Datum:

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Consistency/ Relative Density	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Comments
Top soil					OL: TOPSOIL - ORGANIC SILT: dark brown, low plasticity, with minor fine to medium sand and trace clay	M	S to F			
Hinuera Formation					SM: Sandy SILT: brown to grey, no to low plasticity, fine to coarse sand, with minor fine to medium sub-angular to sub-rounded gravel					
					SP: SAND: pale brown, fine to coarse, poorly graded (predominantly fine), with some fine to coarse sub-angular to sub-rounded gravel *0.5m: sand becomes fine to medium, with minor silt, no gravel*	M to W	L MD			
			1		SW: SAND: brownish grey, fine to coarse, well graded, with trace fine sub-rounded gravel	W	D to VD			
					Borehole terminated at 1.400 m					
			2							
			3							
			4							
			5							
			6							

Dynamic Cone Penetrometer (Blow/100 mm)



Termination reason: Refusal on gravel

Remarks: Groundwater not encountered

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA03

Client: Te Rapa Waikato Racing Club
Project: Te Rapa Racecourse Redevelopment
Site Address: Ken Browne Drive, Hamilton
Project No: HAM2016_0109
Date: 30/05/2017
Borehole Location: Refer to site plan

Scale: 1:30

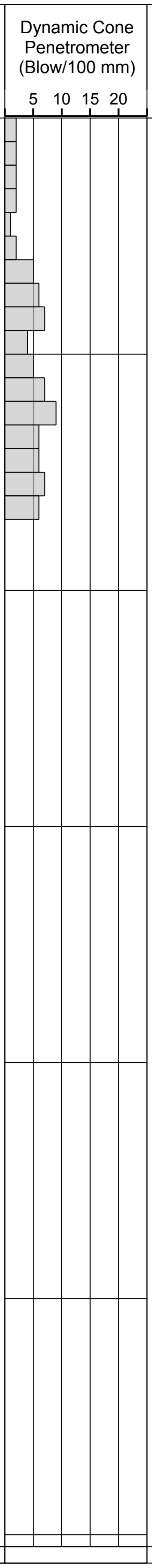
Sheet 1 of 1

Logged by: JC
Checked by: KAL

Position:
Survey Source:

Elevation:
Datum:

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Consistency/ Relative Density	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm)	Comments
Topsoil					OL: TOPSOIL - ORGANIC SILT: dark brown, low plasticity	M					
Hinuera Formation					SM: Sandy SILT: light brown, mottled orange, low plasticity	M to W	Vst to H		204+		
					SP: SAND: light grey, fine, poorly graded, with minor silt	W	MD to D				
					SW: SAND: greyish brown, fine to coarse, well graded						
					Borehole terminated at 2.200 m						



Termination reason: Refusal on gravel

Remarks: Groundwater not encountered, but likely to be near bottom of hole as becoming more wet

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA04

Client: Te Rapa Waikato Racing Club
Project: Te Rapa Racecourse Redevelopment
Site Address: Ken Browne Drive, Hamilton
Project No: HAM2016_0109
Date: 30/05/2017
Borehole Location: Refer to site plan

Scale: 1:30

Sheet 1 of 1

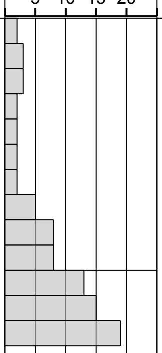
Logged by: JC
Checked by: KAL

Position:
Survey Source:

Elevation:
Datum:

Unit	Groundwater	RL (m)	Depth (m)	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Consistency/ Relative Density	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Comments
Topsoil				OL: TOPSOIL - ORGANIC SILT: dark brown, low plasticity	M				
Hinuera Formation				SM: Silty SAND: light brown, mottled orange, fine grained, poorly graded	M to W	L			
			1	SW: Gravelly SAND: light brown to grey, fine to coarse sand, fine to medium sub-rounded gravel, well graded	W	D			
				Borehole terminated at 1.100 m					
			2						
			3						
			4						
			5						
			6						

Dynamic Cone Penetrometer (Blow/100 mm)



Termination reason: Refusal on gravel

Remarks: Groundwater not encountered

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA05

Client: Te Rapa Waikato Racing Club
Project: Te Rapa Racecourse Redevelopment
Site Address: Ken Browne Drive, Hamilton
Project No: HAM2016_0109
Date: 30/05/2017
Borehole Location: Refer to site plan

Scale: 1:30

Sheet 1 of 1

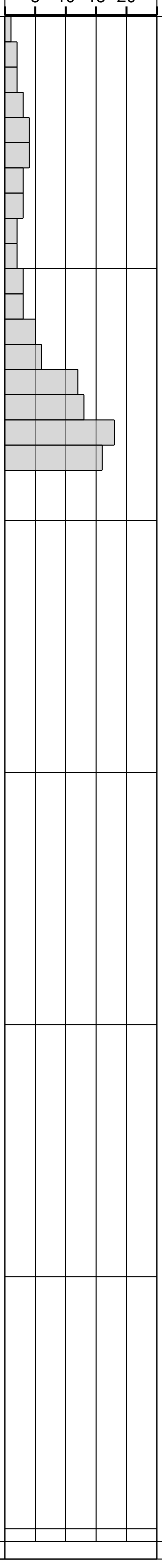
Logged by: AMH
Checked by: KAL

Position:
Survey Source:

Elevation:
Datum:

Unit	Groundwater	RL (m)	Depth (m)	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Consistency/ Relative Density	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Comments
Topsoil				OL: TOPSOIL - ORGANIC SILT: brown, low plasticity, with minor fine to coarse sand and minor fine to coarse sub-angular to sub-rounded gravel, and some rootlets	M			UTP - SV influenced by sand	
Fill				SW: Silty SAND: pale brown, fine to coarse, well graded, with minor fine to coarse angular to sub-rounded gravel		L			
Topsoil				OL: ORGANIC SILT - BURIED TOPSOIL: dark brown, low plasticity, with trace fine to coarse sand, with some rootlets *0.8m: becomes dark brown to black*					
Hinuera Formation			1	SP: Silty SAND: pale brown streaked/mottled orange, fine, poorly graded, slightly dilatant	M to W	MD			
				SW: Gravelly SAND: brown, fine to coarse sand, fine to coarse sub-angular to sub-rounded gravel (some pumice), well graded, with minor silt	W	D			
				Borehole terminated at 1.500 m					
			2						
			3						
			4						
			5						
			6						

Dynamic Cone Penetrometer (Blow/100 mm)



Termination reason: Refusal on gravel

Remarks: Groundwater not encountered

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA06

Client: Te Rapa Waikato Racing Club
Project: Te Rapa Racecourse Redevelopment
Site Address: Ken Browne Drive, Hamilton
Project No: HAM2016_0109
Date: 30/05/2017
Borehole Location: Refer to site plan

Scale: 1:30

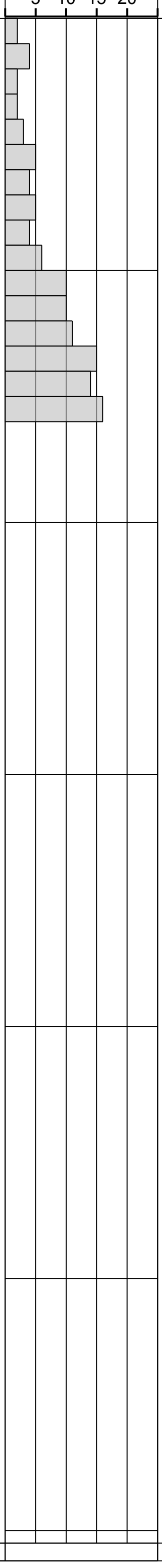
Sheet 1 of 1

Logged by: AMH
Checked by: KAL

Position:
Survey Source:

Elevation:
Datum:

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Consistency/ Relative Density	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm)	Comments
Topsoil					OL: TOPSOIL - ORGANIC SILT: dark brown, low plasticity	M					
Hinuera Formation					SP: Silty SAND: light brown with orange mottling, fine, poorly graded		L				
					SW: SAND: light grey, fine to coarse, well graded, with trace silt		MD				
			1		*1.0m: with some fine to medium sub-rounded gravel*	W	D				
					Borehole terminated at 1.200 m						
			2								
			3								
			4								
			5								
			6								



Termination reason: Refusal on gravel

Remarks: Groundwater not encountered

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA07

Client: Te Rapa Waikato Racing Club
Project: Te Rapa Racecourse Redevelopment
Site Address: Ken Browne Drive, Hamilton
Project No: HAM2016_0109
Date: 30/05/2017
Borehole Location: Refer to site plan

Scale: 1:30

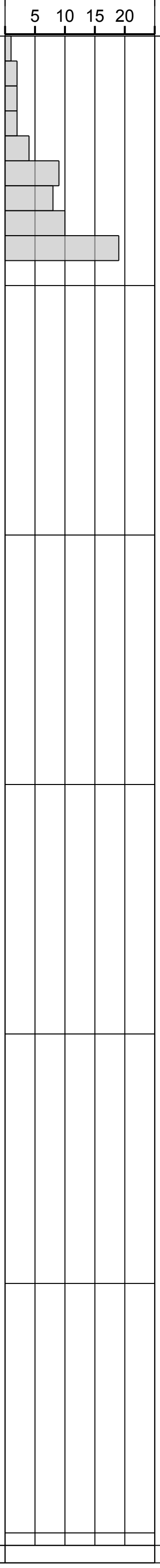
Sheet 1 of 1

Logged by: AMH
Checked by: KAL

Position:
Survey Source:

Elevation:
Datum:

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Consistency/ Relative Density	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm)	Comments
TS	▼				ML: TOPSOIL - Clayey SILT: dark brown, low plasticity, with some rootlets	W					
Hinuera Formation					ML: SILT: grey with orange streaks/mottling, low plasticity, with minor rootlets	M to W	F to St	Q	V-56		
					SW: SAND: grey to pale brown, fine to coarse, well graded, with trace fine sub-rounded gravel (pumice), and trace silt	W	D				
			1		*0.8m: becomes mainly coarse grained*	S					
					Borehole terminated at 1.200 m						
			2								
			3								
			4								
			5								
			6								



Termination reason: No recovery due to saturated sands.

Remarks: Groundwater at surface, however may be influenced by ponding surface water in the area.
Unit TS = Topsoil.

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA08

Client: Te Rapa Waikato Racing Club
Project: Te Rapa Racecourse Redevelopment
Site Address: Ken Browne Drive, Hamilton
Project No: HAM2016_0109
Date: 30/05/2017
Borehole Location: Refer to site plan

Scale: 1:30

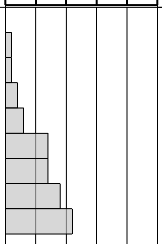
Sheet 1 of 1

Logged by: AMH
Checked by: KAL

Position:
Survey Source:

Elevation:
Datum:

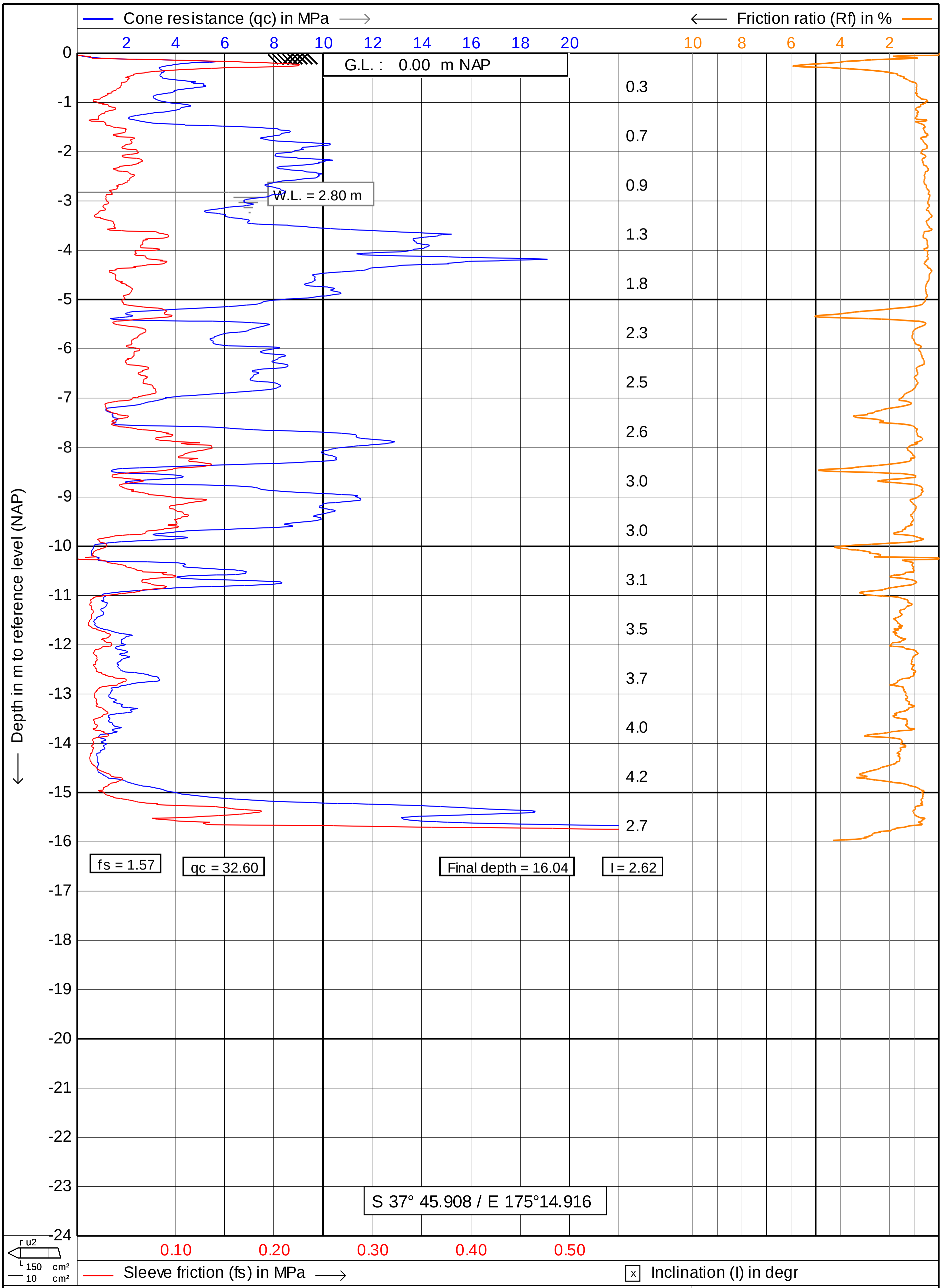
Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Consistency/ Relative Density	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm) 5 10 15 20	Comments
Top soil	▼				OL: TOPSOIL - Clayey SILT: dark brown, low plasticity, with some rootlets	W to S					
Hinuera Formation					ML: SILT: grey with orange streaks/mottling, low plasticity, with trace rootlets *0.5m: with some fine to coarse sand*	M to W	St to VSt	Q	V-152(9) SV influenced by sand		
					SW: SAND: pale grey to pale brown, fine to coarse, well graded, with minor fine to medium sub-rounded gravel (pumice), and trace silt	S	MD to D				
			1		Borehole terminated at 0.900 m						
			2								
			3								
			4								
			5								
			6								



Termination reason: No recovery due to saturated sands.

Remarks: Groundwater at surface, however may be influenced by ponding surface water in the area.

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.



1.47

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

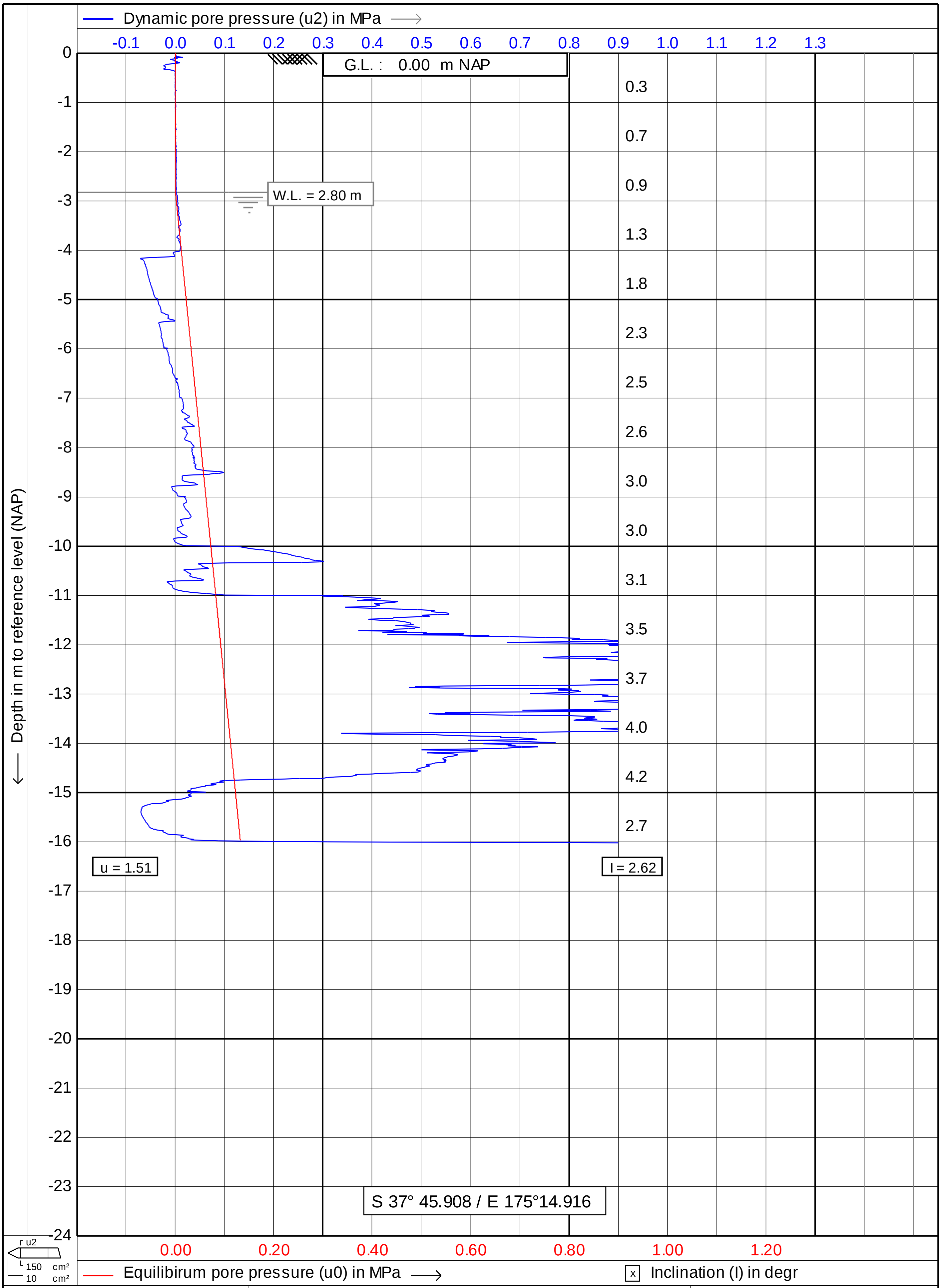
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT01** **1/15**

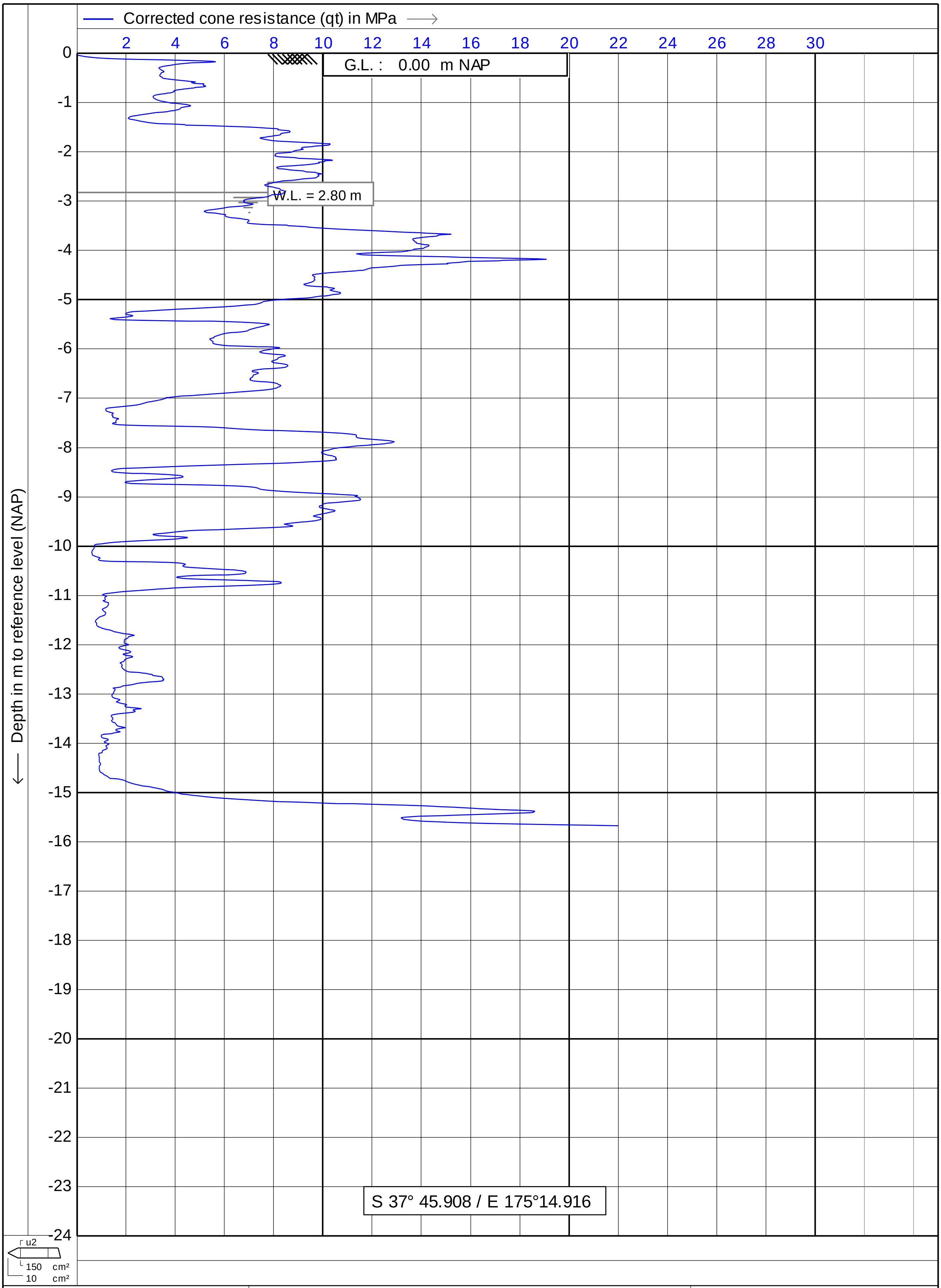


ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT01** **2/15**

1.47



CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

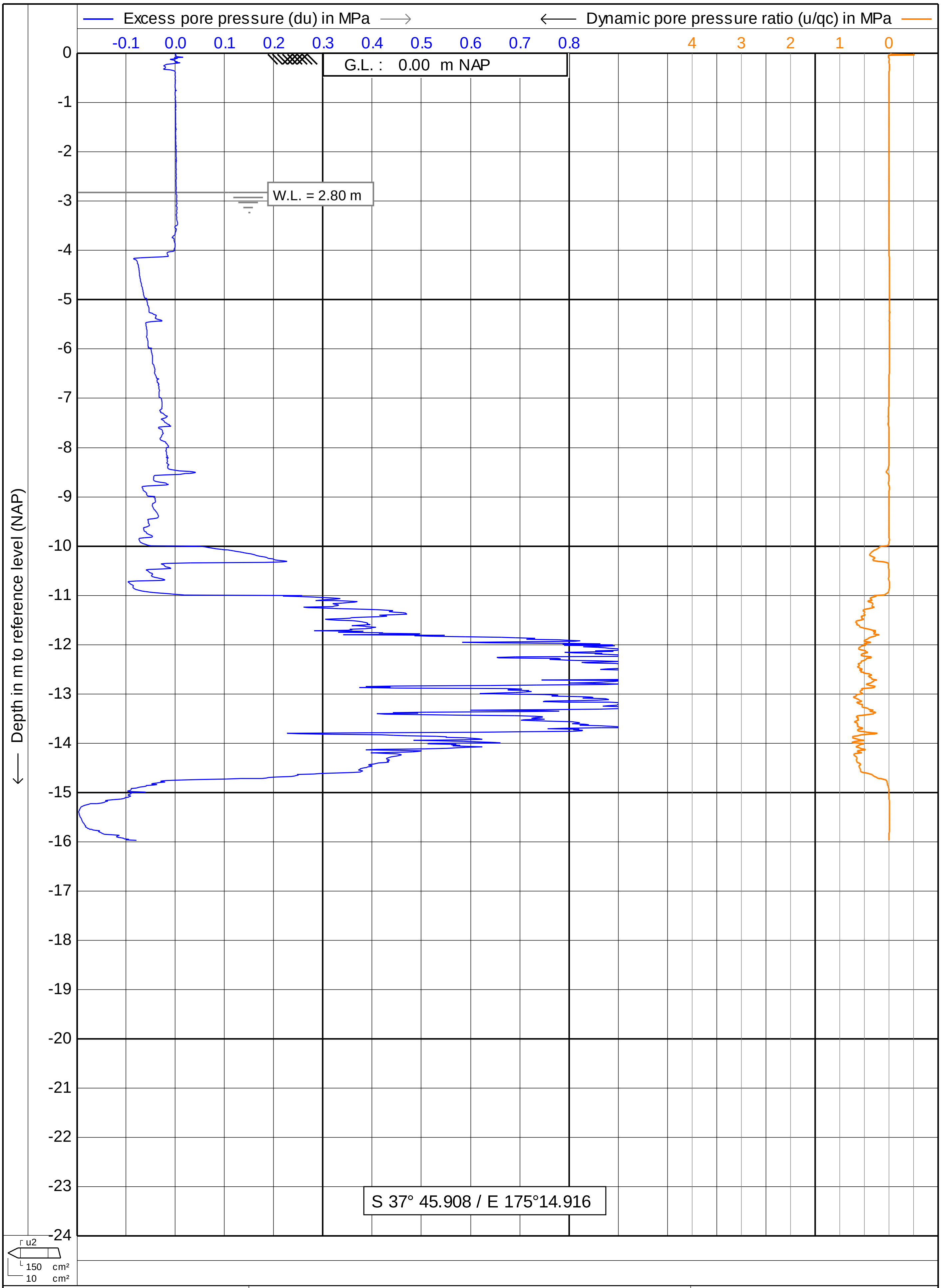
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT01** **3/15**

1.47



CONE PENETROMETER TESTING

1.47

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

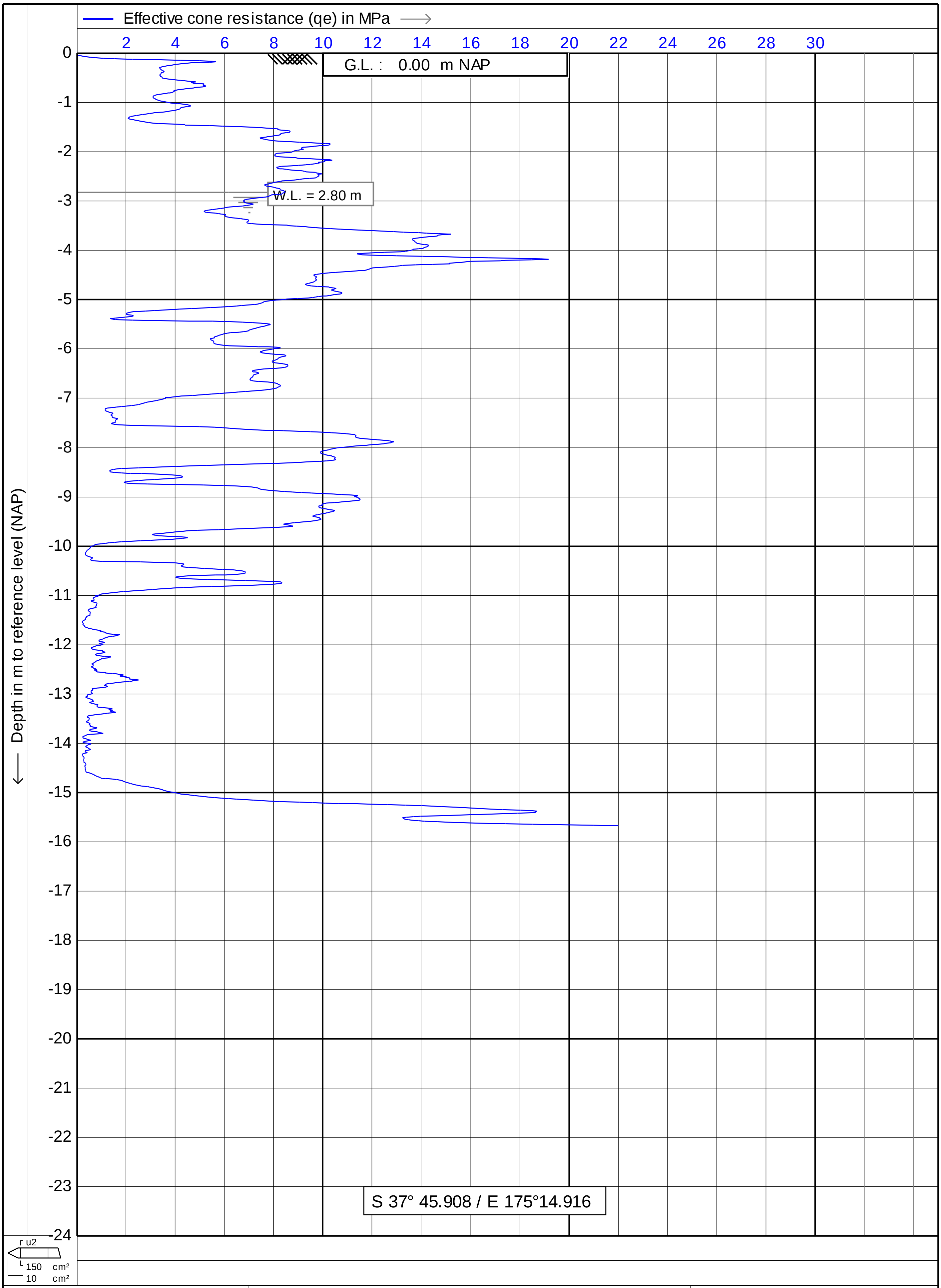
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT01** **4/15**



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

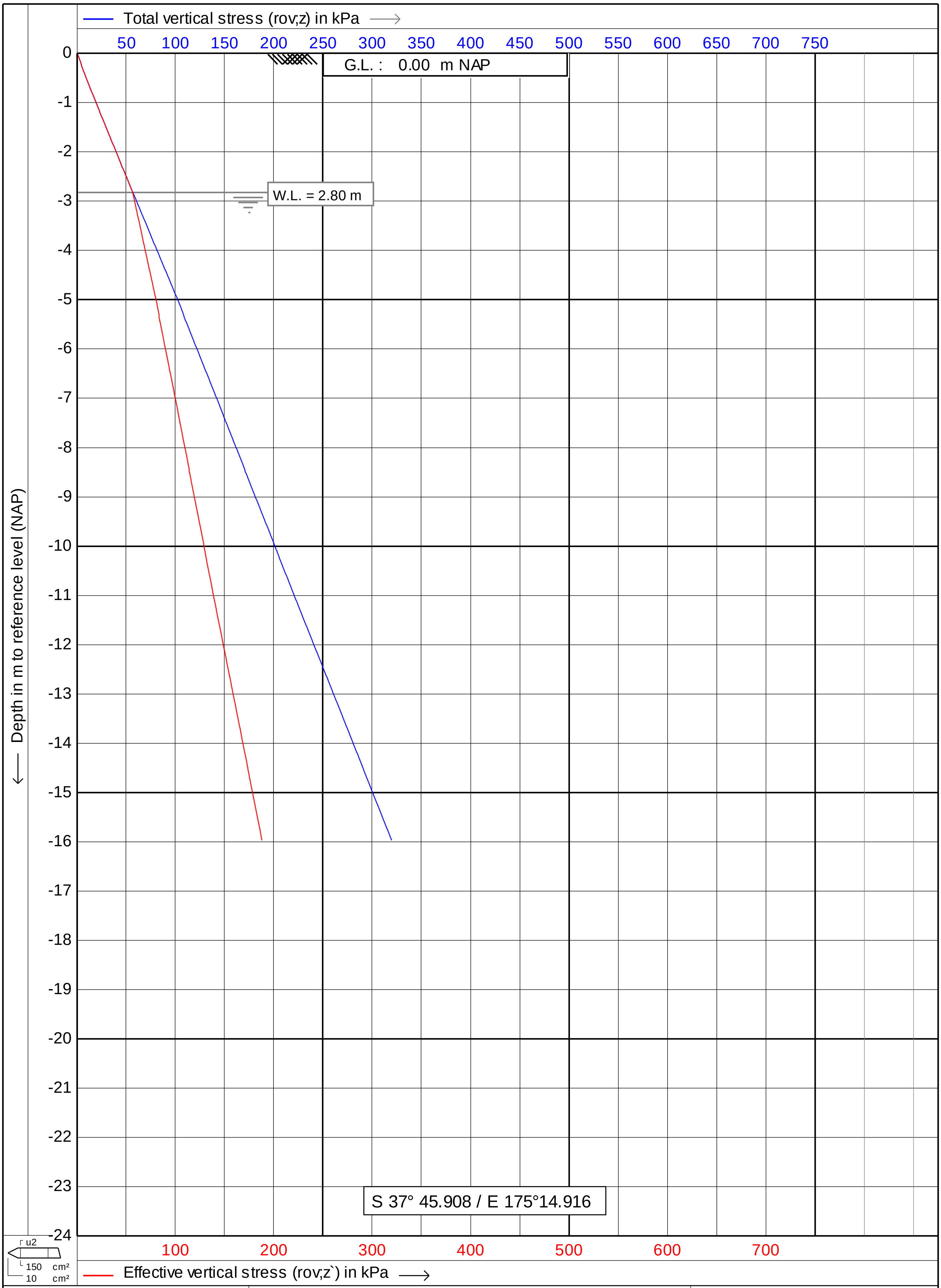
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT01** **5/15**

1.47



CPT it
CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

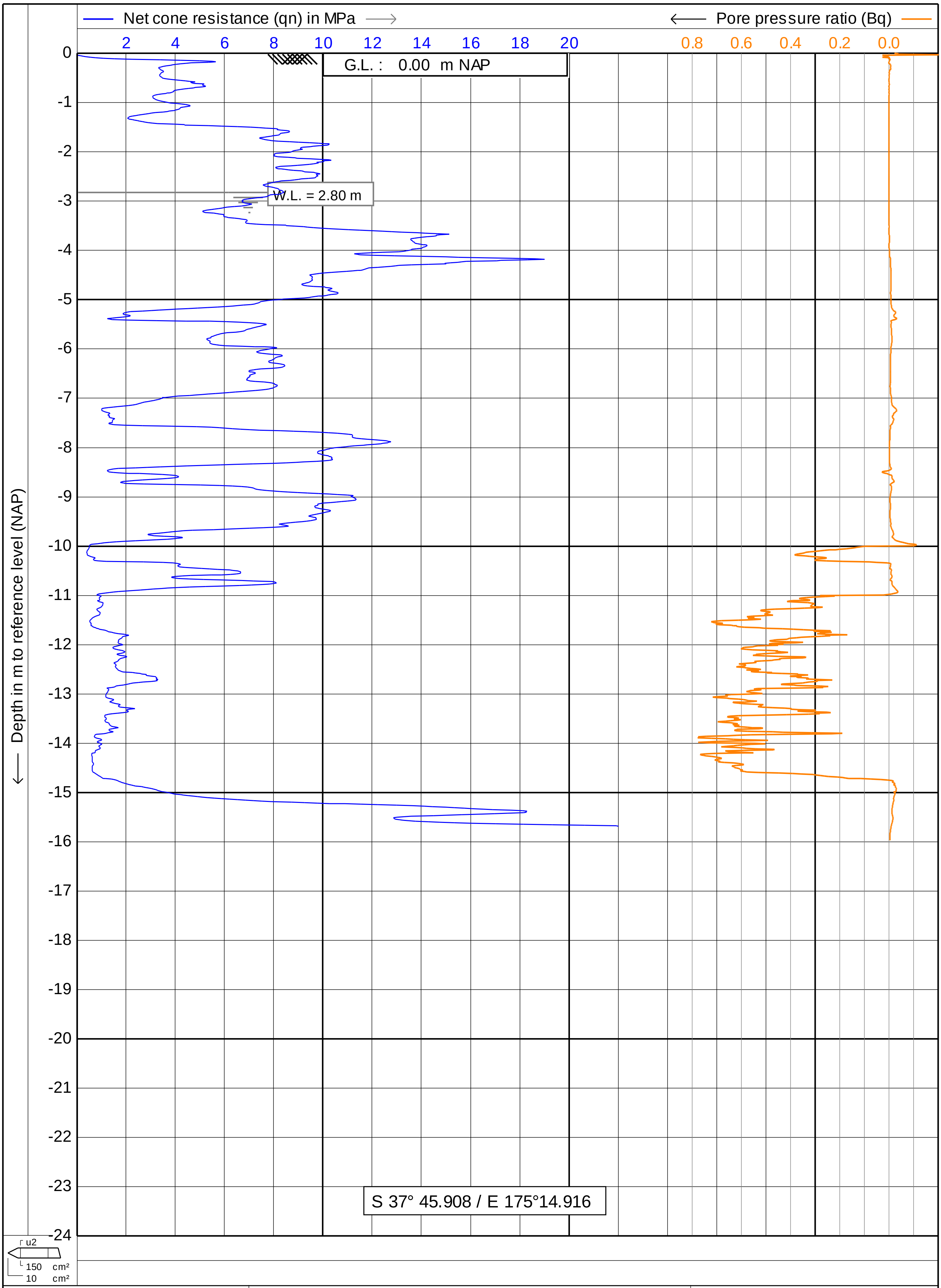
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT01** **6/15**

1.47



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

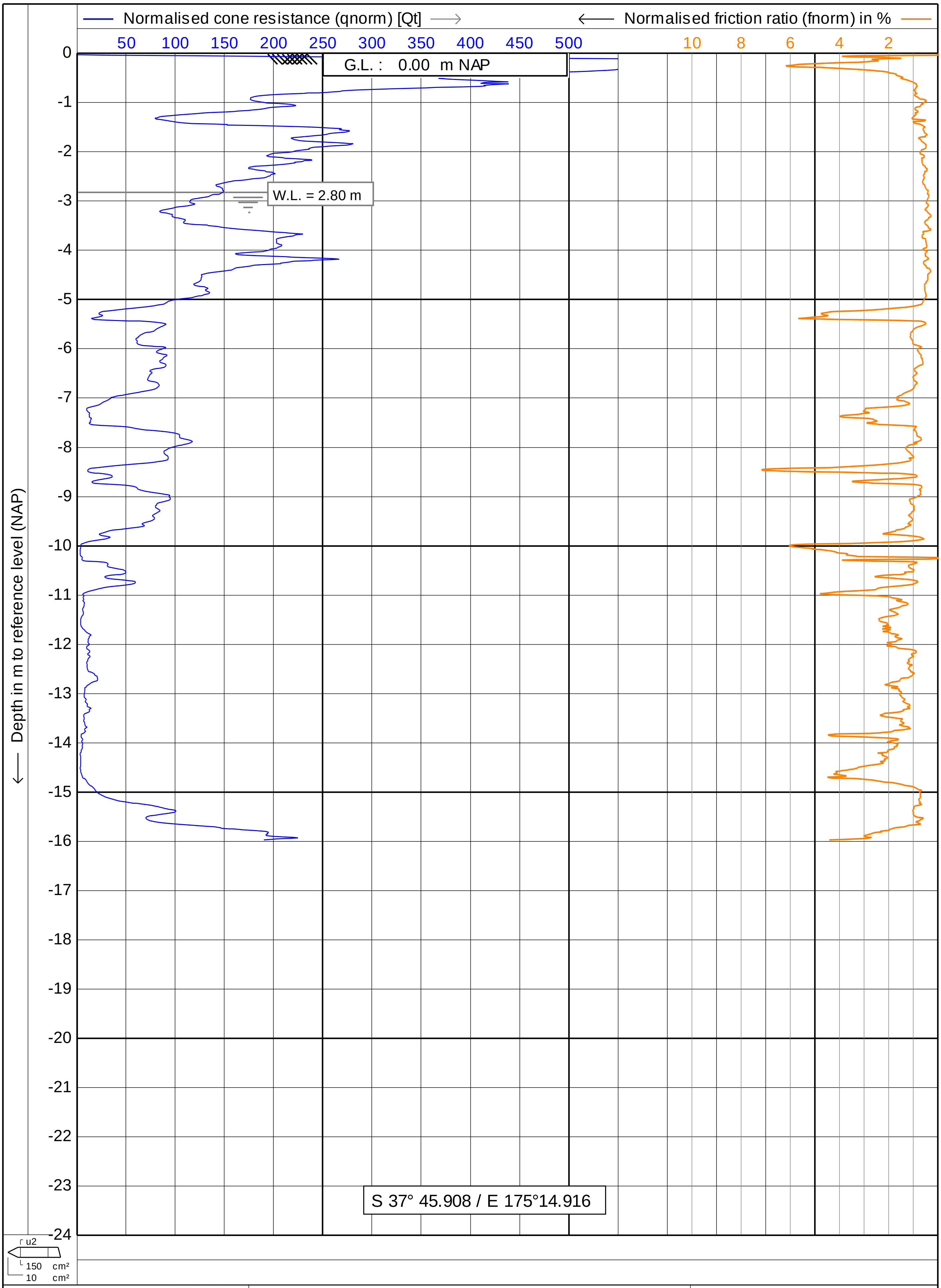
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT01** **7/15**

1.47

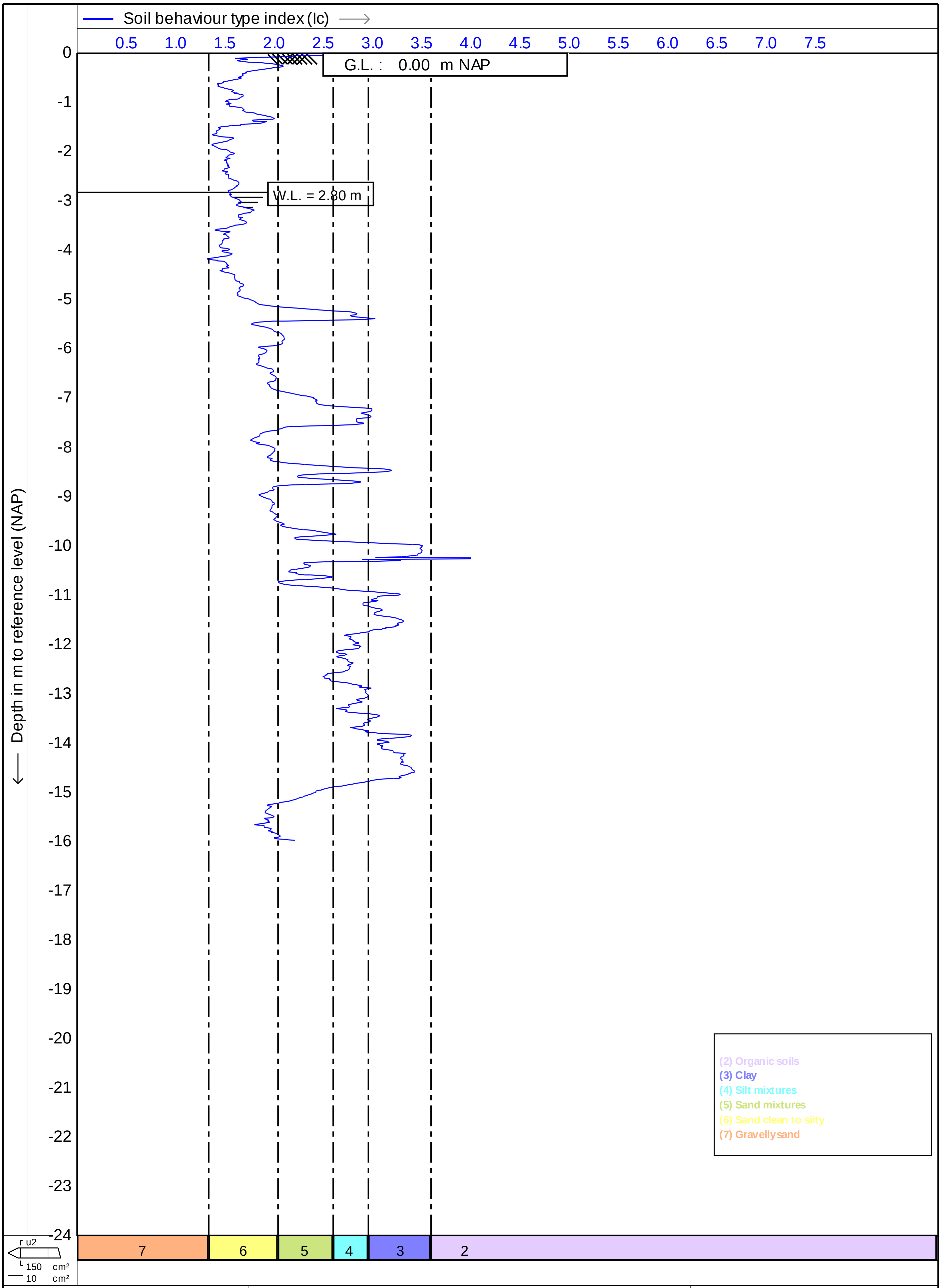


ISO 22476-1:2012 Application class 1 Test type TE1

Project :	**TE RAPA RACECOURSE DEVELOPMENT**
Location:	**TE RAPA RACECOURSE**
Position:	**0, 0**

Date :	**23-May-17**
Cone no. :	**S10CFIIP.S16082**
Project no. :	**17017/HAM2017-109**
CPT no. :	**CPT01** **8/15**

1.47



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

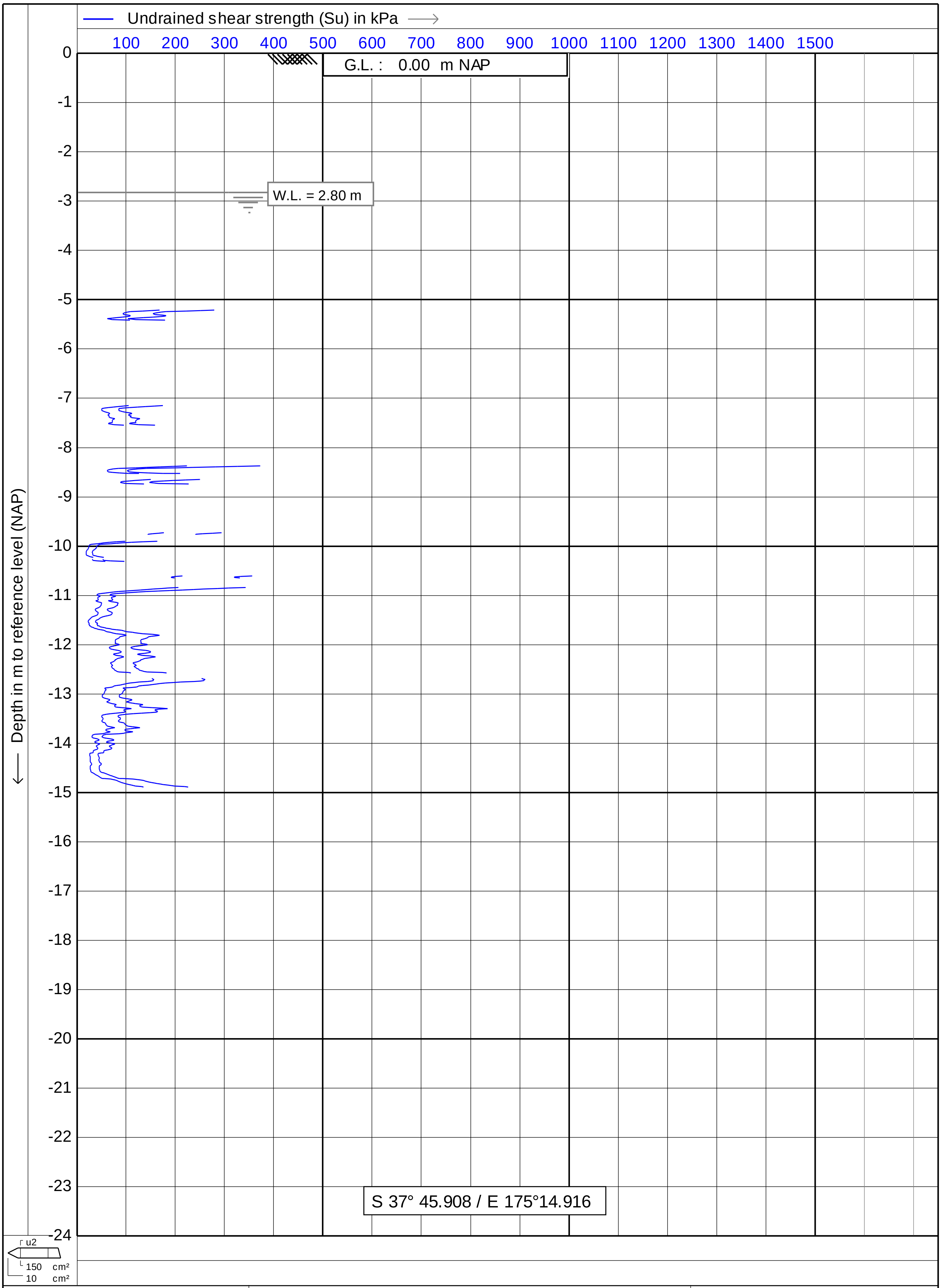
Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT01** **9/15**

CPT it
CONE PENETROMETER TESTING

1.47



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT01**

10/15

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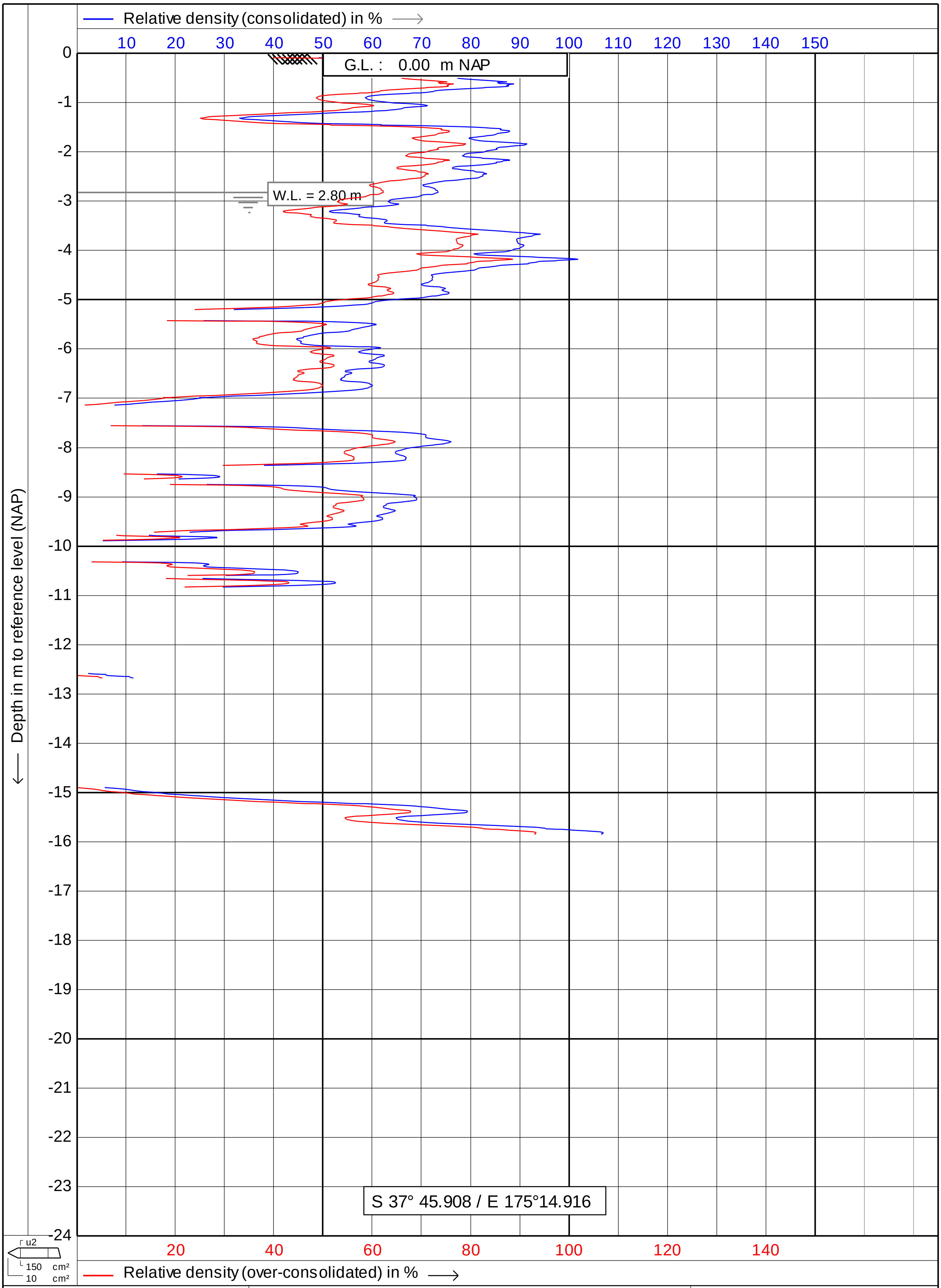

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT01** **11/15**

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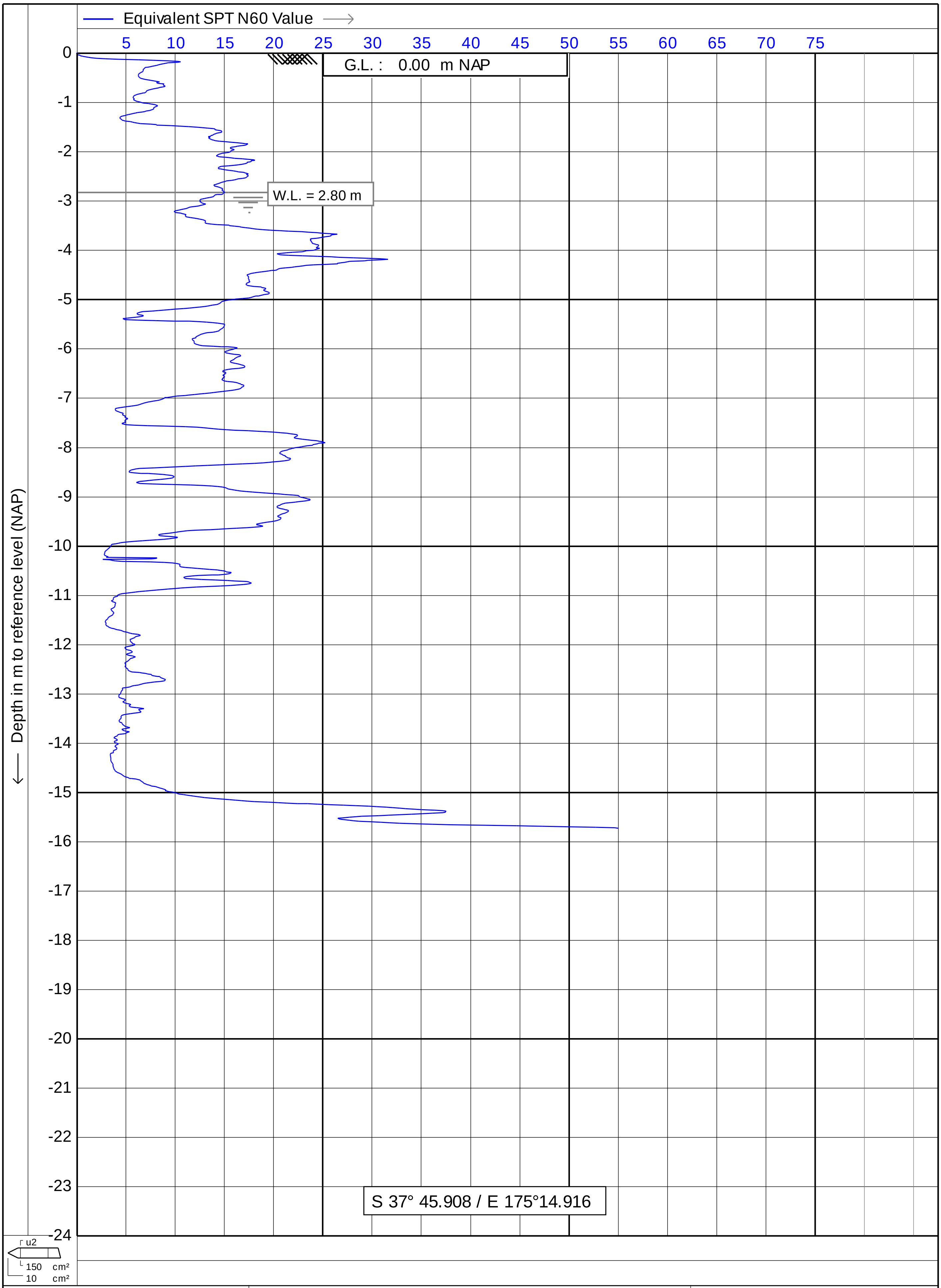

CONE PENETROMETER TESTING

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ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

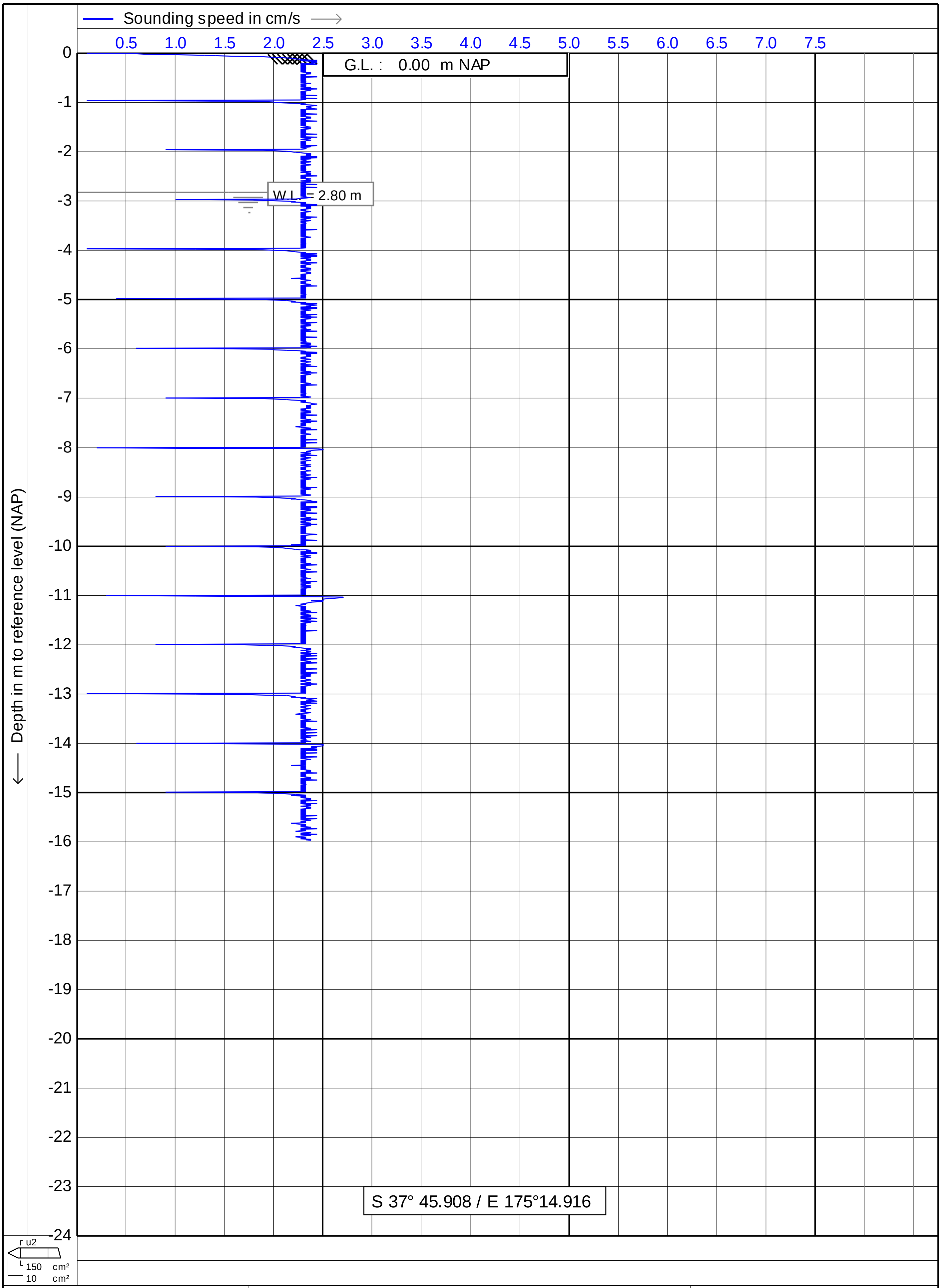
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT01**

12/15



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT01**

13/15

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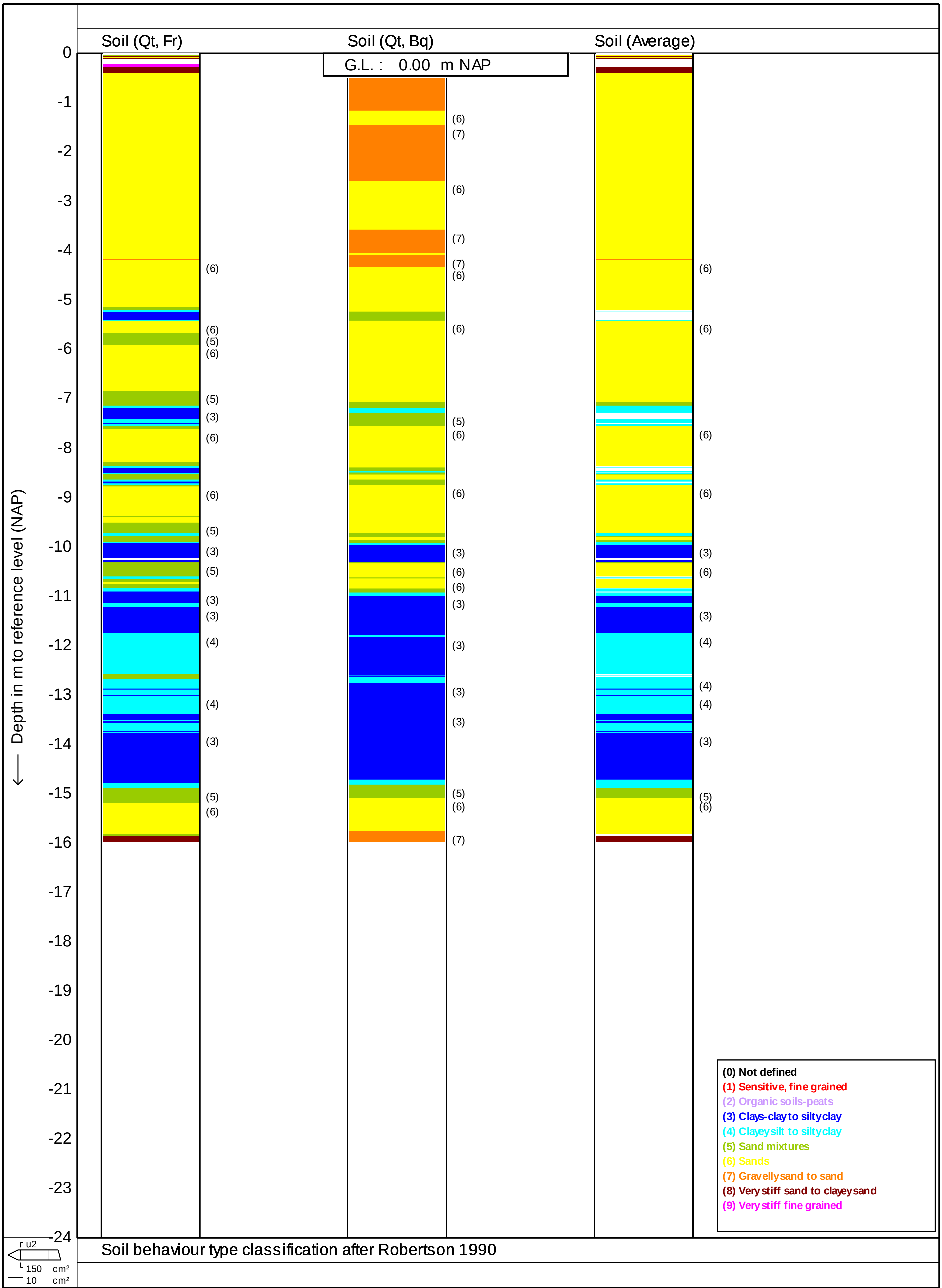

Soil behaviour type classification after Robertson 1990

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT01**

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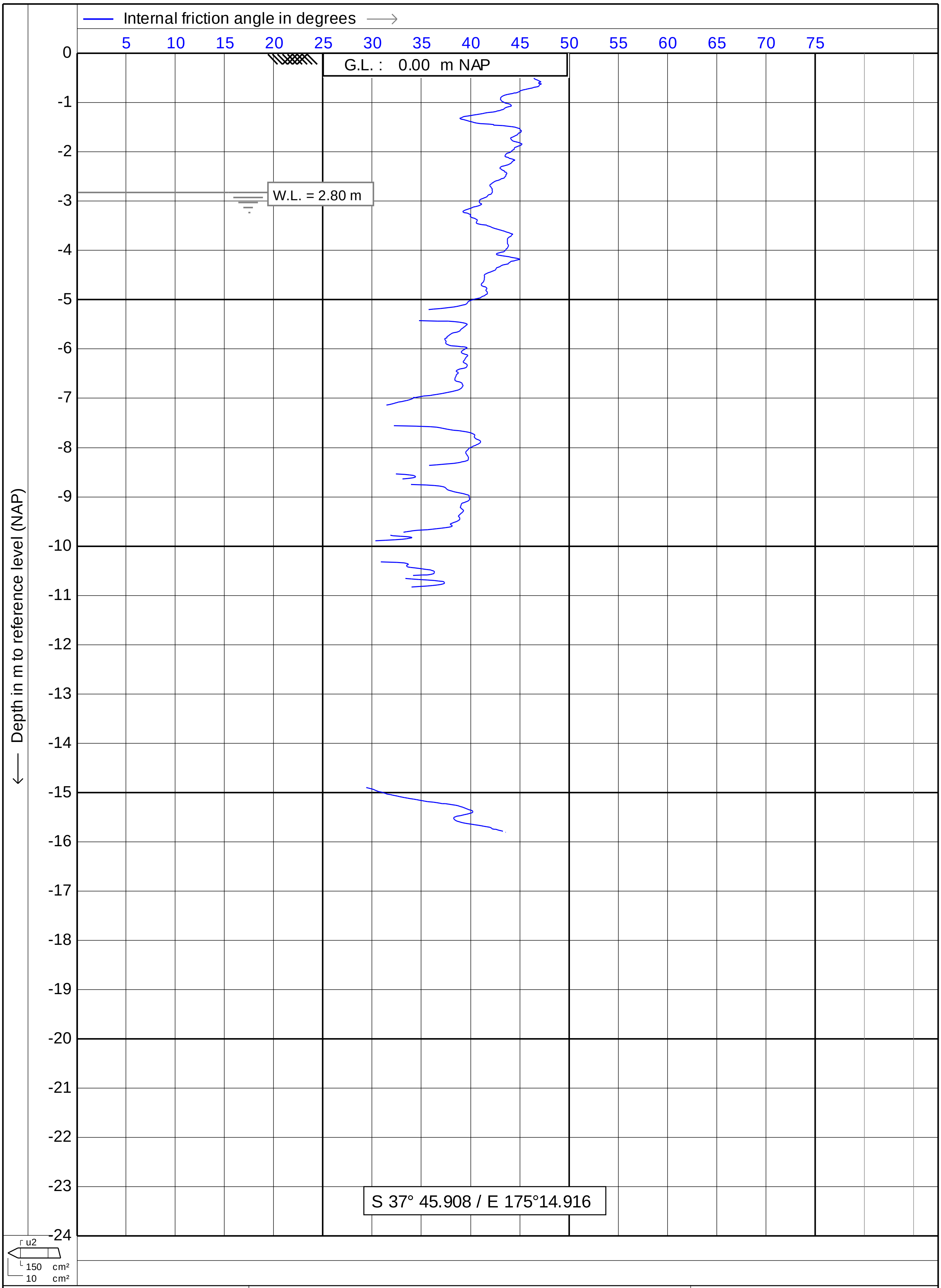

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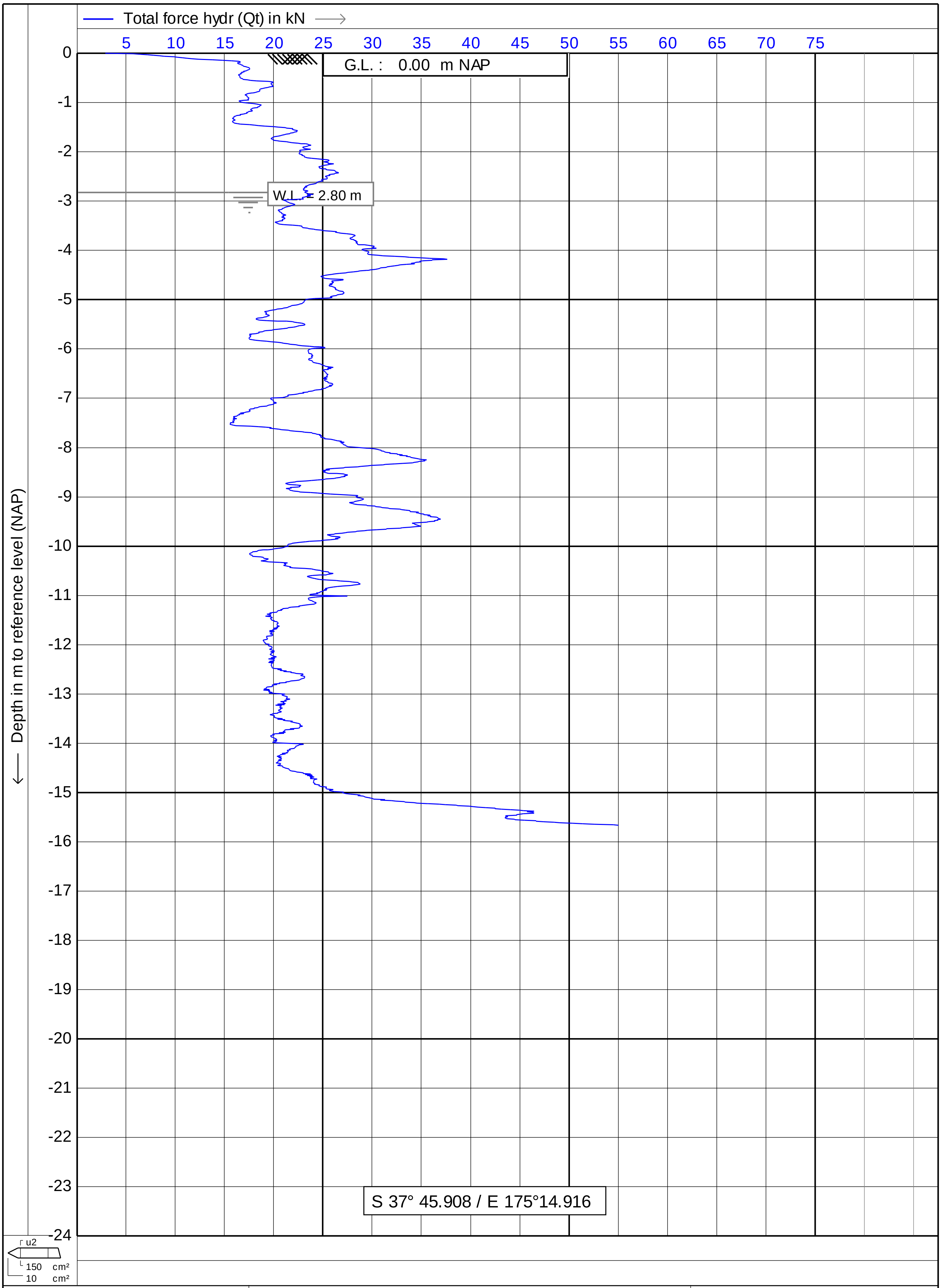
CPT it
CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1	
Project : **TE RAPA RACECOURSE DEVELOPMENT**	Date : **23-May-17**
Location: **TE RAPA RACECOURSE**	Cone no. : **S10CFIIP.S16082**
Position: **0, 0**	Project no. : **17017/HAM2017-109**
	CPT no. : **CPT01** **15/15**



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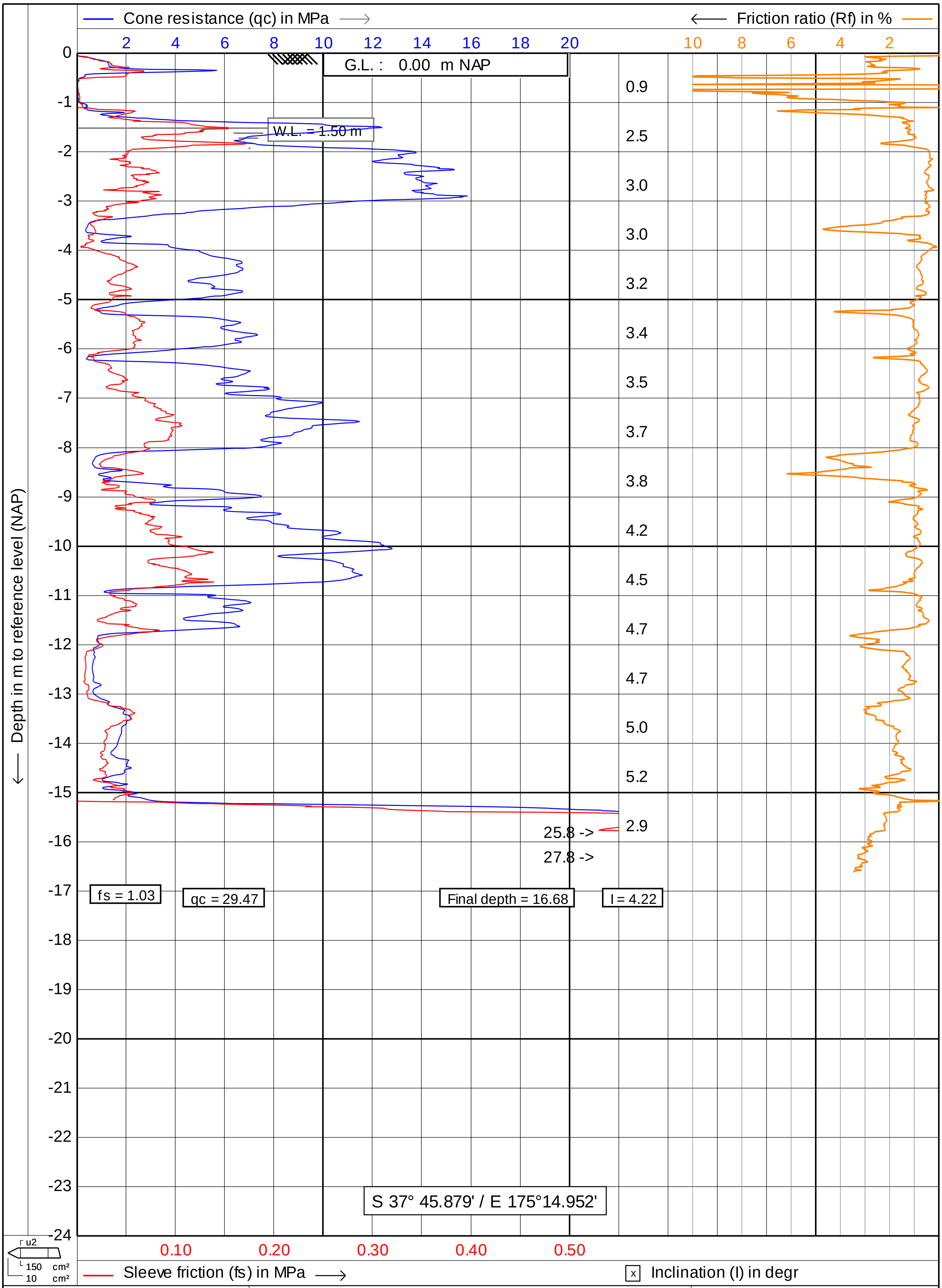
ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT01** **16/15**



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

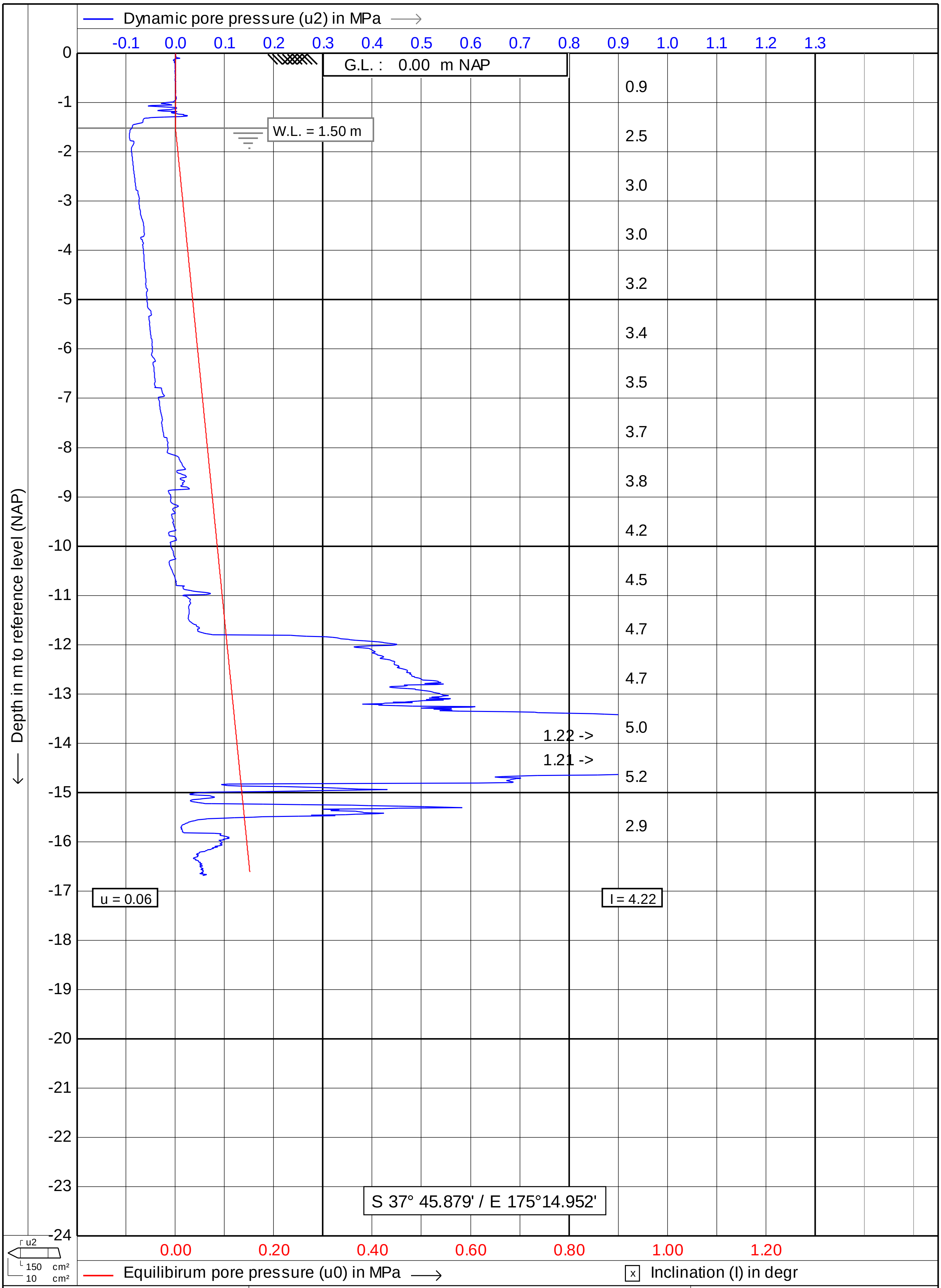
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02** **1/15**



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ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

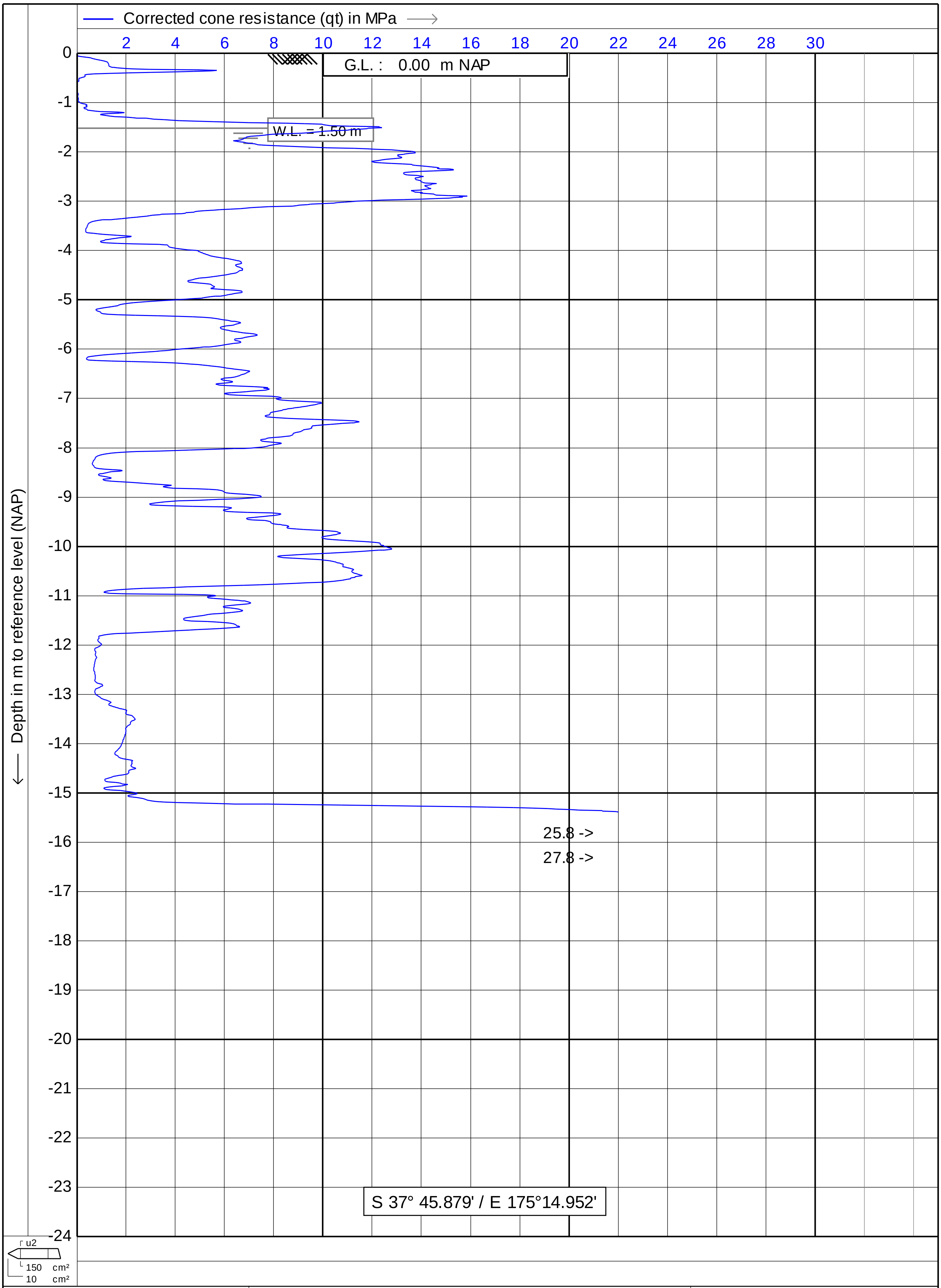
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02**

2/15



CONE PENETROMETER TESTING

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ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

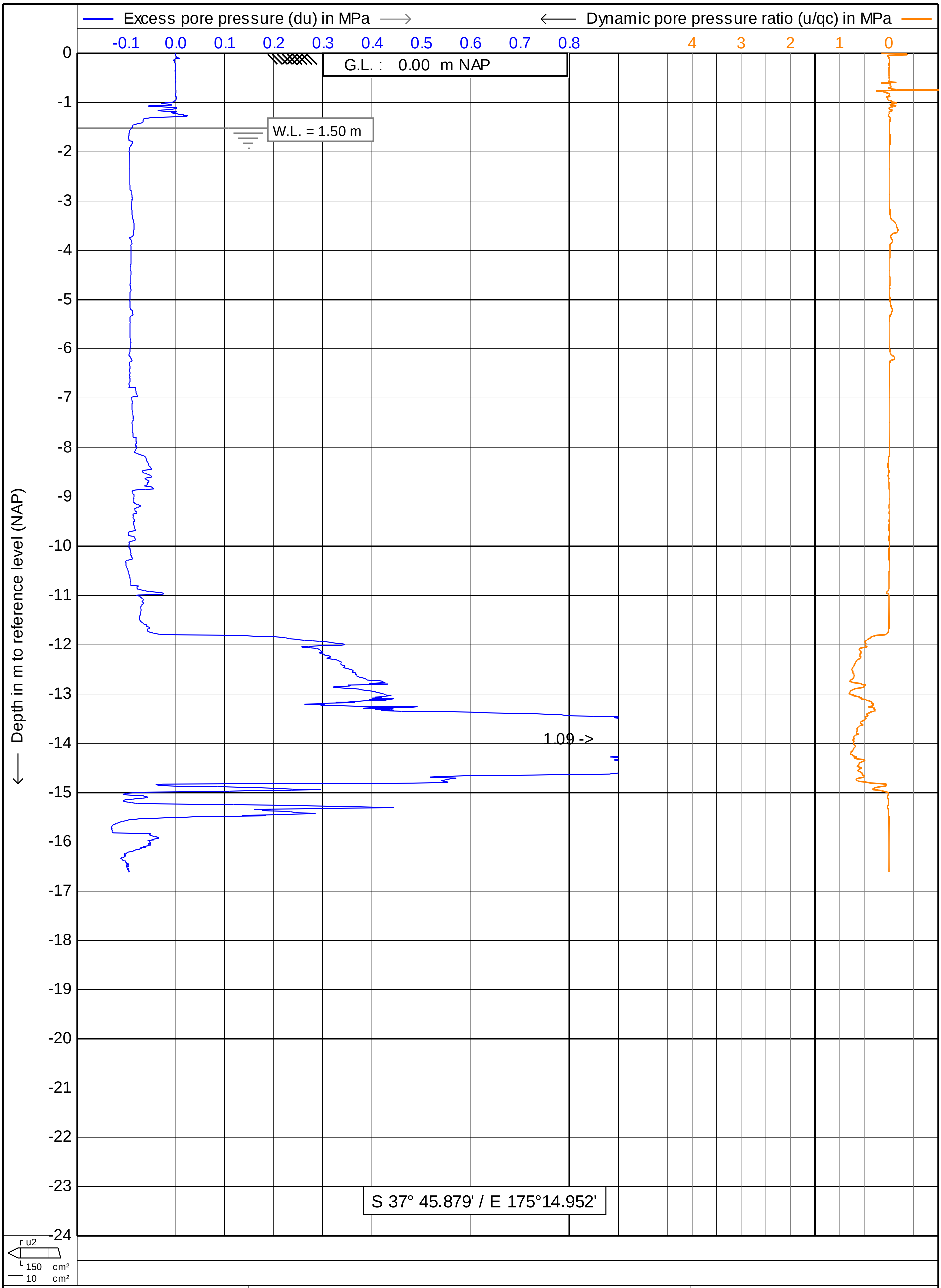
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02**

3/15



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CONE PENETROMETER TESTING

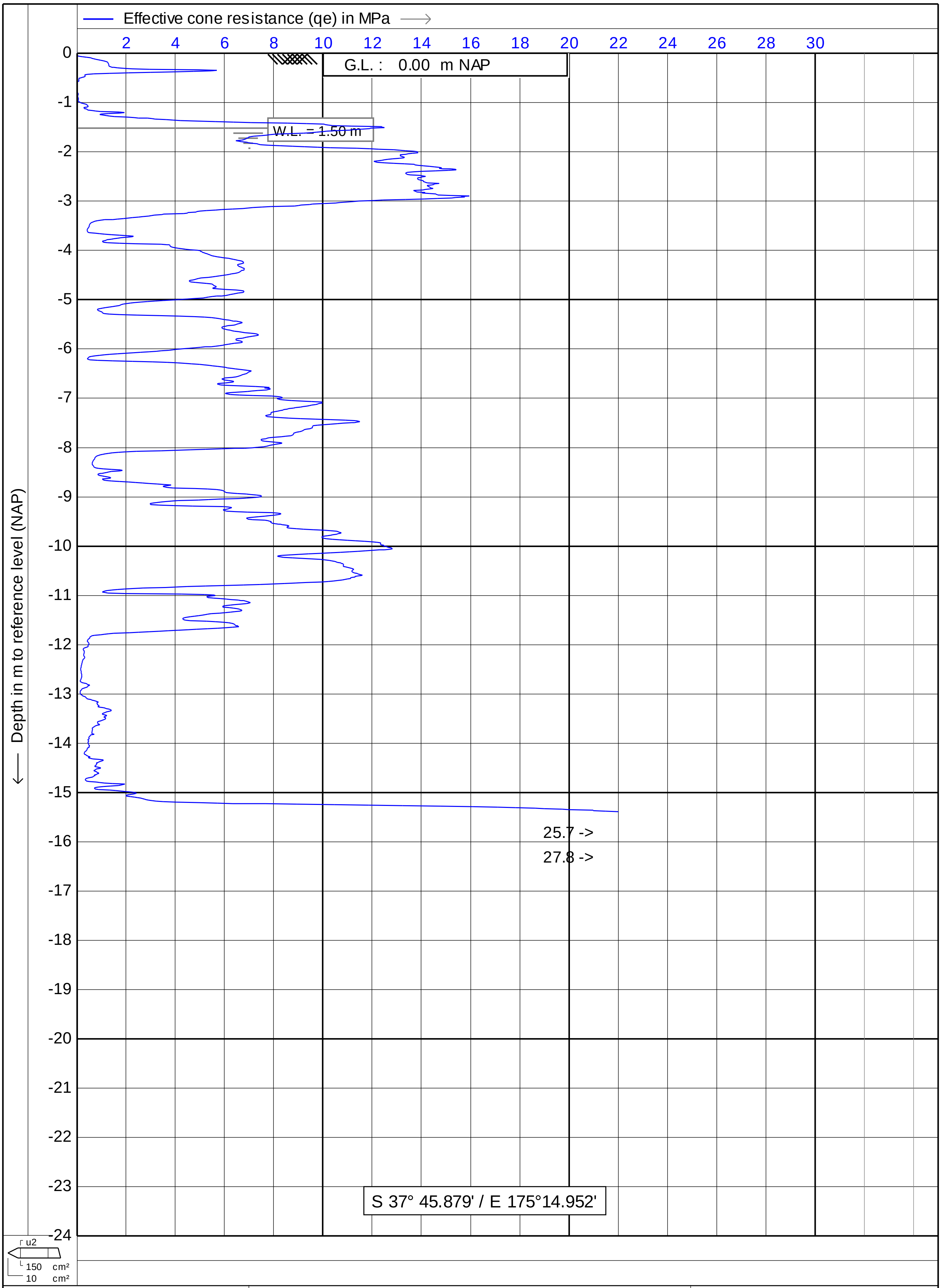
ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no. :	**17017/HAM2017-109**
CPT no.	: **CPT02** **4/15**



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02** **5/15**

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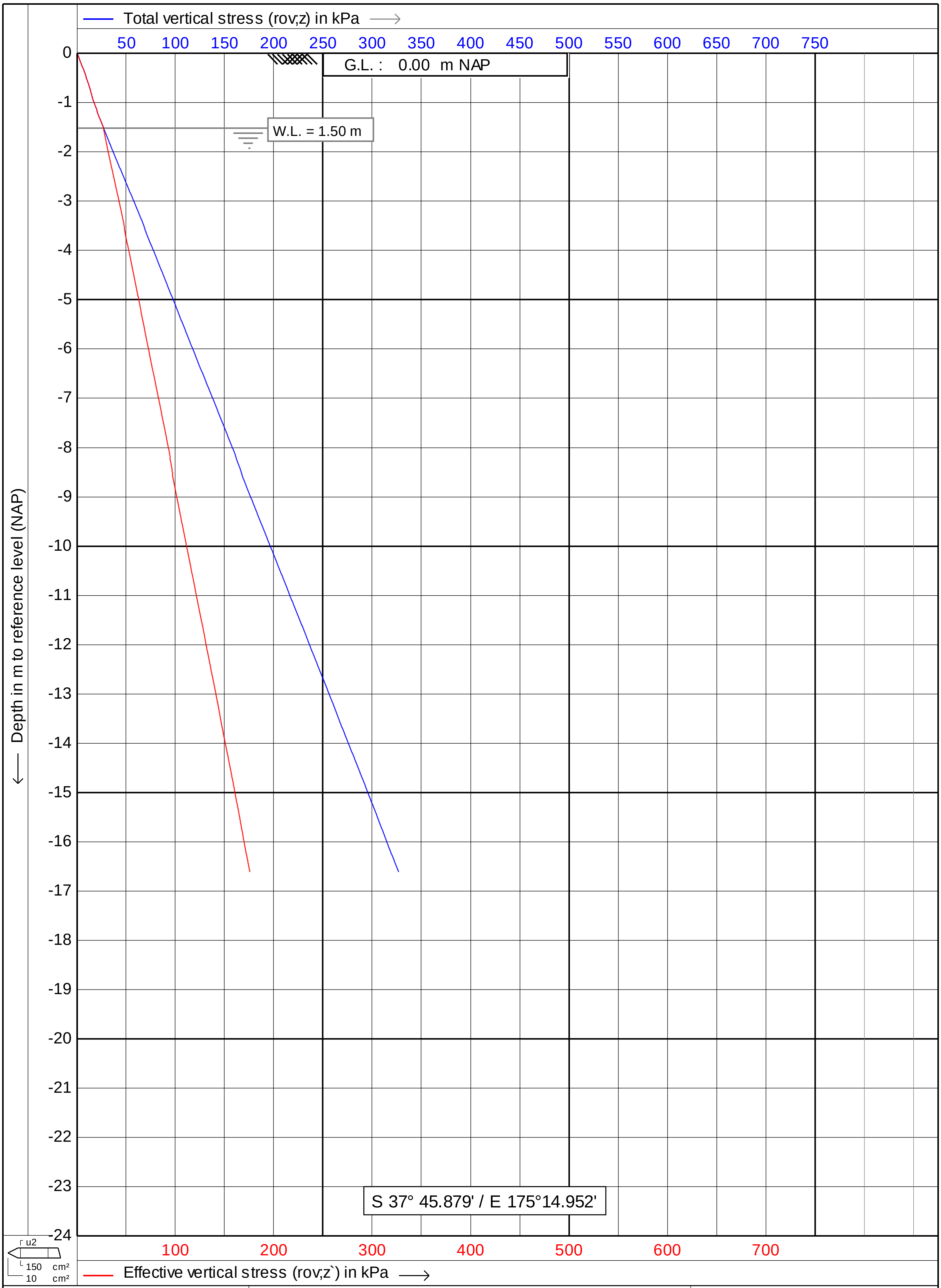
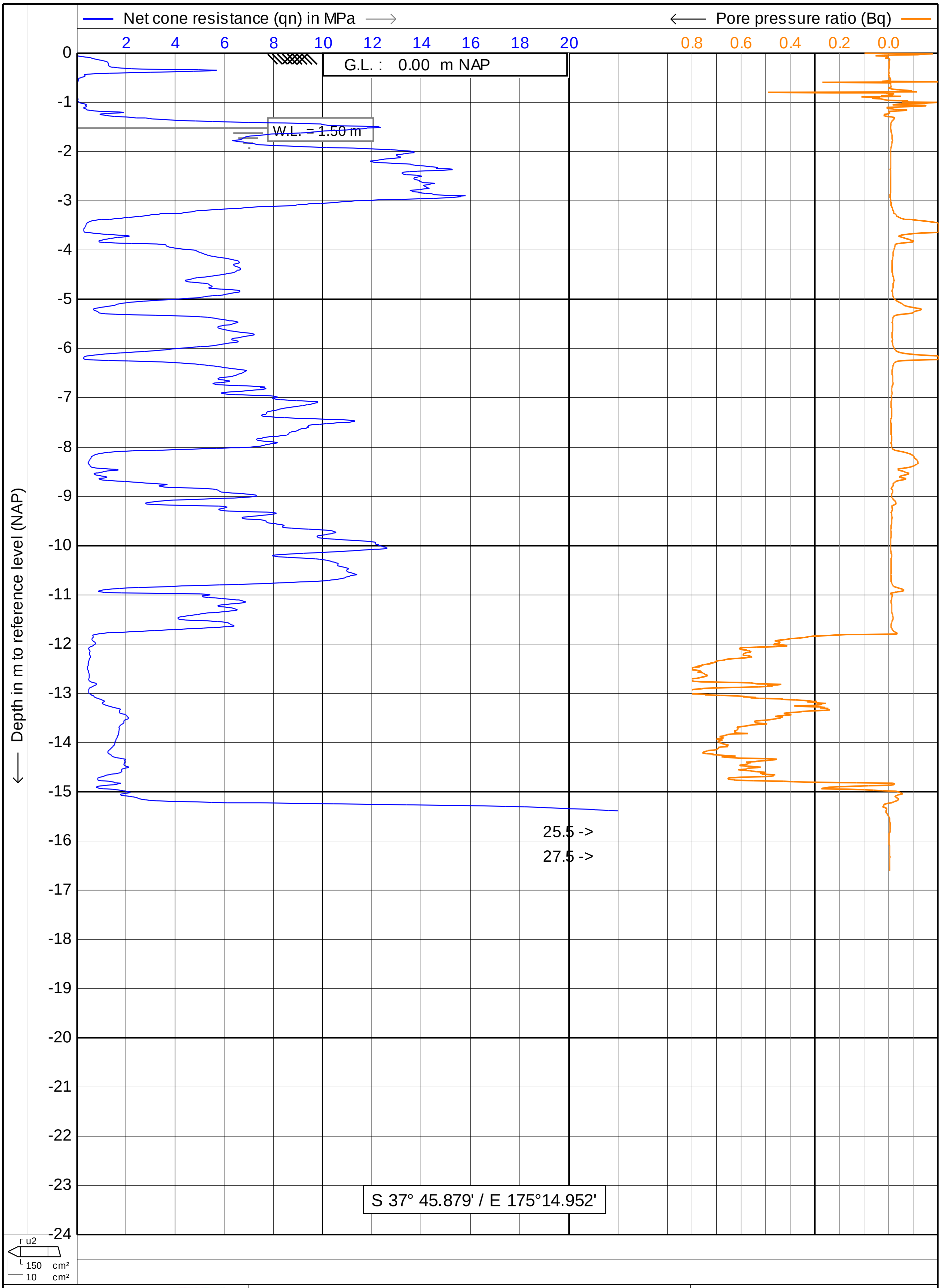


ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT02** **6/15**



CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT02** **7/15**

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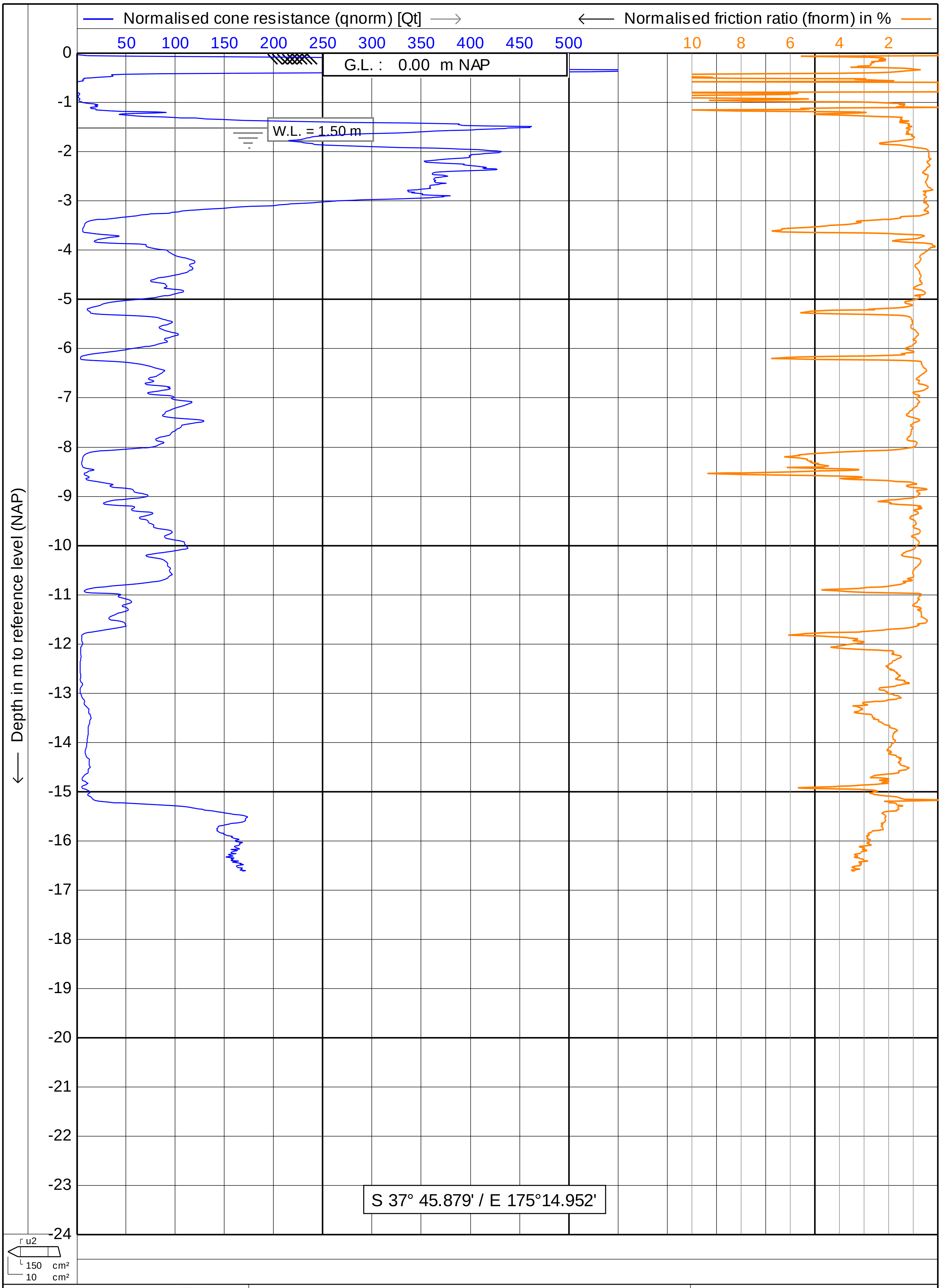


ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

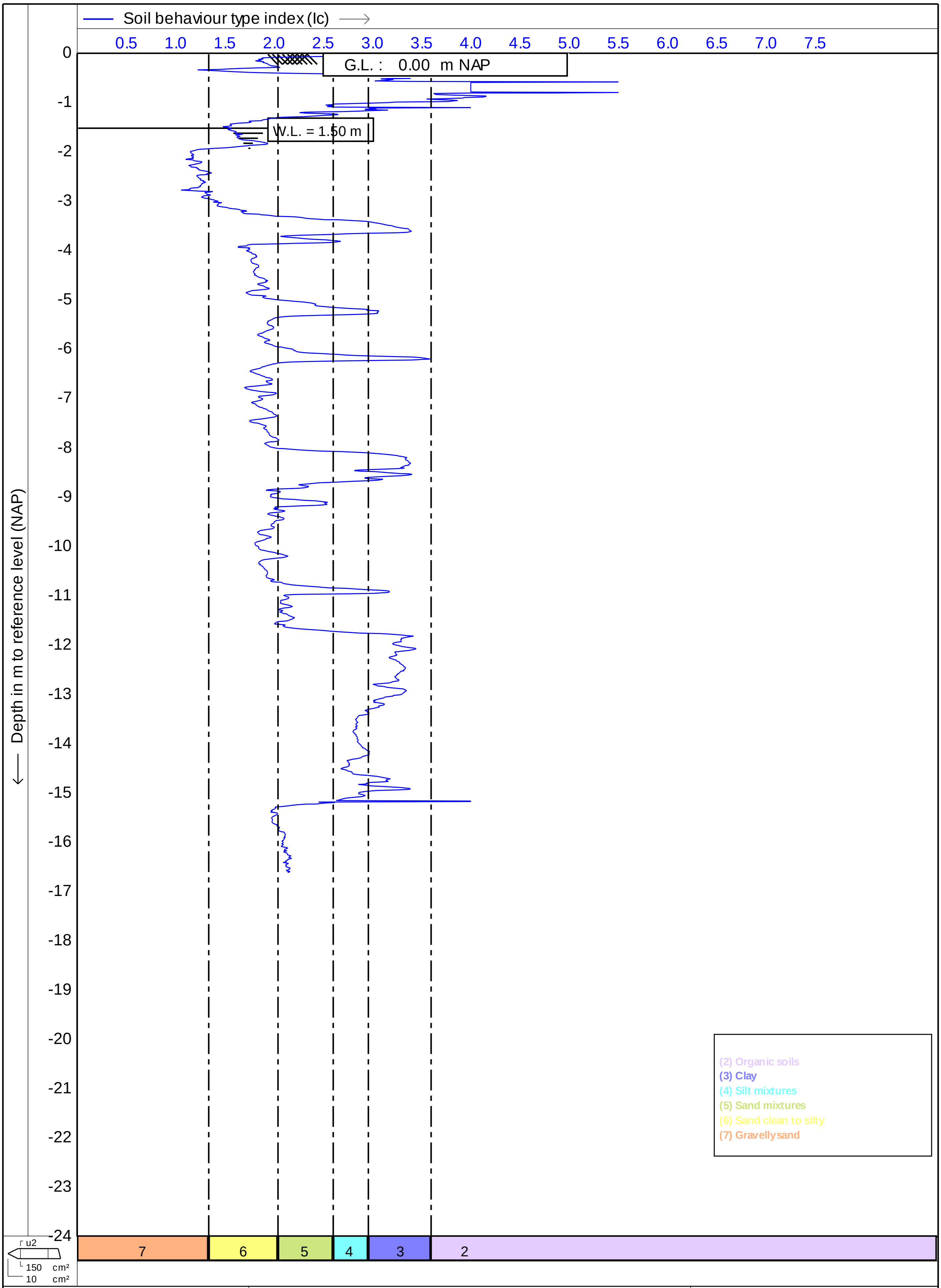
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02** **8/15**

1.47



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CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

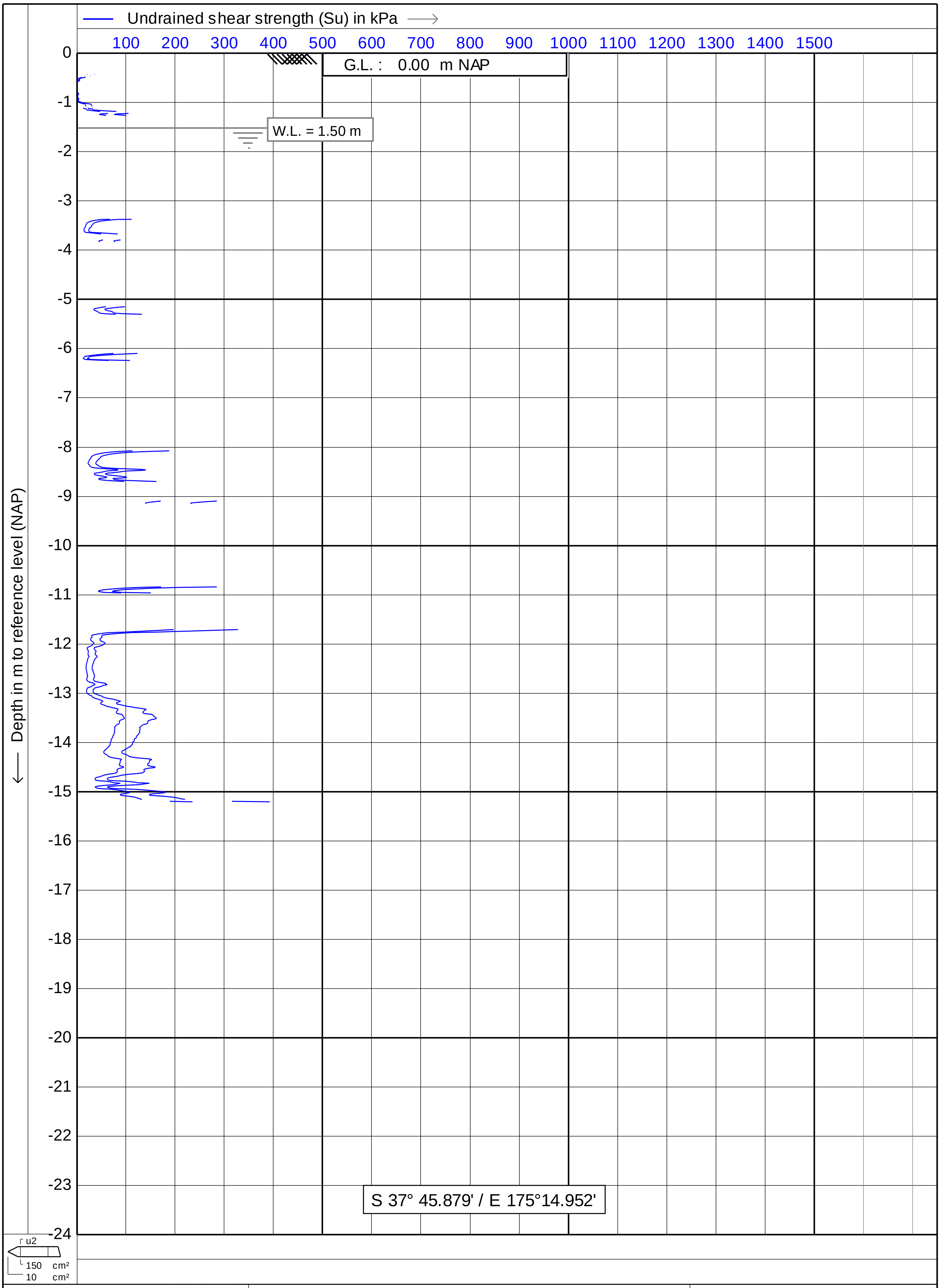
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02**



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CPT it CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

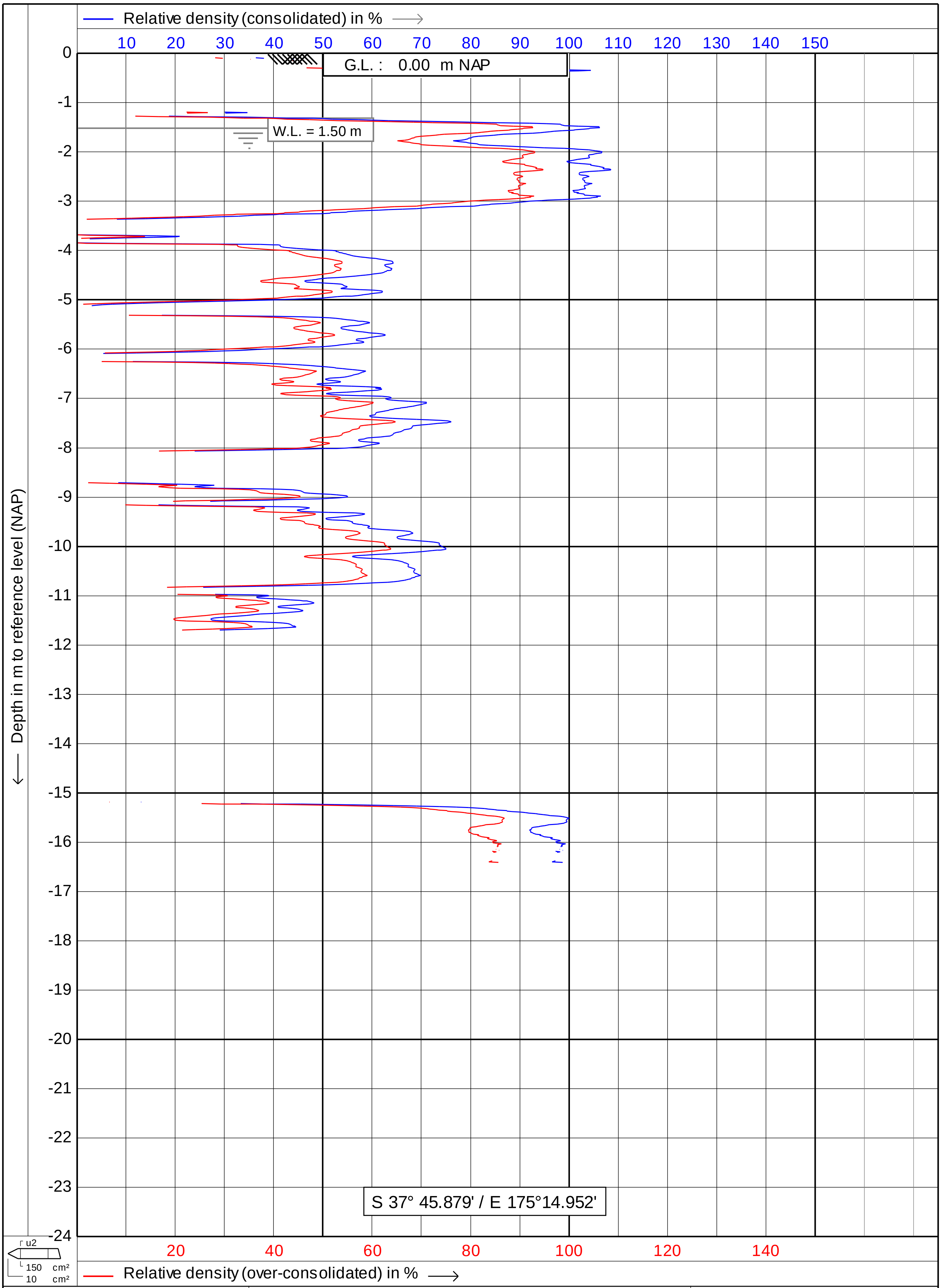
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02**

10/15



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02**

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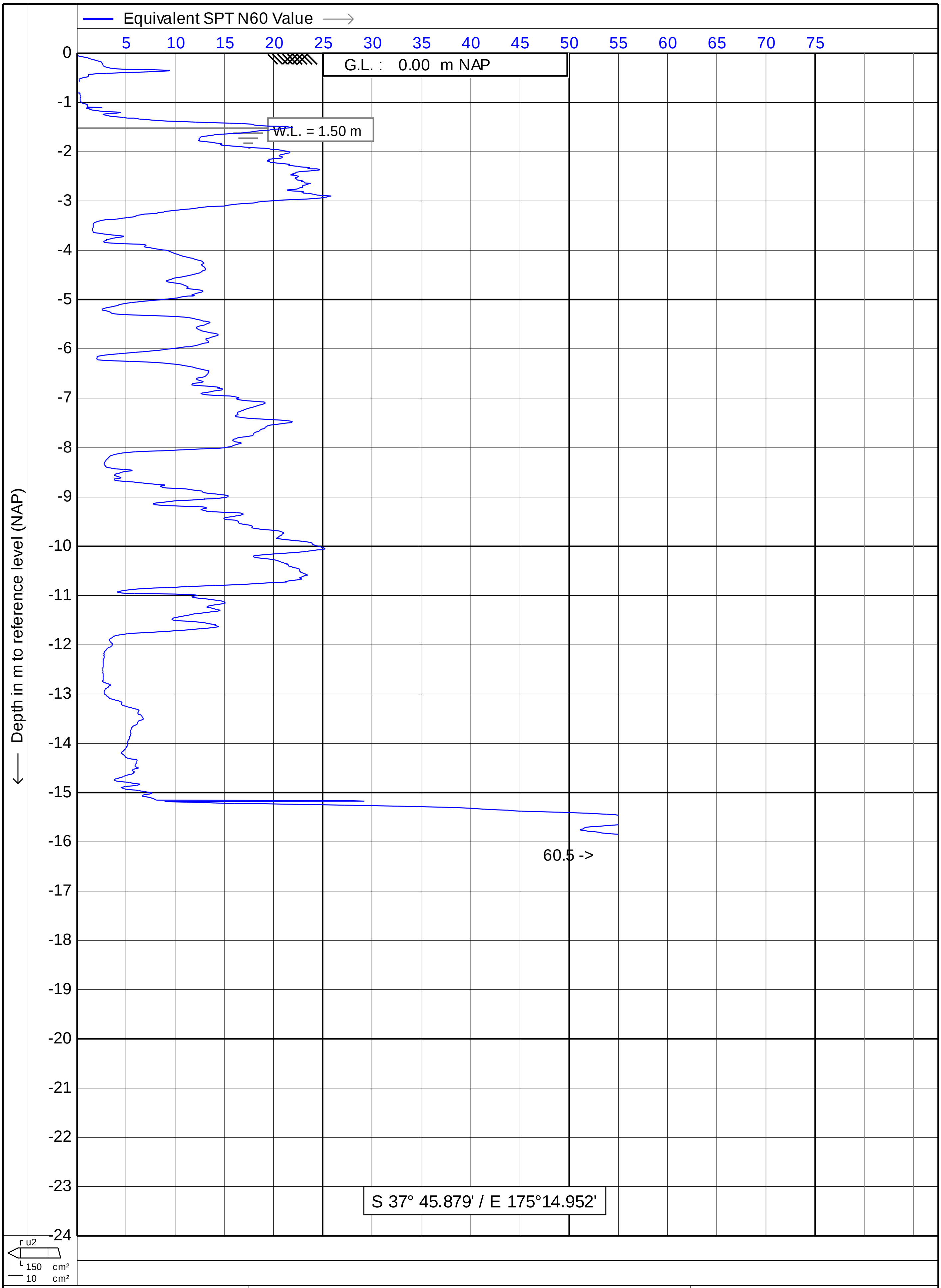
1.47



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CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

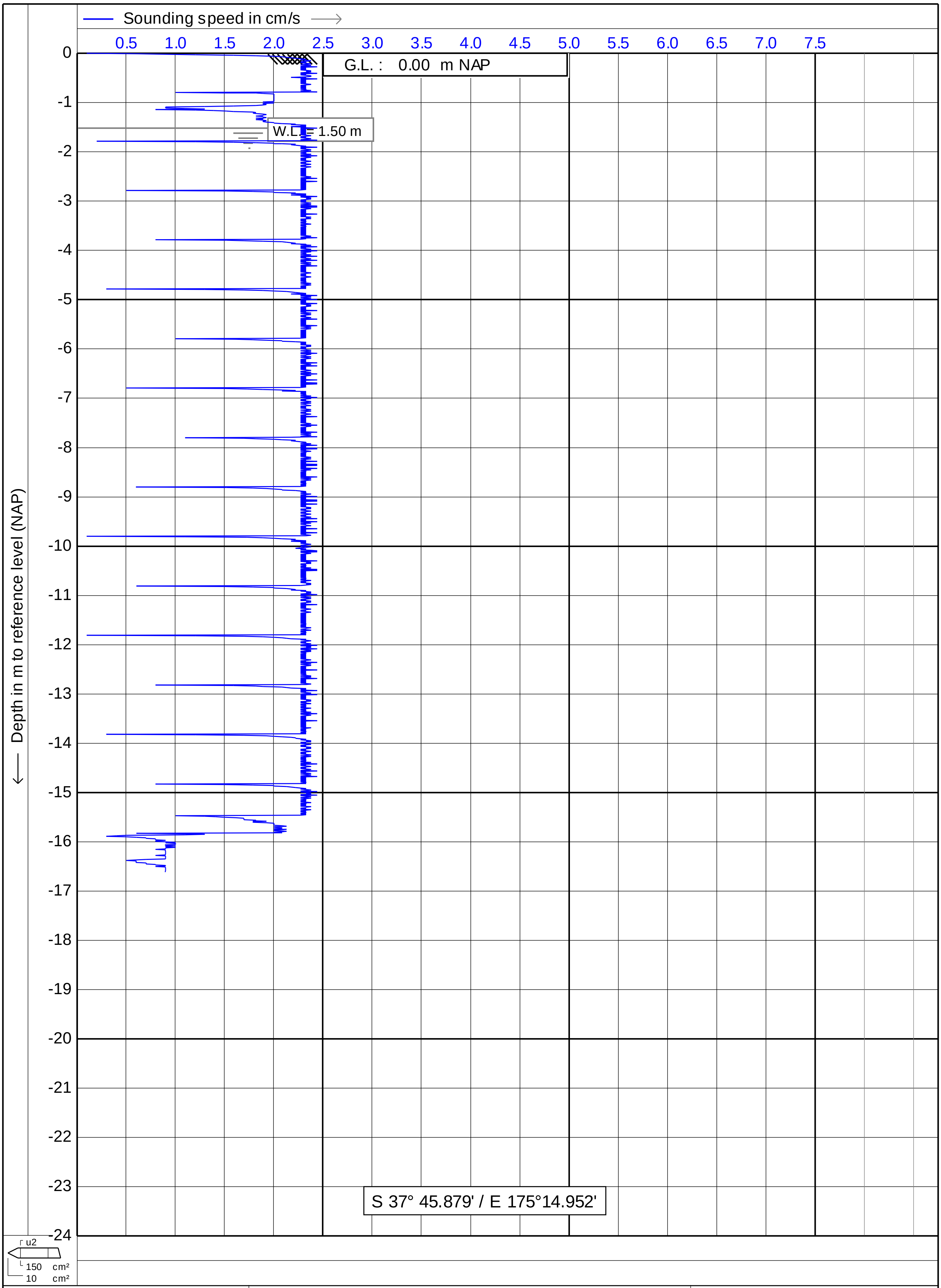
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT02**

12/15



CONE PENETROMETER TESTING

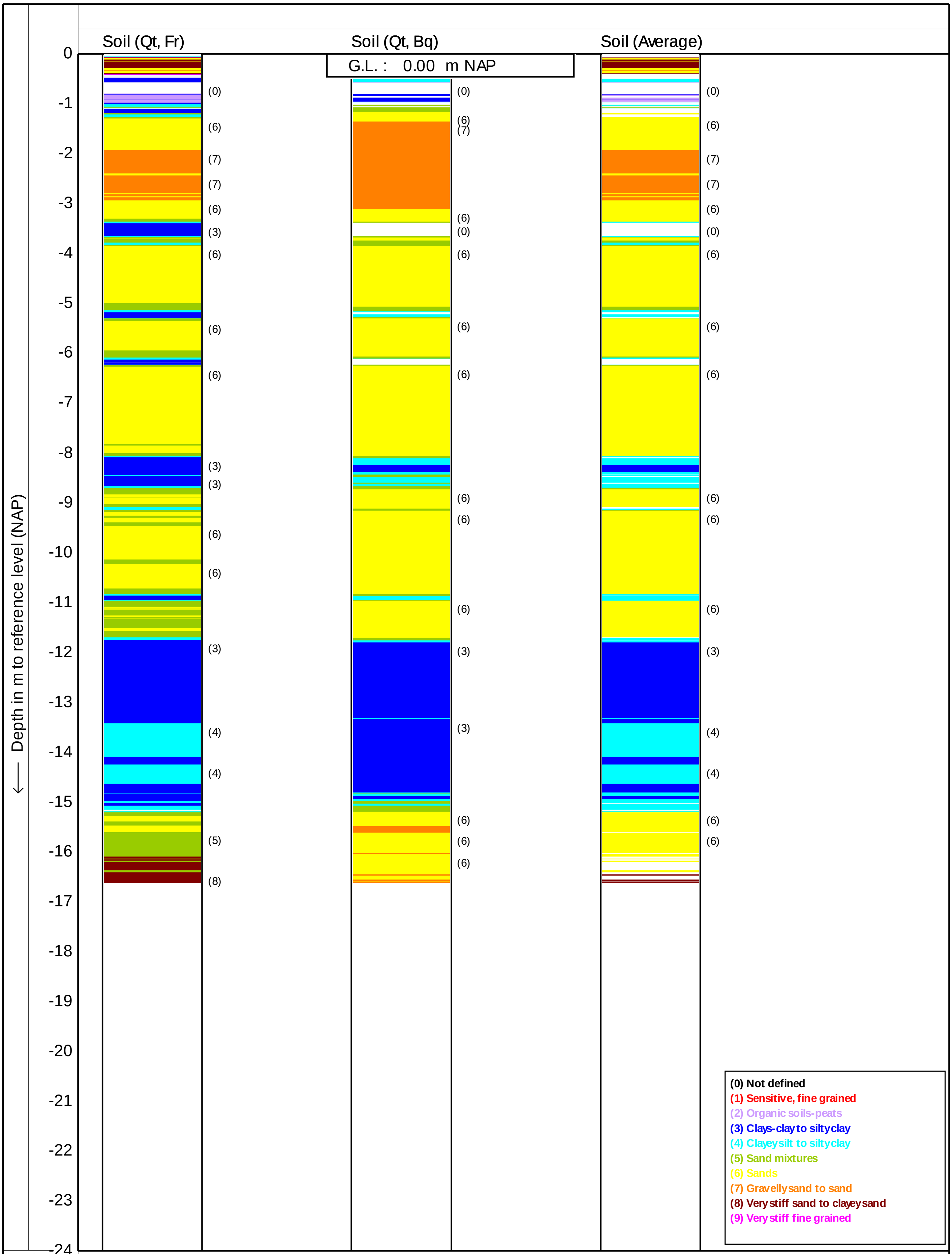
ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT02**

13/15

1.47



Soil behaviour type classification after Robertson 1990

CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT02**

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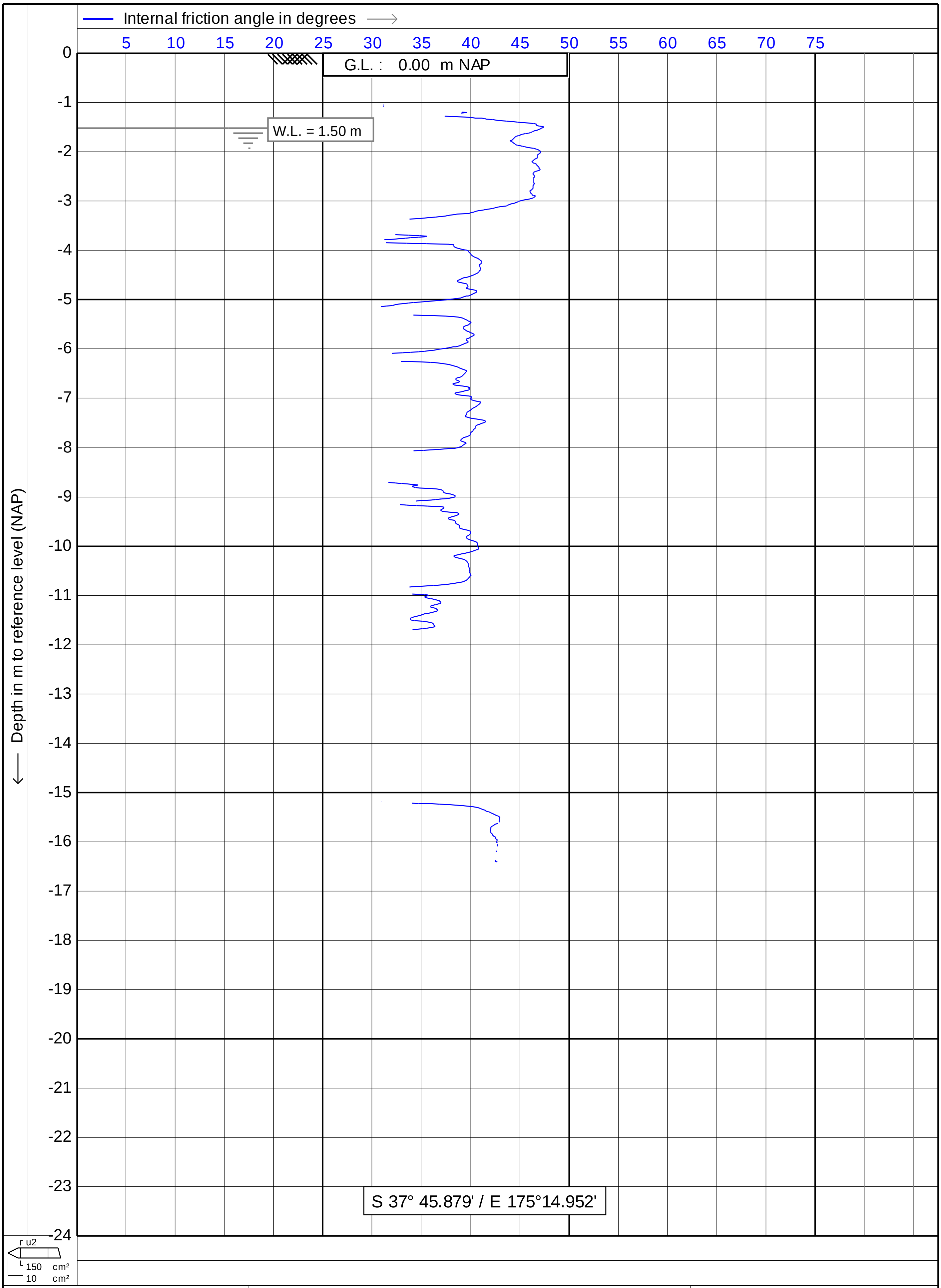
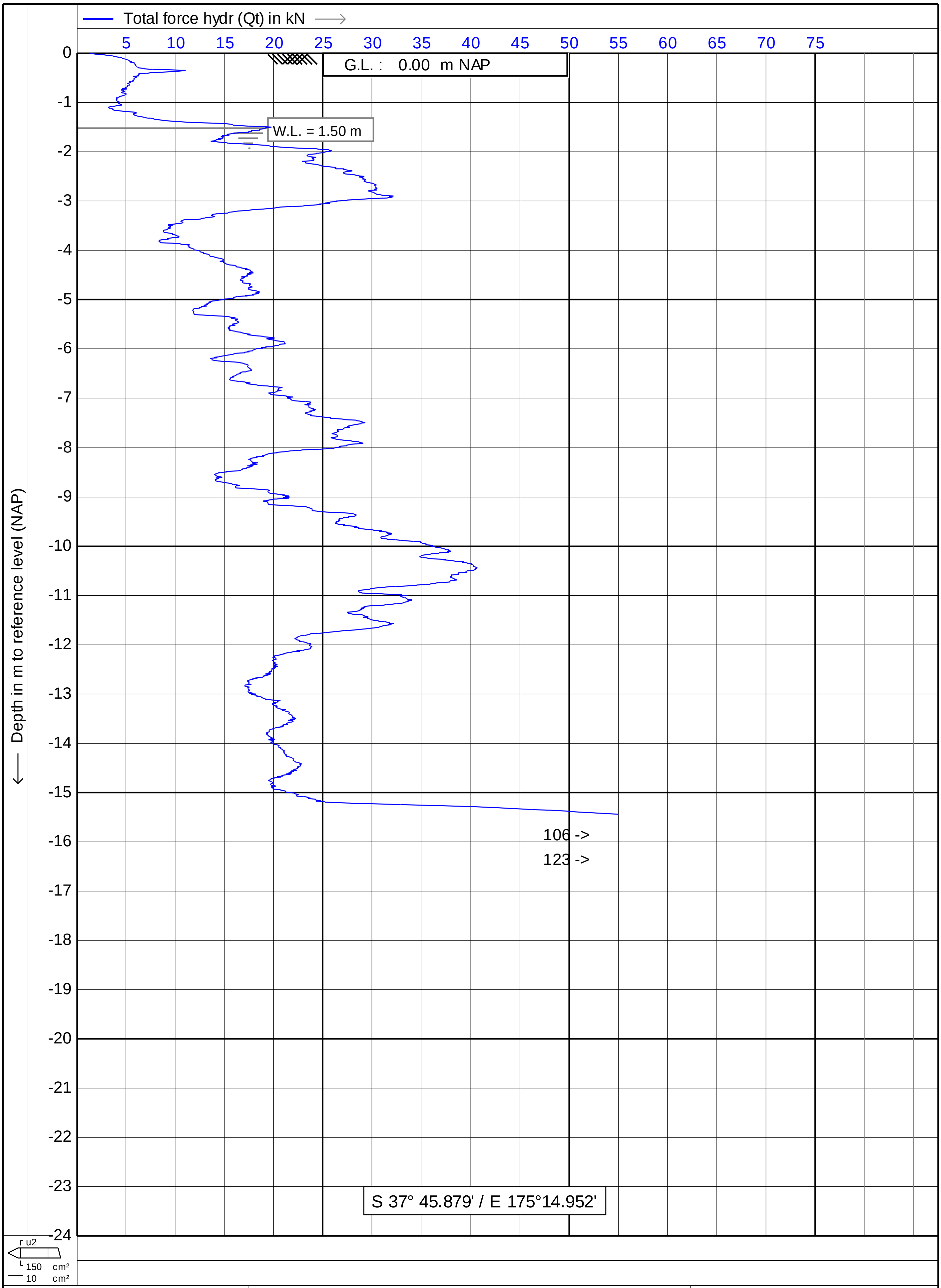


ISO 22476-1:2012 Application class 1 Test type TE1	Date : **23-May-17**
Project : **TE RAPA RACECOURSE DEVELOPMENT**	Cone no. : **S10CFIIP.S16082**
Location: **TE RAPA RACECOURSE**	Project no. : **17017/HAM2017-109**
Position: **0, 0**	CPT no. : **CPT02**

15/15

CPT it CONE PENETROMETER TESTING

1.47



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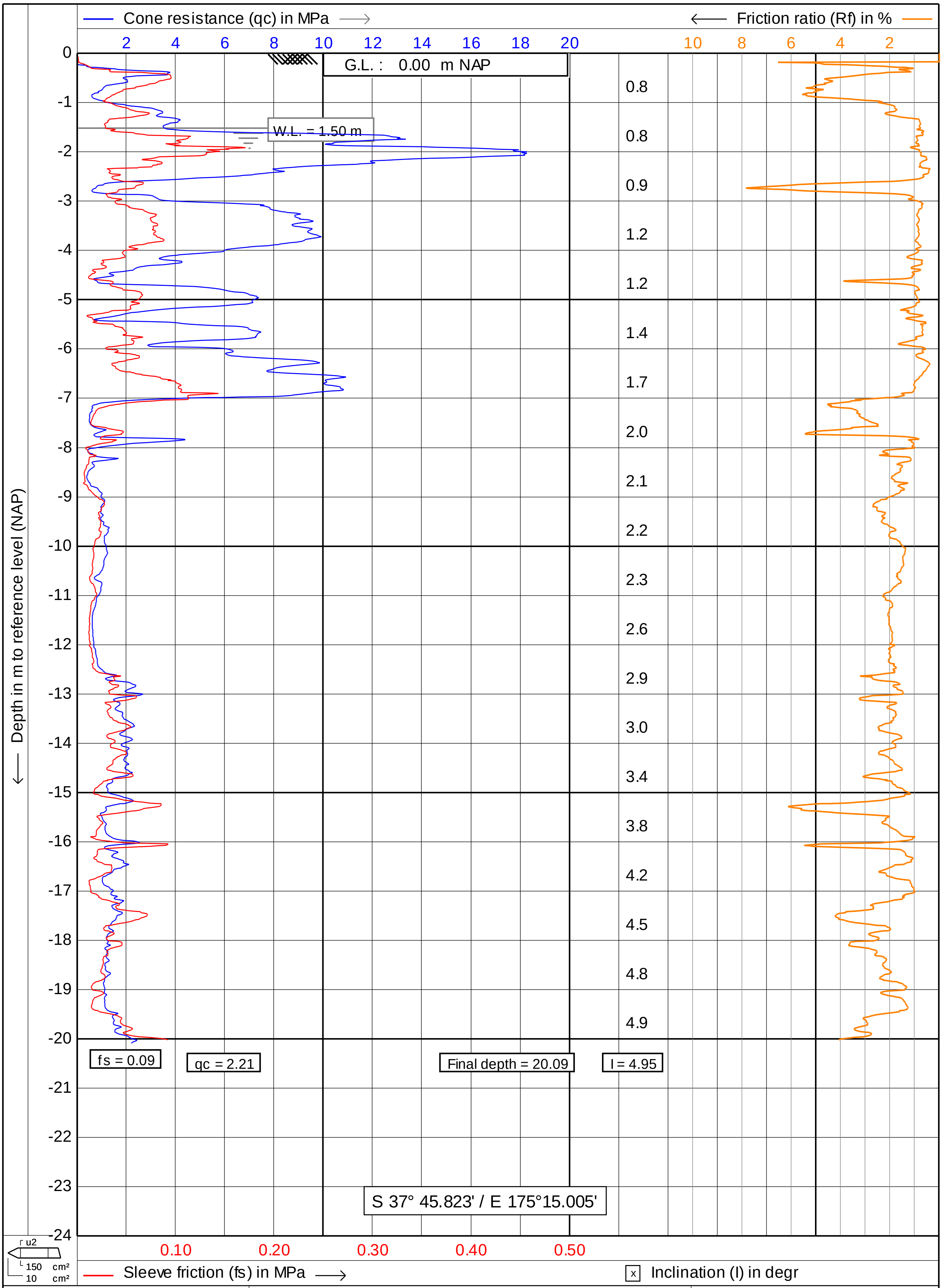
ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT02** **16/15**



CONE PENETROMETER TESTING

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ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

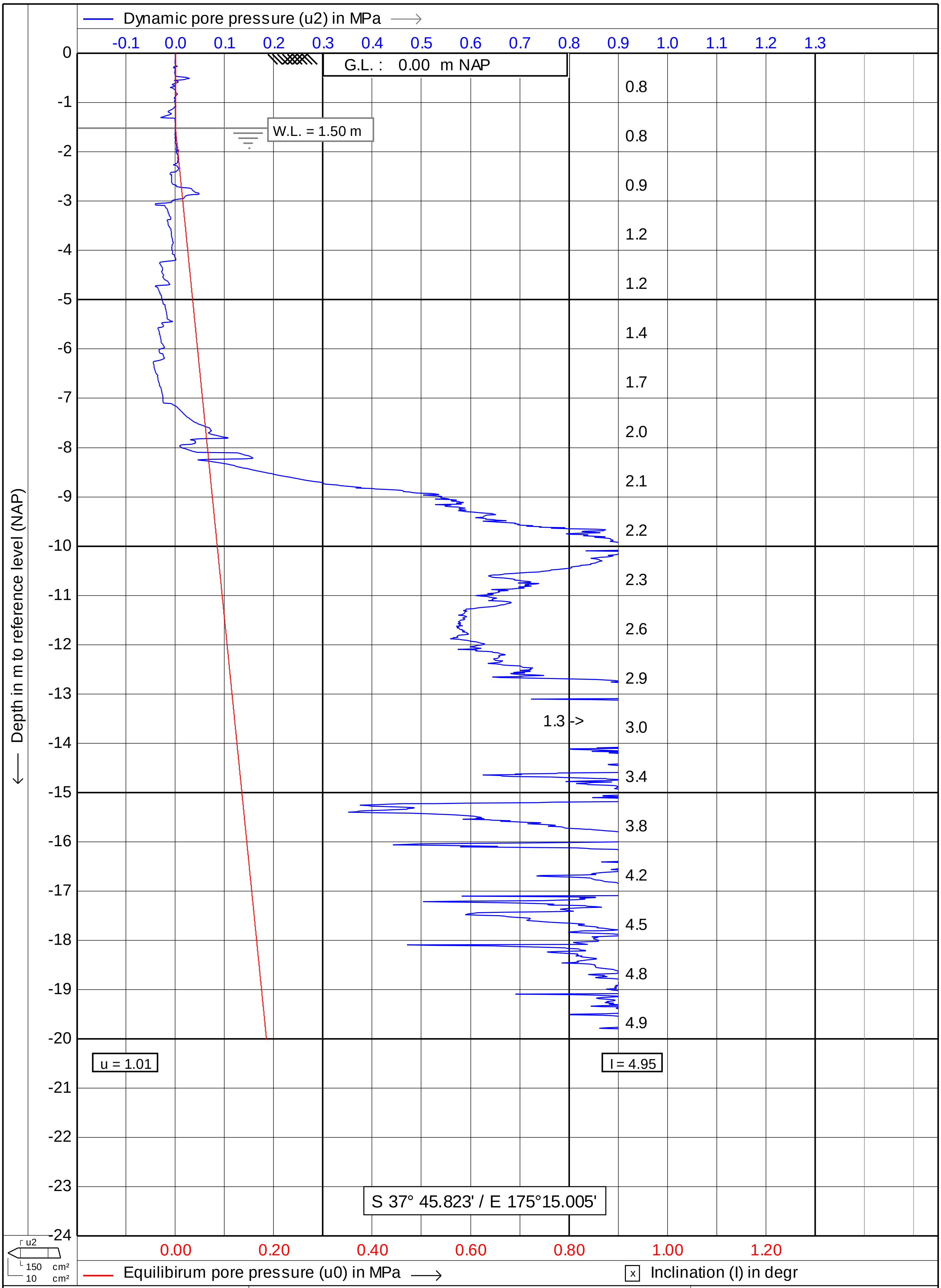
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03** **1/15**



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CPT it CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

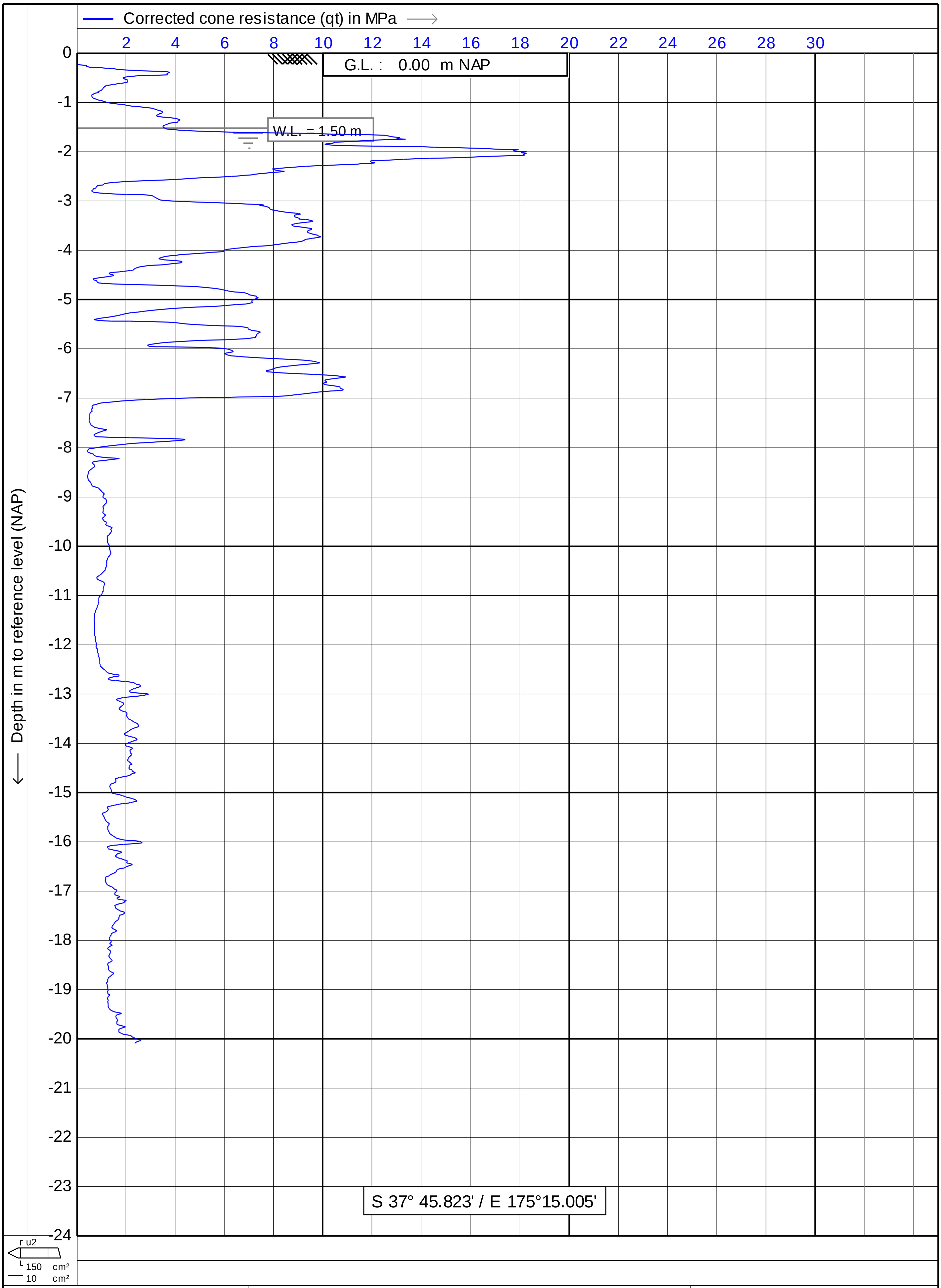
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03**

2/15



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CONE PENETROMETER TESTING

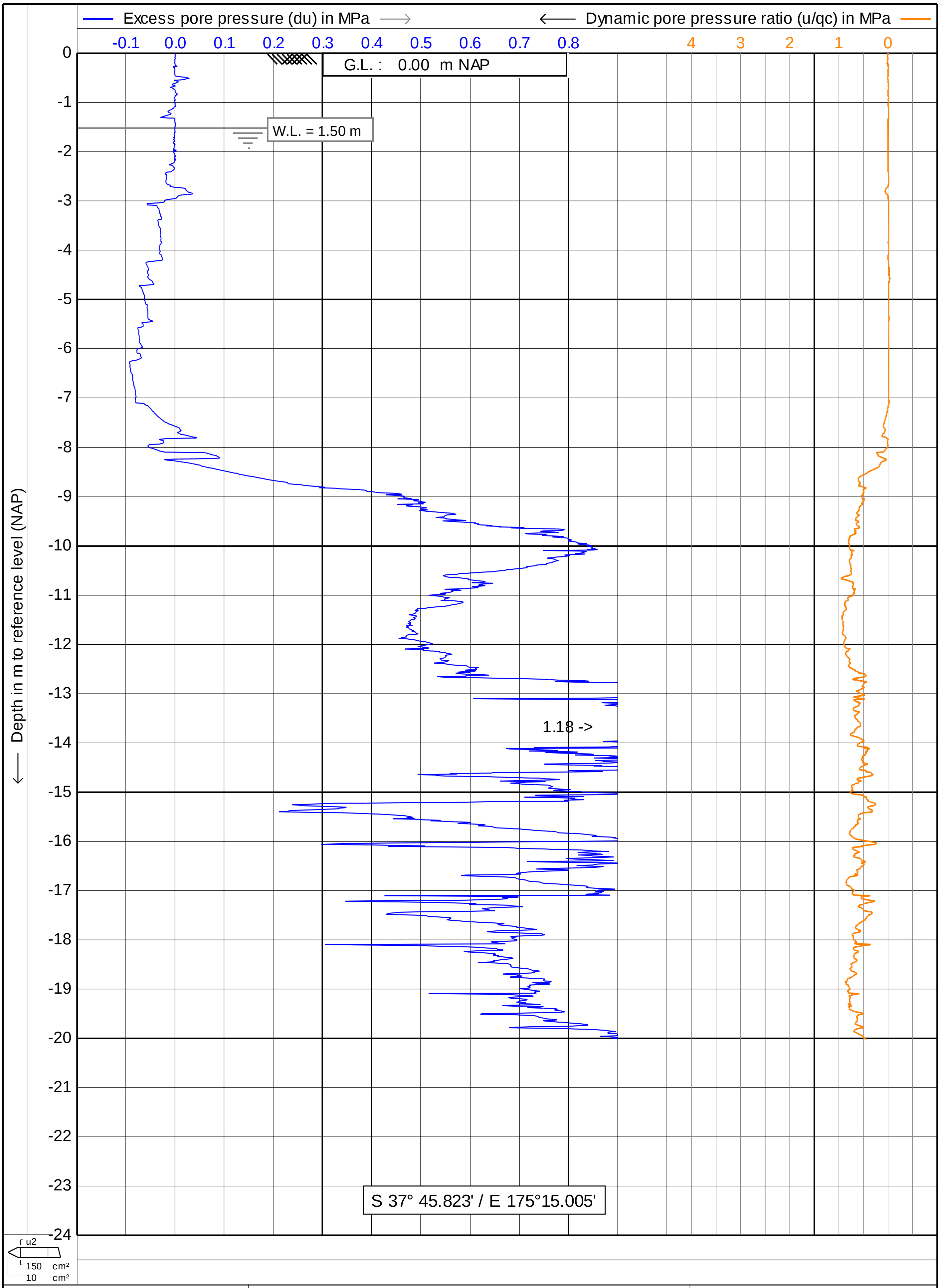
ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT03** **3/15**



CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03** **4/15**

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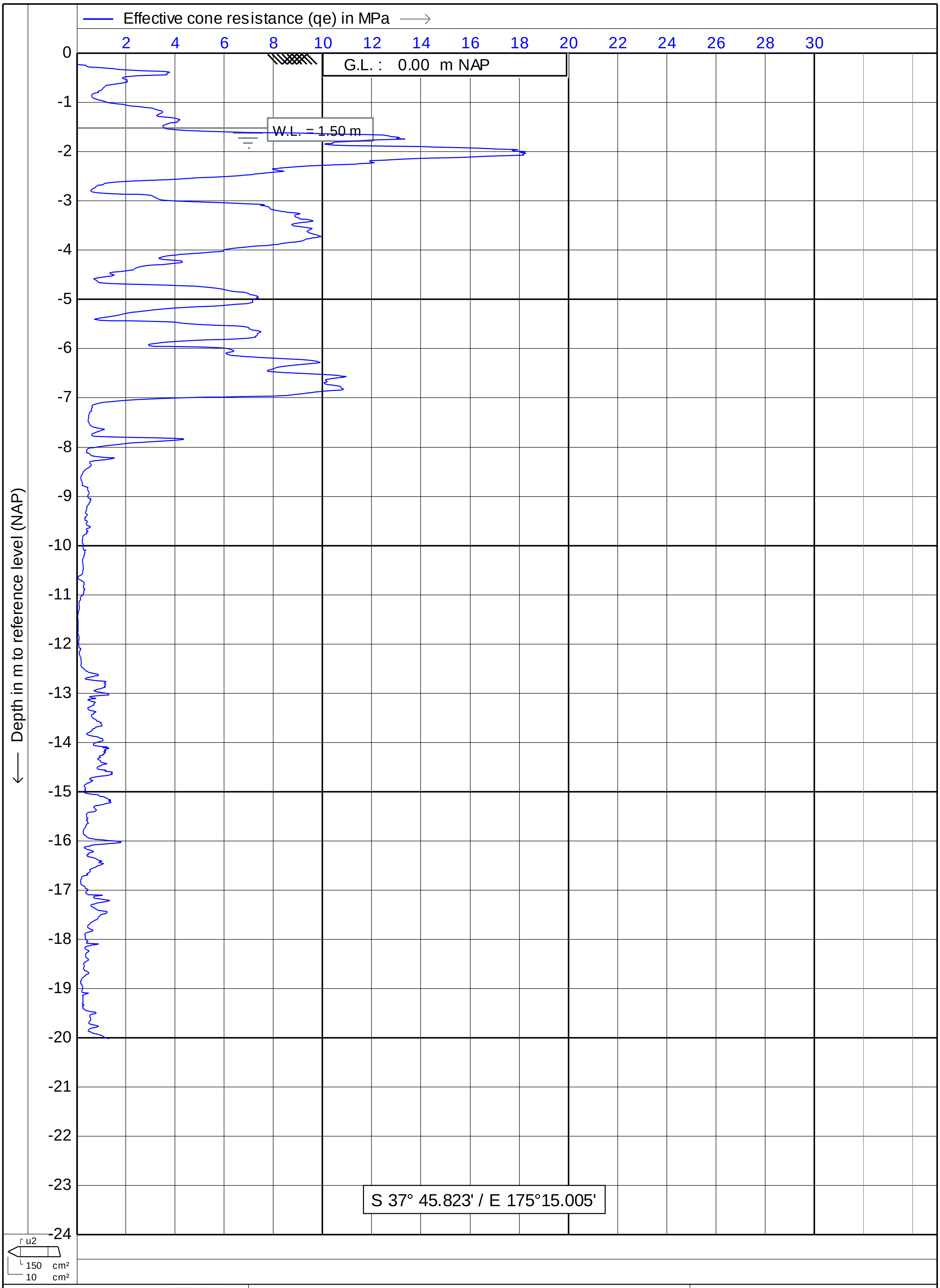

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03** **5/15**

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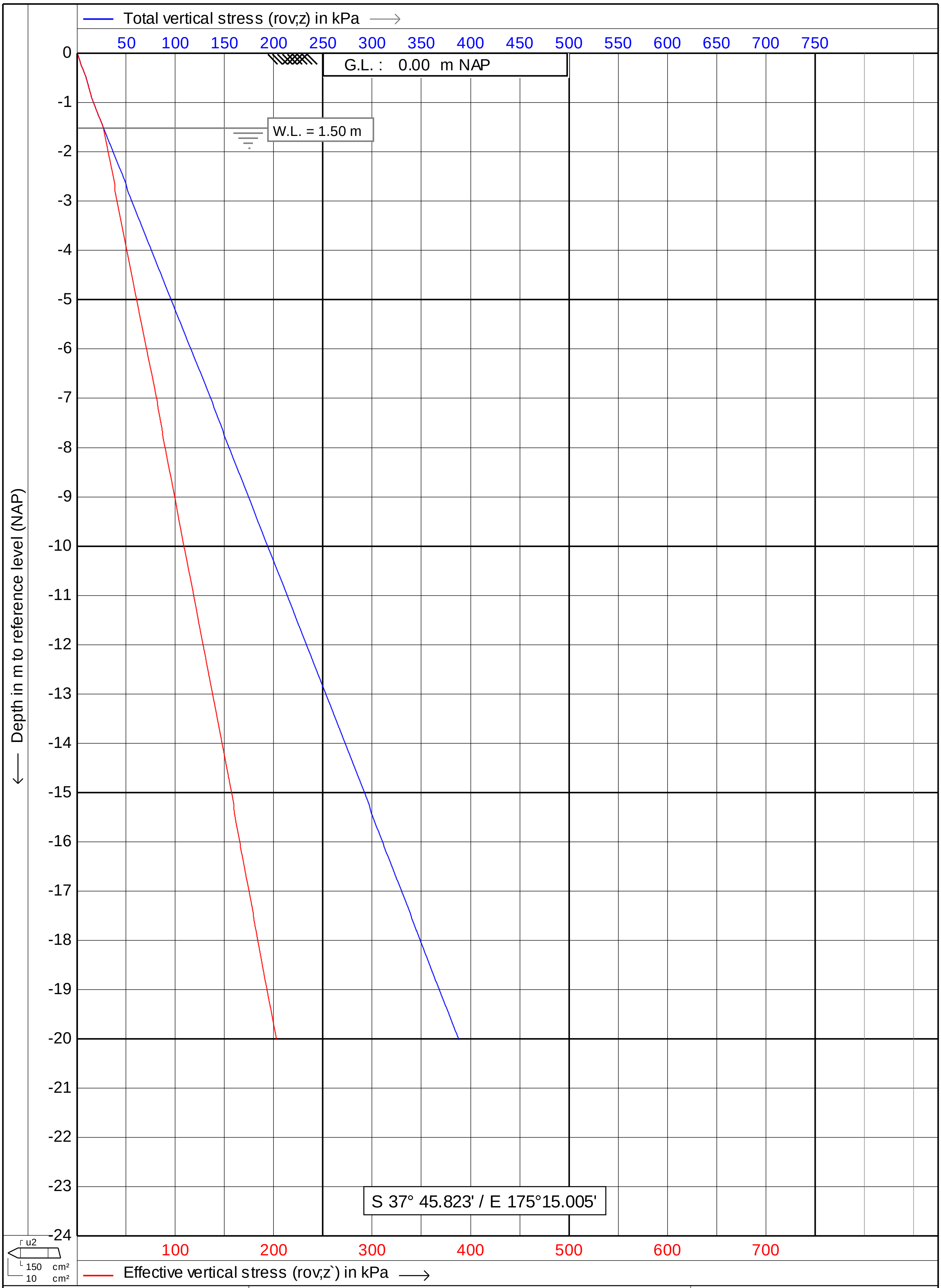

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT03**

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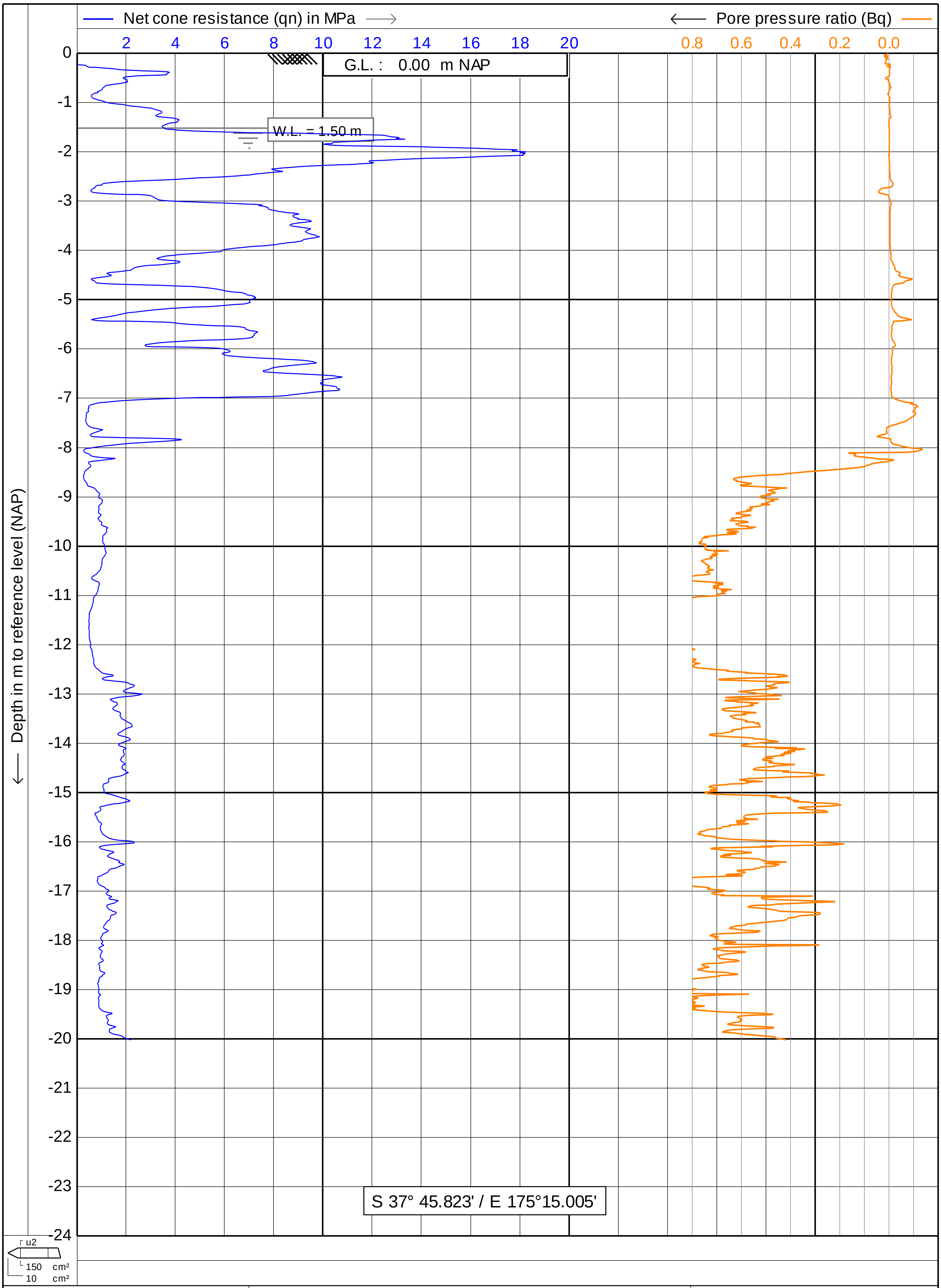

CPT it
CONE PENETROMETER TESTING

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ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

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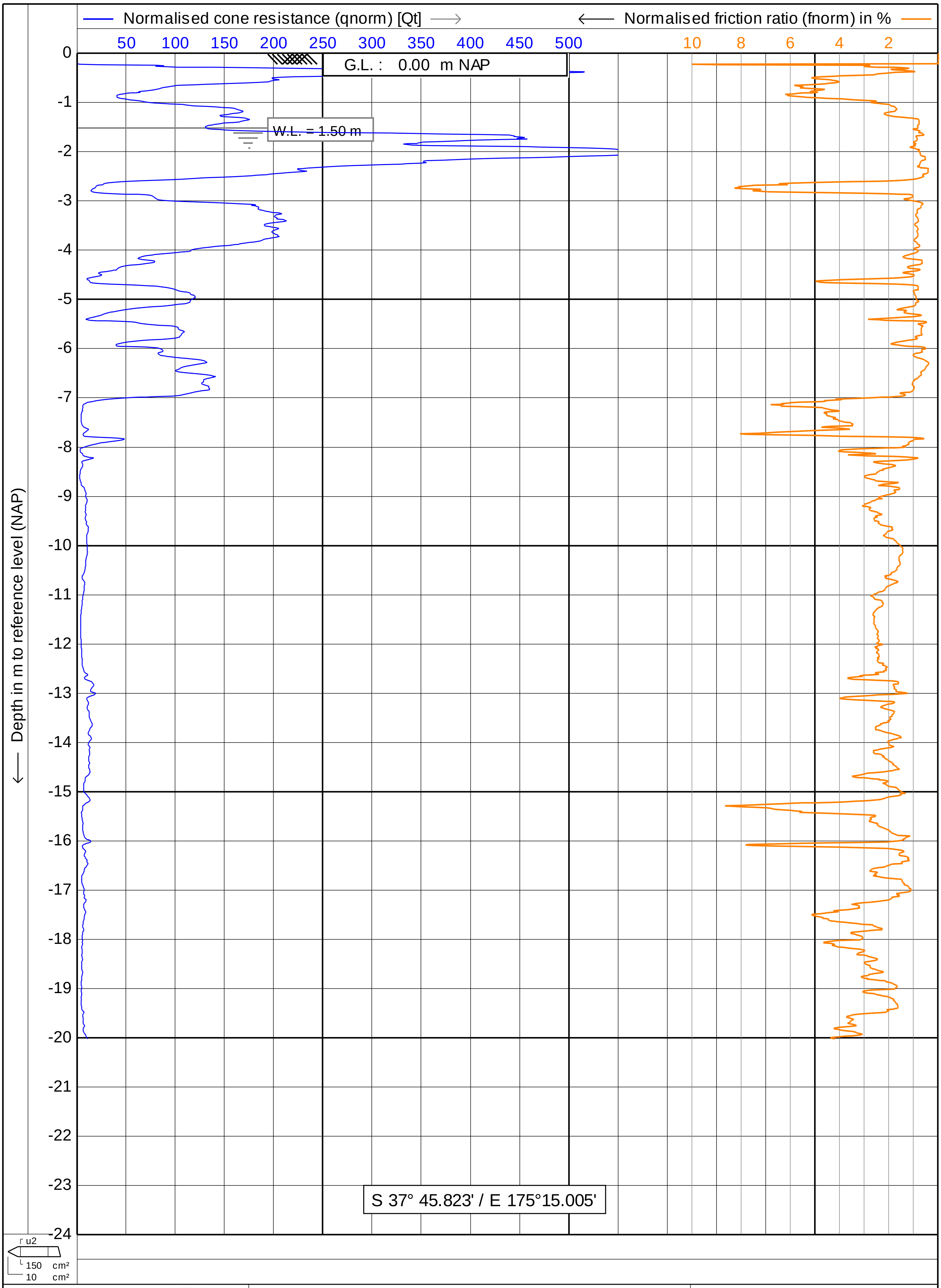
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03** **7/15**



CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

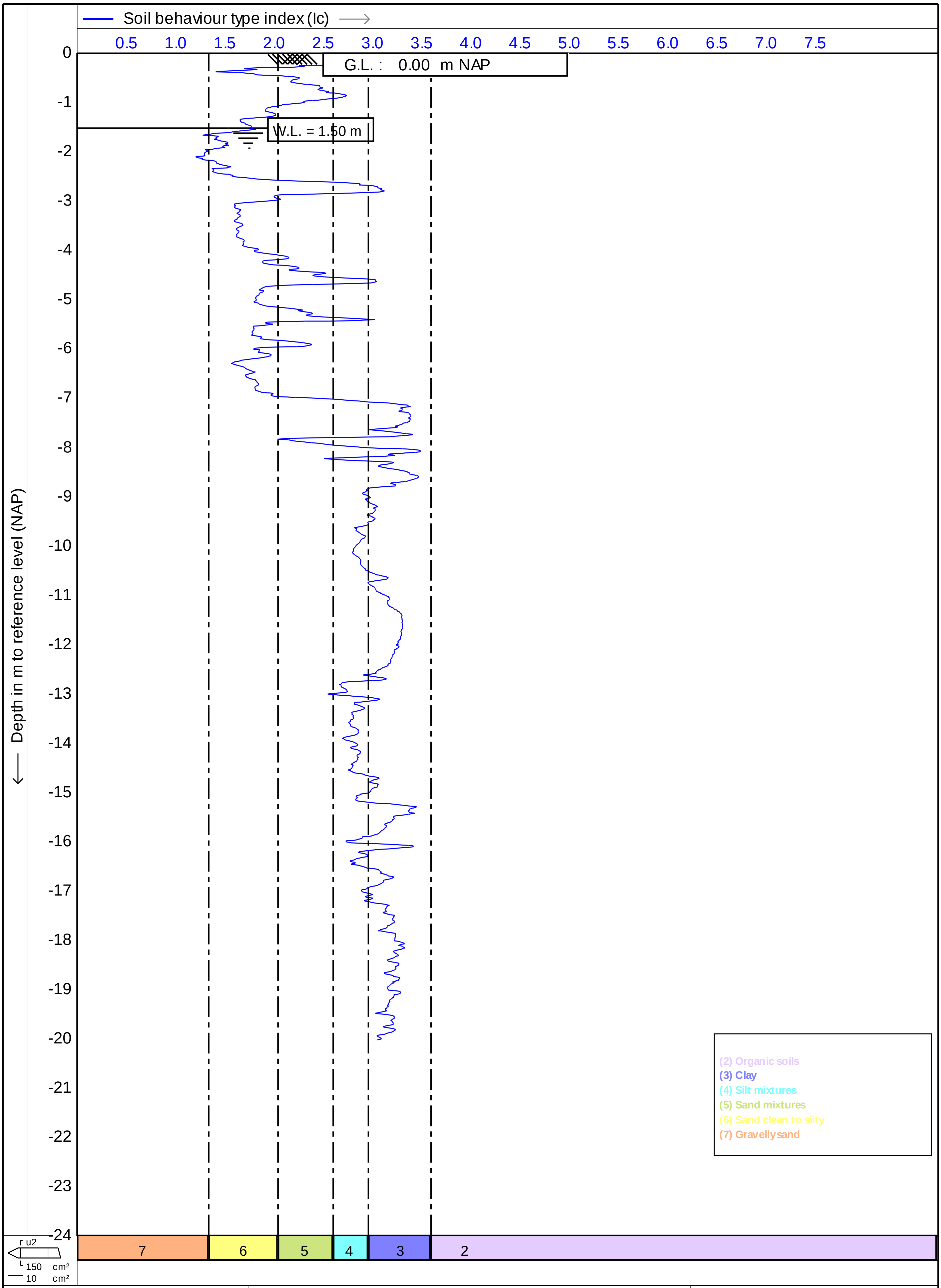
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03** **8/15**

1.47



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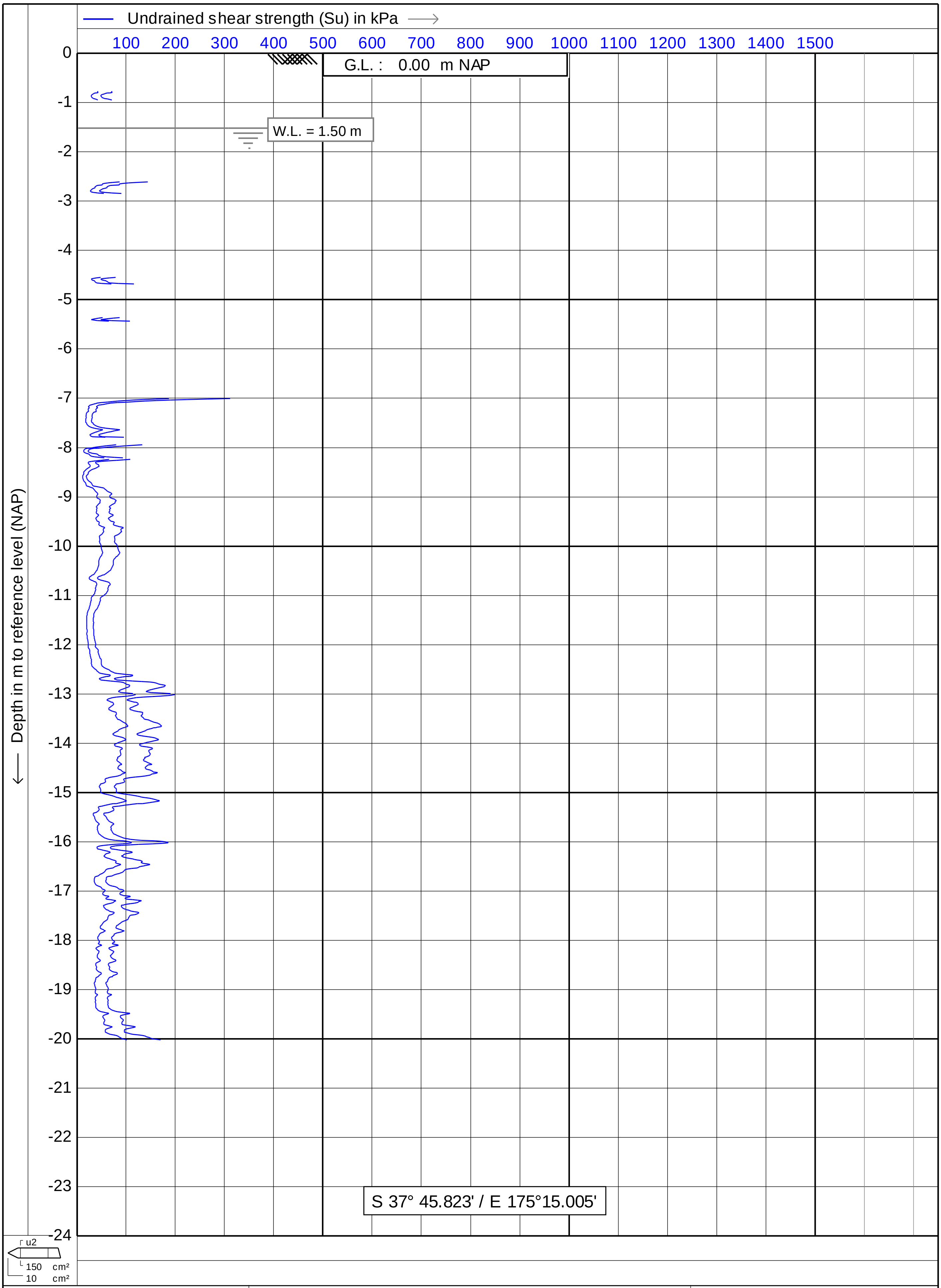
CPT it
CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT03** **9/15**



CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03**

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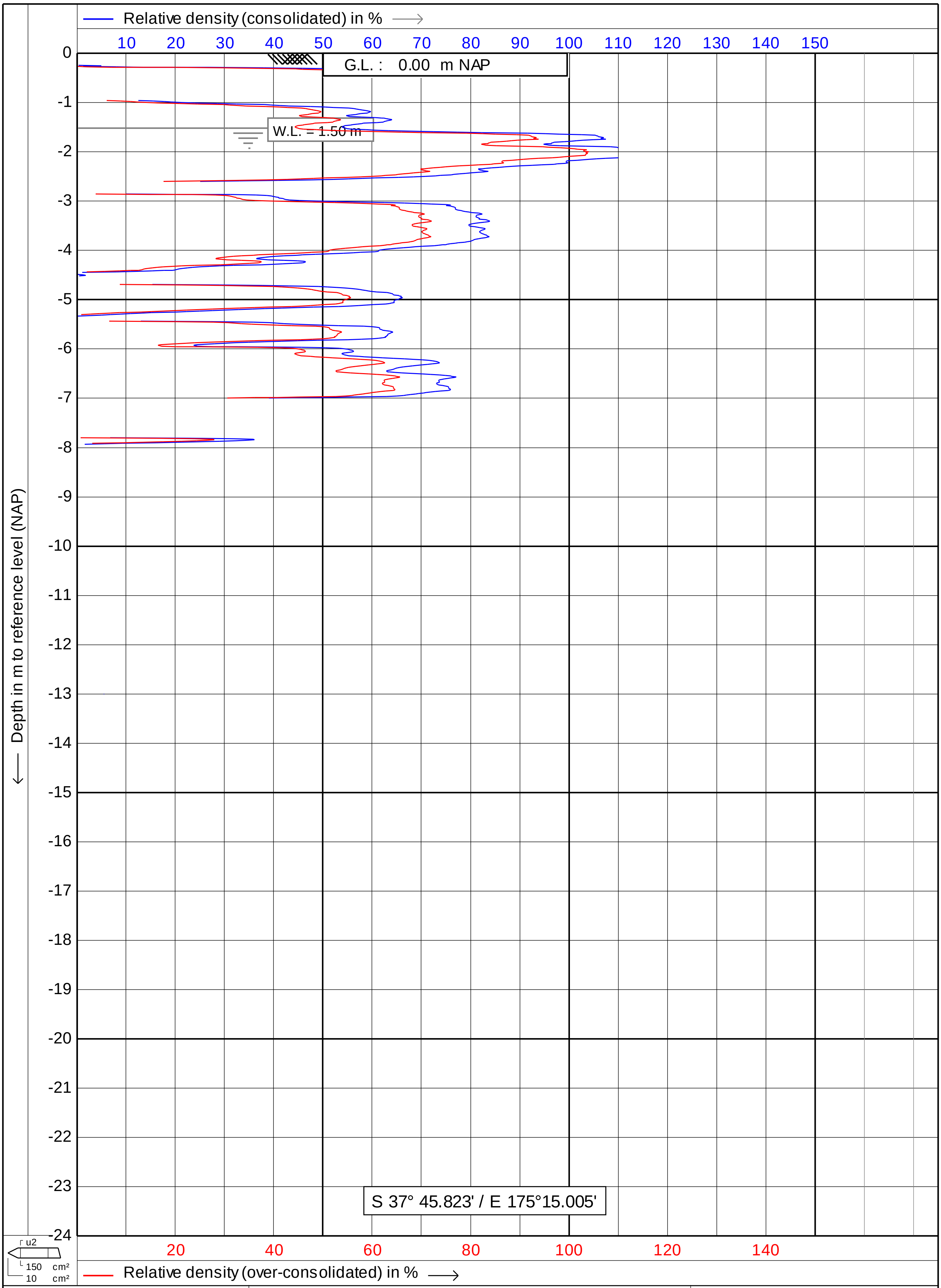

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CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

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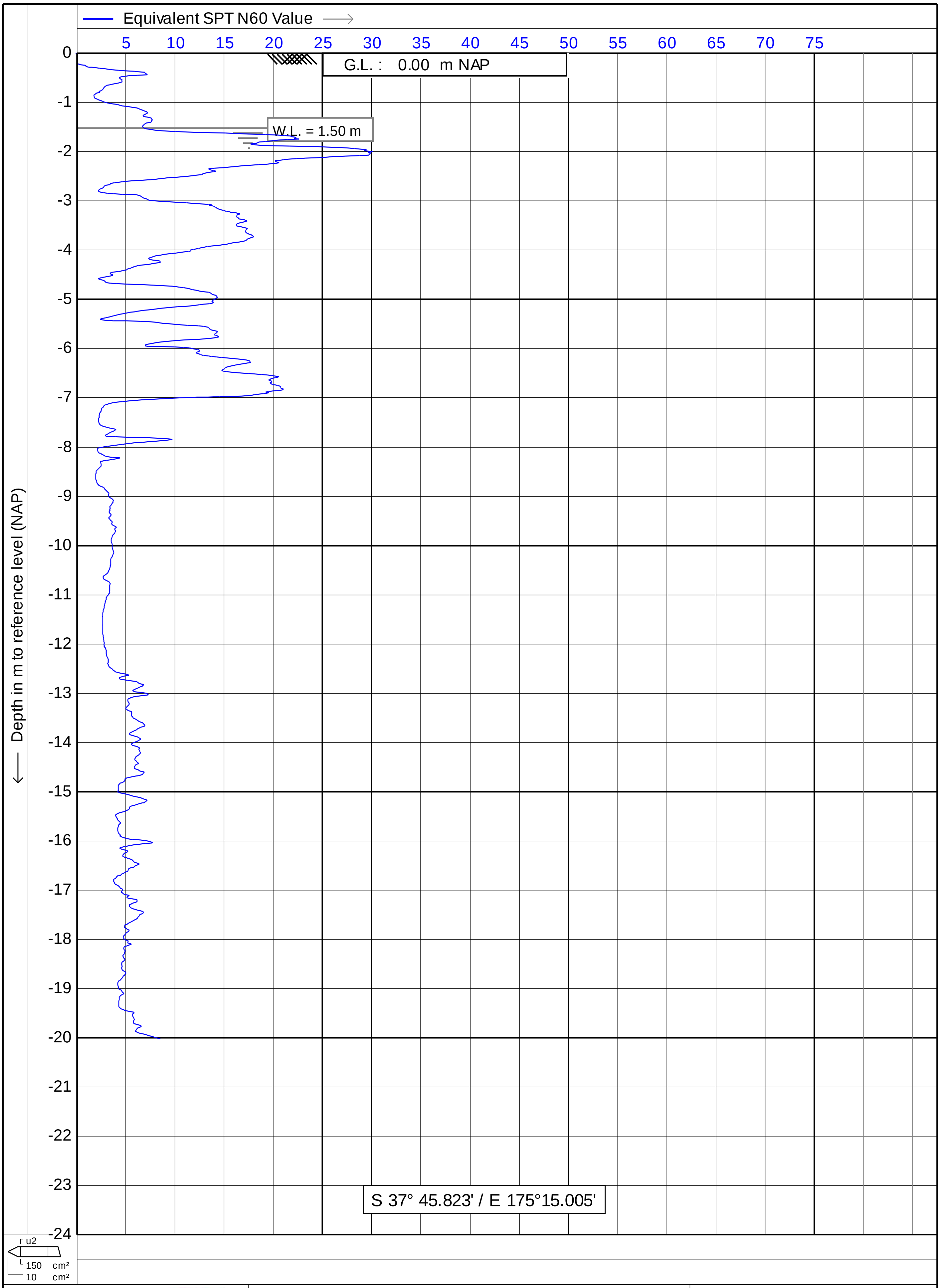
Project no. : **17017/HAM2017-109**

CPT no. : **CPT03**

11/15



CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

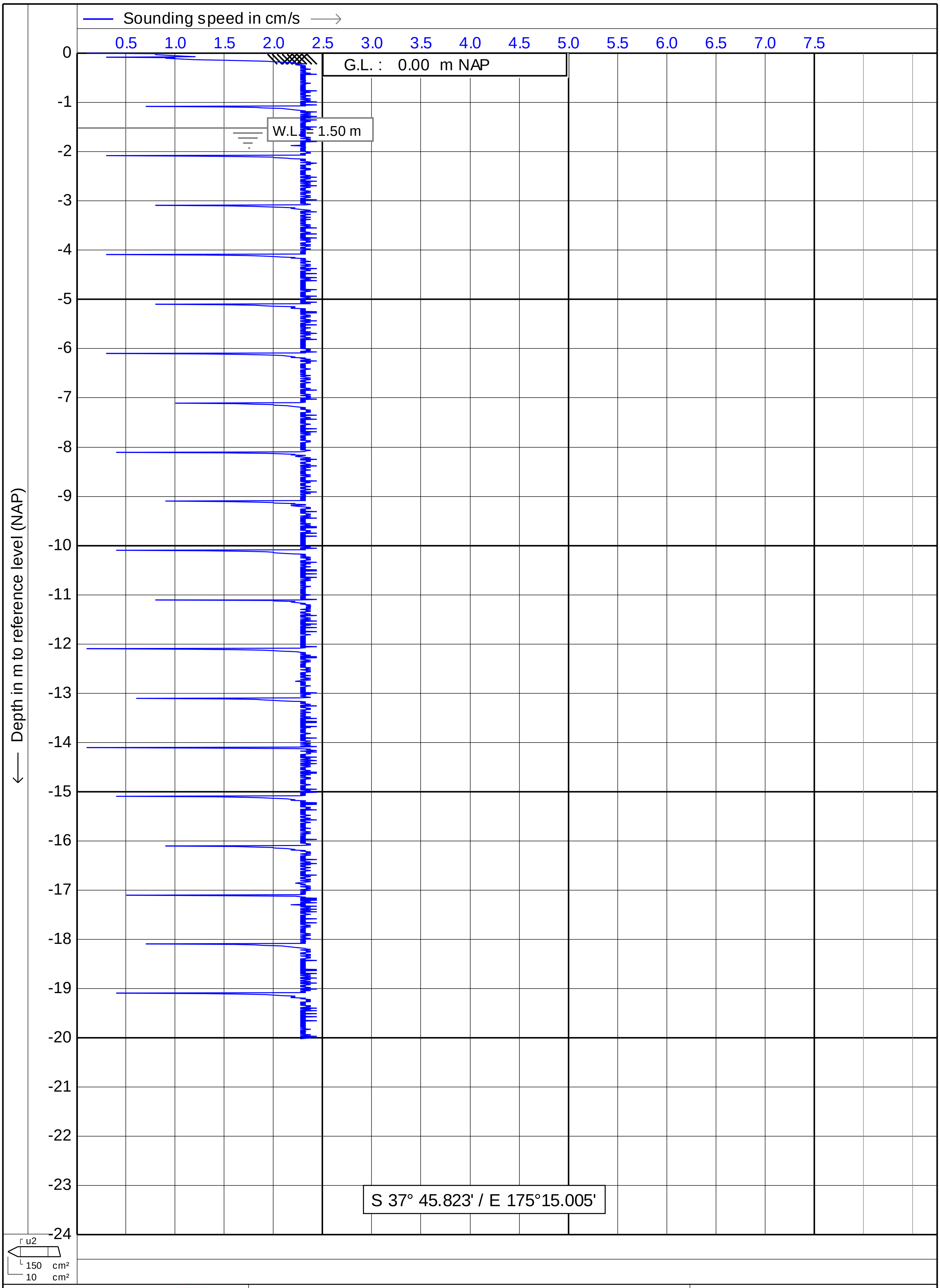
Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03**

12/15

1.47



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CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

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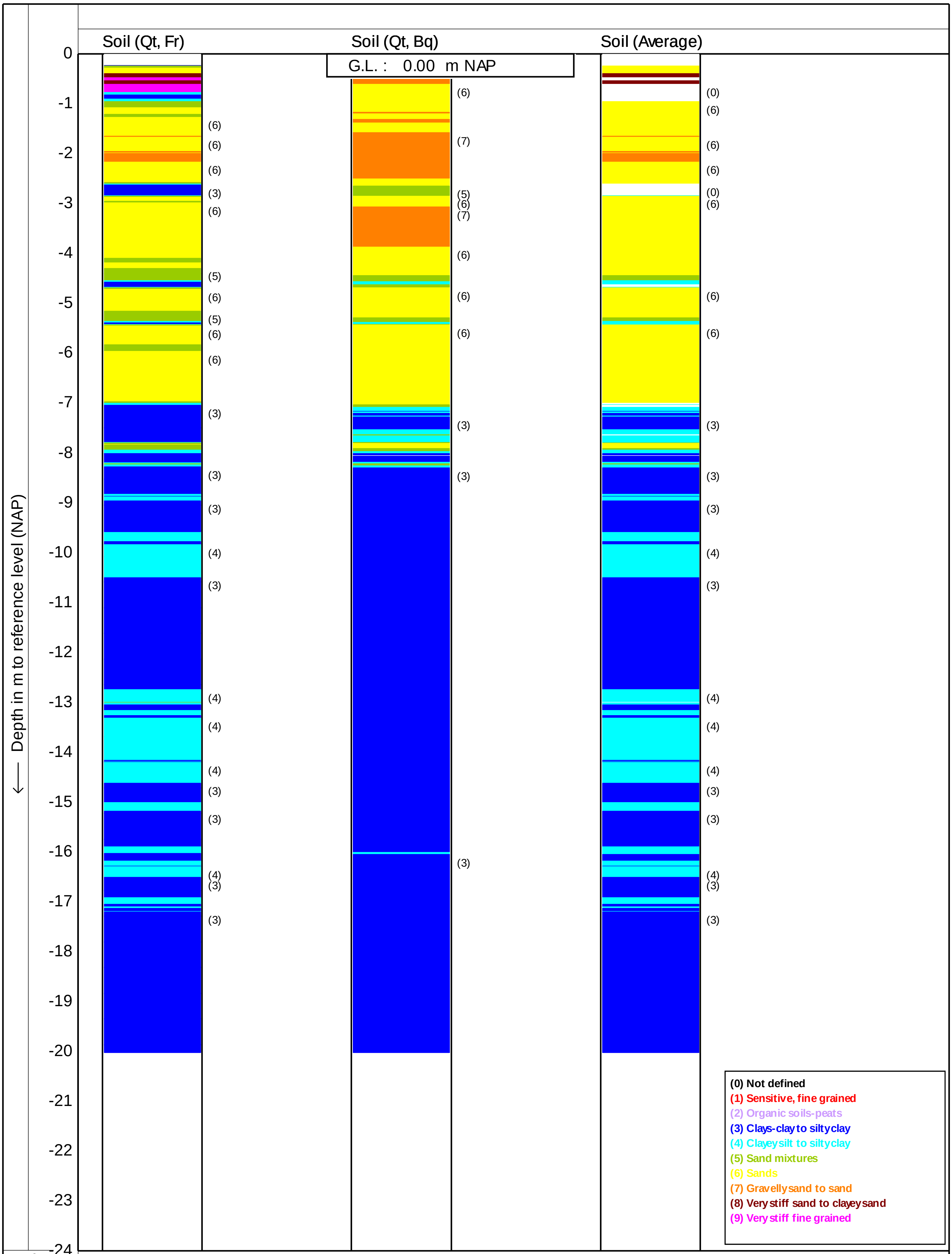
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT03**



Soil behaviour type classification after Robertson 1990

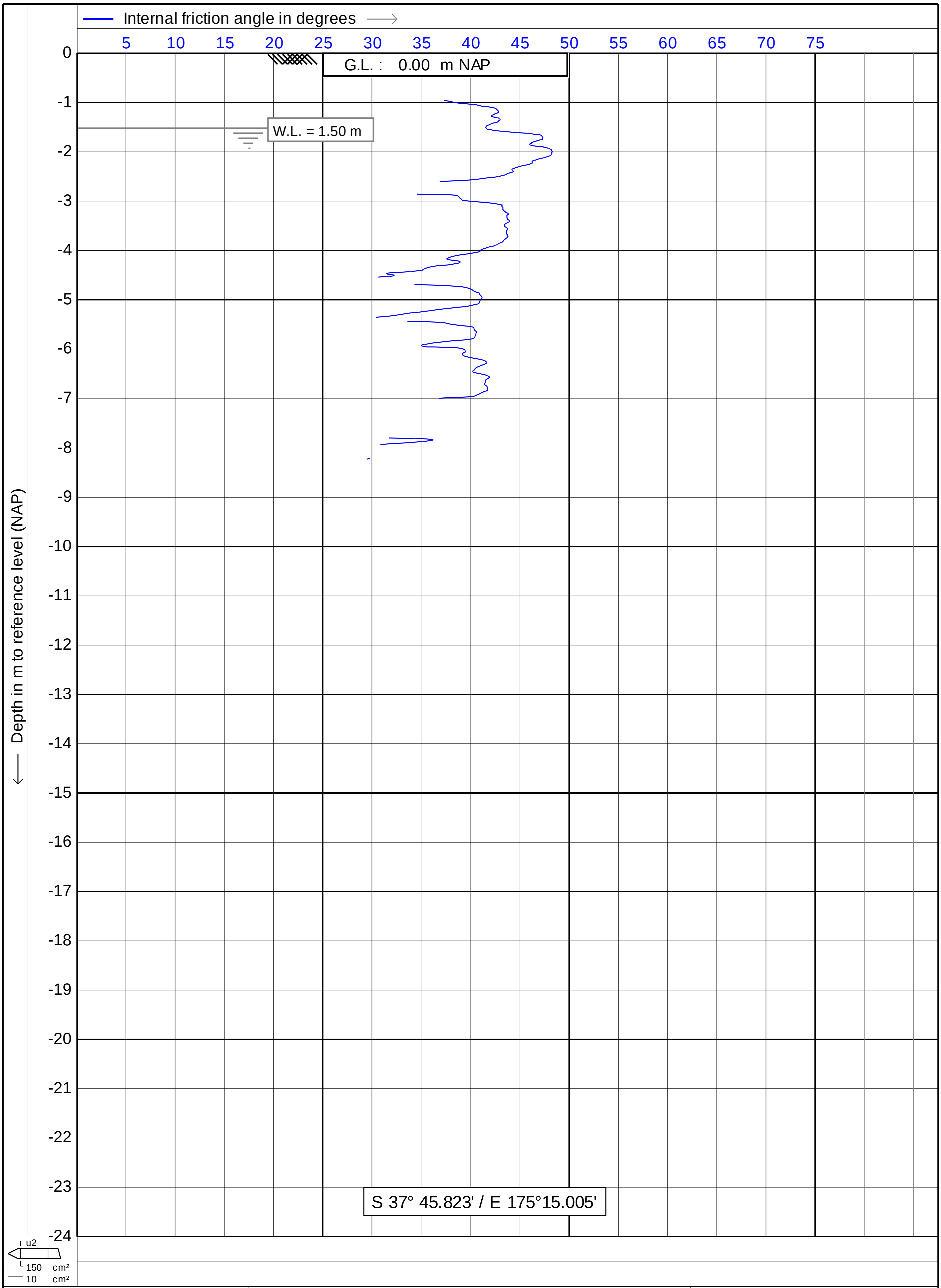
u2
150 cm²
10 cm²

CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT03**



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	Project : **TE RAPA RACECOURSE DEVELOPMENT**	Cone no.	: **S10CFIIP.S16082**
	Location: **TE RAPA RACECOURSE**	Project no.	: **17017/HAM2017-109**
	Position: **0, 0**	CPT no.	: **CPT03** **15/15**

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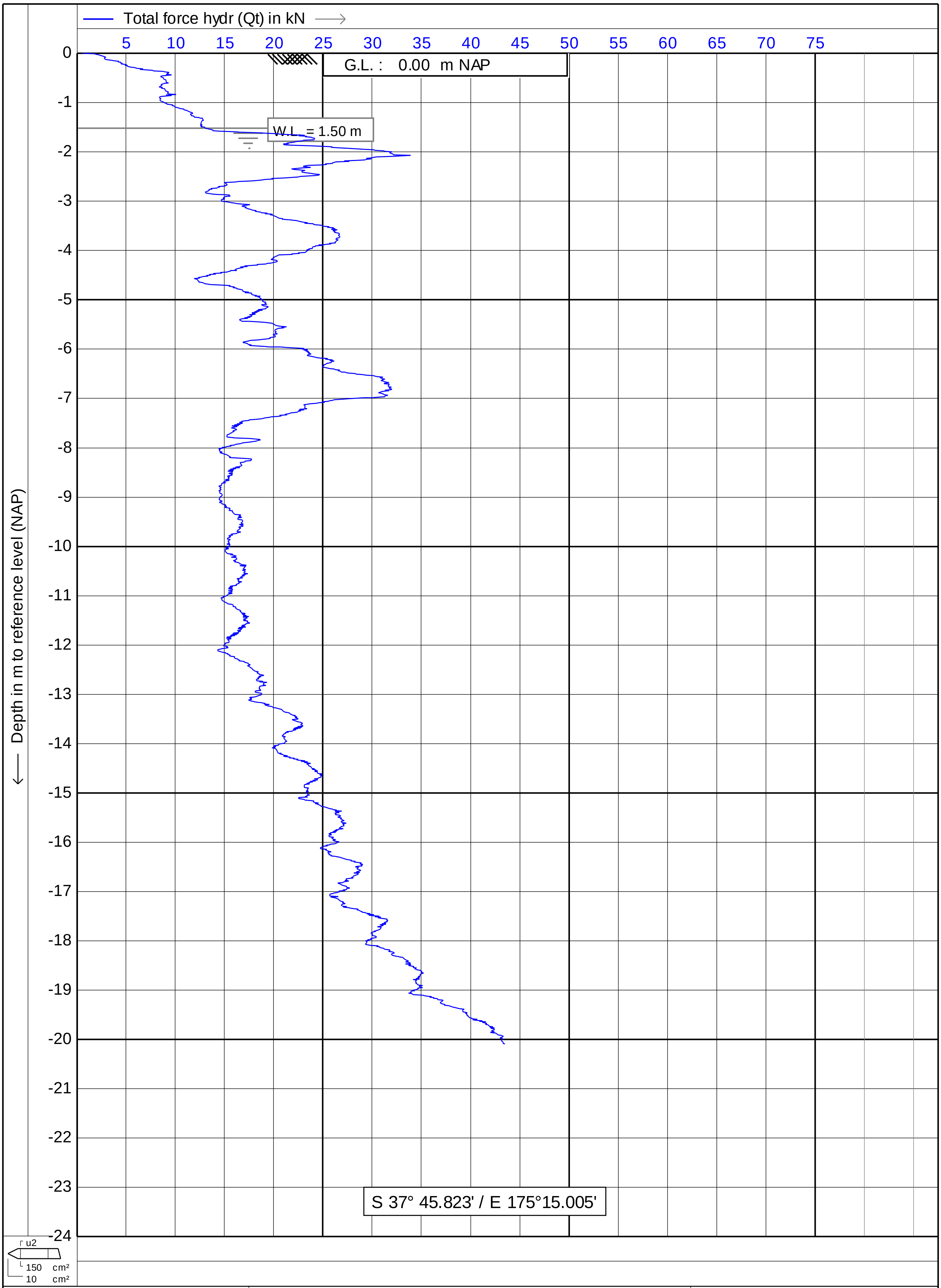

ISO 22476-1:2012 Application class 1 Test type TE1	Date : **23-May-17**	
Project : **TE RAPA RACECOURSE DEVELOPMENT**	Cone no. : **S10CFIIP.S16082**	
Location: **TE RAPA RACECOURSE**	Project no. : **17017/HAM2017-109**	
Position: **0, 0**	CPT no. : **CPT03**	**16/15**

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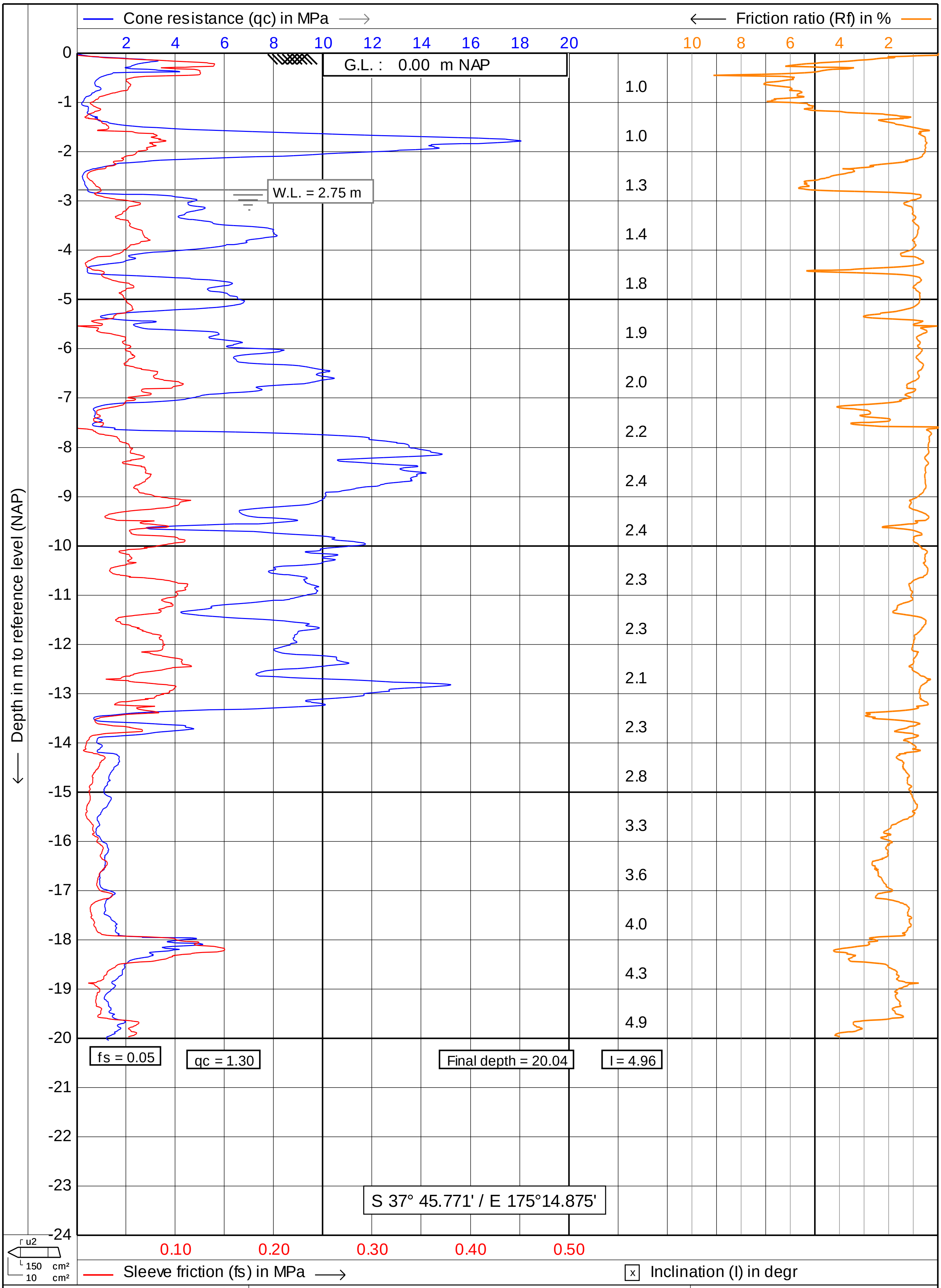


CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

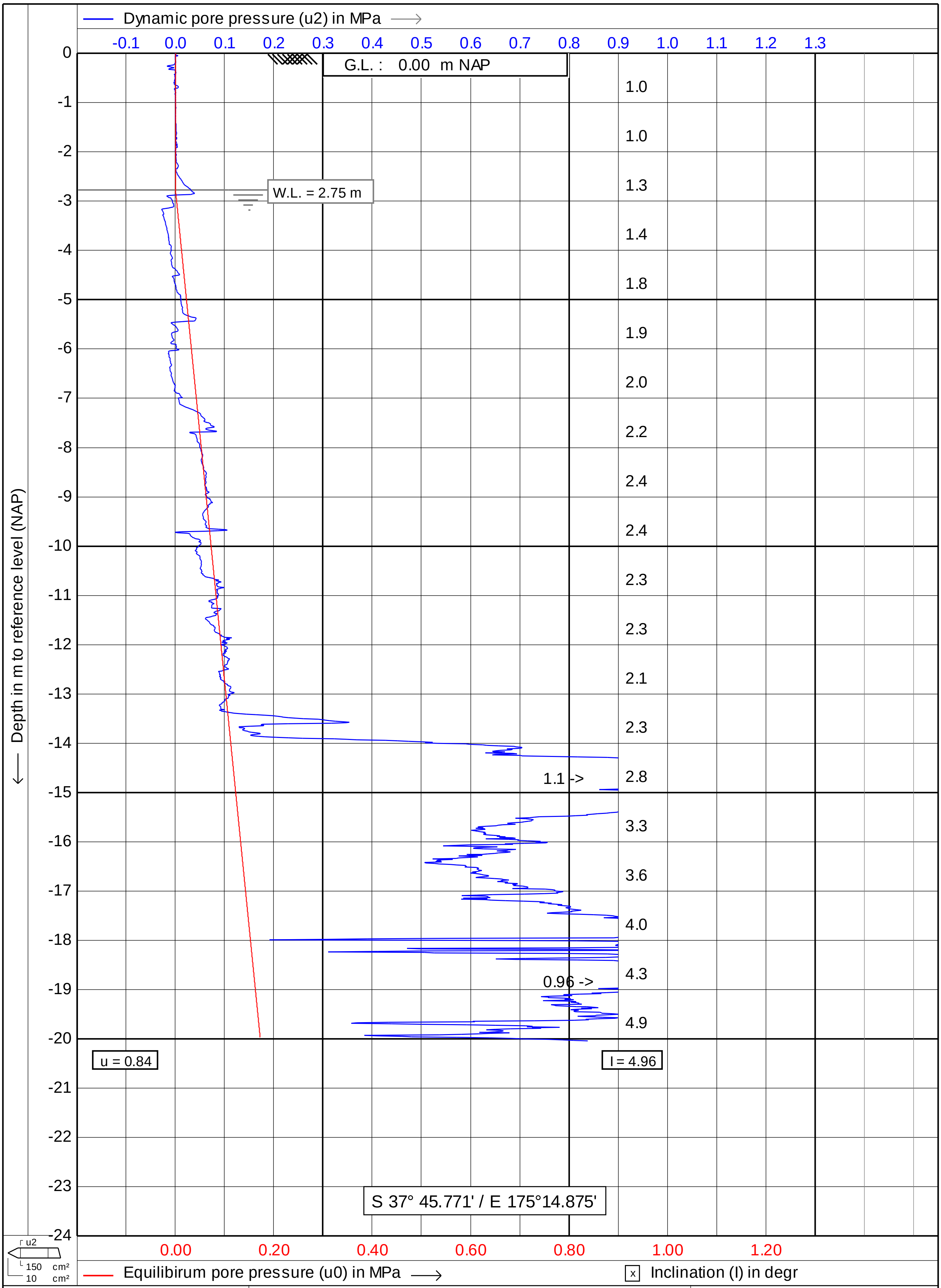
Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04**

1/15

1.47



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ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

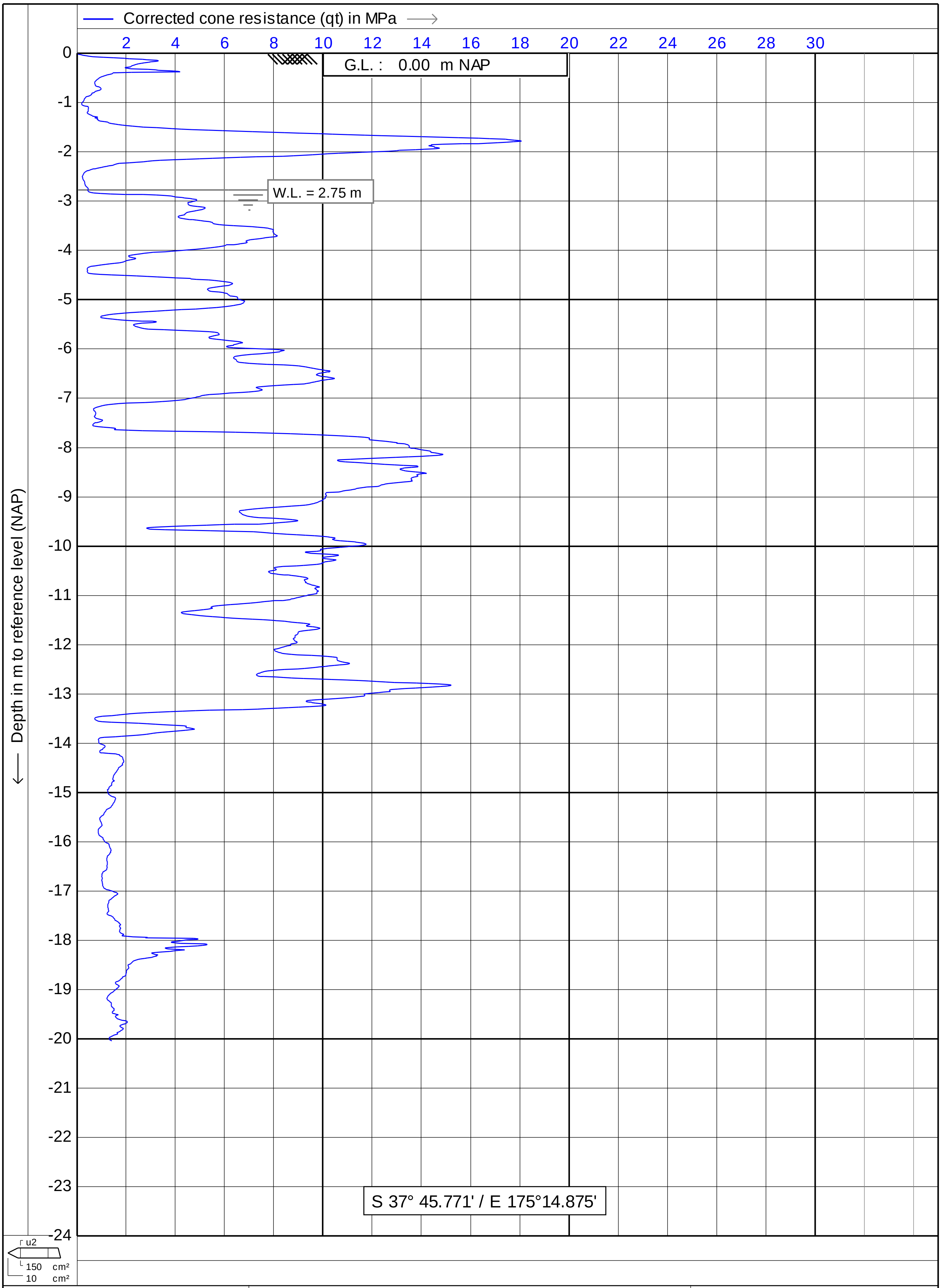
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04** **2/15**



CPT it
CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT04** **3/15**

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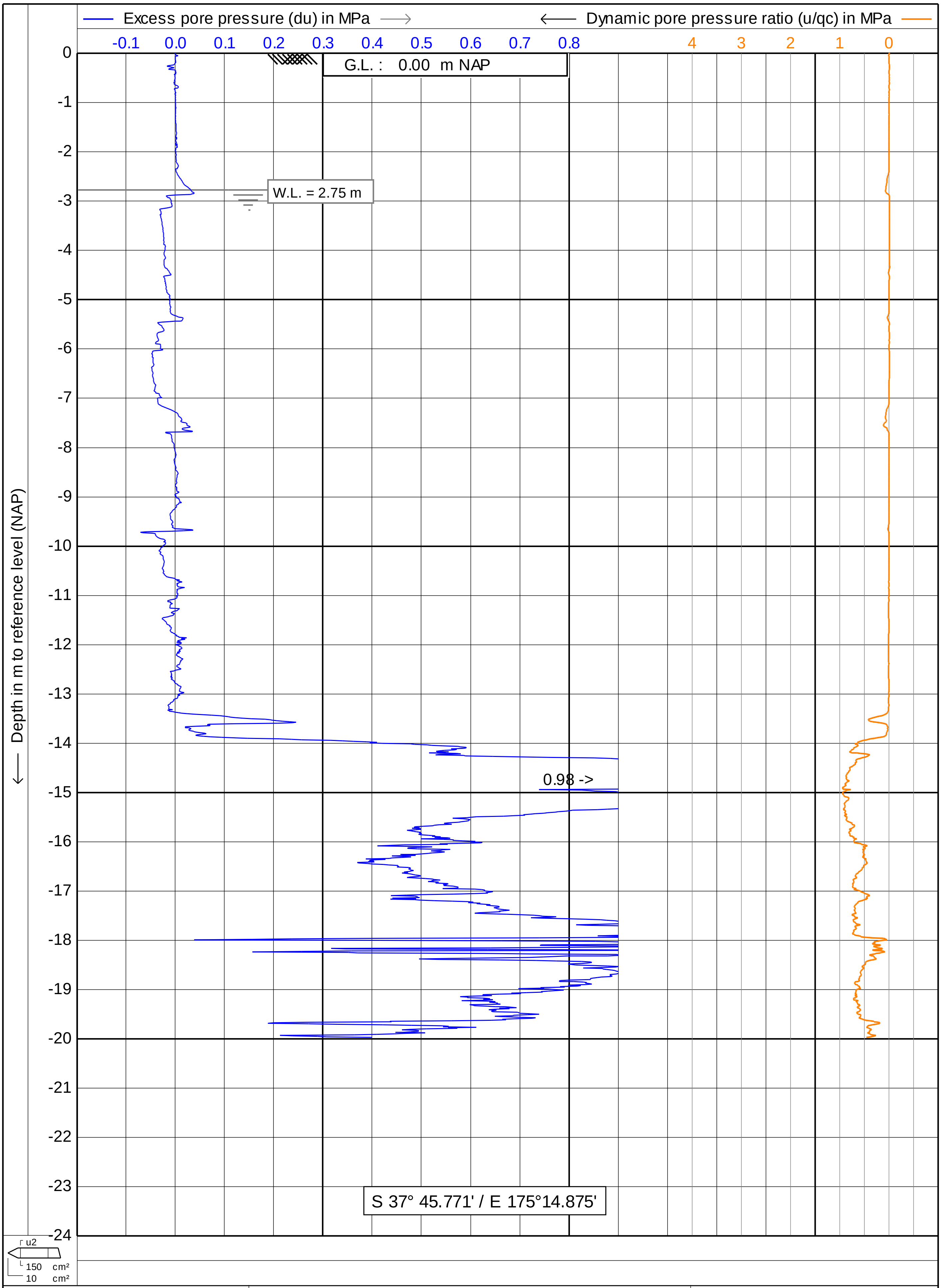

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CPT it CONE PENETROMETER TESTING

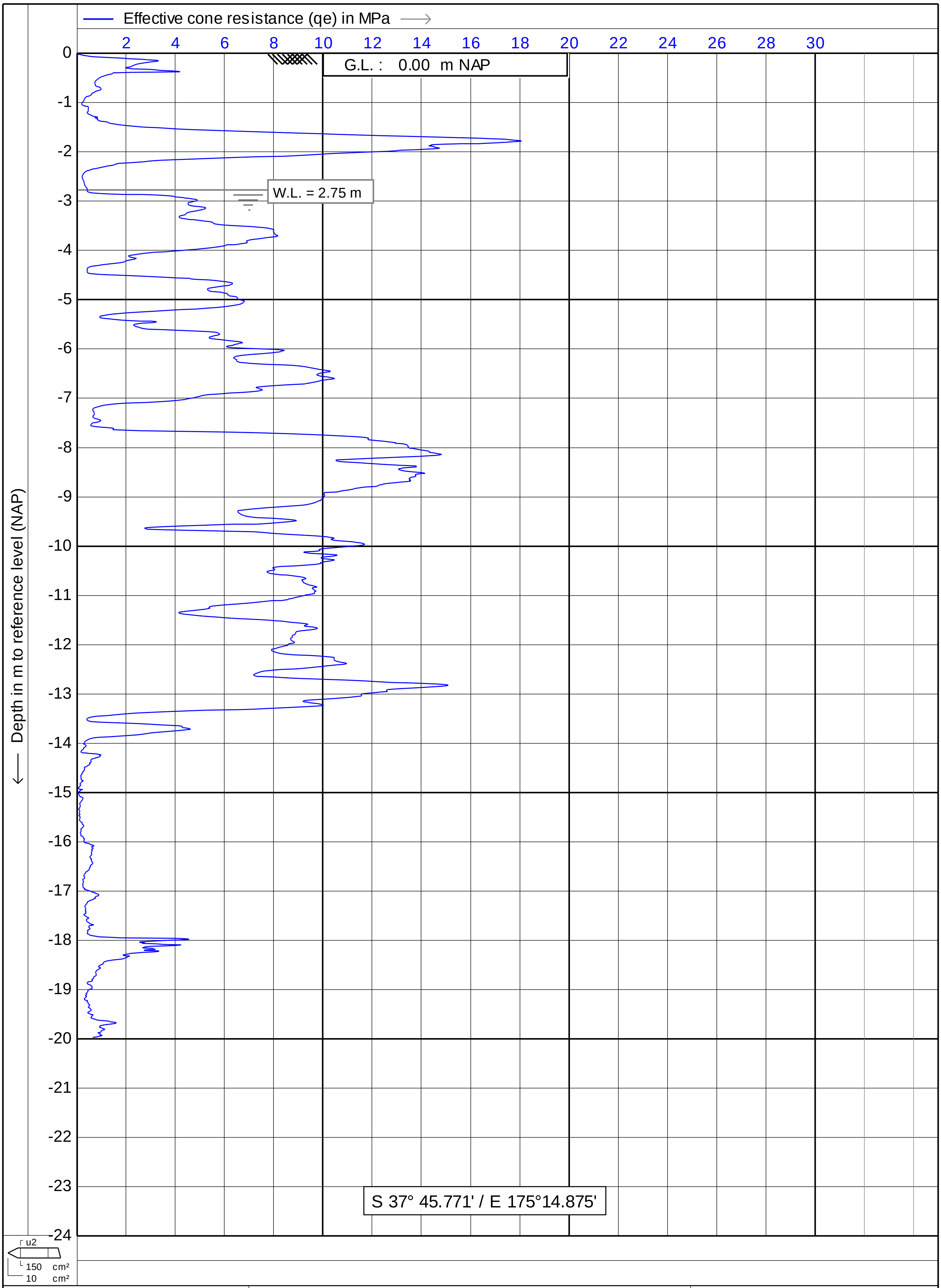
ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT04** **4/15**



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

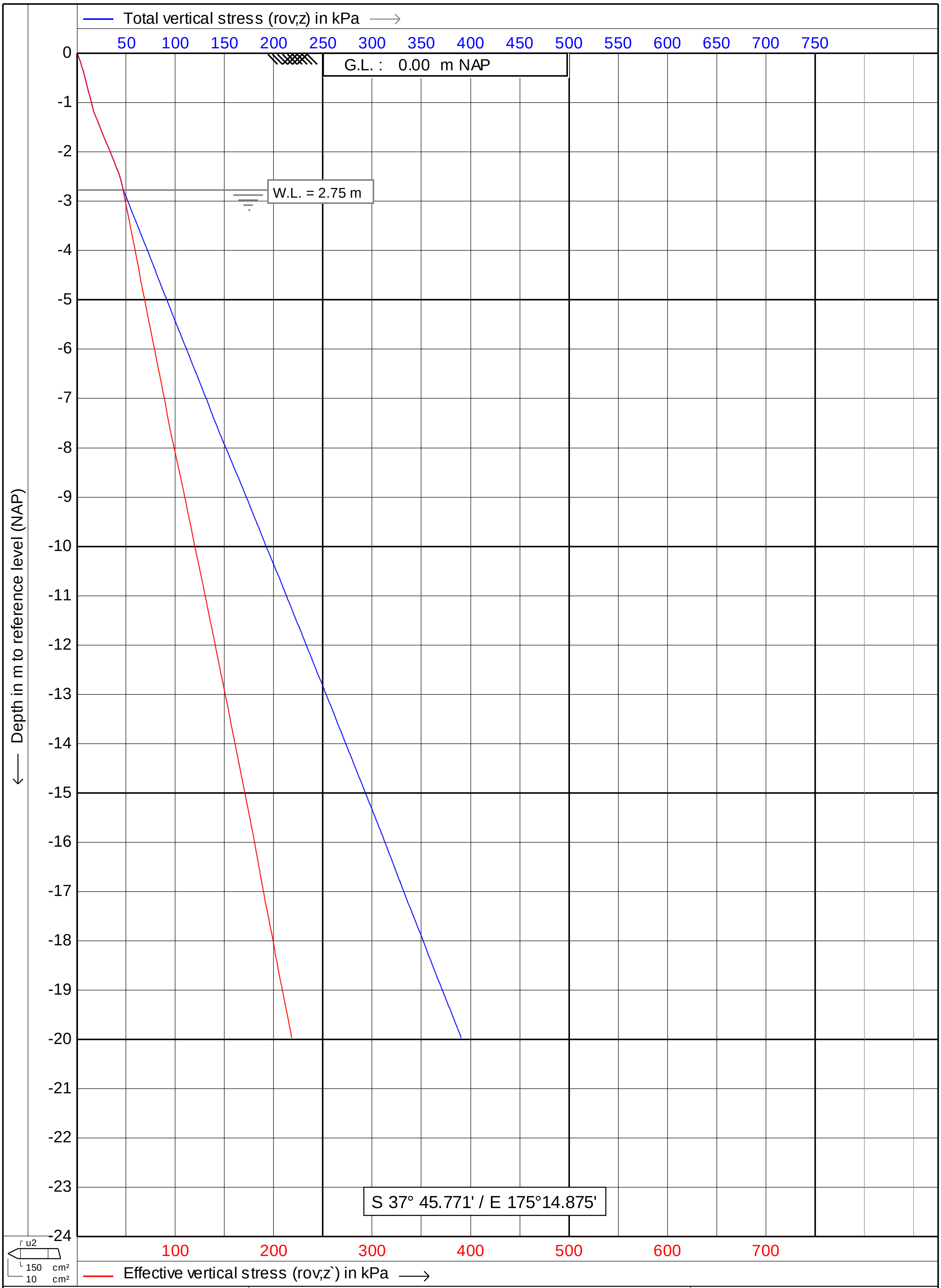
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04** **5/15**

1.47



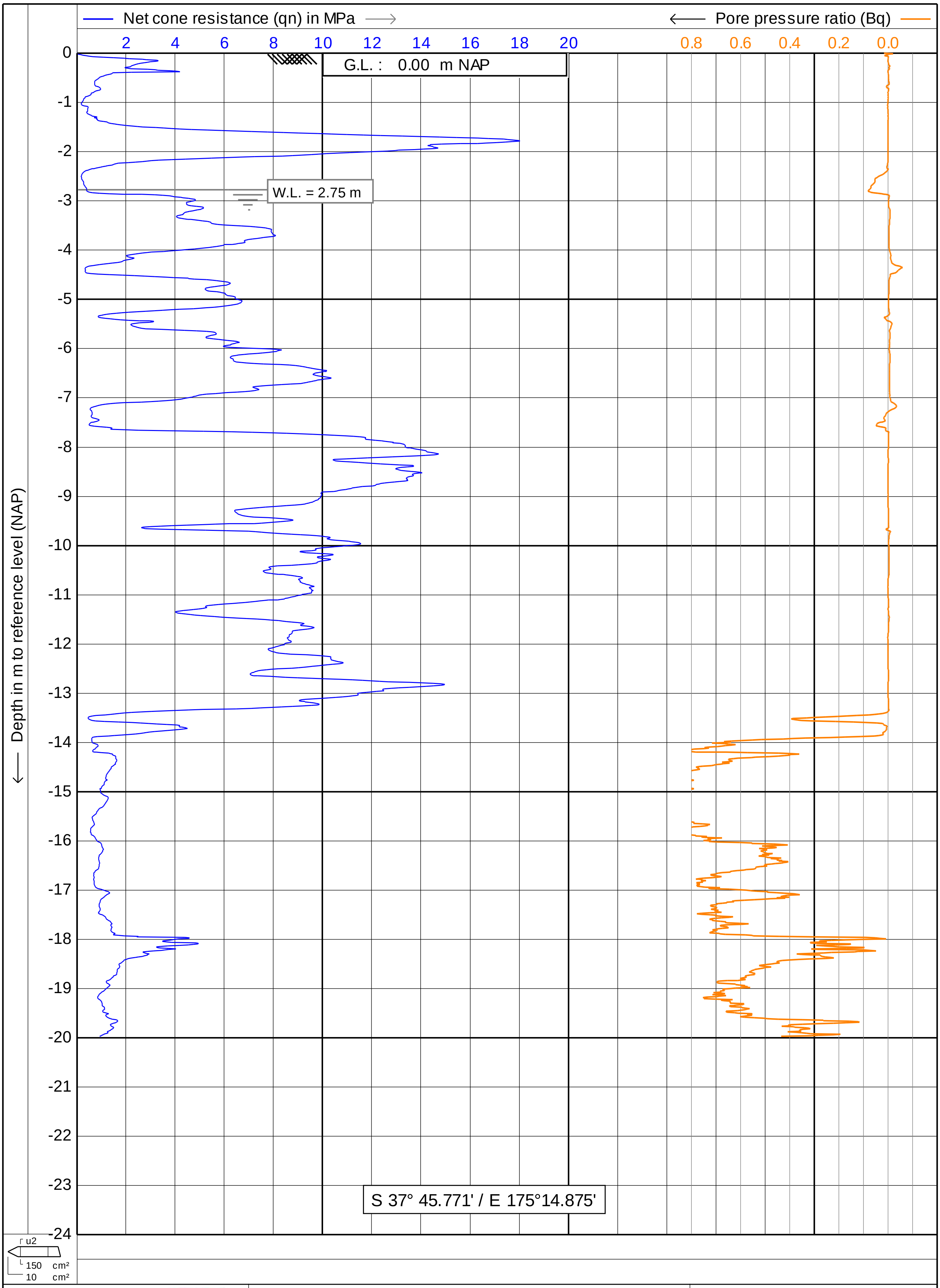
ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date	: **23-May-17**
Cone no.	: **S10CFIIP.S16082**
Project no.	: **17017/HAM2017-109**
CPT no.	: **CPT04**



CPT it
CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

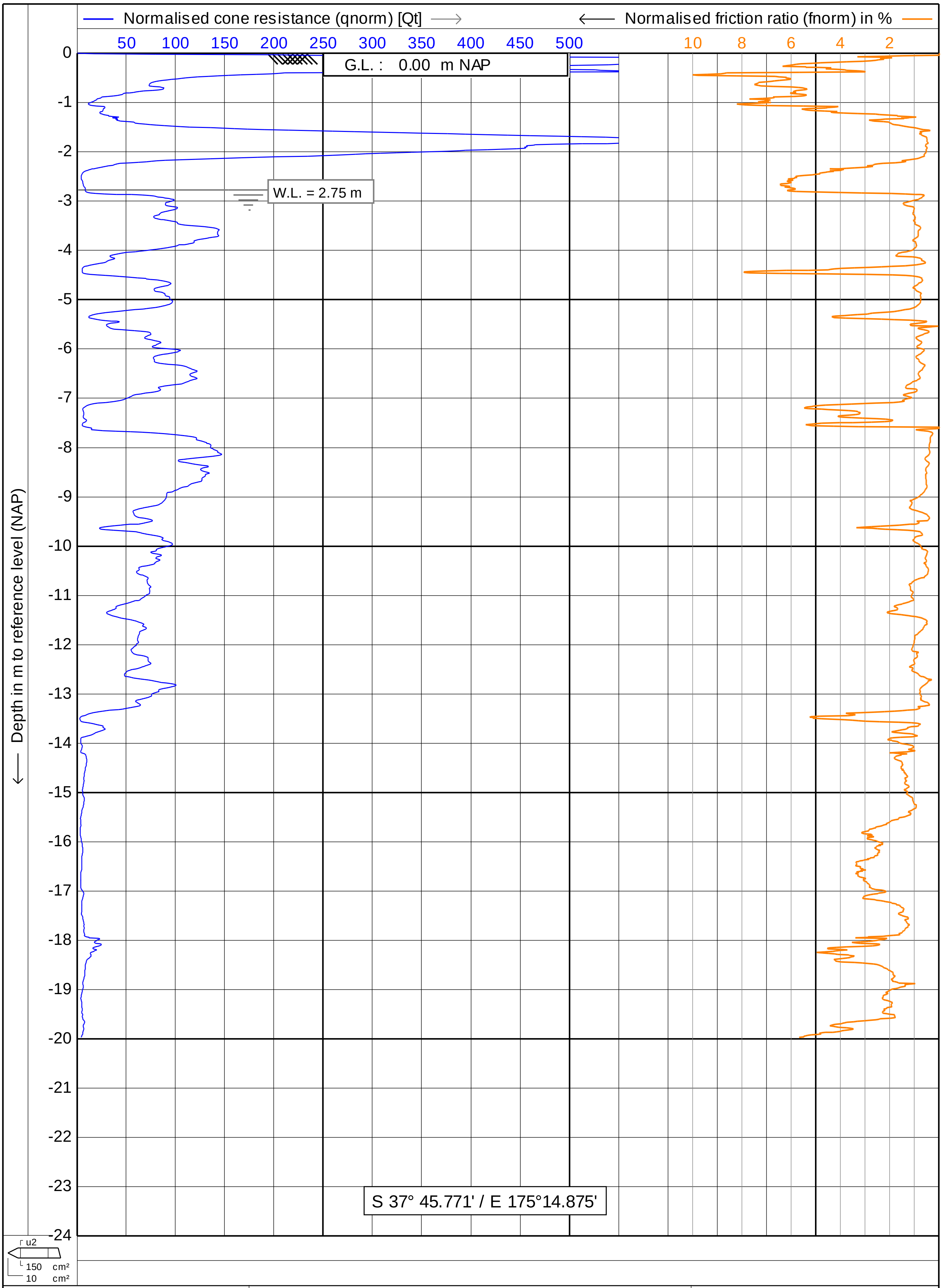
Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04** **7/15**

1.47



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ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

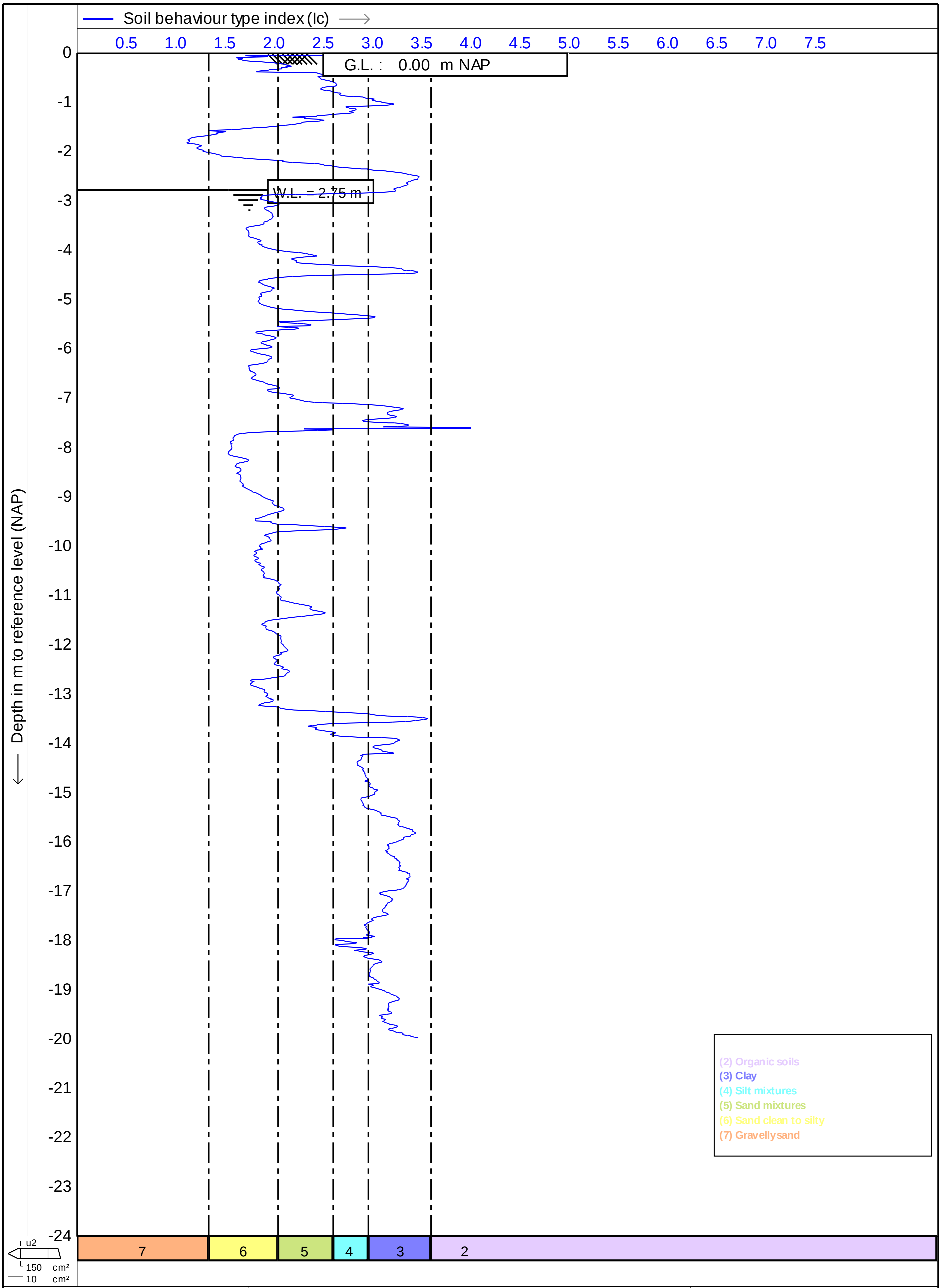
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04** **8/15**



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CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

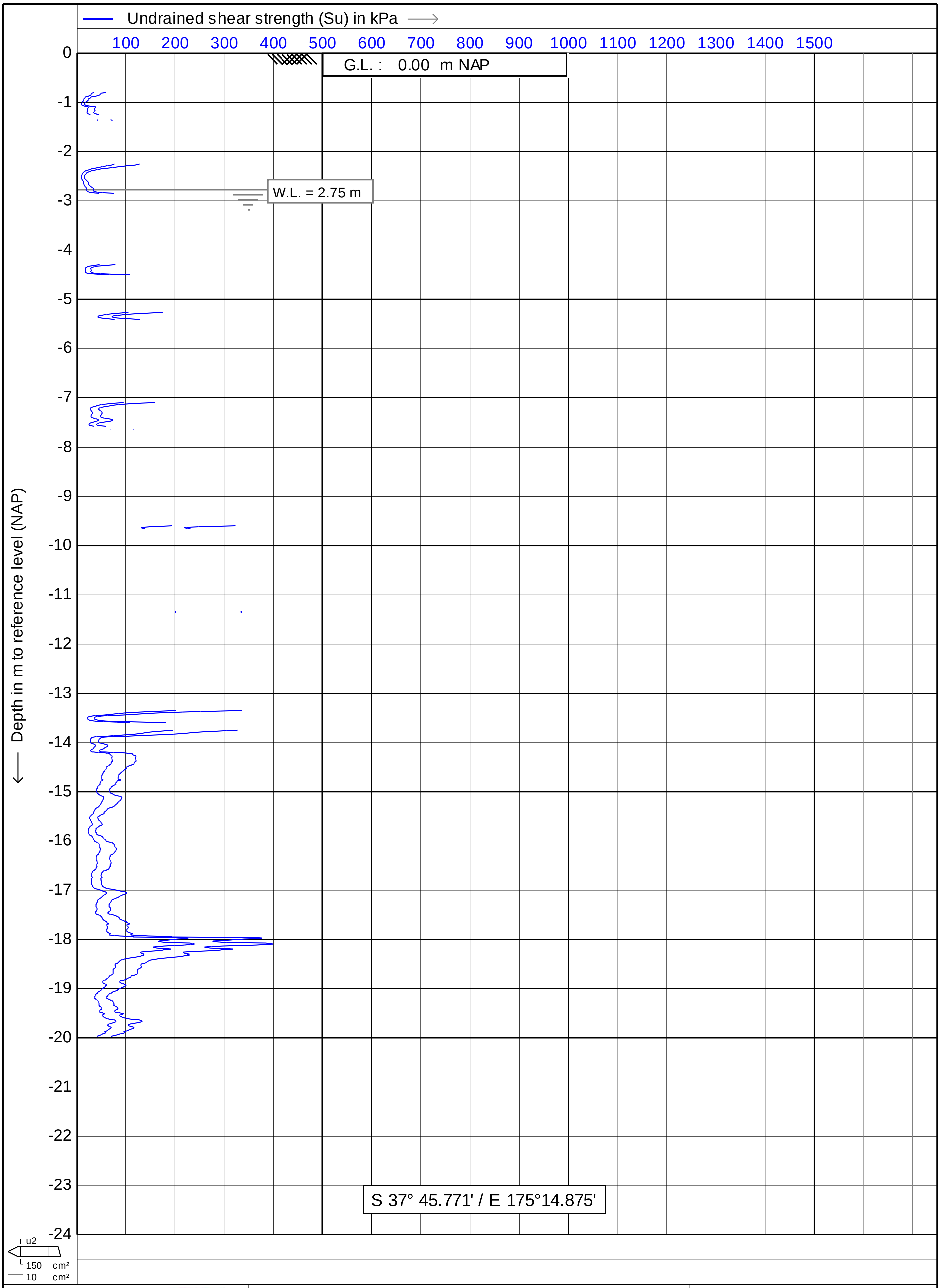
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04** **9/15**



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

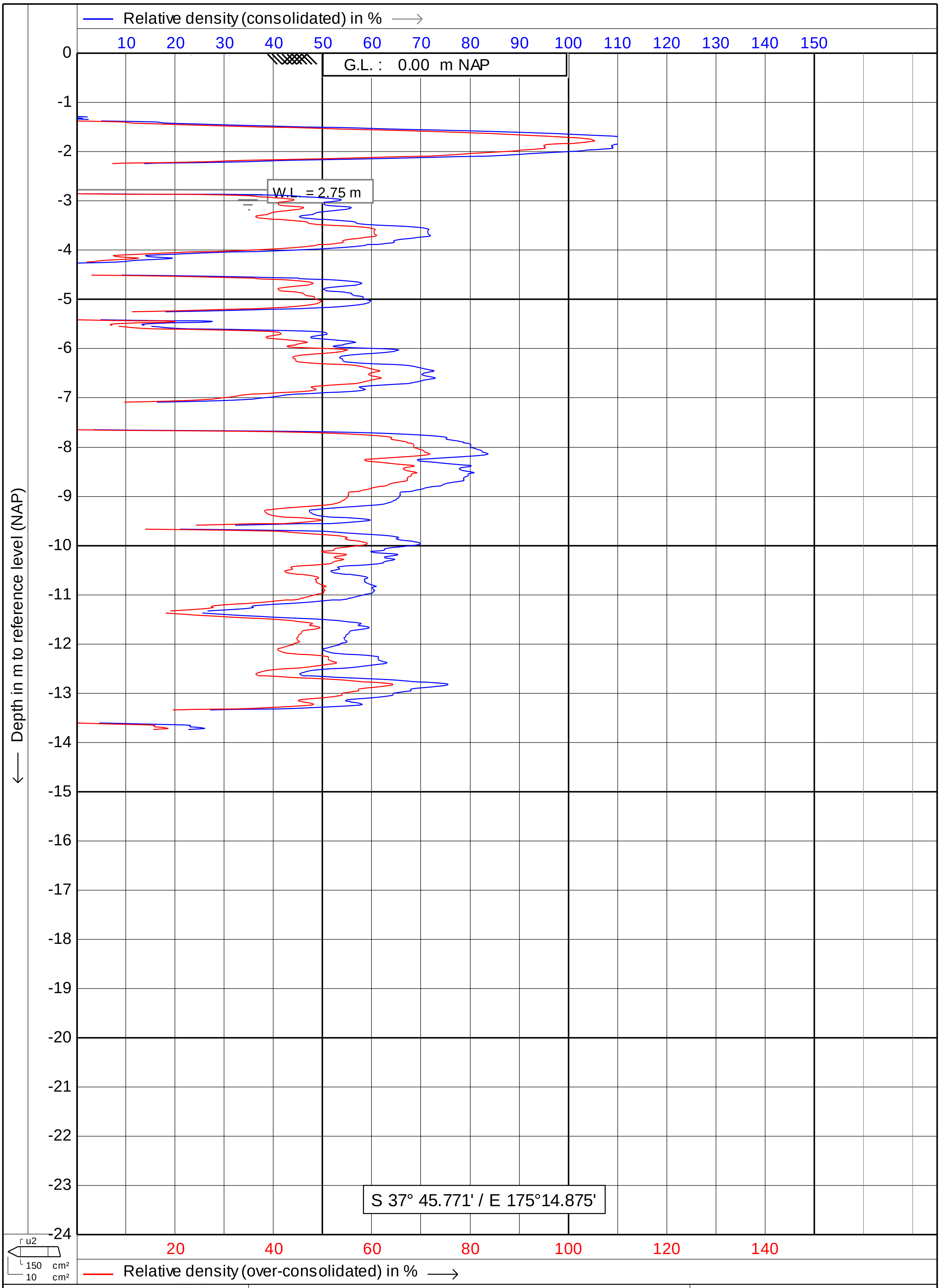
Project no. : **17017/HAM2017-109**

CPT no. : **CPT04**

10/15

CONE PENETROMETER TESTING

1.47



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

Location: **TE RAPA RACECOURSE**

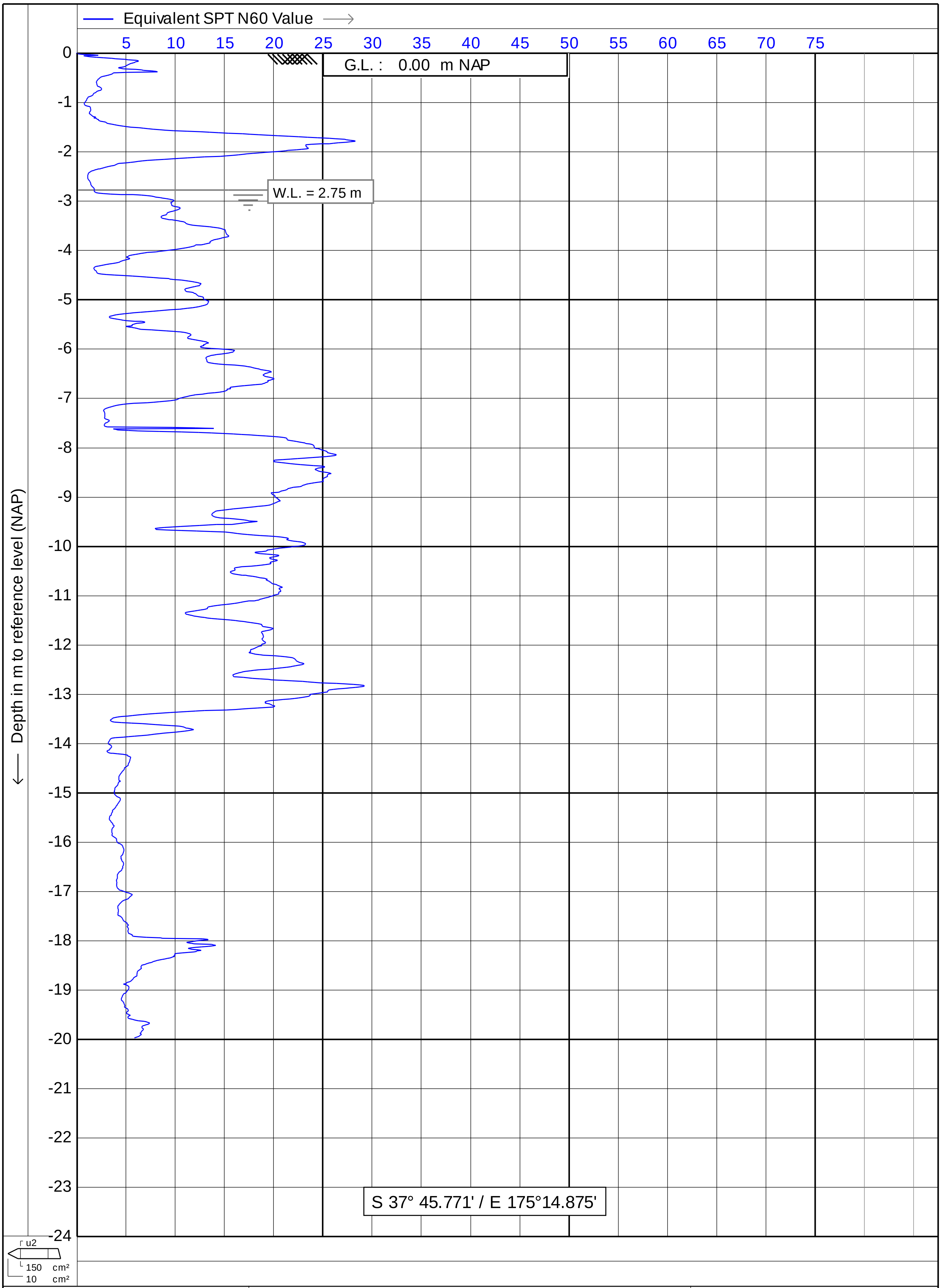
Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04**



S 37° 45.771' / E 175°14.875'

CONE PENETROMETER TESTING

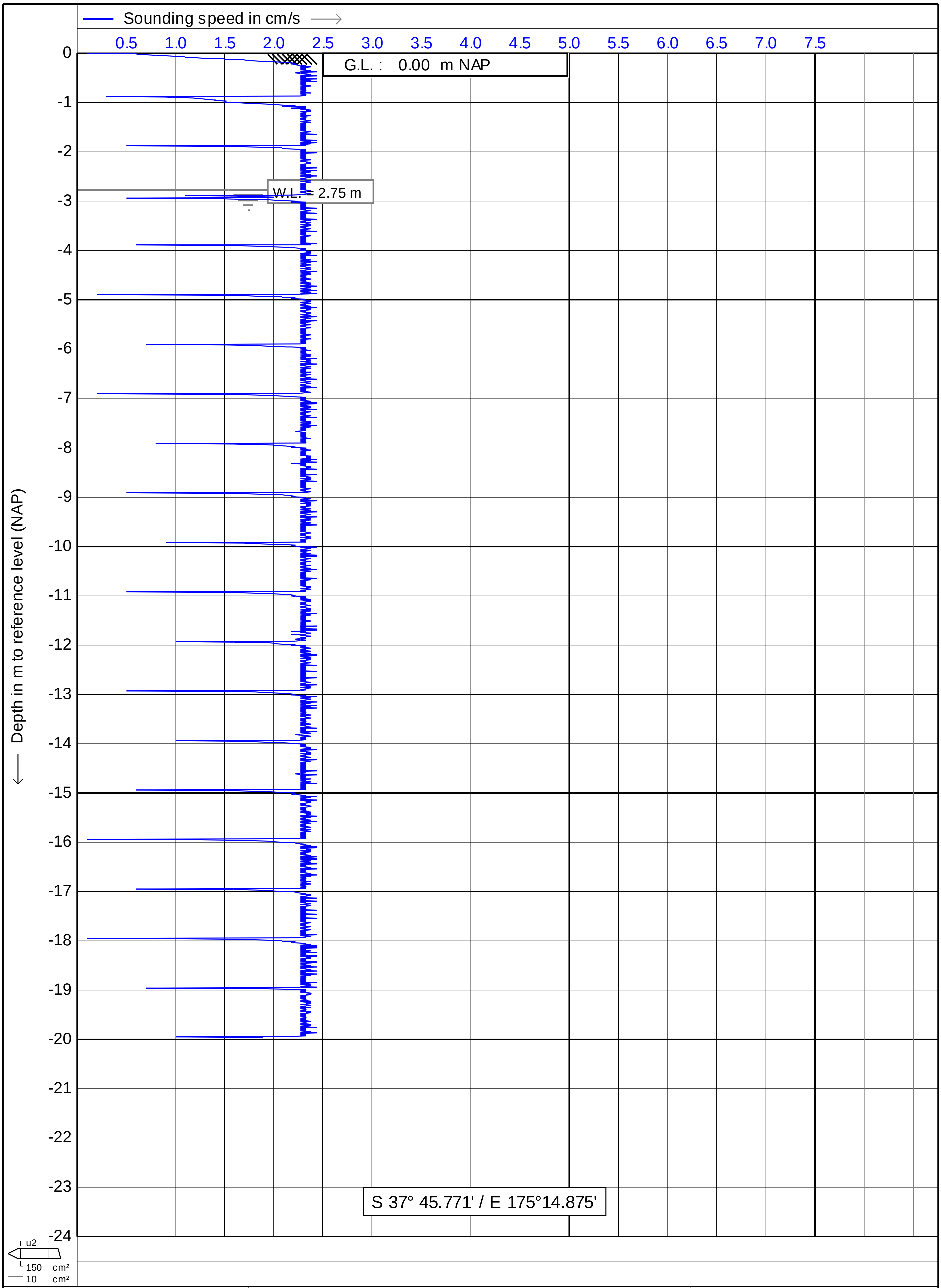
ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**
Location: **TE RAPA RACECOURSE**
Position: **0, 0**

Date : **23-May-17**
Cone no. : **S10CFIIP.S16082**
Project no. : **17017/HAM2017-109**
CPT no. : **CPT04**

12/15

1.47



ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

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Project no. : **17017/HAM2017-109**

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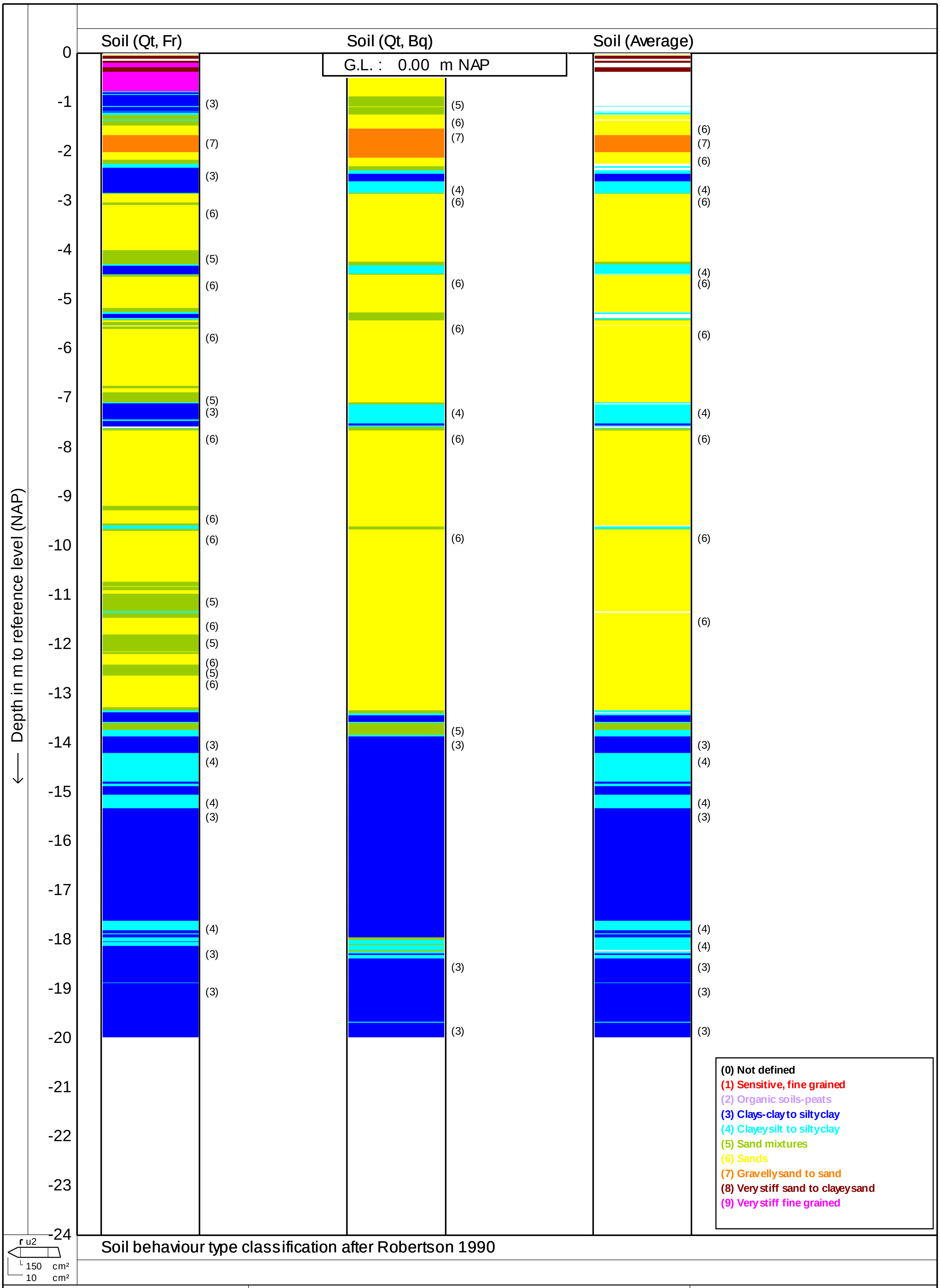
13/15

1.47



Soil behaviour type classification after Robertson 1990

1.47

CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

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Location: **TE RAPA RACECOURSE**

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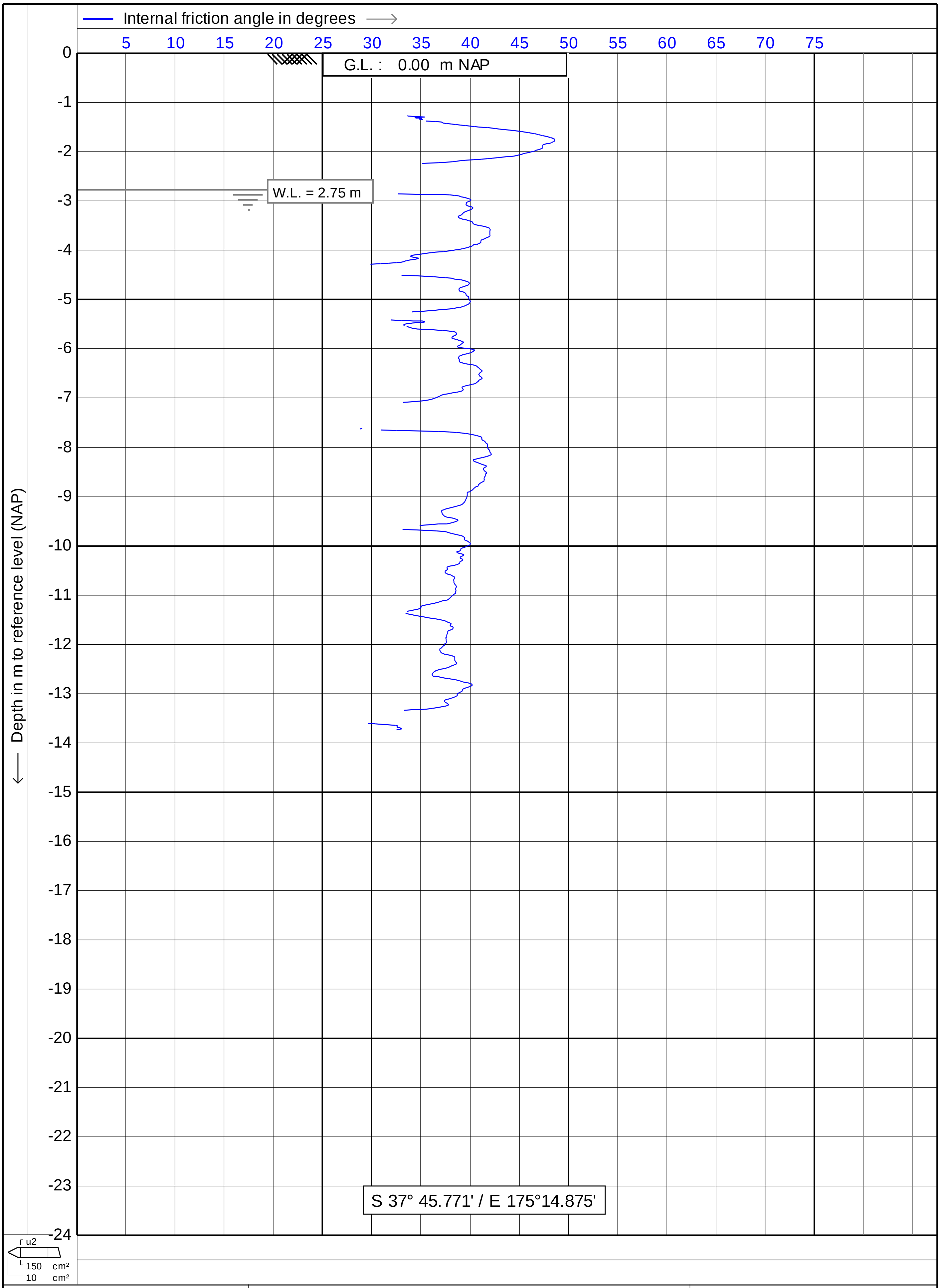

ISO 22476-1:2012 Application class 1 Test type TE1

Project : **TE RAPA RACECOURSE DEVELOPMENT**

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Position: **0, 0**

Date : **23-May-17**

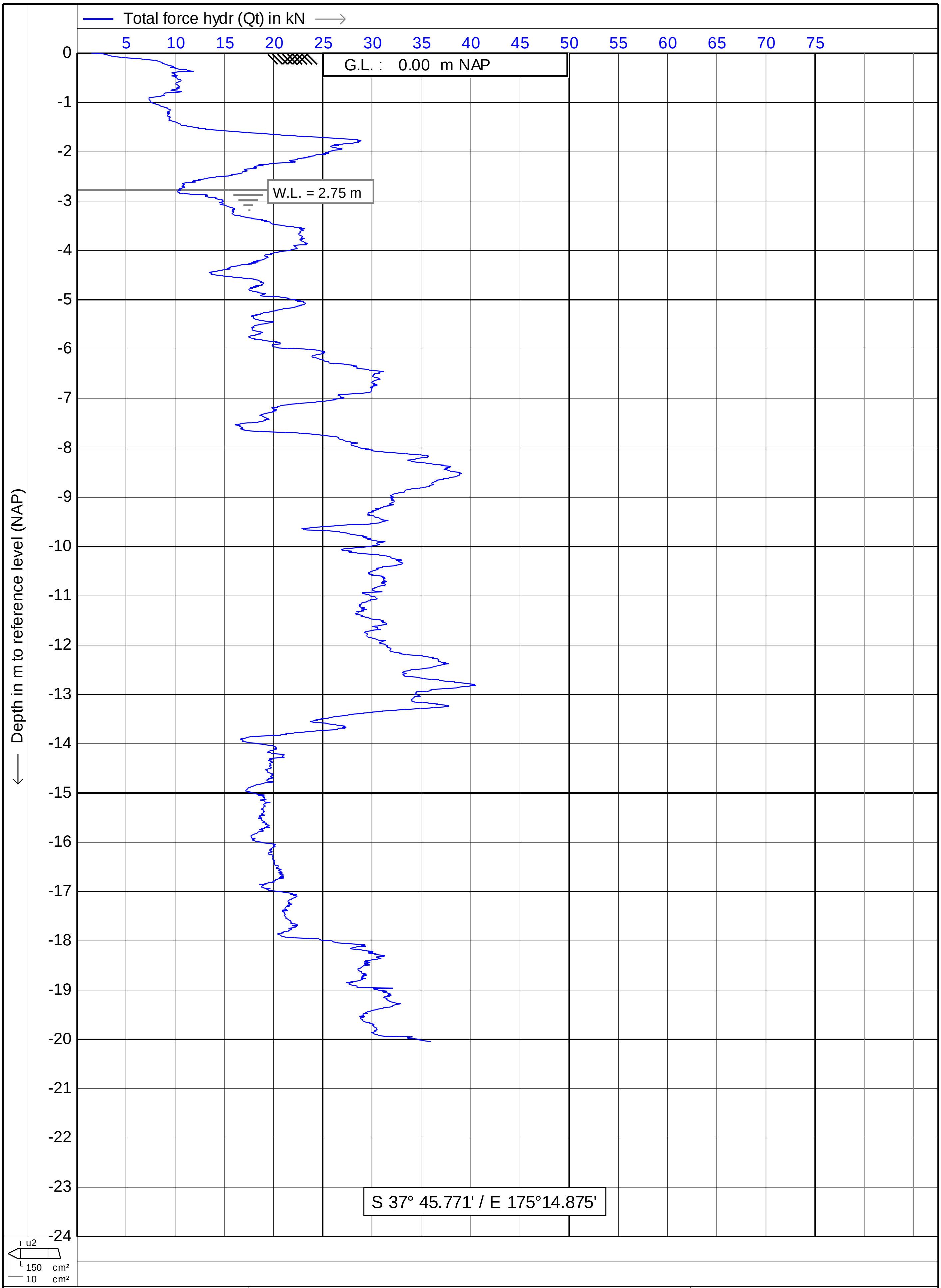
Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04**

15/15

1.47



CONE PENETROMETER TESTING

ISO 22476-1:2012 Application class 1 Test type TE1

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Location: **TE RAPA RACECOURSE**

Position: **0, 0**

Date : **23-May-17**

Cone no. : **S10CFIIP.S16082**

Project no. : **17017/HAM2017-109**

CPT no. : **CPT04**

16/15

APPENDIX B: CHOW:HILL CONCEPT PLAN

Position more intensive lots of approx. 200 sq.m. net area around edge facing WRC

Consider lots of approx. 300sq.m. net area on boundary to industrial land.

Consider more intensive residential development next to track - approx. 200 sq.m. net per unit.

GROSS DEVELOPMENT AREA (YELLOW)

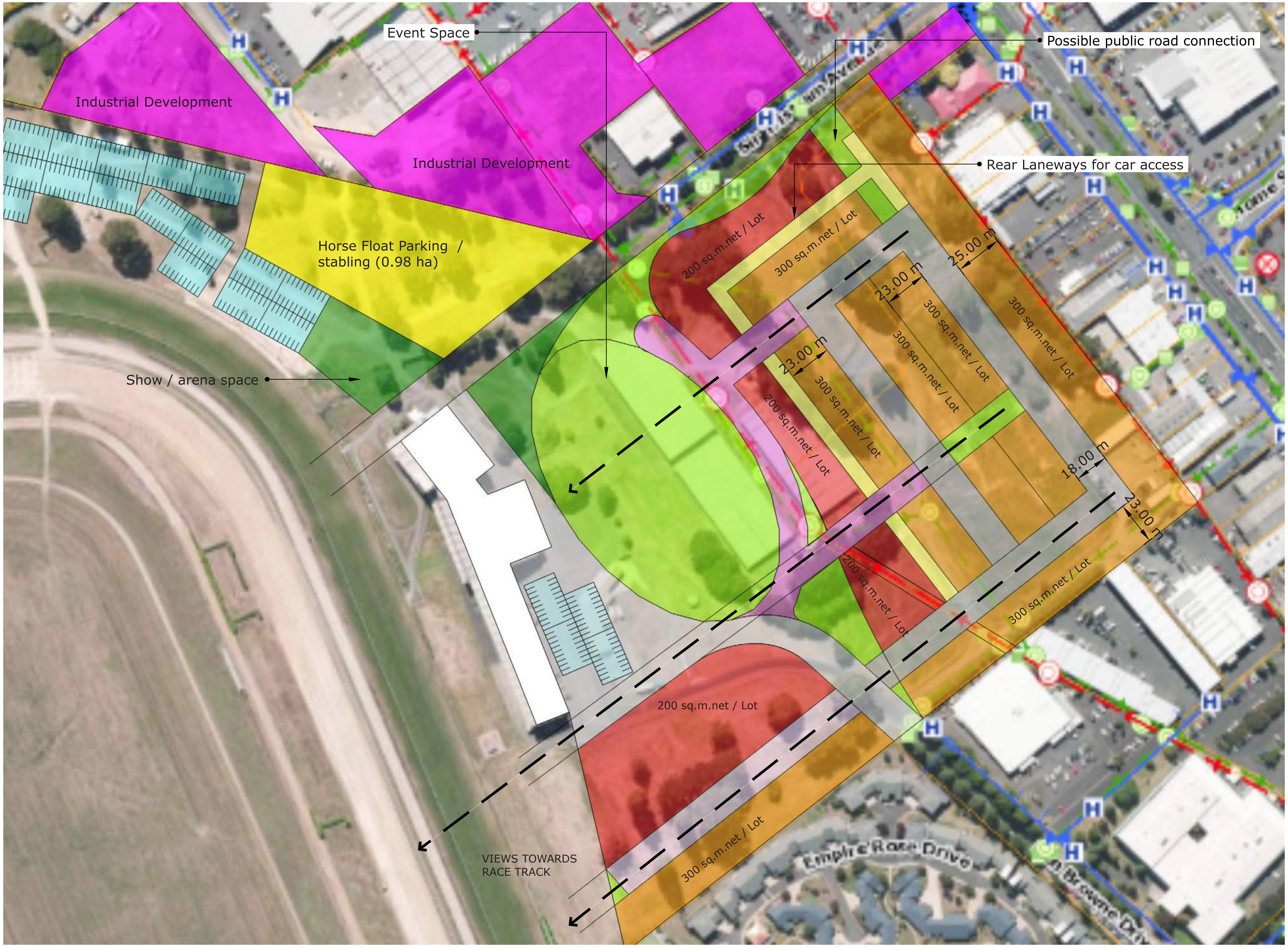
Approx. 6.48 ha.

(Note that Gross Area would include roads, open space and the net area of residential lots)

16704 10 May 2017

SKETCH OPTION 4A - Gross Area

chowhill



GROSS DEVELOPMENT AREA

Approx. 6.48 ha.

(Note that Gross Area would include roads, open space and the net area of residential lots)

NET DEVELOPMENT AREA (ORANGE and RED)

Approx. 4.19 ha

This is based on the sketch plan as shown, and represents about 65% of the Gross Area, which is reasonably efficient.

An alternative location is shown for race day vehicles and stabling (Yellow). Should this be too small,the area could be extended to the west.

16704 10 May 2017

SKETCH OPTION 4A - Indicative Concept Plan

N

0 400m

DEVELOPMENT CONTEXT

This plan shows Option 4A within the wider setting. The key features as shown here are:

Vehicular Access

- Access to the proposed residential areas is intended to be from the south via Ken Browne Drive. There is also the possibility of a vehicle connection from this area to Sir Tristram Avenue to the north.
- Race related traffic associated with horse movement / officials is shown to the north of the existing stands (in yellow). This has separate vehicle access via Sir Tristram Avenue or Mainstreet Place.
- Car parking for race events is shown around the northern side of the track and there is space to go beyond the approximate 1100 spaces shown. Access is shown from Sunshine Avenue to the north or Sir Tristram Avenue.

Flexibility

- The basic concept allows Sir Tristram Avenue and the roads to the south of the stands to extend westward if required, towards the track.

Character

- This plan highlights the possibility of an attractive private open space between the grandstands and the proposed development. This can be used for event purposes and also serves as a buffer space between the different uses.

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SKETCH OPTION 4A - Context Plan

chowhill

Higher building forms are proposed along the edge of the development facing the racing club. These buildings, as the more public aspect of the residential area, could adopt a relatively bold and distinctive character.

The 2 -storey dwellings along this boundary are intended to provide a relatively continuous acoustic barrier between the rest of the development and the industrial uses to the east.

The central open space could reflect the character of the horse racing environment in its treatment, as per the example below.

This depicts a relatively low density apartment scenario. 3 apartments are grouped around each stair and lift core, allowing good views outwards. The car parking as shown on the surface would allow 2 spaces per unit and still retain existing open space and some tree planting.

By adopting a denser form of apartment or a higher building, the number of units in this area could be increased, possibly limited only by economical car parking provision.

DWELLING NUMBERS

RED AREAS

Average net lot area - 158 sq.m.

Area of land - 15,209 sq.m.

Indicative Numbers - 72 Apartments
24 3-3torey townhouses

ORANGE AREAS

Average net lot area - 223 sq.m.

Area of land - 22,795 sq.m.

Indicative Numbers 20 detached dwellings
66 2-storey duplex dwellings
16 3-storey townhouses

Total number of dwellings - 198

This image is indicative of the scale of building forms that could be adopted. Note that this is more diagrammatic rather than a final representation of the architecture.

Within this framework, there is significant scope to incorporate variety and adjust the numbers of dwellings in different parts of the site, either up or down.

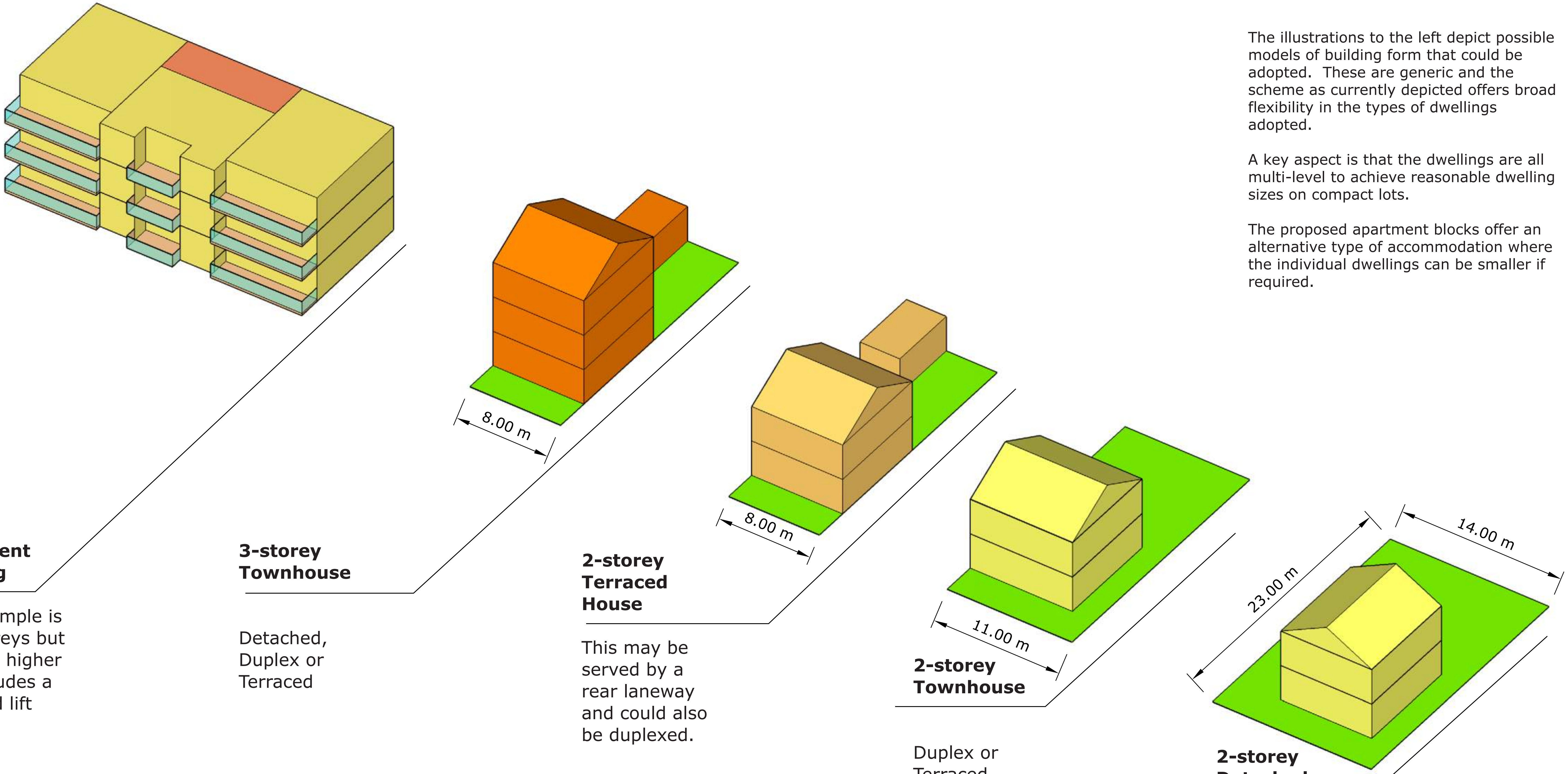
16704 10 May 2017

SKETCH OPTION 4A - Aerial View of Possible Dwelling Typologies

The illustrations to the left depict possible models of building form that could be adopted. These are generic and the scheme as currently depicted offers broad flexibility in the types of dwellings adopted.

A key aspect is that the dwellings are all multi-level to achieve reasonable dwelling sizes on compact lots.

The proposed apartment blocks offer an alternative type of accommodation where the individual dwellings can be smaller if required.



Apartment Building

This example is of 3 storeys but could be higher and includes a stair and lift core

3-storey Townhouse

Detached, Duplex or Terraced

2-storey Terraced House

This may be served by a rear laneway and could also be duplexed.

2-storey Townhouse

Duplex or Terraced

2-storey Detached House

Parnell, Auckland

Parnell, Auckland

Proposed Housing, Hobsonville

Duplex Housing, Hobsonville

Detached housing, Hobsonville

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Possible Dwelling Typologies

chowhill

APPENDIX C: CLIQ OUTPUTS

CMW Geosciences (NZ) Ltd
Suite 2, 5 Hill St, Hamilton 3204
PO Box 995, Waikato Mail Centre, Hamilton 3204
www.cmwgeosciences.co.nz

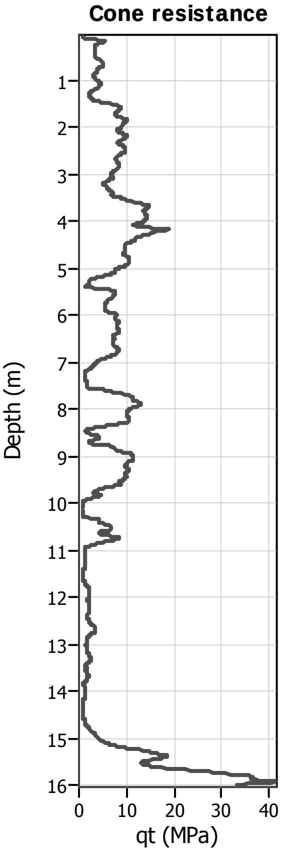
LIQUEFACTION ANALYSIS REPORT

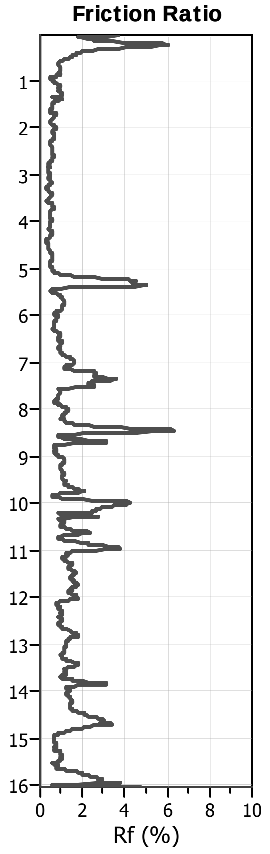
Project title : Te Rapa Racesourse Redevelopment **Location : Hamilton**

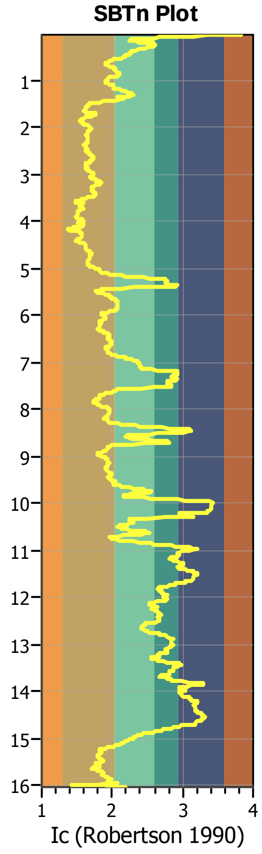
CPT file : CPT01 ULS

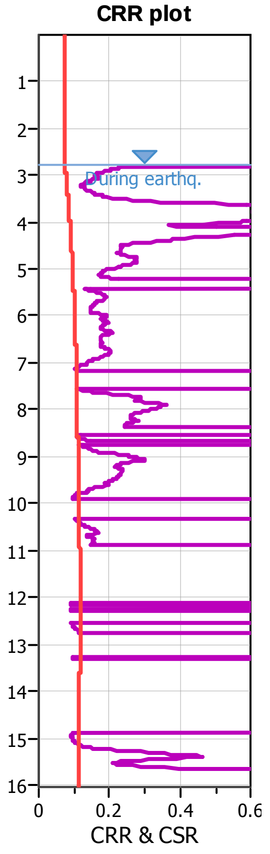
Input parameters and analysis data

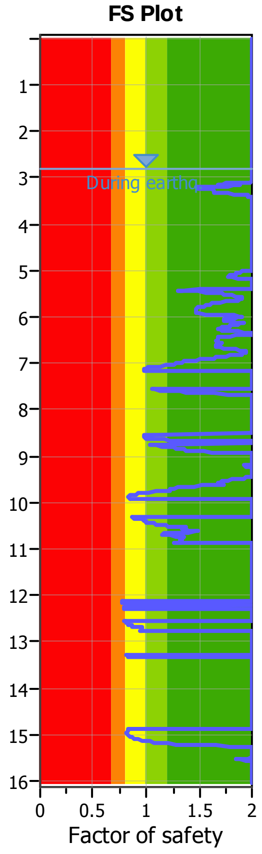
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.80 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.80 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.75	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.23	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

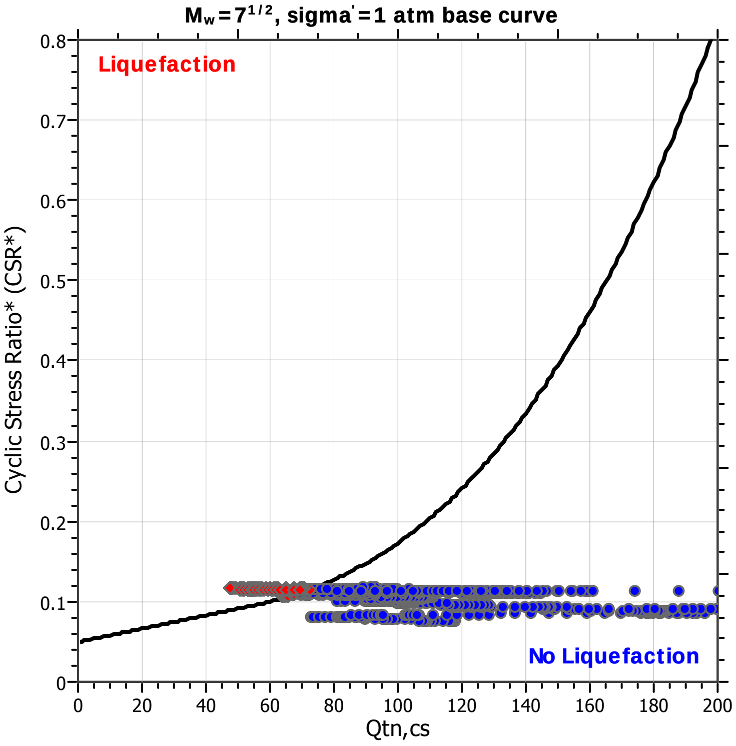

Cone resistance

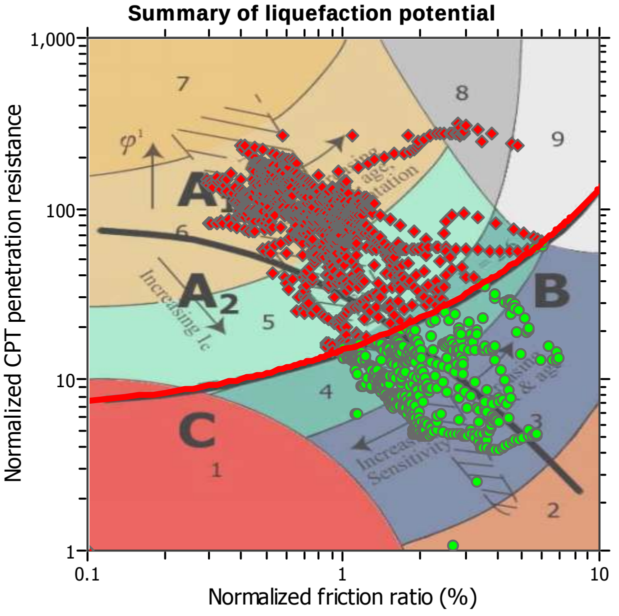

Friction Ratio


SBTn Plot


CRR plot

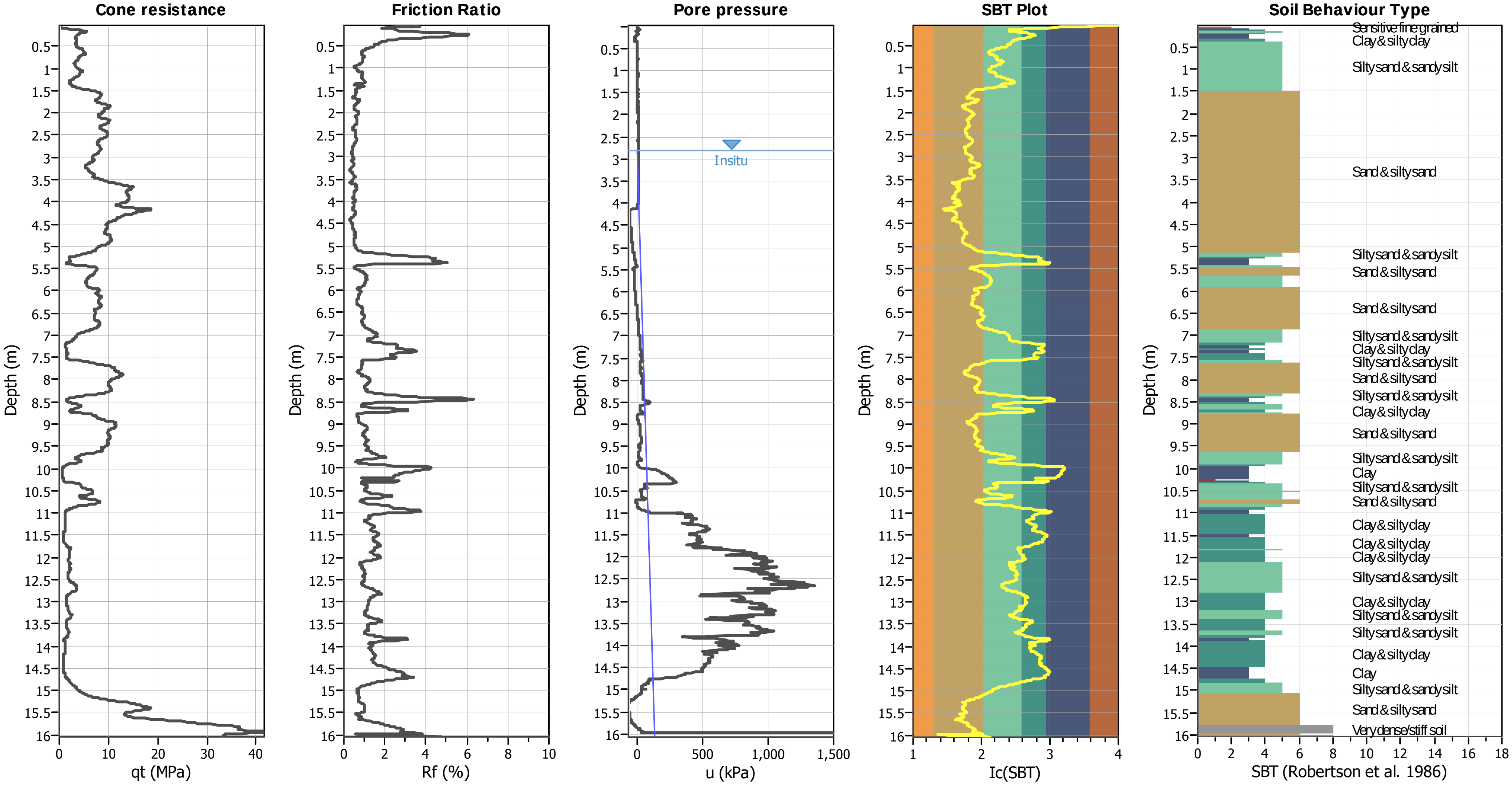

FS Plot


$M_w=7^{1/2}$, sigma'=1 atm base curve


Summary of liquefaction potential

Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



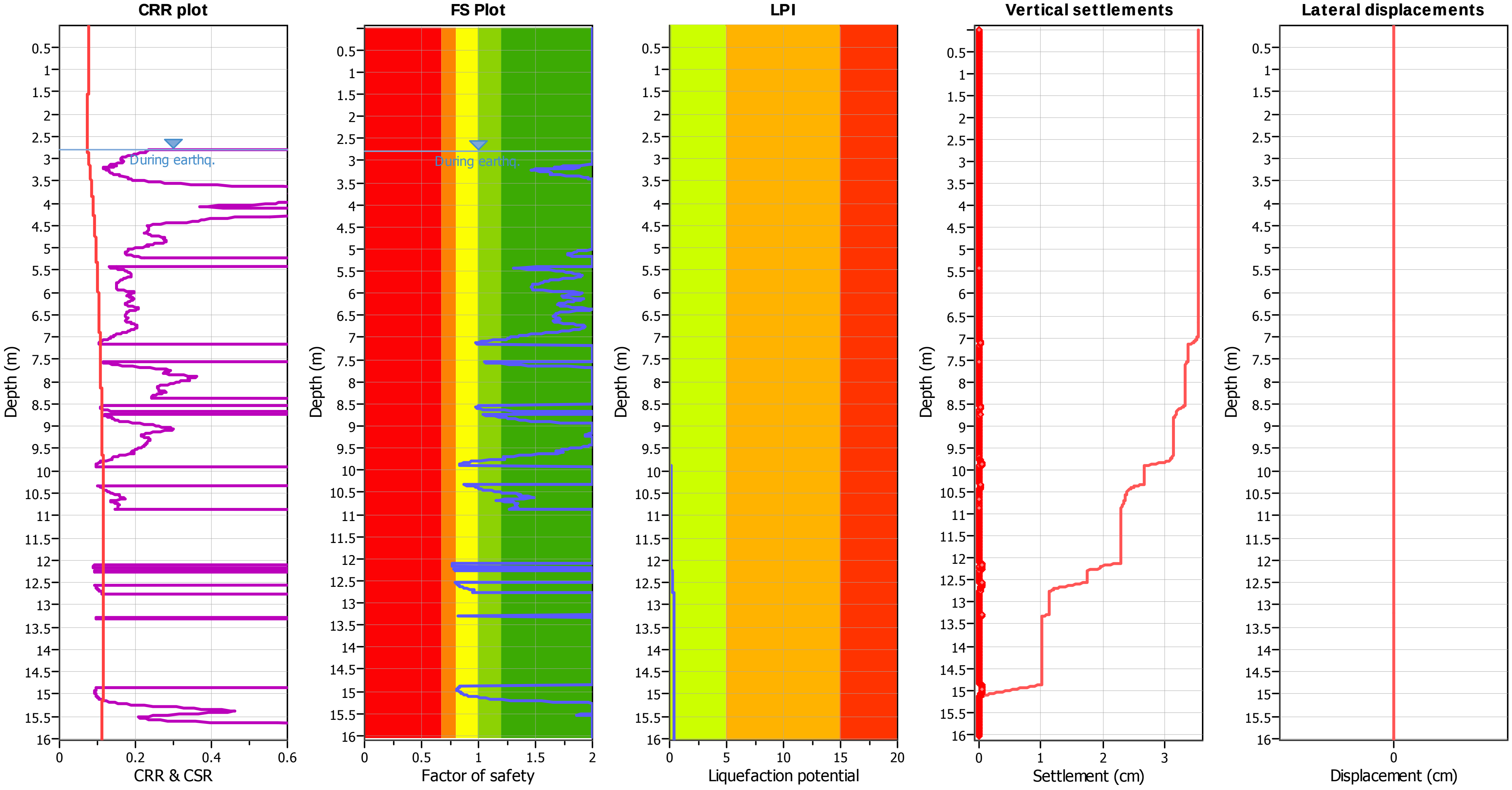
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.80 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.23	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟫 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.80 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.23	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

CMW Geosciences (NZ) Ltd
Suite 2, 5 Hill St, Hamilton 3204
PO Box 995, Waikato Mail Centre, Hamilton 3204
www.cmwgeosciences.co.nz

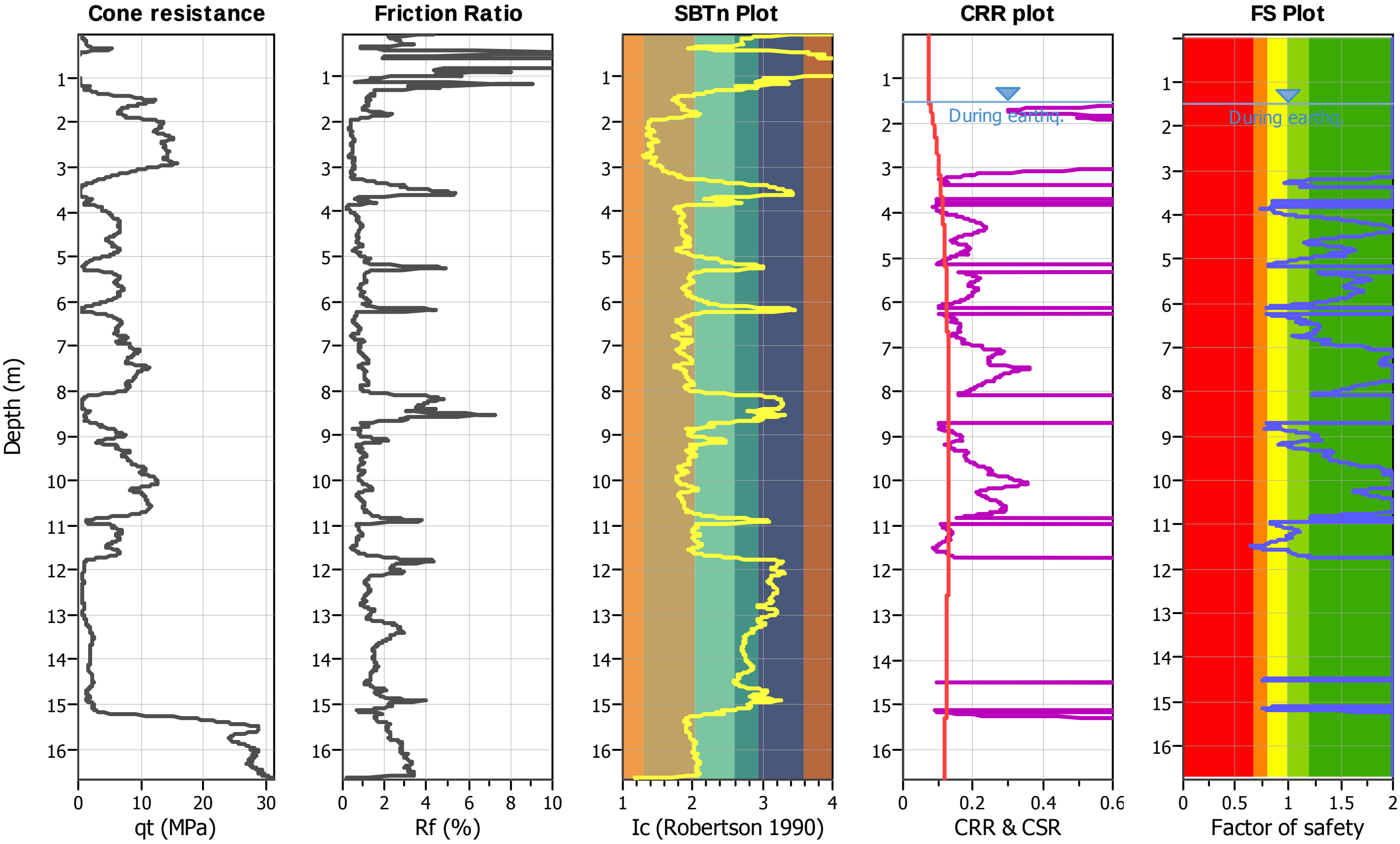
LIQUEFACTION ANALYSIS REPORT

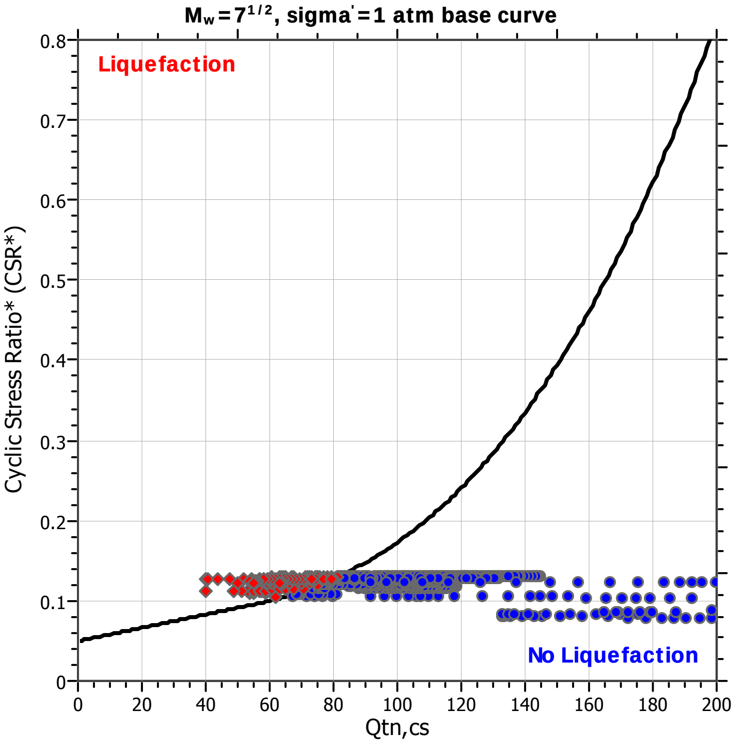
Project title : Te Rapa Racesourse Redevelopment **Location : Hamilton**

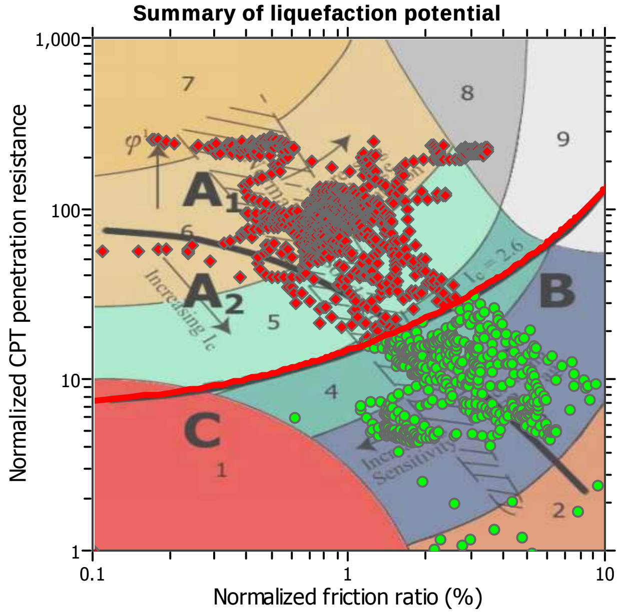
CPT file : CPT02 ULS

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.75	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.23	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

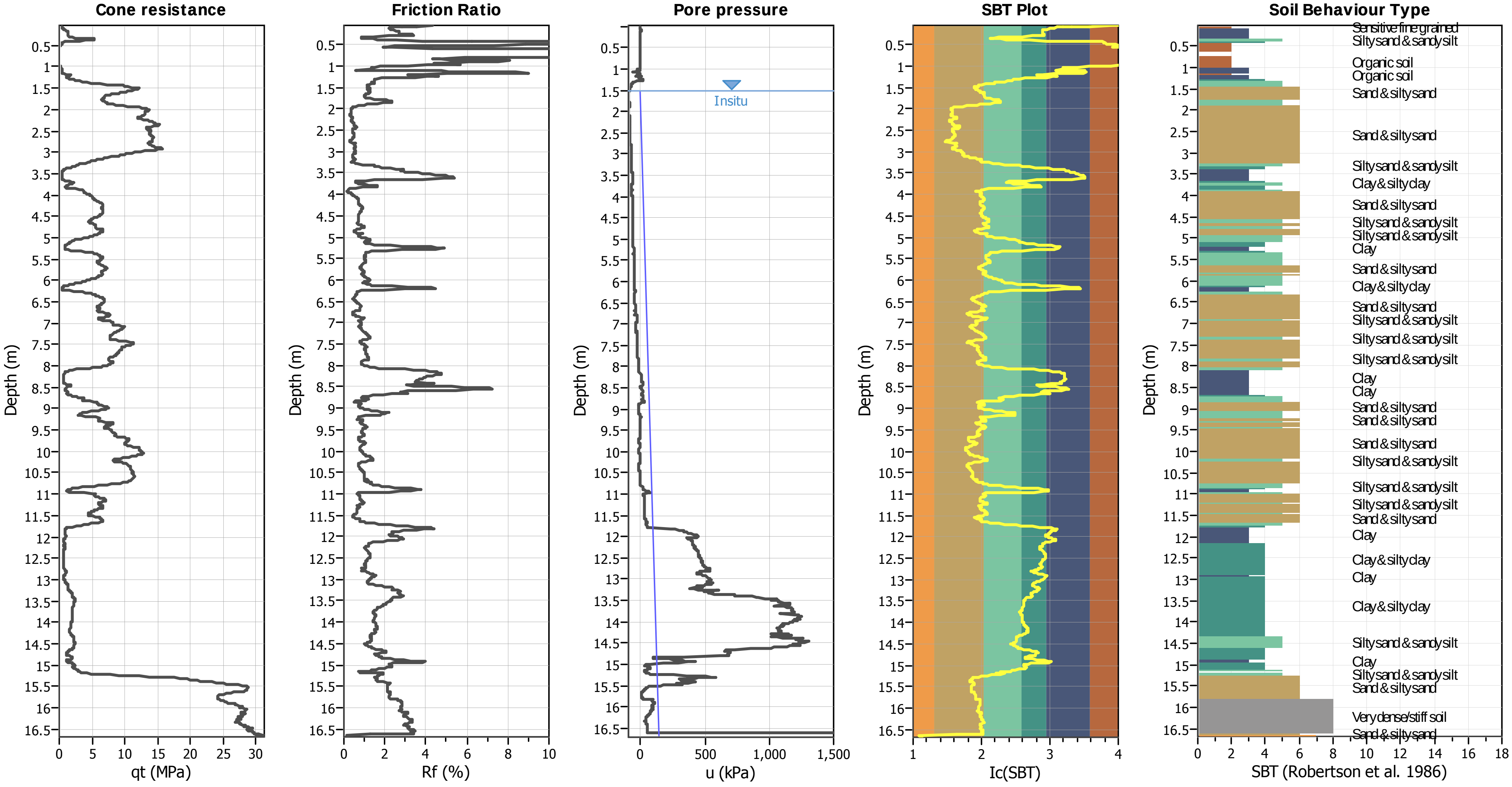






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



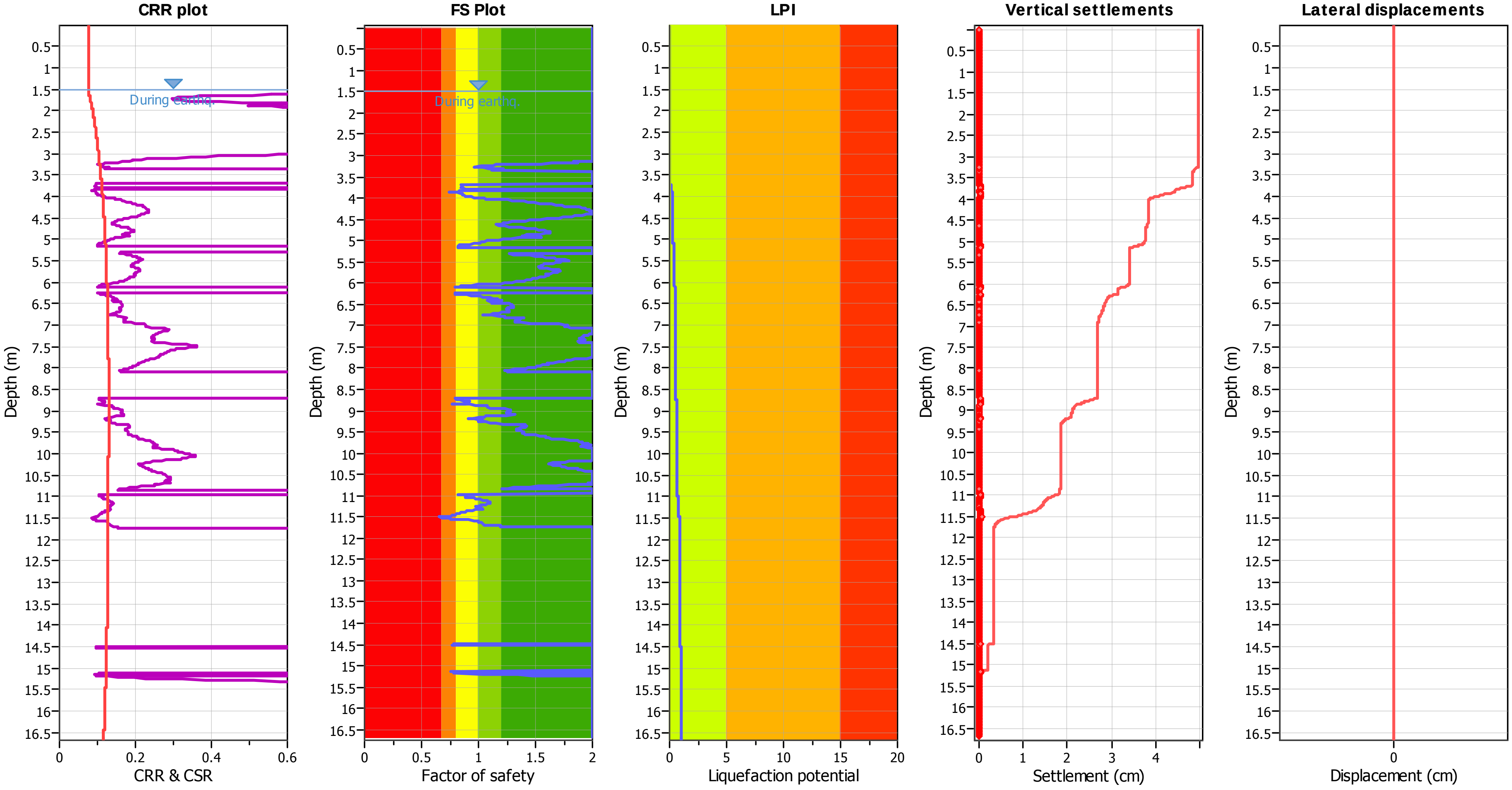
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.23	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬜ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.23	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

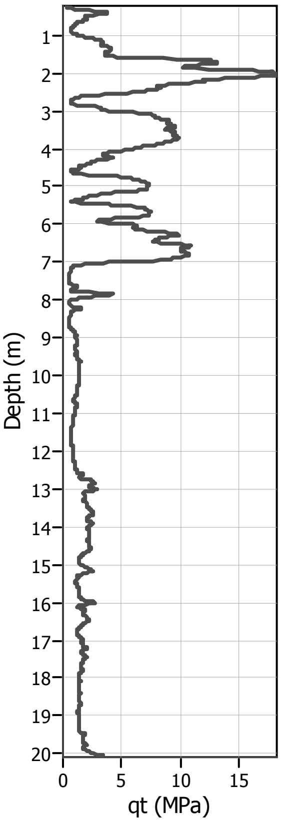
LIQUEFACTION ANALYSIS REPORT

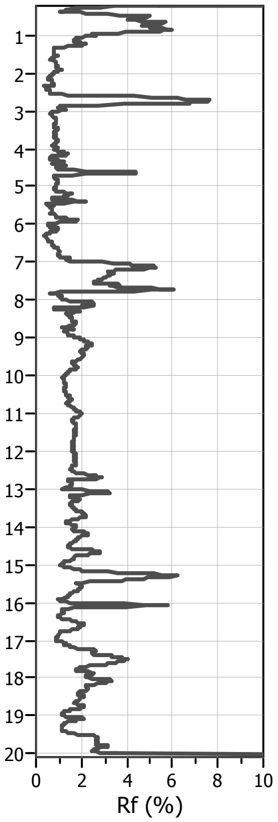
Project title : Te Rapa Racesourse Redevelopment **Location : Hamilton**

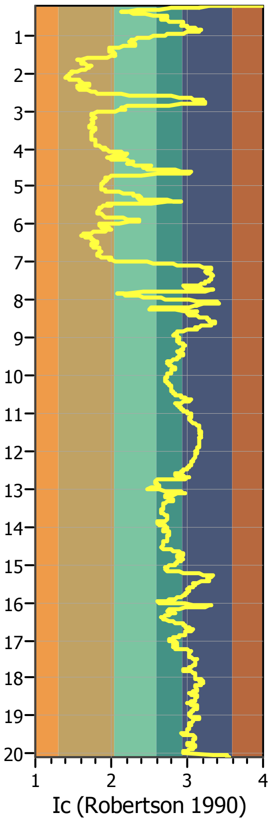
CPT file : CPT03 ULS

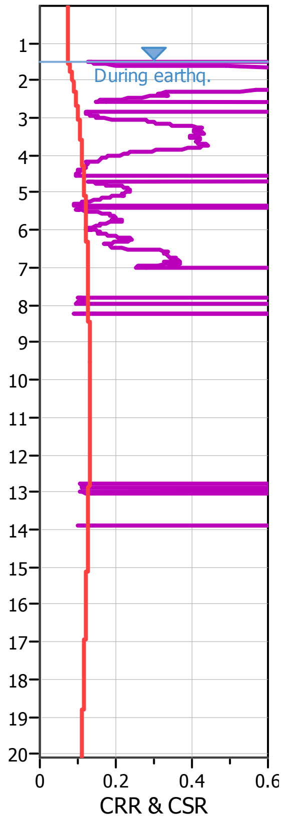
Input parameters and analysis data

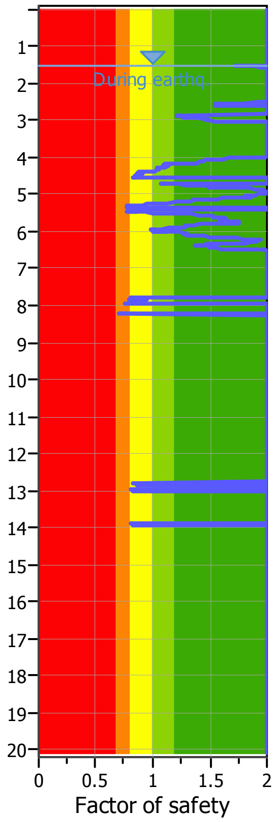
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.75	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.23	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

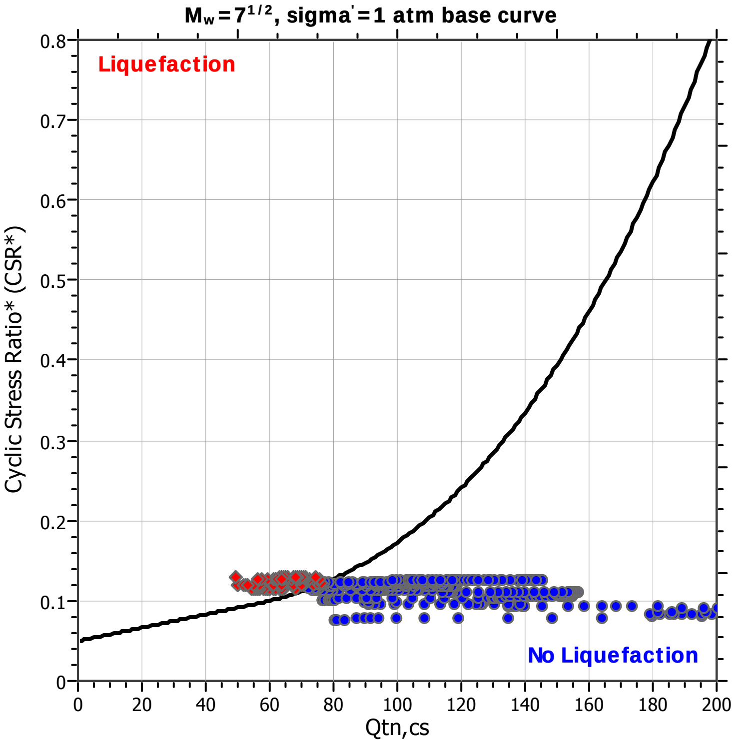

Cone resistance

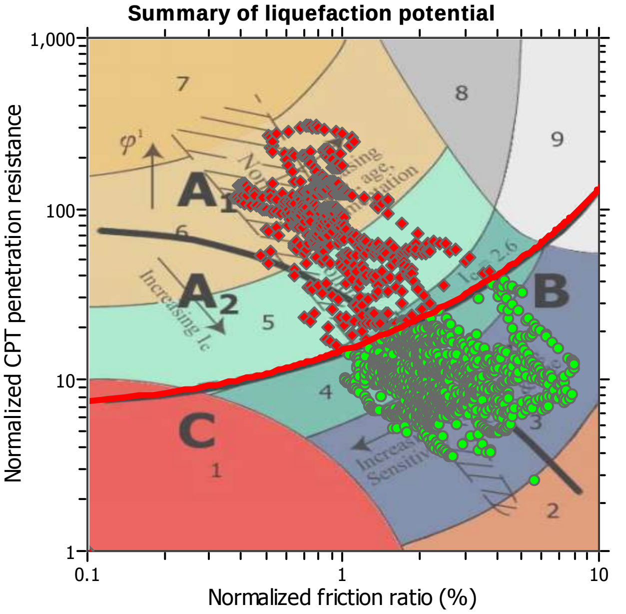

Friction Ratio


SBTn Plot


CRR plot

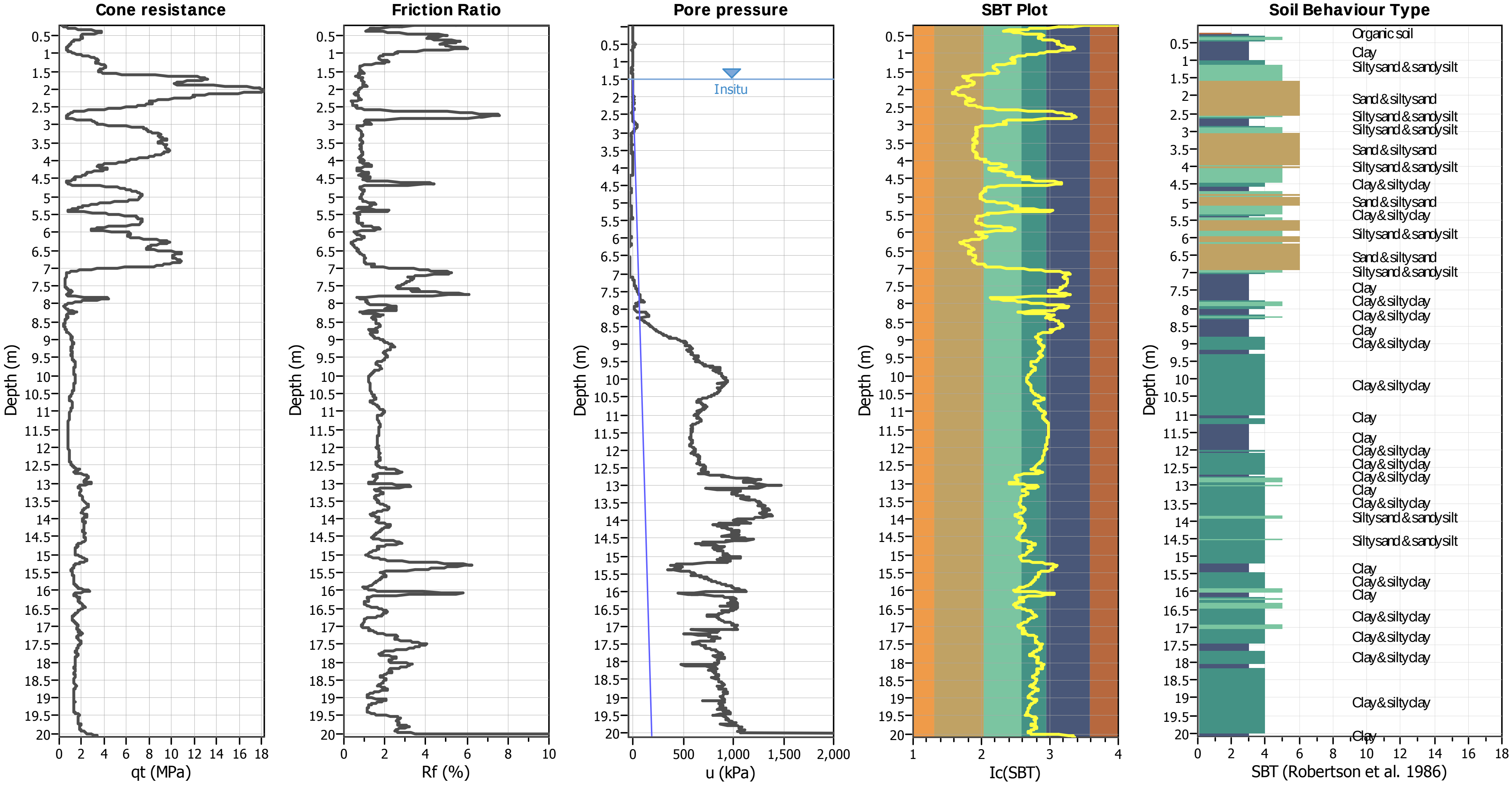

FS Plot


$M_w = 7^{1/2}$, sigma' = 1 atm base curve


Summary of liquefaction potential

Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



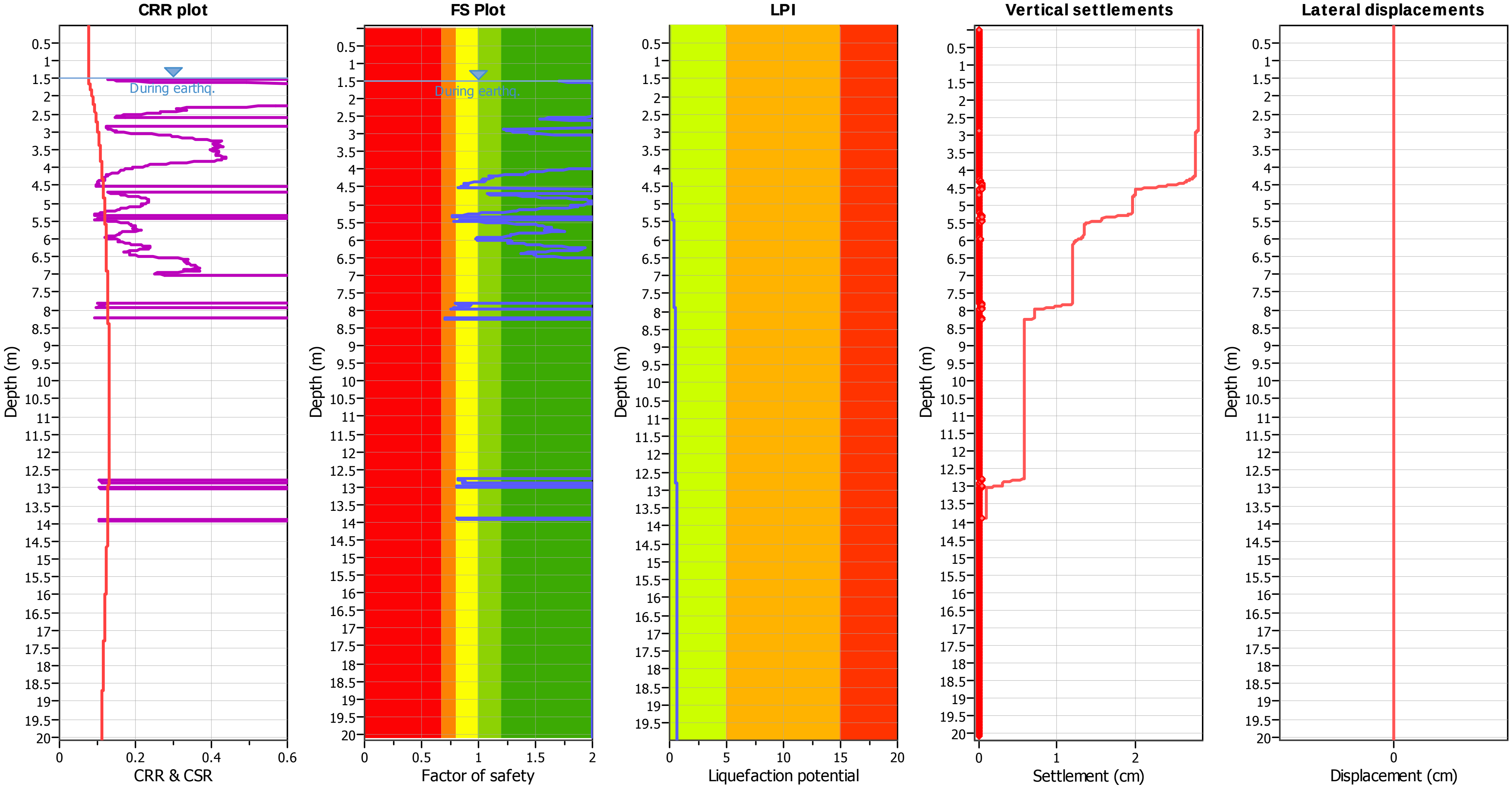
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.23	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.23	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

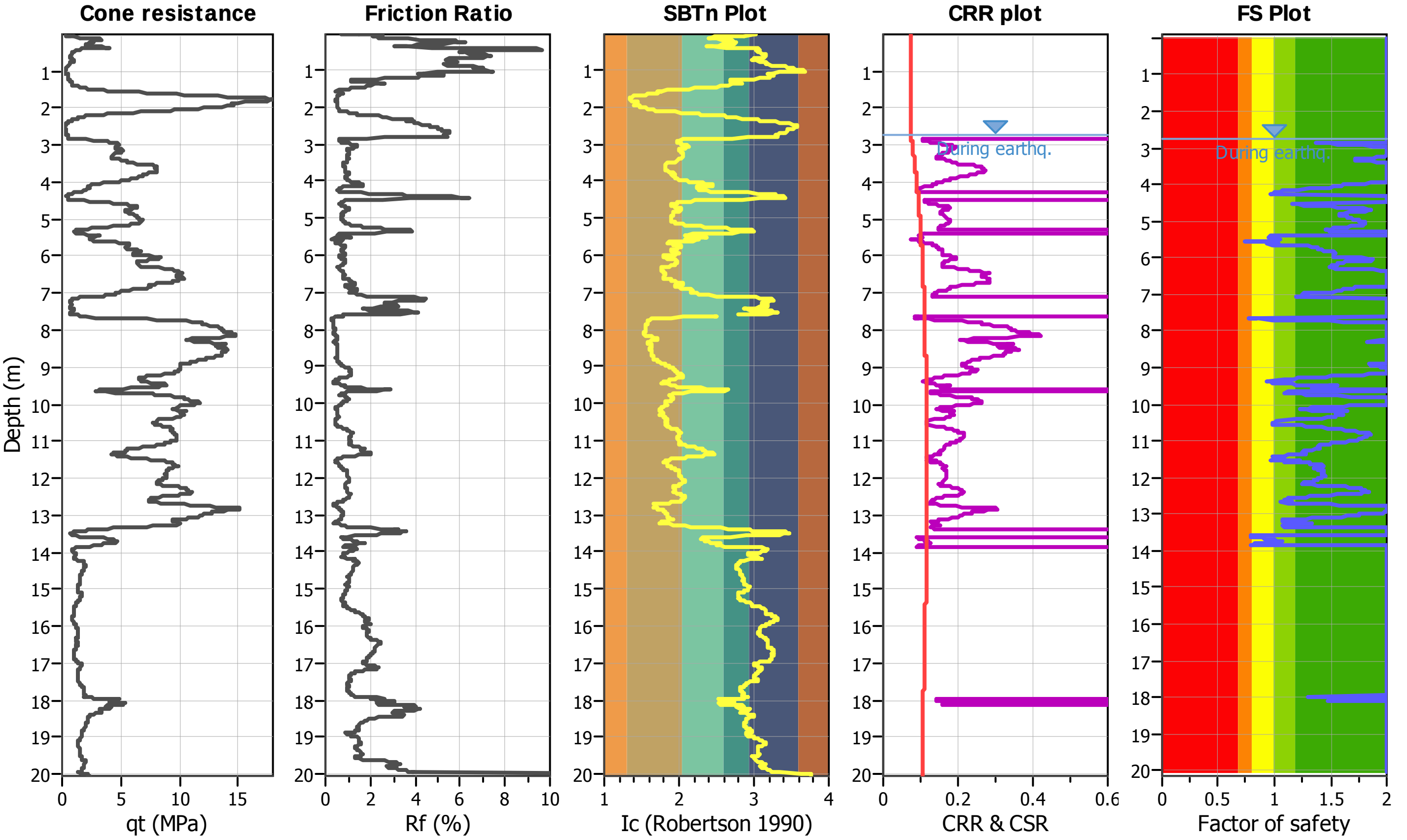
LIQUEFACTION ANALYSIS REPORT

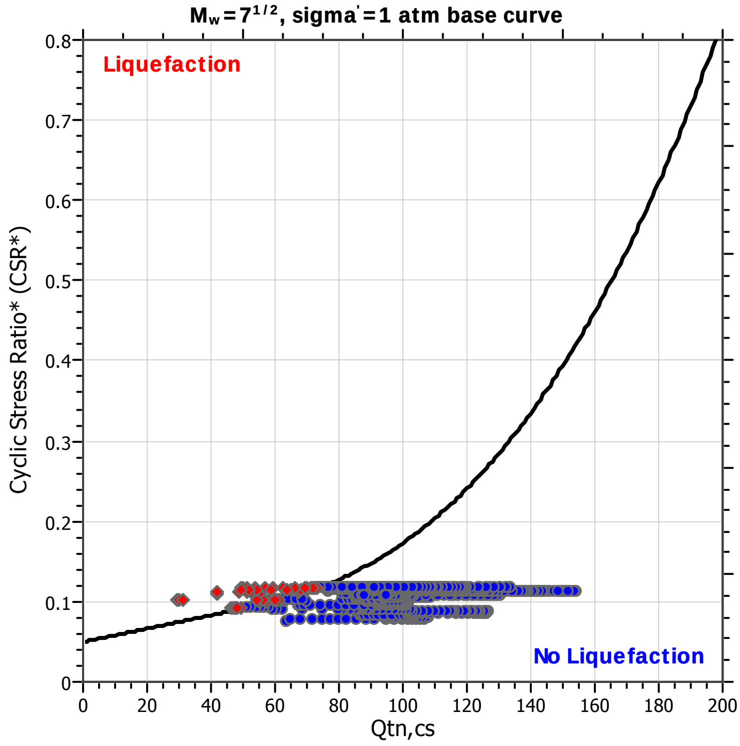
Project title : Te Rapa Racesourse Redevelopment **Location : Hamilton**

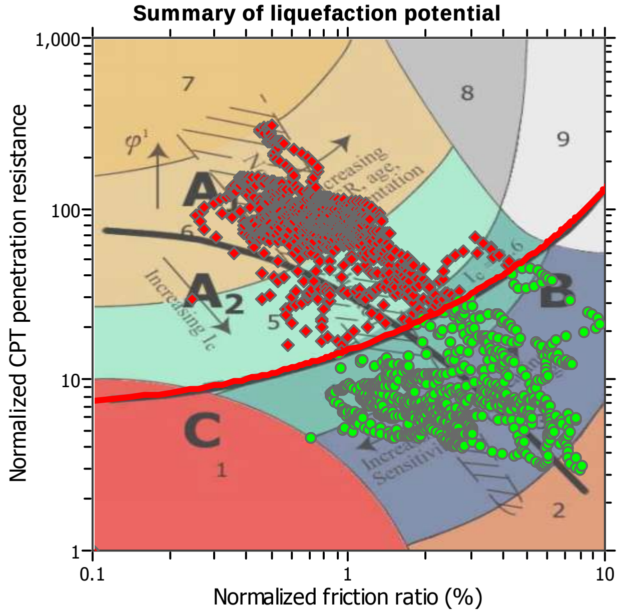
CPT file : CPT04 ULS

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.75 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.75 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	5.75	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.23	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

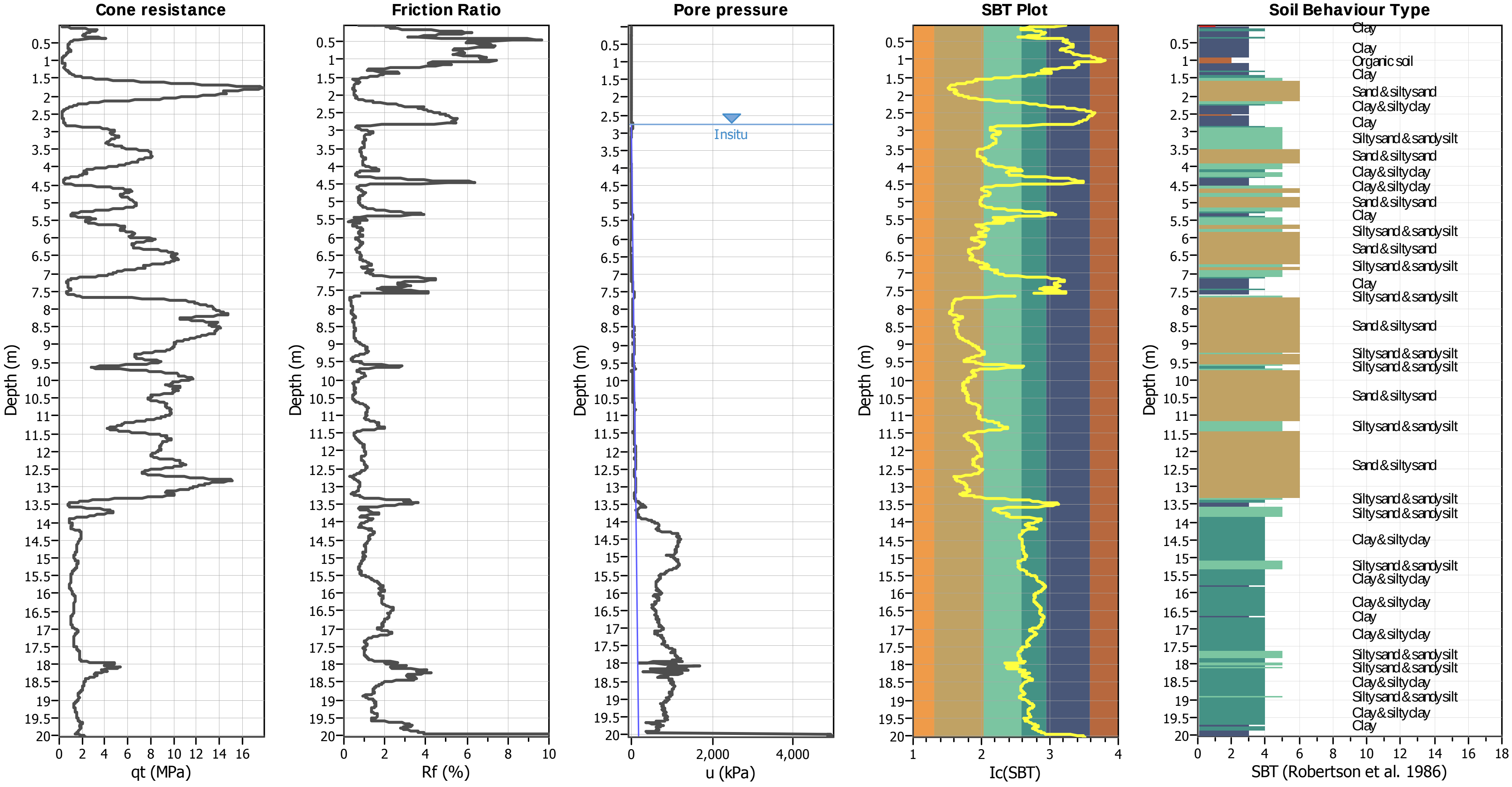






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



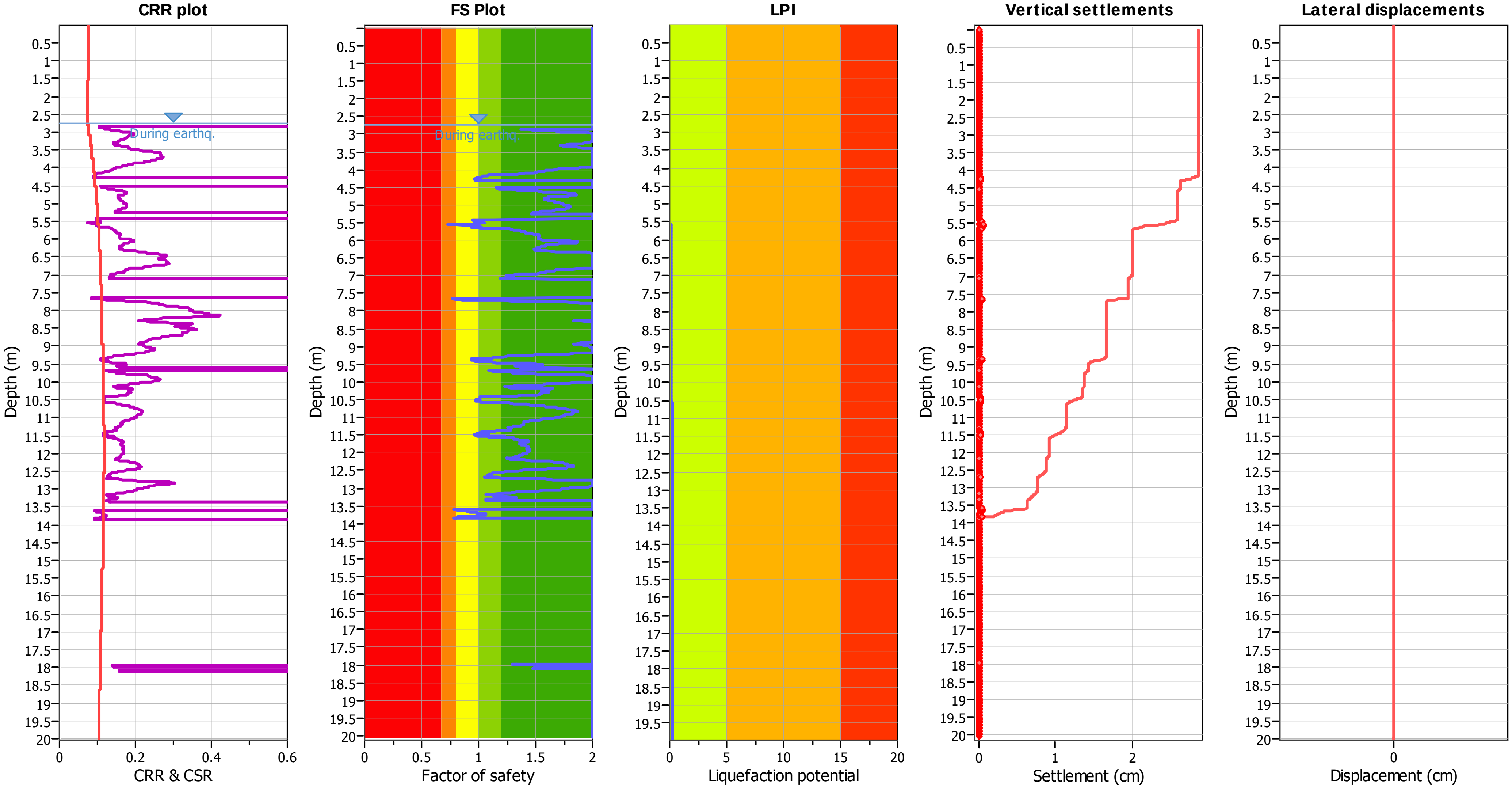
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.75 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.23	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.75 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.75 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.23	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.75 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

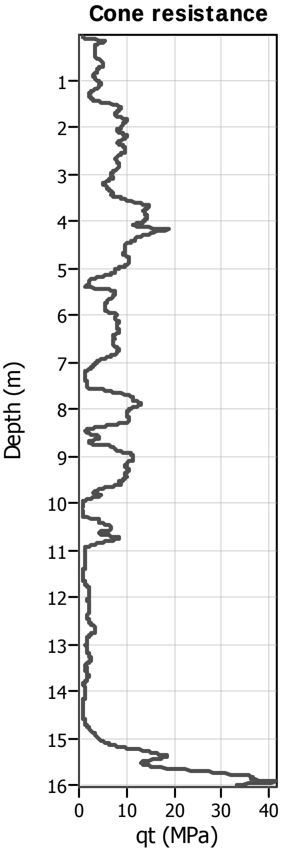
LIQUEFACTION ANALYSIS REPORT

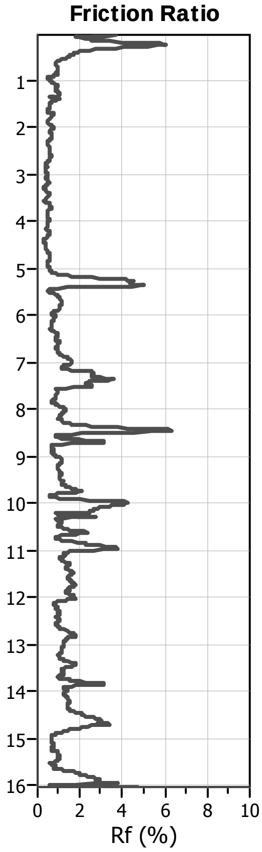
Project title : Te Rapa Racesourse Redevelopment **Location : Hamilton**

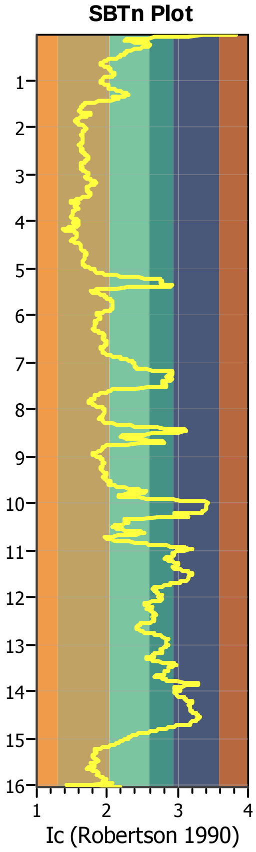
CPT file : CPT01 SLS

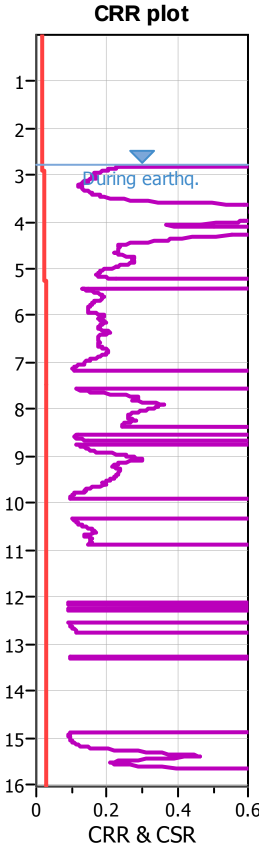
Input parameters and analysis data

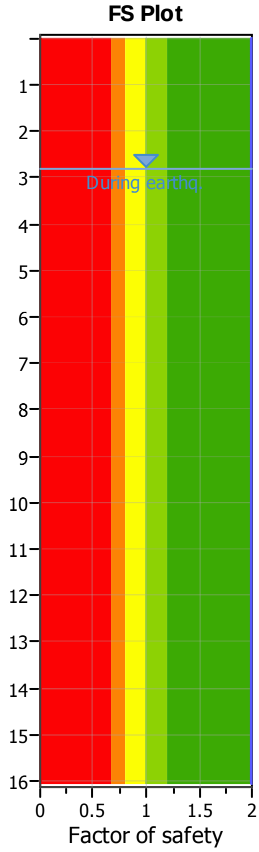
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.80 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.80 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.75	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.06	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

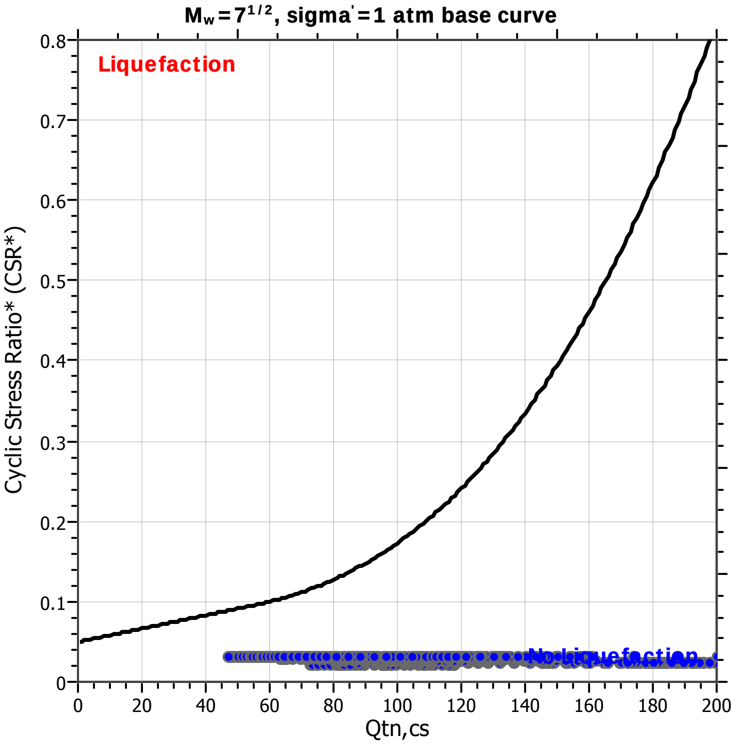


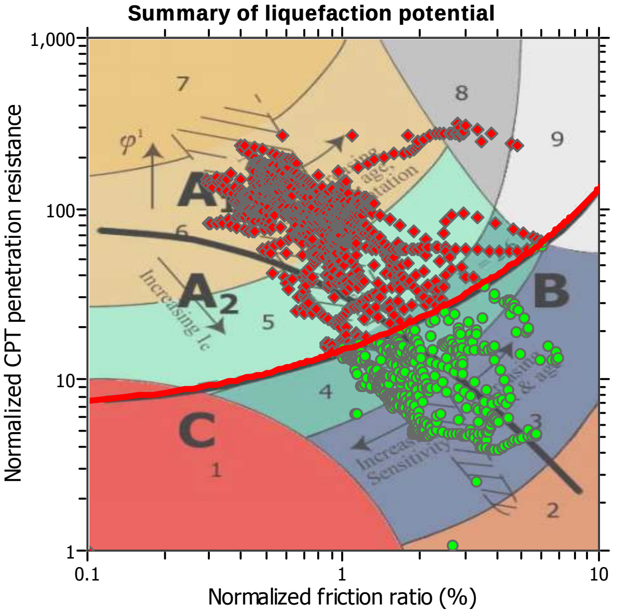






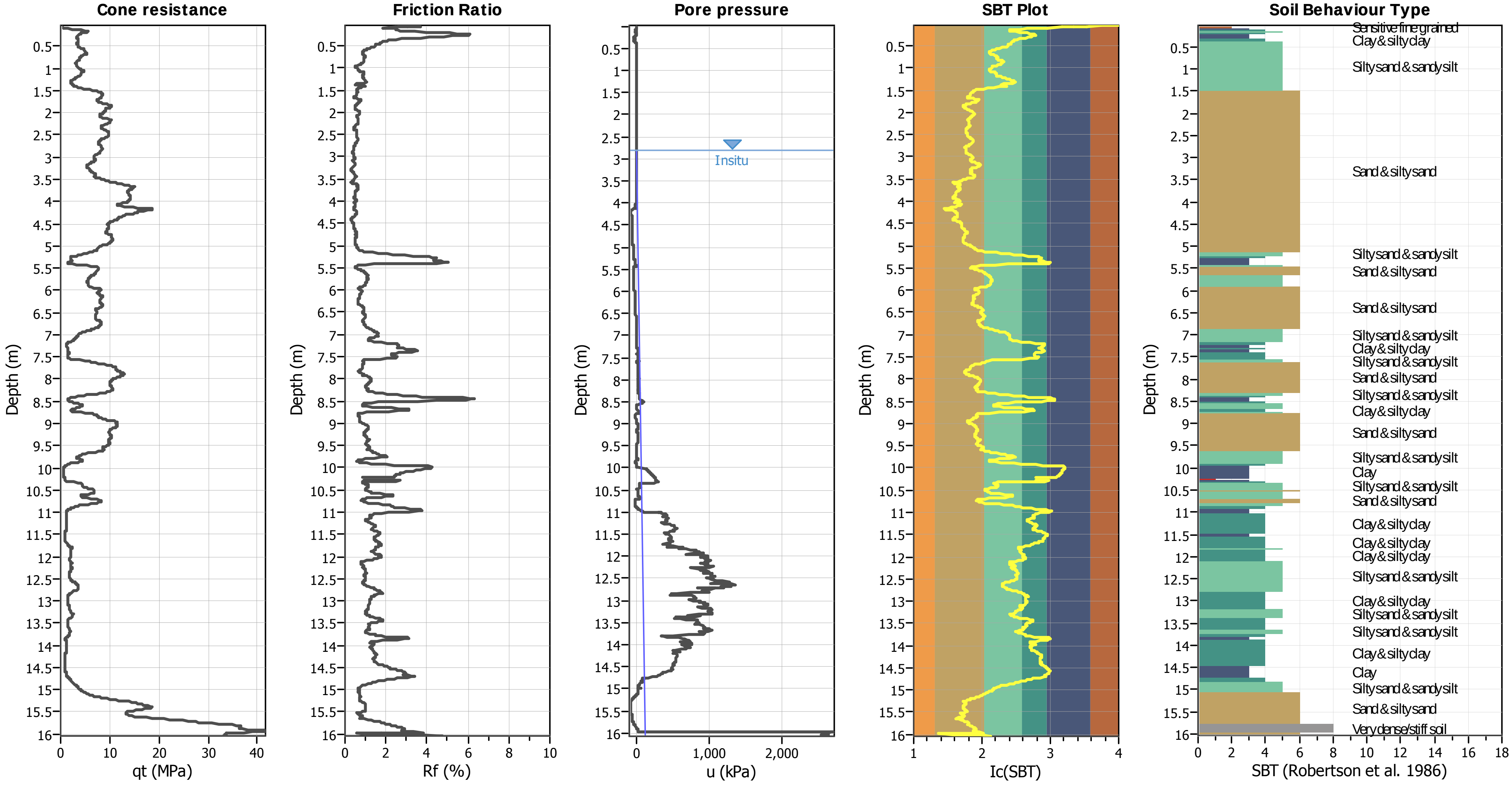






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



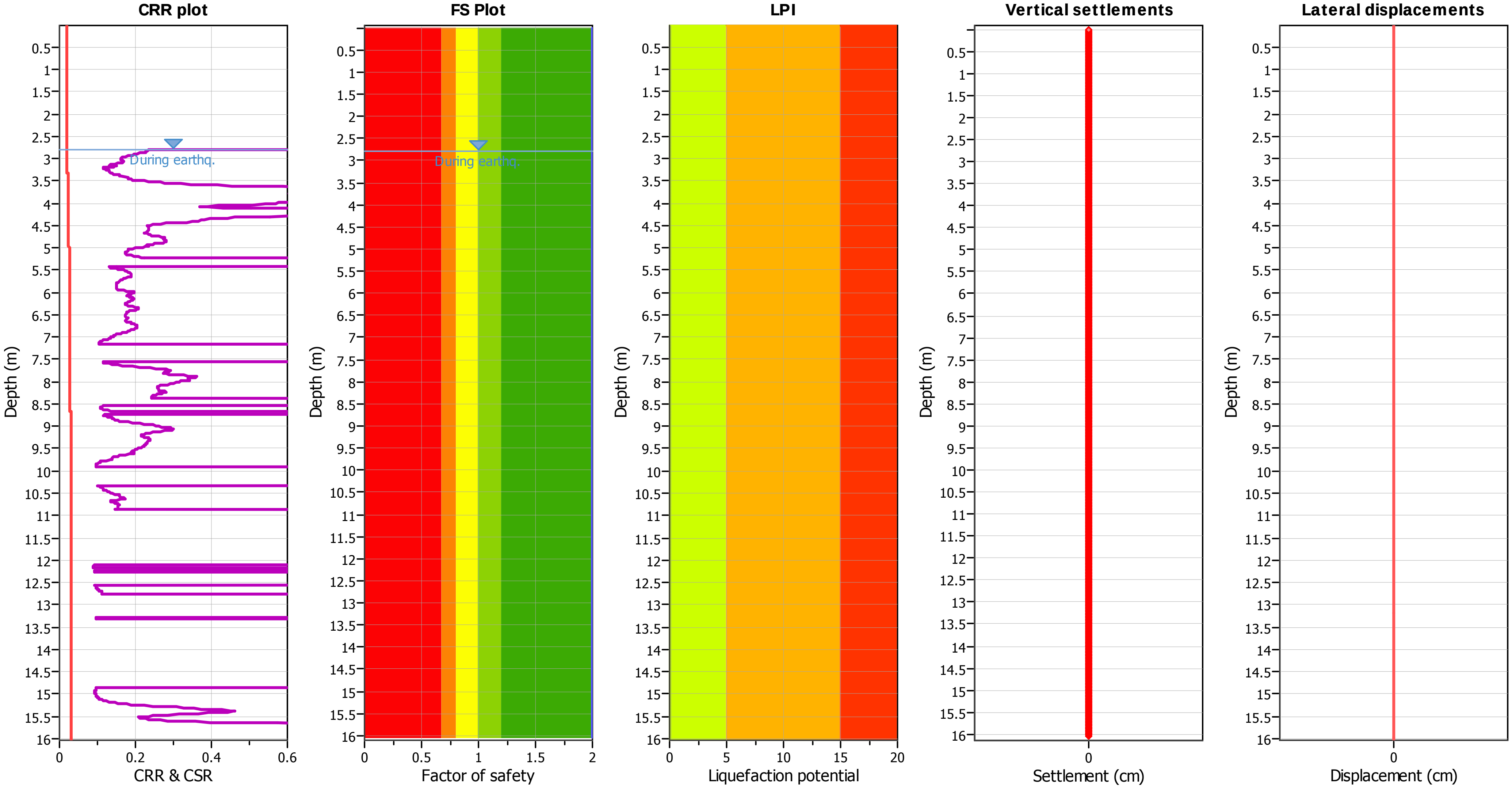
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.80 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟫 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.80 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

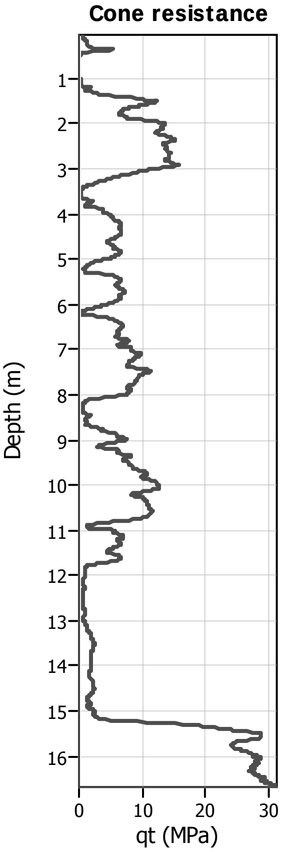
LIQUEFACTION ANALYSIS REPORT

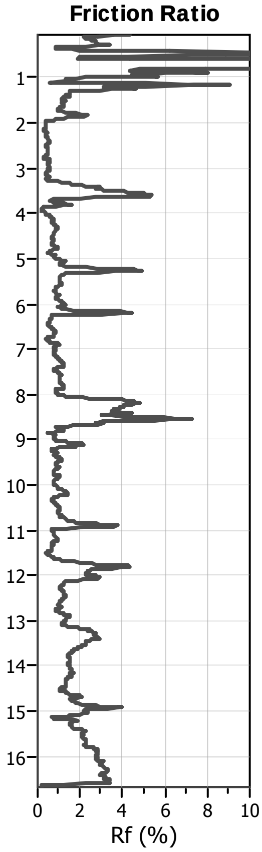
Project title : Te Rapa Racesourse Redevelopment **Location : Hamilton**

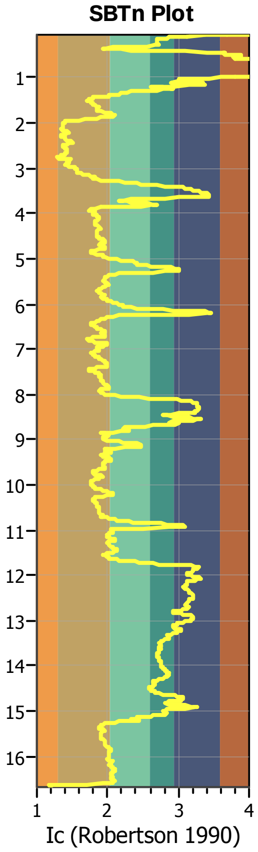
CPT file : CPT02 SLS

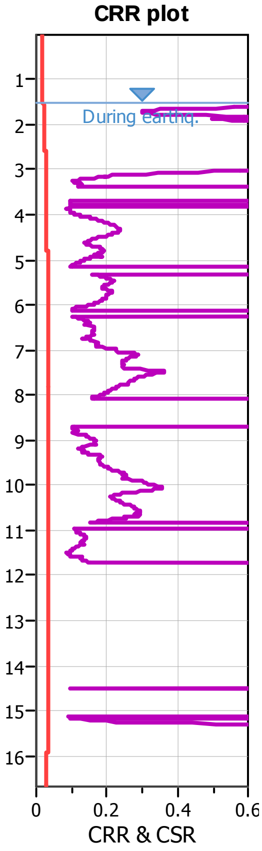
Input parameters and analysis data

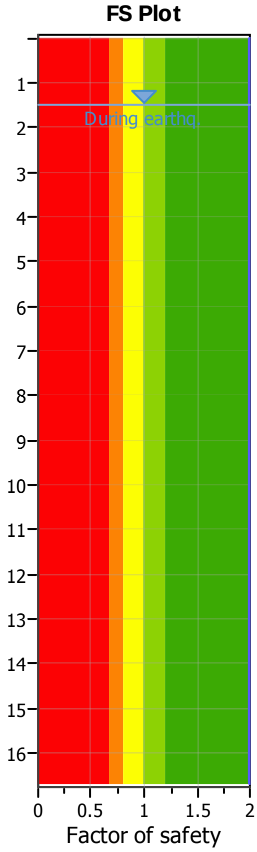
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	5.75	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.06	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

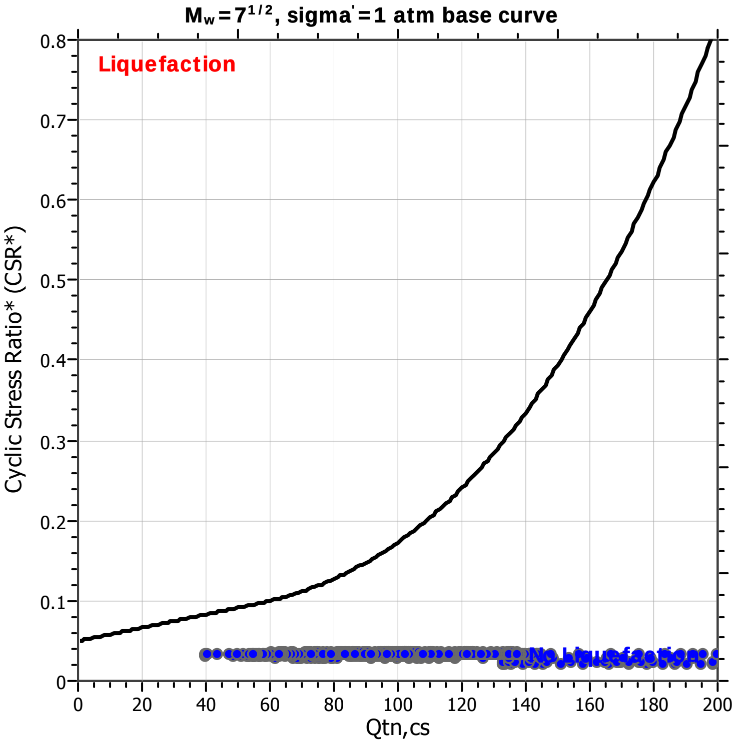

Cone resistance

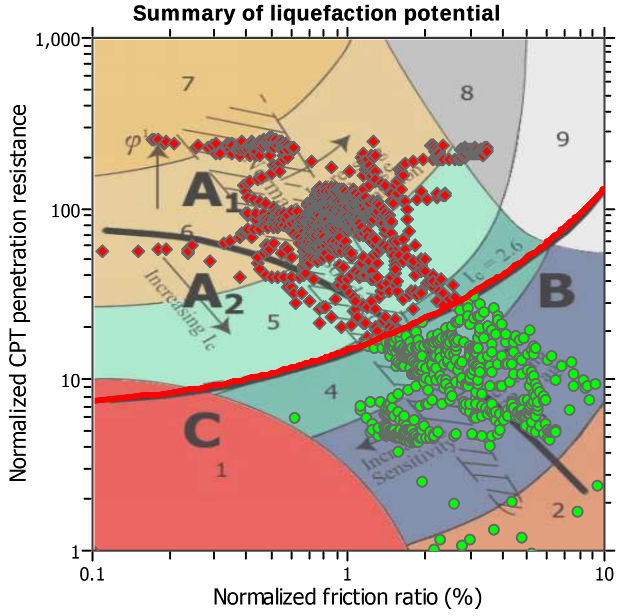

Friction Ratio


SBTn Plot


CRR plot

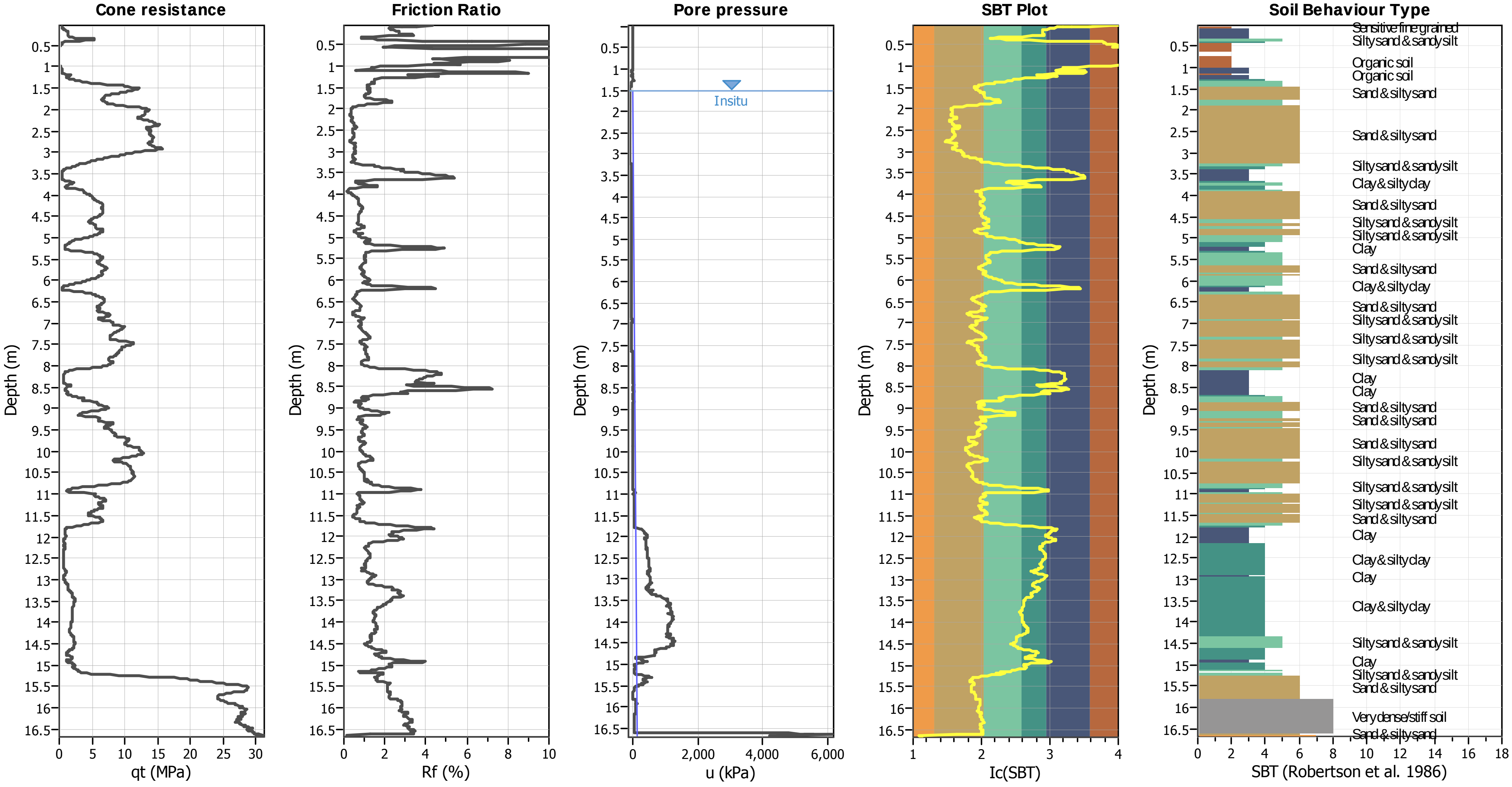

FS Plot


$M_w = 7^{1/2}$, sigma'=1 atm base curve


Summary of liquefaction potential

Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



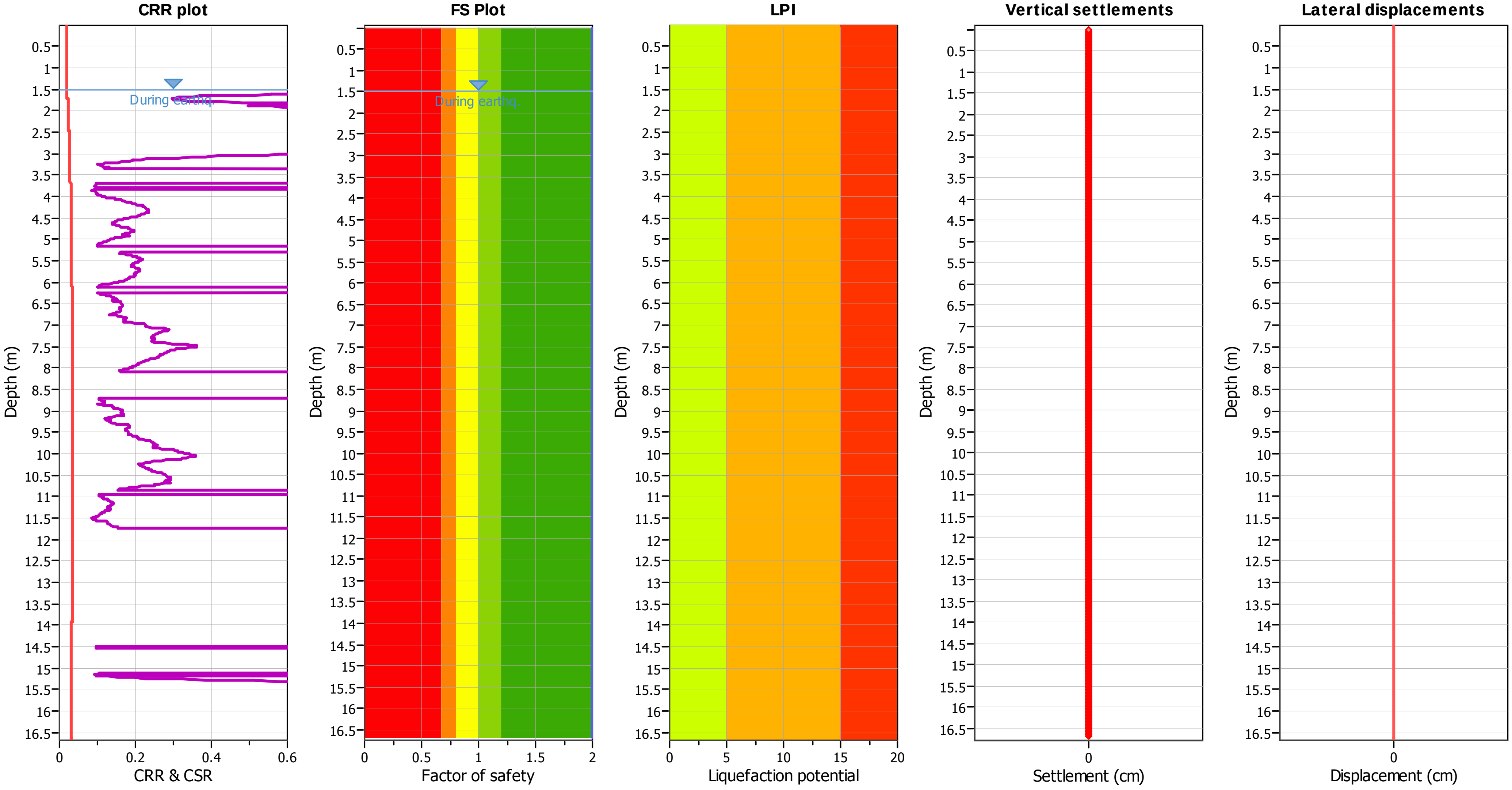
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

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PO Box 995, Waikato Mail Centre, Hamilton 3204
www.cmwgeosciences.co.nz

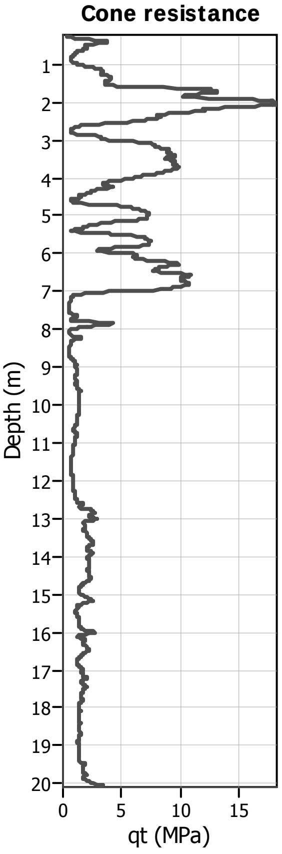
LIQUEFACTION ANALYSIS REPORT

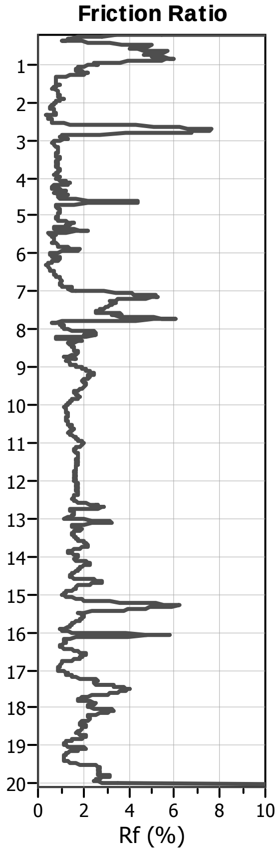
Project title : Te Rapa Racesourse Redevelopment **Location : Hamilton**

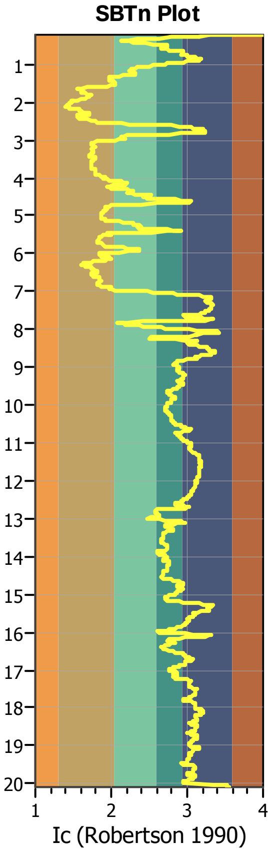
CPT file : CPT03 SLS

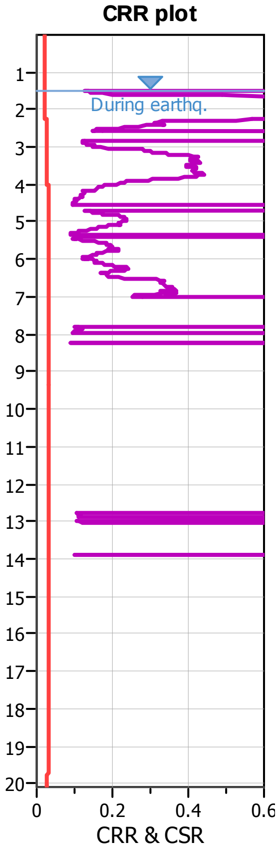
Input parameters and analysis data

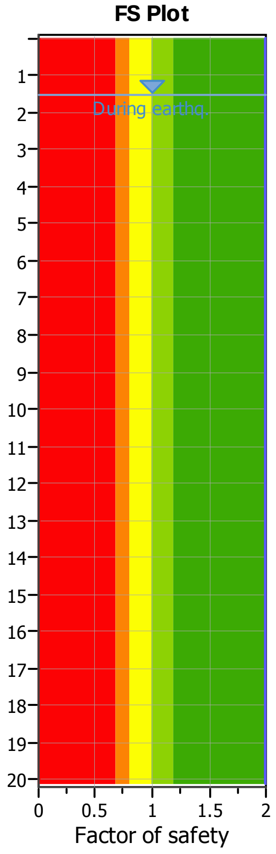
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.75	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.06	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

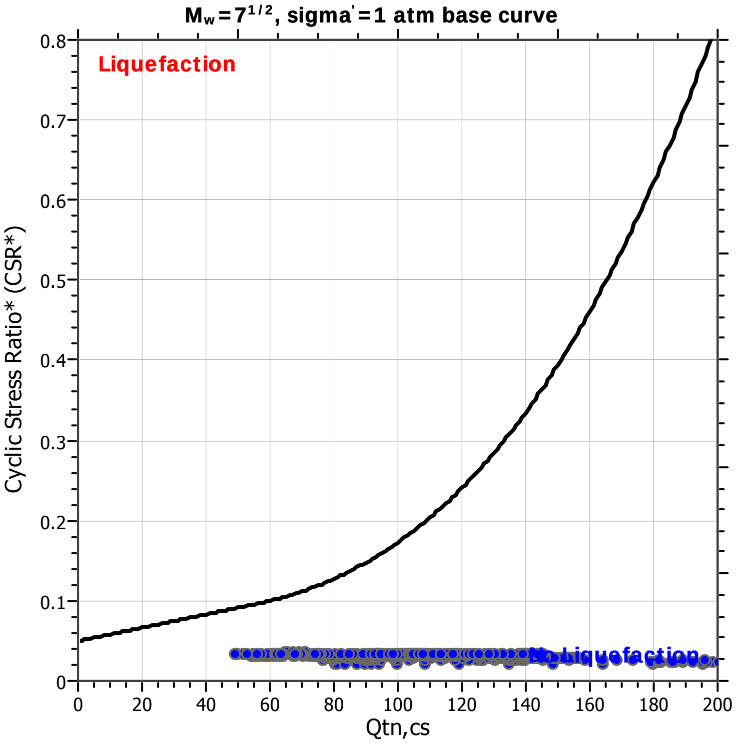


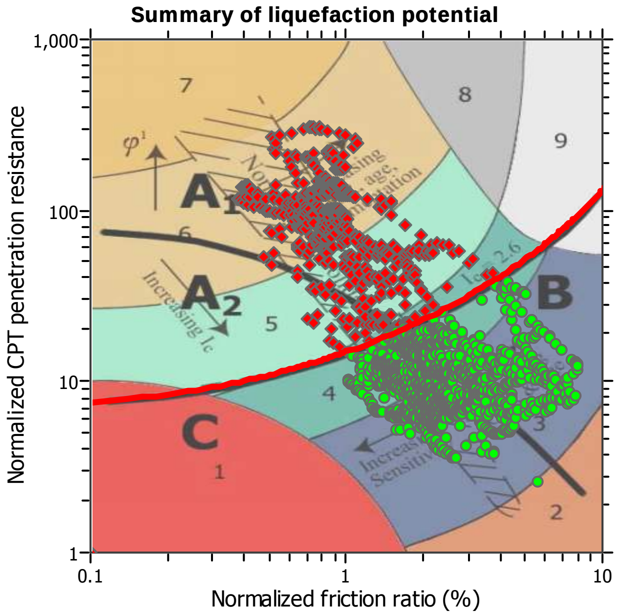






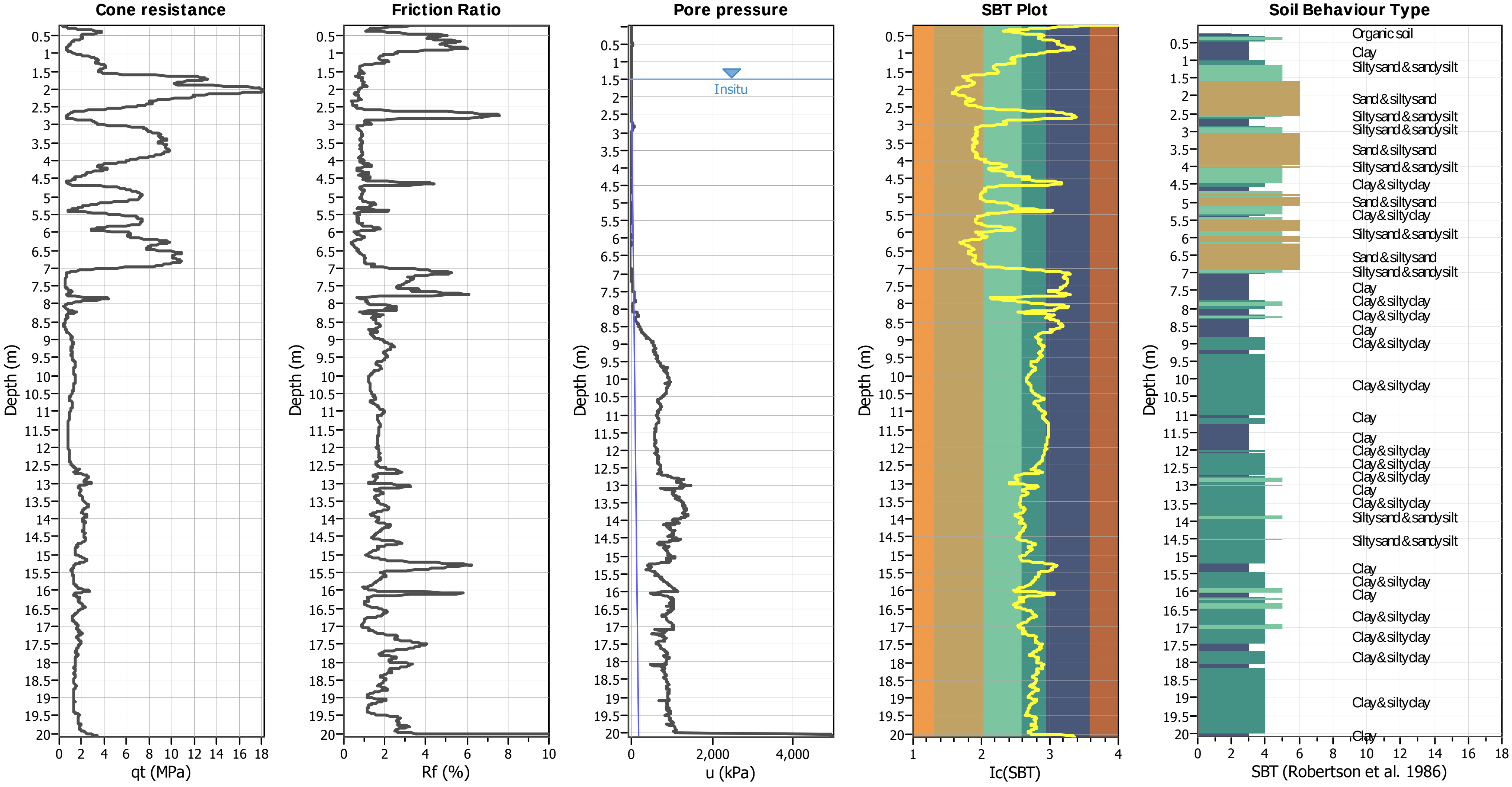






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 5.75
Peak ground acceleration: 0.06
Depth to water table (insitu): 1.50 m

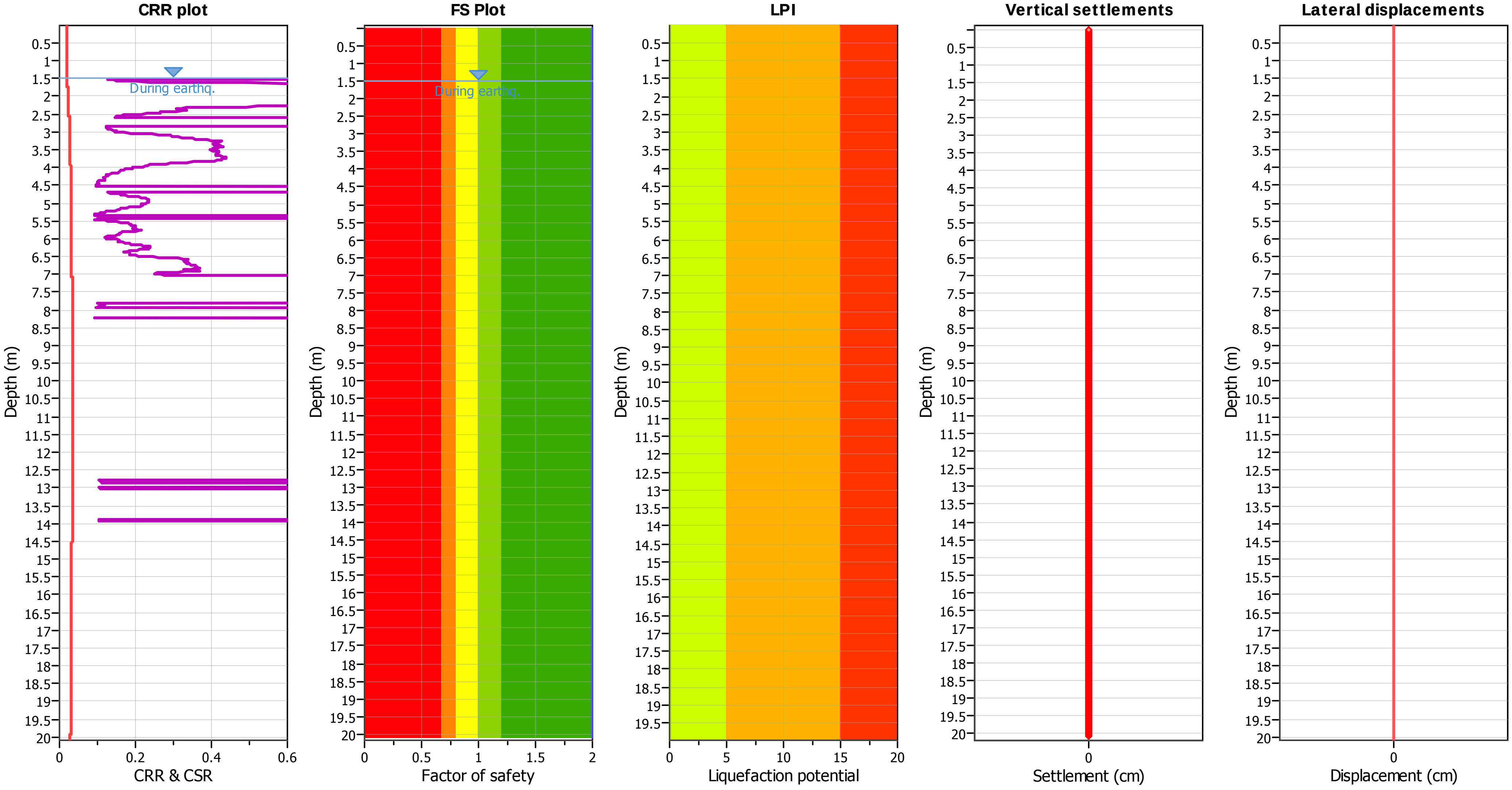
Depth to water table (erthq.): 1.50 m
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: No
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

LIQUEFACTION ANALYSIS REPORT

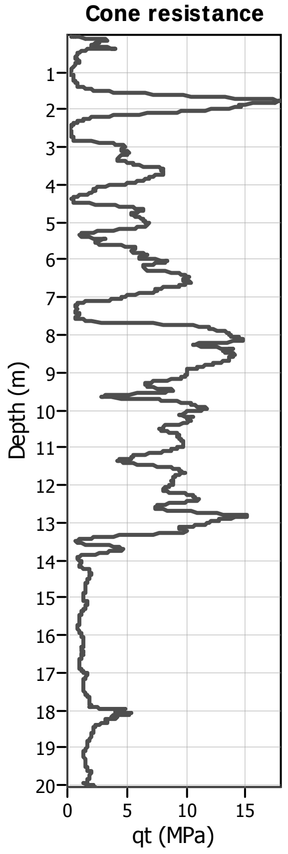
Project title : Te Rapa Racesourse Redevelopment **Location : Hamilton**

CPT file : CPT04 SLS

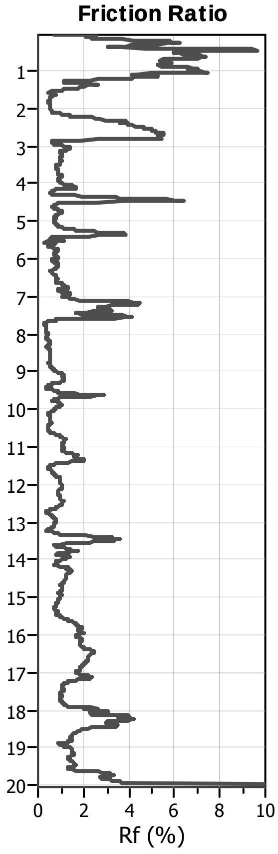
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.75 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.75 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	5.75	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.06	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

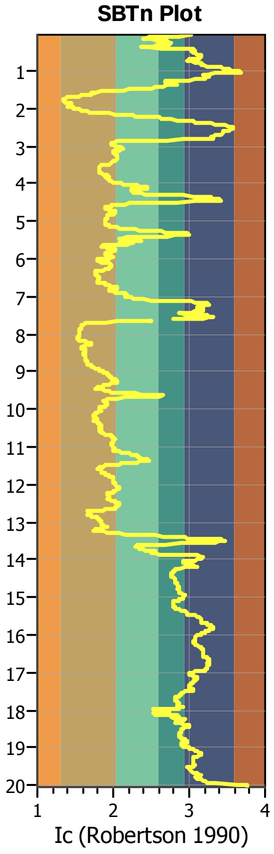
Cone resistance



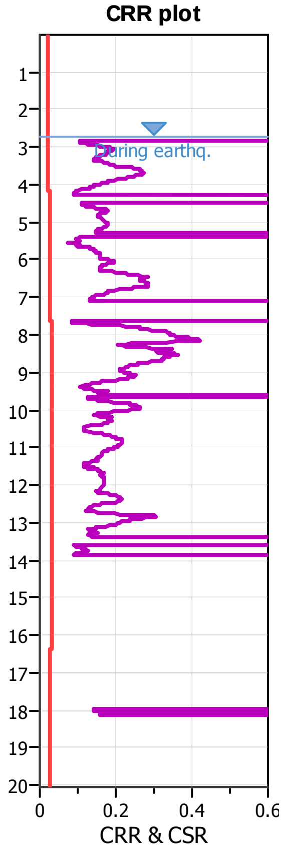
Friction Ratio



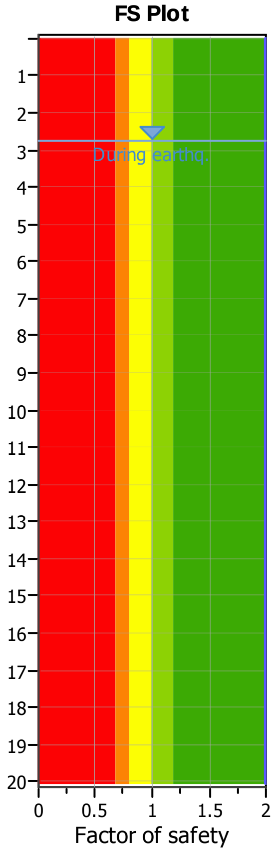
SBTn Plot



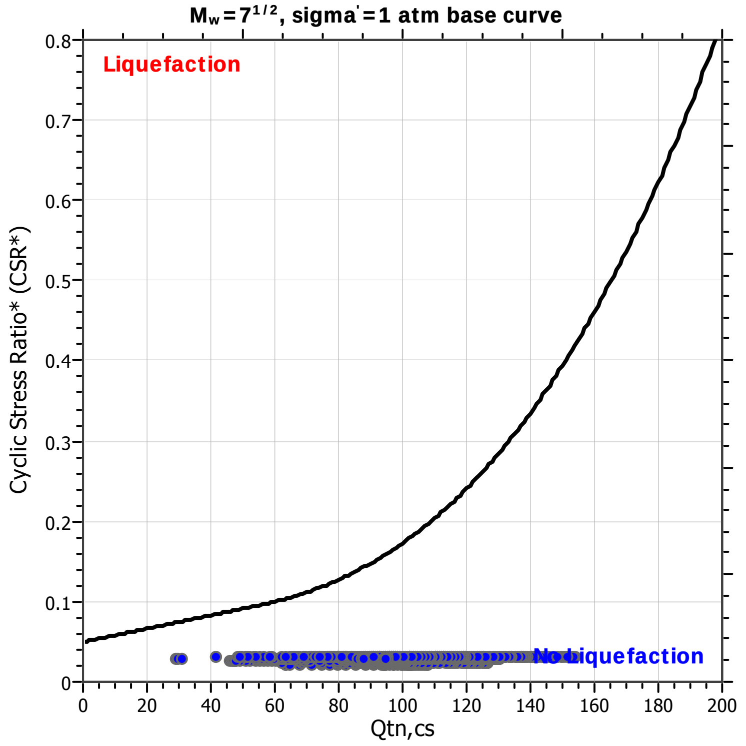
CRR plot



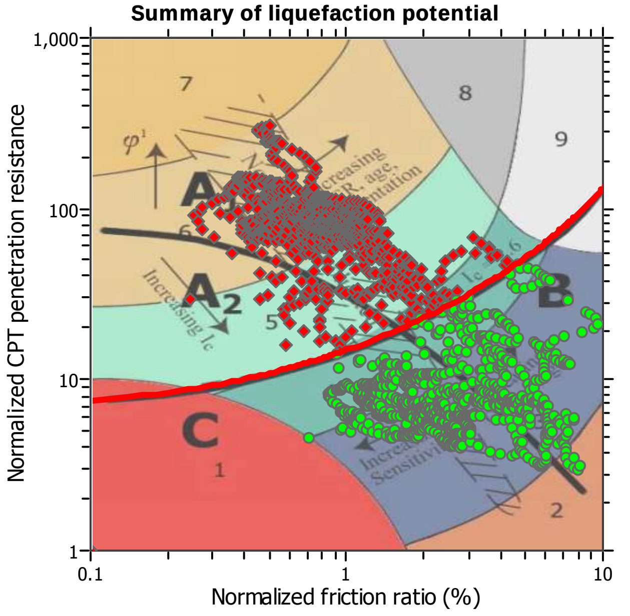
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

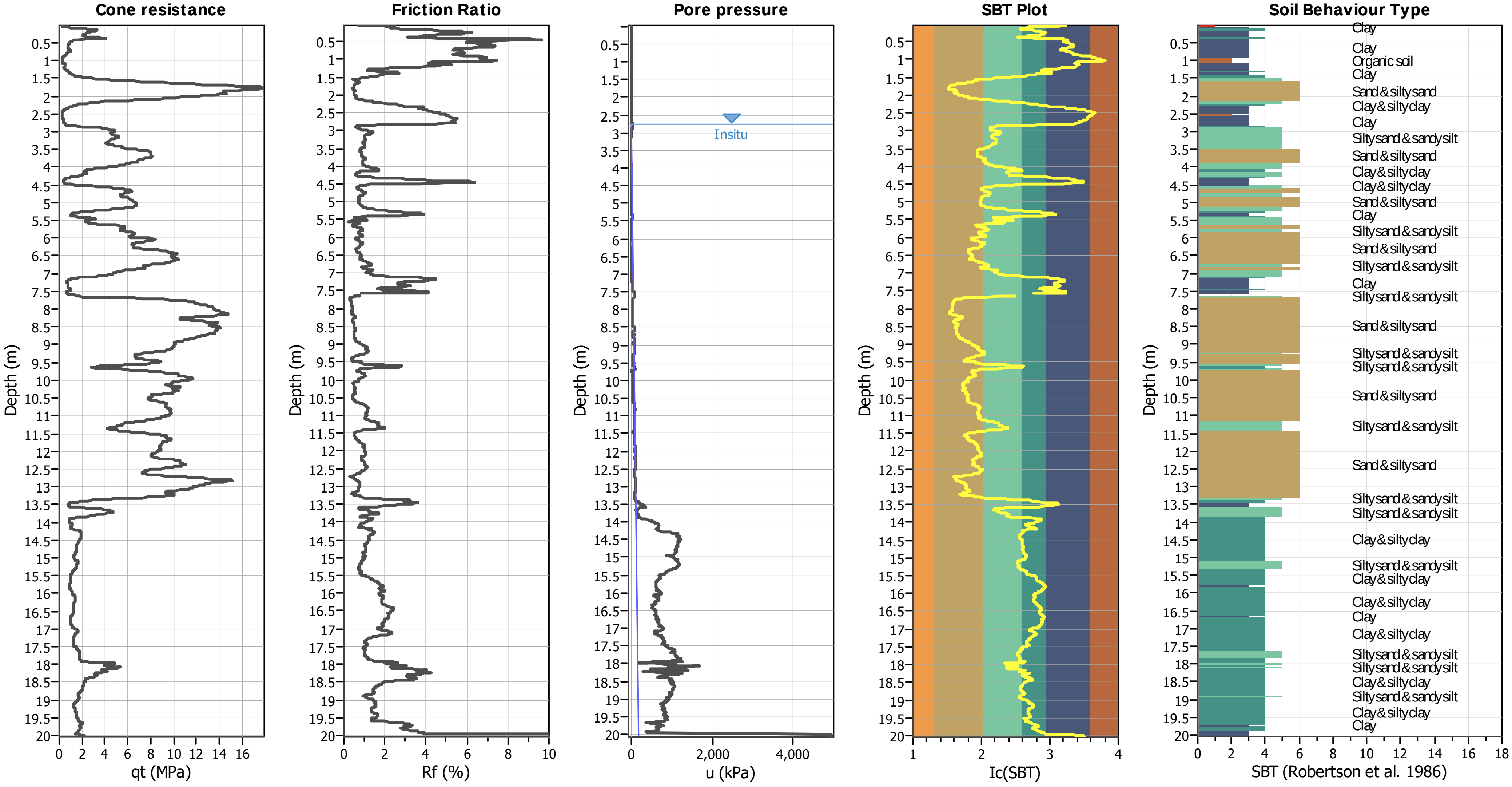


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



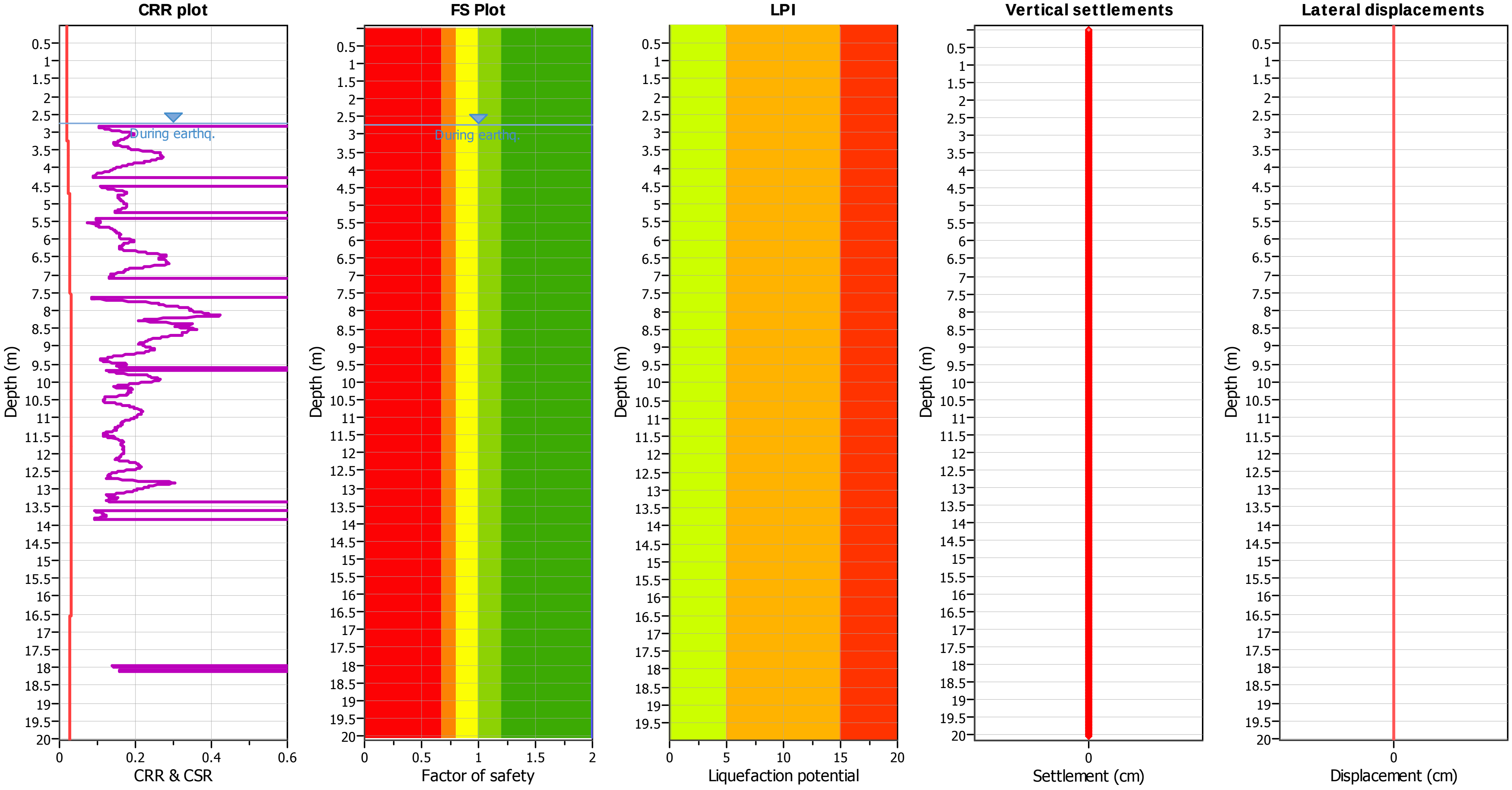
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.75 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.75 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	2.75 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.75 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

APPENDIX D: SETTLEMENT OUTPUT ANALYSIS

Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT01

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		80	kN/m^2
Depth to watertable from ground surface (h_f)		2.8	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 8 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
1.0	30.0	0.5	30.00	**CONTINUOUS**

Footing shape if L/B = 1 Circular or Square Footing (SQU)

if L/B ≥ 10 Continuous Footings (CON)

Depth of influence = 4.5 metres

Circular or Square Shape = Df + 2B = 2.5 metres

Continuous Shape = Df + 4B = 4.5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 8.00 kN/m^2

Where watertable is below base of footing (Df < hf) : σ'_{vo} = (γ_s x D_f)

Where watertable is above base of footing (Df > hf) : σ'_{vo} = (γ_s x h_f) + (γ_s - γ_w) x (D_f - h_f)

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 24.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 16.00 kN/m^2

For h_f < D_f + B/2 $\sigma'_{zp(squ)}$ = (γ_s x h_f) + (γ_s - γ_w) x (D_f + B/2 - h_f)

For h_f > D_f + B/2 $\sigma'_{zp(squ)}$ = γ_s x (D_f + B/2)

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 24.00 kN/m^2

For h_f < D_f + B $\sigma'_{zp(con)}$ = (γ_s x h_f) + (γ_s - γ_w) x (D_f + B - h_f)

For h_f > D_f + B $\sigma'_{zp(con)}$ = γ_s x (D_f + B)

Peak strain influence factor (I_{zp}) $I_{zp(squ)}$ = 0.5 + 0.1 √ (q' / $\sigma'_{zp(squ)}$) = 0.7121

$I_{zp(con)}$ = 0.5 + 0.1 √ (q' / $\sigma'_{zp(con)}$) = 0.6732

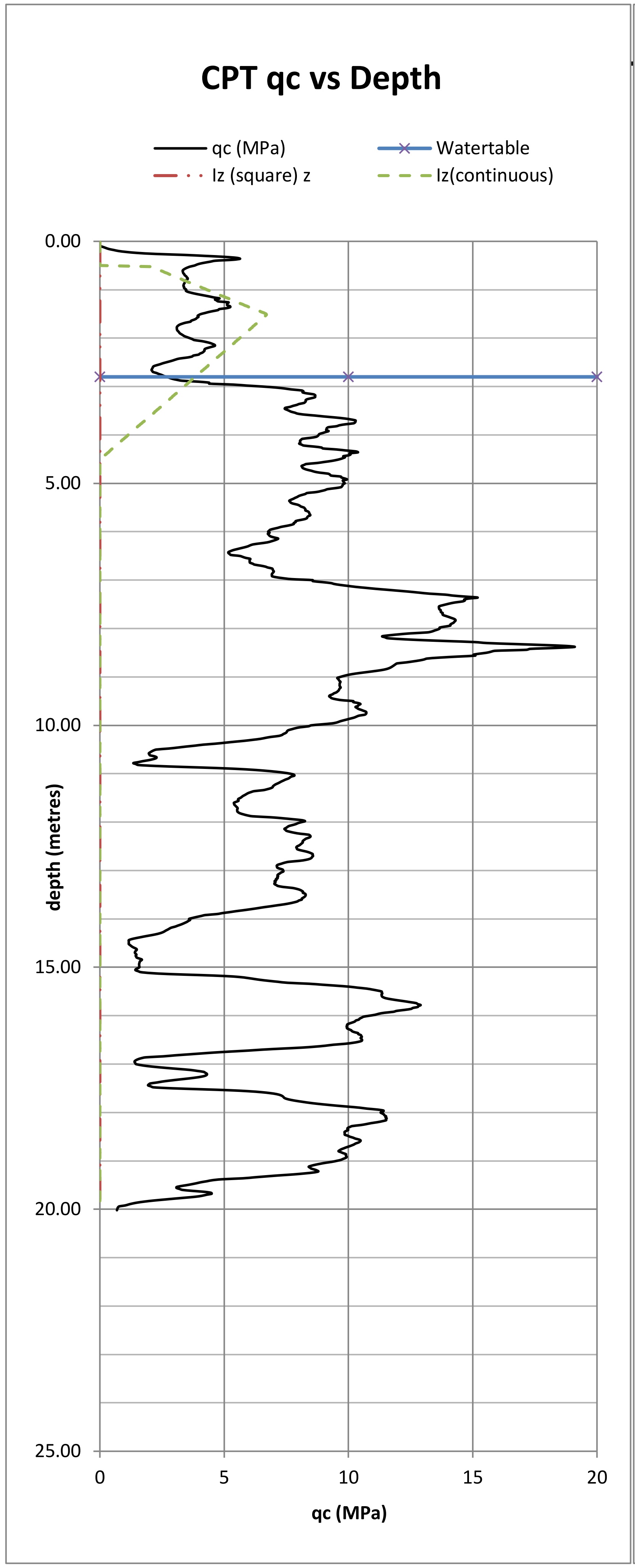
Depth factor (C1) C1 = 1 - 0.5 (σ'_{vo} / q') = 0.9444

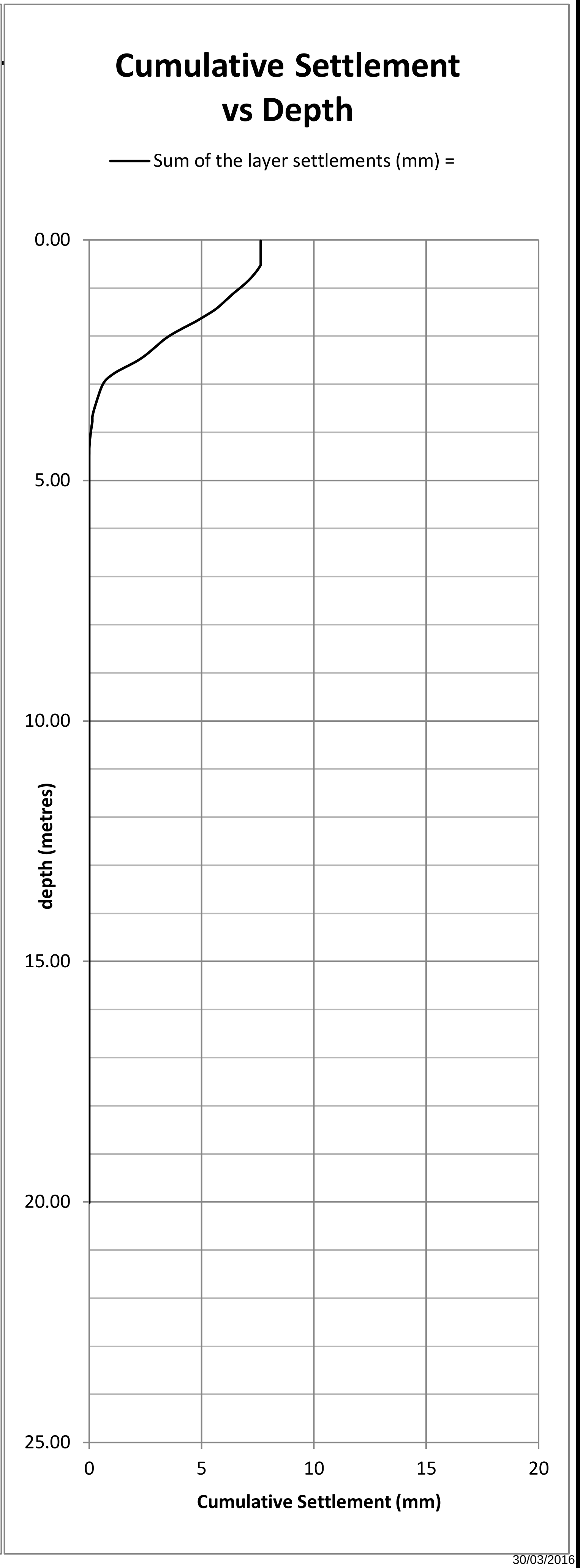
Secondary creep factor (C2) C2 = 1 + 0.2 $\log_{10}$ (t / 0.1) = 1.5398

Summary of settlement calculation

Total settlement = C1.C2.σ'_{vo}.Σ(I_z/xq_c).Δz

Total settlement for SQUARE / CIRCULAR =	-	**in**	-
Total settlement for CONTINUOUS =	**8 mm**	**in**	**50 years**





30/03/2016

Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT01

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		2.8	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 10 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
2.0	2.0	0.5	1.00	**SQUARE / CIRCULAR**

Footing shape — if L/B = 1 — Circular or Square Footing (SQU)

if L/B ≥ 10 — Continuous Footings (CON)

Depth of influence = 4.5 metres

Circular or Square Shape = Df + 2B = 4.5 metres

Continuous Shape = Df + 4B = 8.5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 8.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) — σ'_{zp} = 24.00 kN/m^2

Where, for Square or Circular Shaped Footing — $\sigma'_{zp(squ)}$ = 24.00 kN/m^2

For $h_f < D_f + B/2$ — $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ — $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing — $\sigma'_{zp(con)}$ = 40.00 kN/m^2

For $h_f < D_f + B$ — $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f > D_f + B$ — $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp})

$I_{zp(squ)} = 0.5 + 0.1 \sqrt{} (q' / \sigma'_{zp(squ)}) =$ 0.6958

$I_{zp(con)} = 0.5 + 0.1 \sqrt{} (q' / \sigma'_{zp(con)}) =$ 0.6517

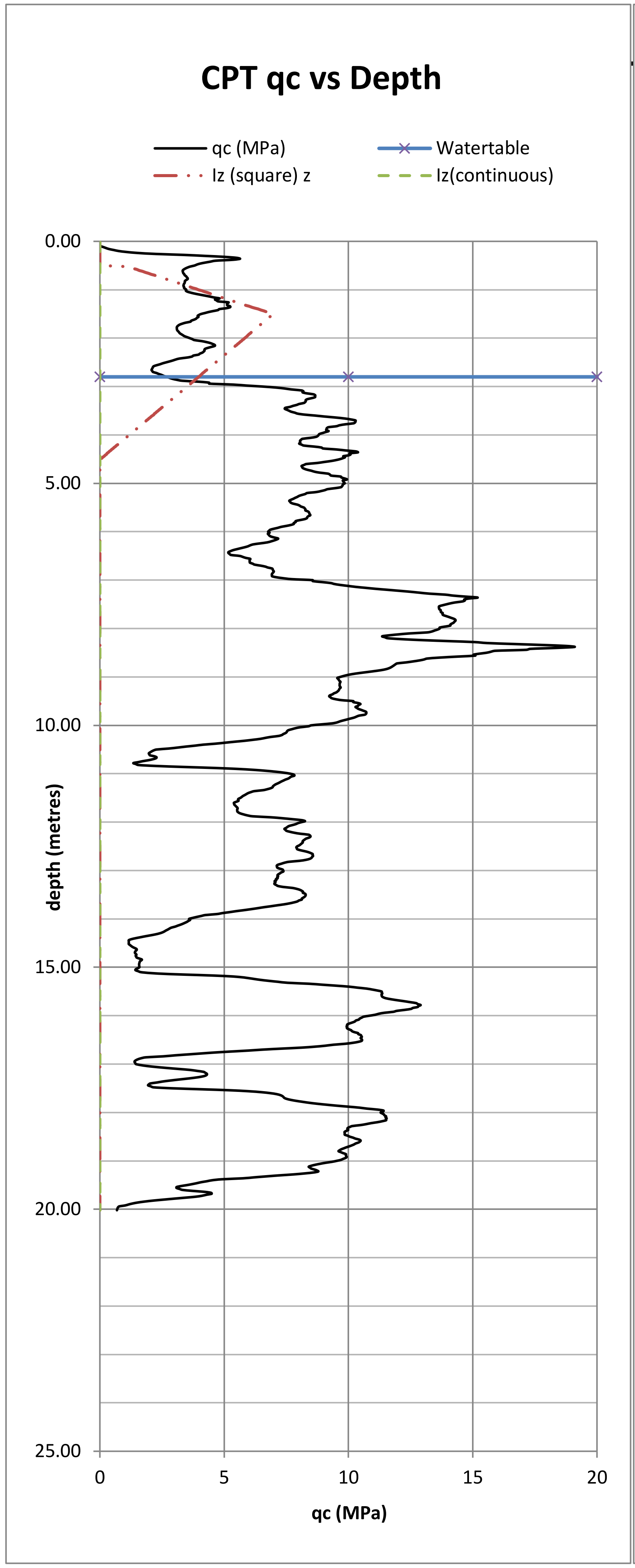
Depth factor (C1) — $C1 = 1 - 0.5 (\sigma'_{vo} / q') =$ 0.9565

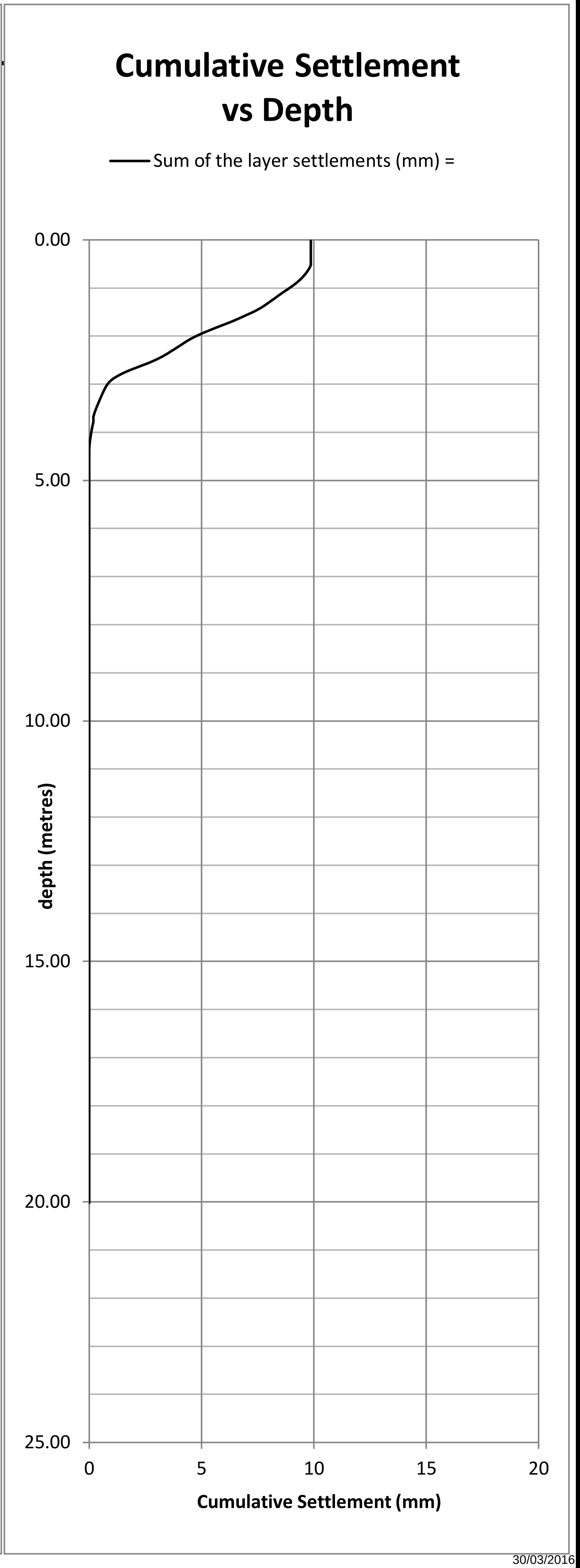
Secondary creep factor (C2) — $C2 = 1 + 0.2 \log_{10} (t / 0.1) =$ 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	**10 mm**	**in**	**50 years**
Total settlement for CONTINUOUS =	**-**	**in**	**-**





Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT01

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		2.8	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 8 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
1.0	30.0	1.0	30.00	**CONTINUOUS**

Footing shape if L/B = 1 Circular or Square Footing (SQU)

if L/B ≥ 10 Continuous Footings (CON)

Depth of influence = 5 metres

Circular or Square Shape = Df + 2B = 3 metres

Continuous Shape = Df + 4B = 5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 16.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 32.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 24.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 32.00 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp}) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{} (q' / \sigma'_{zp(squ)})$ = 0.6871

$I_{zp(con)} = 0.5 + 0.1 \sqrt{} (q' / \sigma'_{zp(con)})$ = 0.6620

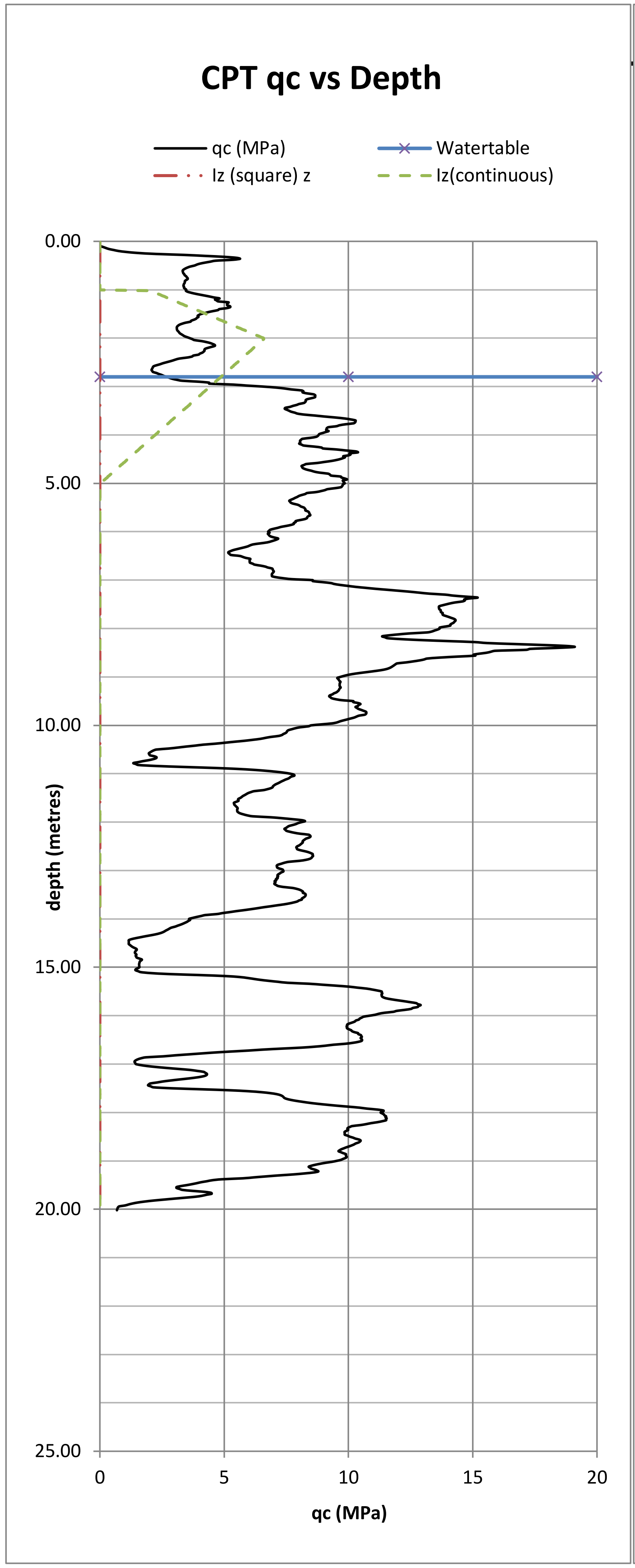
Depth factor (C1) $C1 = 1 - 0.5 (\sigma'_{vo} / q')$ = 0.9048

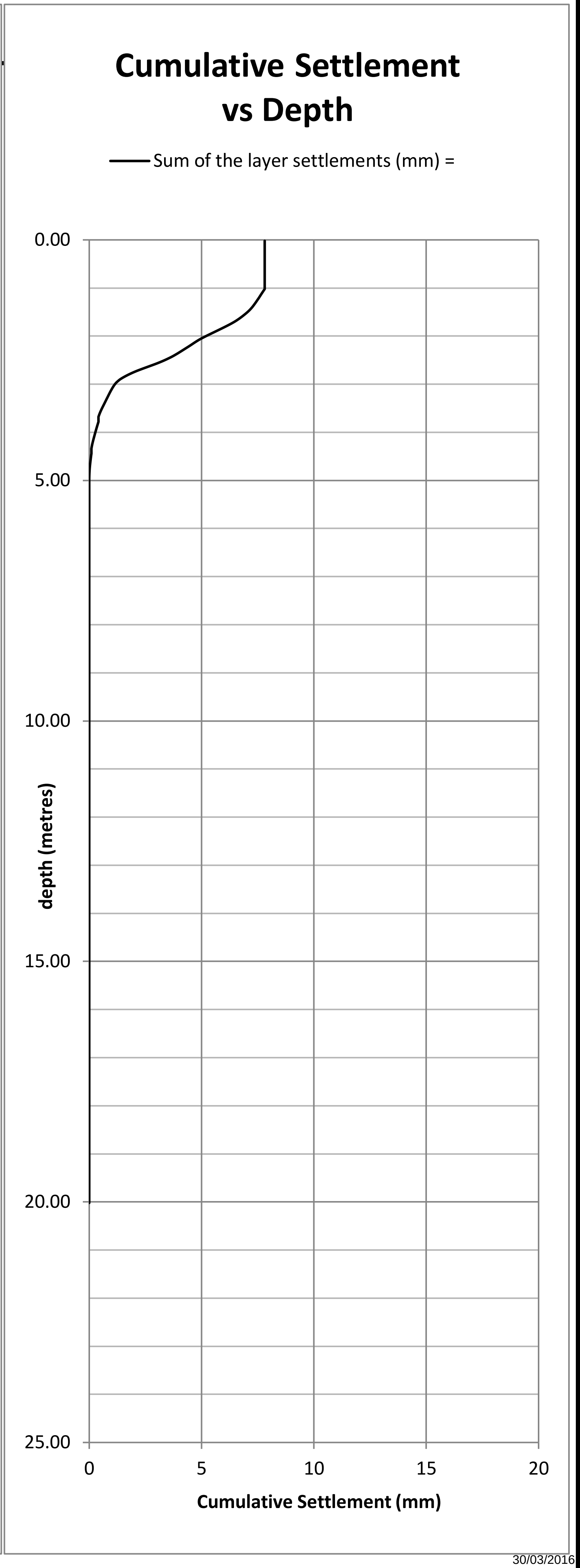
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10} (t / 0.1)$ = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	-	in	-
Total settlement for CONTINUOUS =	**8 mm**	**in**	**50 years**





Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT01

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		2.8	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions Settlement = 8 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
2.0	2.0	1.0	1.00	**SQUARE / CIRCULAR**

Footing shape if L/B = 1 Circular or Square Footing (SQU)

if L/B ≥ 10 Continuous Footings (CON)

Depth of influence = 5 metres

Circular or Square Shape = Df + 2B = 5 metres

Continuous Shape = Df + 4B = 9 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 16.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \text{ x } D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \text{ x } h_f) + (\gamma_s - \gamma_w) \text{ x } (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 32.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 32.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \text{ x } h_f) + (\gamma_s - \gamma_w) \text{ x } (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \text{ x } (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 46.04 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \text{ x } h_f) + (\gamma_s - \gamma_w) \text{ x } (D_f + B - h_f)$

For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \text{ x } (D_f + B)$

Peak strain influence factor (I_{zp}) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{} (q' / \sigma'_{zp(squ)})$ = 0.6620

$I_{zp(con)} = 0.5 + 0.1 \sqrt{} (q' / \sigma'_{zp(con)})$ = 0.6351

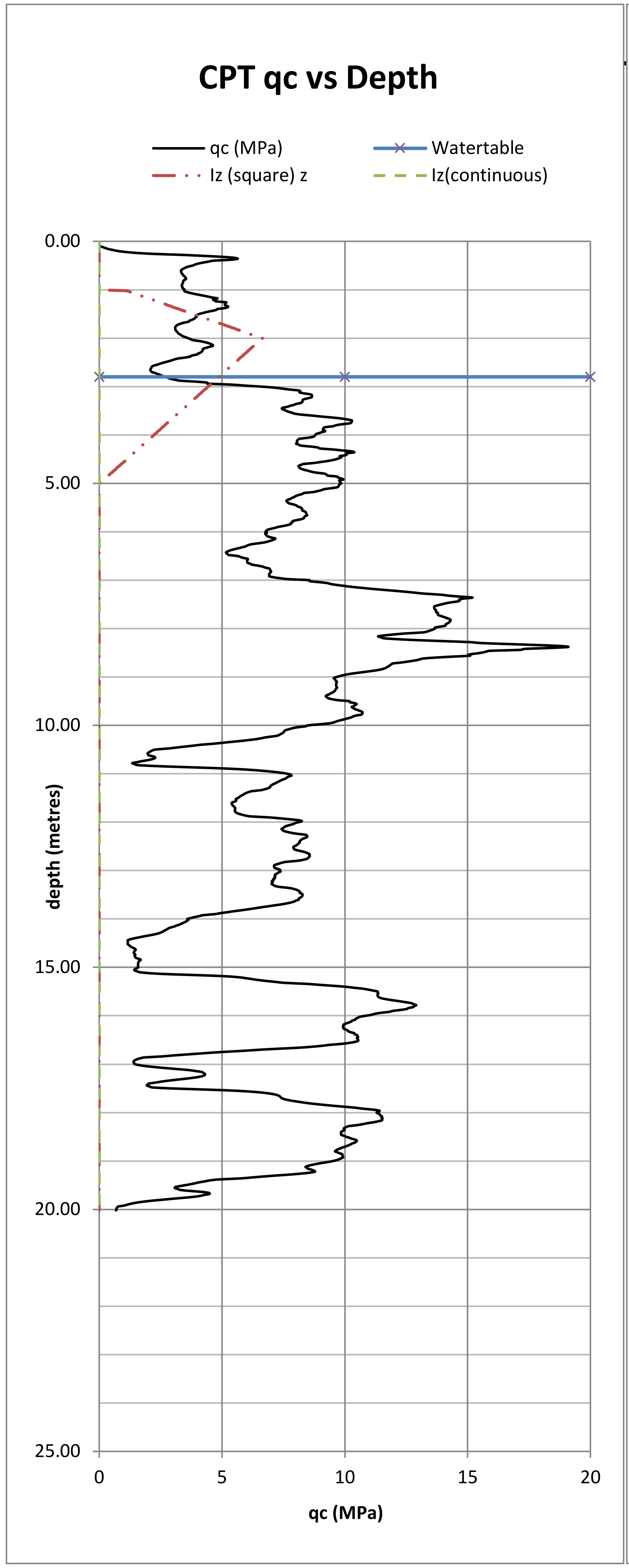
Depth factor (C1) $C1 = 1 - 0.5 (\sigma'_{vo} / q')$ = 0.9048

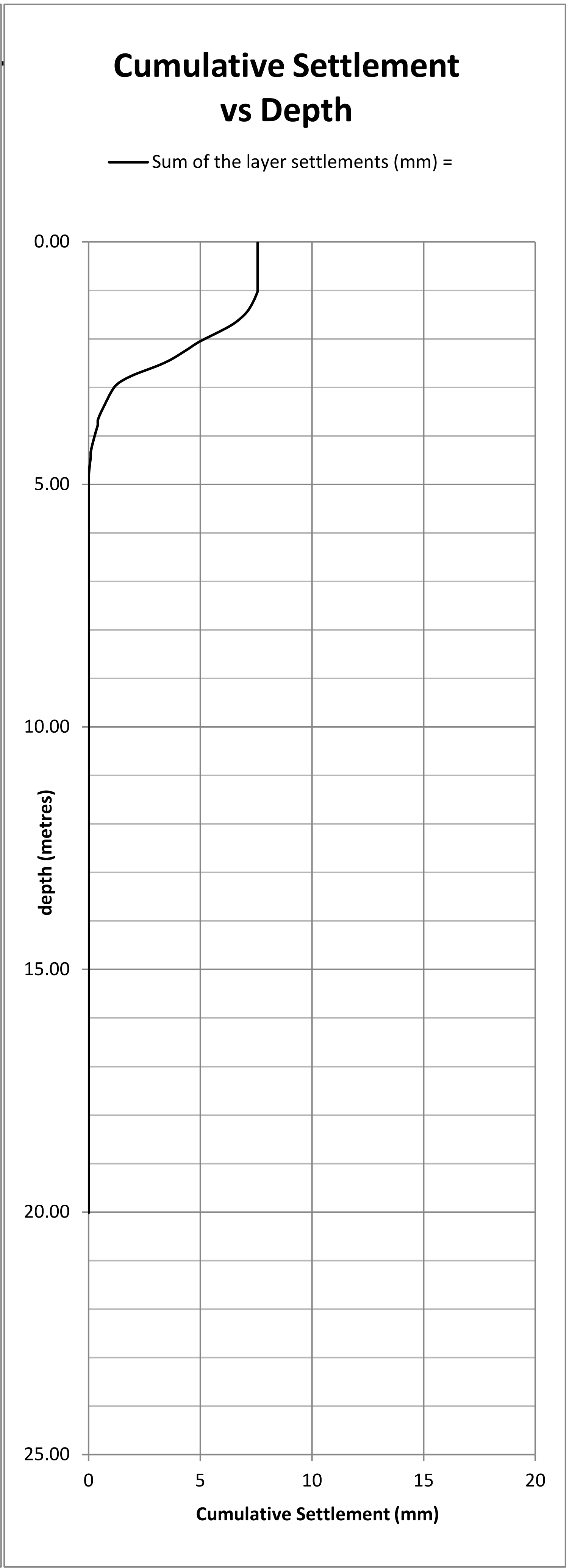
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10} (t / 0.1)$ = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	**8 mm**	**in**	**50 years**
Total settlement for CONTINUOUS =	**-**	**in**	**-**





30/03/2016

Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT02

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		80	kN/m^2
Depth to watertable from ground surface (h_f)		2.8	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 16 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
1.0	30.0	0.5	30.00	**CONTINUOUS**

Footing shape if L/B = 1 Circular or Square Footing (SQU)

if L/B ≥ 10 Continuous Footings (CON)

Depth of influence = 4.5 metres

Circular or Square Shape = Df + 2B = 2.5 metres

Continuous Shape = Df + 4B = 4.5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 8.00 kN/m^2

Where watertable is below base of footing (Df < hf) : σ'_{vo} = $(\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : σ'_{vo} = $(\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 24.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 16.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 24.00 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp}) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ = 0.7121

$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ = 0.6732

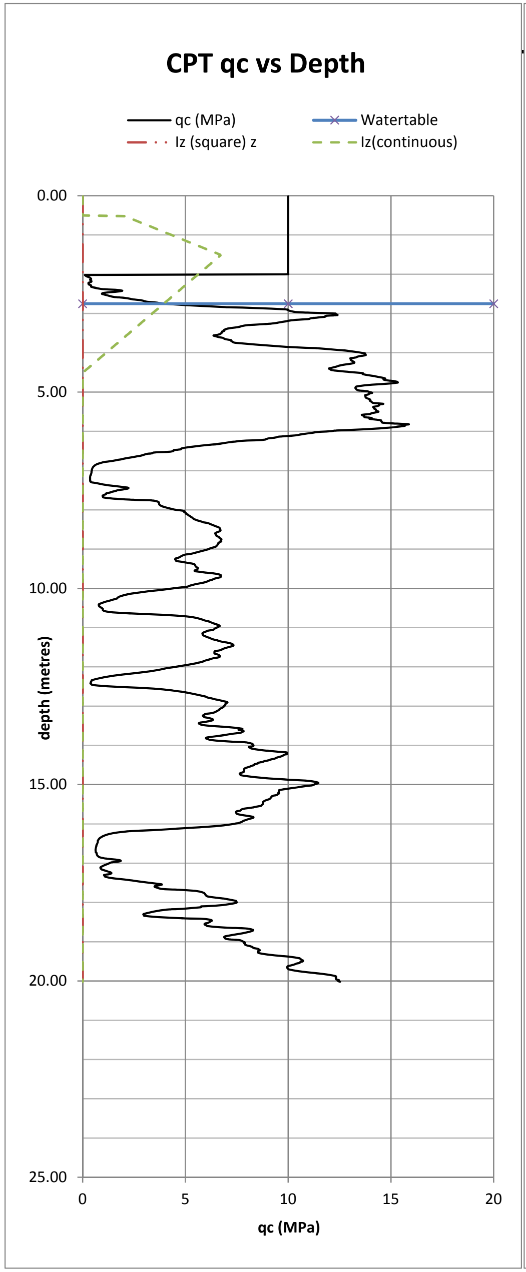
Depth factor (C1) $C1 = 1 - 0.5 (\sigma'_{vo} / q')$ = 0.9444

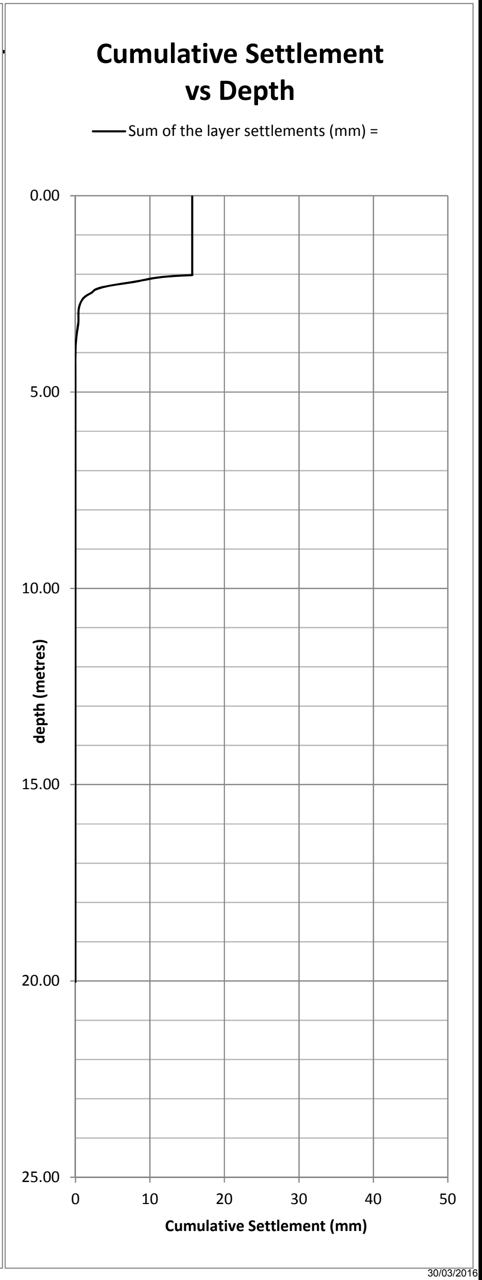
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10} (t / 0.1)$ = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	-	in	-
Total settlement for CONTINUOUS =	16 mm	in	50 years





Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT02

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		2.8	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 21 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
2.0	2.0	0.5	1.00	SQUARE / CIRCULAR

Footing shape	if L/B = 1	Circular or Square Footing (SQU)
	if L/B ≥ 10	Continuous Footings (CON)

Depth of influence = 4.5 metres

Circular or Square Shape = Df + 2B = 4.5 metres

Continuous Shape = Df + 4B = 8.5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 8.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 24.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 24.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 40.00 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp}) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ = 0.6958

$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ = 0.6517

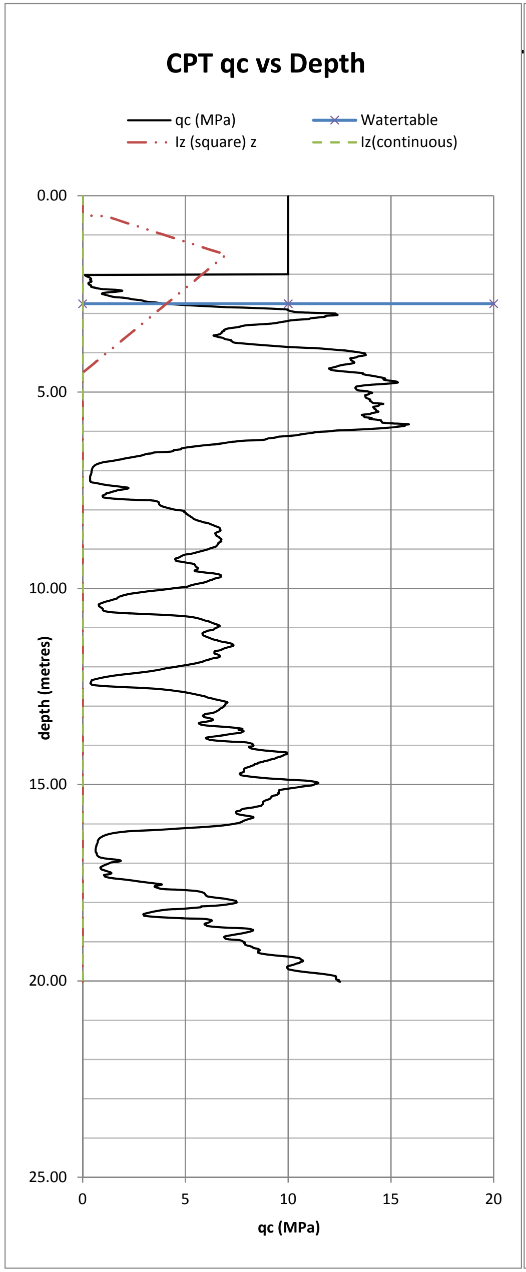
Depth factor (C1) $C1 = 1 - 0.5 (\sigma'_{vo} / q')$ = 0.9565

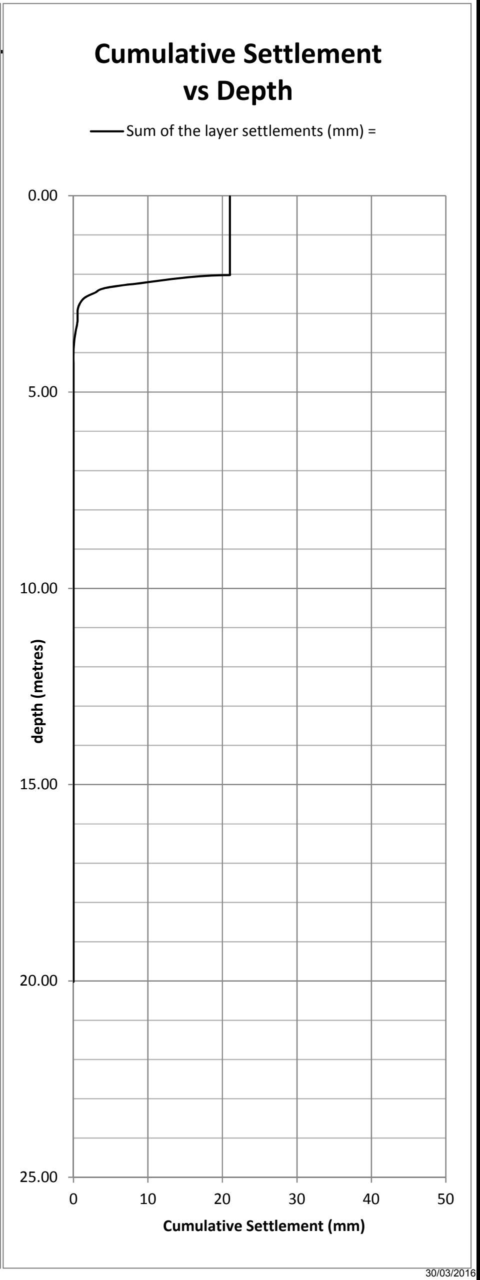
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10} (t / 0.1)$ = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	**21 mm**	**in**	**50 years**
Total settlement for CONTINUOUS =	**-**	**in**	**-**





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Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT02

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		2.8	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 21 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
1.0	30.0	1.0	30.00	**CONTINUOUS**

Footing shape	if L/B = 1	Circular or Square Footing (SQU)
	if L/B ≥ 10	Continuous Footings (CON)

Depth of influence = 5 metres

Circular or Square Shape = Df + 2B = 3 metres

Continuous Shape = Df + 4B = 5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 16.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 32.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 24.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 32.00 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

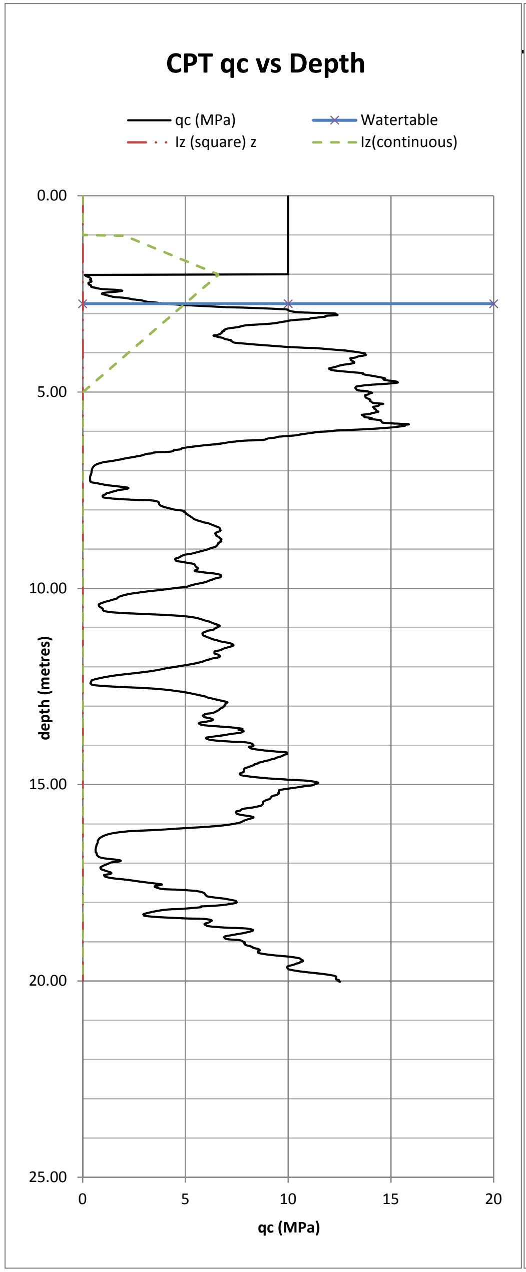
For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp})	$I_{zp(squ)} = 0.5 + 0.1 \sqrt{} (q' / \sigma'_{zp(squ)})$ =	0.6871
	$I_{zp(con)} = 0.5 + 0.1 \sqrt{} (q' / \sigma'_{zp(con)})$ =	0.6620
Depth factor (C1)	C1 = 1 - 0.5 (σ'_{vo} / q') =	0.9048
Secondary creep factor (C2)	C2 = 1 + 0.2 $\log_{10}$ (t / 0.1) =	1.5398

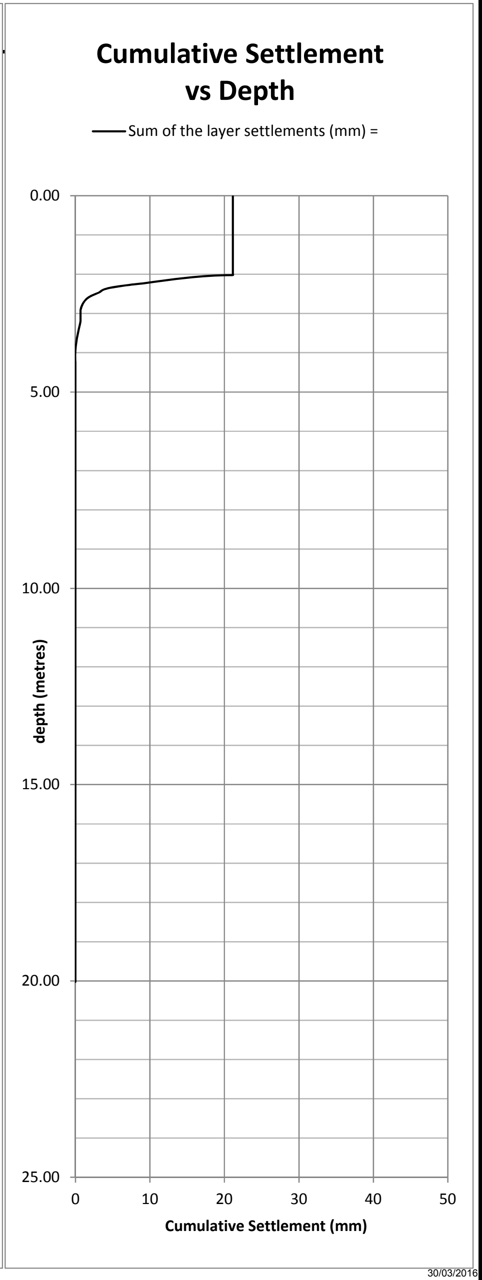
Summary of settlement calculation

Total settlement = C1.C2.σ'_{vo}.Σ(I_z/xq_c).Δz

Total settlement for SQUARE / CIRCULAR =	-	**in**	-
Total settlement for CONTINUOUS =	**21 mm**	**in**	**50 years**



CPT qc vs Depth



Cumulative Settlement vs Depth

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Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT02

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h$_f$)		2.8	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 21 mm

Width (B) (metres)	Length (L) (metres)	Depth (D$_f$) (metres)	L/B	Footing Shape
2.0	2.0	1.0	1.00	SQUARE / CIRCULAR

Footing shape   if L/B = 1   Circular or Square Footing (SQU)

if L/B ≥ 10   Continuous Footings (CON)

Depth of influence = 5 metres

Circular or Square Shape =   Df + 2B =   5 metres

Continuous Shape =   Df + 4B =   9 metres

Eff. stress at a depth D$_f$ below the ground surface (σ'_{vo}) = 16.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I$_{zp}$ (σ'_{zp}) σ'_{zp} = 32.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 32.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 45.55 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I$_{zp}$) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ = 0.6620

$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ = 0.6358

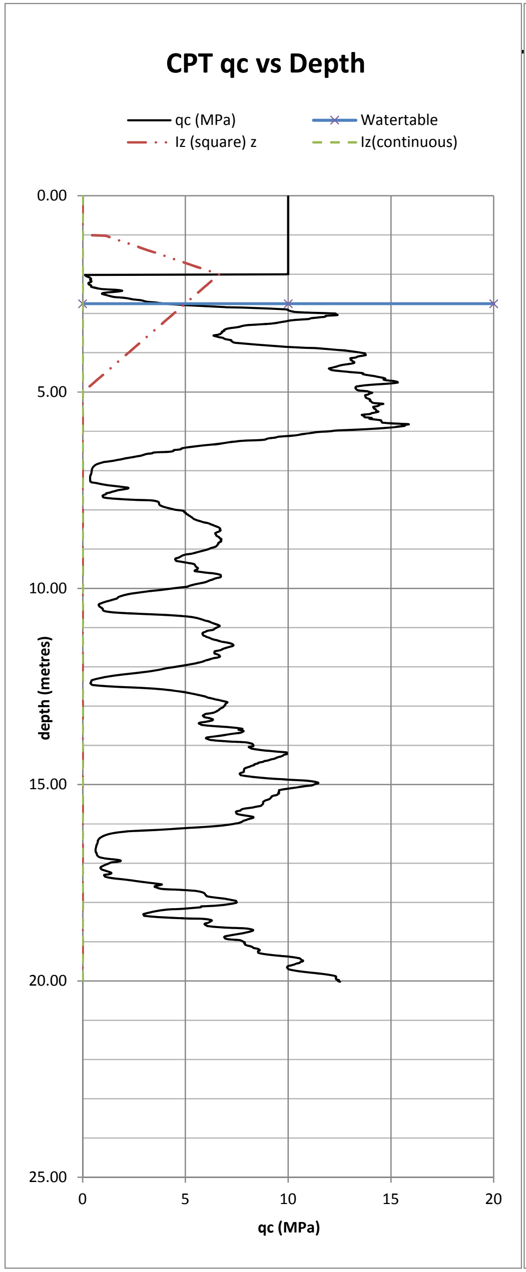
Depth factor (C1) $C1 = 1 - 0.5 (\sigma'_{vo} / q')$ = 0.9048

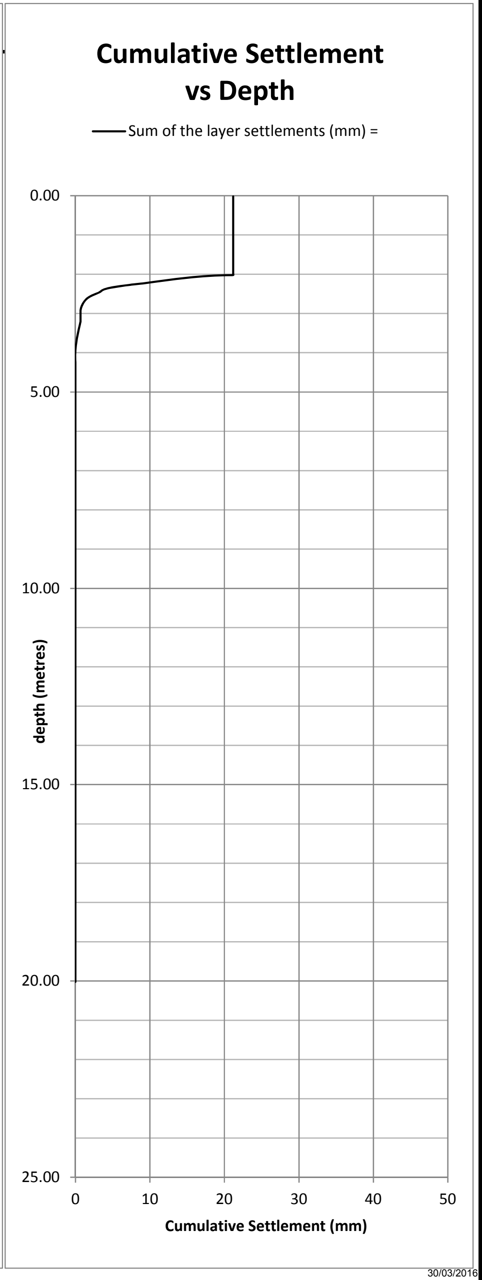
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10} (t / 0.1)$ = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	**21 mm**	**in**	**50 years**
Total settlement for CONTINUOUS =	**-**	**in**	**-**





30/03/2016

Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT03

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		80	kN/m^2
Depth to watertable from ground surface (h_f)		1.5	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 19 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
1.0	30.0	0.5	30.00	**CONTINUOUS**

Footing shape if L/B = 1 Circular or Square Footing (SQU)

if L/B ≥ 10 Continuous Footings (CON)

Depth of influence = 4.5 metres

Circular or Square Shape = Df + 2B = 2.5 metres

Continuous Shape = Df + 4B = 4.5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 8.00 kN/m^2

Where watertable is below base of footing (Df < hf) : σ'_{vo} = (γ_s x D_f)

Where watertable is above base of footing (Df > hf) : σ'_{vo} = (γ_s x h_f) + (γ_s - γ_w) x (D_f - h_f)

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 24.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 16.00 kN/m^2

For h_f < D_f + B/2 $\sigma'_{zp(squ)}$ = (γ_s x h_f) + (γ_s - γ_w) x (D_f + B/2 - h_f)

For h_f > D_f + B/2 $\sigma'_{zp(squ)}$ = γ_s x (D_f + B/2)

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 24.00 kN/m^2

For h_f < D_f + B $\sigma'_{zp(con)}$ = (γ_s x h_f) + (γ_s - γ_w) x (D_f + B - h_f)

For h_f > D_f + B $\sigma'_{zp(con)}$ = γ_s x (D_f + B)

Peak strain influence factor (I_{zp}) $I_{zp(squ)}$ = 0.5 + 0.1 √ (q' / $\sigma'_{zp(squ)}$) = 0.7121

$I_{zp(con)}$ = 0.5 + 0.1 √ (q' / $\sigma'_{zp(con)}$) = 0.6732

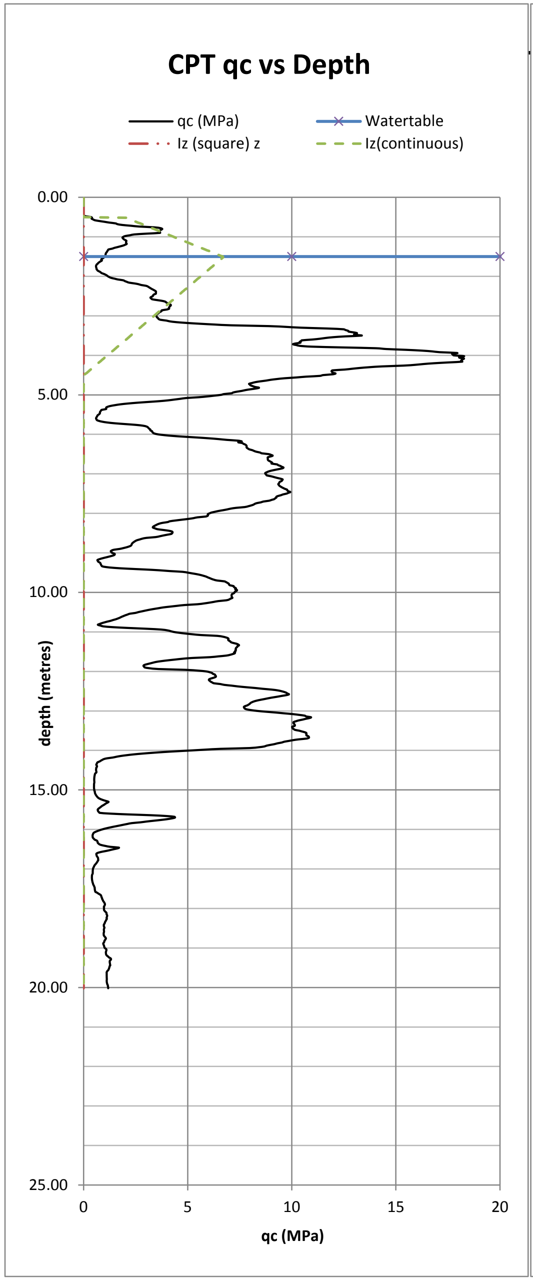
Depth factor (C1) C1 = 1 - 0.5 (σ'_{vo} / q') = 0.9444

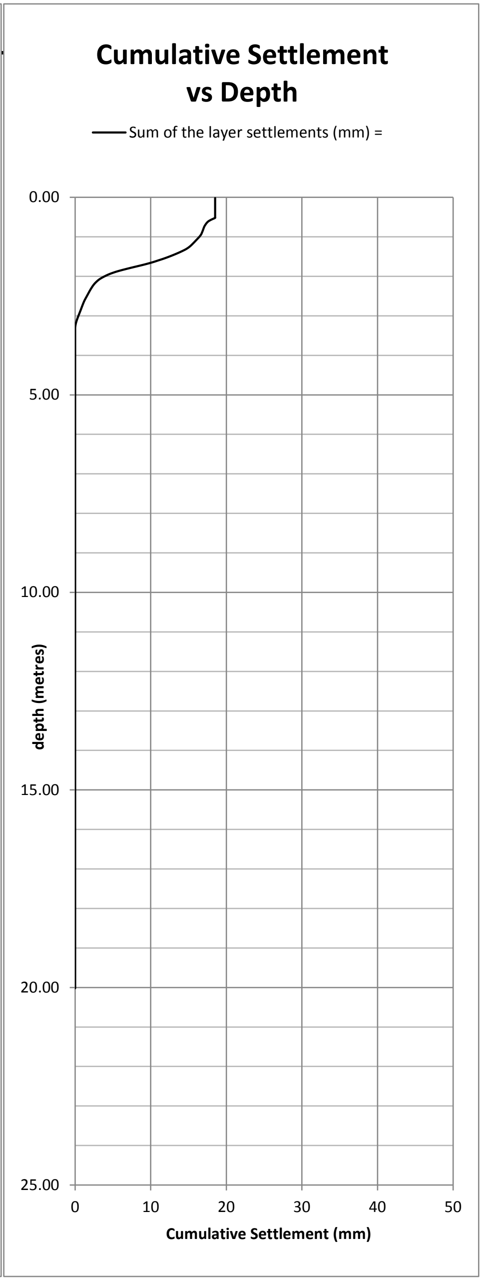
Secondary creep factor (C2) C2 = 1 + 0.2 $\log_{10}$ (t / 0.1) = 1.5398

Summary of settlement calculation

Total settlement = C1.C2.σ'_{vo}.Σ(I_z/xq_c).Δz

Total settlement for SQUARE / CIRCULAR =	-	in	-
Total settlement for CONTINUOUS =	**19 mm**	**in**	**50 years**





Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT03

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		1.5	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 24 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
2.0	2.0	0.5	1.00	**SQUARE / CIRCULAR**

Footing shape	if L/B = 1	Circular or Square Footing (SQU)
	if L/B ≥ 10	Continuous Footings (CON)

Depth of influence = 4.5 metres

Circular or Square Shape = Df + 2B = 4.5 metres

Continuous Shape = Df + 4B = 8.5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 8.00 kN/m^2

Where watertable is below base of footing (Df < hf) : σ'_{vo} = $(\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : σ'_{vo} = $(\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 24.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 24.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 30.19 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp}) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ = 0.6958

$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ = 0.6746

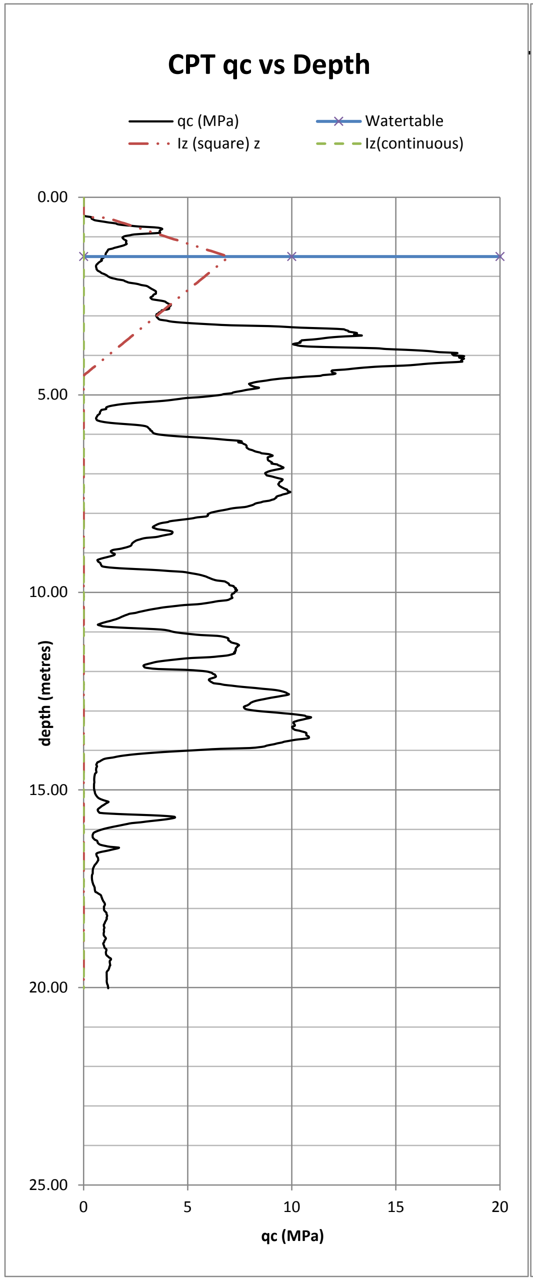
Depth factor (C1) $C1 = 1 - 0.5 (\sigma'_{vo} / q')$ = 0.9565

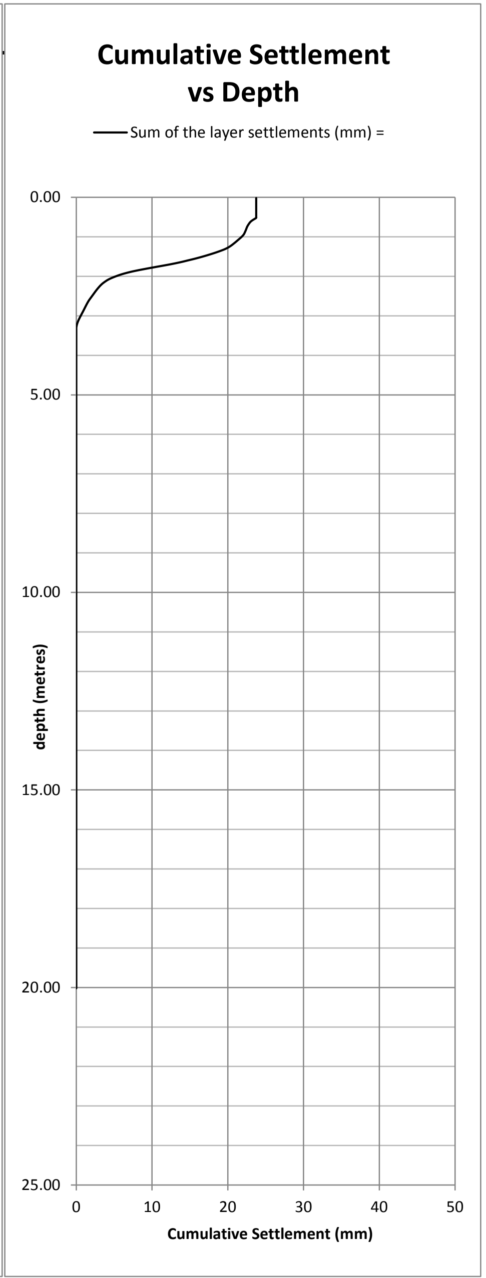
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10} (t / 0.1)$ = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	**24 mm**	**in**	**50 years**
Total settlement for CONTINUOUS =	**-**	**in**	**-**





30/03/2016

Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT03

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h$_f$)		1.5	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 17 mm

Width (B) (metres)	Length (L) (metres)	Depth (D$_f$) (metres)	L/B	Footing Shape
1.0	30.0	1.0	30.00	**CONTINUOUS**

Footing shape	if L/B = 1	Circular or Square Footing (SQU)
	if L/B ≥ 10	Continuous Footings (CON)

Depth of influence = 5 metres

Circular or Square Shape = Df + 2B = 3 metres

Continuous Shape = Df + 4B = 5 metres

Eff. stress at a depth D$_f$ below the ground surface (σ'_{vo}) = 16.00 kN/m^2

Where watertable is below base of footing (Df < hf) : σ'_{vo} = (γ_s x D$_f$)

Where watertable is above base of footing (Df > hf) : σ'_{vo} = (γ_s x h$_f$) + (γ_s - γ_w) x (D$_f$ - h$_f$)

Initial vert eff. stress at a depth of I$_{zp}$ (σ'_{zp}) σ'_{zp} = 27.10 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 24.00 kN/m^2

For h$_f$ < D$_f$ + B/2 $\sigma'_{zp(squ)}$ = (γ_s x h$_f$) + (γ_s - γ_w) x (D$_f$ + B/2 - h$_f$)

For h$_f$ > D$_f$ + B/2 $\sigma'_{zp(squ)}$ = γ_s x (D$_f$ + B/2)

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 27.10 kN/m^2

For h$_f$ < D$_f$ + B $\sigma'_{zp(con)}$ = (γ_s x h$_f$) + (γ_s - γ_w) x (D$_f$ + B - h$_f$)

For h$_f$ > D$_f$ + B $\sigma'_{zp(con)}$ = γ_s x (D$_f$ + B)

Peak strain influence factor (I$_{zp}$) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ = 0.6871

$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ = 0.6761

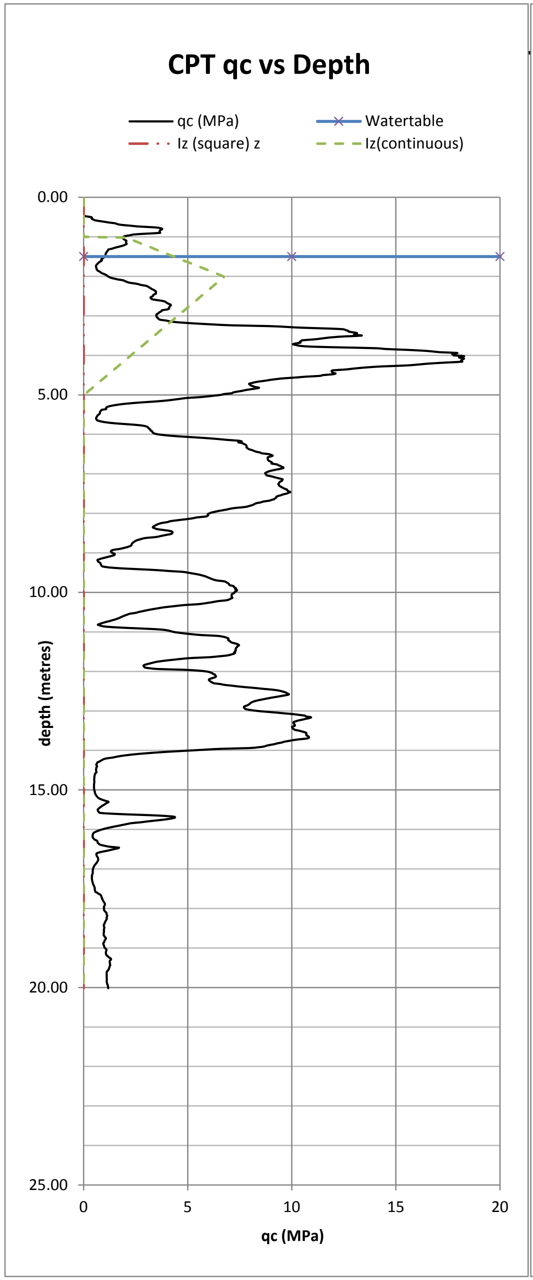
Depth factor (C1) $C1 = 1 - 0.5 (\sigma'_{vo} / q')$ = 0.9048

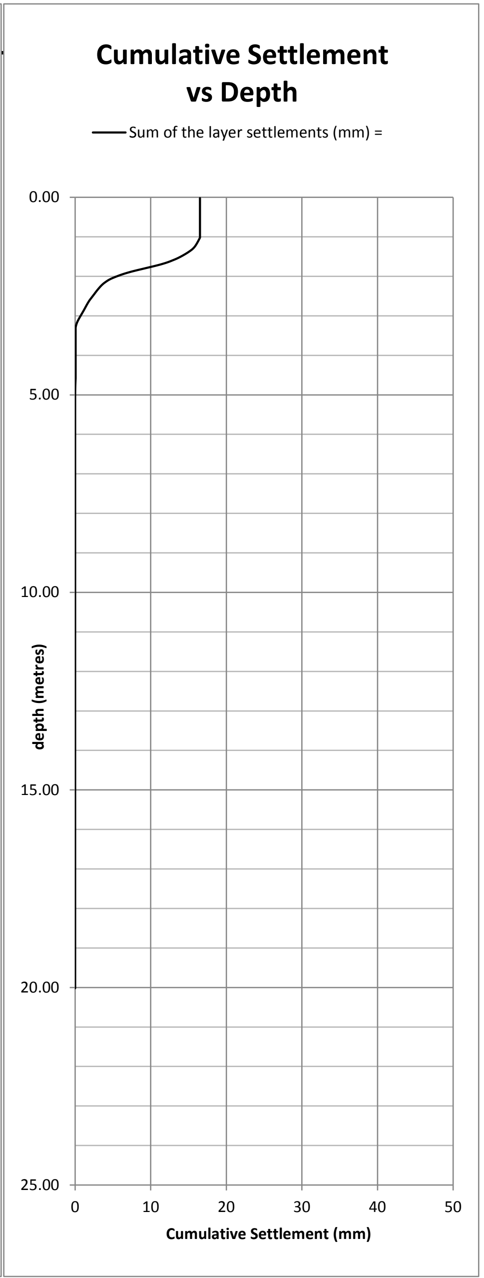
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10} (t / 0.1)$ = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	-	in	-
Total settlement for CONTINUOUS =	**17 mm**	**in**	**50 years**





Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT03

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		1.5	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 16 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
2.0	2.0	1.0	1.00	**SQUARE / CIRCULAR**

Footing shape	if L/B = 1	Circular or Square Footing (SQU)
	if L/B ≥ 10	Continuous Footings (CON)

Depth of influence = 5 metres

Circular or Square Shape = Df + 2B = 5 metres

Continuous Shape = Df + 4B = 9 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 16.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 27.10 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 27.10 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 33.29 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp}) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ = 0.6761

$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ = 0.6589

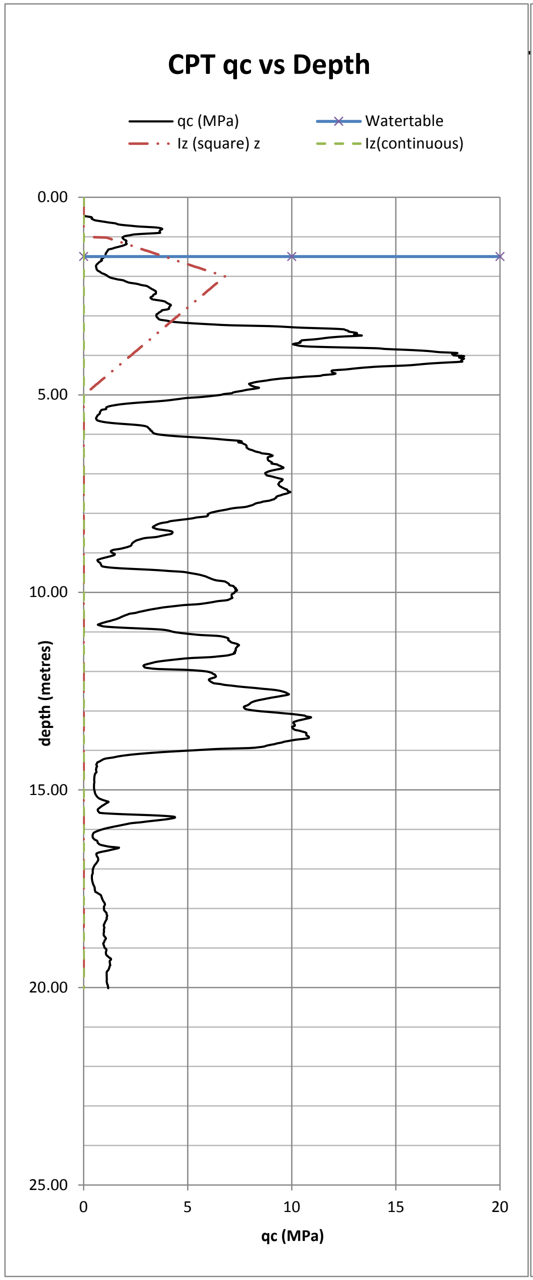
Depth factor (C1) $C1 = 1 - 0.5\,(\sigma'_{vo} / q')$ = 0.9048

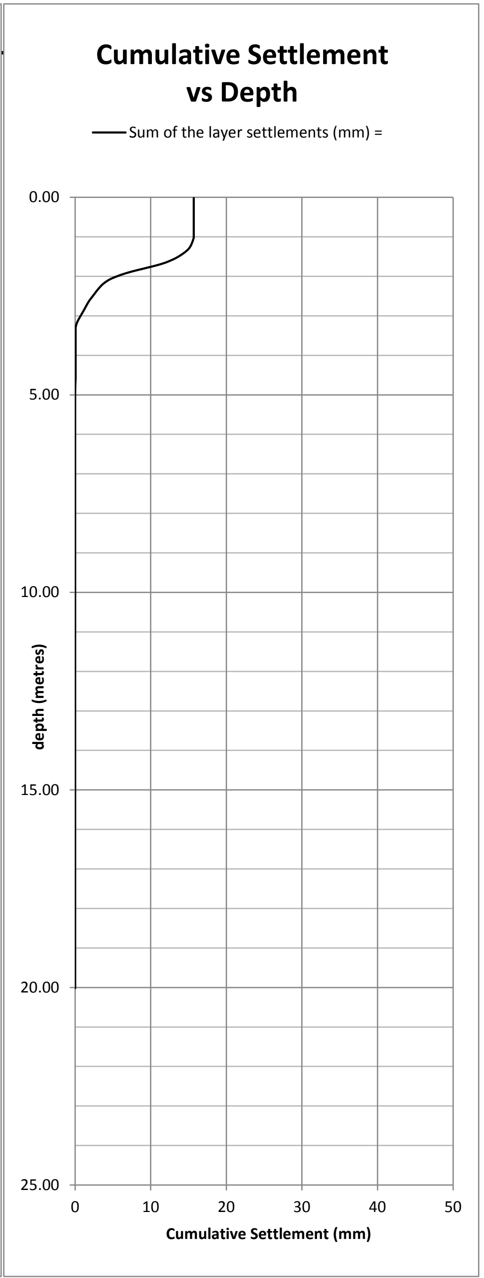
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10}(t / 0.1)$ = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	**16 mm**	**in**	**50 years**
Total settlement for CONTINUOUS =	**-**	**in**	**-**





30/03/2016

Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT04

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		80	kN/m^2
Depth to watertable from ground surface (h_f)		2.75	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 36 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
1.0	30.0	0.5	30.00	**CONTINUOUS**

Footing shape	if L/B = 1	Circular or Square Footing (SQU)
	if L/B ≥ 10	Continuous Footings (CON)

Depth of influence = 4.5 metres

Circular or Square Shape = Df + 2B = 2.5 metres

Continuous Shape = Df + 4B = 4.5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 8.00 kN/m^2

Where watertable is below base of footing (Df < hf) : σ'_{vo} = $(\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : σ'_{vo} = $(\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 24.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 16.00 kN/m^2

For $h_f <$ D_f + B/2 $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f >$ D_f + B/2 $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 24.00 kN/m^2

For $h_f <$ D_f + B $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f >$ D_f + B $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp})

$I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ = 0.7121

$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ = 0.6732

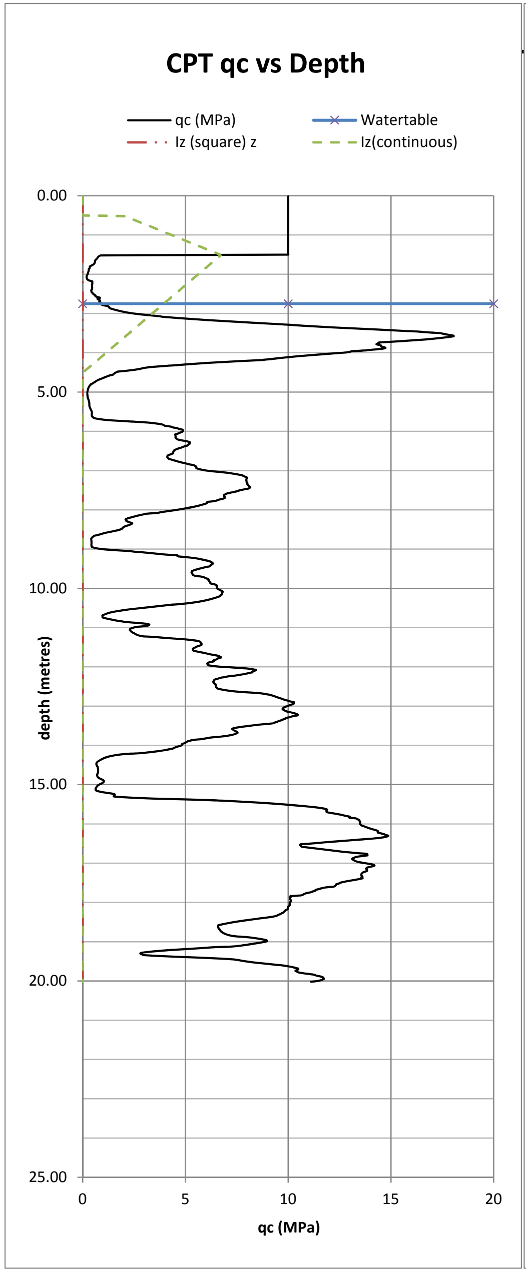
Depth factor (C1) C1 = 1 - 0.5 (σ'_{vo} / q') = 0.9444

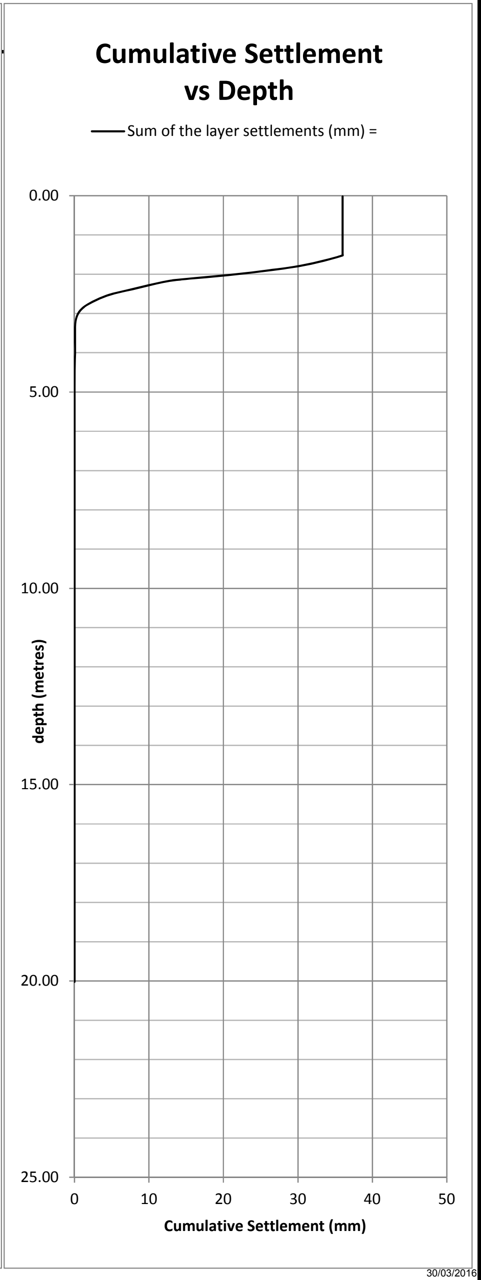
Secondary creep factor (C2) C2 = 1 + 0.2 $\log_{10}$ (t / 0.1) = 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	-	in	-
Total settlement for CONTINUOUS =	**36 mm**	**in**	**50 years**





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Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT04

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		2.75	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 48 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
2.0	2.0	0.5	1.00	**SQUARE / CIRCULAR**

Footing shape if L/B = 1 Circular or Square Footing (SQU)

if L/B ≥ 10 Continuous Footings (CON)

Depth of influence = 4.5 metres

Circular or Square Shape = Df + 2B = 4.5 metres

Continuous Shape = Df + 4B = 8.5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 8.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 24.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 24.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 40.00 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

Peak strain influence factor (I_{zp}) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{}(q' / \sigma'_{zp(squ)}) =$ 0.6958

$I_{zp(con)} = 0.5 + 0.1 \sqrt{}(q' / \sigma'_{zp(con)}) =$ 0.6517

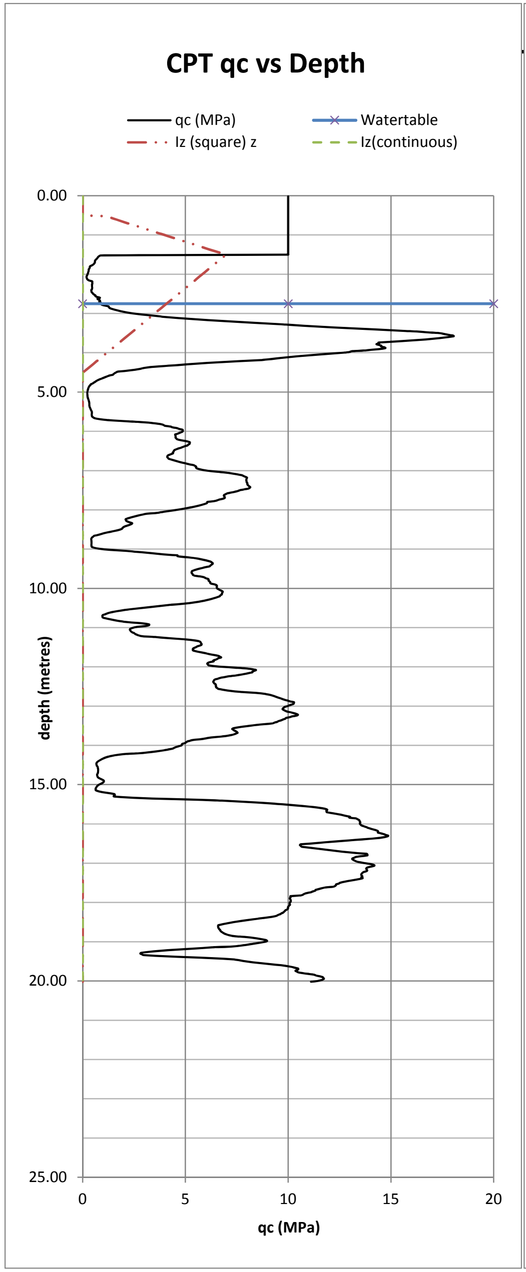
Depth factor (C1) $C1 = 1 - 0.5 (\sigma'_{vo} / q') =$ 0.9565

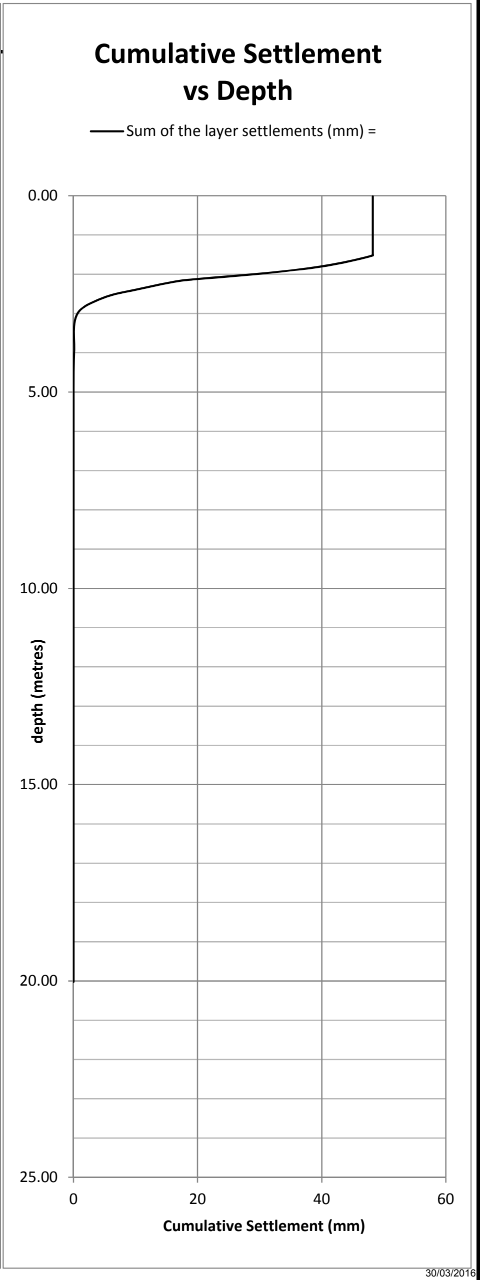
Secondary creep factor (C2) $C2 = 1 + 0.2 \log_{10} (t / 0.1) =$ 1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	**48 mm**	**in**	**50 years**
Total settlement for CONTINUOUS =	**-**	**in**	**-**





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Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT04

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h_f)		2.75	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 45 mm

Width (B) (metres)	Length (L) (metres)	Depth (D_f) (metres)	L/B	Footing Shape
1.0	30.0	1.0	30.00	**CONTINUOUS**

Footing shape if L/B = 1 Circular or Square Footing (SQU)

if L/B ≥ 10 Continuous Footings (CON)

Depth of influence = 5 metres

Circular or Square Shape = Df + 2B = 3 metres

Continuous Shape = Df + 4B = 5 metres

Eff. stress at a depth D_f below the ground surface (σ'_{vo}) = 16.00 kN/m^2

Where watertable is below base of footing (Df < hf) : σ'_{vo} = (γ_s x D_f)

Where watertable is above base of footing (Df > hf) : σ'_{vo} = (γ_s x h_f) + (γ_s - γ_w) x (D_f - h_f)

Initial vert eff. stress at a depth of I_{zp} (σ'_{zp}) σ'_{zp} = 32.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 24.00 kN/m^2

For h_f < D_f + B/2 $\sigma'_{zp(squ)}$ = (γ_s x h_f) + (γ_s - γ_w) x (D_f + B/2 - h_f)

For h_f > D_f + B/2 $\sigma'_{zp(squ)}$ = γ_s x (D_f + B/2)

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 32.00 kN/m^2

For h_f < D_f + B $\sigma'_{zp(con)}$ = (γ_s x h_f) + (γ_s - γ_w) x (D_f + B - h_f)

For h_f > D_f + B $\sigma'_{zp(con)}$ = γ_s x (D_f + B)

Peak strain influence factor (I_{zp}) $I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ = 0.6871

$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ = 0.6620

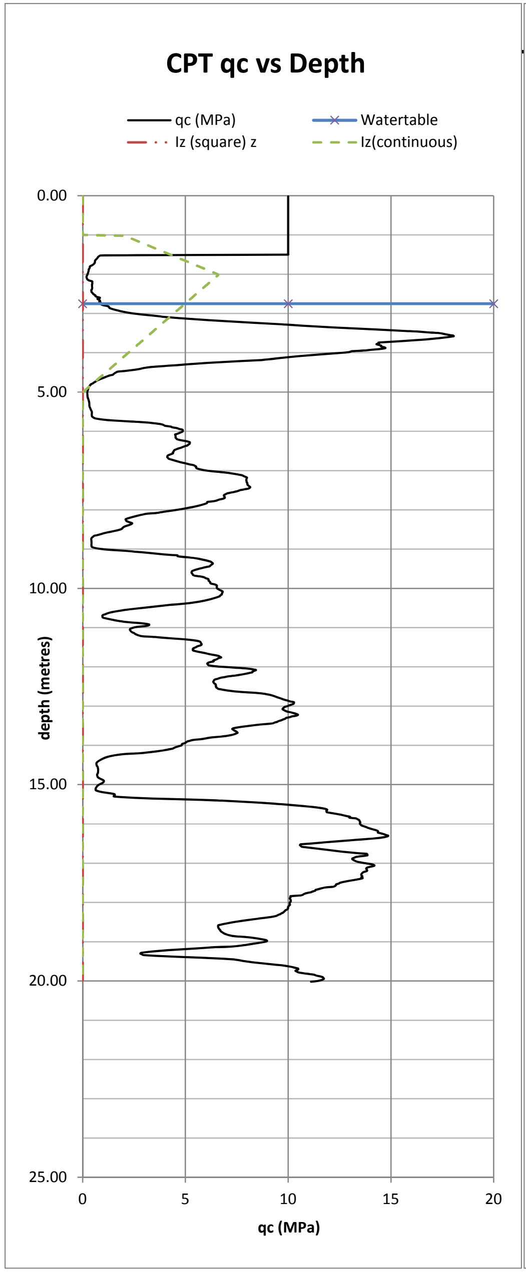
Depth factor (C1) C1 = 1 - 0.5 (σ'_{vo} / q') = 0.9048

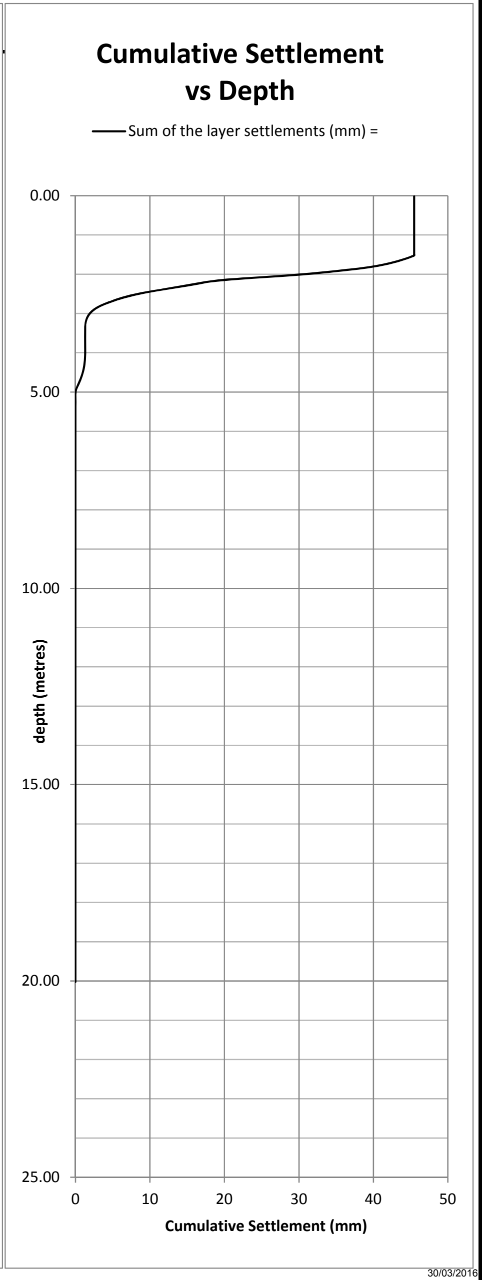
Secondary creep factor (C2) C2 = 1 + 0.2 $\log_{10}$ (t / 0.1) = 1.5398

Summary of settlement calculation

Total settlement = C1.C2.σ'_{vo}.Σ(I_z/xq_c).Δz

Total settlement for SQUARE / CIRCULAR =	-	in	-
Total settlement for CONTINUOUS =	**45 mm**	in	**50 years**





30/03/2016

Settlement Calculation for Cohesionless Soil - Schmertmann's Method

CPT04

Client:	Te Rapa Waikato Racing Club	Job number:	HAM2016_0109
Location:	Te Rapa Racecourse	Date:	14/06/2017

Input parameters

Soil unit weight (kN/m^3)		16.0	kN/m^3
Bearing pressure at base of footing (q)		100	kN/m^2
Depth to watertable from ground surface (h$_f$)		2.75	metres
Time since application of load (t)	(t ≥ 0.1 yr)	50	years
Filter out layer settlement where qc is greater than		10.0	MPa

Footing dimensions

Settlement = 45 mm

Width (B) (metres)	Length (L) (metres)	Depth (D$_f$) (metres)	L/B	Footing Shape
2.0	2.0	1.0	1.00	**SQUARE / CIRCULAR**

Footing shape	if L/B = 1	Circular or Square Footing (SQU)
	if L/B ≥ 10	Continuous Footings (CON)

Depth of influence = 5 metres

Circular or Square Shape =	Df + 2B =	5 metres
Continuous Shape =	Df + 4B =	9 metres

Eff. stress at a depth D$_f$ below the ground surface (σ'_{vo}) = 16.00 kN/m^2

Where watertable is below base of footing (Df < hf) : $\sigma'_{vo} = (\gamma_s \times D_f)$

Where watertable is above base of footing (Df > hf) : $\sigma'_{vo} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f - h_f)$

Initial vert eff. stress at a depth of I$_{zp}$ (σ'_{zp}) σ'_{zp} = 32.00 kN/m^2

Where, for Square or Circular Shaped Footing $\sigma'_{zp(squ)}$ = 32.00 kN/m^2

For $h_f < D_f + B/2$ $\sigma'_{zp(squ)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B/2 - h_f)$

For $h_f > D_f + B/2$ $\sigma'_{zp(squ)} = \gamma_s \times (D_f + B/2)$

Where, for Continuous Shape Footing $\sigma'_{zp(con)}$ = 45.55 kN/m^2

For $h_f < D_f + B$ $\sigma'_{zp(con)} = (\gamma_s \times h_f) + (\gamma_s - \gamma_w) \times (D_f + B - h_f)$

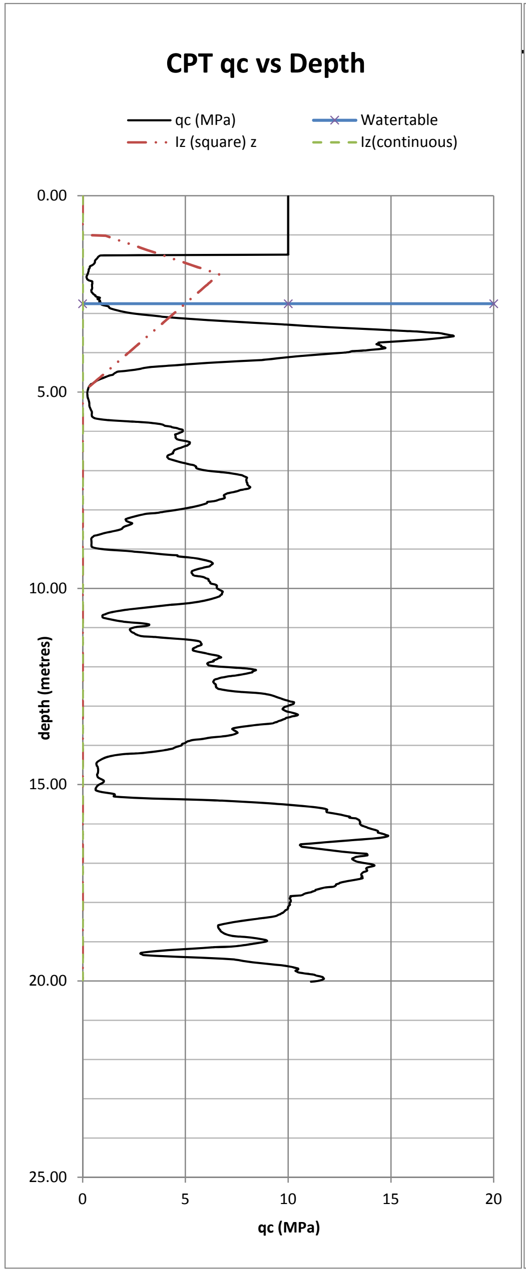
For $h_f > D_f + B$ $\sigma'_{zp(con)} = \gamma_s \times (D_f + B)$

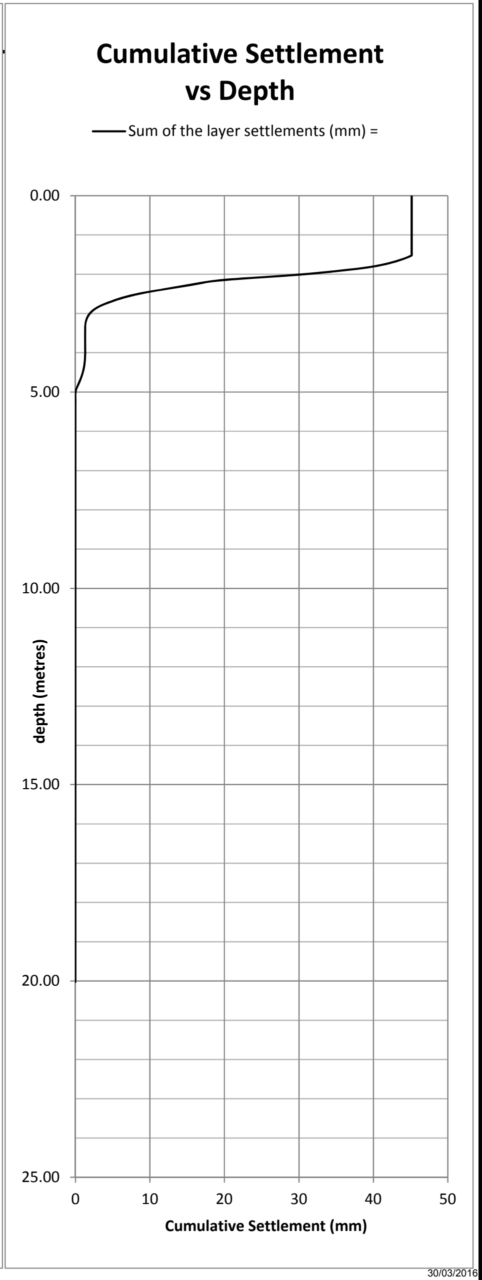
Peak strain influence factor (I$_{zp}$)	$I_{zp(squ)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(squ)})}$ =	0.6620
	$I_{zp(con)} = 0.5 + 0.1 \sqrt{(q' / \sigma'_{zp(con)})}$ =	0.6358
Depth factor (C1)	$C1 = 1 - 0.5 (\sigma'_{vo} / q')$ =	0.9048
Secondary creep factor (C2)	$C2 = 1 + 0.2 \log_{10} (t / 0.1)$ =	1.5398

Summary of settlement calculation

Total settlement = $C1.C2.\sigma'_{vo}.\Sigma(I_z/xq_c).\Delta z$

Total settlement for SQUARE / CIRCULAR =	**45 mm**	**in**	**50 years**
Total settlement for CONTINUOUS =	**-**	**in**	**-**





30/03/2016

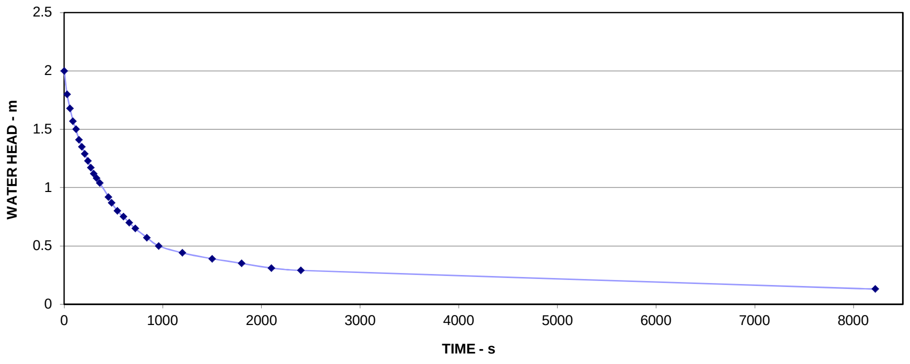
APPENDIX E: PERCOLATION TEST RESULTS

CLIENT: Te Rapa Waikato Racing Club
PROJECT: Te Rapa Racecourse Redevelopment
LOCATION: Ken Browne Drive, Hamilton
JOB NUMBER: HAM2016_0109
TEST DATE: 30/05/2017

Test ID: HA03

Refer to HA03 Engineering Log for soil description.
Following presoaking of hole, groundwater was measured at 2.78m below ground level.

FALLING HEAD PERMEABILITY TEST HA03 RESULT



Reference: Appendix 4, Control of Groundwater for Temporary Works (CIRIA Report No. 113)

Borehole diameter = 100 mm

Hydraulic conductivity $$k = \left(\frac{log\left(\frac{h_1}{h_2}\right) - log\left(\frac{\alpha h_1 + 1}{\alpha h_2 + 1}\right)}{(t_2 - t_1)} \right) \text{ x } l$$

where l = average piezometric head over chosen time interval

$$= \frac{(h_1 + h_2)}{2}$$

h_1 = piezometric head at start of chosen interval (m)
h_2 = piezometric head at end of chosen interval (m)
$t_2 - t_1$ = chosen time interval (seconds)

$$\alpha = \frac{\pi d}{\left(\frac{\pi d^2}{2}\right)} \qquad = \qquad 20.0$$

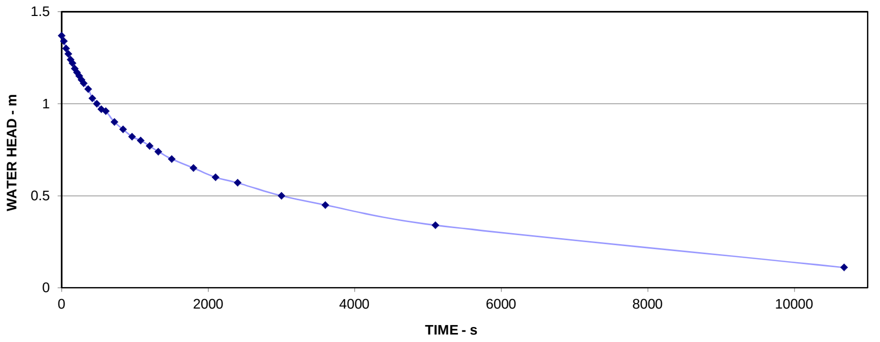
Elapsed Time		t2 - t1	Piezometric Head	l	log (h_1/h_2)	Hydraulic Conductivity	
	(s)	(secs)	h (m)	(m)		k (m/sec)	k (m/day)
0	0		2				
0.5	30	30	1.8	1.90	0.05	7.44E-05	6
1	60	30	1.68	1.74	0.03	4.86E-05	4
1.5	90	30	1.57	1.63	0.03	4.76E-05	4
2	120	30	1.5	1.54	0.02	3.20E-05	3
2.5	150	30	1.41	1.46	0.03	4.33E-05	4
3	180	30	1.35	1.38	0.02	3.04E-05	3
3.5	210	30	1.29	1.32	0.02	3.17E-05	3
4	240	30	1.23	1.26	0.02	3.32E-05	3
4.5	270	30	1.17	1.20	0.02	3.48E-05	3
5	300	30	1.12	1.15	0.02	3.03E-05	3
5.5	330	30	1.08	1.10	0.02	2.52E-05	2
6	360	30	1.04	1.06	0.02	2.61E-05	2
7.5	450	90	0.92	0.98	0.05	2.82E-05	2
8	480	30	0.87	0.90	0.02	3.83E-05	3
9	540	60	0.8	0.84	0.04	2.87E-05	2
10	600	60	0.75	0.78	0.03	2.20E-05	2
11	660	60	0.7	0.73	0.03	2.34E-05	2
12	720	60	0.65	0.68	0.03	2.50E-05	2
14	840	120	0.57	0.61	0.06	2.20E-05	2
16	960	120	0.5	0.54	0.06	2.17E-05	2
20	1200	240	0.44	0.47	0.06	1.05E-05	1
25	1500	300	0.39	0.42	0.05	7.81E-06	1
30	1800	300	0.35	0.37	0.05	6.91E-06	1
35	2100	300	0.31	0.33	0.05	7.64E-06	1
40	2400	300	0.29	0.30	0.03	4.14E-06	0
137	8220	5820	0.13	0.21	0.35	2.61E-06	0

CLIENT: Te Rapa Waikato Racing Club
PROJECT: Te Rapa Racecourse Redevelopment
LOCATION: Ken Browne Drive, Hamilton
JOB NUMBER: HAM2016_0109
TEST DATE: 30/05/2017

Test ID: HA05

CMW Geosciences

FALLING HEAD PERMEABILITY TEST HA05 RESULT



Refer to HA05 Engineering Log for soil description. Following presoaking of hole, groundwater was measured at 1.75m below ground level.

Reference: Appendix 4, Control of Groundwater for Temporary Works (CIRIA Report No. 113)

Borehole diameter = 100 mm

Hydraulic conductivity

$$k = \left(\frac{log\left(\frac{h_1}{h_2}\right) - log\left(\frac{\alpha h_1 + 1}{\alpha h_2 + 1}\right)}{(t_2 - t_1)} \right) \text{x } l$$

where l = average piezometric head over chosen time interval

$$= \frac{(h_1 + h_2)}{2}$$

h_1 = piezometric head at start of chosen interval (m)
h_2 = piezometric head at end of chosen interval (m)
t_2 - t_1 = chosen time interval (seconds)

$$\alpha = \frac{\pi d}{\left(\frac{\pi d^2}{2}\right)} \quad = \quad 20.0$$

Elapsed Time		t2 - t1	Piezometric Head	l	log (h_1/h_2)	Hydraulic Conductivity	
	(s)	(secs)	h (m)	(m)		k (m/sec)	k (m/day)
0	0		1.37				
0.5	30	30	1.34	1.36	0.01	1.55E-05	1
1	60	30	1.3	1.32	0.01	2.11E-05	2
1.5	90	30	1.27	1.29	0.01	1.63E-05	1
2	120	30	1.24	1.26	0.01	1.66E-05	1
2.5	150	30	1.22	1.23	0.01	1.13E-05	1
3	180	30	1.19	1.21	0.01	1.73E-05	1
3.5	210	30	1.17	1.18	0.01	1.18E-05	1
4	240	30	1.15	1.16	0.01	1.20E-05	1
4.5	270	30	1.13	1.14	0.01	1.22E-05	1
5	300	30	1.11	1.12	0.01	1.24E-05	1
6	360	60	1.08	1.10	0.01	9.48E-06	1
7	420	60	1.03	1.06	0.02	1.64E-05	1
8	480	60	1	1.02	0.01	1.02E-05	1
9	540	60	0.97	0.99	0.01	1.05E-05	1
10	600	60	0.96	0.97	0.00	3.57E-06	0
12	720	120	0.9	0.93	0.03	1.11E-05	1
14	840	120	0.86	0.88	0.02	7.79E-06	1
16	960	120	0.82	0.84	0.02	8.14E-06	1
18	1080	120	0.8	0.81	0.01	4.21E-06	0
20	1200	120	0.77	0.79	0.02	6.50E-06	1
22	1320	120	0.74	0.76	0.02	6.75E-06	1
25	1500	180	0.7	0.72	0.02	6.27E-06	1
30	1800	300	0.65	0.68	0.03	5.00E-06	0
35	2100	300	0.6	0.63	0.03	5.37E-06	0
40	2400	300	0.57	0.59	0.02	3.42E-06	0
50	3000	600	0.5	0.54	0.06	4.35E-06	0
60	3600	600	0.45	0.48	0.05	3.46E-06	0
85	5100	1500	0.34	0.40	0.12	3.64E-06	0
178	10680	5580	0.11	0.23	0.49	4.16E-06	0

APPENDIX F – AECOM WASTEWATER ASSESSMENT

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31 August 2017

Jackie Colliar
Infrastructure Engineer - Waters
City Development

Hamilton City Council

Dear Jackie

PSP15290 Misc. Wastewater Modelling Services - Te Rapa Racecourse Development Wastewater Capacity Assessment

1.0 Introduction

In August 2017, under the commission of PSP15290, HCC engaged AECOM to undertake a wastewater capacity assessment for the Te Rapa Race Course development. The location of the area of interest is shown in Figure 1 of Appendix A.

The objective of this assessment is to determine if the network is likely to have sufficient spare capacity to accommodate the proposed increased discharge. This assessment is undertaken for the 2061 horizon using the HCC Wastewater Model (the Model).

2.0 Assessment Methodology

The modelled system performance results were assessed for a section of the Central Interceptor (CI) and the surrounding trunk network in terms of the following:

- Spare pipe capacity.
- Maximum water level in the pipe network.
- Predicted overflows, if any.

This assessment was undertaken for the dry weather flow (DWF) simulation and for the largest wet weather event in the 10 year rainfall time series. This is an actual rainfall event that occurred on 23 January 2011.

It is assumed that if the wastewater system can cope with the proposed development for the largest actual rainfall event, then the network should be able to meet the spill frequency objective of having no more than one spill every 10 years at any location (in the vicinity of the development discharge location).

The single event simulation duration was 4.5 days, starting on 21 January 2011 at 12 pm, and ending on 26 January 2011 at 12 am. This event has been used to represent the wet weather flow (WWF).

2.1 Te Rapa Racecourse Development

Development information was provided by Hayden Vink of Wainui Environmental (WE) and is briefly summarised below. The flow calculation sheet provided by WE is shown in Figure 3 of Appendix A.

The development block is located between Sir Tristram Avenue, Te Rapa Road, and Empire Rose Drive. The following information applies to the proposed development:

i. The approximate area of the development is 6.87 hectares.

ii. The development has an estimated population density of 78 people per hectare. This is equivalent to 535 people. The current population projection for this area in the 2061 horizon is 34 people. The current projection is based on employee numbers within this development block provided in the GIS layer named *HCCNonResidentialEmployeesMay2017*.

iii. The development has an estimated peak DWF of 4.26 L/s.

iv. The development has an estimated peak WWF of 5.57 L/s. This accounts for the inflow and infiltration allowance as per the HCC ITS.

v. Development discharge into the existing manhole WWO15001 (directly into the CI).

2.2 Assessment Outcomes

Model results for the 2061 horizon were assessed and are provided for the section of the CI from one pipe section upstream of the development discharge point (WWO15002) to where it connects into the Western Interceptor at existing manhole WWM13006. The following system performance measures are provided:

- Pipe utilisation, which is the percent of pipe full. Pipe utilisation is based on the maximum water level attained within a pipe during the model simulation.
- Pipe spare capacity, which indicates how much capacity is available.

The system performance measures are provided for the following simulation events:

i. Dry weather flow (DWF).

ii. 10 year average recurrence interval (ARI) overflow event.

Pipes have been assessed based on the ITS requirement that pipelines do not flow more than 100% full during wet weather. Additional capacity may be available up to the overflow point, but this has not been assessed.

The key model findings are as follows:

- During dry weather the pipeline is between 41% and 51% full, with an estimated average spare capacity of 188 L/s for the 675mm diameter network, and 275 L/s for the 750mm diameter network.
- During wet weather the pipeline is predicted to be between 65% and 95% full with an estimated average spare capacity of 51 L/s for the 675mm diameter network, and 93 L/s for the 750mm diameter network.
- No manhole spills are predicted during the 10 year ARI event.

Table 1 Existing and proposed development flows in the 2061 horizon.

Te Rapa Racecourse Development	**Population**	**PDWF (L/s)**	**PWWF (L/s)**[1]
6.87 ha			
Calculations using HCC growth figures	34	0.21	0.41
Calculations using proposed WE figures	535	4.26	5.57
Increase in flow		**4.05**	**5.16**

The estimated increase in peak dry and wet weather flows due to the proposed development is provided in Table 1. The wet weather increase is about 10% of the estimated minimum spare capacity. It is therefore concluded that the CI has sufficient capacity to accommodate additional development flows without causing new or worsening any existing network system performance issues.

To support the outcomes of the initial assessment the Model was re-run with inclusion of the constant flow of 5.57 L/s discharging into the manhole WWO15001. The result of this simulation also confirmed no manhole spills are predicted and the maximum water level in the CI remains below pipe full. A longitudinal profile showing the maximum water level in the CI is provided in the Figure 2 of Appendix A.

Table 2 summarises the system performance results in the 2061 horizon.

---

[1] PWWF assumes to be two times of PDWF.

Table 2 Summary of system performance results

Pipe ID (From/To Manhole ID)	Pipe Diameter (mm)	Discharge at Pipe Full (L/s)	Dry Weather (DWF) Results		Wet Weather Results (10 year ARI event)	
			Maximum Pipe Full (%)	Spare Pipe Capacity (L/s)	Maximum Pipe Full (%)	Spare Pipe Capacity (L/s)
402160 (WWO15002 -> WWO15001)	675	322	45	190	78	61
402161 (WWO15001 -> WWN15030)	675	319	46	187	82	60
402162 (WWN15030 -> WWN15029)	675	314	46	183	88	56
402165 (WWN15029 -> WWN15028)	675	305	52	173	95	32
402164 (WWN15028 -> WWN15027)	675	311	51	178	92	37
402166 (WWN15027 -> WWN14008)	675	318	49	186	88	44
402167 (WWN14008 -> WWN14007)	675	330	45	198	84	55
402168 (WWN14007 -> WWN14006)	675	343	46	210	85	64
33788 (WWN14006 -> WWN14005)	750	351	49	215	87	53
33787 (WWN14005 -> WWN14004)	750	364	50	228	86	64
33786 (WWN14004 -> WWM14042)	750	409	44	270	77	96
33686 (WWM14042 -> WWM14041)	750	422	46	282	80	100
399047 (WWM14041 -> WWM14040)	750	431	43	291	76	109
399064 (WWM14040 -> WWM14039)	750	434	45	294	78	112
399069 (WWM14039 -> WWM13013)	750	438	45	298	77	103
389269&389263_1 (WWM13013 -> WWM13006-1)	750	438	45	298	71	102
389269&389263_2 (WWM13006-1 -> WWM13006)	750	438	41	298	65	100

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3.0 Disclaimer

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Yours faithfully

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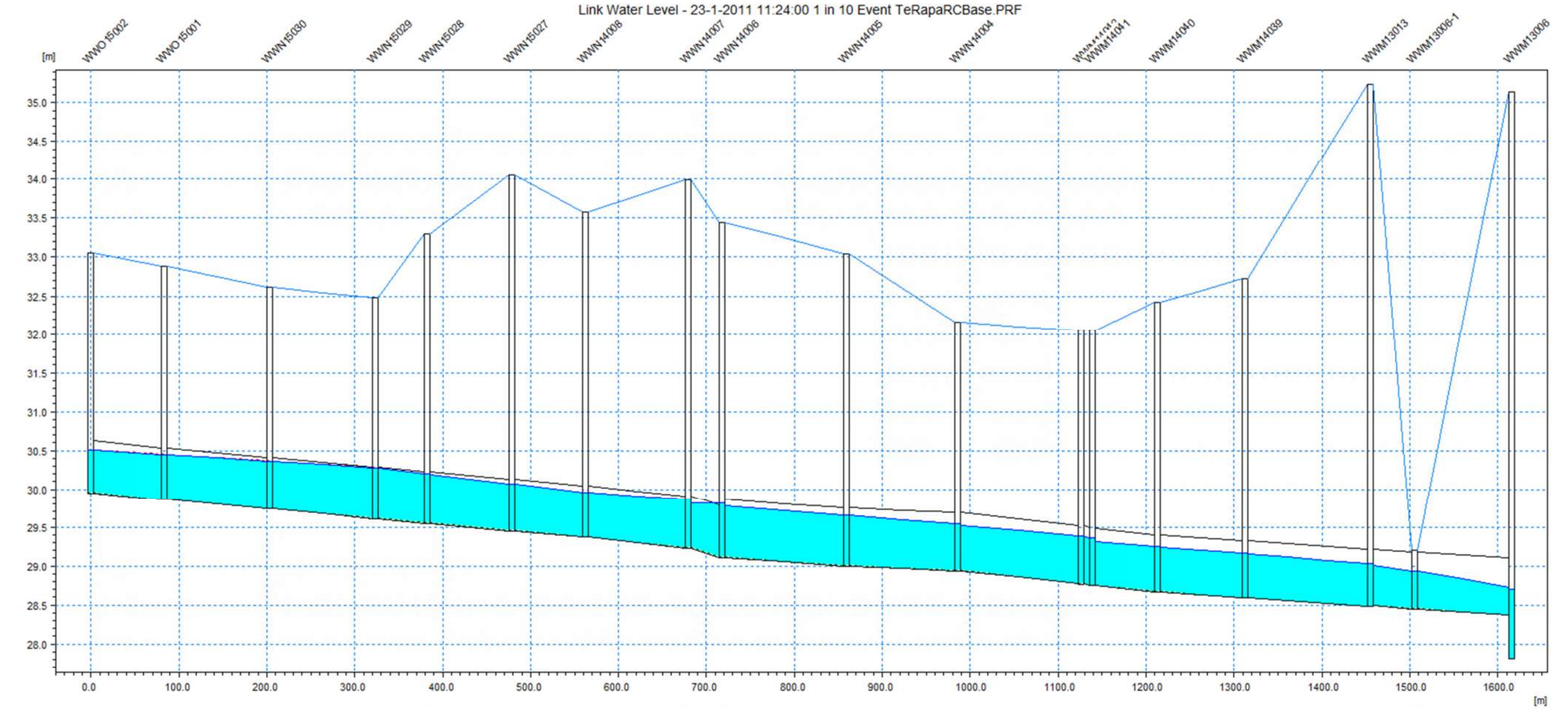
Appendix A

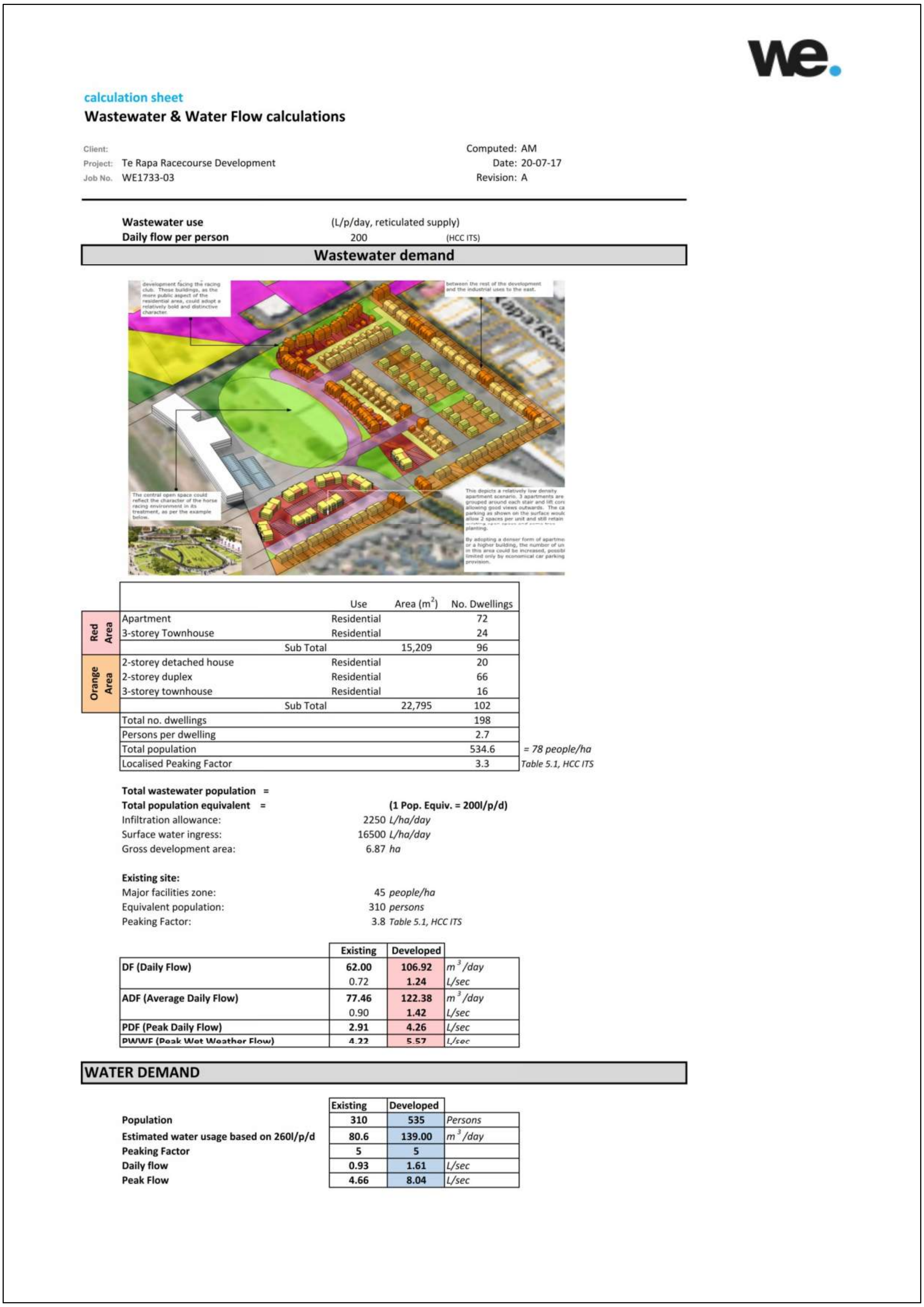
Figure 1 Location map.

Figure 2 Longitudinal profile showing the maximum water level.



Ground Lev. [m]	Invert lev. [m]	Length [m]	Diameter [m]	Slope o/oo
33.05	29.95			
		83.26	0.68	1.08
32.88	29.86			
		119.68	0.68	1.00
32.61	29.74			
		120.49	0.68	1.08
32.48	29.61			
		59.11	0.68	1.02
33.30	29.55			
		96.71	0.68	1.03
34.05	29.45			
		82.85	0.68	0.97
33.58	29.37			
		117.34	0.68	1.19
33.99	29.23			
		37.95	0.68	3.16
33.45	29.11			
		141.53	0.75	0.78
33.04	29.00			
		126.97	0.75	0.47
32.16	28.94			
		139.56	0.75	1.21
32.05	28.77			
		74.03	0.73	1.08
32.42	28.67			
		99.07	0.73	0.81
32.72	28.59			
		142.57	0.73	0.77
35.24	28.48			
		50.36	0.73	0.79
29.20	28.45			
		110.60	0.73	0.72

Figure 3 Calculation sheet provided by WE.



we.

calculation sheet

Wastewater & Water Flow calculations

Client:
Project: Te Rapa Racecourse Development
Job No. WE1733-03

Computed: AM
Date: 20-07-17
Revision: A

Wastewater use	(L/p/day, reticulated supply)	
Daily flow per person	200	(HCC ITS)

Wastewater demand

development facing the racing club. These buildings, as the more public aspect of the residential area, could adopt a relatively bold and distinctive character.

between the rest of the development and the industrial uses to the east.

The central open space could reflect the character of the horse racing environment in its treatment, as per the example below.

This depicts a relatively low density apartment scenario. 3 apartments are grouped around each stair and lift cor allowing good views outwards. The ca parking as shown on the surface woul allow 2 spaces per unit and still retain existing open space and some tree planting.

By adopting a denser form of apartm or a higher building, the number of un in this area could be increased, possib limited only by economical car parking provision.

		Use	Area (m^2)	No. Dwellings	
Red Area	Apartment	Residential		72	
	3-storey Townhouse	Residential		24	
	Sub Total		15,209	96	
Orange Area	2-storey detached house	Residential		20	
	2-storey duplex	Residential		66	
	3-storey townhouse	Residential		16	
	Sub Total		22,795	102	
	Total no. dwellings			198	
	Persons per dwelling			2.7	
	Total population			534.6	*= 78 people/ha*
	Localised Peaking Factor			3.3	*Table 5.1, HCC ITS*

Total wastewater population =
Total population equivalent = **(1 Pop. Equiv. = 200l/p/d)**

Infiltration allowance:	2250	*L/ha/day*
Surface water ingress:	16500	*L/ha/day*
Gross development area:	6.87	*ha*

Existing site:

Major facilities zone:	45	*people/ha*
Equivalent population:	310	*persons*
Peaking Factor:	3.8	*Table 5.1, HCC ITS*

	Existing	Developed	
DF (Daily Flow)	**62.00**	**106.92**	*m^3/day*
	0.72	**1.24**	*L/sec*
ADF (Average Daily Flow)	**77.46**	**122.38**	*m^3/day*
	0.90	**1.42**	*L/sec*
PDF (Peak Daily Flow)	**2.91**	**4.26**	*L/sec*
PWWF (Peak Wet Weather Flow)	**4.22**	**5.57**	*L/sec*

WATER DEMAND

	Existing	Developed	
Population	**310**	**535**	*Persons*
Estimated water usage based on 260l/p/d	**80.6**	**139.00**	*m^3/day*
Peaking Factor	**5**	**5**	
Daily flow	**0.93**	**1.61**	*L/sec*
Peak Flow	**4.66**	**8.04**	*L/sec*

APPENDIX G – MOTT MACDONALD WATER ASSESSMENT

MOTT
MACDONALD

Wainui Environmental
12 Manukau Road,
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Raglan 3265,
New Zealand

Waikato Racing Club Proposed Development – Water Impact Assessment –

Our Reference
367916

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mottmac.com

20 October 2017

This letter summarises the results of the assessment undertaken for the proposed Waikato Racing Club development consisting of 198 dwellings between Te Rapa Rd and Sir Tristram Ave. This development will be serviced from the Hamilton water supply network and will be included as part of the planned Pukete Zone.

1 Background

In September 2017 Mott MacDonald was commissioned by Wainui Environmental to assess the system performance in terms of level of Service (LOS) and firefighting capacity in the proposed Waikato Racing Club development. Further to the preliminary verification it was found that modelling should be considered to assess the impact of the additional development demand prior to the Pukete Zone closure.

In this analysis, the latest HCC water supply model was used. The existing network was updated with all recently constructed water mains in this area. One scenario was investigated, with and without additional demand from the proposed development for existing operational conditions. These are detailed in the Scenario Investigation section of this letter.

2 Assumptions

2.1 Demand Calculations

The Waikato Racing Club demand has been calculated based on a per capita flow of 260 l/day/person and a peaking factor of 5 as specified in the Hamilton City Development Manuals and confirmed by Wainui Environmental Ltd. This results in a total instantaneous peak flow of 8l/s. The demand calculation provided by Wainui Environmental is summarised in Table 1 below:

Table 1 - Wainui Environmental Demand Calculation

	Existing	Developed
Population	310	535
Estimated Average Day Water Usage (m^3/day)	80.6	139.0
Average Daily Flow (l/s)	0.93	1.61
Instantaneous Peak Flow (l/s)	4.66	8.04

2.2 Proposed Connection Points

As per client requirements, it was assumed that the development would be connected to the 200m pipeline along Tristam Ave, and to the 150mm pipe off Garnett Ave. Figure 1 below shows the proposed connection points.

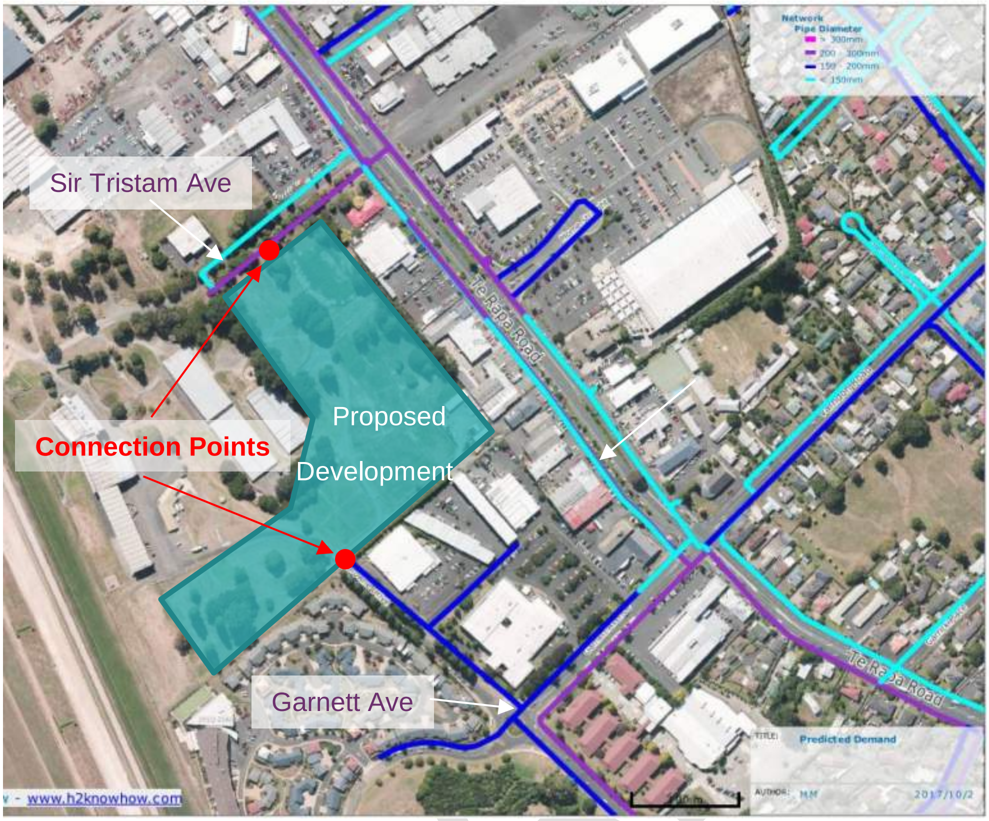

Figure 1 – Proposed Connection Points

3 Scenario Investigated

One scenario was investigated, including the following demand and zone implementation:

- **Demand**: 2021 Peak Day
- **Zone Closure**: Pukete zone open. Orange Zone extended, Maeroa, Whitiora and Rototuna Zones closed.
- **Proposed infrastructure**: it was assumed that the proposed development would be serviced through a 150mm pipe connecting the two connection points.

4 Model Results

4.1 System Performance Analysis in the proposed Development

This section describes the results of the system performance analysis undertaken for the above scenario after including the proposed development demand (maximum elevation provided by client: 37m). Results have been analysed to check that levels of service can be met in the Waikato Racing Club development without any network modification. The table below summarises the results in terms of minimum pressure and fire flow capacity.

Scenario	Minimum pressure (m)	Maximum Head losses (m/km)	Fire Flow
Prior Pukete Zone closure	21.1	1.6	Can meet residential fire flow (FW2 – 25 l/s with 10m residual pressure)

As shown in the table above, levels of service can be met in the proposed development.

4.2 System Performance Analysis in the Remaining of the Network

This section describes the results of the system performance in the remaining of the existing West Blue Zone. Results have been analysed to assess the effect of the proposed development for each scenario.

Figure 2 below shows the system performance prior to the Pukete Zone closure, after the Rototuna and Maeroa zones closure, including 2021 peak demand, prior Waikato Racing Club Development, while Figure 3 includes the proposed development demand.

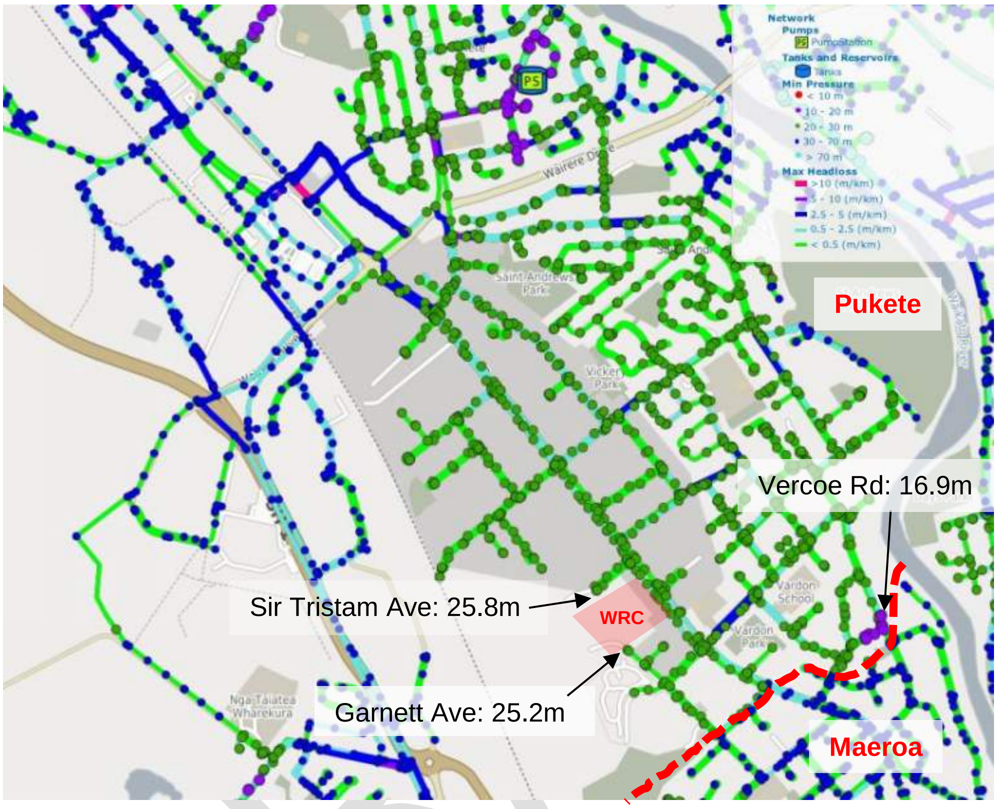

Figure 2 - System Performance excluding Waikato Racing Club Development

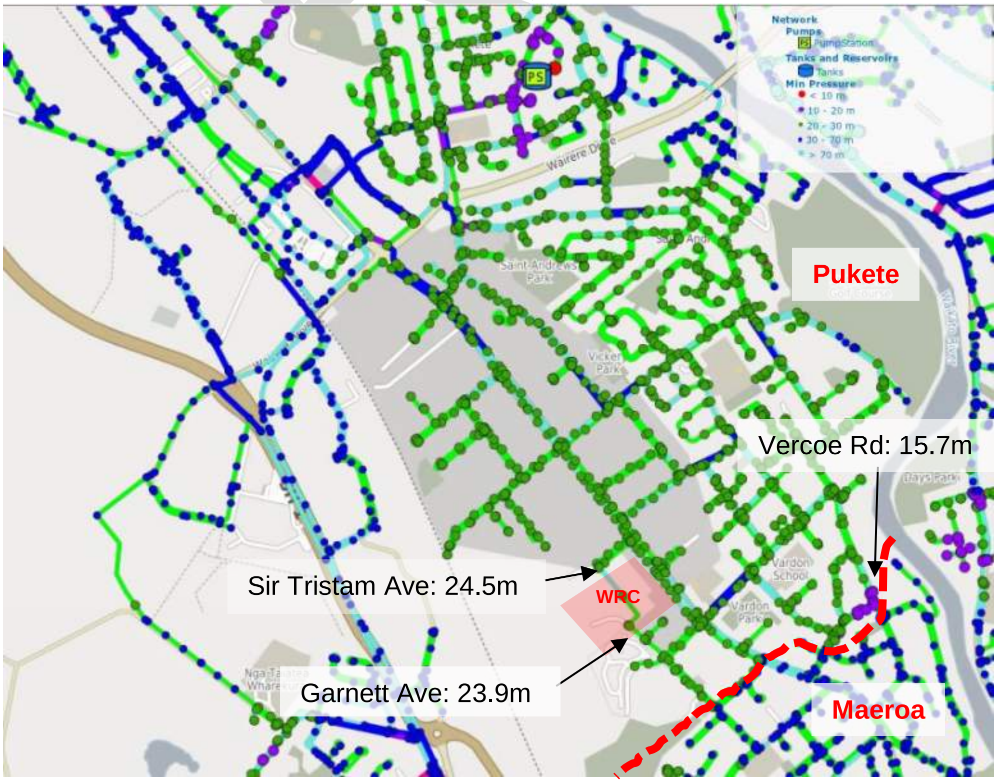

Figure 3 – System Performance including Waikato Racing Club Development

As shown in the pictures above, the proposed development is predicted to have a noticeable impact on the remaining of the water network with a maximum pressure drop of 1.3m. However, pressures are predicted to remain above the recommended level of service (20m), except along Beerescourt Rd and Vercoe Rd, where minimum pressure is predicted to drop from 16.9m to 15.7m. Minimum pressure occur when the Pukete reservoir level falls below 3m and therefore cannot service the area during peak demand period. In these operational conditions, pressure below 30m and pockets of pressure between 15 and 20m are predicted. When the Pukete reservoir pump station is active, pressure remain above 20m throughout the zone.

The table below summarises the minimum pressure forecasted at the supply point and along Vercoe Rd, before and after the proposed development:

Location	Min pressure before development (m)	Min pressure after development (m)	Pressure drop (m)
Sir Tristram Ave	25.8	24.5	1.3
Garnett Ave	25.2	23.9	1.3
Vercoe Rd	16.9	15.7	1.2

Originally, properties along Vercoe Rd and Beerescourt Rd were not included in the Maeroa Zone as additional valves and road crossings would be required. Once the Pukete Zone is closed, pressure in the service zone will remain above 30m throughout the zone.

5 Conclusions and Recommendations

Demand from the proposed Waikato Racing Club has been added to the network for short term horizon conditions (prior Pukete Zone closure) to determine if suitable levels of service could be obtained.

Levels of service are expected to be met in the proposed development in terms of pressure, head losses and firefighting capacity. However minimum pressures in the remaining network are forecasted to drop by 1.3m due to the additional demand. Pressure along Vercoe Rd is predicted to drop from 16.9 to 15.7m. This is an existing level of service issue related to the operation of the Pukete Reservoir. To maintain pressure above 20m throughout the zone, the Pukete reservoir pump station needs to be active during peak demand periods.

Julie Plessis
Hydraulic Engineer
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APPENDIX F – AECOM WASTEWATER ASSESSMENT

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31 August 2017

Jackie Colliar
Infrastructure Engineer - Waters
City Development

Hamilton City Council

Dear Jackie

PSP15290 Misc. Wastewater Modelling Services - Te Rapa Racecourse Development Wastewater Capacity Assessment

1.0 Introduction

In August 2017, under the commission of PSP15290, HCC engaged AECOM to undertake a wastewater capacity assessment for the Te Rapa Race Course development. The location of the area of interest is shown in Figure 1 of Appendix A.

The objective of this assessment is to determine if the network is likely to have sufficient spare capacity to accommodate the proposed increased discharge. This assessment is undertaken for the 2061 horizon using the HCC Wastewater Model (the Model).

2.0 Assessment Methodology

The modelled system performance results were assessed for a section of the Central Interceptor (CI) and the surrounding trunk network in terms of the following:

- Spare pipe capacity.
- Maximum water level in the pipe network.
- Predicted overflows, if any.

This assessment was undertaken for the dry weather flow (DWF) simulation and for the largest wet weather event in the 10 year rainfall time series. This is an actual rainfall event that occurred on 23 January 2011.

It is assumed that if the wastewater system can cope with the proposed development for the largest actual rainfall event, then the network should be able to meet the spill frequency objective of having no more than one spill every 10 years at any location (in the vicinity of the development discharge location).

The single event simulation duration was 4.5 days, starting on 21 January 2011 at 12 pm, and ending on 26 January 2011 at 12 am. This event has been used to represent the wet weather flow (WWF).

2.1 Te Rapa Racecourse Development

Development information was provided by Hayden Vink of Wainui Environmental (WE) and is briefly summarised below. The flow calculation sheet provided by WE is shown in Figure 3 of Appendix A.

The development block is located between Sir Tristram Avenue, Te Rapa Road, and Empire Rose Drive. The following information applies to the proposed development:

i. The approximate area of the development is 6.87 hectares.

ii. The development has an estimated population density of 78 people per hectare. This is equivalent to 535 people. The current population projection for this area in the 2061 horizon is 34 people. The current projection is based on employee numbers within this development block provided in the GIS layer named *HCCNonResidentialEmployeesMay2017*.

iii. The development has an estimated peak DWF of 4.26 L/s.

iv. The development has an estimated peak WWF of 5.57 L/s. This accounts for the inflow and infiltration allowance as per the HCC ITS.

v. Development discharge into the existing manhole WWO15001 (directly into the CI).

2.2 Assessment Outcomes

Model results for the 2061 horizon were assessed and are provided for the section of the CI from one pipe section upstream of the development discharge point (WWO15002) to where it connects into the Western Interceptor at existing manhole WWM13006. The following system performance measures are provided:

- Pipe utilisation, which is the percent of pipe full. Pipe utilisation is based on the maximum water level attained within a pipe during the model simulation.
- Pipe spare capacity, which indicates how much capacity is available.

The system performance measures are provided for the following simulation events:

i. Dry weather flow (DWF).

ii. 10 year average recurrence interval (ARI) overflow event.

Pipes have been assessed based on the ITS requirement that pipelines do not flow more than 100% full during wet weather. Additional capacity may be available up to the overflow point, but this has not been assessed.

The key model findings are as follows:

- During dry weather the pipeline is between 41% and 51% full, with an estimated average spare capacity of 188 L/s for the 675mm diameter network, and 275 L/s for the 750mm diameter network.
- During wet weather the pipeline is predicted to be between 65% and 95% full with an estimated average spare capacity of 51 L/s for the 675mm diameter network, and 93 L/s for the 750mm diameter network.
- No manhole spills are predicted during the 10 year ARI event.

Table 1 Existing and proposed development flows in the 2061 horizon.

Te Rapa Racecourse Development	Population	PDWF (L/s)	PWWF (L/s)[1]
6.87 ha			
Calculations using HCC growth figures	34	0.21	0.41
Calculations using proposed WE figures	535	4.26	5.57
Increase in flow		**4.05**	**5.16**

The estimated increase in peak dry and wet weather flows due to the proposed development is provided in Table 1. The wet weather increase is about 10% of the estimated minimum spare capacity. It is therefore concluded that the CI has sufficient capacity to accommodate additional development flows without causing new or worsening any existing network system performance issues.

To support the outcomes of the initial assessment the Model was re-run with inclusion of the constant flow of 5.57 L/s discharging into the manhole WWO15001. The result of this simulation also confirmed no manhole spills are predicted and the maximum water level in the CI remains below pipe full. A longitudinal profile showing the maximum water level in the CI is provided in the Figure 2 of Appendix A.

Table 2 summarises the system performance results in the 2061 horizon.

---

[1] PWWF assumes to be two times of PDWF.

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Table 2 Summary of system performance results

Pipe ID (From/To Manhole ID)	Pipe Diameter (mm)	Discharge at Pipe Full (L/s)	Dry Weather (DWF) Results		Wet Weather Results (10 year ARI event)	
			Maximum Pipe Full (%)	Spare Pipe Capacity (L/s)	Maximum Pipe Full (%)	Spare Pipe Capacity (L/s)
402160 (WWO15002 -> WWO15001)	675	322	45	190	78	61
402161 (WWO15001 -> WWN15030)	675	319	46	187	82	60
402162 (WWN15030 -> WWN15029)	675	314	46	183	88	56
402165 (WWN15029 -> WWN15028)	675	305	52	173	95	32
402164 (WWN15028 -> WWN15027)	675	311	51	178	92	37
402166 (WWN15027 -> WWN14008)	675	318	49	186	88	44
402167 (WWN14008 -> WWN14007)	675	330	45	198	84	55
402168 (WWN14007 -> WWN14006)	675	343	46	210	85	64
33788 (WWN14006 -> WWN14005)	750	351	49	215	87	53
33787 (WWN14005 -> WWN14004)	750	364	50	228	86	64
33786 (WWN14004 -> WWM14042)	750	409	44	270	77	96
33686 (WWM14042 -> WWM14041)	750	422	46	282	80	100
399047 (WWM14041 -> WWM14040)	750	431	43	291	76	109
399064 (WWM14040 -> WWM14039)	750	434	45	294	78	112
399069 (WWM14039 -> WWM13013)	750	438	45	298	77	103
389269&389263_1 (WWM13013 -> WWM13006-1)	750	438	45	298	71	102
389269&389263_2 (WWM13006-1 -> WWM13006)	750	438	41	298	65	100

3.0 Disclaimer

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Yours faithfully

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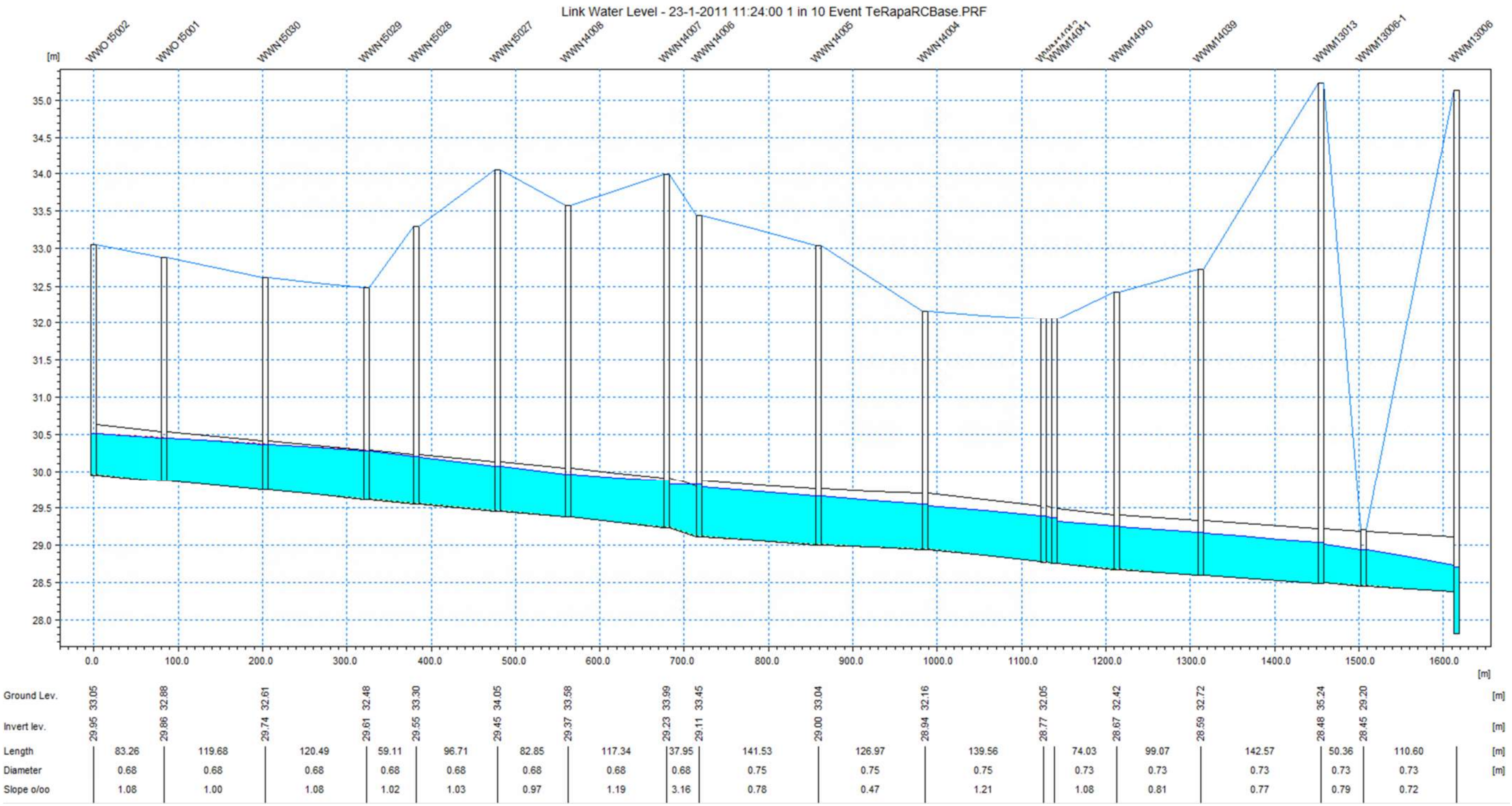
Appendix A

Figure 1 Location map.

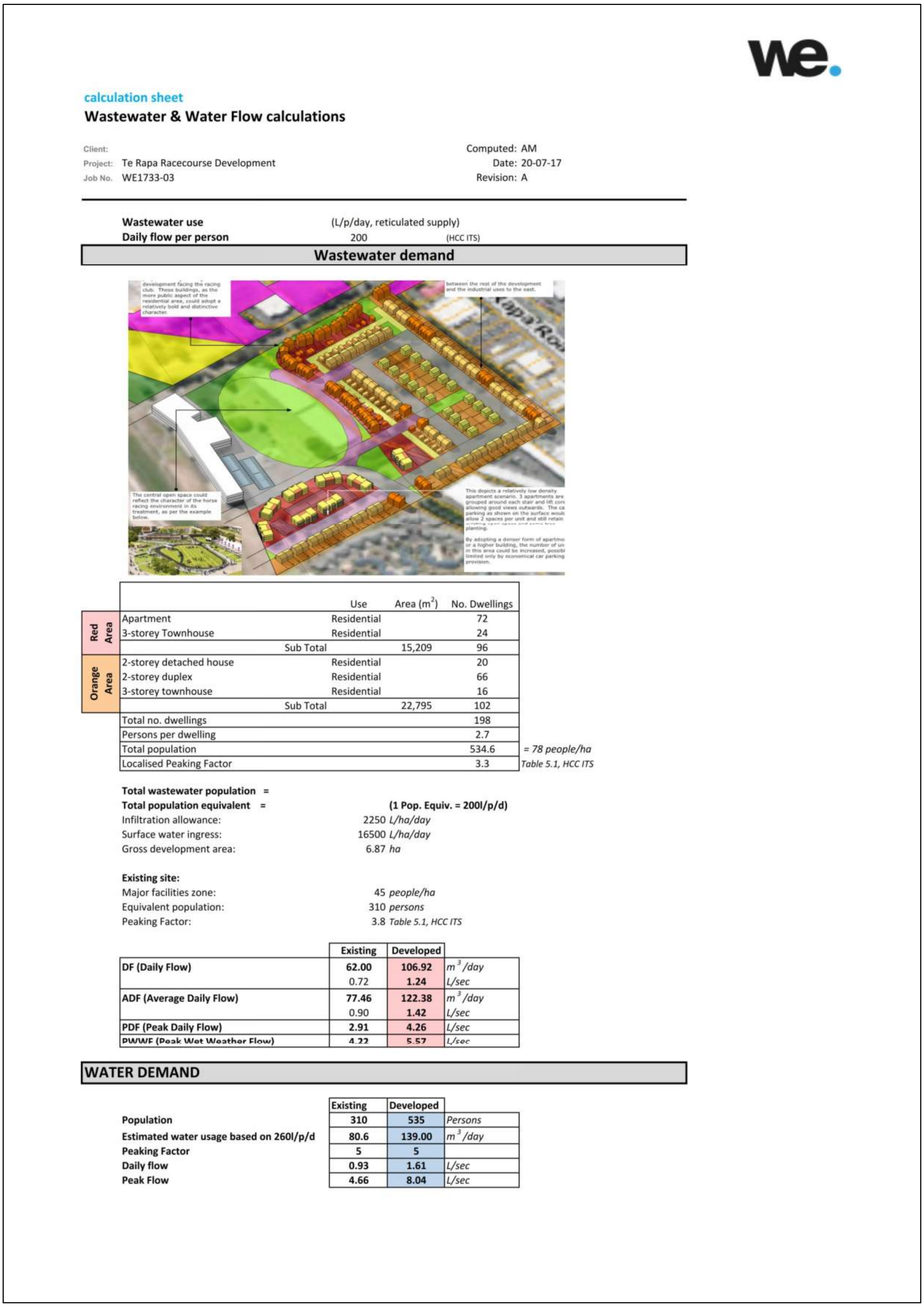
Figure 2 Longitudinal profile showing the maximum water level.



Ground Lev.	33.05	32.88	32.61	32.48	33.30	34.05	33.58	33.99	33.45	33.04	32.16	32.05	32.42	32.72	35.24	29.20	[m]
Invert lev.	29.95	29.86	29.74	29.61	29.55	29.45	29.37	29.23	29.11	29.00	28.94	28.77	28.67	28.59	28.48	28.45	[m]

Length	83.26	119.68	120.49	59.11	96.71	82.85	117.34	37.95	141.53	126.97	139.56	74.03	99.07	142.57	50.36	110.60	[m]
Diameter	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.75	0.75	0.75	0.73	0.73	0.73	0.73	0.73	[m]
Slope o/oo	1.08	1.00	1.08	1.02	1.03	0.97	1.19	3.16	0.78	0.47	1.21	1.08	0.81	0.77	0.79	0.72	

Figure 3 Calculation sheet provided by WE.



we.

calculation sheet

Wastewater & Water Flow calculations

Client:
Project: Te Rapa Racecourse Development
Job No. WE1733-03

Computed: AM
Date: 20-07-17
Revision: A

Wastewater use (L/p/day, reticulated supply)
Daily flow per person 200 (HCC ITS)

Wastewater demand

development facing the racing club. These buildings, as the more public aspect of the residential area, could adopt a relatively bold and distinctive character.

between the rest of the development and the industrial uses to the east.

The central open space could reflect the character of the horse racing environment in its treatment, as per the example below.

This depicts a relatively low density apartment scenario. 3 apartments are grouped around each stair and lift cor allowing good views outwards. The ca parking as shown on the surface woul allow 2 spaces per unit and still retain planting.

By adopting a denser form of apartme or a higher building, the number of un in this area could be increased, possib limited only by economical car parking provision.

		Use	Area (m^2)	No. Dwellings	
Red Area	Apartment	Residential		72	
	3-storey Townhouse	Residential		24	
	Sub Total		15,209	96	
Orange Area	2-storey detached house	Residential		20	
	2-storey duplex	Residential		66	
	3-storey townhouse	Residential		16	
	Sub Total		22,795	102	
	Total no. dwellings			198	
	Persons per dwelling			2.7	
	Total population			534.6	*= 78 people/ha*
	Localised Peaking Factor			3.3	*Table 5.1, HCC ITS*

Total wastewater population =
Total population equivalent = **(1 Pop. Equiv. = 200l/p/d)**
Infiltration allowance: 2250 *L/ha/day*
Surface water ingress: 16500 *L/ha/day*
Gross development area: 6.87 *ha*

Existing site:
Major facilities zone: 45 *people/ha*
Equivalent population: 310 *persons*
Peaking Factor: 3.8 *Table 5.1, HCC ITS*

	Existing	Developed	
DF (Daily Flow)	**62.00**	**106.92**	*m^3/day*
	0.72	**1.24**	*L/sec*
ADF (Average Daily Flow)	**77.46**	**122.38**	*m^3/day*
	0.90	**1.42**	*L/sec*
PDF (Peak Daily Flow)	**2.91**	**4.26**	*L/sec*
PWWF (Peak Wet Weather Flow)	**4.22**	**5.57**	*L/sec*

WATER DEMAND

	Existing	Developed	
Population	**310**	**535**	*Persons*
Estimated water usage based on 260l/p/d	**80.6**	**139.00**	*m^3/day*
Peaking Factor	**5**	**5**	
Daily flow	**0.93**	**1.61**	*L/sec*
Peak Flow	**4.66**	**8.04**	*L/sec*

APPENDIX G – MOTT MACDONALD WATER ASSESSMENT

MOTT MACDONALD

Wainui Environmental
12 Manukau Road,
Raglan,
Raglan 3265,
New Zealand

Waikato Racing Club Proposed Development – Water Impact Assessment –

20 October 2017

Our Reference
367916

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Wynyard Quarter
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This letter summarises the results of the assessment undertaken for the proposed Waikato Racing Club development consisting of 198 dwellings between Te Rapa Rd and Sir Tristram Ave. This development will be serviced from the Hamilton water supply network and will be included as part of the planned Pukete Zone.

1 Background

In September 2017 Mott MacDonald was commissioned by Wainui Environmental to assess the system performance in terms of level of Service (LOS) and firefighting capacity in the proposed Waikato Racing Club development. Further to the preliminary verification it was found that modelling should be considered to assess the impact of the additional development demand prior to the Pukete Zone closure.

In this analysis, the latest HCC water supply model was used. The existing network was updated with all recently constructed water mains in this area. One scenario was investigated, with and without additional demand from the proposed development for existing operational conditions. These are detailed in the Scenario Investigation section of this letter.

2 Assumptions

2.1 Demand Calculations

The Waikato Racing Club demand has been calculated based on a per capita flow of 260 l/day/person and a peaking factor of 5 as specified in the Hamilton City Development Manuals and confirmed by Wainui Environmental Ltd. This results in a total instantaneous peak flow of 8l/s. The demand calculation provided by Wainui Environmental is summarised in Table 1 below:

Table 1 - Wainui Environmental Demand Calculation

	Existing	Developed
Population	310	535
Estimated Average Day Water Usage (m^3/day)	80.6	139.0
Average Daily Flow (l/s)	0.93	1.61
Instantaneous Peak Flow (l/s)	4.66	8.04

2.2 Proposed Connection Points

As per client requirements, it was assumed that the development would be connected to the 200m pipeline along Tristam Ave, and to the 150mm pipe off Garnett Ave. Figure 1 below shows the proposed connection points.

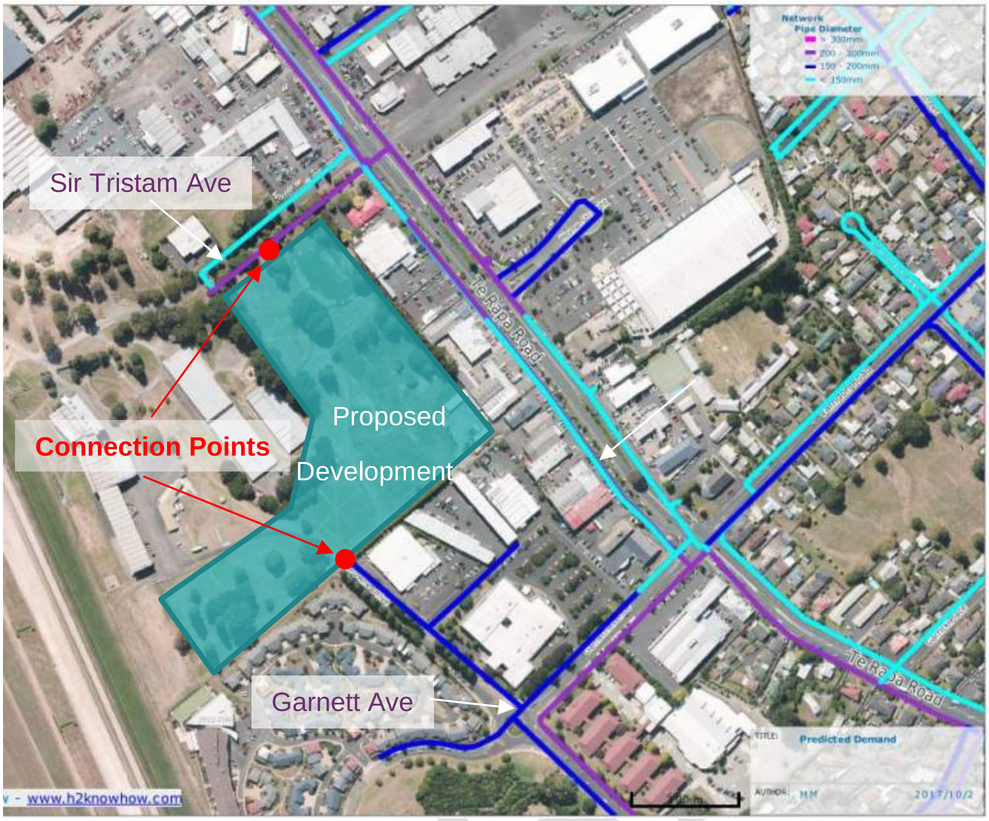


Figure 1 – Proposed Connection Points

3 Scenario Investigated

One scenario was investigated, including the following demand and zone implementation:

- **Demand**: 2021 Peak Day
- **Zone Closure**: Pukete zone open. Orange Zone extended, Maeroa, Whitiora and Rototuna Zones closed.
- **Proposed infrastructure**: it was assumed that the proposed development would be serviced through a 150mm pipe connecting the two connection points.

4 Model Results

4.1 System Performance Analysis in the proposed Development

This section describes the results of the system performance analysis undertaken for the above scenario after including the proposed development demand (maximum elevation provided by client: 37m). Results have been analysed to check that levels of service can be met in the Waikato Racing Club development without any network modification. The table below summarises the results in terms of minimum pressure and fire flow capacity.

Scenario	Minimum pressure (m)	Maximum Head losses (m/km)	Fire Flow
Prior Pukete Zone closure	21.1	1.6	Can meet residential fire flow (FW2 – 25 l/s with 10m residual pressure)

As shown in the table above, levels of service can be met in the proposed development.

4.2 System Performance Analysis in the Remaining of the Network

This section describes the results of the system performance in the remaining of the existing West Blue Zone. Results have been analysed to assess the effect of the proposed development for each scenario.

Figure 2 below shows the system performance prior to the Pukete Zone closure, after the Rototuna and Maeroa zones closure, including 2021 peak demand, prior Waikato Racing Club Development, while Figure 3 includes the proposed development demand.

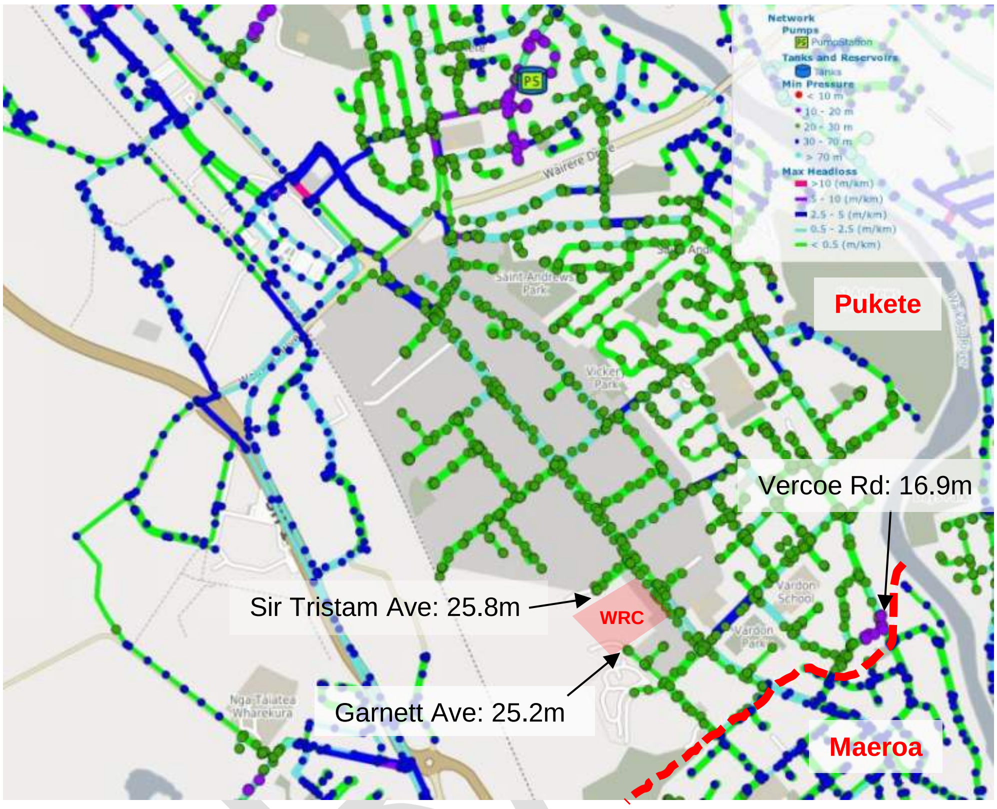


Figure 2 - System Performance excluding Waikato Racing Club Development

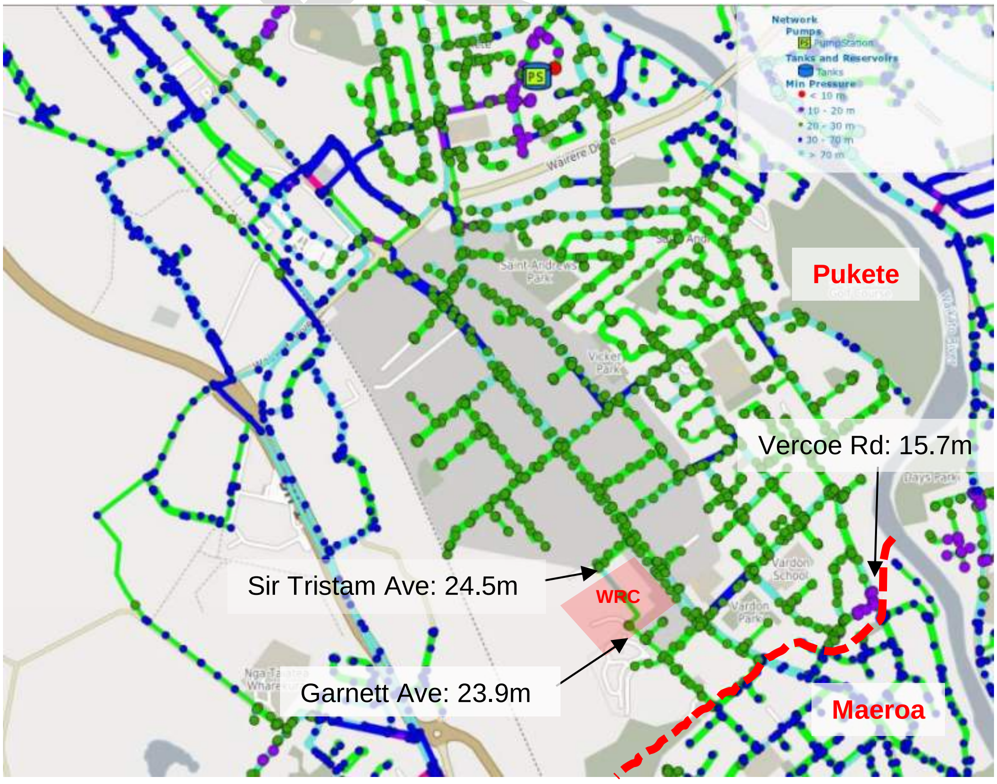


Figure 3 – System Performance including Waikato Racing Club Development

As shown in the pictures above, the proposed development is predicted to have a noticeable impact on the remaining of the water network with a maximum pressure drop of 1.3m. However, pressures are predicted to remain above the recommended level of service (20m), except along Beerescourt Rd and Vercoe Rd, where minimum pressure is predicted to drop from 16.9m to 15.7m. Minimum pressure occur when the Pukete reservoir level falls below 3m and therefore cannot service the area during peak demand period. In these operational conditions, pressure below 30m and pockets of pressure between 15 and 20m are predicted. When the Pukete reservoir pump station is active, pressure remain above 20m throughout the zone.

The table below summarises the minimum pressure forecasted at the supply point and along Vercoe Rd, before and after the proposed development:

Location	Min pressure before development (m)	Min pressure after development (m)	Pressure drop (m)
Sir Tristram Ave	25.8	24.5	1.3
Garnett Ave	25.2	23.9	1.3
Vercoe Rd	16.9	15.7	1.2

Originally, properties along Vercoe Rd and Beerescourt Rd were not included in the Maeroa Zone as additional valves and road crossings would be required. Once the Pukete Zone is closed, pressure in the service zone will remain above 30m throughout the zone.

5 Conclusions and Recommendations

Demand from the proposed Waikato Racing Club has been added to the network for short term horizon conditions (prior Pukete Zone closure) to determine if suitable levels of service could be obtained.

Levels of service are expected to be met in the proposed development in terms of pressure, head losses and firefighting capacity. However minimum pressures in the remaining network are forecasted to drop by 1.3m due to the additional demand. Pressure along Vercoe Rd is predicted to drop from 16.9 to 15.7m. This is an existing level of service issue related to the operation of the Pukete Reservoir. To maintain pressure above 20m throughout the zone, the Pukete reservoir pump station needs to be active during peak demand periods.

Julie Plessis
Hydraulic Engineer
Julie.plessis@mottmac.com

APPENDIX H – STORMWATER RETICULATION CALCULATIONS

calculation sheet

MANNING'S PIPE DESIGN

Client:	C/- BBO Ltd		**Computed:**	MRS
Project:	TE RAPA RACECOURSE		**Date:**	28/07/2021
Job No.	WE1733-03		**Revision:**	A

RATIONAL METHOD CALCULATIONS

Notes:

1. C FACTORS calcuated based on 0.9 for impervious surfaces and 0.3 for pervious surfaces (refer NZBC E1 - Surface Water, table 1 - Run-off Co-efficients)
2. ToC assumed as 10 minutes
3. Impervious areas for developed area is assumed as 80% impervious

C perv = 0.30

C Impervious = 0.90

(Hirds v4 RCP6.0 for the period 2081-2100 for climate change)

	EXISTING CLIMATE		**CLIMATE CHANGE**
Rainfall Intensity (2yr ARI-10 mins storm) =	59.1	mm/hr	75.7
Rainfall Intensity (2yr ARI-20mins storm) =	40.5	mm/hr	51.9
Rainfall Intensity (5yr ARI-10 mins storm) =	77.6		100.4
Rainfall Intensity (5yr ARI-20mins storm) =	53.1		68.7
Rainfall Intensity (10yr ARI-10 mins storm) =	91.7	mm/hr	119.3
Rainfall Intensity (10yr ARI-20mins storm) =	62.7	mm/hr	81.6
Rainfall Intensity (100yr ARI-10 mins storm) =	145.0		190.4
Rainfall Intensity (100yr ARI-20mins storm) =	98.9		129.8

Climate change 2.3 degrees

CATCHMENT ID	TOTAL CATCHMENT AREA (m^2)	IMPERVIOUS AREA %	IMPERVIOUS AREA m^2	C-FACTOR	ToC	PEAK FLOWS 2YR-ARI Q2 (L/s)	PEAK FLOWS 5YR-ARI Q5 (L/s)	PEAK FLOWS 10YR-ARI Q10 (L/s)	PEAK FLOWS 100YR-ARI Q100 (L/s)
UPSTREAM CATCHMENT TO SWO15031	79550.0	57%	45318.1	0.64	10mins	1073.36	1424.54	1692.35	2699.66
UPSTREAM CATCHMENT TO SWO15004	119544.3	28%	33713.9	0.47	20mins	808.10	1070.92	1271.28	2022.97
UPSTREAM CATCHMENT TO SWO15006	5989.9	94%	5601.7	0.86	10mins	108.44	143.91	170.97	272.74
UPSTREAM CATCHMENT TO CONNECTION	35417.7	90%	31875.9	0.84	10mins	625.46	830.09	986.15	1573.13

MANNING'S PIPE DESIGN

Client:	PRAGMA	Computed:	MRS	AM
Project:	126 HORSHAM ROAD (STAGE 3A)	Date:	14/04/2021	18/05/2021
Job No.	WE1636-55-02	Revision:	A	B

SW NETWORK

EXISTING PIPE - 2YR ARI

SWMH		CATCHMENT	Q (cumulative)	DIAM	SLOPE	U/S IL	D/S IL	LENGTH	MANHOLE INTERNAL DROP (m)	CHANGE IN HEIGHT	MANNINGS	PIPE CAPACITY	VEL	PIPE TIME	capacity
FROM	TO		L/S	mm	%	mRL	mRL	m		m	n	L/S	m/s	min	check
SWO15031	SWO15004		1073.36	1050	0.21	30.490	30.050	214.00	0.06	0.44	0.012	1341.4	1.55	2.30	Ok
SWO15004	SWO15006		1881.45	1050	0.16	29.990	29.840	94.00	0.05	0.15	0.012	1181.7	1.36	1.15	No
SWO15006	SWO16057		2615.35	1050	0.11	29.790	29.560	213.60	-	0.23	0.012	970.7	1.12	3.18	No

EXISTING PIPE - 5YR ARI

SWMH		CATCHMENT	Q (cumulative)	DIAM	SLOPE	U/S IL	D/S IL	LENGTH	MANHOLE INTERNAL DROP (m)	CHANGE IN HEIGHT	MANNINGS	PIPE CAPACITY	VEL	PIPE TIME	capacity
FROM	TO		L/S	mm	%	mRL	mRL	m		m	n	L/S	m/s	min	check
SWO15031	SWO15004		1424.54	1050	0.21	30.490	30.050	214.00	0.06	0.44	0.012	1341.4	1.55	2.30	No
SWO15004	SWO15006		2495.46	1050	0.16	29.990	29.840	94.00	0.05	0.15	0.012	1181.7	1.36	1.15	No
SWO15006	SWO16057		3469.47	1050	0.11	29.790	29.560	213.60	-	0.23	0.012	970.7	1.12	3.18	No

EXISTING PIPE - 10YR ARI

SWMH		CATCHMENT	Q (cumulative)	DIAM	SLOPE	U/S IL	D/S IL	LENGTH	MANHOLE INTERNAL DROP (m)	CHANGE IN HEIGHT	MANNINGS	PIPE CAPACITY	VEL	PIPE TIME	capacity
FROM	TO		L/S	mm	%	mRL	mRL	m		m	n	L/S	m/s	min	check
SWO15031	SWO15004		1692.35	1050	0.21	30.490	30.050	214.00	0.06	0.44	0.012	1341.4	1.55	2.30	No
SWO15004	SWO15006		2963.63	1050	0.16	29.990	29.840	94.00	0.05	0.15	0.012	1181.7	1.36	1.15	No
SWO15006	SWO16057		4120.75	1050	0.11	29.790	29.560	213.60	-	0.23	0.012	970.7	1.12	3.18	No

EXISTING PIPE - 100YR ARI

SWMH		CATCHMENT	Q (cumulative)	DIAM	SLOPE	U/S IL	D/S IL	LENGTH	MANHOLE INTERNAL DROP (m)	CHANGE IN HEIGHT	MANNINGS	PIPE CAPACITY	VEL	PIPE TIME	capacity
FROM	TO		L/S	mm	%	mRL	mRL	m		m	n	L/S	m/s	min	check
SWO15031	SWO15004		2699.66	1050	0.21	30.490	30.050	214.00	0.06	0.44	0.012	1341.4	1.55	2.30	No
SWO15004	SWO15006		4722.64	1050	0.16	29.990	29.840	94.00	0.05	0.15	0.012	1181.7	1.36	1.15	No
SWO15006	SWO16057		6568.50	1050	0.11	29.790	29.560	213.60	-	0.23	0.012	970.7	1.12	3.18	No

calculation sheet

SSA RESULTS - SHEET 1 OF 2

Client:	C/- BBO Ltd	Computed:	MRS
Project:	TE RAPA RACECOURSE	Date:	8/08/2021
Job No.	WE1733-03	Revision:	A

PLAN VIEW

Analisys Assumptions:

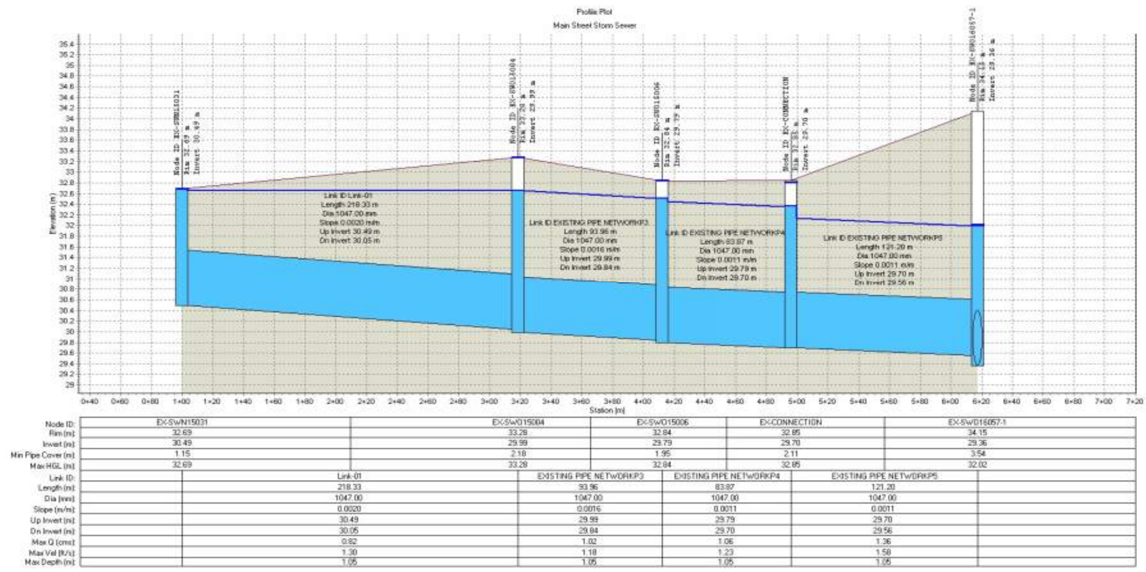
1. Hydrology method used is Rational.
2. Rainfall was obtained from HIRDS V4 with 2.3 degrees for climate change.

RAINFALL INTENSITY (mm/h)	10 mins	20 mins
2-YR ARI	75.68	51.86
5-YR ARI	100.45	68.73
10-YR ARI	119.33	81.59
100-YR ARI	190.36	129.84

3. ToC assumed 10 minutes for the existing developed catchments and 20 minutes for the undeveloped catchment.
4. Imperviousness was measured from aerial image.
5. C FACTORS calcuated based on 0.9 for impervious surfaces and 0.3 for pervious surfaces (refer NZBC E1 - Surface Water, table 1 - Run-off Co-efficients)
6. Hydraulic routing method is hydrodynamic.
7. Exisitng pipes and manhole parameters were obtained from HCC GIS Viewer.

LONG SECTIONS RESULTS

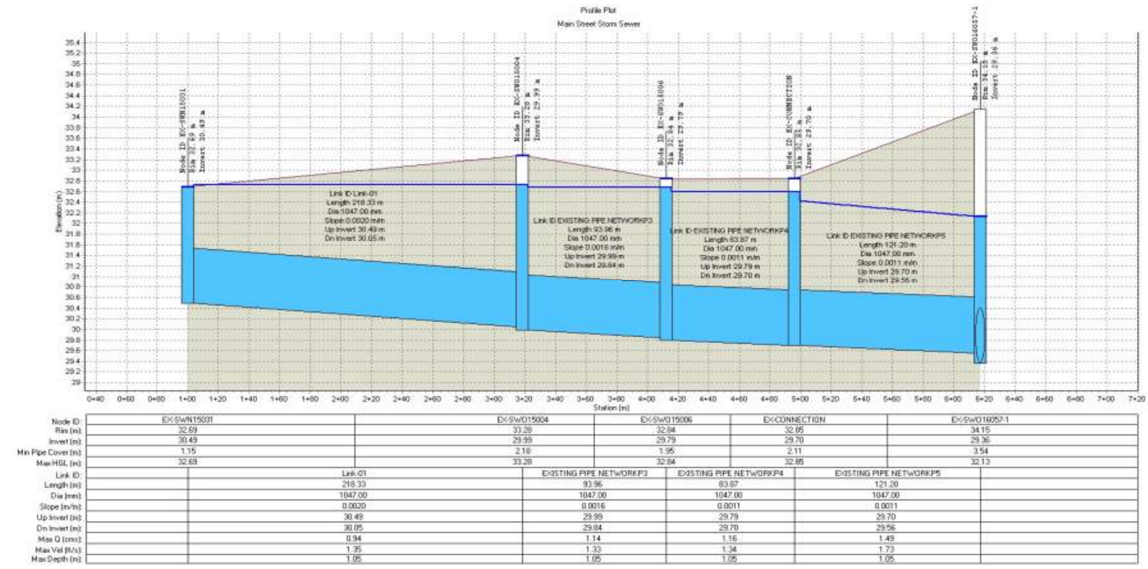
2 YR ARI



Node ID:	EX-SWN15031	EX-SWO15004	EX-SWO15006	EX-CONNECTION	EX-SWO16057-1
Rim (m):	32.69	33.28	32.84	32.85	34.15
Invert (m):	30.49	29.99	29.79	29.70	29.36
Min Pipe Cover (m):	1.15	2.18	1.95	2.11	3.54
Max HGL (m):	32.69	33.28	32.84	32.85	32.02

Link ID:	Link-01	EXISTING PIPE NETWORKP3	EXISTING PIPE NETWORKP4	EXISTING PIPE NETWORKP5
Length (m):	218.33	93.96	83.87	121.20
Dia (mm):	1047.00	1047.00	1047.00	1047.00
Slope (m/m):	0.0020	0.0016	0.0011	0.0011
Up Invert (m):	30.49	29.99	29.79	29.70
Dn Invert (m):	30.05	29.84	29.70	29.56
Max Q (cms):	0.82	1.02	1.06	1.36
Max Vel (ft/s):	1.30	1.18	1.23	1.58
Max Depth (m):	1.05	1.05	1.05	1.05

5 YR ARI



Node ID:	EX-SWN15031	EX-SWO15004	EX-SWO15006	EX-CONNECTION	EX-SWO16057-1
Rim (m):	32.69	33.28	32.84	32.85	34.15
Invert (m):	30.49	29.99	29.79	29.70	29.36
Min Pipe Cover (m):	1.15	2.18	1.95	2.11	3.54
Max HGL (m):	32.69	33.28	32.84	32.85	32.13

Link ID:	Link-01	EXISTING PIPE NETWORKP3	EXISTING PIPE NETWORKP4	EXISTING PIPE NETWORKP5
Length (m):	218.33	93.96	83.87	121.20
Dia (mm):	1047.00	1047.00	1047.00	1047.00
Slope (m/m):	0.0020	0.0016	0.0011	0.0011
Up Invert (m):	30.49	29.99	29.79	29.70
Dn Invert (m):	30.05	29.84	29.70	29.56
Max Q (cms):	0.94	1.14	1.16	1.49
Max Vel (ft/s):	1.35	1.33	1.34	1.73
Max Depth (m):	1.05	1.05	1.05	1.05

calculation sheet

SSA RESULTS - SHEET 2 OF 2

Client:	C/- BBO Ltd	**Computed:**	MRS
Project:	TE RAPA RACECOURSE	**Date:**	8/08/2021
Job No.	WE1733-03	**Revision:**	A

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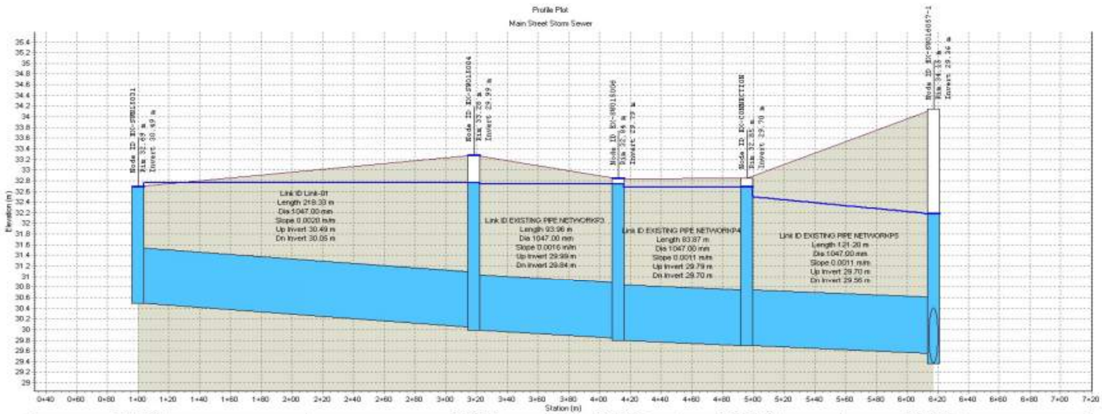
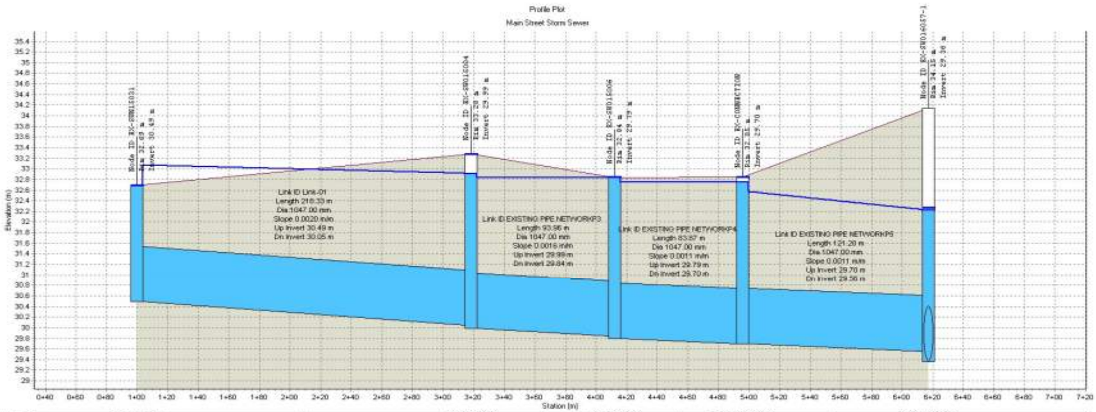
10 YR ARI



Node ID:	EX-SWN15031	EX-SWO15004	EX-SWO15006	EX-CONNECTION	EX-SWO16057-1
Rim (m):	32.69	33.28	32.84	32.85	34.15
Invert (m):	30.49	29.99	29.79	29.70	29.36
Min Pipe Cover (m):	1.15	2.18	1.95	2.11	3.54
Max HGL (m):	32.69	33.28	32.84	32.85	32.18

Link ID:		Link-01	EXISTING PIPE NETWORKP3	EXISTING PIPE NETWORKP4	EXISTING PIPE NETWORKP5	
Length (m):		218.33	93.96	83.87	121.20	
Dia (mm):		1047.00	1047.00	1047.00	1047.00	
Slope (m/m):		0.0020	0.0016	0.0011	0.0011	
Up Invert (m):		30.49	29.99	29.79	29.70	
Dn Invert (m):		30.05	29.84	29.70	29.56	
Max Q (cms):		1.01	1.19	1.19	1.55	
Max Vel (ft/s):		1.38	1.39	1.39	1.80	
Max Depth (m):		1.05	1.05	1.05	1.05	

100 YR ARI



Node ID:	EX-SWN15031	EX-SWO15004	EX-SWO15006	EX-CONNECTION	EX-SWO16057-1
Rim (m):	32.69	33.28	32.84	32.85	34.15
Invert (m):	30.49	29.99	29.79	29.70	29.36
Min Pipe Cover (m):	1.15	2.18	1.95	2.11	3.54
Max HGL (m):	32.69	33.28	32.84	32.85	32.27

Link ID:		Link-01	EXISTING PIPE NETWORKP3	EXISTING PIPE NETWORKP4	EXISTING PIPE NETWORKP5	
Length (m):		218.33	93.96	83.87	121.20	
Dia (mm):		1047.00	1047.00	1047.00	1047.00	
Slope (m/m):		0.0020	0.0016	0.0011	0.0011	
Up Invert (m):		30.49	29.99	29.79	29.70	
Dn Invert (m):		30.05	29.84	29.70	29.56	
Max Q (cms):		1.18	1.28	1.28	1.66	
Max Vel (ft/s):		1.47	1.49	1.49	1.93	
Max Depth (m):		1.05	1.05	1.05	1.05	