Tonkin+Taylor

Job No: 1007466
21 September 2018

Ministry of Education
West End
Christchurch 8013

(via email: Deb.Taylor@education.govt.nz)

Attention: Deb Taylor

Dear Deb

TKKM o Horouta Wananga – 17 Ranfurly Street, Gisborne
Geotechnical investigation and assessment report

1 Introduction

This letter presents the results of a preliminary geotechnical investigation and assessment completed by Tonkin & Taylor Ltd (T+T) for the property at 17 Ranfurly Street in Gisborne (the Site). The work described in this document was commissioned by the Ministry of Education (MoE) and has been completed in accordance with the terms and conditions outlined in the T+T letter of engagement dated 17 July 2018.

The information contained in this report is intended to support the pre-purchase assessment of the Site by MoE and identify any major geotechnical-related considerations for the potential school development. The Site is a former Mobil Oil New Zealand (Mobil) terminal that was decommissioned during the late 1980s/early 1990s. Since that time the Site has remained unused and covered with grass.

2 Scope

T+T has carried out the following scope of work for the purposes of this letter report:

- Review of existing geotechnical information for the Site (and surrounding area) from T+T files and the NZGD[1].
- Site walkover by a T+T ground contamination specialist.
- Site-specific geotechnical investigations comprising six Cone Penetrometer Tests (CPTs) advanced to depths between 14 and 18 m below the existing ground level (bgl).
- Preparation of a geotechnical model for the Site.
- Analysis of the geotechnical investigation data using current liquefaction analysis methods to assess potential liquefaction during different levels of earthquake shaking along with liquefaction-related settlement and lateral spreading that may occur.

[1] New Zealand Geotechnical Database.

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- Identification of other potential geotechnical issues associated with the Site.
- Preliminary assessment of possible foundation options and associated geotechnical design parameters for typical sorts of MoE buildings.
- Preparation and issue of this letter report.

3 Proposed development

We are currently unaware of the form of the proposed development. However, the buildings are likely to comprise one- or two-storey lightweight structures, as currently built at other MoE schools around New Zealand.

4 Geotechnical information

4.1 Published geotechnical information

Published geological information[2] describes the Site as being underlain by predominantly Holocene Age[3] gravel, sand and silt river deposits.

4.2 Previous geotechnical investigations

Pattle Delemore Partners (PDP)[4] undertook investigations on the Site in 2009 as part of an environmental investigation. These comprised 89 boreholes drilled to a depth of approximately 2.7 m below ground level. A total of 72 logs from these shallow boreholes are presented in their report along with logs from drilling for the installation of two piezometers. These boreholes indicate that the Site is underlain by up to 2 m of fill (generally comprising silt with some clay, gravel and sand), with interbedded clay, silty clay, silt and sand deposits beneath this material. Sandy gravels and cobbles were also identified over the upper 0.6 m of the soil profile in parts of the Site. No information was sighted indicating how the fill material was selected, placed or what level of compaction was achieved. The locations of the boreholes used for the purposes of this assessment are shown on Figure A1 (Appendix A). This information has been used to supplement our recent geotechnical investigations and is accounted for in the ground model described in Section 4.4 below.

4.3 Current geotechnical investigations

Six CPTs were carried out on 16 and 17 August 2018 by LDE Ltd at the request of T+T. The CPTs met effective refusal conditions at depths between approximately 14.1 m and 19.1 m bgl. The locations of the investigations across the Site are shown on Figure A1 (refer Appendix A) and the logs from these tests are provided in Appendix B.

4.4 Generalised subsurface profile

The generalised subsurface profile inferred from the geotechnical investigations is provided in Tables 1 to 3 (below) and reflects the considerable variation across the Site from west to east. Figure A2 provides our interpretation of fill thickness across the Site. The nature and continuity of subsurface conditions away from the investigation locations is inferred and it must be appreciated that the actual conditions may vary from the assumed model. The geotechnical zoning is discussed in Section 5.3.

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[2] Mazengarb, C.; Speden, I.G. (compilers). (2000). Geology of the Raukumara area. Institute of Geological and Nuclear Sciences Ltd, New Zealand.

[3] The Holocene Age is the geological epoch beginning 11,700 years ago and continuing to the present day.

[4] Pattle Delamore Partners (16 November 2009) *Phase 2 Environmental Site Investigation, Former Mobil Gisborne Terminal (Site No. 410-280).* PDP Ref AJ22304R001.

Table 1: Generalised subsurface profile beneath the Western Zone

Soil layer no.	Typical depth to top of layer (m bgl)	Typical layer thickness (m)	Typical lithology
0	0	0.1 - 0.2	Topsoil
1	0.0 – 0.2	0 – 2.7	Fill –sandy silt and gravelly sand with some siltstone cobbles, bricks and brick fragments, rare shells, rusted steel pipewood pieces, rare timber pieces, rare concrete with rebar, glass fragments
2	0.2 - 1	0 – 3	Clayey to sandy silt, loose to medium dense
3	4.0 – 4.5	5.3 – 7.5	Clayey silt and silty clay, very soft to firm / medium dense
4	9.8 – 11.5	3.5 – 5.7	Silty sand to clayey silt, very soft to firm / medium dense to very dense
5	15 – 15.5	> 2.7	Sandy silt to clean sand, loose to very dense

Table 2: Generalised subsurface profile beneath the Central Zone

Soil layer no.	Typical depth to top of layer (m bgl)	Typical layer thickness (m)	Typical lithology
0	0	0.1 - 0.2	Topsoil
1	0.0 – 0.2	1.2 – 2.4	Fill - sandy silt and gravelly sand with siltstone cobbles, bricks and brick fragments, rare wire, some wood pieces rusted steel pipewood pieces, rare timber pieces, rare concrete pieces
2[a]	1.2	2.8	Clayey to sandy silt, loose to medium dense
3	2 - 4	5.8 – 9.0	Clayey silt and silty clay, very soft to firm / medium dense
4[b]	7.8	7.8	Silty sand to clayey silt, very soft to firm / medium dense to very dense
5	13 – 15.6	> 2.4	Sandy silt to clean sand, loose to very dense

a Not present at CPT03.

b Not present at CPT04.

Table 3: Generalised subsurface profile beneath the Eastern Zone

Soil layer no.	Typical depth to top of layer (m bgl)	Typical layer thickness (m)	Typical lithology
0	0	0.1 - 0.2	Topsoil
1	0.0 – 0.2	0 – 0.2	Fill - sandy silt and gravelly sand with some siltstone cobbles,
2[a]	0.2	1	Clayey to sandy silt, soft to firm; loose to medium dense
3	0.2 – 1.0	9.6 – 12.8	Clayey silt and silty clay, very soft to firm / medium dense
4	10.6 – 13	1.5 - 1.9	Silty sand to clayey silt, very soft to firm / medium dense to very dense
5	12.5 – 14.5	> 0.5	Sandy silt to clean sand, loose to very dense

a Not present at CPT06.

4.5 Groundwater

Groundwater information is not available on the NZGD[5] for this Site. The investigations by PDP in 2009 indicated that groundwater was encountered between the ground surface and approximately 1.2 m depth. The Site-specific investigations undertaken in August 2018 indicate groundwater at depths between 0.5 m and 2 m bgl. Shallower groundwater was observed in the south-eastern part of the Site. Artesian pressure was observed following the withdrawl of the CPT probe at the location of CPT05 (i.e. from a depth of 19 m bgl). The artesian water level was measured at approximately 0.4 m above ground level.

Based on this information, we have adopted an inferred groundwater depth of 1.0 m bgl across the Site for the purposes of our liquefaction analysis. We note that the depth to groundwater can have significant implications for the static design of shallow foundations and any excavations carried out on the Site and that it can be variable, especially after periods of prolonged rainfall or due to seasonal variation. Further discussion is provided in Section 6.

5 Liquefaction assessment

5.1 Seismic shaking hazard

Evaluation of the seismic performance of the Site (including liquefaction effects) is guided by the appropriate seismic shaking hazard and the requirements of the New Zealand Building Code, which considers the design earthquake scenarios derived from “NZS 1170 – Structural Design Actions” representing the following design performance requirements:

- Serviceability limit state 1 (SLS1) – the building should suffer little or no structural damage, and remain accessible and safe to occupy. There may be minor damage to building fabric that is readily repairable.
- Ultimate limit state (ULS) – the building is expected to suffer moderate to significant structural damage, but not to collapse.

The design earthquake scenarios are described in terms of an event moment magnitude (M_w) and Peak Horizontal Ground Acceleration (PGA_H) and were derived assuming a building design life of 50 years and an Importance Level (IL) of IL2, as set out in NZS 1170. The earthquake scenarios adopted for analysis are presented in Table 4 (below).

The MoE has published a structural and geotechnical guideline[6] (SGG) for the engineering associated with MoE buildings. Following the SGG we have also presented an SLS2 scenario for an event[7] with a return period of around 250 years. At around this shaking level the majority of the potentially liquefaction-susceptible soils within the investigated soil profile are expected to have triggered.

[5] New Zealand Geotechnical Database.

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[6] Ministry of Education (2016). *Designing Schools in New Zealand. Structural and Geotechnical Guidelines. Version 2.0, March 2016.* Endorsed by the Ministry of Business, Innovation and Employment.

[7] Serviceability limit state 2 (SLS2) – the building may suffer “*tolerable damage”* where the building may be used for its intended purpose but with reduced amenity.

Table 4: Design earthquake scenarios

	Design earthquake scenario		
	SLS1	**SLS2**	**ULS IL2**
Return period (years)	25	250	500
Moment magnitude (M_w)	6.0	6.25	6.25
Peak Horizontal Ground Acceleration (PGA_H)	0.21 g	0.64 g	0.85 g

5.2 Liquefaction analysis

The liquefaction triggering analyses have been carried out using the methodologies presented in Boulanger and Idriss (2014), with corresponding liquefaction-related consolidation settlements (S_{v1d}) calculated using the method of Zhang et al (2002)[8]. Liquefaction severity number (LSN)[9] has also been used as a guide in order to assess the potential liquefaction-induced land damage.

The results of the liquefaction triggering analyses indicate that:

- Surface effects of land damage when described in terms of NZGS guidelines are expected to be relatively mild in a SLS1-level event over the eastern part of the Site (S_{v1d} values of less than 30 mm and LSN approximately 4).
- In the central and western areas the surface effects are expected to be mild to moderate (S_{v1d} values of the order of 60 to 90 mm and LSN 9 to 13). Differential foundation settlements may also occur across individual buildings and these may be in the order of 40 mm over a typical building width of 10 – 12 m. This assessment is informed by a combination of the calculated settlement values and our engineering judgement.
- The majority of liquefaction triggering (and related damage) occurs at a return period of approximately 100 – 250 years. Hence, the results from SLS2 and ULS-level events are not markedly different from one another. This opinion also applies whether buildings are considered to be IL2 or IL3, since the level of shaking that triggers a significant extent of liquefaction within the investigated soil profile is less than the ULS PGA_H.
- Under ULS-level shaking (IL2) the following observations are made:
  - Liquefaction-related settlements are estimated to be approximately 15 to 45 mm, 80 to 90 mm, and 120 mm to 140 mm in the eastern, central and western areas of the Site, respectively.
  - The expected liquefaction-related ground damage is expected to be moderate. Based on the calculated settlement values, the depth where liquefaction is expected to trigger and our engineering judgment, in our opinion differential foundation settlements may be in the order of 40 – 50 mm over a typical building width of 10 – 12 m.

We note that the settlement values stated above correspond to the settlements calculated over the entire length of the CPTs (approximately 15 m), and that the settlements that occur in the upper 10 m of the soil profile typically vary between 10% and 30% of these. Experience from the Canterbury earthquake sequence (CES), which is relevant to this Site, suggests that liquefaction-related

[8] Zhang, Robertson and Brachman (2002). *Estimating liquefaction-induced ground settlements from CPT for level ground.* Canadian Geotechnical Journal.
[9] Cubrinovski et al (2016) Earthquake geotechnical engineering practice, Module 3: Identification, assessment and mitigation of liquefaction hazards, New Zealand Geotechnical Society and Ministry of Business, innovation and Employment.

settlements below 10 m depth are likely to be minor and have a much reduced effect at the ground surface (in terms of expected ground damage or potential differential building settlements).

We also note that the future site investigations would enable the liquefaction analysis and foundation design implications to be refined during further stages of development. However, we do not expect the findings to materially affect our recommendations, rather they will be used to optimise geotechnical design details.

Liquefaction-related lateral soil displacement is not expected to occur at the Site in future earthquake events as the majority of the liquefiable material is present at significant depth.

5.3 Geotechnical "zones" and associated building foundation recommendations

For the purposes of this geotechnical assessment, we believe that the Site can be considered to consist of three geotechnical "zones" – namely an Eastern, Central and Western Zone. These have been developed based on the variability of the fill deposits and the liquefaction susceptibility of the underlying materials. The three zones are presented on Figure A3.

6 Geotechnical implications for Site development

6.1 General

The recommendations and opinions that are contained in this report are based upon data from geotechnical investigations on the Site and surrounding areas. The nature and continuity of subsurface conditions away from the investigation locations is inferred, and it must be appreciated that the actual conditions may vary from the assumed geotechnical model.

All of the recommendations and interpretations that are presented in this report are preliminary in nature and must be reviewed and confirmed as part of the future design process for any development works. This confirmation is likely to require further location-specific geotechnical investigations, depending on the final chosen nature and locations of specific buildings.

Final selection of the building foundation systems should be made in collaboration with the Structural Engineer, the Geotechnical Engineer, the Client, and the MoE once more detail of the actual building locations, configurations and foundation loads is available. This should ensure that a more complete consideration of seismic performance expectations, financial issues, and constructability issues is undertaken.

6.2 Key geotechnical considerations

The main geotechnical considerations for design and construction of the proposed development include:

- The presence of fill on the Site.

  This has two main potential considerations from a geotechnical perspective:

  - Structural competence/physical composition (i.e. density/adequate bearing). There is likely to be some degree of variability across the Site (both with depth and spatially) given the demolition of the previous structures on the property and within the surrounding areas (refer Figure A2).
  - Cost implications with regards to health and safety practices, soil management requirements including disposal, and construction-related resource consents, which is discussed in the separate ground contamination-related report for the Site.
- The liquefaction potential of the underlying silt and sand deposits, mainly in the south-western part of the Site.

- Static settlements due to consolidation of the existing fill and the relatively soft underlying clay/silt deposits.
- Parts of the Site will have been previously loaded by tank structures, there is a potential for differential settlement to occur on Site. Differential settlement will have to be considered especially for any new buildings that will straddle the site of a former structure.
- The relatively shallow depth to groundwater is a consideration for foundation design and construction (and trenching for services). Dewatering may be required depending on actual excavation depths, especially if undercutting of unsuitable material beneath foundations (i.e. uncontrolled fill, rubbish or organic material) is required in order to achieve an adequate foundation subgrade.

These issues are discussed below in conjunction with the key geotechnical aspects of the development.

6.3 General development considerations

In general, observations made throughout Christchurch during the CES indicated that buildings that were clad with light-weight wall and roof materials performed better under earthquake loading than those which were clad with heavy-weight materials. Therefore, we recommend that new structures proposed for the Site are also constructed from light-weight materials. If heavy-weight materials are to be used then it should be restricted to single-storey structures and the bottom level of multi-storey buildings. Alternatively, if heavy-weight cladding materials are used on all levels and/or buildings higher than two- or three-storeys, then more robust foundation works, possibly in conjunction with some form of shallow ground improvement, are likely to be required so that satisfactory seismic performance is achieved.

Buildings that have a regular or symmetrical footprint have also been observed to perform better during the CES i.e. less damage and generally easier to repair. Therefore, we recommend that regular building shapes be adopted for the proposed development.

Mixed foundation systems within the same structure are not recommended, e.g. suspended timber floor with slab on grade, unless appropriate allowance is made for differential movement under strong earthquake shaking.

Generally, the recommendations for shallow foundation systems as set out in the MBIE Guidance for residential-type buildings are considered suitable for new single- or two-storey structures on the Site if such structures are constructed using light- to moderate-weight materials with similar structural stiffness. Some classroom-type structures that are built using light- to moderate-weight materials can be considered similar to residential buildings with regards to likely performance characteristics.

Generally it is not recommended that buildings are founded within non-engineered fill. However, in this case, the CPT information indicates that the existing fill material could have sufficient strength to support shallow foundations. This may also be of particular relevance where readily re-levellable foundations are used. The variability of the fill properties beneath a particular building footprint is a key consideration and further shallow investigations would be required to confirm the viability of founding within the non-engineered fill.

We note that handling and compaction of silty and clayey soils can be problematic, especially when they become saturated. Where such silty soils are encountered during any site earthworks then due consideration should be given to limiting the extent of area potentially exposed to rainfall and/or inundation. We also recommend that adequate surface drainage is installed to prevent the ponding of surface water on top of prepared subgrade areas. Management of sediment-laden stormwater will also need careful consideration.

6.4 Site subsoil class

In terms of NZS 1170.5[10] the site subsoil class is likely to be Class C (shallow soil). This recommendation is based on published geological information[11] indicating that sandstone bedrock is present beneath the nearby Titirangi Domain, and the thickness of silt, clay and sand deposits beneath the site. The presence of the hill (bedrock deposit) to the south of the Site may also mean that wave amplification effects are felt at the Site during an earthquake event.

If the site is purchased, we recommend that the structural and geotechnical engineers discuss the implications of the selected subsoil classification on the structural design. A deep cored borehole may be required to determine the depth to rock and verify the appropriate subsoil classification.

6.5 Foundation options

The underlying silts and sands are susceptible to liquefaction under future moderate to strong earthquake shaking, however the magnitude of potential settlement associated with this is less over the eastern portion of the Site. While the boundaries between geotechnical zones cannot be precisely defined due to the limited data available, we would recommend that structures are positioned wholly within each of the zones to more readily provide for consistent building foundation conditions beneath individual structures. At this stage we would also consider it preferable to construct any buildings within the eastern zone where the geotechnical constraints are less.

We recommend that structures could be founded within the near surface crust. Foundation options that are likely to be viable include:

- Specifically designed Type A or B residential-type footings for lightly loaded timber-framed structures that are readily re-levellable.
- Relatively stiff reinforced concrete slab-type foundations, such as a specifically designed waffle slab, potentially in conjunction with a raft of engineered fill. The engineered fill raft may be constructed above the existing ground level to enhance the surface crust, however consideration would need to be given to potential consolidation of the underlying soft silt/clay soil. This sort of foundation system could also help to reduce the potential adverse impacts of the shallow groundwater.

Larger-scale ground improvement is not currently believed to be warranted to support shallow foundations for this development. However, some localised ground replacement-type measures may be beneficial to optimise some shallow foundation elements. For example, over-excavation of soft or loose silts or non-engineered fill and replacement with compacted hardfill, or similar, may enable footings sizes to be reduced and the potential adverse impacts of any near-surface liquefaction to be better managed. This is particularly relevant to more heavily loaded pad footings or individual ground beams. Such an approach would be in line with the intent behind the SGG and may warrant further exploration in conjunction with future geotechnical investigations.

We have undertaken a high level assessment to estimate the static settlement of a one to two storey structure on the Site. For this assessment, we assumed that the building had a rectangular footprint of 30 m by 60 m and a uniform load of 10 to 20 kPa was applied over the footprint. Our analyses indicated that static settlement beneath the centre of the raft foundation may be in the order of 50 to 150 mm. Preloading (i.e. placement of fill to surcharge the Site at the location of the proposed development) may be a viable method to reduce the effects of static settlement. Further

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[10] Standards New Zealand (2004) – *NZS 1170.5:2004 – Structural Design Actions Part 5: Earthquake Actions – New Zealand.*
[11] Mazengarb, C.; Speden, I.G. (compilers). (2000). Geology of the Raukumara area. Institute of Geological and Nuclear Sciences Ltd, New Zealand.

information on likely building location, size and foundation loading at the Site would be needed to inform any design of such preloading.

Due to the potential magnitude of static settlement and the high probability that liquefaction-induced settlement occurs following design level earthquakes, flexible service connections at the building envelope should be used. These will reduce the potential for damage to, or loss of, services which result from strong shaking and vibrations associated with an earthquake and other post-construction settlements. Connections should be located and detailed to facilitate ease of repair if required in future.

Consideration could also be given to designing the foundations to sustain a 4 m loss of internal ground support and 2 m loss of support at the edges/corners, depending on the level of resilience (or robustness) that is desired under future potential earthquake shaking.

Piled foundations are also an option, particularly for more heavily loaded structures. Driven piles are expected to be the most cost-effective and would need to be founded within suitable material below any liquefaction-susceptible soils, which could be of the order of 18 to 20 m bgl, or more. Further machine-drilled boreholes would be required to confirm likely pile founding materials and depths. Any pile design would need to take into consideration negative skin friction due to liquefaction of soils along the pile shaft. It may also be necessary to design any floor slabs for piled structures to be fully self-supporting. The design of piled foundations on liquefiable soils also needs to carefully consider allowances for post-event inspection, assessment and repair. For these reasons we expect that shallow foundations in conjunction with relatively light-weight structures of uniform shape located within the eastern part of the Site would be more cost-effective, in the long term.

6.6 Further investigations

If the Site is selected for purchase and development, and once specific building locations and loads are known, then further location-specific ground investigations will be necessary to confirm geotechnical foundation design parameters and depth to groundwater. Further relatively deep machine-drilled boreholes would be required if pile foundations are contemplated.

Additional shallow investigations, comprising hand-augered boreholes and Scala Penetrometer tests would be required at the location of any proposed building to confirm whether footings for specific structures could be designed and founded within existing non-engineered fill. Other geotechnical-related work that we believe should be considered if the project proceeds include:

- Inclusion of geotechnical input at the master planning stage of any development, involving discussions with parties such as the architect and structural engineer to collaboratively consider the various constraints and opportunities presented by the geotechnical conditions at this Site.
- Geotechnical review of any structural foundation design. This may involve further discussion/collaboration with the structural engineer to optimise any proposed building foundations.
- Development of a project-specific earthworks specification covering geotechnical aspects of the construction works.
- Suitable oversight of relevant construction works by a geotechnical engineer, which we would expect would also enable a construction producer statement (PS4) to be issued for these works.

7 Applicability

This report has been prepared for the exclusive use of our client Ministry of Education, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

All of the recommendations and interpretations presented in this report are preliminary in nature and must be reviewed as part of the future design process for any development works.

The liquefaction susceptibility analyses carried out represent probabilistic analyses of empirical liquefaction databases under various earthquakes. Earthquakes are unique and impose different levels of shaking in different directions on different sites. The results of the liquefaction susceptibility analyses and the estimates of consequences presented within this document are based on regional seismic demand and published analysis methods, but it is important to understand that the actual performance may vary from that calculated.

Recommendations and opinions presented in this report are based on a limited number of discrete data points. The nature and continuity of the subsoil away from the sample locations is inferred and it must be appreciated that the actual conditions could vary from the assumed model.

Final selection of the building foundation system should be made in collaboration with the project design team, once more detail of the building configurations and foundation loads are available.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
Kirsti Murahidy Geotechnical Engineer CM EngNZ, CPEng, Int PE (NZ)	Gordon Ashby Project Director

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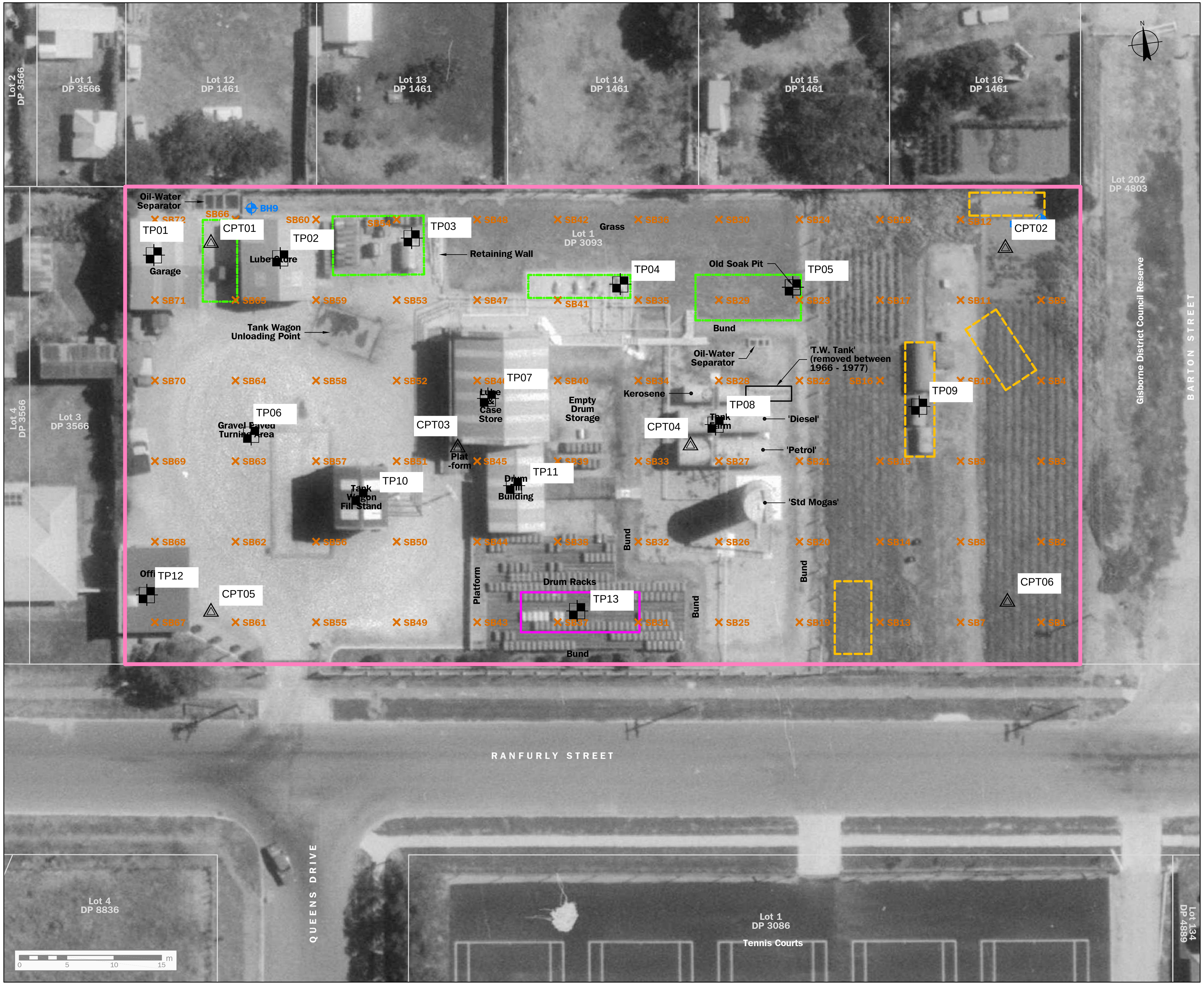
Appendix A: Figures



Legend

- SB1 — Soil bore (7-16 July 2009)
- BH9 — Monitoring well (installed July 2009)
- Empty AST storage, historical
- Former location of original AST c.1942
- Historical drum storage
- Site boundary

T+T investigation locations

- TP01 — Test pit (August 2018)
- CPT01 — Cone Penetrometer Test (August 2018)

Figure A1. TKKM o Horouta Wananga, 17 Ranfurly Street, Gisborne

Site Investigation Plan

T+T Ref:1007466

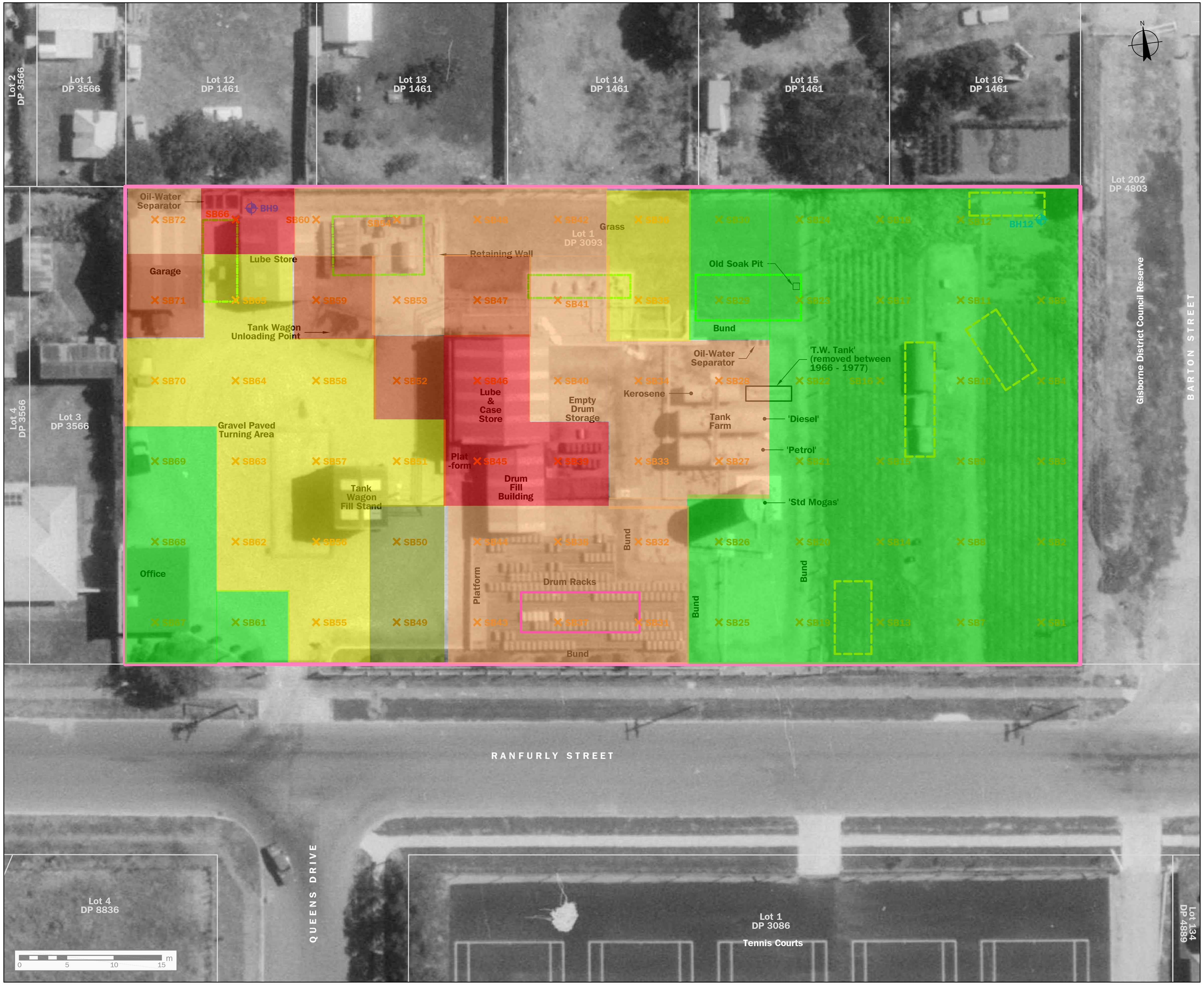
KCC 19/09/2018

Base drawing from Pattle Delamore Partners (2009)

Notes:
1) Aerial photography supplied by NZ Aerial Mapping Ltd. (may not be spatially accurate).
Date of photography 28 Jan 1986.
2) Cadastral information derived from LINZ data.

Scale 1:400 (A3)

Figure 2 : HISTORICAL SITE LAYOUT (1986) & SAMPLE LOCATION PLAN



Legend

- SB1 — Soil bore (7-16 July 2009)
- BH9 — Monitoring well (installed July 2009)
- Empty AST storage, historical
- Former location of original AST c.1942
- Historical drum storage
- Site boundary

Fill depth

- 0 - 0.19 m
- 0.2 - 0.49m
- 0.5 - 0.99 m
- 1.0 - 1.49 m
- 1.5 - 1.99 m
- : >2m

Figure A2. TKKM o Horouta Wananga, 17 Ranfurly Street, Gisborne

Fill Depth Plan

T+T Ref:1007466

KCC 19/09/2018

Base drawing from Pattle Delamore Partners (2009)

Notes:
1) Aerial photography supplied by NZ Aerial Mapping Ltd. (may not be spatially accurate).
Date of photography 28 Jan 1986.
2) Cadastral information derived from LINZ data.

Scale 1:400 (A3)

Figure 2 : HISTORICAL SITE LAYOUT (1986) & SAMPLE LOCATION PLAN

AJ22304D001.dwg [AJ22302I002.jpg] Sep-09 Final

Legend

- SB1 — Soil bore (7-16 July 2009)
- BH9 — Monitoring well (installed July 2009)
- Empty AST storage, historical
- Former location of original AST c.1942
- Historical drum storage
- Site boundary

Approximate fill depth

- 0 - 0.19 m
- 0.2 - 0.49m
- 0.5 - 0.99 m
- 1.0 - 1.49 m
- 1.5 - 1.99 m
- >2m

T+T investigation locations

- CPT01 Cone Penetrometer Test (August 2018)

Figure A3. TKKM o Horouta Wananga, 17 Ranfurly Street, Gisborne

Geotechnical Zoning

T+T Ref:1007466

KCC 19/09/2018

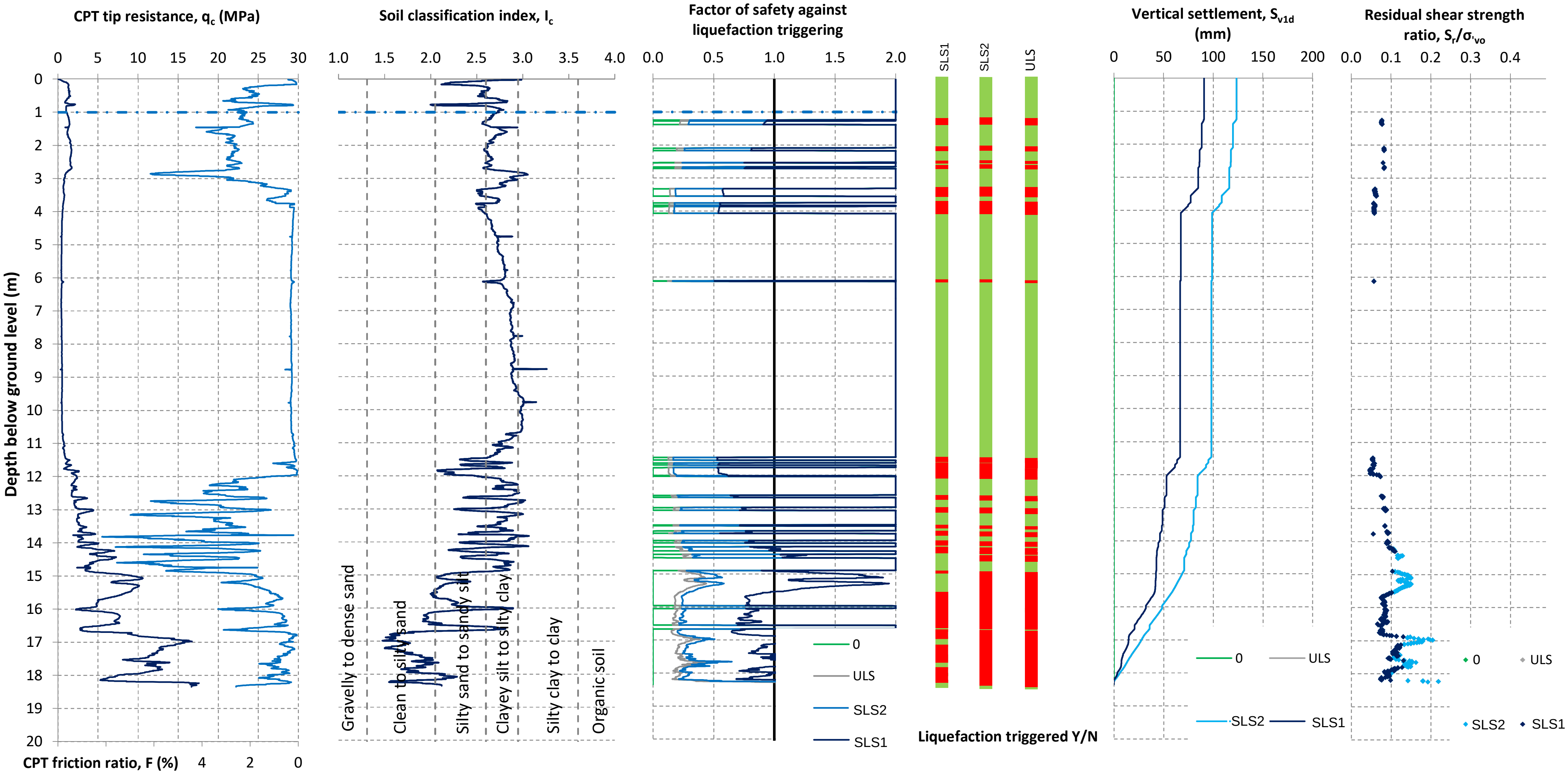
Base drawing from Pattle Delamore Partners (2009)

Notes:
1) Aerial photography supplied by NZ Aerial Mapping Ltd. (may not be spatially accurate).
Date of photography 28 Jan 1986.
2) Cadastral information derived from LINZ data.

Figure 2 : HISTORICAL SITE LAYOUT (1986) & SAMPLE LOCATION PLAN

Scale 1:400 (A3)

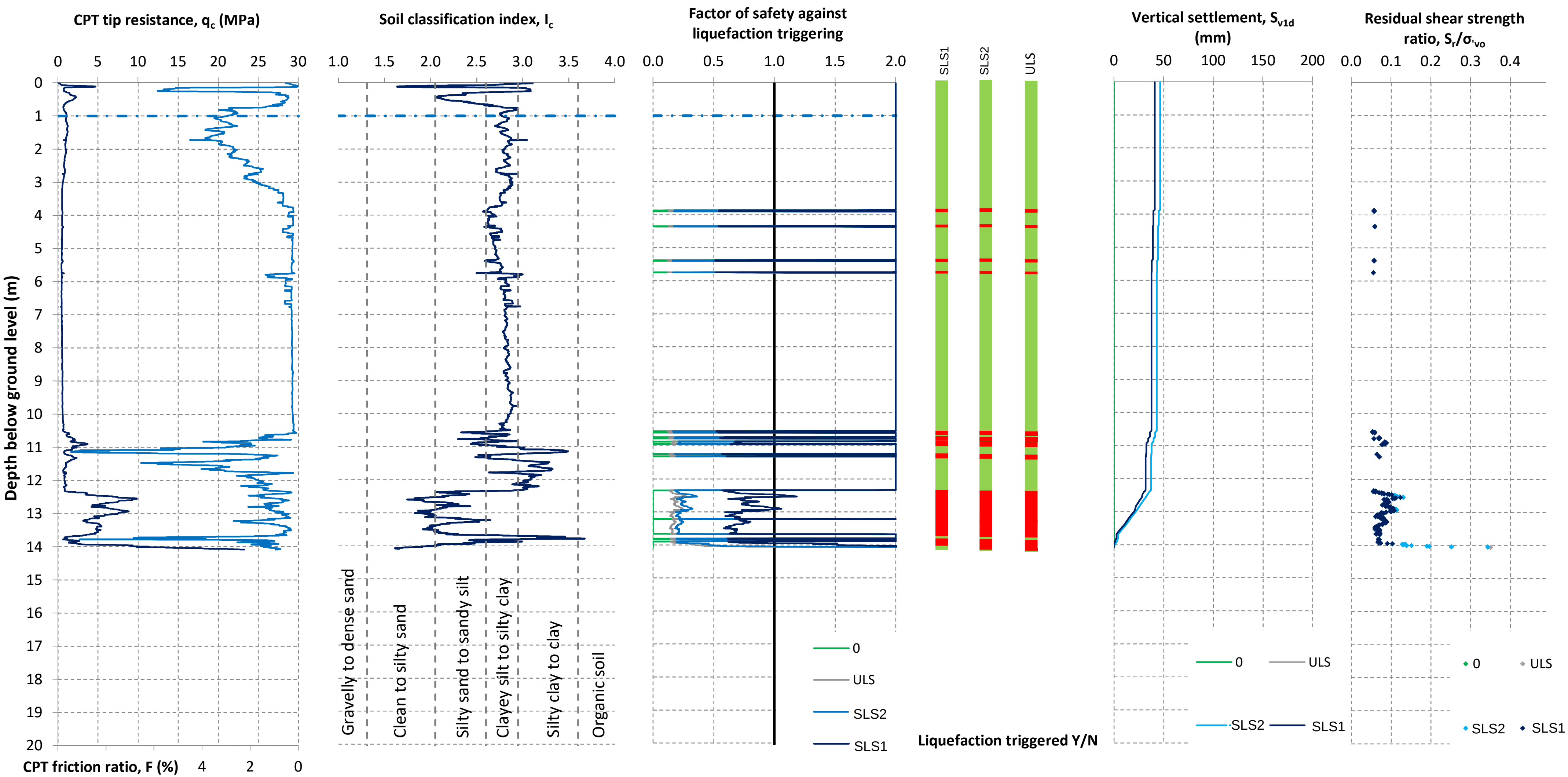
Appendix B: Liquefaction analysis outputs



INPUT	CPT Name	Investigation Date	GWD (m)	P_L (%)	Trigger Method	Settlement Method	Res. Strength Method	Pre-drill Depth (m)	γ (kN/m^3)	Surcharge load (kPa)
	CPT01	8/16/2018	1.0	15	B & I (2014)	ZRB (2002)	I & B (2007)	0	18	0

OUTPUT	Event	PGA (g)	Magnitude, M_w	S_{v1d} (mm)	LSN	CTL (m)
	SLS1	0.21	6.0	91	13	3.6
	SLS2	0.64	6.25	123	16	5.0
	ULS	0.85	6.25	124	16	5.0

Tonkin+Taylor	**Client:** Ministry of Education **Project:** 17 Ranfurly Street **Description:** Liquefaction Analysis of CPT Data	**Location:** Ranfurly St, Gisborne **Job Number:** 1007466 **Date:** 19/09/2018	**Version:** 1.4 **Analysed by:** PEKI **Checked by:** KCC



INPUT	CPT Name	Investigation Date	GWD (m)	P_L (%)	Trigger Method	Settlement Method	Res. Strength Method	Pre-drill Depth (m)	γ (kN/m^3)	Surcharge load (kPa)
	CPT02	8/17/2018	1.0	15	B & I (2014)	ZRB (2002)	I & B (2007)	0	18	0

OUTPUT	Event	PGA (g)	Magnitude, M_w	S_{v1d} (mm)	LSN	CTL (m)
	SLS1	0.21	6.0	41	4	1.6
	SLS2	0.64	6.25	47	4	1.8
	ULS	0.85	6.25	47	4	1.8

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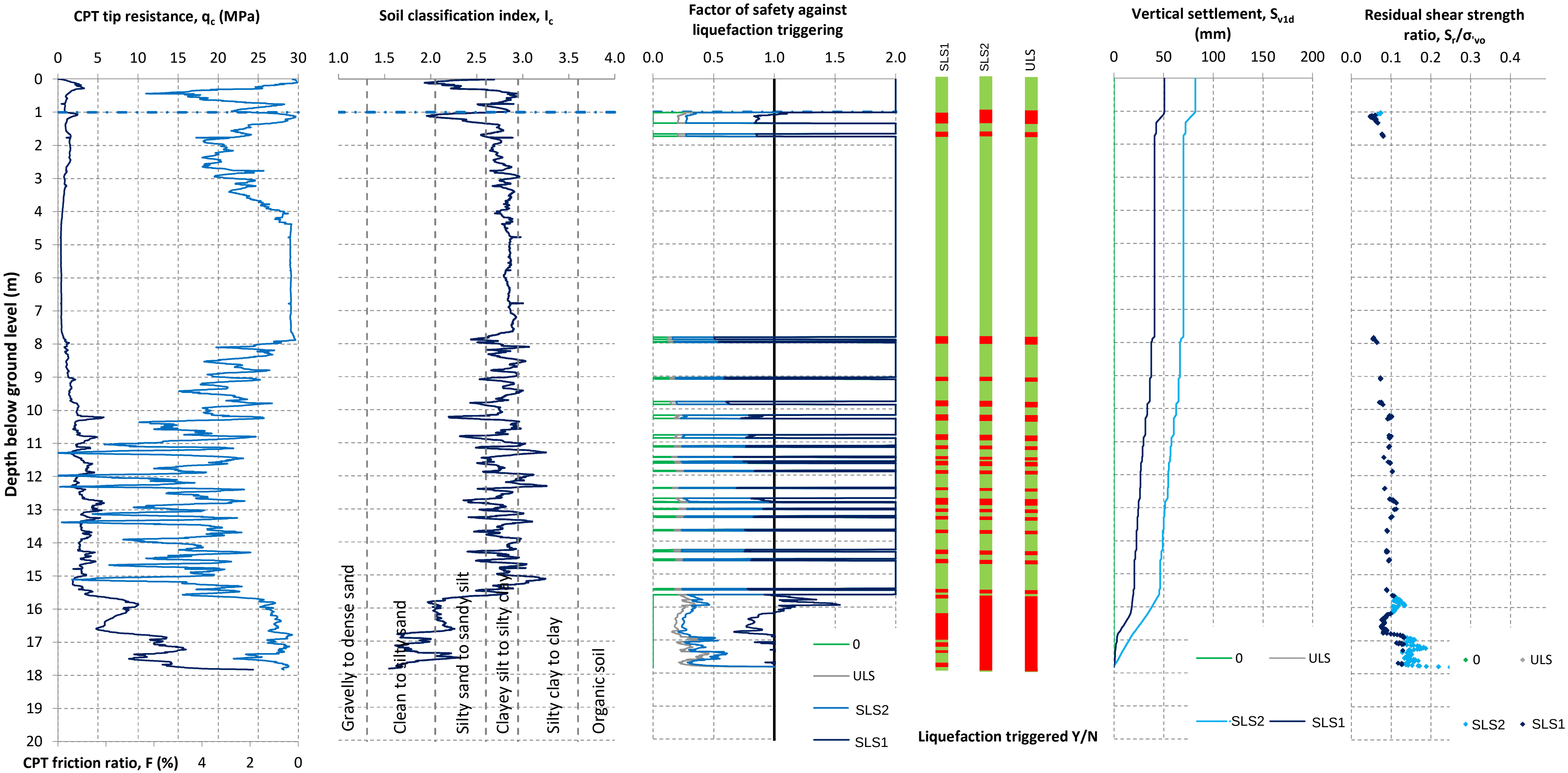

INPUT	CPT Name	Investigation Date	GWD (m)	P_L (%)	Trigger Method	Settlement Method	Res. Strength Method	Pre-drill Depth (m)	γ (kN/m^3)	Surcharge load (kPa)
	CPT03	8/17/2018	1.0	15	B & I (2014)	ZRB (2002)	I & B (2007)	0	18	0

OUTPUT	Event	PGA (g)	Magnitude, M_w	S_{v1d} (mm)	LSN	CTL (m)
	SLS1	0.21	6.0	51	11	2.2
	SLS2	0.64	6.25	82	15	3.6
	ULS	0.85	6.25	82	15	3.6

Tonkin+Taylor

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Location: Ranfurly St, Gisborne
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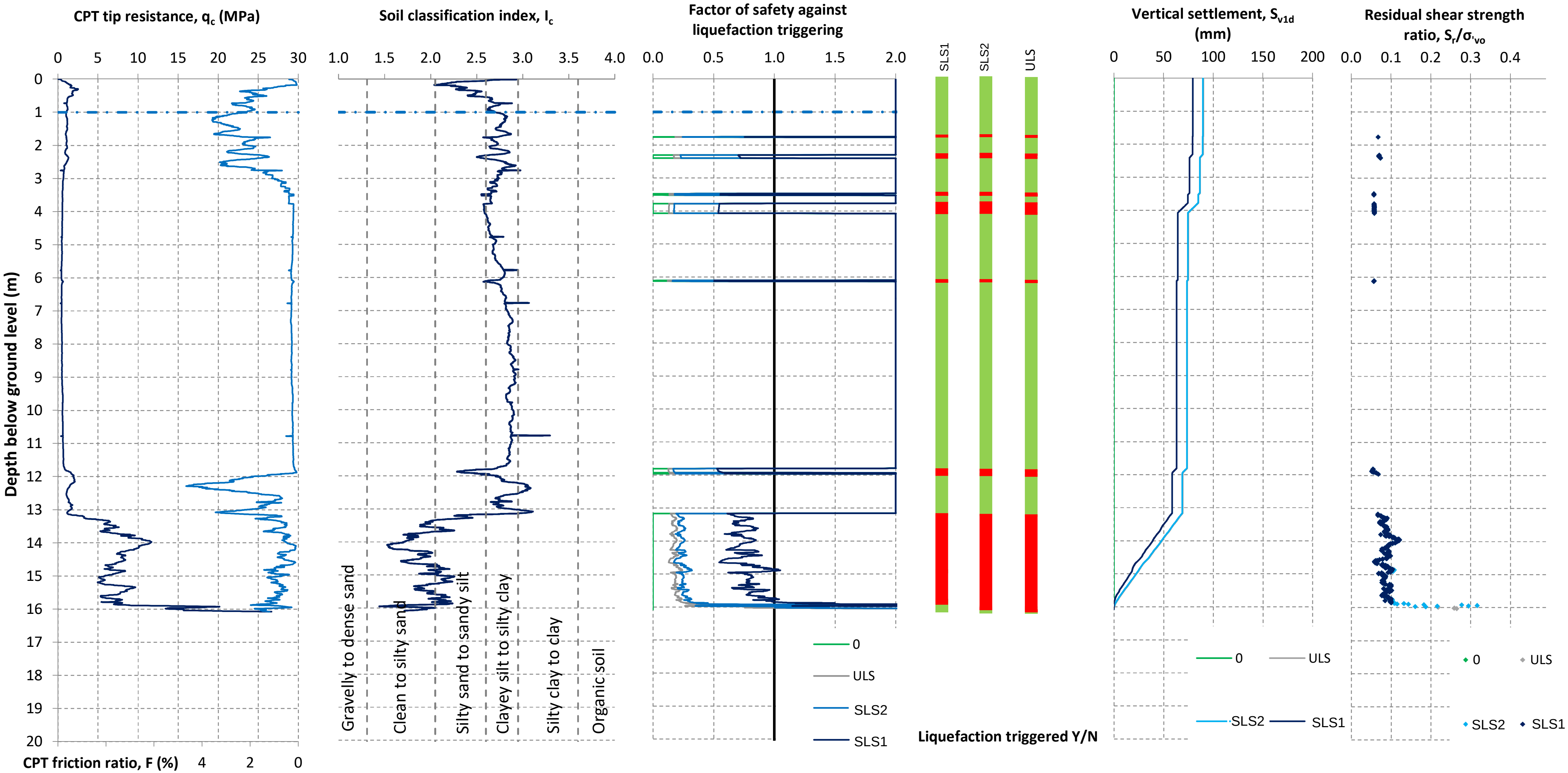
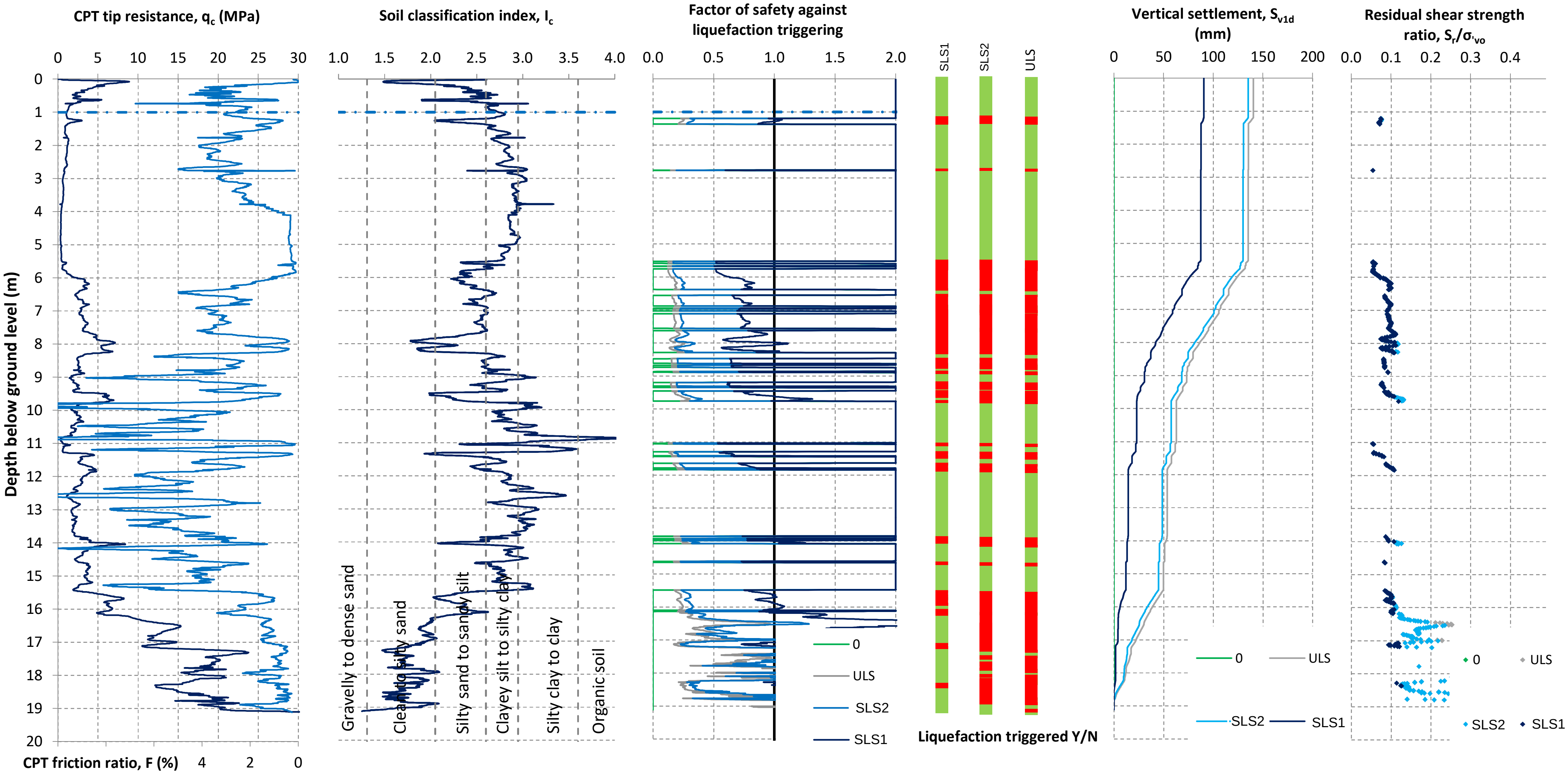


INPUT	CPT Name	Investigation Date	GWD (m)	P_L (%)	Trigger Method	Settlement Method	Res. Strength Method	Pre-drill Depth (m)	γ (kN/m^3)	Surcharge load (kPa)
	CPT04	8/17/2018	1.0	15	B & I (2014)	ZRB (2002)	I & B (2007)	0	18	0

OUTPUT	Event	PGA (g)	Magnitude, M_w	S_{v1d} (mm)	LSN	CTL (m)
	SLS1	0.21	6.0	79	9	3.3
	SLS2	0.64	6.25	90	10	3.4
	ULS	0.85	6.25	90	10	3.5

Tonkin+Taylor	**Client:** Ministry of Education	**Location:** Ranfurly St, Gisborne	**Version:** 1.4
	Project: 17 Ranfurly Street	**Job Number:** 1007466	**Analysed by:** PEKI
	Description: Liquefaction Analysis of CPT Data	**Date:** 19/09/2018	**Checked by:** KCC



INPUT	CPT Name	Investigation Date	GWD (m)	P_L (%)	Trigger Method	Settlement Method	Res. Strength Method	Pre-drill Depth (m)	γ (kN/m^3)	Surcharge load (kPa)
	CPT05	8/16/2018	1.0	15	B & I (2014)	ZRB (2002)	I & B (2007)	0	18	0

OUTPUT	Event	PGA (g)	Magnitude, M_w	S_{v1d} (mm)	LSN	CTL (m)
	SLS1	0.21	6.0	91	13	3.9
	SLS2	0.64	6.25	135	18	6.1
	ULS	0.85	6.25	140	18	6.5

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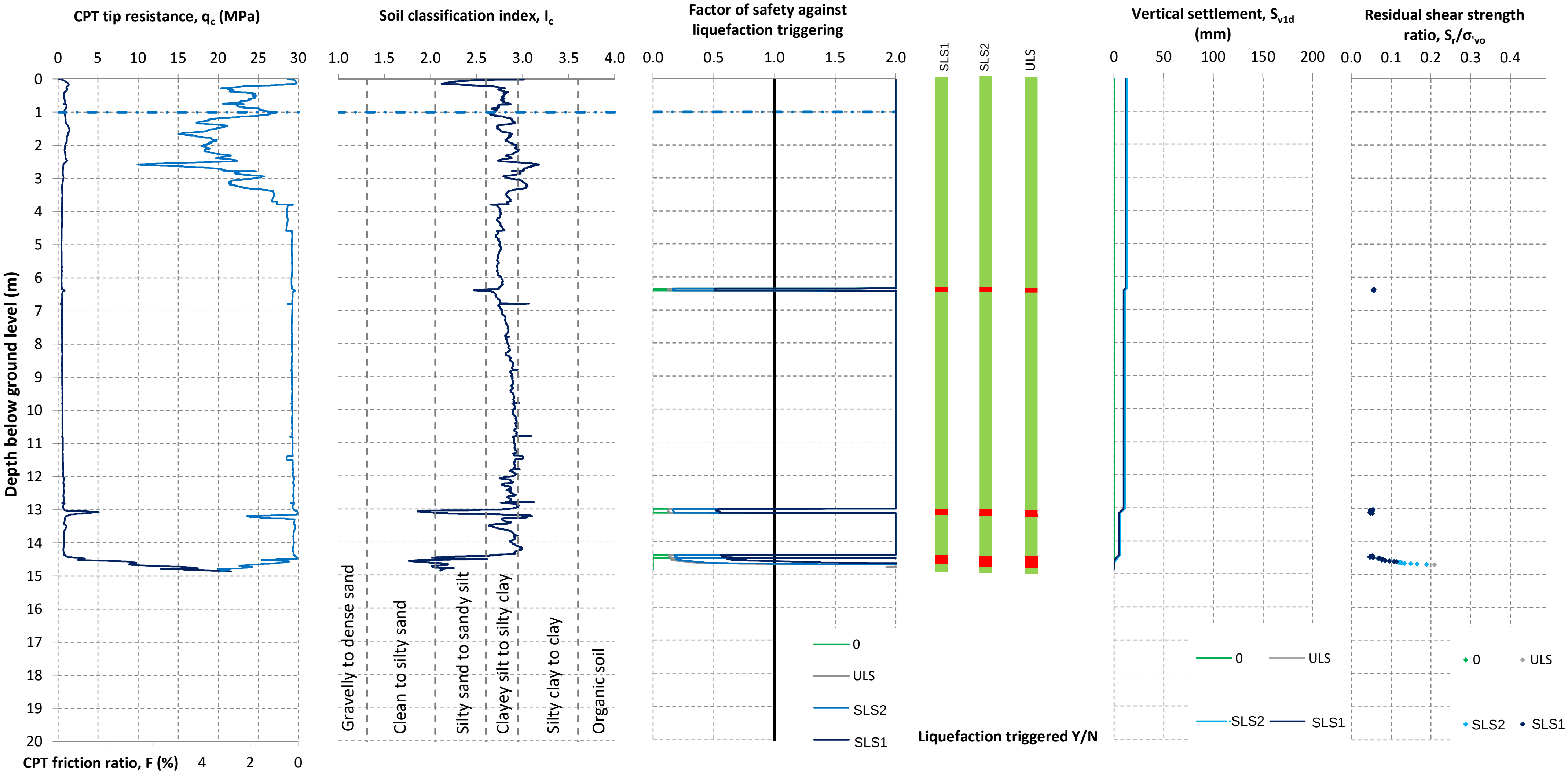

INPUT	CPT Name	Investigation Date	GWD (m)	P_L (%)	Trigger Method	Settlement Method	Res. Strength Method	Pre-drill Depth (m)	γ (kN/m^3)	Surcharge load (kPa)
	CPT06	8/17/2018	1.0	15	B & I (2014)	ZRB (2002)	I & B (2007)	0	18	0

OUTPUT	Event	PGA (g)	Magnitude, M_w	S_{v1d} (mm)	LSN	CTL (m)
	SLS1	0.21	6.0	12	1	0.4
	SLS2	0.64	6.25	13	1	0.5
	ULS	0.85	6.25	13	1	0.5

Tonkin+Taylor	**Client:** Ministry of Education **Project:** 17 Ranfurly Street **Description:** Liquefaction Analysis of CPT Data	**Location:** Ranfurly St, Gisborne **Job Number:** 1007466 **Date:** 19/09/2018	**Version:** 1.4 **Analysed by:** PEKI **Checked by:** KCC