PRELIMINARY DESIGN GEOTECHNICAL ASSESSMENT REPORT

FOR PROPOSED MANGAPAPA SCHOOL RE-DEVELOPMENT

5 RUA STREET, MANGAPAPA, GISBORNE

Project Reference: 15344
26 July 2019

EXECUTIVE SUMMARY

Based on the investigation and appraisal of the site reported herein, the proposed administration block and classroom block for the re-development at Mangapapa School at 5 Rua Street, Mangapapa, Gisborne has been assessed as stable and is generally considered suitable for modified conventional construction in accordance with the relevant codes of practice, providing that the recommendations of this report are adhered to. This report has been prepared in support of the Preliminary Design phase of the project, and is expected to be suitable for future resource and building consent applications.

'Good Ground' with a geotechnical ultimate bearing capacity of at least 300kPa was not consistently encountered at a shallow depth beneath the site. Based on the test information a geotechnical ultimate bearing capacity of 200kPa is only available from between 0.7m to 2.1m depth. This increases to 300kPa from generally below 2.0m depth. The foundation designs given in NZS3604 (2011) will need to be modified to address this issue.

The detailed liquefaction and sensitivity analyses indicated that the site has an overall low to moderate susceptibility to liquefaction, however with potential risk for excessive lateral spreading to occur. This conclusion is supported by the theoretically low liquefaction potential susceptibility (LPI & LSN) values both under SLS, MLS, ULS IL2 and IL3 seismic conditions.

The slope stability modelling indicates that under static conditions the slope has a reasonable level of stability, however is likely to trend towards potential instability of the slope in extreme event situations. In order to mitigate the both the slope instability hazard and lateral spreading risk, we recommend that a palisade wall be constructed behind the segmental retaining wall. In addition, the wall will eliminate the requirement for TC3 foundations to be considered.

The liquefaction analysis indicates that TC2 type ground performance is expected in a IL2 ULS seismic event (0.32g) and as per the MBIE Guidelines[1] either SED timber floor foundations or enhanced Concrete Raft/Waffle Slab foundations are required (Options 1 to 4 from Part A, Section 5.3.1 of the guidance document). Alternatively, construction of standard waffle slabs on a reinforced gravel raft are suitable for this site.

All other geotechnical hazards at the site have been assessed as either not present or of acceptable risk provided that the various mitigation measures, inspection and certification requirements, and good practice recommendations made in this report are adopted. This executive summary must not be taken out of context with the balance of this report. Additional recommendations and considerations are made in the body of the report which provide the context for the above key findings relating to the development under consideration.

[1] MBIE 2015. Repairing and rebuilding houses affected by the Canterbury earthquakes.

CONTENTS

1 INTRODUCTION

LDE Ltd (Land Development & Engineering) was engaged by DCA Architects Ltd C/- Ministry of Education (MOE) to undertake a Preliminary Design geotechnical assessment and report (PDGAR) for the proposed re-development of Mangapapa School, located at 5 Rua Street, Mangapapa, Gisborne (Figure 1).

The site is legally defined as comprising Lot 1, Pt Lot 2 and 3 DP 1813 and Lots 9 and 10 DP 539 to the north (numbered as 2 Rua Street) and Lot 1 DP 2603 and Lot 21 DP 539 to the south (5 Rua Street), with a total site area of some 2.1ha. The main school precinct is located to the south of Rua Street and is the principal subject of this assessment (Figure 2).

Figure 1: Site location relative to Gisborne City and surrounding communities (**Source:** Google Maps[2]).

The proposal involves the demolition and replacement of several classrooms, the administration building, the redevelopment of carparking on the southern side of the site, and the construction of a new parking area and drop-off zone on the northern side. The proposed re-development plans are illustrated in Figures 3 and 4 and are also appended to this report in full. In all, three classrooms and the administration building are to be removed with two large 565m^2 to 800m^2 teaching spaces proposed to be constructed in addition to a 300m^2 new administration block and a 36m^2 caretakers building.

The objectives of this preliminary geotechnical assessment were to use all the existing geotechnical investigation data in addition to newly acquired data determine the nature, strength and variability beneath the subject site, assess and quantify the liquefaction potential

[2] Google Maps (www.google.co.nz/maps)

and lateral spreading risk to the proposed development using best practice analysis methodology, assess the current stability of the riverbank slope in relation to the position of the proposed classrooms and make recommendations relative to the proposed site development and building foundation systems. This report has been prepared in support of the intended building consent application and any associated resource consent application to Gisborne District Council (GDC) following the Developed Design and Detailed Design reviews by LDE Ltd.

Note that although this report has been prepared for the MOE Preliminary Design Phase assessment, it is also expected to be satisfactory for the subsequent Developed Design and Detailed Design phases of the project without additional site investigations, unless significant changes to the locations and layout of the proposed buildings occurs.

Figure 2: Site location relative to Rua and Perry Streets and Ormond Road. Site boundaries shown in blue (**Source:** Tairawhiti Maps[3])

[3] Tairawhiti Maps (https://maps.gdc.govt.nz/H5V2_10/)

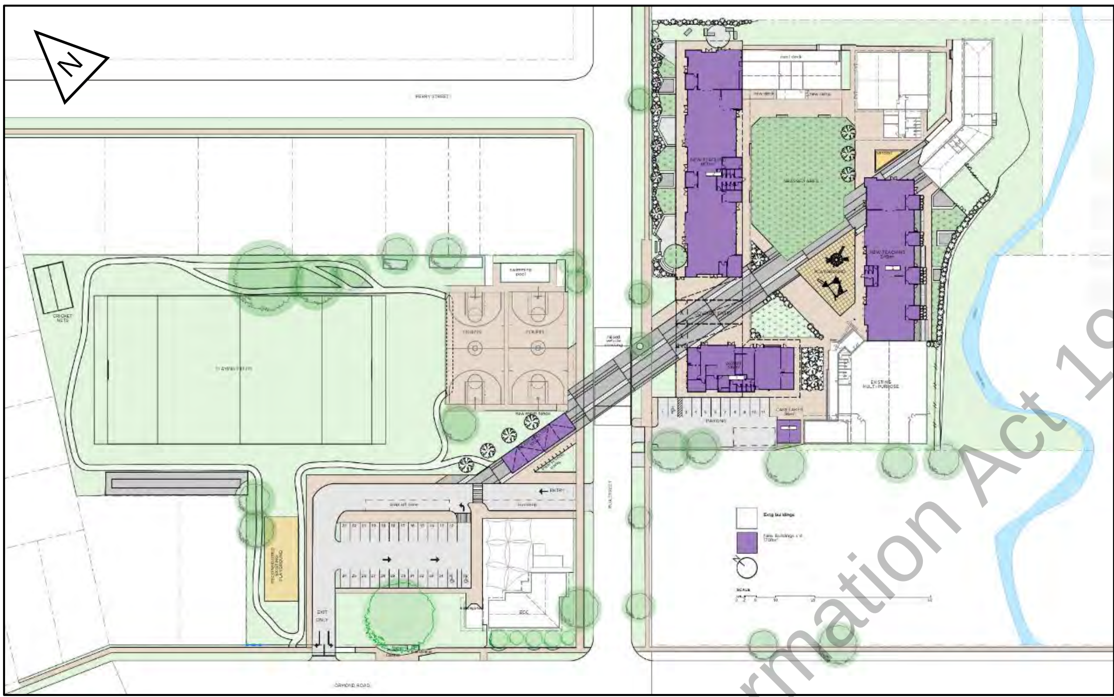

Figure 3: Site Re-Development Plan for entire school site (**Source:** DCA Ltd, Mangapapa School | Preliminary Design | Proposed Site Plan, V0604, Dated: 27/05/2019). New structures shown as purple shaded areas.

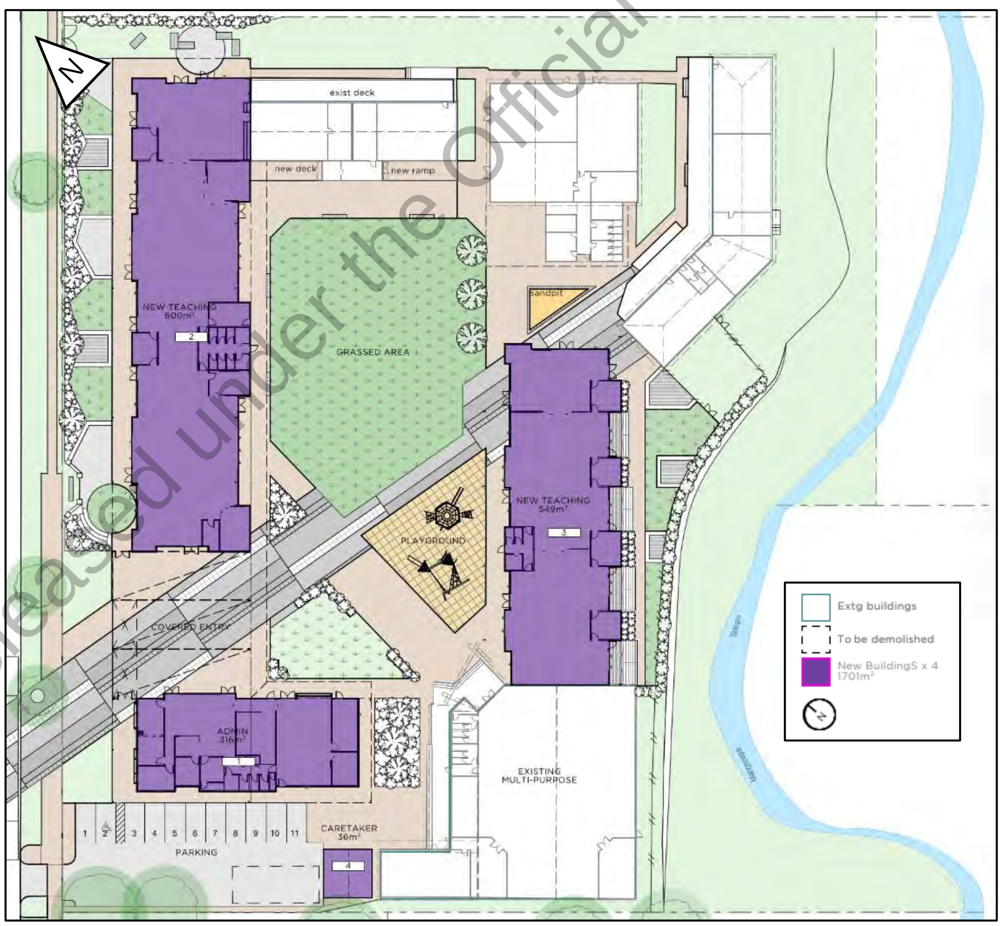

Figure 4: Enlarged Site Re-Development Plan displaying positions of two new teaching blocks, an admin building and caretakers structures (purple shaded areas). (**Source:** DCA Ltd, Mangapapa School | Preliminary Design | Proposed Site Plan, V0604, Dated: 27/05/2019).

2 SITE SETTING

2.1 Desktop Review

2.1.1 General

The Mangapapa Primary School is situated in the Mangapapa suburb within the north-eastern reaches of the wider Gisborne Urban area. The school is bordered to the southeast by the Mangapapa Stream and is located within an urban area with well-established residential buildings. As a part of this desktop review, the relevant hazards mapping and all available and relevant geotechnical reports were reviewed.

A review of the Tairawhiti Maps Hazards[4] overlay noted the following:

- The is within a moderate (orange) liquefaction risk zone.
- The site is outside of the modelled 100yr, 500yr, 1000yr and 2,500yr Tsunami inundation zones.
- The is not shown to be crossed by any active or inactive faults or folds.
- The Mangapapa Stream riverbanks are included within the stability alert mapping.
- The site underlying geology is indicated to comprise Te Hapara sandy loams and Muriwai clay loam and recent alluvium (as per the Poverty Bay Soils overlay).
- A review of the New Zealand Geotechnical Database (NZGD) indicates no available data nearby to review and correlate with.

2.1.2 Previous Technical Reports

The following information has been reviewed as a part of this assessment, with all pertinent information and geotechnical data contained within these assessments synthesised within this report:

1. **LDE Ltd** Geotechnical Investigation Report for Proposed Hall Extension, reference: 9824, dated: 22 July 2010.
2. **LDE Ltd** letter to Mangapapa School titled 'Recommendations for Stream Erosion Protection and Wall Stabilisation at Mangapapa School', reference: 9824, dated: 2 September 2011.
3. **LDE Ltd** Specification for Stream Erosion Protection & Retaining Wall Stabilisation, reference: 10251, dated: March 2012.
4. **Frequency Ltd** report titled 'Mangapapa School Preliminary Investigation Works', dated: March 2018.
5. Appendix 1 of that report, being the **BCD Group Ltd** Geotechnical Investigation Report, project no. 17-0708, dated: 19 January 2018.

[4] Tairawhiti Hazard Maps (https://maps.gdc.govt.nz/H5V2_10/)

6. **LDE Ltd** Geotechnical Assessment for Masterplan Options Report, Mangapapa School, Gisborne, reference: 15344, dated: 13 December 2018. This assessment summarises the foregoing eight years of previous works.
7. **DCA Architects** drawings titled Mangapapa School | Preliminary Design | Proposed Site Plan, reference: 181128 V0604, Dated: 27/05/2019.

The previous geotechnical investigations have generally identified the site to be underlain by 5-6m of low to medium strength sands with low strength cohesive clay materials beneath the sands. The sands are identified as being susceptible to liquefaction and having a susceptibility to lateral spreading towards the Mangapapa Stream, which runs along the southeastern boundary of the site (Figure 1). Aside from the 5-7m high stream bank, the site is essentially flat.

The BCD January 2018 report did not identify any significant soil variations across the site, and review of the LDE 2010 investigation for the then extension to the now existing gymnasium also confirmed a generally consistent subsoil profile across the site.

The BCD January 2018 report generally identified no significant liquefaction potential during SLS earthquake events, and a moderate susceptibility to liquefaction during ULS earthquake events, with theoretical vertical settlements up to 35mm and lateral spreading movements of up to 450mm presented in the report. Review of the liquefaction plots indicates however that these values are worst-case scenarios, and no assessment or discussion of the variation within or across the site is described. In particular, no commentary is provided around the calculation of the maximum distance for lateral spread movement being identified in the test location furthest from the stream bank, which appears to have been influenced by the elevated groundwater level recorded in that test location, with said commentary ordinarily forming a part of the context for the overall assessment of the liquefaction potential hazard at the site for planning purposes.

Fundamentally, the slope instability of the steep stream bank leading down to Mangapapa Stream was not assessed in the BCD January 2018 report, nor was any provisional allowance for a conservative set-back to allow for potential slope instability specified. It is also noted that the Frequency March 2018 report identifies the stream bank stability as an issue in the Planning Assessment by the identification of a 'Site Caution Stability Overlay' in this area, but no mention of this classification is made in or crossed referenced with the geotechnical assessment.

From LDE's previous involvement with the site as part of the assessment of the 2010 extension to the gymnasium, the combination of liquefaction potential, lateral spreading, slope instability, and low strength ground posed a complex issue to the site that resulted in the foundations of the gymnasium next to the Mangapapa Stream comprising a palisade wall with 10m deep piles.

It should be specifically noted that the assessed Preliminary Design Site Plan shows Building 3 to be located in line with the gymnasium and will be a similar distance from the crest of the stream bank. A review of GDC 1m contours indicates that a 3m to 4m distance between Building 3 and the crest of slope. The LDE 2018 assessment indicated that a similar level of deep slope reinforcement should therefore be allowed for in project estimates until a detailed assessment was undertaken to establish the exact nature of the potential issue, unless alternative building configurations were otherwise considered.

The report recommends that this is further explored and refined during the detailed assessment and evaluation of the slope stability conditions with allowance made for the stream bank erosion protection measures installed c.2012.

The previous reports based on the results of the field-testing data recommend Specific Engineer Design (SED) timber pole foundations and suspended timber floors for the site, with piles founded at greater than conventional depths due to the low bearing strata in the near surface subsoils.

It should however be noted that such foundations are only suitable without ground improvement or additional design measures for 'minor to moderate' lateral stretch zones (100mm to 200mm stretch in ULS events). As the calculated lateral spread value presented in the BCD Ltd assessment is up to 450mm, this indicates that some additional ground improvement or building/foundation considerations may also be required, and a contingency allowance was recommended until such time the geotechnical investigations and assessment to support the preliminary design was carried out.

Expected ground improvement works would likely take the form of excavation to the near surface deposits to at least 0.8m followed by construction of reinforced gravel rafts to support shallow braced timber pole foundation systems. If the depth of very low strength soils extends beyond 0.8m, the gravel raft may also require driven timber poles to support the base of the raft. Note that this type of ground improvement works are expected to allow concrete rib-raft style foundation slabs to be adopted should they be preferred.

However, it should also be noted that if a palisade wall is constructed at the site, then this may provide the necessary degree of ground improvement / lateral spread mitigation for the buildings at the site, in which case the conventional timber pole foundations with deepening to beyond the low strength near surface soils to approximately 3m depth as recommended by BCD Ltd is expected to be suitable.

2.1.3 Desktop Study Synthesis

Based on the review of the geotechnical information available to date, the LDE Ltd 2018 assessment recommended that the foundation considerations for the proposed new buildings at Mangapapa School take into account the following:

1. Building 3 is likely to require additional foundation measures such as a leading edge of piles designed to allow for potential loss of support due to slope instability, unless the building is otherwise moved 10m or greater away from the existing crest of slope.
2. A buried palisade wall up to 10m depth may be required to improve the stability of the site in the vicinity of Building 3 and provide the combined slope stability / lateral spreading mitigation.
3. The foundation systems for the remainder of the building developments should be designed as SED timber pole foundations, deepened to take into account the low strength soils at the site.
4. Following the detailed assessment of the liquefaction potential of the site and the lateral spreading characteristics in particular, ground improvement beneath the building footprints may be required.
5. If ground improvement proves necessary, it will likely take the form of reinforced gravel raft construction followed by shallow braced timber pole or waffle slab foundations.

Based on the forgoing and confirmation of the Preliminary Design Building configurations (Figures 3 & 4) this PDGAR aims to specifically address the following:

1. Confirmation of the vertical and lateral spread liquefaction potential across the site.
2. Confirmation of the stability of the stream bank adjacent to Building 3.
3. Comprehensive evaluation of the identified hazards and synthesis of the foundation systems and ground improvement options available to develop the most economical effective mitigation of the hazards for the building development.

2.2 Mangapapa Stream Erosion Protection Works 2009 - 2012

Due to instability (c.2006) and erosion of the Mangapapa north streambank and slope works were undertaken between 2009 and 2012 to remediate the situation the following was carried out:

1. Construction of a **segmental block retaining wall** (designed by John. H Klimenko & Associates). The wall consists of a approx.4.5m high wall with anchor diamond pro segmental wall units forming an 83° from the horizontal face, with a 2.7m to 3.5m wide compacted sand back-fill (engineered fill) reinforced with Fortrac 55/30-20 Biaxial geogrids at 0.6m spacing. The wall also includes the relevant geofabric linings and novaflo subsoil drainage. A min.250mm depth, 600mm wide footing of compacted GAP40 is shown on the plans, however LDE understand that contractor increased the

footing depth during construction with site concrete. All relevant drawings are included in Appendix F.

2. A combination of a **timber pole palisade wall**, **reno-mattress** and **gabion baskets** were recommended, modelled and specified in the LDE Ltd 2011 and 2012 reports with subsequent construction and monitoring occurring May 2012 (Figure 5a – 5i). The design drawings (Appendix F) specify either 150mm SED timber poles installed at 1.5m centres or 200mm SED poles at 2.0m centres to 5.0m depth. Note that a review of our construction records and PS4 certificate did not establish which timber poles were installed as per the specification, with the only apparent photos of the area suggesting undersized poles (possibly 100mm SED at 1.0m centres) (Figure 5c).

It should be noted that the wall has been emplace since 2009 and gabions since 2012 with the only minor slumping in the gabions observed, likely to due to poor packing of the rip-rap gravel within. Otherwise the wall and gabions appear in good condition with no evidence of subsidence, overturning, bulging, rotation, or other deformation of the walls.

Figure 5a: 16/05/2012 – initial undercut.

Figure 5b: 17/05/2012 – reno-mattress placement.

Figure 5c: 16/05/2012: General view of excavation with possible timber pole driven piles exposed in excavation wall.

Figure 5d: 17/05/2012 – gabion in-construction.

Figure 5e: 22/05/2012 – completed segment of gabions. Note old wooden piles and concrete behind gabion.

Figure 5f: 25/05/2012 – completed south gabions.

Figure 5g: 28/05/2012 – completed center and north gabions.

Figure 5h: 28/05/2012 – completed center and north gabions.

Figure 5i: 30/05/2012 – completed center and north gabions.

Figure 5: Selection of LDE Construction Monitoring Photographs detailing construction of reno-mattress and gabion basket installation during May 2012.

2.3 Published Geology

The 1:250,000 geological map of the region[5] shows the site as being underlain by Holocene Age Ocean Beach Deposits consisting of sand. These soils are consistent with dune and beach sand formations.

[5] Mazengarb C. and Speden I. (compilers) 2000: "Geology of the Raukumara area. Institute of Geological and Nuclear Sciences 1:250,000 geological map 6".

No active, or in-active fault traces have been mapped near the site. The nearest active fault trace is the concealed, approximately located Repongaere Fault situated 14km northwest of the site.

2.4 Site Characteristics

The existing Mangapapa School buildings are located on the southern side of Rua Street with the Mangapapa Kindergarten, grassed playing fields, sealed courts and swimming pool on the northern side of Rua Street. The vast majority of the subject site is located on a level Holocene Age dune and beach derived landform which has a slight fall towards the south (Figure 6).

The only feature of geomorphic significance within the investigation area is the meandering Mangapapa Stream located along the southeastern boundary of the site. The steep and high stream bank introduces a primary stability hazard to the site development. The stream trends northeast to southwest across the boundary and is separated from the school by a length of palisade wall supported gabion baskets at the stream edge, a narrow strip of floodplain, and a geogrid reinforced segmental retaining wall up to 4.8m in height which was constructed in 2009 prior to LDE's involvement (Figures 7 - 9).

Figure 6: General current view of proposed classroom 'New Teaching 2'. Photograph taken facing northeast. Note Rua Street in the left-hand-side of image.

Figure 7: General view of segmental retaining wall during construction c 2009 – photograph supplied to LDE Ltd.

Figure 8: General view of the current Mangapapa Stream condition facing north. Note position of gabion baskets and 4.8m high segmental retaining wall.

Figure 9: General view from top of segmental retaining wall facing south.

3 GROUND CONDITIONS

3.1 General

The nature of the ground beneath the site is summarised below and in the appended cross sections. It is based on an integration of published and unpublished data, the geomorphology of the site, surface exposures of the underlying geology, and subsurface investigations carried out at discrete locations. The nature of the ground between the investigation points is inferred and may vary from that described. For details of the materials encountered and measurements of their respective strengths please review the appended investigation logs.

3.2 Geotechnical Investigations

The following details the suburface investigations carried out at Mangapapa School over the past nine years:

1. **LDE Ltd 2010 (Gymnasium Extension)**:
   - Analysis of historic sterographic aerial photographs taken from 1942, 1953 and 1988 to assess key geomorphical features of the site and surrounding area.
   - Two electronic cone penetrometer tests (CPT1 and CPT2) put down using a specialist CPT rig to 15m depth.
   - Six 50mm hand augered boreholes (HA1 to HA6) put down to a target depth of 3m depth or refusal. Measurements of the undrained shear strength were taken at 200mm intervals within cohesive soils encountered down through the boreholes using a calibrated shear vane.

- Five dynamic penetrometer tests (P1 to P5) put down to a target depth of 3m or refusal with measurements taken in 50mm increments.
- One slope profile using a tape and abney level (Cross Section A-A').

2. **BCD Ltd 2018**:
   - One HQ machine borehole (MBH01) put down to 15m depth with standard penetration tests (SPT-N) tests undertaken at 1.5m centres.
   - Four electronic cone penetrometer tests (CPT01 to CPT04) put down using a specialist CPT rig to 20m - 30m depth.
   - Three 50mm hand augered boreholes (HA01 to HA03) put down next to the associated borehole to a target depth of 3m depth or refusal.
   - Three dynamic penetrometer tests put down to a target depth of 3m or refusal with measurements taken in 100mm increments.

3. **LDE Ltd 2019:**
   - Eight electronic cone penetrometer tests (CPT01 to CPT07 & CPT07A) put down using a specialist CPT rig to 20m depth.
   - Eighteen 50mm hand augered boreholes (HA01 to HA18) put down to a target depth of 3-4m depth or refusal. Measurements of the undrained shear strength were taken at 200mm intervals within cohesive soils encountered down through the boreholes using a calibrated shear vane. Shear vane measurements also undertaken in non-cohesive materials to capture indicative strengths in transitional soils.
   - Eighteen dynamic penetrometer tests put down next to the associated borehole to a target depth of 3m or refusal with measurements taken in 50mm increments.
   - Two slope profiles using a tape and abney level (Cross Sections B-B'& C-C').

The locations of the subsurface investigations are shown in Figure 9 below and on the Geotechnical Investigation Plan in Appendix A. The current CPT and hand auger / Scala penetrometer data obtained herein is presented in Appendix B. Logs of the previous machine borehole, hand augered boreholes, Scala penetrometer tests and CPTs are presented in Appendix C.

The LDE investigations during 2010 & 2019 were carried out during winter whereas the BCD Ltd 2018 investigation was undertaken during the summer of 2017/2018.

Figure 10a (upper) and 10b (lower): Clips of Geotechnical Investigation Plan showing locations of all test data sites within property. Note overlap and offset of clips. Refer to Appendix A for full size drawing and legend.

3.3 Engineering Geology

In summary, our investigations generally encountered dune and beach soils consistent with the sediments shown on the published geology for the site underlain by a deep layer of estuarine/marine clay rich sediments. The following is generalised in Table 1 and illustrated in Figure 11.

Specifically, the investigations indicate that the site is underlain by a surface layer of slightly to moderately organic SILT **(Topsoil)** with variable sand content down to 0.1m to 0.7m depth below ground level (bgl). Nominally, the topsoil extends to 0.4m bgl with HA3 noting a deeper layer of surficial organic soils.

The numerous shallow investigations indicate that underlying the surficial topsoil and fill the site is underlain by low to high strength layers of fine-coarse SAND with variable silt, pumice, gravel and shell content and non-plastic SILT **(Holocene Dune & Beach Deposits)** with variable sand content down to between 4.7m and 5.6m bgl (extrapolated from deep testing). Shear strengths recorded *in-situ* with a downhole shear vane indicated undrained shear strengths within the near surface cohesive soils (silts) to be between 35kPa and 132kPa (average of 60kPa) corresponding to a firm to very stiff consistency.

Scala penetrometers undertaken beside each of the hand augered boreholes generally indicated low to moderate strengths (<1 to 6 blows/50mm, average of 1 – 2 blows/50) within the upper 2.0m with a gradual increase in soil strength and density below this depth (1 - >10 blows/50mm). The Scala testing across the site generally reached effective refusal between 1.9m and 3.4m bgl within very dense sands. The CPTs indicated cone tip resistances (qc) of between 2MPa and >20MPa (average of 16MPa) within the Dune & Beach Deposits. SPT N60 values within this deposit were noted to be between 18 and 19, which corresponds to a medium dense consistency.

It should be noted that the following lower strength layers were encountered within the near surface including:

- BCD HA01 between 0.0 – 2.1m (blow counts of <1/50mm),
- BCD HA02 between 1.5m and 1.9m (0.5blows/50mm),
- HA3 between 0.0m and 0.9m (<1/50mm),
- HA4 between 0.0m and 1.0m (<1/50mm),
- HA5 between 1.5m and 1.9m (<1 – 1blow/50mm),
- HA15 between 0.7m and 1.5m (<1 – 2blow/50mm),
- HA17 between 0.4m – 1.7m (<1 – 1blows/50mm) which additionally encountered a lense of non-plastic PEAT between 1.0m and 1.3m bgl,

The shallow auger and Scala investigations generally terminated within the very dense sand layer, and did not penetrate into the subsoils below. The deep CPT testing indicates that the Dune & Beach Deposits extend down to approximately 4.7m and 5.6m bgl.

Underlying the Dune & Beach Deposits the investigations show that the site is consistently underlain by low strength, high plasticity CLAY and silty CLAY (**Late Quaternary Estuarine/Marine Deposits)** with variable occasional fibrous organics and shell content to >30m bgl. Occasional thin lenses of sand between 50mm and 200mm were also observed in the logged machine borehole and CPT profiles (inferred tephra layers). The CPTs indicated cone tip resistances (qc) of between <1MPa and 1MPa. SPT N60 values within this deposit were noted to be between 0 and 1, which corresponds to a very soft soil.

Based on our review of LDE investigation data within Gisborne and a review of the literature[6], it would appear the estuarine/marine muds extend down to approximately 50m depth before

[6] L. J. Brown (1995): Holocene Shoreline Depositional Processes at Poverty Bay, A Tectonically Active Area, Northeastern North Island, New Zealand. *Quaternary International*, Vol. 26, pp. 21-33, 1995.

encountering 'basement' **bedrock** of Tertiary Age. Note however the depth to bedrock is influenced by the proximity to Kaiti Hill and the angle it descends beneath the Poverty Bay flats.

Table 1: Simplified stratigraphic profile beneath the site and assigned geotechnical parameters to subsoil layers.

Depth (m)	Graphic Log	Simple Description	Assigned Geotechnical Parameters
0.0m to 0.7m bgl		Sandy organic SILT **(Topsoil)**	γ = 18 kN/m³
n/a		Rip – Rap Gravel **(Gabion Baskets)**	C = 0 kPa ϕ = 40° γ = 19 kN/m³
n/a		Compacted Sandy Gravel **(Engineered Fill)**	C = 1 kPa ϕ = 35° γ = 18 kN/m³
0.1m to 2.3m bgl		Sand & Silt Mixtures Low to Moderate Strength **(Holocene Dune & Beach Deposits)**	C = 2 kPa ϕ = 30° γ = 18 kN/m³ qc = 2 – 4 MPa SPT N60 = 5
1.75m to 5.5m bgl		Sand & Silt Mixtures Moderate to High Strength **(Holocene Dune & Beach Deposits)**	C = 1 kPa ϕ = 33° γ = 18 kN/m³ qc = 3 – 25 MPa SPT N60 = >15-20
4.7m to >30m bgl		Clay with Clay & Silt Mixtures Low Strength **(Late Quaternary Estuarine & Marine Deposits)**	C = 3 kPa ϕ = 26° γ = 18 kN/m³ qc = <1 – 1 MPa SPT N60 = 0 – 1 Su = 20 – 60kPa
Note: Su (shears strength) inferred from CPT profile correlations.			

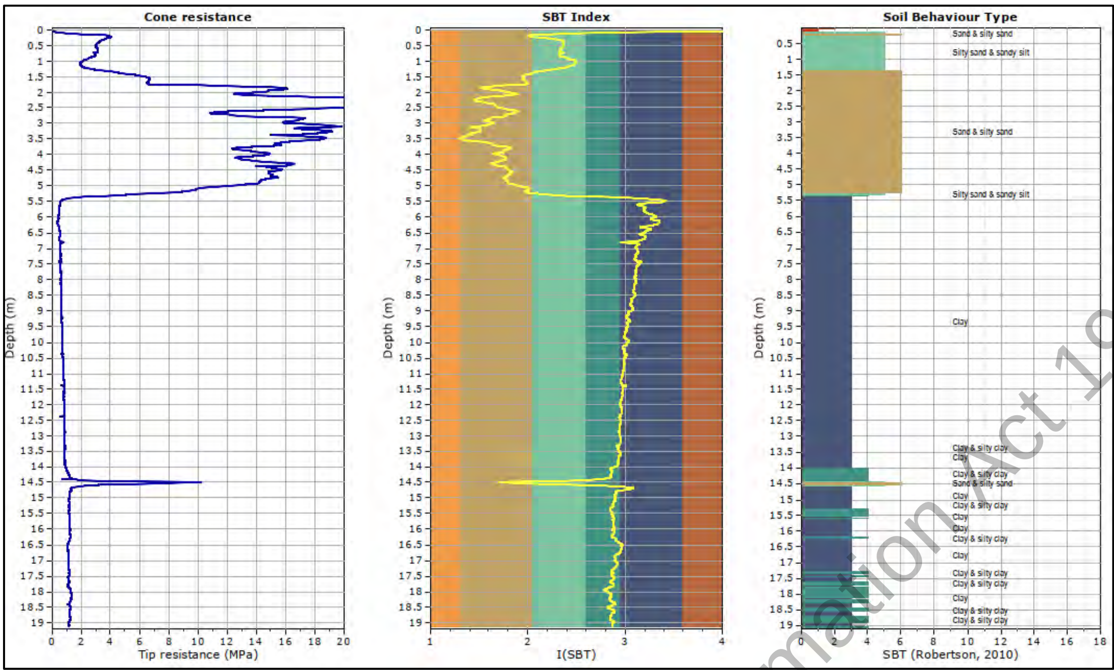

Figure 11: Representative soil profile of CPT-01. Note upper 5.5m of dune and beach deposits (sands) overlying deep deposit of low strength estuarine/marine deposits (silts and clays).

3.4 Soil Moisture Profile and Groundwater Conditions

The soils beneath the site were generally moist to wet down to 3.0m to 3.4m depth, becoming saturated below this depth.

The groundwater table beneath the site was measured between 3.0m and 3.7m depth on completion of the subsurface investigations which is generally consistent with control of the water level by the nearby Mangapapa Stream (Table 2). The phreatic surface (groundwater level) is noted to deepen from west to east across the site. Note that BCD – CPT04 has recorded an anomalously high GWL, which does not match the readings across the site, with the BCD machine borehole beside CPT04 has no recorded static groundwater level measurements. Therefore, it is most likely that the dipper probe has hit a wet patch of side wall and returned a false positive or after penetrating the lower clays the static GWL has under artesian pressures pushed up the hole. Likewise, BCD – CPT01 notes a groundwater level of 2.1m, however the hand augered borehole completed beside it to 3.0m notes no GWL.

The moisture content of the near surface soils is expected to be higher during the winter months or extended periods of wet weather resulting in their saturation at times. The extent of the wetting front will be dependent on the duration of the period of rainfall, but may extend down some 1m to 2m of the surface. Similarly, the groundwater table is expected to rise some 1m to 2m during extended periods of wet weather. In our opinion complete saturation of the ground is possible, but is a low probability occurrence. Complete saturation of the slope is unlikely to occur.

Table 2: Recorded Static Groundwater Levels.

Test No#	Static GWL (m, bgl)	Test No#	Static GWL (m, bgl)
BCD – HA01	NE* >3.0m	**HA1**	3.4m
BCD – HA02	NE* >3.0m	**HA2**	H/C* 2.9m
BCD – HA03	NE* >3.0m	**HA3**	H/C* 2.6m
BCD – CPT01	2.1m	**HA4**	H/C* 2.5m
BCD – CPT02	H/C* 3.1m	**HA5**	3.4m
BCD – CPT03	H/C* 3.5m	**HA6**	3.4m
BCD – CPT04	1.3m	**HA7**	3.5m
BCD – BH01	Not recorded	**HA8**	3.5m
2010 CPT-01b	3.0m	**HA9**	3.6m
2010 CPT02	3.0m	**HA10**	3.8m
CPT-01	H/C* 3.75m	**HA11**	3.5m
CPT-02	H/C* surface	**HA12**	H/C* 2.4m
CPT-03	3.7m	**HA13**	NE^
CPT-04	NE^	**HA14**	3.6m
CPT-05a	H/C* 3.75m	**HA15**	NE^
CPT-06	H/C* 3.8m	**HA16**	NE^
CPT-07	NE^	**HA17**	3.2m
CPT-07a	3.65m	**HA18**	3.5m

Notes:
H/C*: Hole collapse after probe withdrawal at Xm depth, groundwater level unable to be measured accurately.
NE* – GW Not encountered in test hole up to 3.0m. GWL is therefore >3.0m bgl.
NE^ - Shallow refusal, groundwater table not reached.

3.5 Seismic Subsoil Category

The previously completed BCD Ltd report suggests that a Class C – 'Shallow Soil' classification is appropriate for this site, however, then suggests that a more conservative Class D – 'Deep Soil' classification may be used for structural engineering design. The site straddles the borders between a Class C & D site subsoil classification.

Taking into consideration that overall expected performance of the site, the literature[7] indicating a 50m depth to the Tertiary basement bedrock, required Class C maximum thickness limits for soft to stiff soils (soft 20m, firm 25m, and stiff 40m), then based on the measured CPT profiles at the site up to 30m and the inferred extrapolation of similar soils to 50m depth (consistent with subsurface data elsewhere in the Gisborne area), it is therefore our expectation that it is likely that >25m of soft to firm soils and almost definitely in excess of 40m of stiff soils located beneath the site, and accordingly a Class D classification is considered to be most appropriate for this site.

We therefore consider that the site is a Class D deep or soft soil site as defined by NZS 1170.5 (2004) "Structural Design Actions: Part 5: Earthquake actions – New Zealand". It should be noted that the updated National Seismic Model for Gisborne has not yet been published (cf. GNS Report 2015-186 for Hawke's Bay region[8]).

[7] L. J. Brown (1995): Holocene Shoreline Depositional Processes at Poverty Bay, A Tectonically Active Area, Northeastern North Island, New Zealand. *Quaternary International*, Vol. 26, pp. 21-33, 1995.
[8] Rosser BJ, Dellow, GD, compliers, October 2017. GNS Report 2015-186 'Assessment of Liquefaction Risk In Hawke's Bay'

To compensate for the potential update and soil class uncertainty, further sensitivity analysis (Figure 12) has been included in the following liquefaction assessment for the site which replicates and straddles the Class PGA considerations for Class C/D soils.

Table A1: Return Periods for Seismic Design of School Buildings

Building Use[1]	SLS1	SLS2	ULS
Small (< 30m^2) ancillary buildings that are not usually occupied (IL1)	1 in 25	n/a	1 in 100
Larger ancillary buildings (IL2)	1 in 25	n/a	1 in 500
Buildings of lightweight construction, with less than 250 occupants in block (IL2)[2]	1 in 25	n/a	1 in 500
Buildings of lightweight construction, with 250 or more occupants (IL3)	1 in 25	n/a	1 in 1000
All buildings of more than one suspended level and single storey classrooms of heavy construction, with less than 250 occupants in block (IL2)[2]	1 in 25	1 in 100[3]	1 in 500
All buildings of more than one suspended level and single storey classrooms of heavy construction, with 250 or more occupants (IL3)	1 in 25	1 in 250[3]	1 in 1000
Assembly halls, gymnasiums etc. where occupants may congregate (IL3)	1 in 25	1 in 250[3]	1 in 1000

Figure 12: Table A1 – Return Periods of Seismic Design of School Buildings (**Source:** MOE 2016, Designing Schools in New Zealand, Structural and Geotechnical Guidelines, Version 2.0, pg.14). **Note:** Red boxes are indicative of assessed criterion.

4 NATURAL HAZARDS AND GROUND DEFORMATION POTENTIAL

4.1 General

This section summarises our assessment of the natural hazards within the property as generally defined in Section 106 of the Resource Management Act (1991 and subsequent amendments) and the Building Act (2004) and the potential risk that these present to the proposed building in terms of vertical and lateral ground deformation. This section also includes our assessment of ground beneath the building site which is outside the definition of "Good Ground" as defined by the Compliance Document for the NZ Building Code, NZS3604 (2011) "Timber Framed Buildings" and NZS4229 (2013) "Concrete Masonry Buildings Not Requiring Specific Engineering Design". This is any ground which could foreseeably experience movement of 25mm or greater for any reason including one or a combination of compressible ground, land instability, ground creep, subsidence, seasonal swelling and shrinking, frost heave, changing groundwater level, erosion, dissolution of soil in water, and the effect of tree roots.

4.2 Earthquake Hazards

4.2.1 Earthquake Shaking

The site is located in a region of high seismicity. As such, the site can be expected to be subject to moderate high levels of earthquake shaking generated by large distant earthquakes every 20 to 25 years, with a 10% probability of a significant local earthquake occurring within the 50 year design life of the structure. Potential ground deformation resulting from earthquake shaking is discussed in the following sections.

The Ministry of Business Innovation & Environment released draft guidelines for Earthquake Geotechnical Engineering Practice (Module 1, Rev 0, March 2016) for adopting a revised methodology via the NZTA Bridge Manuel (2014) for determining peak ground accelerations under Section 175 of the Building Act. Current best practice in seismic design and ground performance is to adopt this methodology over NZS1170.5 (2004).

Taking the seismic subsoil category of the site into consideration, NZTA Bridge Manual (2014) indicates a peak ground acceleration of 0.32g can be expected during an Ultimate Limit State (ULS) earthquake event and 0.08g during a Serviceability Limit State (SLS) earthquake event for the proposed IL2 buildings.

The forgoing SLS and ULS events correspond to design event return periods of 1/25 years and 1/500 years respectively. To inform the seismic liquefaction analysis carried out, a design event return period of 1/100 years has also been considered, which corresponds to a PGA value of 0.16g define a Mid-range Limit State (MLS) earthquake event. Additionally, the a IL3 ULS design event has been evaluated (1/1000 year return period event) to provide additional correlation and context to address any uncertainty in building use or soil class assessment.

4.2.2 Fault Line Surface Rupture

The GNS NZ Geology Webmap and Active Faults Database[9] do not show any faults passing beneath the site. There also does not appear to be any surface expressions which would indicate the presence of an active fault line beneath or within close proximity to the site. We therefore consider that the surface fault line rupture risk to be low.

4.3 Liquefaction

Liquefaction is the term used to describe the severe strength loss which can occur when saturated loose to medium dense sands and low plasticity silts are subject to seismic shaking. In addition to strength loss, liquefaction may also result in the expulsion of sand, silt and water

[9] http://data.gns.cri.nz/geology/ & http://data.gns.cri.nz/af/

at the surface, post seismic settlement, and in lateral movement towards areas of lower elevation such as rivers or streams, referred to as lateral spreading. Differences in the level of underlying liquefaction due to variations in the ground can result in differential surface settlement. In addition, significant building settlement can occur due to the severe loss of strength and subsequent bearing capacity of the ground.

The deep CPT testing has indicated that the underlying geology consists of variable density upper sands and silts with layers of dense of sand present to approximately 5.0m to 5.5m depth, underlain by a very low to low strength clay and silty clay to >30m. All of the coarse and silty soils below the groundwater table are likely prone to liquefaction. Furthermore, as the site is situated beside the Mangapapa Stream free-face, there is also the potential for a lateral spread component of the liquefaction response of the site. Note that CPT04 & CPT07 have been excluded from this analysis due to their shallow refusal depths.

Two sets of analyses have been undertaken, the first comprising the CPT obtained for the purposes of this Preliminary Design investigation; and the second comprising the analysis of the historic CPT data from the previous investigations under the same analysis assumptions for comparative / correlation purposes. The results are presented in Appendix D.

Analyses have been carried out to determine:

- What material layers beneath the site are likely to be prone to liquefaction under SLS, MLS, and ULS seismic shaking.
- The severity of surface damage due to liquefaction.
- The potential magnitude of surface settlement due to consolidation of the liquefied layers.
- The potential for site extension due to lateral spreading.

The analyses adopted the data from each CPT using geotechnical software (CLiq3). The analyses assumed a peak ground acceleration of 0.08g for the design SLS conditions, 0.16g for SLS2/MLS conditions, 0.32g for IL2 ULS, and 0.41g for the IL3 ULS conditions. An earthquake shaking magnitude of M6.4 (Bridge Manual) has been selected for all scenarios. A conservative average groundwater depth of 2.0m was used in the analyses to model long term average groundwater conditions during an earthquake and future potential sea level rise, representing an average 1m elevation above as measured winter static levels.

We have adopted the CPTu-based calculation method for assessing liquefaction triggering of the soil profile across the site, using the methodology by Boulanger & Idriss 2014, as recommended by the New Zealand Geotechnical Society Guidelines (NZGS; 2016). Liquefaction-induced free-field vertical volumetric strains were estimated for the SLS and ULS design seismic events using the method of Zhang et al. (2002). Default assessment values were utilised within CLiq3 during the liquefaction analyses. These include, but are not limited to, assuming the existing ground is level, utilising an Ic cut-off of 2.6, applying clean sand and overburden corrections, automatic calculations for soil unit weights, applying automatic corrections to the input data at soil transition layers, and a calculation for assessing the "ev weighting factor" which calculates the risk of liquefaction induced settlements within the entire soil profile and assesses what impact the deep liquefiable soil layers have on the overall estimated settlements.

4.3.1 Layers Subject to Liquefaction

4.3.1.1 Serviceability Limit State Conditions

The analyses indicate that under SLS loading (0.08g, M6.4) there is little to no potential for liquefaction to occur (Figure 10). Liquefaction Potential Index (LPI) values of 0 were returned, as well as default low overall probabilities for liquefaction for the current data set. Similarly, values of 0 or at most <1 were returned for the historical data set, as well as default or near-default low overall probabilities for liquefaction. Except for a trivial result in CPT03, all Liquefaction Severity Numbers (LSN) values were zero.

4.3.1.2 Mid-Range Limit State Conditions

The analyses indicate that under MLS loading (0.16g, M6.4) a step-change to ULS behaviour does not occur and that the results are similar to or marginally elevated to the SLS site response to liquefaction (Figure 10). Some zero values of LPI and default overall probabilities for both data sets of liquefaction were returned, with the remainder being near negligible results (LPI <1, low / <5% overall probability). LSN values between 0 and 2 were returned.

4.3.1.3 IL2 Ultimate Limit State Conditions

The analyses show the most of the low to moderate strength sands, silty sands and silt layers within the upper 5.5m beneath the water table are likely to liquefy during an IL2 ultimate limit state earthquake event (0.32g, M6.4). It should be noted that below the Holocene Age Dune & Beach Deposits with the exception of the occasional lense of sand, the Late Quaternary Deposits which account for the majority of the subsurface strata are shown to be non-liquefiable (Figure 10).

A 'low' threshold value (LPI) between < 1 and 5 was returned from the analysis with only CPT-07a returning a 'high' threshold value of 6 (8% of total results), with overall probabilities also low. Note that there is no intermediate category between 'low' and 'high'. In addition, the LSN predictor were noted to remain low on average, with values returned between <1 and 12.

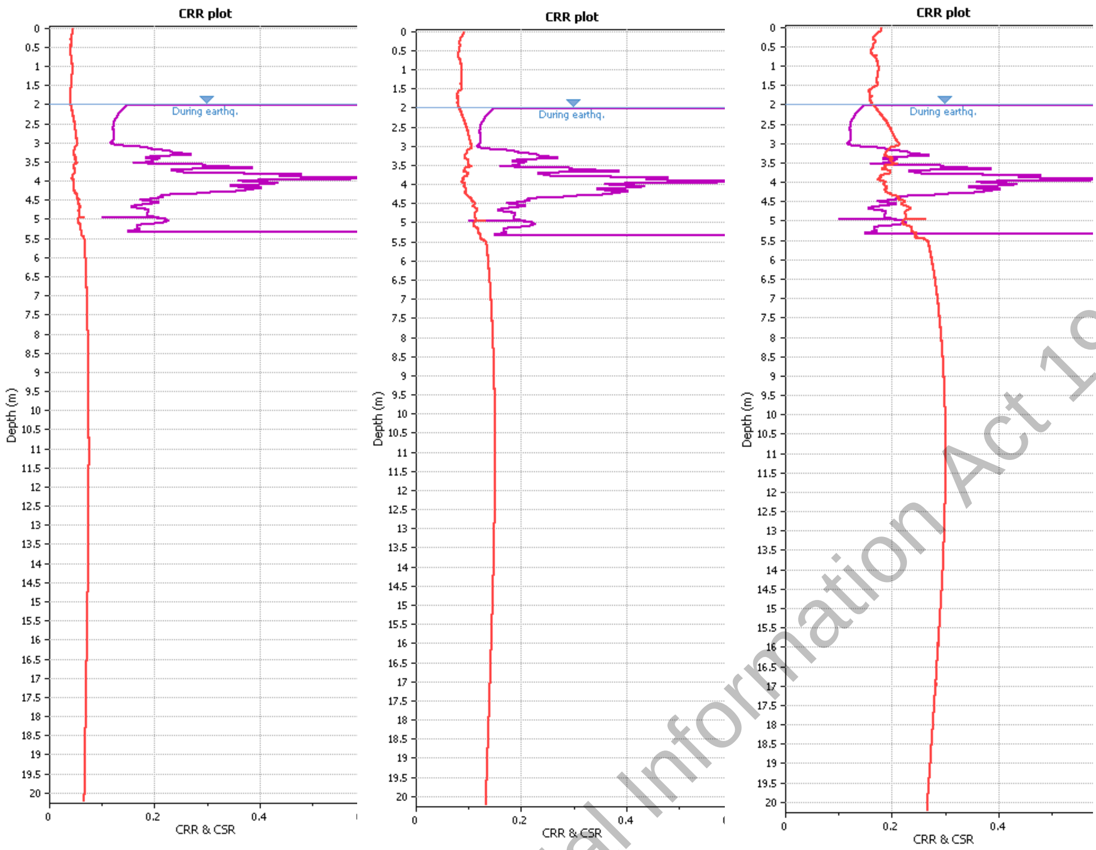

Figure 13: Representative profile layers from CPT105a which are susceptible to liquefaction under SLS, MLS and ULS IL2 seismic conditions (susceptible layers plot to the left of the red line).

4.3.1.4 IL3 Ultimate Limit State Conditions

The analysis shows that for IL3 ULS Conditions (0.41g, M6.4) that the site response is similar to that of the IL2 ULS response with the low to moderate strength sands, silty sands and silt layers and lenses beneath the water table being prone to liquefaction. However; similarly, the dense to very dense sand layers in the upper 4.5m to 5.5m remain unlikely to liquefy. The analysis showed that 'low to high' threshold values (LPI) of between < 1 and 8 were returned with the majority being 'low' < 5. The overall probabilities also remaining low, except for CPTs 05a and 07a indicating higher probabilities. LSN values also remained similar to the IL2 ULS however remain in the lower ranges (LSNs of <1 to 14) with.

Based on foregoing, only minor variances are noted between IL2 and IL3 ULS events, with a plateau in the 'theoretical effects' of liquefaction observed.

4.3.2 Liquefaction Severity Number (LSN)

The Liquefaction Severity Number[10] provides an indication of the likely future performance of the site due to underlying liquefaction. The determination of the number takes into account the thickness of the layers subject to liquefaction and their proximity to the surface. The magnitude of land damage that may be expected has been categorised into ranges Table 3.

Table 3: LSN ranges and observed land effects

LSN Range*	Predominant performance based on actual observations
0-10	Little to no expression of liquefaction, minor effects
10-20	Minor expression of liquefaction, some sand boils
20-30	Moderate expression of liquefaction, with some sand boils and some structural damage
30-40	Moderate to severe expression of liquefaction, settlement can cause structural damage
40-50	Major expression of liquefaction, undulations and damage to ground surface, severe total and differential settlement of structures
>50	Severe damage, extensive evidence of liquefaction at the surface, severe total and differential settlements affecting structures, damage to services.

* LSN is an approximate indicator of liquefaction effects and do not always reflect the actual liquefaction from seismic events. Note that the LSN estimates average ground performance and that a range can occur in each category i.e. some buildings may incur significant damage within the 0-10 range and some buildings may suffer no damage in LSN >30 areas.

The analysis of the data from the CPTs put down within the site return LSN values of 0 in SLS conditions. Under MLS conditions the range of LSN values increased marginally to between 0 and 2. This indicates for SLS and MLS that little to no expression of liquefaction may be expected with this level of shaking.

Under ULS IL2 conditions the LSN values ranged from <1 to 12, indicating little to no expression of liquefaction to tat minor expression of liquefaction with some sand boils may be expected even with this increased level of shaking. Under ULS IL3 LSN values of between <1 and 14 are yielded, which is equivalent to the ULS IL2 classification. These total results suggest that the overall liquefaction vulnerability of the site is low for planning purposes.

4.3.3 Surface Settlement from Liquefaction

Analyses of the potential settlement that could potentially occur following the liquefaction of the sand and silt layers beneath the site have been carried out using CLiq software. The predicted settlements at the surface[11] of the site as a result of possible future SLS, MLS, ULS and IL3 ULS earthquake events are summarised in Table 4 below. The MBIE guidelines reference 'index' values of settlement, this being the settlement occurring within 10m of the ground

[10] Tonkin and Taylor (2013) Liquefaction vulnerability study, Tonkin and Taylor Report 52020.0200/v1.0. February 2013. 52 pages and 14 appendices.
[11] Note: Settlement of the ground surface only. This excludes potential for additional building settlement due to bearing capacity reduction as a result of liquefaction.

surface. Accordingly, for reference and comparison with the MBIE Technical Categories, the 10m depth limited values are presented in brackets for each CPT data set.

Table 4: Potential surface settlements* due to liquefaction. 'Index' settlement within 10m of the surface in brackets. All results rounded to nearest 5mm.

CPT No.	Settlement (mm) for SLS Event (25 Yr) (M6.4, 0.08g)	Settlement (mm) for MLS Event (100 Yr) (M6.4, 0.16g)	Settlement (mm) for ULS IL2 Event (500 Yr) (M6.4, 0.32g)	Settlement (mm) for ULS IL3 Event (1000 Yr) (M6.4, 0.41g)
CPT-01	0 [0]	0 [0]	<1 [<1]	<5 [<5]
CPT-02	0 [0]	<5 [<5]	10 [10]	15 [15]
CPT-03	0 [0]	<1 [<1]	5 [5]	5 [5]
CPT-05a	0 [0]	<5 [<5]	35 [35]	40 [40]
CPT-06	0 [0]	<5 [<5]	10 [10]	20 [20]
CPT-07a	0 [0]	5 [5]	35 [35]	40 [40]
2010-CPT1b	0 [0]	5 [5]	15 [15]	20 [20]
2010-CPT02	0 [0]	<1 [<1]	5 [5]	15 [10]
BCD-CPT01	0 [0]	0 [0]	<1 [<1]	<5 [<5]
BCD-CPT02	0 [0]	<1 [<1]	15 [15]	20 [20]
BCD-CPT03	0 [0]	5 [5]	25 [25]	25 [25]
BCD-CPT04	0 [0]	<1 [<1]	15 [15]	20 [15]

*Calculation assumes no sand boils are present. When sand boils are present, the estimated total settlement is unpredictable.

In summary, negligible liquefaction induced settlement is calculated to occur during a SLS or MLS earthquake event, with calculated theoretical vertical settlements indicated to be between 0mm and 5mm [0mm to 5mm indexed values]. This indicates that in SLS or SLS2 the values are within the typically acceptable limit of 25mm accepted by the Compliance Document for the NZ Building Code[12]. Therefore, under SLS and SLS2/MLS it can be confirmed that the site falls within the TC1 ground performance category in respect of vertical settlements. This is also reinforced by the low overall probability, LPI and LSN predictor values.

Under IL2 ULS events (IL2 structures) theoretical vertical settlements are expected to be between <5mm and 35mm [<5 and 35mm index values] with the average being 15mm of predicted settlement. This level of predicted settlement is indicative of TC1 to TC2 type ground performance (TC1 being equivalent to 0 – 25mm, and TC2 25mm – 100mm). It should be noted that only two of the CPTs exceed the 25mm TC1 tolerances for ULS IL2 (Table 4).

For IL3 ULS conditions, theoretical vertical settlements are expected to be between <5mm and 40mm [<5mm to 40mm indexed values] which is also indicative of TC1 to TC2-like ground performance, as well as suggesting an effective cap to liquefaction damage.

A series of sensitivity checks were undertaken as part of the analyses to determine the effect of the various transition layer and interbedded soil layer influences, as well as the effect of the

[12] MBIE 2012. Repairing and rebuilding houses affected by the Canterbury earthquakes.

groundwater assumption for the site. While a minor shift towards greater settlements and liquefaction vulnerability occurs, these shifts are not substantial and in terms of the index values and conservative groundwater assumption, the site remains within an overall TC2-like category for expected ground performance.

4.3.4 Lateral Spreading

4.3.4.1 General

Lateral spreading typically occurs in sloping ground or level ground close to waterways (e.g. riverbanks, streams, in the backfills behind quay walls). Even a very gentle slope in the ground (of several degrees) will create a bias in the cyclic loads acting on the soil mass during earthquakes which will drive the soil to move in the down-slope direction.

Lateral spreading generally manifests within 100m to 150m of a free-face. The entire site is within 100m of the approximately 2.0m wide, 2.5m deep Mangapapa Stream which is located along the southeastern site boundary. While it is uncertain whether such narrow stream channels will act as a true free-face during a liquefaction event, the presence of the channel in addition to a relatively continuous liquefiable layer between 0.0m and 5.5m bgl could pose a risk of lateral spreading during elevated groundwater conditions. Tape and clinometer cross sections run from the school down the segmental wall to the base of the gabions positioned in the stream note a vertical difference of 6.9m to 7.1m between the existing ground level at the school and the base of the stream. Hence, a 7.0m has been selected as the H value for vertical for this assessment.

The lateral spreading potential assessment has been divided into two sections, the first using Cliq3 to determine theoretical lateral displacements and the second by using a comparison of free-face distance, LSN values and estimate vertical settlements. It should be noted that lateral spreading calculations are less robust than vertical settlements and overall liquefaction susceptibility calculations, and that the added complexity of a retained very steep, high 'free face' is difficult to appraise with a high degree of reliability and hence introduces the greatest uncertainty into the liquefaction assessment.

4.3.4.2 Cliq Lateral Displacement Assessment

The MBIE guidelines outline the following technical category classifications for lateral displacements as a result of lateral spreading under ULS conditions; TC1 – nil, TC2 – <100mm and TC3 Minor to Moderate <200mm and TC3 Major 200mm – 500mm.

Table 5 below summarises the theoretical lateral displacements towards the stream free-face. The analysis shows that under SLS conditions (0.08g) that no lateral spreading is expected to manifest with a resulting TC1 ground performance indicated. In addition, under SLS the liquefaction potential index (LPI) and overall probability show that liquefaction has a very low to low chance of occurring, and therefore any lateral spread movement is considered unlikely to be manifested at this level of ground shaking (i.e. TC1-like ground performance).

The site is shown under ULS IL2 conditions (0.32g) to experience a range of theoretical lateral movements from TC2 ULS < 100mm to movements to in excess or within the TC3 major range (200mm to 500mm). It should be noted that the ULS lateral stretch generally lessens with distance from the free-face, with the highest theoretical values closest to the stream and very steep, high slope face (i.e. CPTs 07a & 1b reporting the highest values due to this proximity).

It should also be noted that the general range of reported values of 50 – 80m from the free face are still relatively high, indicating the potential for large lateral movements due to the liquefaction potential of the low to moderate strength sands below the water table.

It should also be noted that under ULS conditions the data indicates a low liquefaction potential index (LPI), liquefaction severity and overall probability, and similarly the full amount of calculated lateral spread movement is considered unlikely to be manifested within the soil column.

Furthermore, it is specifically noted that the calculated lateral stretch of 3,200mm (Table 5) is excessive and is likely an artefact of the calculation in the programme, and inability to model in retained and reinforced segmental wall and various rows of palisade walls and gabions into the equation.

Table 5: Potential lateral displacements due to lateral spreading. All results rounded to nearest 5mm.

CPT No.	Lateral Displacements (mm)				
	Distances to Free-Face (m)	SLS Event (25 Yr) (M6.4, 0.08g)	MLS Event (100 Yr) (M6.4, 0.16g)	ULS IL2 Event (500 Yr) (M6.4, 0.32g)	ULS IL3 Event (1000 Yr) (M6.4, 0.41g)
CPT-01	55m	0	< 1	15	30
CPT-02	75m	0	< 1	75	105
CPT-03	60m	0	5	50	60
CPT-05a	80m	0	15	440	520
CPT-06	27m	0	25	175	290
CPT-07a	10m	0	50	3,200	3,320
2010-CPT1b	15m	0	190	970	1,080
2010-CPT02	25m	0	10	145	240
BCD-CPT01	80m	0	< 1	5	20
BCD-CPT02	50m	0	15	145	200
BCD-CPT03	60m	0	15	305	325
BCD-CPT04	80m	0	5	165	200

The MLS analysis (0.16g) generally falls within the TC2 range of lateral movements (5 – 50mm) with the exception of CPTs 1b and 07a which are the closest to the free-face. The ULS IL3 calculations additionally increase from the ULS IL2 calculations as demonstrated on the appended plots and in the values presented in Table 5, however do not show a significant increase between the IL2 and IL3 values, unlike the step-change suggested between MLS and ULS IL2.

A TC3 category site performance for the foregoing has been applied in respect of lateral stretch. A sensitivity analysis was also undertaken which involved overall parametric analysis utilising different liquefaction calculation methodologies all four scenarios examined (Figures 14). The analysis shows that the selected methodology is not over reporting values in respect of the other methodologies.

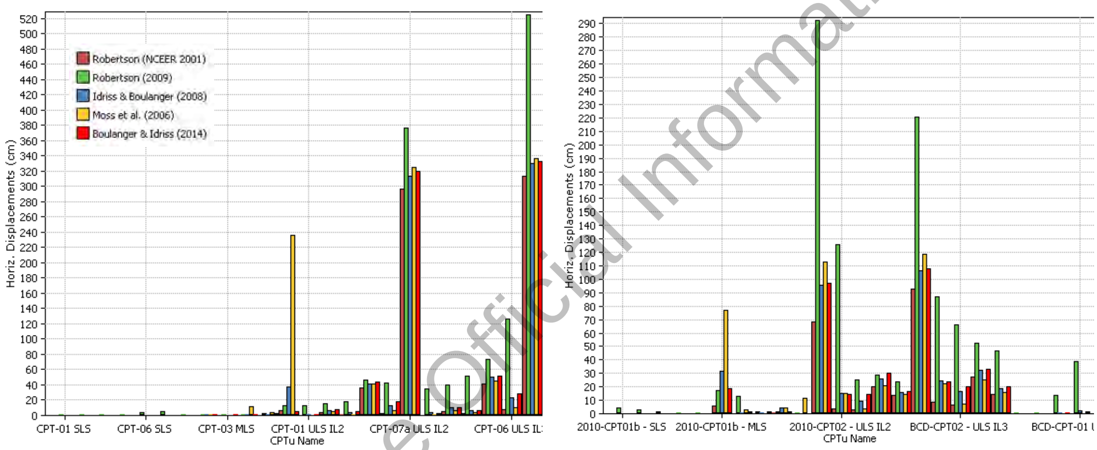

Figure 14: Overall parametric assessment of various liquefaction calculation methodologies in respective of lateral displacements for all four assessment criteria (SLS, MLS, ULS IL2 and ULS IL3). Note the red columns are representative of the B&I 2014 methodology which this assessment is based on.

4.3.4.3 GNS 2015 Methodology

The appendices to the GNS Report 2015-186-Assessment of Liquefaction Risk in Hawke's Bay give an indication whether lateral spreading is likely to occur at a particular site. Using Table 5.2 (referring to channels 2-3m in depth) of the Appendices, the relationship between the distance from the top of the bank and the calculated vertical settlement (primary factor) and LSN values (secondary factor) for the site and is reproduced in Table 6 below.

Table 6: Reproduction of Table 5.2 of Appendices volume of GNS report 2015/186.

Distance from the riverbank (m)	Settlement (mm)	LSN Value
0 to 10m	>25	>7
10 to 25m	>50	>15
25 to 50m	>100	>20
50 to 100m	>200	>30

In the first instance it should be noted that the GNS Report indicates that for channels of this size the maximum extent of lateral spreading is limited to approximately 100m from the stream banks. The commentary in the report supports this conclusion, as it alludes to the difficulty in the current modelling software to accurately predict lateral spread movements, plus the applicability of the Christchurch lateral spread mapping to highly interbedded soils (more relevant for the Heretaunga Plains and likely less so for Poverty Bay).

The data for the site setting out the distance to free-face, vertical settlements, and associated LSN values are presented in Table 7 below. Note that for comparison with the values presented in Table 6 above the values in Table 6 are not tied to any particular level of seismic shaking, but are able to be used to infer whether the triggering condition is met in the various SLS to ULS seismic conditions.

Table 7: Summary of CPT proximity to free-face, theoretical vertical settlements and LSN values for SLS, MLS and ULS. Potential surface settlements* due to liquefaction. All settlements rounded to nearest 5mm. LSNs rounded to the nearest 1d.p.

CPT No.	Distances to Free-Face (m)	Settlements SLS (mm)*	LSN SLS	Settlements MLS (mm)*	LSN MLS	Settlements IL2 ULS (mm)*	LSN IL2 ULS
CPT-01	55m	0 [0]	0	0 [0]	0	<1 [<1]	<1
CPT-02	75m	0 [0]	0	<5 [<5]	< 1	10 [10]	3
CPT-03	60m	0 [0]	0	<1 [<1]	< 1	5 [5]	1
CPT-05a	80m	0 [0]	0	<5 [<5]	1	35 [35]	12
CPT-06	27m	0 [0]	0	<5 [<5]	< 1	10 [10]	3
CPT-07a	10m	0 [0]	0	5 [5]	2	35 [35]	11
2010-CPT1b	15m	0 [0]	0	5 [5]	1	15 [15]	5
2010-CPT02	25m	0 [0]	0	<1 [<1]	1	5 [5]	2
BCD-CPT01	80m	0 [0]	0	0 [0]	0	<1 [<1]	<1
BCD-CPT02	50m	0 [0]	0	<1 [<1]	< 1	15 [15]	4
BCD-CPT03	60m	0 [0]	0	5 [5]	1	25 [25]	10
BCD-CPT04	80m	0 [0]	0	<1 [<1]	< 1	15 [15]	5

*[MBIE Indexed Settlement Values]. Distances to free-face rounded to the nearest 5m.

The guideline conditions as illustrated in Tables 6 and results in Table 7 suggest that lateral spreading is only possible within the 0m – 10m criterion for CPT-07a in ULS which notes a 35mm vertical settlement and an LSN of 11 (criterion being >25mm and LSN >7).

All of the other CPTs do not meet the threshold targets required in Table 6.

The guideline conditions as illustrated in Tables 5 and 6 suggest that lateral spreading is possible within a zone of 0m to 10m of the free-face in ULS seismic events, with the remainder of the site not meeting the indicated trigger values.

Given the limited channel width (≤ 2m), limited channel base (approx. 1-2m), >10m distance to the building from the free-face and the overall site's liquefaction potential, particularly with deep winter groundwater conditions, in our opinion the risk of lateral spreading compromising the proposed structures are low for the site, however a residual risk does exist, particularly to proposed Structure 3, closest the stream.

Taking into account the general TC2-type ground response expected at this site, the enhanced foundations (e.g. reinforced/stiffened raft) are likely to be sufficient in the event that lateral spreading manifests and that any 'block sliding' or 'lateral stretch' like movements will be well within the foundation design tolerances. It is also important to note that the theoretical lateral spread movement must be taken within the context of the overall potential for the site to liquefy. This is further discussed in Section 4.3.5 below.

4.3.5 Liquefaction Analysis Conclusions

Based on the overall low potential susceptibility to liquefaction (LPI), low to moderate calculated vertical settlements, low to minor predication of ground damage and liquefaction effects at the ground surface (LSN, severity) and low to moderate risk of lateral displacements; it is considered that the site has an overall 'low' to 'moderate' susceptibility to liquefaction. This conclusion is supported by the theoretically 'low' to 'minor' liquefaction severity (LSN) values under SLS, MLS and both ULS seismic conditions. This low to moderate risk conclusion is also reinforced by the considerably deep winter groundwater table, a non-liquefiable soil column below 4.7m to 5.5m depth, and the presence of layers of dense to very dense sand within the upper meters of the site.

Based on the liquefaction analysis the results show that under SLS the site is generally within TC1 tolerances, with a step change to ULS not occurring under MLS conditions. However, in respect of lateral spreading, the theoretical MLS calculations (5 – 190mm) reflect a TC2 to TC3 minor – moderate classification. In both ULS IL2 and IL3 the site loadings result in a TC2 site classification in respect of vertical settlements (5mm and 40mm [5mm to 40mm indexed values]).

However, as previously discussed, the site in respect of lateral spreading in ULS conditions is noted to be between 5mm and >500mm with majority of results >100mm; this directly correlates to a TC3 classification in respect of lateral stretch. These values should not be taken out of the context for the overall potential for the site to liquefy as the overall probability remains low with matching low range LPI and LSN indicators. The exception is CPT07a closets the free-face which has a high probability for liquefaction and similar high risk LPI.

It should be noted that, the Ministry of Business Innovation & Environment guidelines for Earthquake Geotechnical Engineering Practice (Module 4, Rev 0, November 2016) indicates that *"resistance to lateral seismic loading is not necessarily critical to the safe performance of the buildings during ULS events"* and advises that *"Building damage should be limited and controlled when subject to the ULS earthquake shaking so that the risk of building collapse is very low and so that evacuation of the building occupants may be safely carried out"*.

In summary, there is generally considerably more uncertainty involved with the prediction of lateral spreading distances compared to the estimation of vertical settlements, in particular with the arrangement of this site and the influence of the existing reinforced wall. Therefore, based on the foregoing, the risk of lateral spreading negatively impacting the site (with a TC3 site classification) can be mitigated by the emplacement of a palisade wall (also known as an "in-ground retaining wall or soldier pile wall") behind the segmental wall as illustrated in Figure 15, by mitigating this risk an overall TC2 site classification can be considered for lateral spreading being required. The wall would also have the dual added benefit of reinforcing the slope behind the wall. If this is adopted than TC2-like style of foundation solutions (Type 1 - 4[13]) are expected to be suitable.

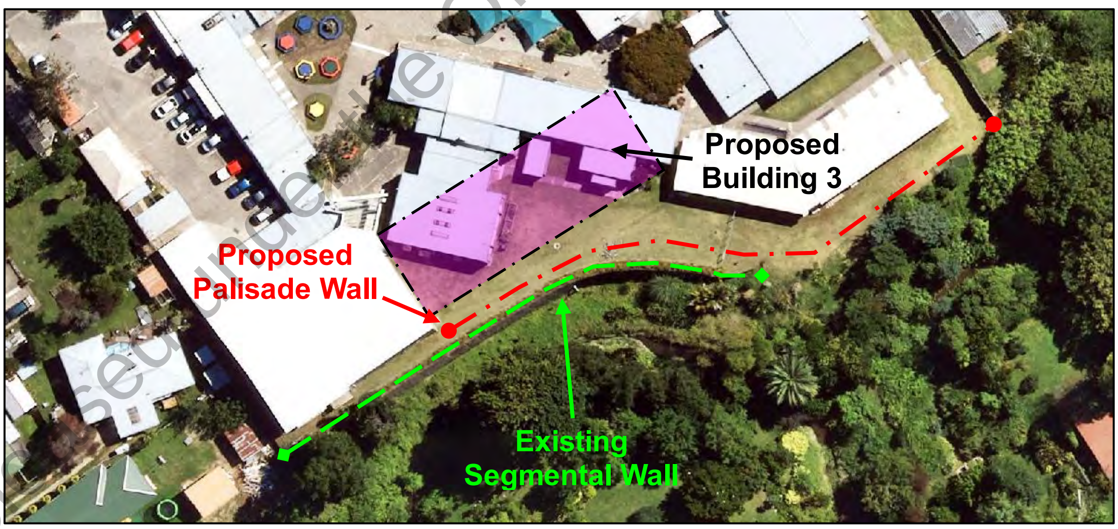

Figure 15: Lateral Displacement Mitigation – Palisade Wall Position.

Continuation of the wall past the proposed Building 3 has dual benefit of mitigating the risk of lateral displacements to the other classroom perpendicular to the slope and the proposed Building 2. The purpose of this wall is further discussed in Section 4.4.

[13] MBIE (2015). Repairing and rebuilding houses affected by the Canterbury earthquakes.

4.4 Slope Instability

4.4.1 Assessment Methodology

The stability of the site has been assessed based on the geomorphology of the surrounding slopes, and numerical stability analyses carried out using specialist geotechnical software on a cross section developed of the underlying engineering geology. The location of the cross sections are shown on the investigation plan in Appendix A.

The numerical analyses included assessments of the slope stability under likely worst case groundwater conditions over the life of the structure (design conditions), the extreme condition of a near fully saturated slope, and ULS and SLS seismic conditions. The soil strength parameters used in the analyses are shown in the appended slope stability analysis print outs and were generally derived from published and unpublished correlation charts and tables for the particular materials encountered in the investigation. Results from laboratory testing of similar materials were also taken into consideration, as were results from numerical back analyses. Consideration has been given to the behaviour of the materials with long term loading, and also their strength under likely worst case moisture content levels.

Minimum factor of safety criteria used in the analyses were a Factor of Safety ≥1.5 for design groundwater conditions and ≥1.2 for extreme groundwater conditions. A displacement based approach was used to assess the seismic performance, with the criteria used being a maximum of 25mm of lateral movement under Serviceability Limit State seismic loads and a maximum of 100mm of lateral movement for Ultimate Limit State seismic load conditions.

The slope stability analysis summaries are presented in Appendix E. A circular and non-circular analysis was carried out to determine the overall slope stability and to model a more realistic potential failure mode due to the inclusion of the retaining walls.

4.4.2 Stability Assessment

4.4.2.1 General

The vast majority of the subject site is located on a level Holocene Age Dune and Beach derived landform which has a slight fall towards the south. The only feature of geomorphic significance within the investigation area is the meandering Mangapapa Stream located along the southeastern boundary of the site. A very steep slope and high slope separate the level site and stream. The slope has been retained by a segmental retaining wall with geogrid reinforcement and granular compacted fill. The wall has been constructed to a very steep angle of 84° from the horizontal and is retaining between 4.4m and 4.7m of slope and has a very shallow footing for the size of the slope it is retaining. The slope bank has been reinforced with a row of gabions supported by reno-mattress. A palisade wall was proposed to support the gabions however, it has not been proven that they were constructed. The retaining wall as illustrated in Figure 15 is non-continuous and does not extend along the entire slope.

The slope stability analysis has been carried out on Cross Section B-B' and is presented in Appendix E of this report (refer Geotechnical Investigation Plan for Section Lines).

4.4.2.2 Circular Analysis

The numerical slope stability analysis for the circular analysis shows the under static long term dry conditions a minimum theoretical Factor of Safety (FOS) value of 1.6 is obtained with 1.4 obtained applying the winter ground water condition (refer to Appended Stability Plots). The FOS values for dry/static and winter groundwater conditions are considered satisfactory, being greater than 1.5 and 1.2 respectively. Under SLS seismic conditions, the minimum FOS of the slope is noted to be 1.3, which indicates that the site is stable under SLS seismic loads and ground deformation as a result of SLS sized earthquake events (0.08g) is unlikely. This indicates that under regular conditions the slope is fundamentally stable.

Under ULS State seismic loads (0.32g) the slope returned a minimum FOS value of 0.9, which indicates there is potential for the slope to undergo movement in large earthquake events. Critical slip circles are shown to extend from the Mangapapa Stream, under the segmental wall and daylight within the slope grounds by some 18m from the slope crest. While catastrophic failure has a low probability of occurrence, the soils beneath the site could be expected to be subject to yielding, resulting in a predicted ground deformation of < 5mm based on a yield PGA value (FoS of 1.0) of 0.31g calculated using the Martin & Qui (1994) methodology. This generally indicates that deep seated mass-wasting of the slope is unlikely.

4.4.2.3 Non-Circular Analysis

Due to the presence of the retaining wall, a non-circular analysis was also undertaken as non-circular slip surface can be expected to be generated. A Cuckoo Search methodology was utilised for this analysis.

The numerical stability analysis returned minimum theoretical FOS values of 1.1 and 1.0 for dry/static and winter groundwater conditions. The FOS values are considered unsatisfactory, being less than 1.5 and 1.2 respectively for steady state long term conditions. Under SLS seismic conditions, the minimum FOS of the slope is noted to be 1.0, which indicates that the site is marginally stable under SLS seismic loads and earthquake events (0.08g).

Under ULS seismic loads the slope returned a minimum FOS value of 0.7, which indicates that in a large earthquake there is the potential for the slope to undergo movement. Critical slip circles are shown to extend from the Mangapapa Stream, under the segmental wall and daylight within the slope grounds by some 20m from the slope crest. While catastrophic failure has a low probability of occurrence, the soils beneath the site could be expected to be subject to yielding, resulting in a predicted ground deformation of approximately 100mm based on a yield PGA value (FoS of 1.0) of 0.31g calculated using the Martin & Qui (1994) methodology. This generally indicates that mass-wasting of the slope is approaching the threshold trigger value and is therefore possible.

4.4.3 Stability Analysis Conclusions

These analyses are suggestive of a significant trend towards potential instability of the slope in extreme event scenarios. Therefore, in order to protect both the existing classrooms (futureproof) and the currently proposed classroom block near the slope (Building 3) we recommend that a palisade wall be constructed behind the existing segmental retaining wall (Figure 15). The palisade wall should extend at least 1.0m below the Mangapapa Stream level which indicates an 8.0m founding depth. The wall should be designed for 5.0m of retained height in the event that the segmental wall fails in a significant event and allowance for surcharge of any relevant building loads.

The wall will serve the dual purpose of mitigating the lateral spreading risk to the proposed development also thereby eliminating the requirement for TC3 foundations to be considered.

4.5 Compressible Ground and Consolidation Settlement

The moderately organic SILT (Topsoil) encountered beneath the site is expected to be subject to consolidation due to loading. This will need to be removed and replaced with engineered fill to avoid long term deformation of the building. A thin (0.3m) layer of peat was encountered in TS17 at 1.0m to 1.3m, but was not encountered elsewhere within the site. Apart from these layers, there does not appear to be any compressible ground beneath the building site as defined by NZS3604 (2011).

4.6 Ground Shrinkage and Swelling Potential

Plastic soils can be subject to shrinkage and swelling due to soil moisture content variations which can result in apparent heaving and settlement of buildings, particularly between seasons.

The near surface soils appear to be non-plastic soils with a liquid limit below 50% and a linear shrinkage value below 15% based on their physical characteristics determined during the investigation. Based on the non-plastic nature of the near surface soils, the foundation conditions are not considered to be expansive and therefore no modifications to the foundation designs given in NZS3604 (2011) are required.

4.7 Tree Root Deformation

Trees within close proximity to buildings can result in potentially significant building damage due to heaving as a result of tree root growth, and also settlement due to soil shrinkage from the moisture uptake of the roots.

There are currently large trees along the northwest side of proposed Building 2 at the school frontage. Removal of the trees or thickening and deepening of the foundations adjacent to the tree will be required to address this issue.

4.8 Conclusions

From our assessment of the natural hazard and ground deformation risks presented to the proposed development we consider that a building can be safely located on the site, provided that the recommendations given in Section 5 are adhered to.

5 ENGINEERING RECOMMENDATIONS

5.1 General

It should be appreciated that the recommendations given below are based on the surface and subsurface conditions encountered at the time of the investigation. In addition to the possible variations in the subsurface conditions away from the investigation points within and around the site, changes to the site levels can have a dramatic effect on the recommendations given. Furthermore, cuts into the slopes above and below the site can significantly jeopardise its stability, unless an appropriate measure is put in place to restore the stability of the slope. Accordingly, we should be contacted prior to commencing any earthworks within the slopes to assess how this may affect the subject development. We should also be contacted immediately should the ground conditions encountered vary from that described in this report.

5.2 Site Contouring and Topsoiling

As soon as possible, all final cut-slopes and fill slopes should be covered with topsoil a minimum of 0.10m thick to prevent the ground from drying out readily resulting in the development of cracks.

The finished ground level should be graded so that water cannot pond against, beneath or around the building for the economic life of structure. To achieve this it will be important that the building platform beneath the topsoil grades away from the site.

Contouring should avoid the potential for concentration and discharge of surface water over point locations which could result in soil erosion or instability.

5.3 Palisade Walls

A palisade wall (also known as an “in-ground retaining wall”, “soldier pile wall”, “buried pile wall”) has been recommended for this site. We recommend that palisade wall be emplaced between the segmental retaining wall and the classrooms and extend to the southeast boundary (Figure 15, Section 4.3.5). The wall serves a dual purpose of retaining the site in a slope failure event in terms of slope stability and also mitigating against the effects of lateral spreading to the site.

This “wall” comprises a row of closely spaced piles generally at centres equal to three times the diameter of the piles to retain material by bridging behind the piles in the event that ground in front of the piles is lost due to slope instability. The piles also provide a reinforcing element within the slope.

The piles are likely to be comprised of bored and cast *in situ* timber poles or reinforced concrete piles with a minimum hole diameter of 500mm (i.e. pile spacing 1.5m). Larger diameter, reinforced concrete piles may be required depending on the structural engineering design (SED).

Detailed design of the piles will need to be undertaken by a Chartered Professional Engineer with experience in the structural design of palisade walls.

The following recommendations are made to assist with the engineering design of the reinforced pile retaining wall:

1. The piles should extend across the length of the slope crest as illustrated in Figure 15.
2. The pile diameter should be 500mm or greater with pile spacings at a maximum of three pile diameters (centre to centre) to provide sufficient shear and bending capacity.
3. Allowance should be made for the building loads behind the wall to be imposed on the wall.
4. The piles should be designed to restrain ground down to a vertical depth of 5.0m from the top of the wall. Soils below the wall down to this depth should be assumed to be completely absent. Passive pressure resistance in this zone should not be assumed.
5. The soil parameters given in Section 3.3 should be used for the determination of the active pressures acting on the piles. Full saturation of the ground should also be assumed, unless subsurface drainage is installed.
6. The pile depth should be determined as required to generate sufficient lateral capacity below the depth of potential ground loss. A minimum pile depth of 8.0m is recommended.
7. A capping beam along the top of the piles is recommended.

During construction stage the pile holes should be inspected by a Suitably Qualified Professional to ensure that the ground conditions are as assumed in the engineering design. Modifications to the design may be required. We should be contacted without delay should the ground conditions vary from that described in this report.

5.4 Foundation Design and Construction Recommendations

5.4.1 General

Ground with a geotechnical ultimate bearing capacity of at least 300kPa was not consistently encountered at a shallow depth beneath the site. Based on the test information a geotechnical ultimate bearing capacity of 200kPa is only available from between 0.7m to 2.1m depth. This increases to 300kPa from generally below 2.0m depth. The foundation designs given in NZS3604 (2011) will need to be modified to address this issue.

Additional piled foundations may also be required in the vicinity of HA17 (Building 3) to ensure the foundations extend below the localised area of peat encountered.

Due to the low strength material encountered, it is considered that modified NZS3604 (2011) foundation solutions are suitable for the site. No consideration for lateral spreading is required as this expected to be mitigated by the palisade wall

It is recommended that either:

1. SED driven or augered timber piled solutions be developed on the basis of an available bearing capacity of 200kPa; or
2. An enhanced raft foundation slab be constructed at the site; or
3. The foundations of the proposed buildings be founded on a reinforced gravel raft.

Enhanced slab foundation systems (Item 2 above) should comprise a 300mm pod floor enhanced waffle slab foundation as per Option 4 of Section 5.3.1 of the MBIE “Guidance: Repairing and rebuilding houses affected by the Canterbury earthquakes” guidelines (updated March 2017).

Conventional shallow foundations or conventional waffle slabs designed and constructed in accordance with NZS3604 (2011) can be constructed at the site, provided that they are founded on a reinforced gravel raft and that the foundations do not extend below 0.5m into the surface of the gravel raft. The specification for the gravel raft is set out in Section 5.4.2 below. Note that cantilevered foundations may be required for parts of Building 2 and Building 3 where the construction of the gravel raft is restricted.

5.4.2 Reinforced Gravel Raft

All fill forming part of the building platform needs to be placed in a controlled manner to an engineering specification that follows the general methodology given in NZS 4431 (1989) "Code of practice for earthfill for residential development". This includes the design, inspection and certification of the fill by a Chartered Professional Engineer or Professional Engineering Geologist. This will be particularly important to enable the building proposed for the site to be able to be constructed in accordance with NZS3604 (2011) "Timber Framed Buildings" or NZS 4229 (2013) "Concrete Masonry Buildings Not Requiring Specific Design".

The reinforced gravel raft will be required to have one layer of basal geogrid and geofabric allowing for conventional ribraft type foundations to be utilised. The geogrid should extend to half raft height (0.5m) and be lapped back by a minimum of 1.0m and tensioned – this will also allow for a minimum of 0.5m clearance for services. The geofabric should be lapped back at 0.1m depth and covered by 100mm of hard fill. The raft construction should follow the specification provided below.

The topsoil, un-controlled fill, variably low strength near surface soils present will need to be removed from the footprint areas to a minimum depth of 1.0m and replaced with engineered fill. Care must be taken to not over-excavate the site, particularly during wet periods. Groundwater is located several metres below the ground surface and is unlikely to be encountered during the earthworks.

The excavation for the replacement fill needs to extend beyond the footprint of the building by at least 1.0m to provide the lateral support for the building loads. The cut slopes on the side of the excavation should be shaped to 2V:1H (63° from the horizontal) to allow for short term stability of the cuts during construction and easier construction of the gravel raft in relation to placement of geogrid and geofabric.

Following an inspection of the excavation by the certifying Engineer or their representative, its base should be proof rolled. Following inspection, a nonwoven geotextile fabric separation layer (Bidim A19 or equivalent) should be placed to prevent the potential migration of fines into the aggregate fill material.

The basal geogrid (e.g. Tensar Triaxial TX160, Duragrid Biaxial 40/40, or equivalent) should be placed into the undercut overlying the geofabric with compacted aggregate placed above in lifts not exceeding 150mm with a certifying fill test carried out once the engineered fill has been brought up to the design level.

The following fill specification is recommended:

1. All unsuitable materials, including low strength organic silt and uncontrolled fill etc shall be stripped from the building footprint area.
2. The fill footprint area shall be inspected by the certifying engineer's representative prior to the placement of fill.
3. The replacement fill should comprise suitable well graded aggregate (e.g. GAP40 or GAP65, or other well graded hardfill), placed uniformly into the excavation in layers not exceeding 150mm in thickness. The fill should be placed at the optimum moisture content recommended by the suppliers of the material. Alternatively, the material should be inspected and approved as suitable material by a Suitably Qualified Professional. Material which is wet or saturated shall not be placed unless that is the optimum moisture content for the fill. The fill should be compacted to achieve the strengths given in the Table 8 below.

Table 8: Recommended Fill Compaction Criteria

Dynamic penetrometer (non-cohesive fill)		
	Average value not less than	3 blows/50mm
	Minimum single value	2 blows/50mm
Air voids percentage		
	Average value not more than	10%
	Maximum single value	12%
Maximum dry density percentage		
	Average value not less than	95%
	Minimum single value	92%

Compaction should be carried out using 5 – 7 passes over each lift with a steel drum roller. Compaction using a Bobcat, excavator, truck or other vehicle other than a compactor is not likely to achieve the required strength for the fill to be certified.

Provision should be made to ensure that the earthworks are conducted with due respect for the weather. The fill should not be placed on to wet ground, especially if ponded water is present.

Vibration compaction should not be used if the base of the excavation is wet or if the fill is wet of optimum otherwise the fill strength may be significantly reduced from the resultant moisture uptake until the excess pore pressures have dissipated. The time for this to occur is variable but is likely to take more than one day.

5.5 Verification Checks

5.5.1 Fill Placed beneath Foundations

As required by NZS3604 (2011) and NZS4229 (2013), any fill beneath the building will need to be certified by a Chartered Professional Engineer or Professional Engineering Geologist in accordance with NZS4431 (1989). A "Certificate of Suitability of Earthfill for Residential Development" will also be required in accordance with NZS3604 (2011) and NZS4229 (2013).

In order for the fill to be certified, the excavation will need to be inspected by the certifying Engineer or Engineer's representative to ensure that all compressible materials are removed prior to the placement of the new fill.

Verification strength testing of the backfill by the certifying Engineer or Engineer's representative will also be required to ensure that the minimum fill strengths specified in this report have been achieved.

5.5.2 Foundation Excavations

Verification testing of the ground by a Building Inspector or Suitably Qualified Professional is recommended to ensure that the ground conditions at the base of the foundation excavations are as described in this report, and that all unsuitable and loose materials have been removed as required by NZS3604 (2011) and NZS4229 (2013). We should be contacted immediately if these conditions vary from that described in this report. Deepening of the foundations or a modification to the recommendations or design may be required.

5.6 Surface Water Disposal

The site is proposed to be connected to the council stormwater system. On-site disposal is not proposed.

5.7 Wastewater Disposal

The site is proposed to be connected to the council sewerage system. On-site disposal is not proposed.

5.8 Service Pipes

All service pipes, stormwater structures, and culverts should be designed and constructed to ensure adequate capacity, strength, and water tightness to prevent leakage into the platform through blockage, running under pressure, or structural failure.

All service pipes installed within the fill should be flexible, or flexibly joined, so that they may deflect without breaking if the ground settles. Services should be laid down in a 'snaking' pattern, which provides extra length should ground extensions or settlement occurs. Consideration should also be given to providing greater falls than minimums required by the standards for sewer lines for additional resilience.

Gully traps on the perimeter of buildings should be encapsulated in concreted and tied to the building foundation via two hooped reinforcing bars, which are expected to maintain serviceability and prevent differential movement should there be lateral spreading or ground settlement adjacent to the foundation.

In addition, all efforts must be made to keep service pipes trench depths to a maximum of 300mm deep into any gravel rafts with particular attention made to not over-excavate and cut the geogrid layers. Prior consideration should be given to the position of the geogrid reinforcing in gravel raft foundations before services are run to the building.

A record should be kept of the position, type, and size of all subsoil drains, and in particular of their outlets.

5.9 Garden Trees and Shrubs

We consider that that gardens and trees can be established adjacent to the building, however due to the detrimental effect that these can have on the building (particularly trees) we suggest the following be taken into consideration:

The development of the gardens should not interfere with any subfloor ventilation or the drainage system for the building. Care should be taken to avoid the over watering of gardens close to building footings. To reduce the potential for heave damage associated with tree root growth or foundation settlement due to soil shrinkage due to moisture uptake by the trees, trees should be planted a minimum of 0.5 times the mature height of the tree away from the foundation.

5.10 Site Maintenance

Prompt repair of plumbing leaks should be undertaken. Blocked, broken or faulty spouting should be attended to immediately. The discharge of uncontrolled surface water over the site and surrounding areas should be avoided at all costs.

Areas of slope movement and/ or erosion which may occur in the vicinity of the site should be assessed to determine the possible cause and implications that it may have on the stability of the building site. Remediation measures to limit the potential for further loss of land may need to be implemented to protect the value of the property.

6 OTHER CONSIDERATIONS

This report has been prepared exclusively for by DCA Architects Ltd C/- Ministry of Education (MOE) with respect to the particular brief given to us. Information, opinions and recommendations contained in it cannot be used for any other purpose or by any other entity without our review and written consent. LDE Ltd accepts no liability or responsibility whatsoever for or in respect of any use or reliance upon this report by any third party.

This report was prepared in general accordance with current standards, codes and practice at the time of this report. These may be subject to change.

Opinions given in this report are based on visual methods, and subsurface investigations at discrete locations. It must be appreciated that the nature and continuity of the subsurface materials between these locations are inferred and that actual conditions could vary from that described herein. We should be contacted immediately if the conditions are found to differ from that described in this report.

This report should be read in its entirety to understand the context of the opinions and recommendations given.

Our analyses and opinions of the stability of the site have been based on the site geomorphology and ground conditions at the time of the investigation. Alteration of the slope gradients by cutting or filling could result in significant changes to the stability of the site which could be detrimental. We should be contacted immediately if there are any proposed changes to the slope profile, as well as the incidence of landslippage within the vicinity of the site.

The wall design is based on the ground conditions and ground profiles at the time of design. Changes to the surface profile and design use could have detrimental consequences to the stability of the wall. We should be contacted immediately if there are any changes eventuating or proposed to the ground immediately behind or below the wall. This includes the incidence of landslippage below the wall, the stockpiling of material behind the wall, or changes in the use of the wall (e.g. to support a building or vehicles).

For and on behalf of LDE Ltd

Report prepared by:	Report reviewed & authorised by:
Jonathan-Adam Mukhtar	Dave Dravitzki
MEngNZ (EngGeol)	*CMEngNZ (PEngGeol)*
Engineering Geologist	*Chartered Professional Engineering Geologist*

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APPENDIX A

GEOTECHNICAL INVESTIGATION PLAN

CLIENT	PROJECT	DRAWING TITLE
DCA Architects PO Box 5111 Rotorua 3044	Mangapapa School, Gisborne.	Geotechnical Investigation Plan

LAND DEVELOPMENT & ENGINEERING LTD.

No.	REVISION	BY	DATE

DESIGN:	JM	PROJECT STATUS:	Construction
DRAWN:	JB	PROJECT:	15344
DATE:	19.07.19	SHEET:	1 of 1
CHECKED:	JM	DRAWING No:	G2
SCALE A3:	1:500	REV:	A

APPENDIX B

SUBSURFACE INVESTIGATION DATA

LDE LAND DEVELOPMENT & ENGINEERING LTD.

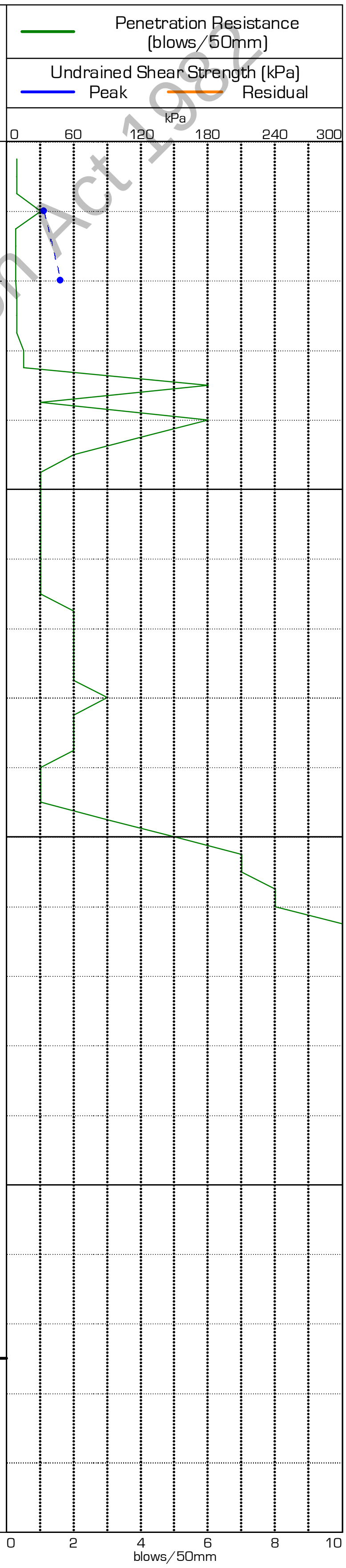
BOREHOLE LOG

Test ID:	HA1
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m   N: 0.0 m   Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0	0.0		M		OL	SILT, organic, sandy, dark brown, moist	Topsoil
		SV 33kPa		F		firm	
		SV 48kPa	M	F	ML	SILT, sandy, light brown, firm, non-plastic, moist	Dune/Beach Sand
-0.5	0.5						
			M	VL	SM	SAND, medium, silty, light brown, very loose, moist	
				D		dense	
				MD L		medium dense loose	
-1.0	1.0						
				MD		medium dense	
-1.5	1.5						
				L		loose	
				MD D		no silt, light grey medium dense dense	
-2.0	2.0						
				VD		very dense	
-2.5	2.5						
						fine, light greyish brown, trace orange mottling	
-3.0	3.0		W			wet	
			S			saturated	
-3.5	3.5					End of Borehole at 3.5m depth/ *Refusal due to collapse of borehole/ *Water table at 3.4m depth/ *	
-4.0	4.0						



BOREHOLE LOG

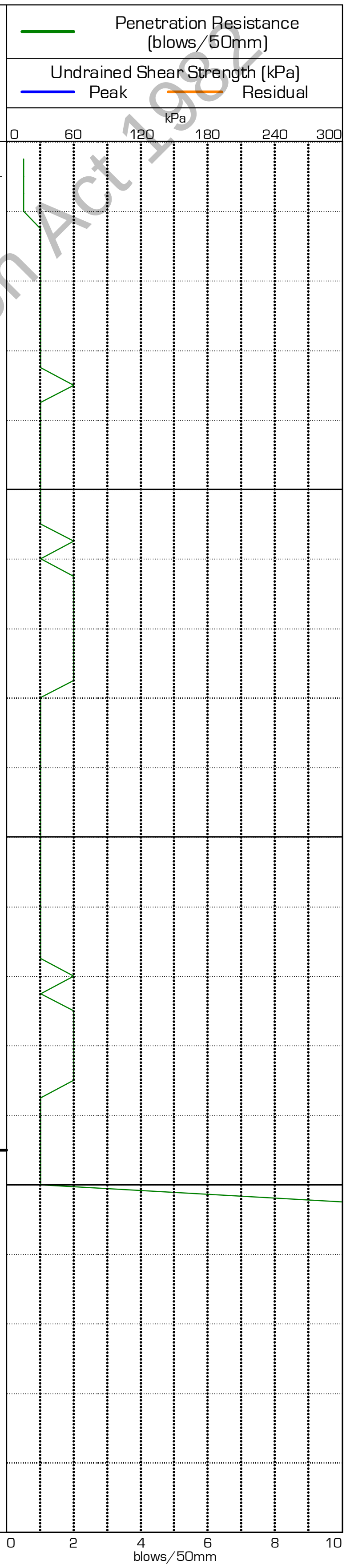
LAND DEVELOPMENT & ENGINEERING LTD.

Test ID:	HA2
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M		OL		SILT, organic, dark brown, moist	Topsoil
			M		ML		SILT, sandy, light brown, non-plastic, moist	Dune/Beach Sand
-0.5	0.5		W	L			SAND, medium, dark brown, loose, wet	
-1.0	1.0						brown	
			M				light grey, moist	
-1.5	1.5			L			loose trace pinkish colouring	
							light brown	
-2.0	2.0						fine, silty, light yellowish brown, trace orange mottling	
							medium, no silt, light grey	
-2.5	2.5			MD			shell fragments, medium dense	
				L			loose	
-3.0	3.0			VD			End of Borehole at 2.9m depth/ *Refusal due to collapse of borehole/ *No watertable encountered/ *	
-3.5	3.5							
-4.0	4.0							



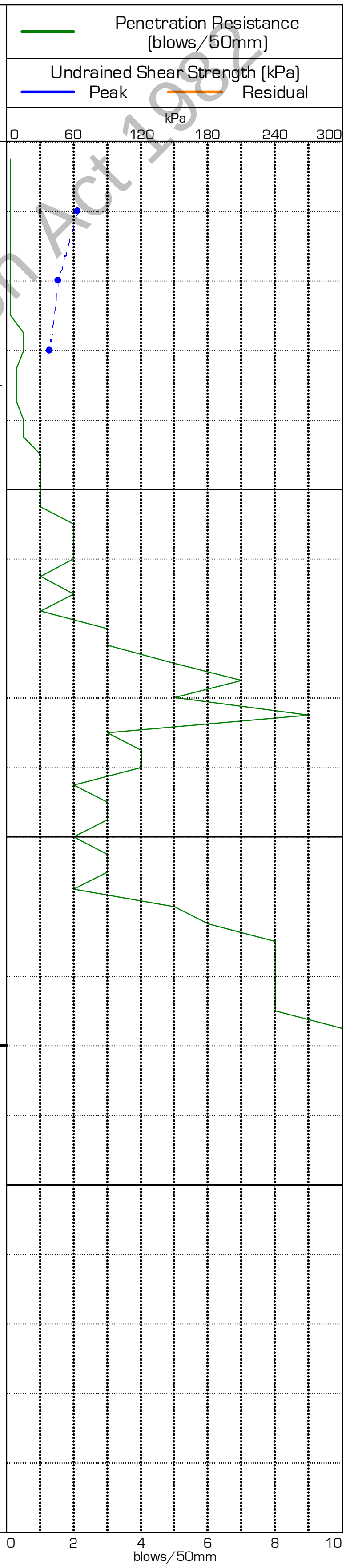
BOREHOLE LOG

LDE — LAND DEVELOPMENT & ENGINEERING LTD.

		Test ID:	HA3
		Sheet:	1 of 1
Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger — Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0	0.0		M		OL	SILT, organic, dark brown, moist	Topsoil
		SV 63kPa		St		stiff	
		SV 46kPa		F		firm	
-0.5	0.5						
		SV 38kPa					
			M	VL	SM	SAND, fine, silty, brown, very loose, moist	Dune/Beach Sand
				L		loose	
-1.0	1.0		M			medium, no silt, light grey, shell fragments, moist	
				L-MD		loose to medium dense	
						fine, trace dark mottling, medium dense	
-1.5	1.5			D		dense	
				VD		very dense	
				MD-D		medium dense to dense	
-2.0	2.0						
				D		silty, trace orange mottling , dense	
				VD		light yellowish brown, very dense	
-2.5	2.5						
						End of Borehole at 2.6m depth/ *Refusal due to auger spin/ *No watertable encountered/ *	
-3.0	3.0						
-3.5	3.5						
-4.0	4.0						



LDE LAND DEVELOPMENT & ENGINEERING LTD.

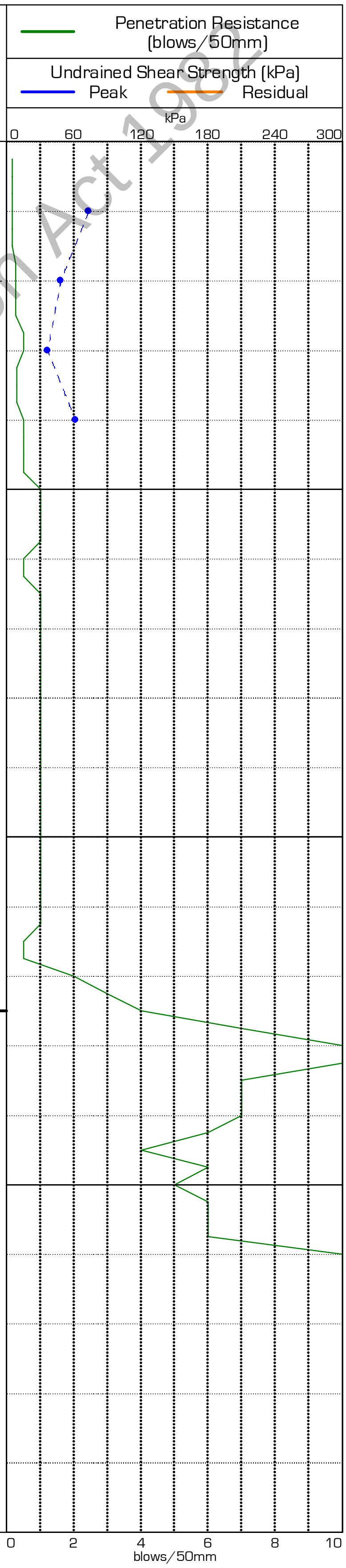
BOREHOLE LOG

Test ID:	HA4
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger — Vane ID: 1609	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0	0.0		M		OL	SILT, organic, dark brown, moist	Topsoil
		SV 73kPa		St		stiff	
		SV 48kPa	M	F	ML	SILT, light brown, firm, non-plastic, moist	Dune/Beach Sand
-0.5	0.5						
		SV 36kPa					
		SV 61kPa		St		sandy, stiff	
-1.0	1.0		M	L	SM	SAND, fine, silty, brown, loose, moist	
				VL		very loose	
				L		loose	
-1.5	1.5						
-2.0	2.0					greyish brown	
				VL		very loose	
				MD		medium, no silt, light grey, shell fragments, medium dense	
-2.5	2.5					End of Borehole at 2.5m depth/ *Refusal due to collapse of borehole/ *No watertable encountered/ *	
-3.0	3.0						
-3.5	3.5						
-4.0	4.0						



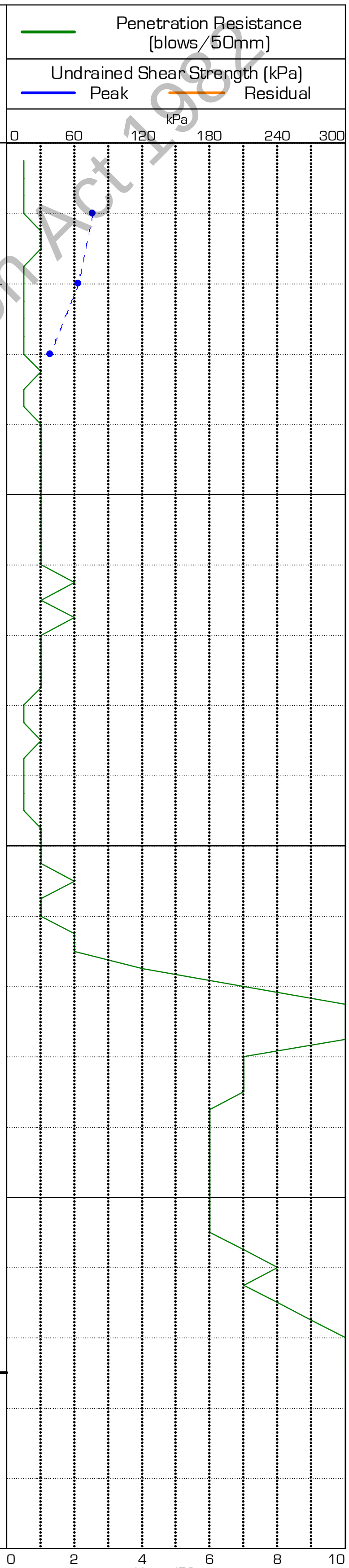
BOREHOLE LOG

LDE LAND DEVELOPMENT & ENGINEERING LTD.

Test ID:	HA5		
Sheet:	1 of 1		
Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger / Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0	0.0		M			SILT, dark brown, moist	Topsoil
		SV 76kPa		St		stiff	
		SV 63kPa	M	St	ML	SILT, light brown, stiff, non-plastic, moist	Dune/Beach Sand
-0.5	0.5						
		SV 38kPa		F		sandy, firm	
			M	L	SM	SAND, fine, silty, light brown, loose, moist	
-1.0	1.0					brown	
-1.5	1.5						
				VL		medium, no silt, light grey, very loose	
						trace brown mottling,	
-2.0	2.0			L		loose fine, silty	
				MD		medium dense	
				D		medium, light yellowish brown dense	
-2.5	2.5						
				D		no silt, trace orange mottling	
			M	F	ML	SILT, sandy, light brown, firm, non-plastic, moist	
-3.0	3.0		W	D		SAND, medium, light grey, shell fragments, trace brown mottling, dense to very dense, wet	
			S			saturated	
-3.5	3.5					End of Borehole at 3.5m depth/*Refusal due to collapse of borehole/*Water table at 3.4m depth/*	
-4.0	4.0						



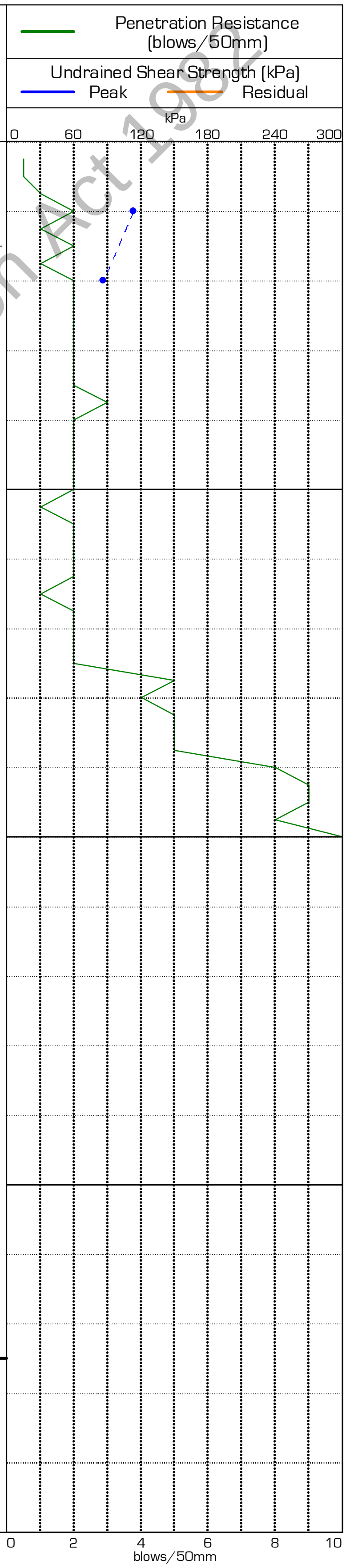
BOREHOLE LOG

Test ID:	HA6
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger — Vane ID: 1609	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M		OL		SILT, organic, dark brown, moist	Topsoil
		SV 113kPa		VSt			very stiff	
			M	VSt	ML		SILT, sandy, light brown, very stiff, non-plastic, moist	Dune/Beach Sand
		SV 86kPa		St			stiff	
-0.5	0.5		M	MD	SM		SAND, fine, silty, light brown, medium dense, moist	
-1.0	1.0							
-1.5	1.5			D			medium, no silt, light grey dense	
				VD			fine, shell fragments, very dense	
-2.0	2.0							
							medium, no shell fragments	
-2.5	2.5						fine	
							medium, shell fragments	
-3.0	3.0		W				fine, silty, no shell fragments, wet	
							trace orange & brown mottling	
			S				medium, saturated	
-3.5	3.5						End of Borehole at 3.5m depth/ *Refusal due to collapse of borehole/ *Water table at 3.4m depth/ *	
-4.0	4.0							



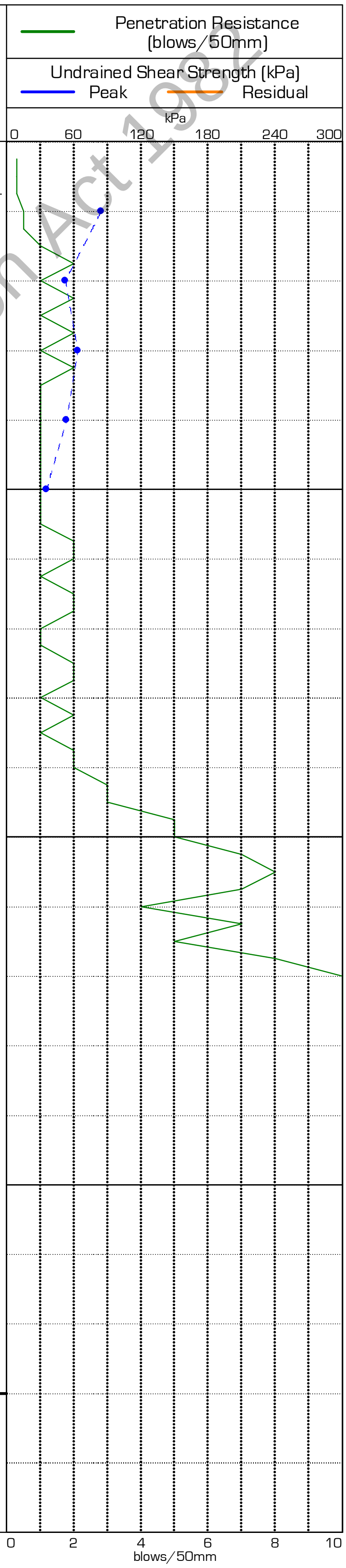
BOREHOLE LOG

Test ID:	HA7
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m   N: 0.0 m   Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0	0.0		M		OL	SILT, organic, dark brown, moist	Topsoil
		SV 84kPa	M	St	ML	SILT, sandy, light brown, non-plastic, moist stiff	Dune/Beach Sand
		SV 52kPa					
-0.5	0.5	SV 63kPa					
		SV 53kPa					
-1.0	1.0	SV 35kPa		F		firm	
			M	L-MD		SAND, medium, light grey, loose to medium dense, moist loose	
-1.5	1.5						
-2.0	2.0			D		dense shell fragments	
				VD D		very dense dense fine, no shell fragments, trace brown mottling	
				VD		very dense shell fragments	
-2.5	2.5						
						trace of gravel, medium, subrounded	
-3.0	3.0		W			silty, no gravel, trace orange mottling, wet	
-3.5	3.5		S			saturated	
						End of Borehole at 3.6m depth/ *Refusal due to collapse of borehole/ *Water table at 3.5m depth/ *	
-4.0	4.0						



BOREHOLE LOG

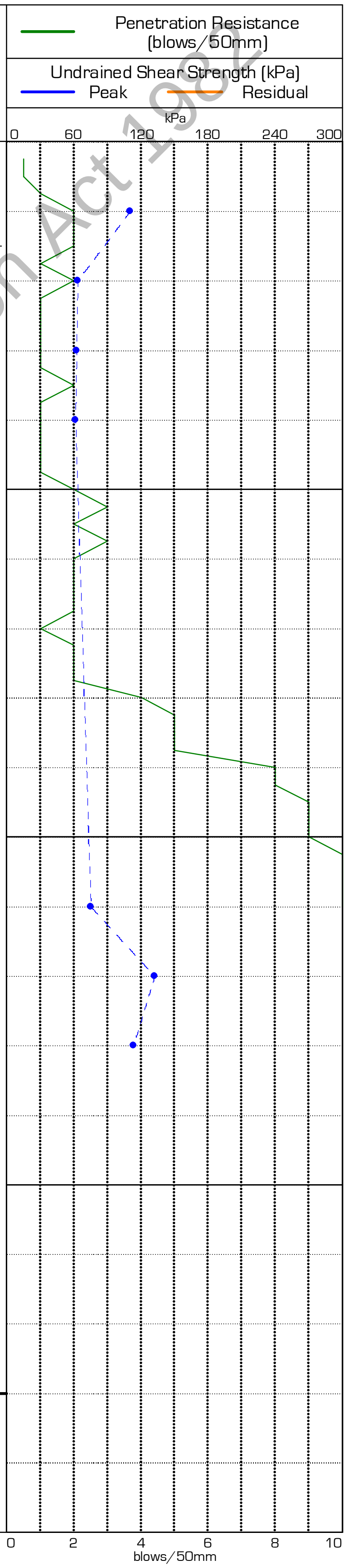
LAND DEVELOPMENT & ENGINEERING LTD.

Test ID:	HA8
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger Vane ID: 1609	Checked by:	DD

Position: E: 0.0 m   N: 0.0 m   Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0	0.0		M		OL	SILT, organic, dark brown, moist	Topsoil
		SV 110kPa		VSt		very stiff	
			M	VSt	ML	SILT, light brown, very stiff, non-plastic, moist	Dune/Beach Sand
		SV 63kPa		St		stiff	
-0.5	0.5						
		SV 62kPa				sandy	
		SV 61kPa					
-1.0	1.0		M	MD	SM	SAND, fine, silty, brown, medium dense, moist	
				L		loose	
-1.5	1.5			MD		medium dense	
			W	D		dense, wet	
				VD		very dense	
			M			medium, no silt, light brown, shell fragments, moist	
-2.0	2.0						
						no shell fragments, trace brown mottling	
		SV 75kPa	M	St	ML	SILT, sandy, light yellowish brown, stiff, non-plastic, moist	
		SV 132kPa		VSt		very stiff	
-2.5	2.5						
		SV 113kPa	M			SAND, fine, grey, shell fragments, moist	
						medium, light grey, no shell fragments	
-3.0	3.0		W			fine, silty, shell fragments, trace orange mottling, wet	
						medium, no silt	
-3.5	3.5		S			saturated	
						End of Borehole at 3.6m depth/ *Refusal due to collapse of borehole/ *Water table at 3.5m depth/ *	
-4.0	4.0						



BOREHOLE LOG

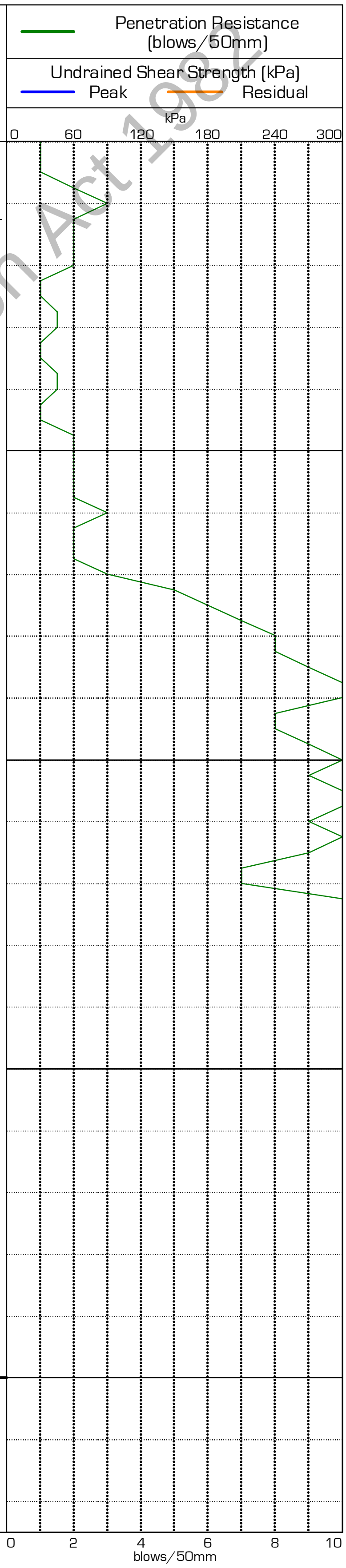
LAND DEVELOPMENT & ENGINEERING LTD.

Test ID:	HA9
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	3/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	JMMA
Test Method:	Hand Auger Vane ID:	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M				SILT, sandy, dark brown, moist	Topsoil
			M	MD			SAND, brown, medium dense, moist	Dune/Beach Sand
-0.5	0.5			L			loose	
-1.0	1.0			MD			medium dense wet	
							light greyish brown	
-1.5	1.5			D			dense	
				VD			very dense	
-2.0	2.0							
							some broken shell and gravel	
				D			some gravel dense	
-2.5	2.5			VD			very dense	
							some silt, brown, some red stains	
-3.0	3.0						no red stains	
							light brown, shells	
							some red stains	
-3.5	3.5		W				light greyish brown, no red stains, wet	
			S				saturated	
							red stains	
-4.0	4.0						End of Borehole at target depth of 4m/ *Water table at 3.6m depth/ *	
-4.5	4.5							



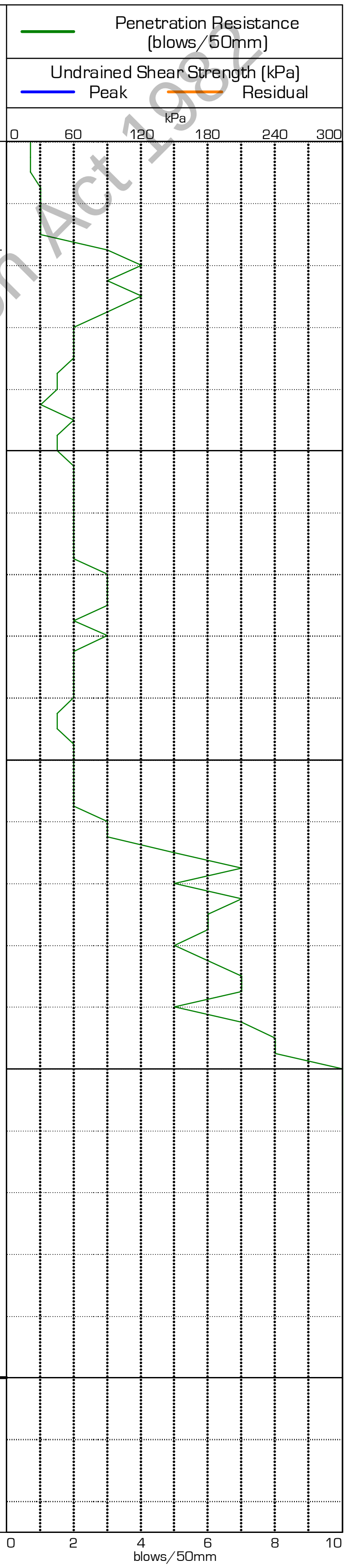
BOREHOLE LOG

Test ID:	HA10
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	3/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	JMMA
Test Method:	Hand Auger — Vane ID:	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m) / Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0 / 0.0		M			SILT, sandy, dark brown, moist	Topsoil
					mixed with brown sand	
					black	
		M	MD-D		SAND, brown, medium dense to dense, moist	Dune/Beach Sand
-0.5 / 0.5						
			L		loose	
-1.0 / 1.0			MD		wet medium dense	
		W			dark brown, wet	
		M			greyish brown, moist	
-1.5 / 1.5					dark brown	
-2.0 / 2.0						
					brown, some grey veins	
			D		greyish brown dense	
-2.5 / 2.5						
			VD		light grey some silt very dense	
-3.0 / 3.0					some shells	
					brownish grey	
					some silt, grey, some red stains	
-3.5 / 3.5		W			wet	
		S			saturated	
-4.0 / 4.0					End of Borehole at target depth of 4m/ *Water table at 3.8m depth/ *	
-4.5 / 4.5						



BOREHOLE LOG

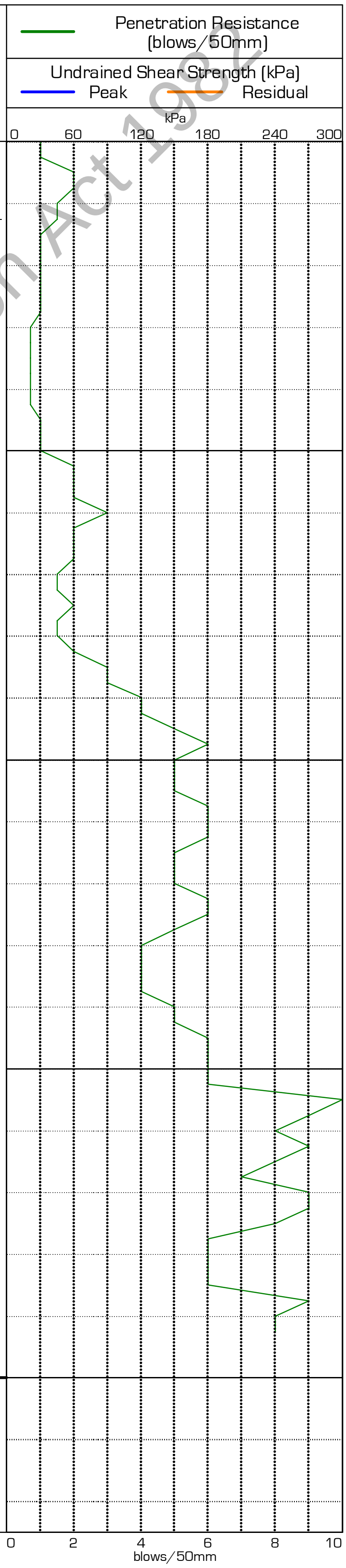
LDE Land Development & Engineering Ltd.

Test ID:	HA11
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	10/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	JMMA
Test Method:	Hand Auger Vane ID:	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M	L			SAND, dark brown, loose, moist	Topsoil
				MD			medium dense	
			M				SILT, sandy, dark brown, moist	
			M	L			SAND, minor silt, light brown, loose, moist	Dune/Beach Sand
-0.5	0.5							
				VL			very loose greyish brown	
				L			loose	
-1.0	1.0							
				MD			brown, medium dense	
				L-MD			loose to medium dense	
-1.5	1.5							
				D			dense	
-2.0	2.0							
			D				grey, dry	
			M				some shells, moist	
-2.5	2.5							
							orangish brown brownish grey	
-3.0	3.0							
				VD			very dense	
			W	D-VD			dense to very dense wet	
-3.5	3.5		S				saturated	
							shells	
-4.0	4.0						End of Borehole at target depth of 4m/ *Water table at 3.5m depth/*	
-4.5	4.5							



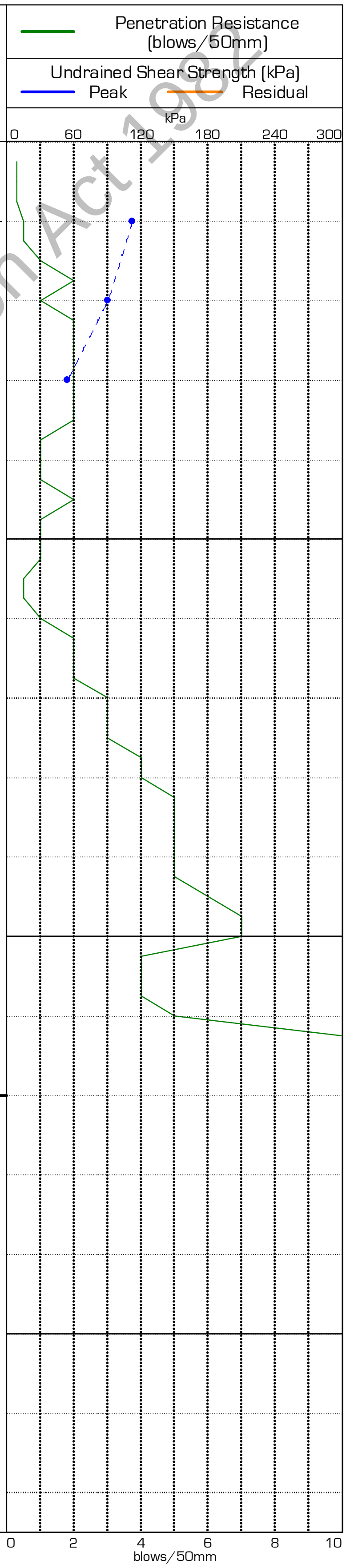
BOREHOLE LOG

LDE Land Development & Engineering Ltd.

		Test ID:	HA12
		Sheet:	1 of 1
Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger — Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0	0.0		M		OL	SILT, organic, dark brown, moist	Topsoil
		SV 112kPa	M	VSt	ML	SILT, sandy, light brown, very stiff, non-plastic, moist	Dune/Beach Sand
		SV 90kPa		St		stiff	
-0.5	0.5	SV 54kPa					
			M	L-MD	SM	SAND, fine, silty, light brown, loose to medium dense, moist	
						brown	
-1.0	1.0			VL		very loose	
				L		loose	
				MD		medium dense	
						medium, no silt, light grey	
-1.5	1.5			D		dense	
-2.0	2.0					silty	
				VD		very dense no silt, some shell fragments	
-2.5	2.5					End of Borehole at 2.4m depth/ *Refusal due to collapse of borehole/ *No watertable encountered/ *	
-3.0	3.0						



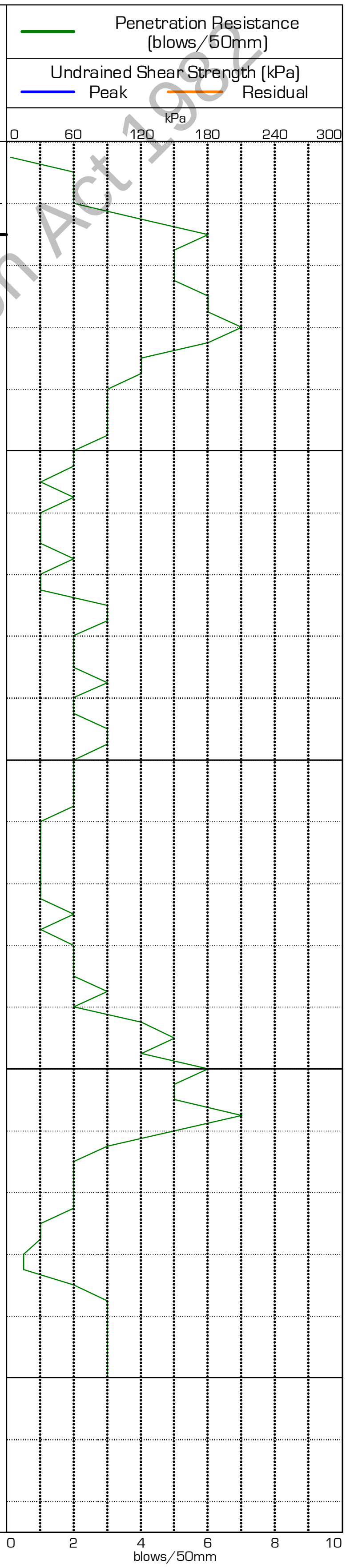
BOREHOLE LOG

LAND DEVELOPMENT & ENGINEERING LTD.

		Test ID:	HA13
		Sheet:	1 of 1
Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M		OL		SILT, organic, dark brown, moist	Topsoil
		SV UTP	M	MDD	SC		SAND, medium, clayey, light yellowish brown, fine to medium subangular fill, medium dense to dense, moist	Dune/Beach Sand
							End of Borehole at 0.3m depth/ *Refusal due to impenetrable material/ *No watertable encountered/ *	
-0.5	0.5							
-1.0	1.0							
-1.5	1.5							
-2.0	2.0							
-2.5	2.5							
-3.0	3.0							
-3.5	3.5							
-4.0	4.0							
-4.5	4.5							



LDE LAND DEVELOPMENT & ENGINEERING LTD.

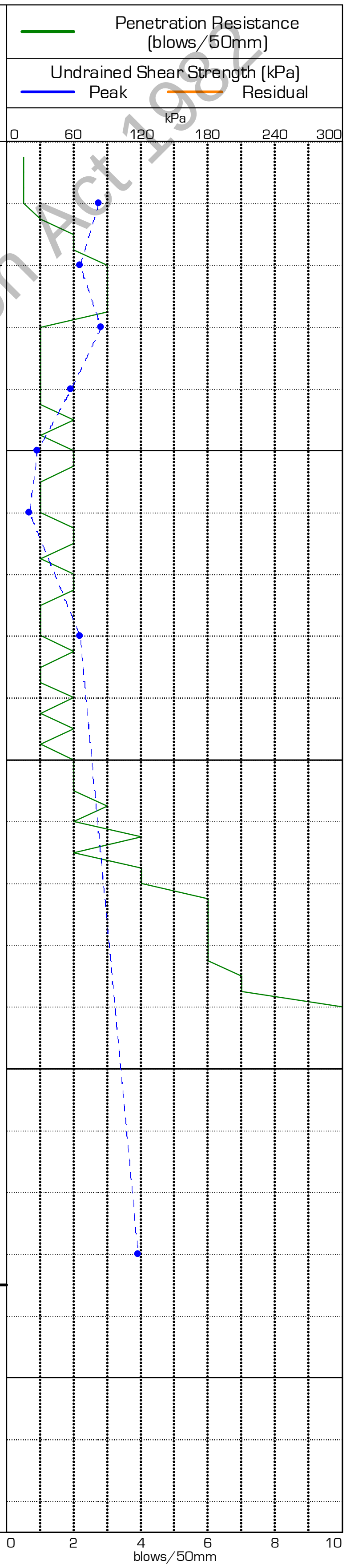
BOREHOLE LOG

Test ID:	HA14
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger — Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Soil Description	Geology
0.0	0.0		M		OL	SILT, organic, dark brown, moist	Topsoil
		SV 82kPa		St		stiff	
		SV 65kPa	M	St	ML	SILT, sandy, brown, stiff, non-plastic, moist	Dune/Beach Sand
-0.5	0.5						
		SV 84kPa					
		SV 57kPa					
-1.0	1.0	SV 27kPa		F		firm	
		SV 20kPa		S		soft	
			M	L-MD		SAND, medium, light grey, loose to medium dense, moist	
-1.5	1.5						
		SV 65kPa	M	St	ML	SILT, sandy, light brown, stiff, non-plastic, moist	
			M	L-MD	SM	SAND, fine, silty, light yellowish brown, loose to medium dense, moist	
-2.0	2.0					light grey, trace orange mottling	
				MD-D		medium, no silt, light greyish brown, shell fragments medium dense to dense	
-2.5	2.5						
						fine, silty, grey, no shell fragments, trace brown mottling	
						medium, no silt, light brown, shell fragments, no mottling	
				VD		fine, trace brown mottling, very dense	
-3.0	3.0					light grey, no shell fragments, no mottling	
						medium, trace brown mottling	
			W			shell fragments, trace orange mottling, wet	
-3.5	3.5						
		SV 117kPa	S			fine, brown saturated	
						End of Borehole at 3.7m depth/ *Refusal due to collapse of borehole/ *Water table at 3.6m depth/ *	
-4.0	4.0						
-4.5	4.5						



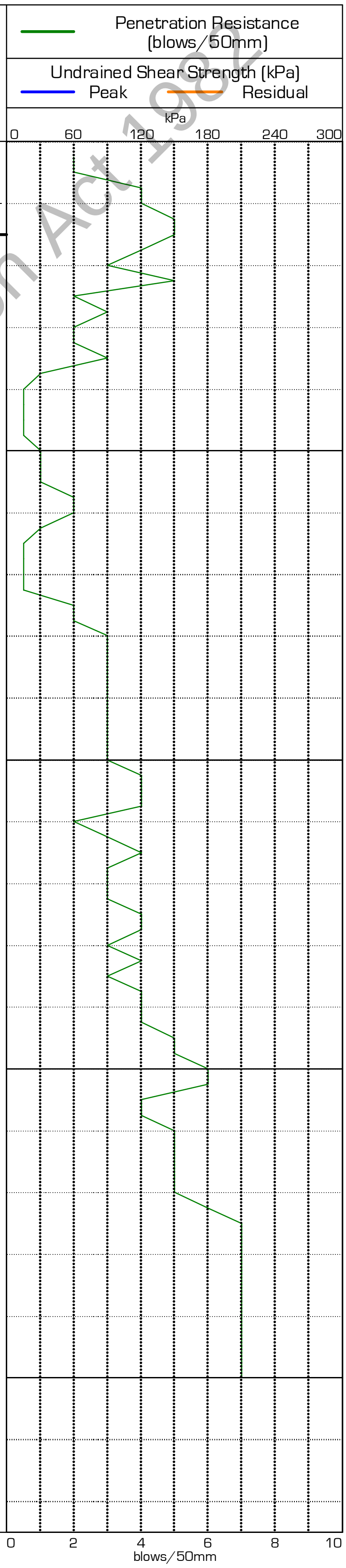
BOREHOLE LOG

Test ID:	HA15
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M		OL		SILT, organic, dark brown, moist	Topsoil
		SV UTP	M	D	SC		SAND, medium, clayey, light yellowish brown, fine to medium subangular fill, dense, moist	Dune/Beach Sand
							End of Borehole at 0.3m depth/ *Refusal due to impenetrable material/ *No watertable encountered/ *	
-0.5	0.5							
-1.0	1.0							
-1.5	1.5							
-2.0	2.0							
-2.5	2.5							
-3.0	3.0							
-3.5	3.5							
-4.0	4.0							
-4.5	4.5							



BOREHOLE LOG

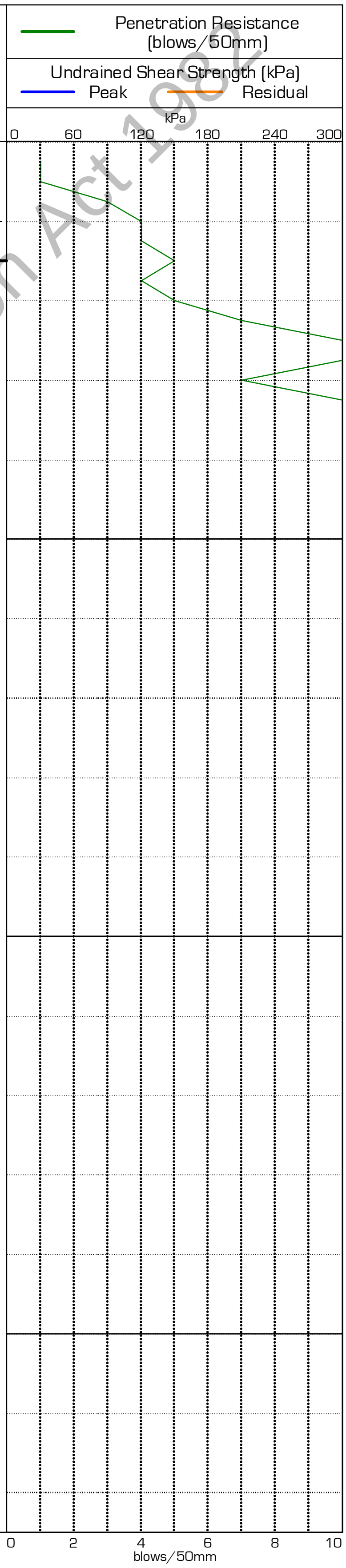
LDE LAND DEVELOPMENT & ENGINEERING LTD.

Test ID:	HA16
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger — Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M		OL		SILT, organic, dark brown, moist	Topsoil
		SV UTP	M	D	SC		SAND, medium, clayey, light yellowish brown, fine to medium subangular fill, dense, moist	Dune/Beach Sand
							End of Borehole at 0.3m depth/ *Refusal due to impenetrable material/ *No watertable encountered/ *	
-0.5	0.5							
-1.0	1.0							
-1.5	1.5							
-2.0	2.0							
-2.5	2.5							
-3.0	3.0							



BOREHOLE LOG

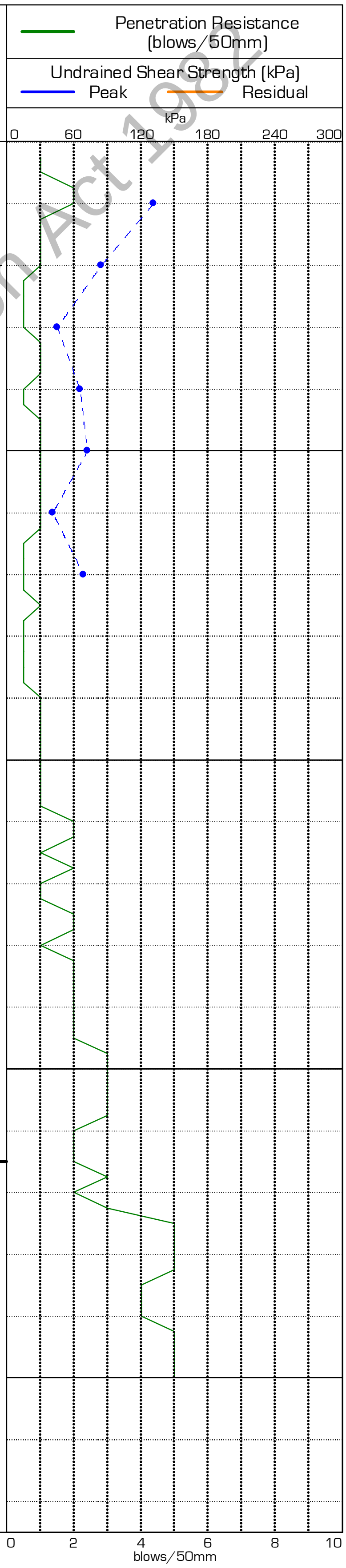
LAND DEVELOPMENT & ENGINEERING LTD.

Test ID:	HA17
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	1/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	CBK
Test Method:	Hand Auger Vane ID: 2554	Checked by:	DD

Position: E: 0.0 m   N: 0.0 m   Elevation: 0.0 m

RL (m)	Depth (m)	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M		OL		SILT, organic, dark brown, moist	Topsoil
		SV 131kPa		VSt			very stiff	
		SV 84kPa	M	St	ML		SILT, sandy, light brown, trace orange mottling, stiff, non-plastic, moist	Dune/Beach Sand
-0.5	0.5							
		SV 45kPa		F			dark brown, firm	
		SV 65kPa		St			stiff	
-1.0	1.0	SV 72kPa	M		Pt		PEAT, black, non-plastic, moist	
		SV 41kPa						
			W	F	ML		SILT, sandy, light brown, firm, non-plastic, wet	
		SV 68kPa		St			stiff	
-1.5	1.5							
			W	VL			SAND, medium, light brown, very loose, wet	
				L			loose	
							fine, silty	
-2.0	2.0							
				L-MD			trace orange mottling, loose to medium	
							medium, light grey, no mottling,	
-2.5	2.5							
							fine, light brownish grey	
-3.0	3.0						trace dark mottling	
			S				grey, saturated	
							End of Borehole at 3.3m depth/ *Refusal due to collapse of borehole/ *Water table at 3.2m depth/ *	
-3.5	3.5							
-4.0	4.0							
-4.5	4.5							



BOREHOLE LOG

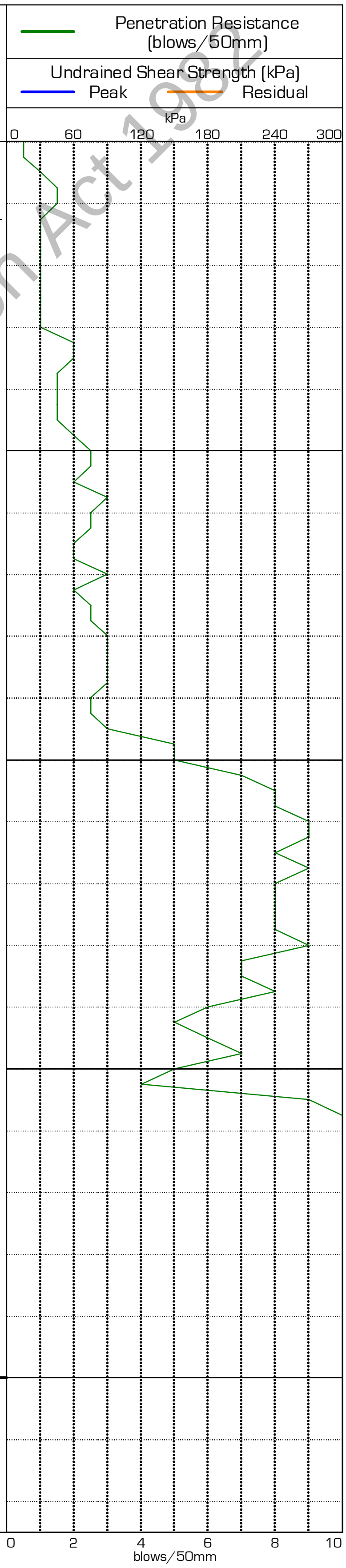
LAND DEVELOPMENT & ENGINEERING LTD.

Test ID:	HA18
Sheet:	1 of 1

Client:	DCA Architects Ltd	Project number:	15344
Project:	Preliminary Design Investigation for Mangapapa School, Gisborne	Date:	10/07/2019
Address:	5 Rua Street, Mangapapa, Gisborne 4010	Logged by:	JMMA
Test Method:	Hand Auger Vane ID:	Checked by:	DD

Position: E: 0.0 m N: 0.0 m Elevation: 0.0 m

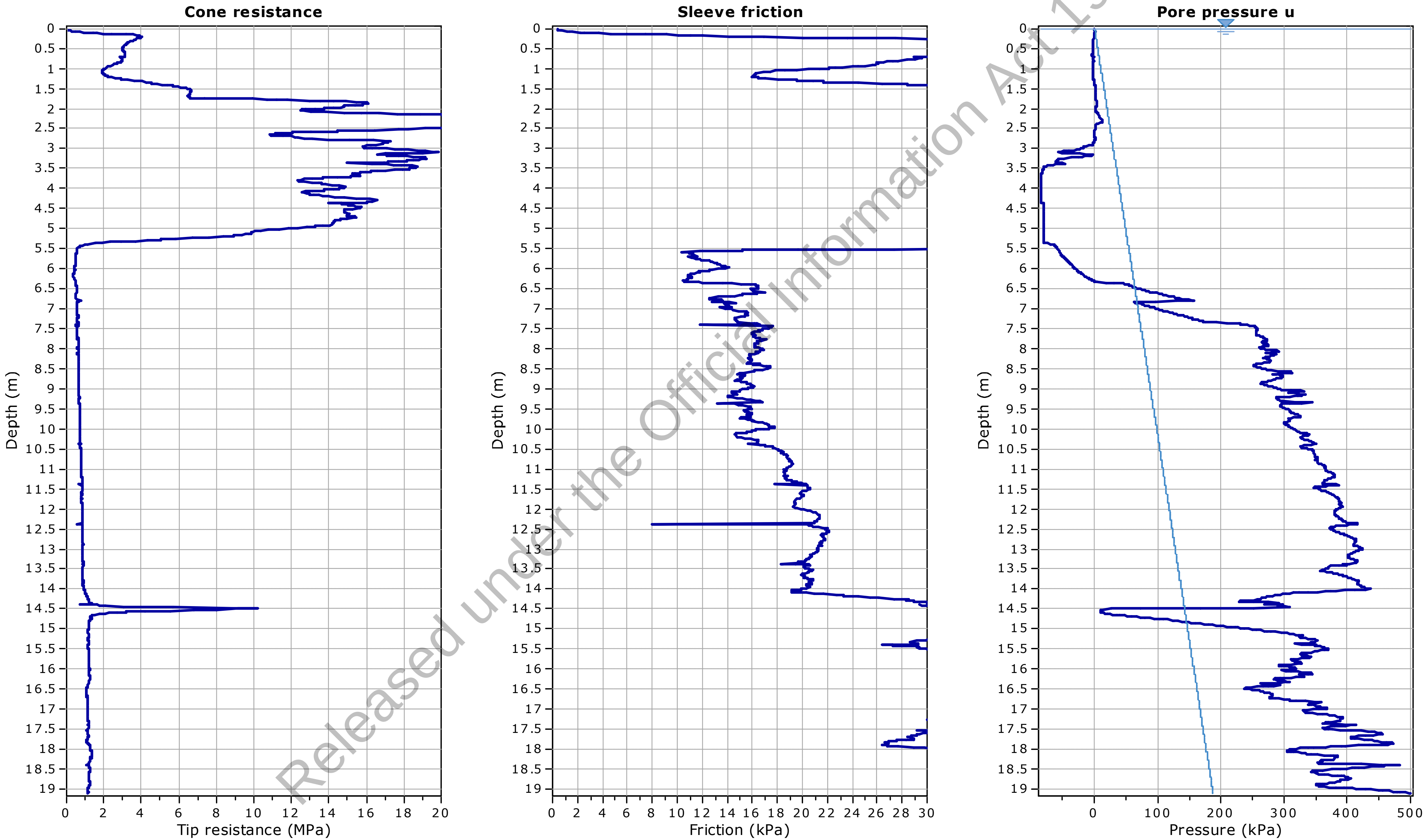
RL [m]	Depth [m]	Samples and Field Tests	Moisture	Strength	Classification	Graphic Log	Soil Description	Geology
0.0	0.0		M				SILT, sandy, dark brown, moist	Topsoil
			M	L			SAND, minor silt, light brown, loose, moist	Dune/Beach Sand
-0.5	0.5						brown	
				L-MD			loose to medium dense	
-1.0	1.0						no silt	
			D				greyish brown, dry	
-1.5	1.5		M				light brown, moist	
							brown	
							greyish brown	
-2.0	2.0			D			dense	
				VD			grey, very dense	
							orangish brown, some gravel and shells	
-2.5	2.5						greyish brown, no gravel or shells	
				D-VD			orangish brown, some shells, dense to very dense	
							greyish brown, no shells	
-3.0	3.0							
			W	VD			some gravel, very dense no gravel, wet	
-3.5	3.5		S				saturated	
-4.0	4.0						End of Borehole at target depth of 4m/*Water table at 3.5m depth/*	
-4.5	4.5							



LDE Ltd
33 Grey Street
Gisborne
www.lde.co.nz

CPT: CPT-01
Total depth: 19.10 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **geotechnical Investigation**
Location: **Mangapapa School**



LDE Ltd
33 Grey Street
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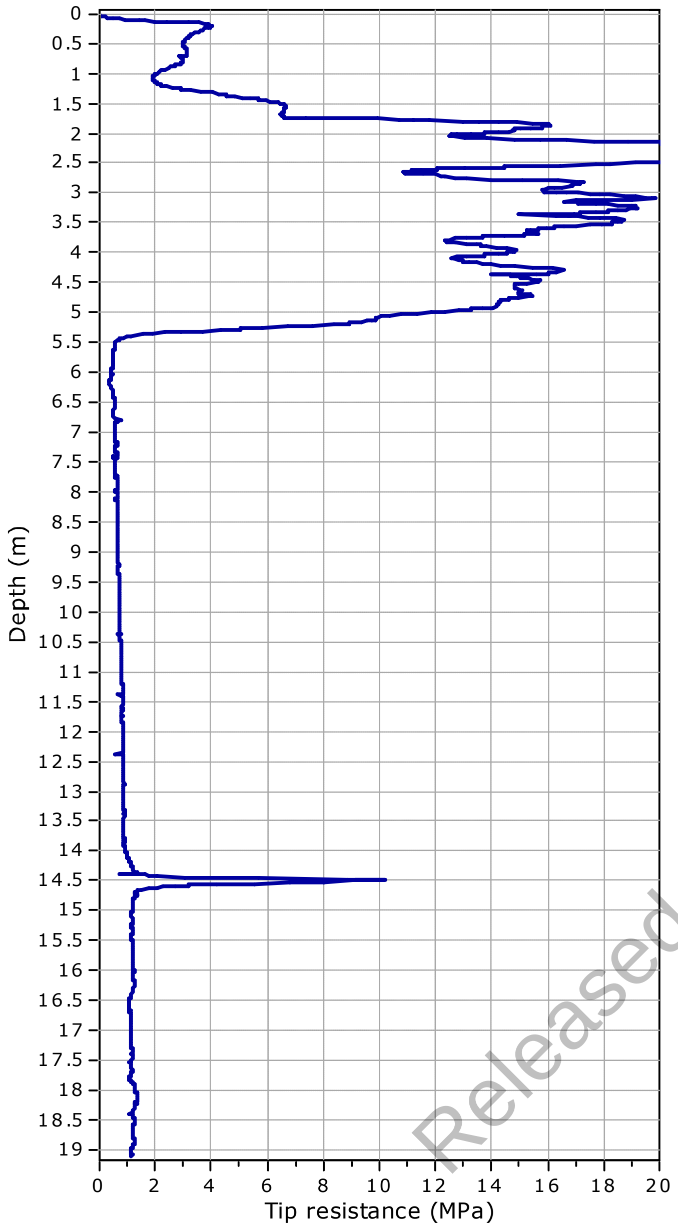
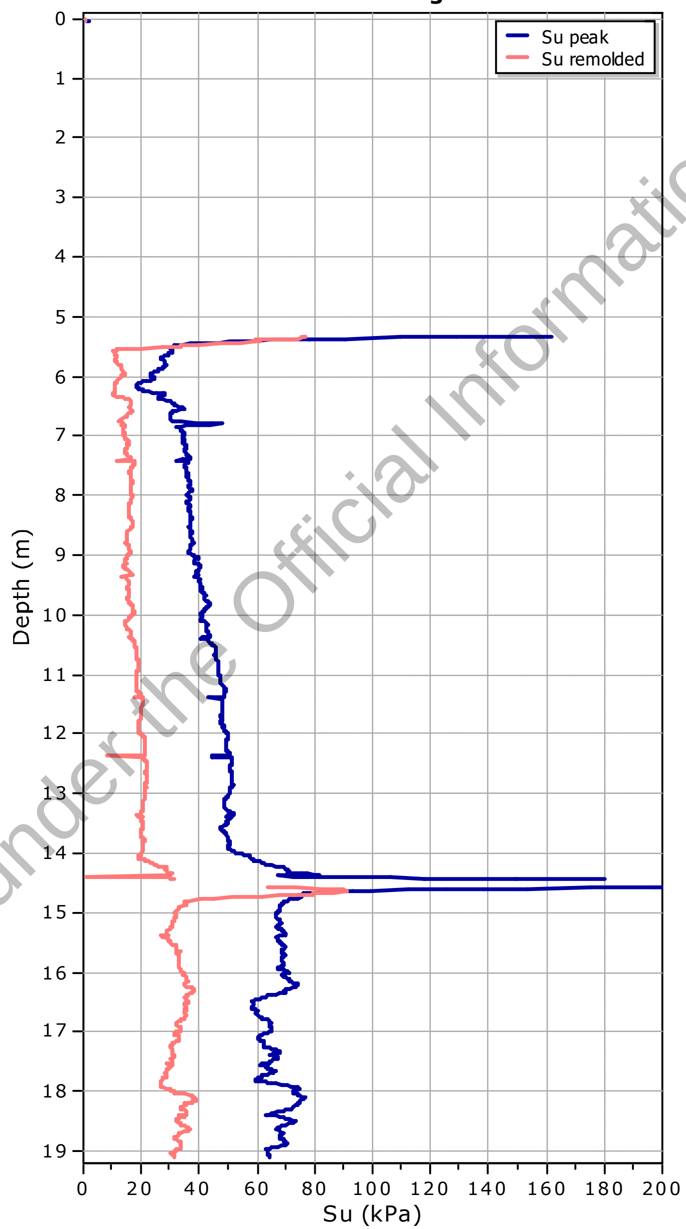
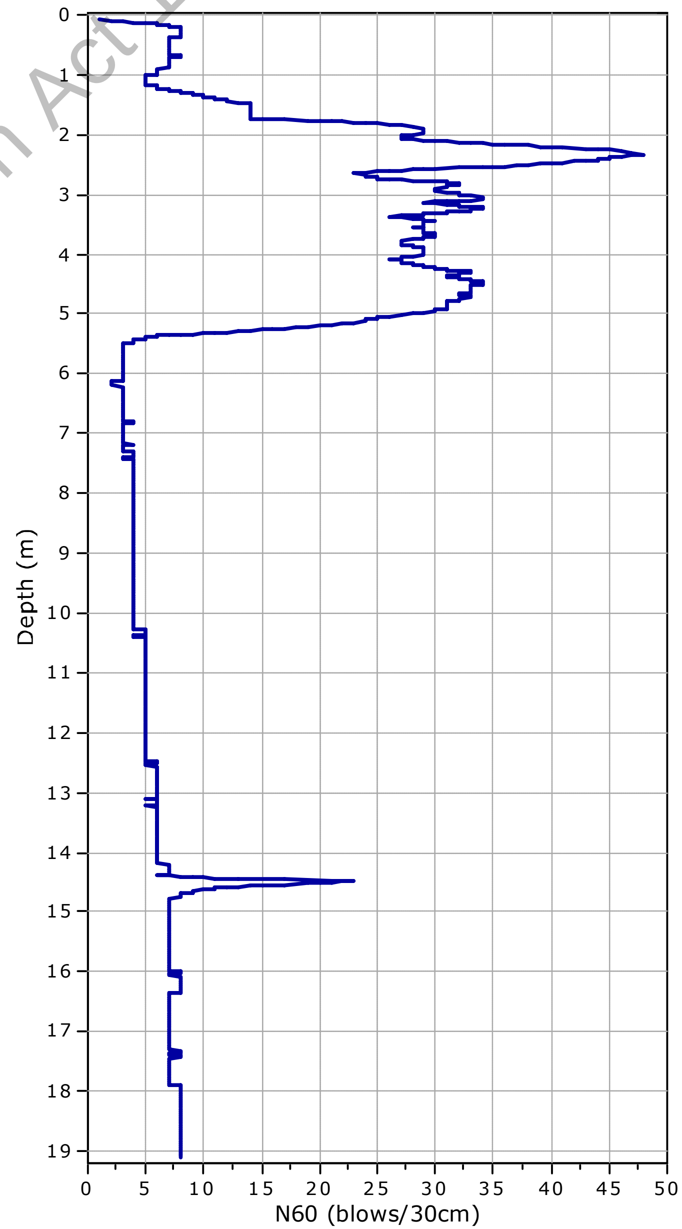
CPT: CPT-01
Total depth: 19.10 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: geotechnical Investigation
Location: Mangapapa School







LDE Ltd
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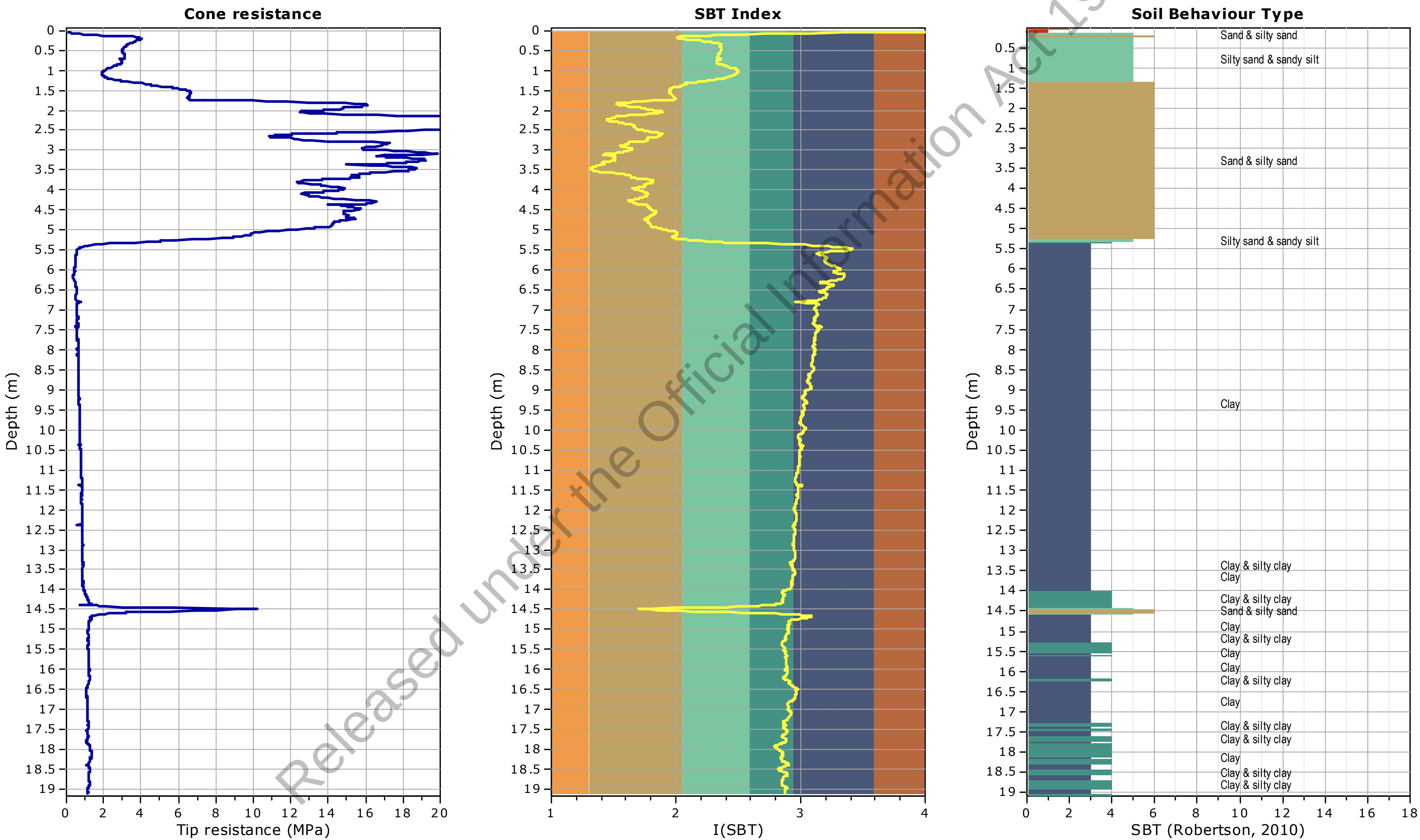
CPT: CPT-01
Total depth: 19.10 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: geotechnical Investigation
Location: Mangapapa School



LDE Ltd
33 Grey Street
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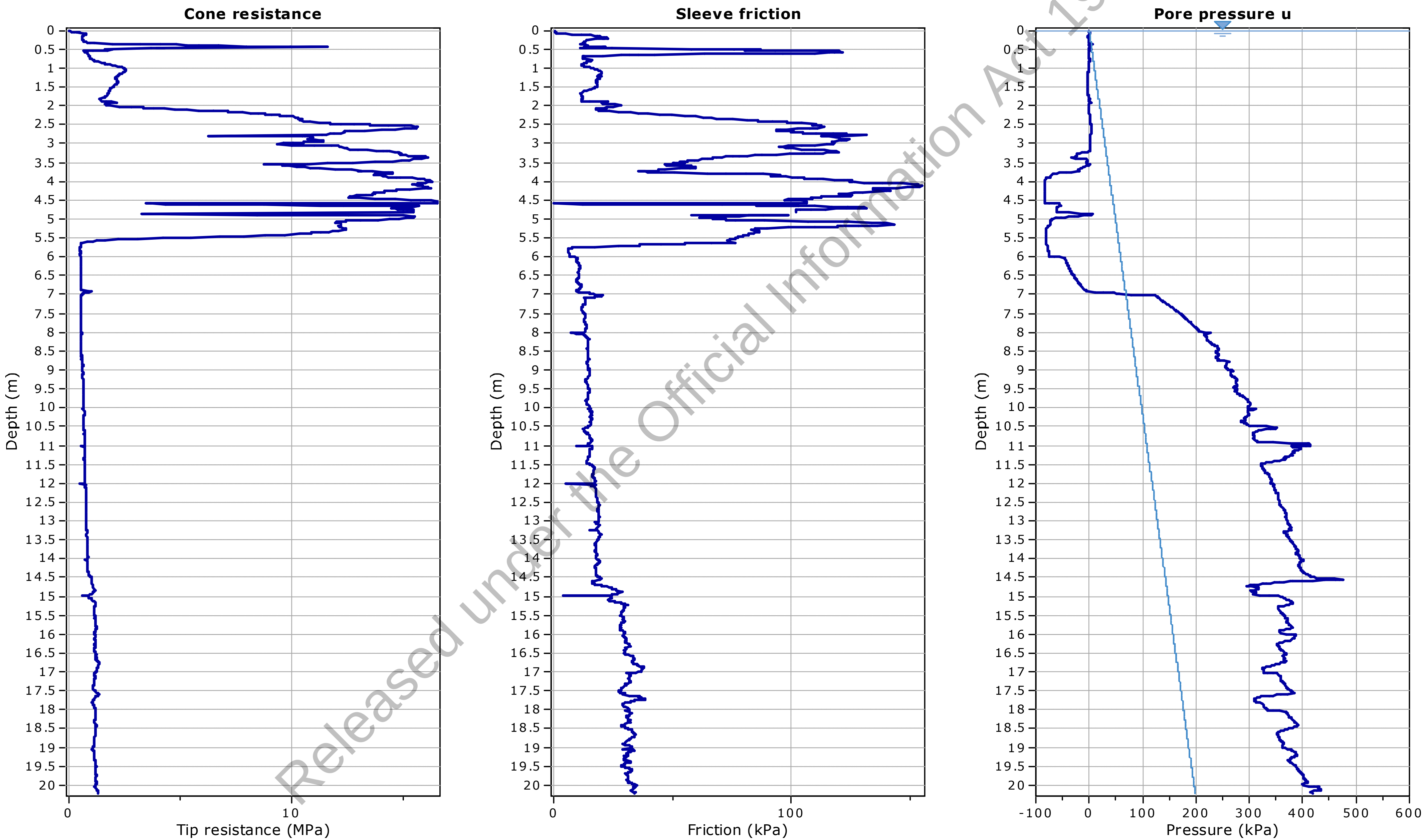
CPT: CPT-02
Total depth: 20.22 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechical

Project: **geotechnical Investigation**
Location: **Mangapapa School**



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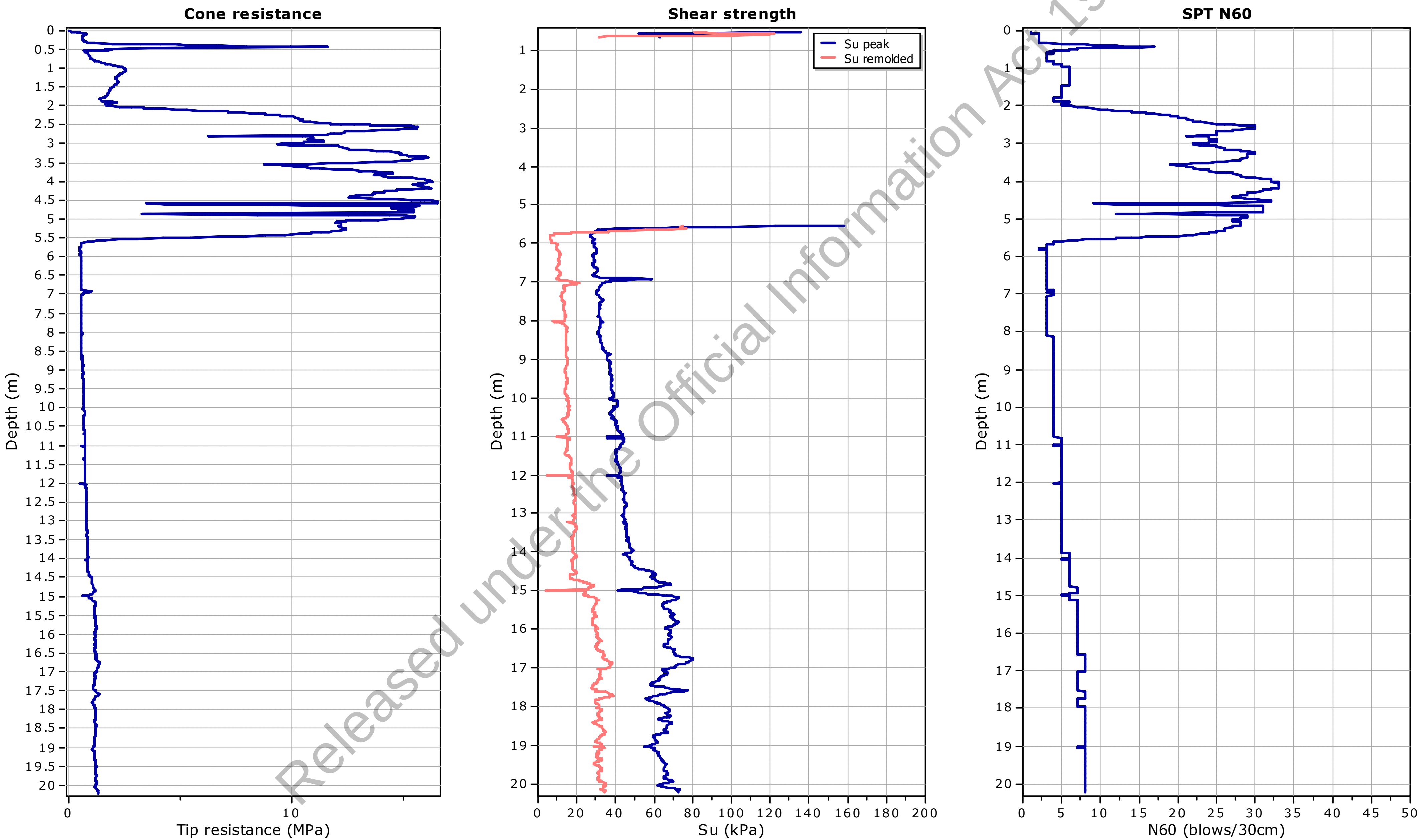
CPT: CPT-02
Total depth: 20.22 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechical

Project: **geotechnical Investigation**
Location: **Mangapapa School**



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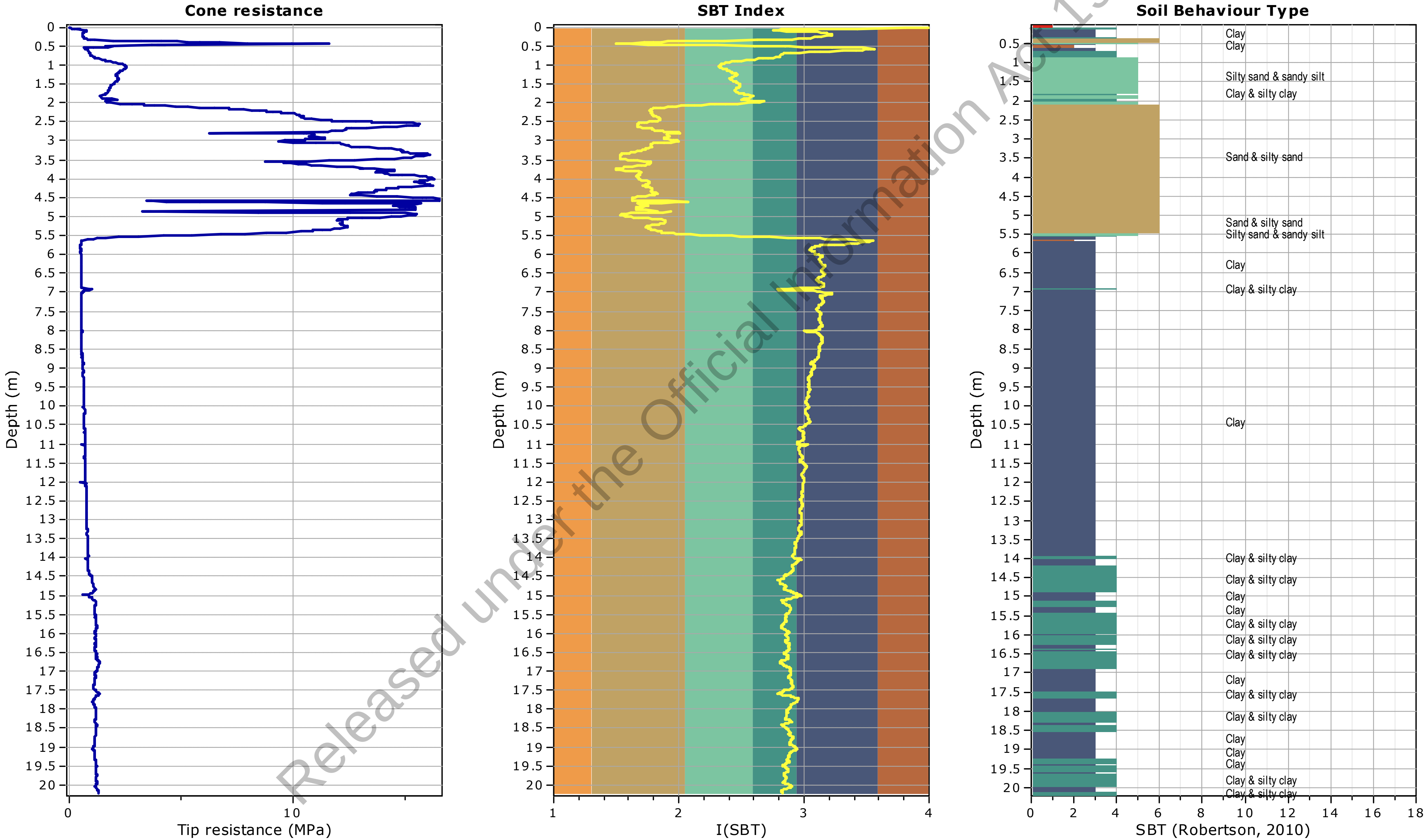
Project: **geotechnical Investigation**
Location: **Mangapapa School**

CPT: CPT-02
Total depth: 20.22 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechical



LDE Land Development & Engineering Ltd.

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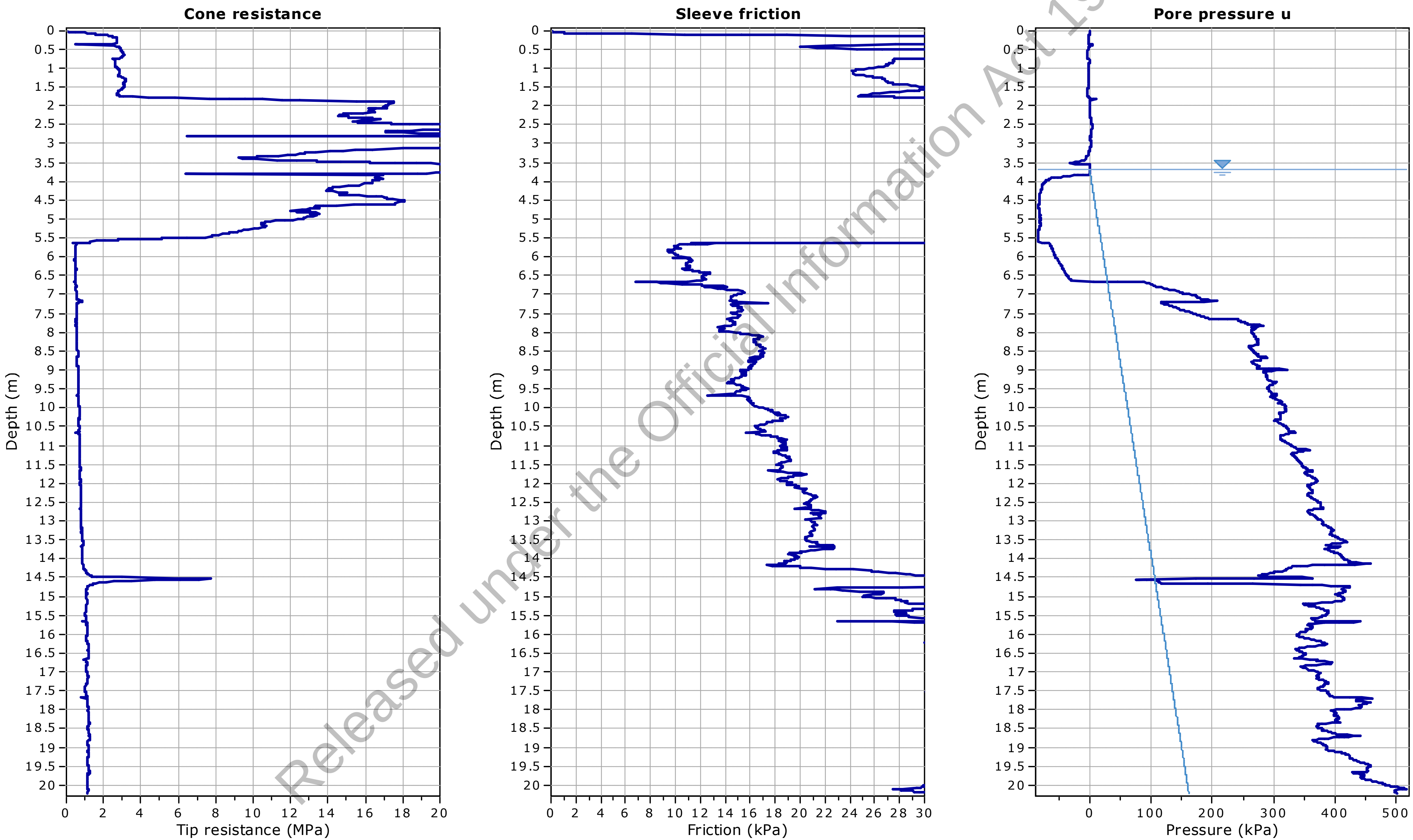
CPT: CPT-03
Total depth: 20.22 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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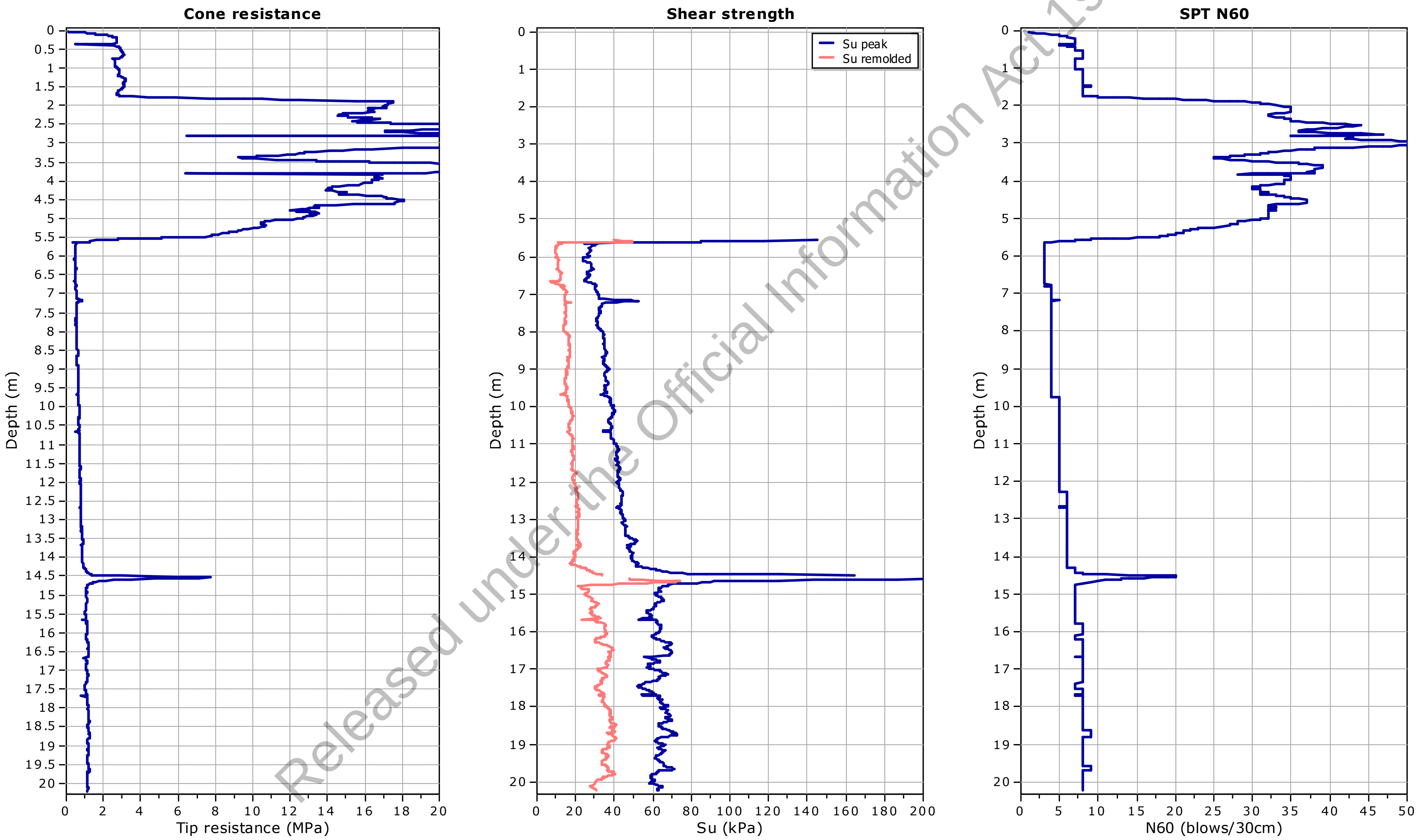
CPT: CPT-03
Total depth: 20.22 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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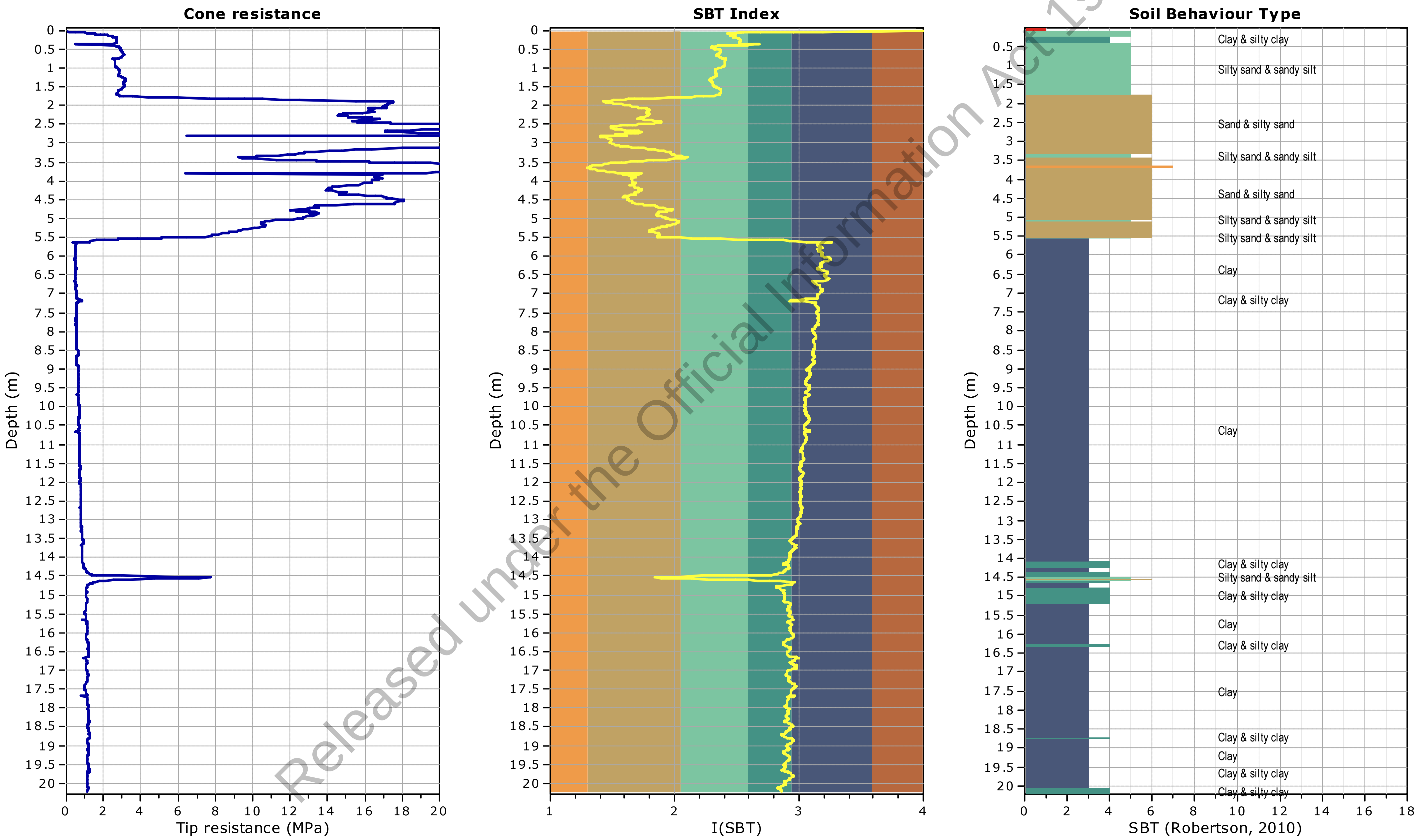
Project: Geotecnical Investigation
Location: Mangapapa School

CPT: CPT-03
Total depth: 20.22 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical



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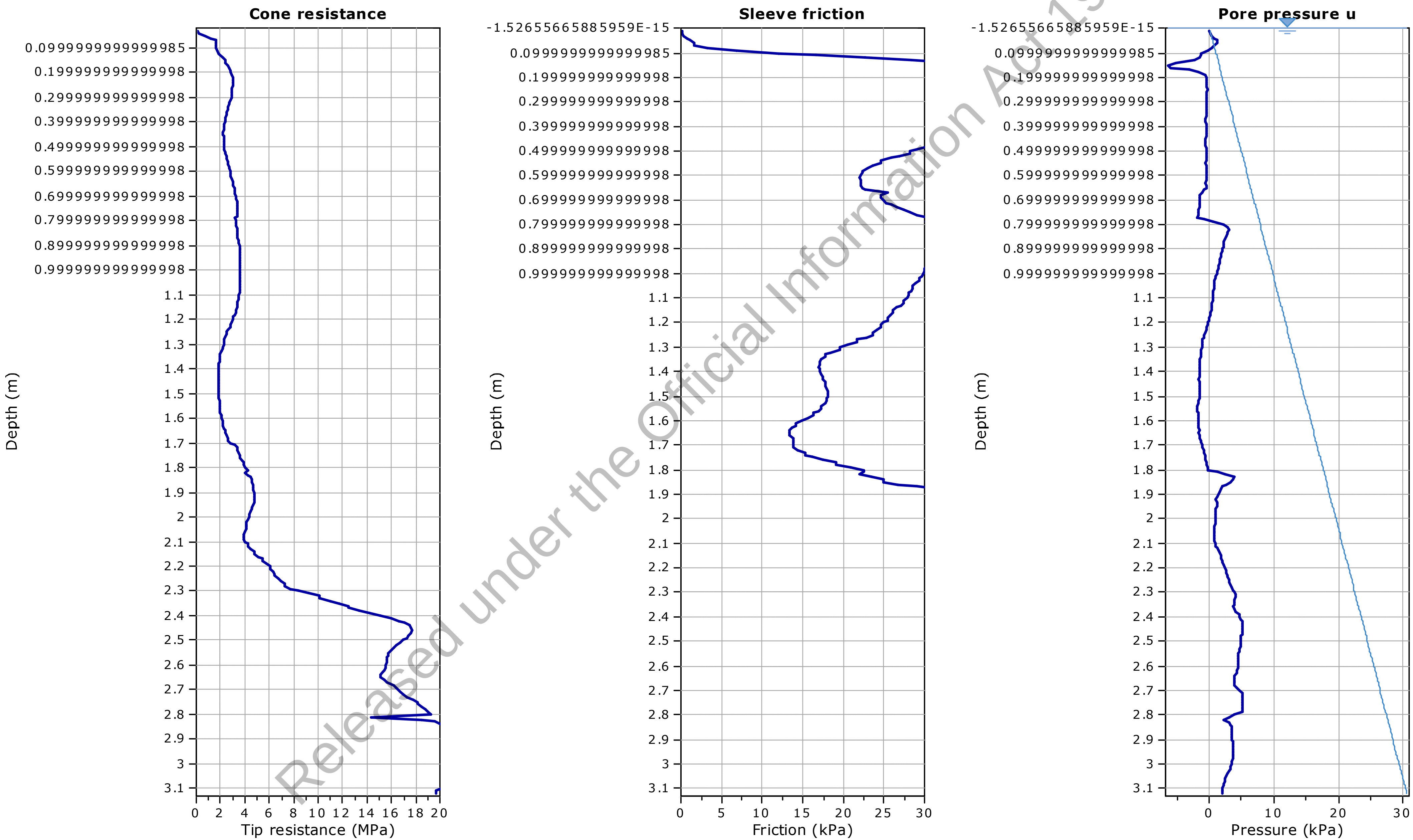
CPT: CPT-04
Total depth: 3.12 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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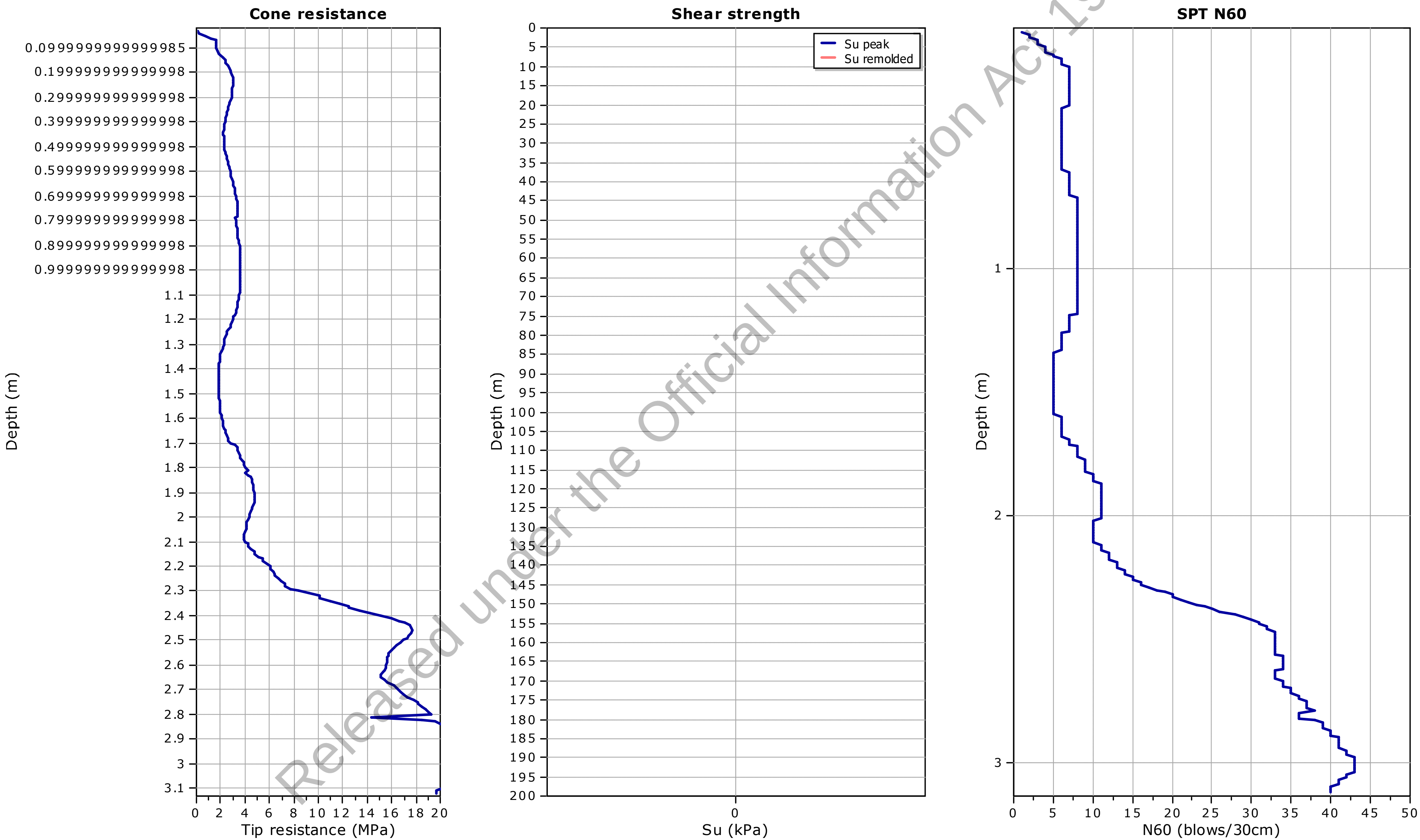
CPT: CPT-04
Total depth: 3.12 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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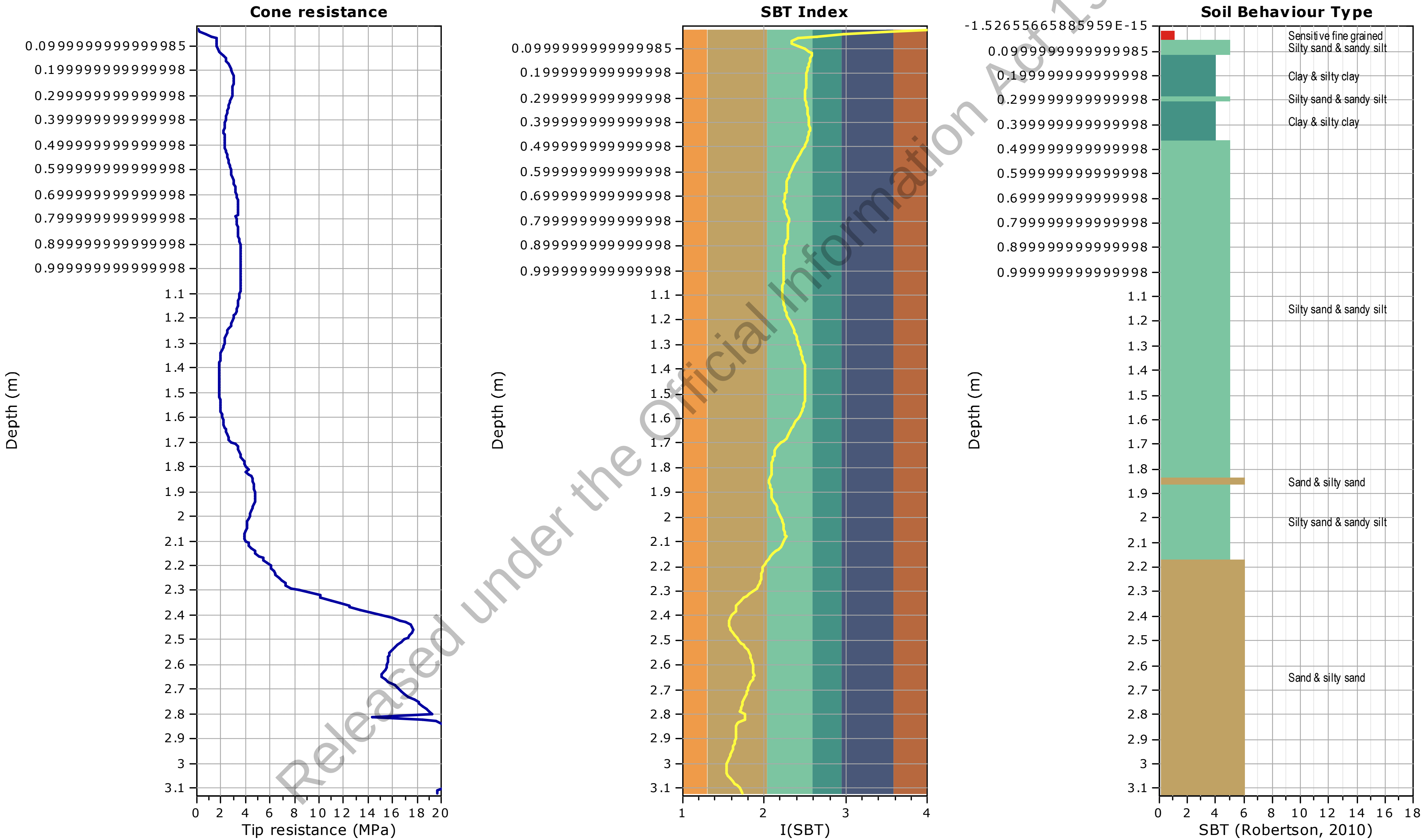
CPT: CPT-04
Total depth: 3.12 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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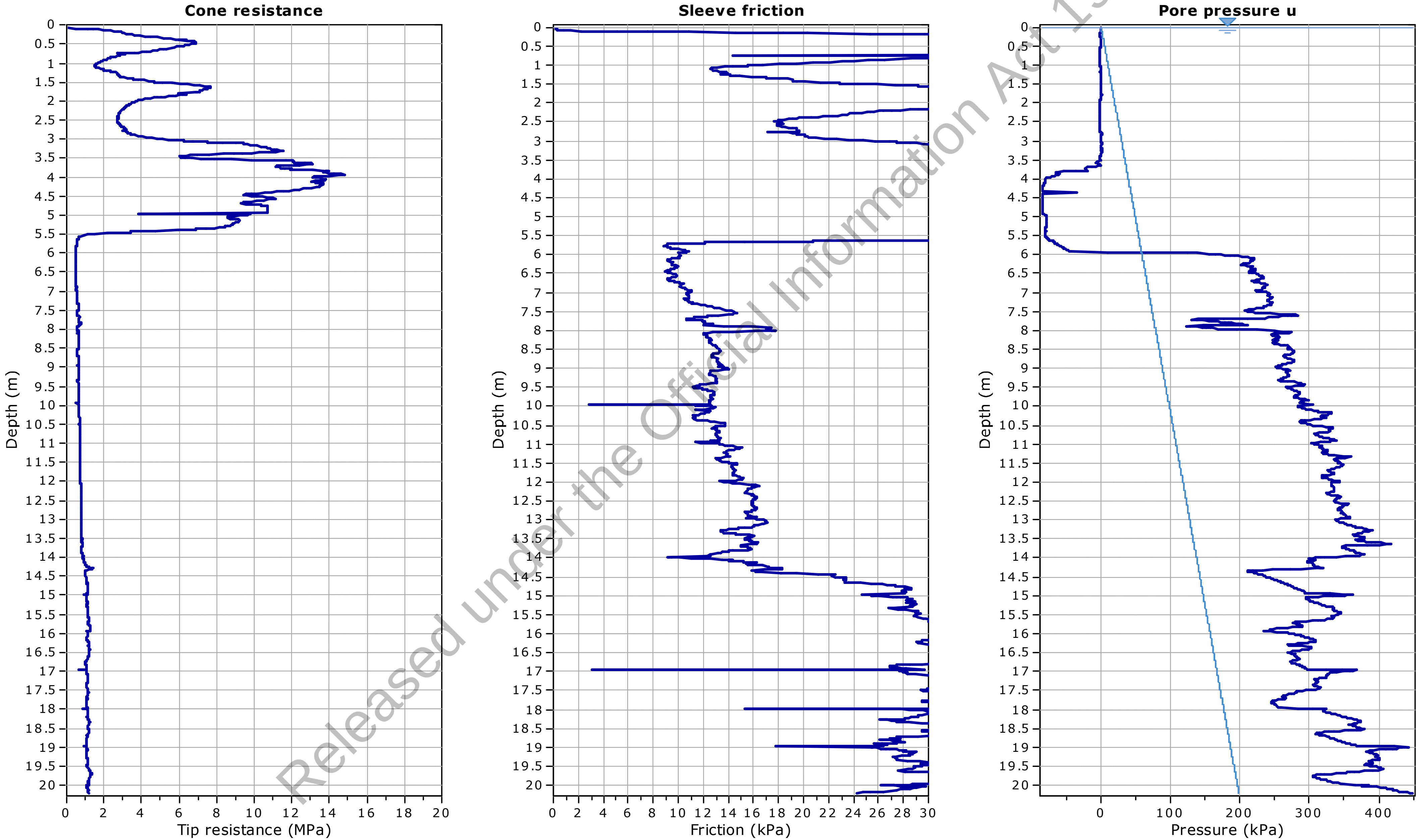
CPT: CPT-05A
Total depth: 20.21 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: Geotecnical Investigation
Location: Mangapapa School



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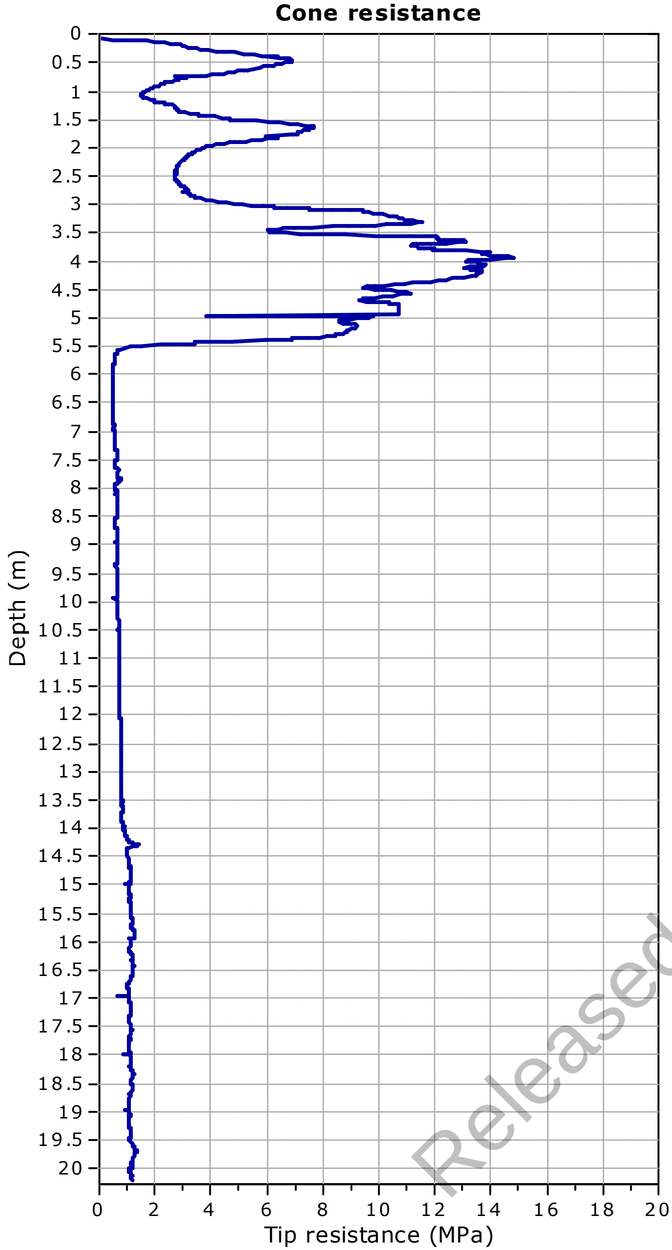
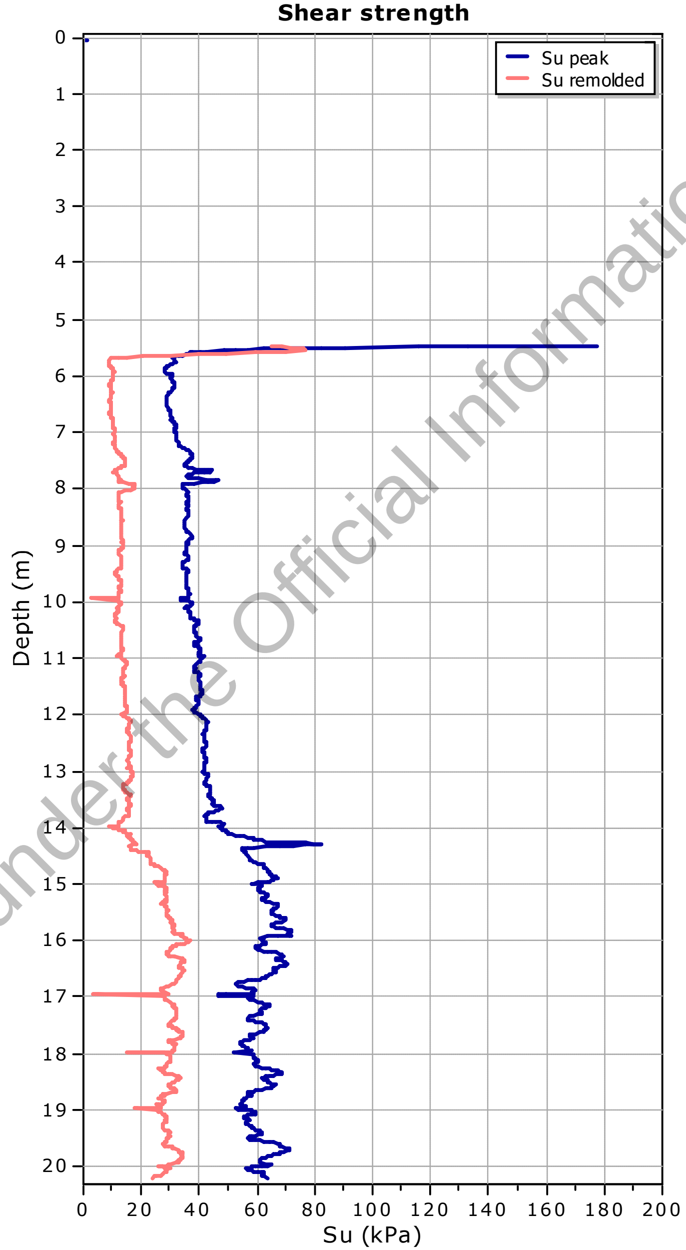
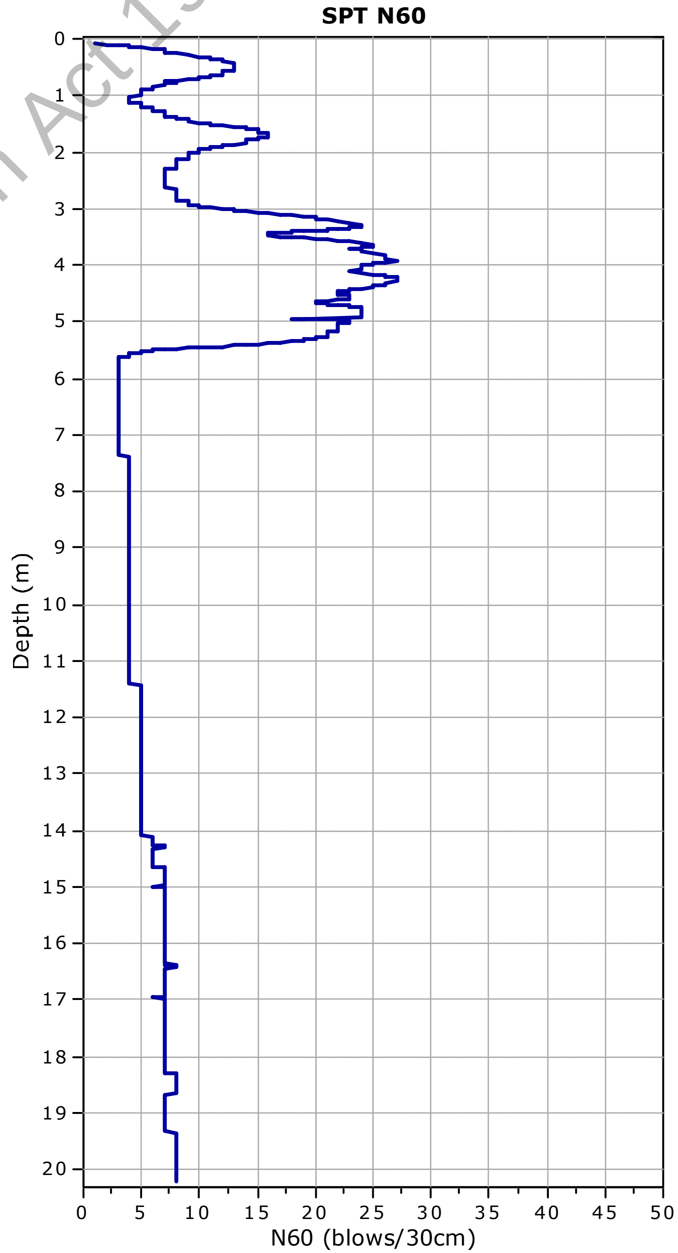
CPT: CPT-05A
Total depth: 20.21 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**







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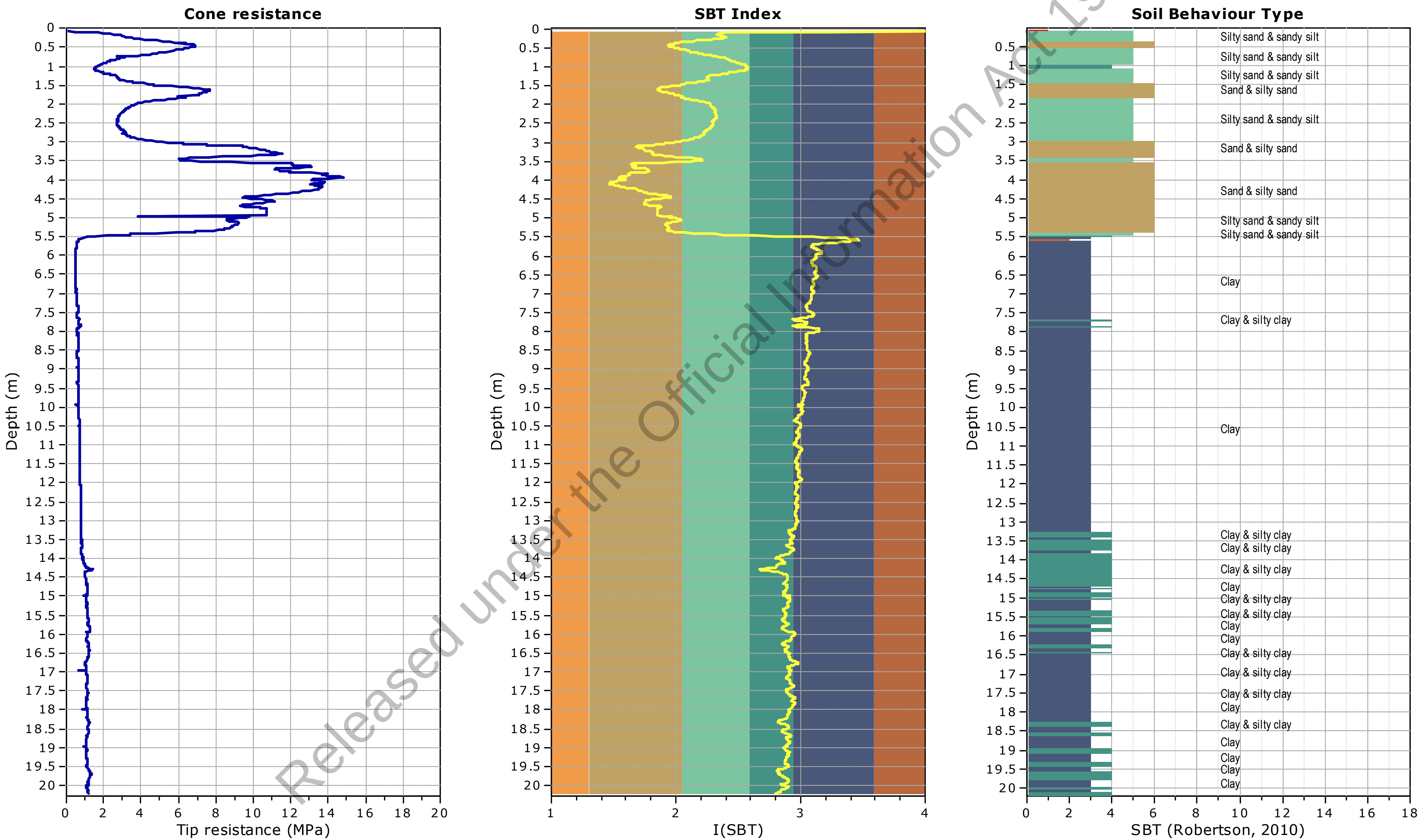
CPT: CPT-05A
Total depth: 20.21 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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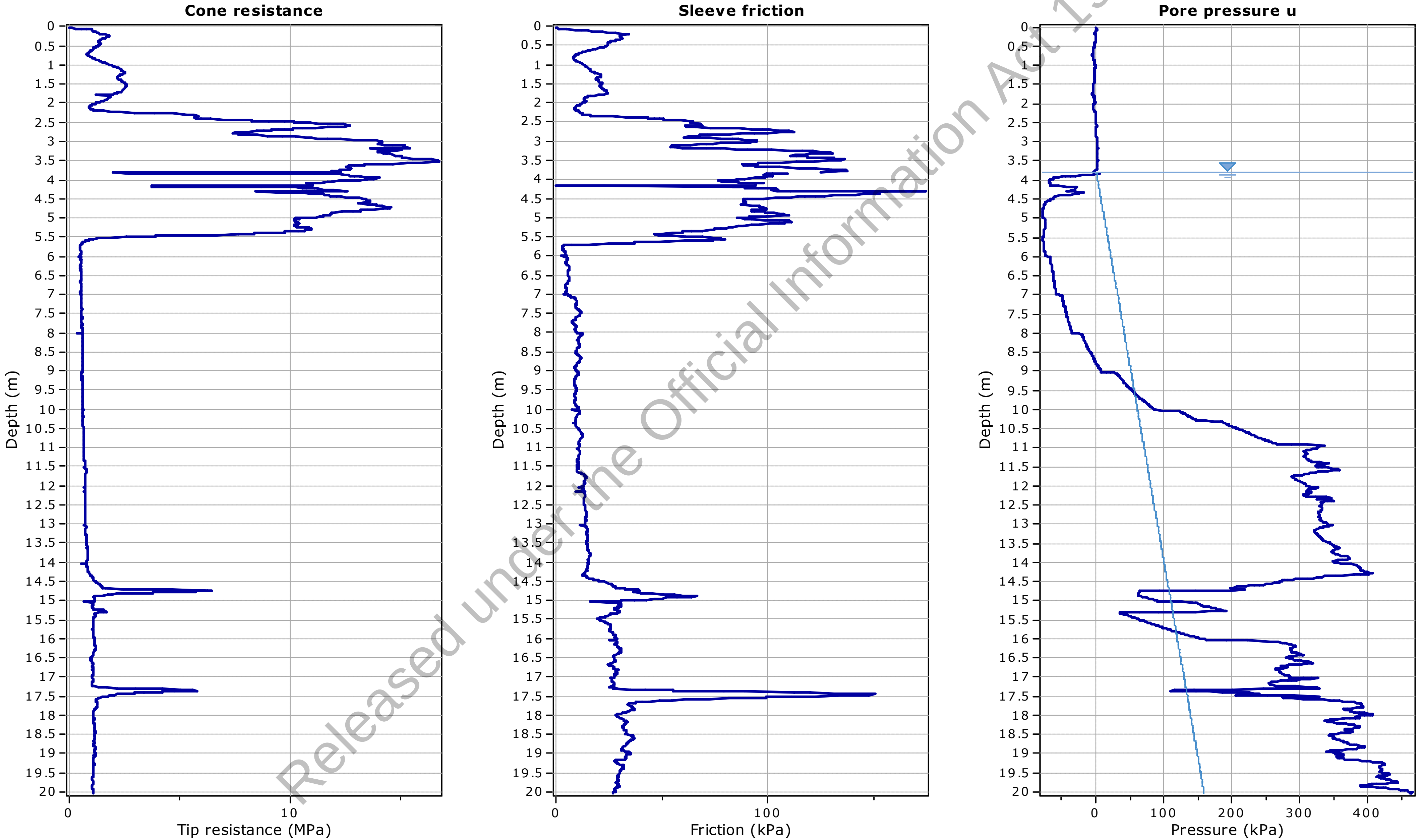
CPT: CPT-06
Total depth: 20.04 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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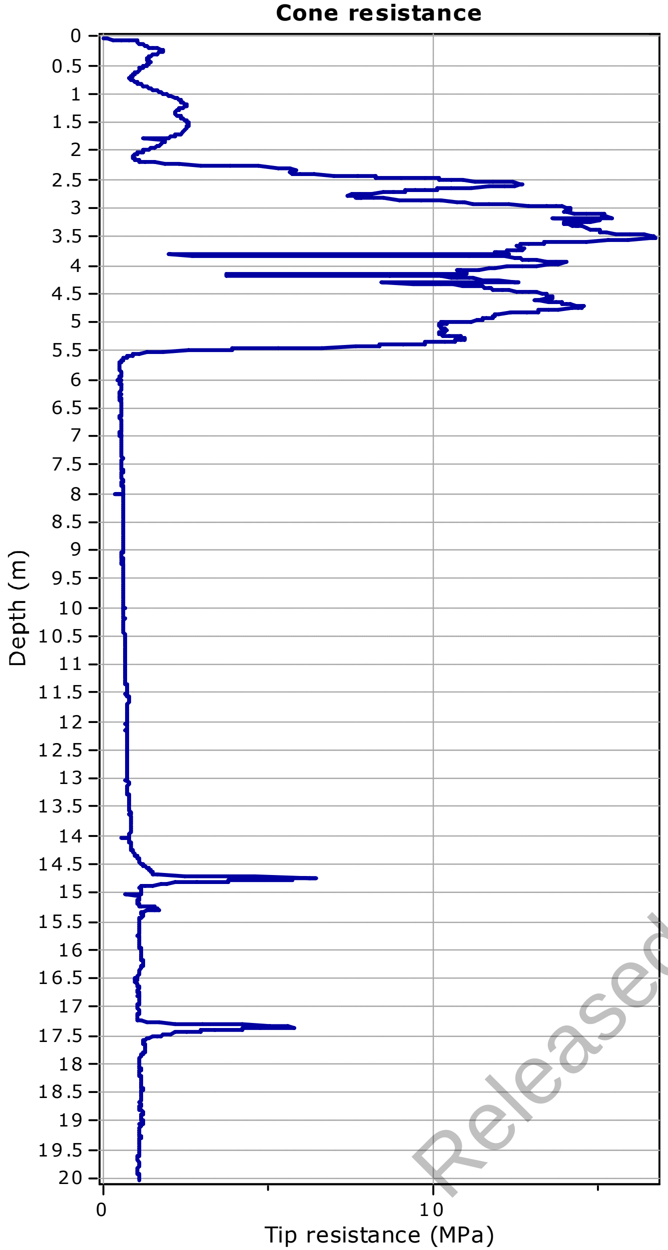
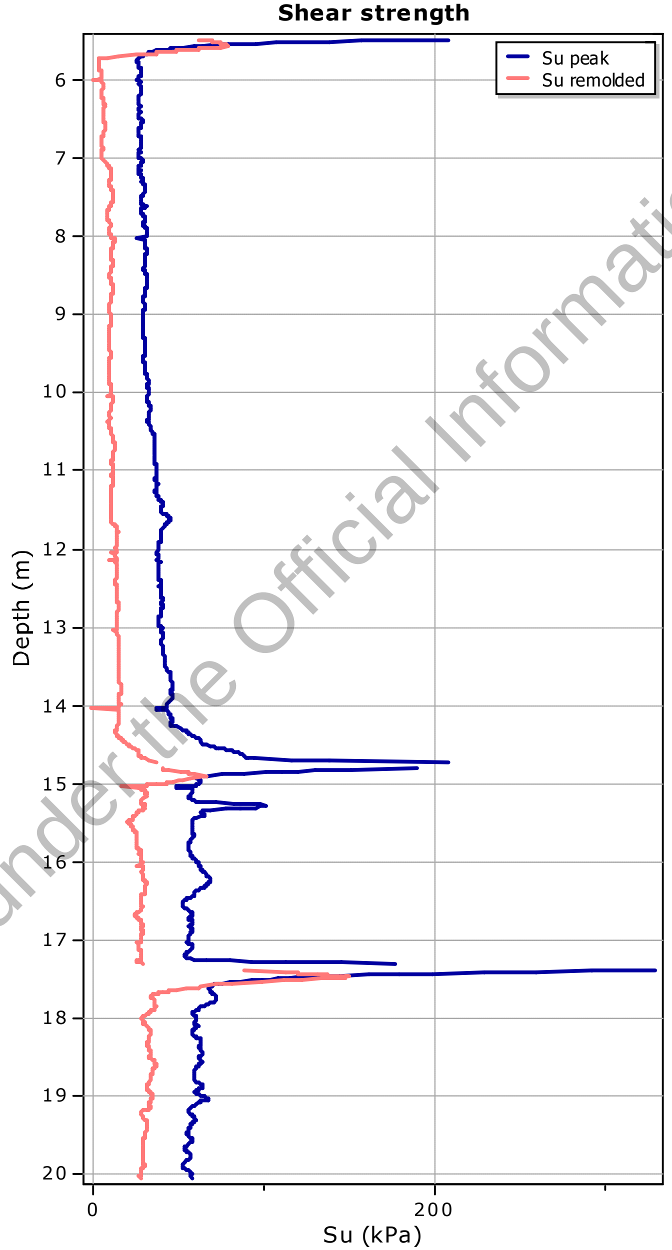
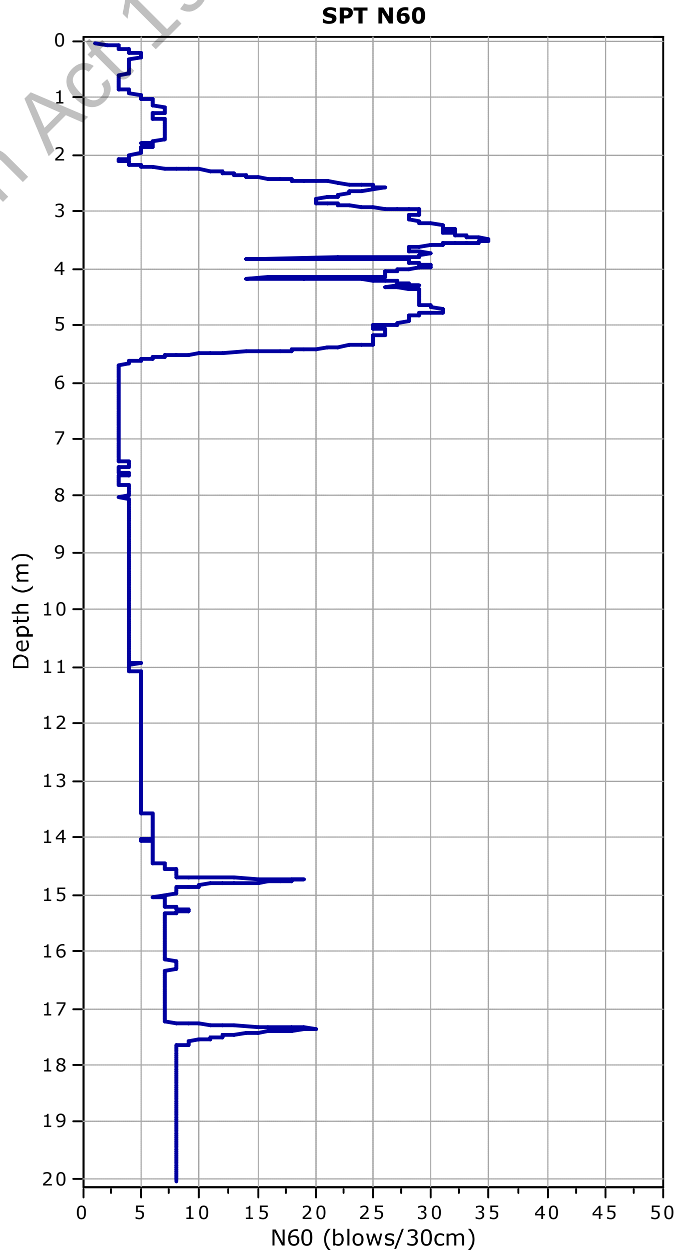
CPT: CPT-06
Total depth: 20.04 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: Geotecnical Investigation
Location: Mangapapa School







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33 Grey Street
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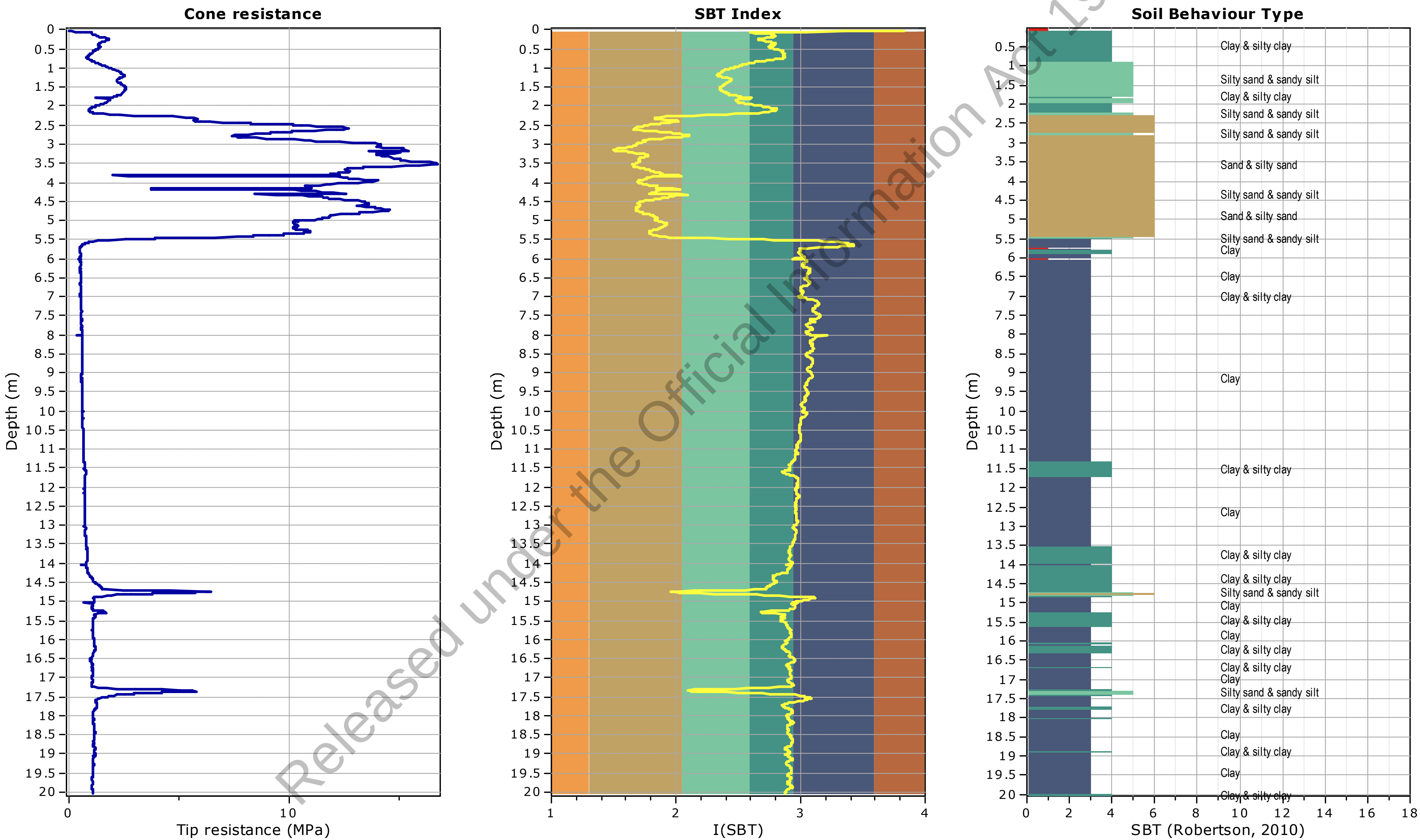
CPT: CPT-06
Total depth: 20.04 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: Geotecnical Investigation
Location: Mangapapa School



LDE Ltd
33 Grey Street
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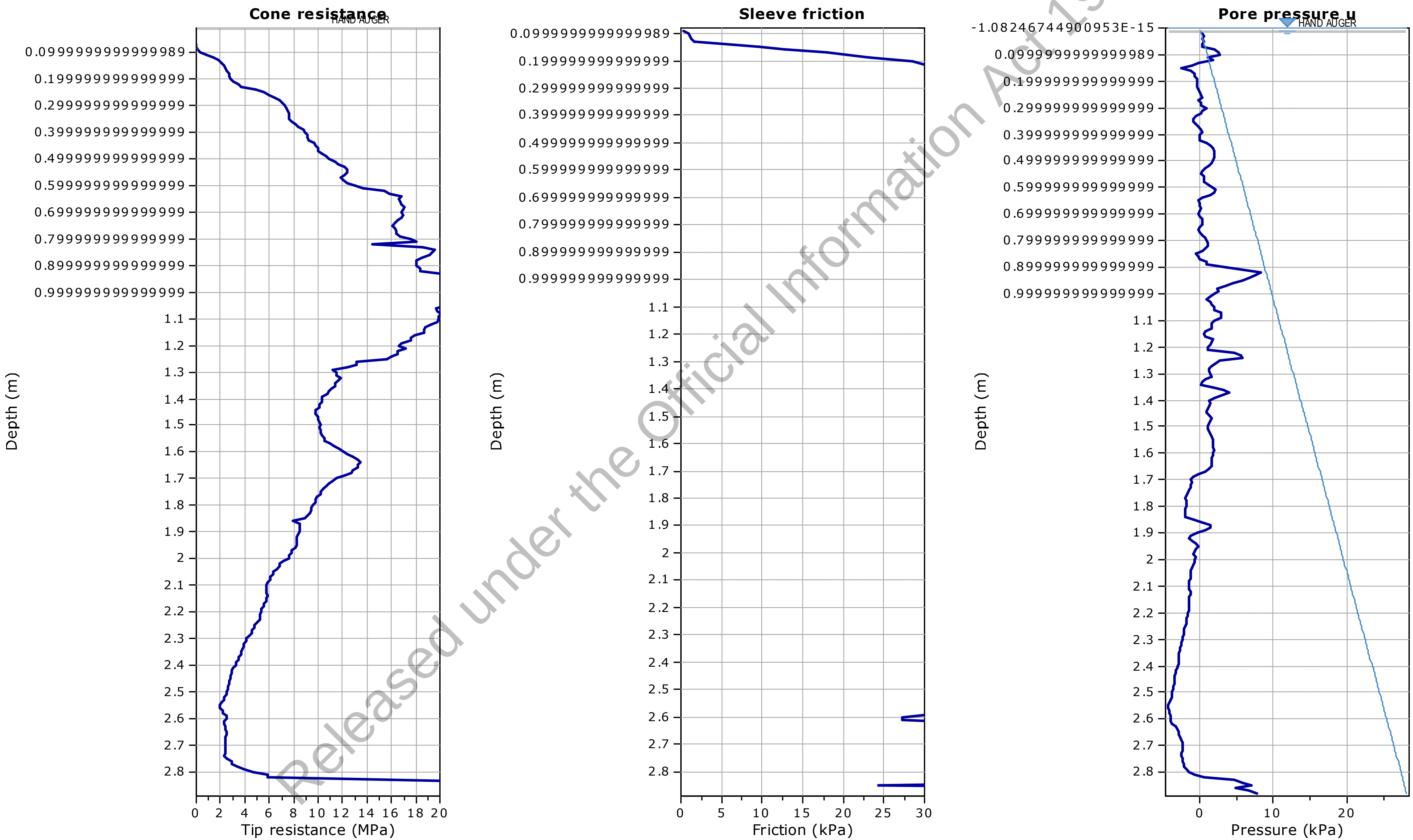
CPT: CPT-07

Total depth: 2.88 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



LDE Ltd
33 Grey Street
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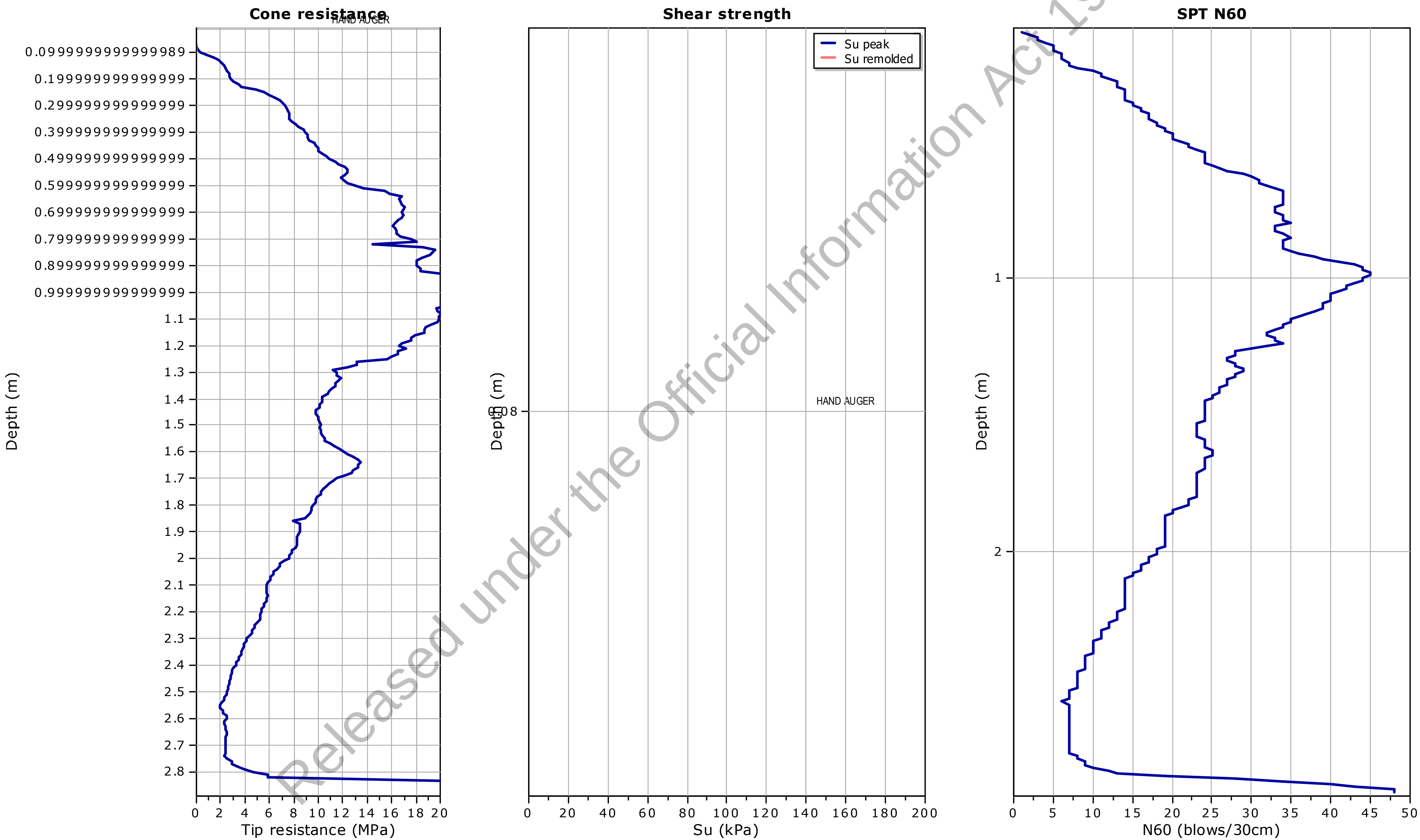
CPT: CPT-07
Total depth: 2.88 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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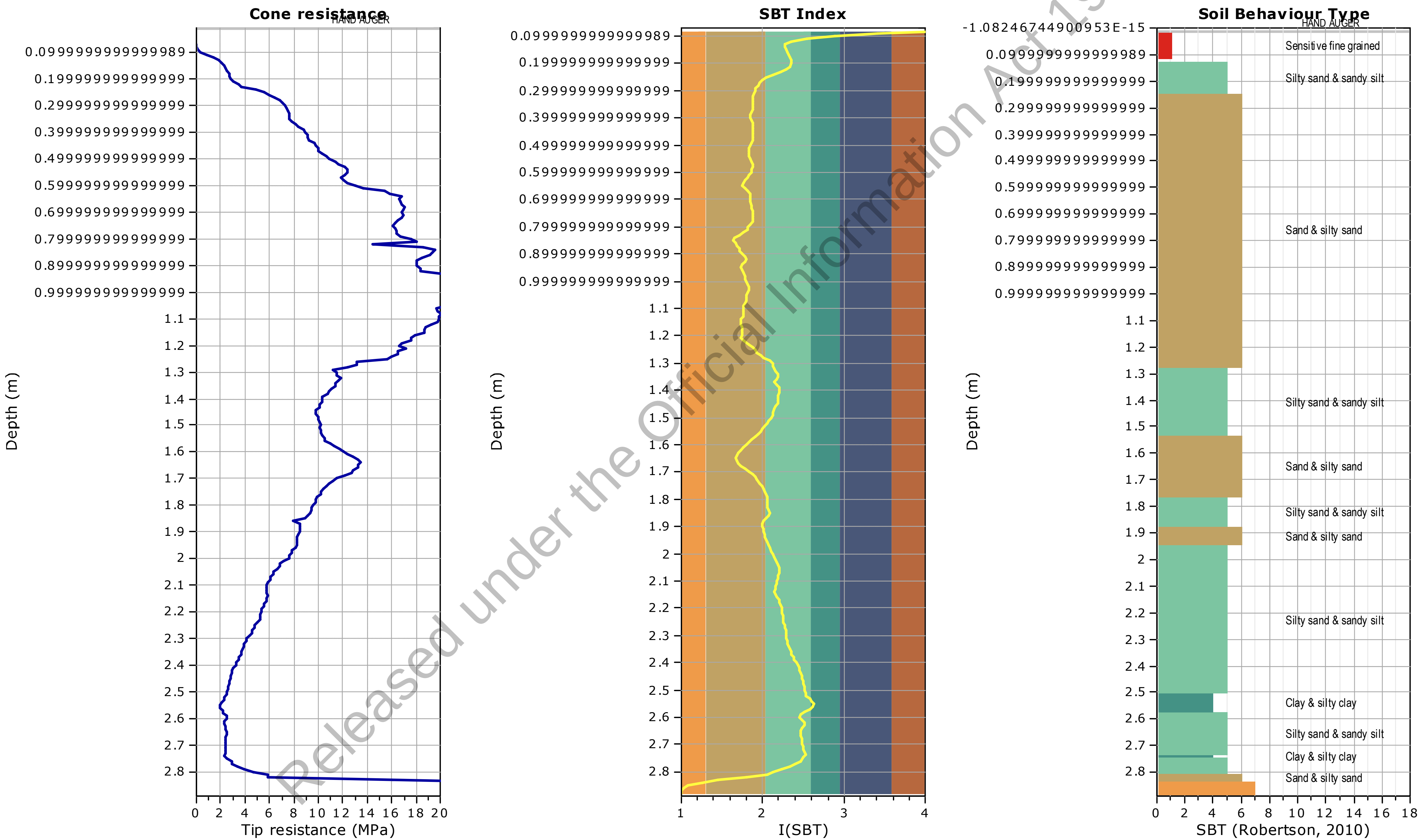
CPT: CPT-07
Total depth: 2.88 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Piezo Cone
Cone Operator: Explore Geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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33 Grey Street
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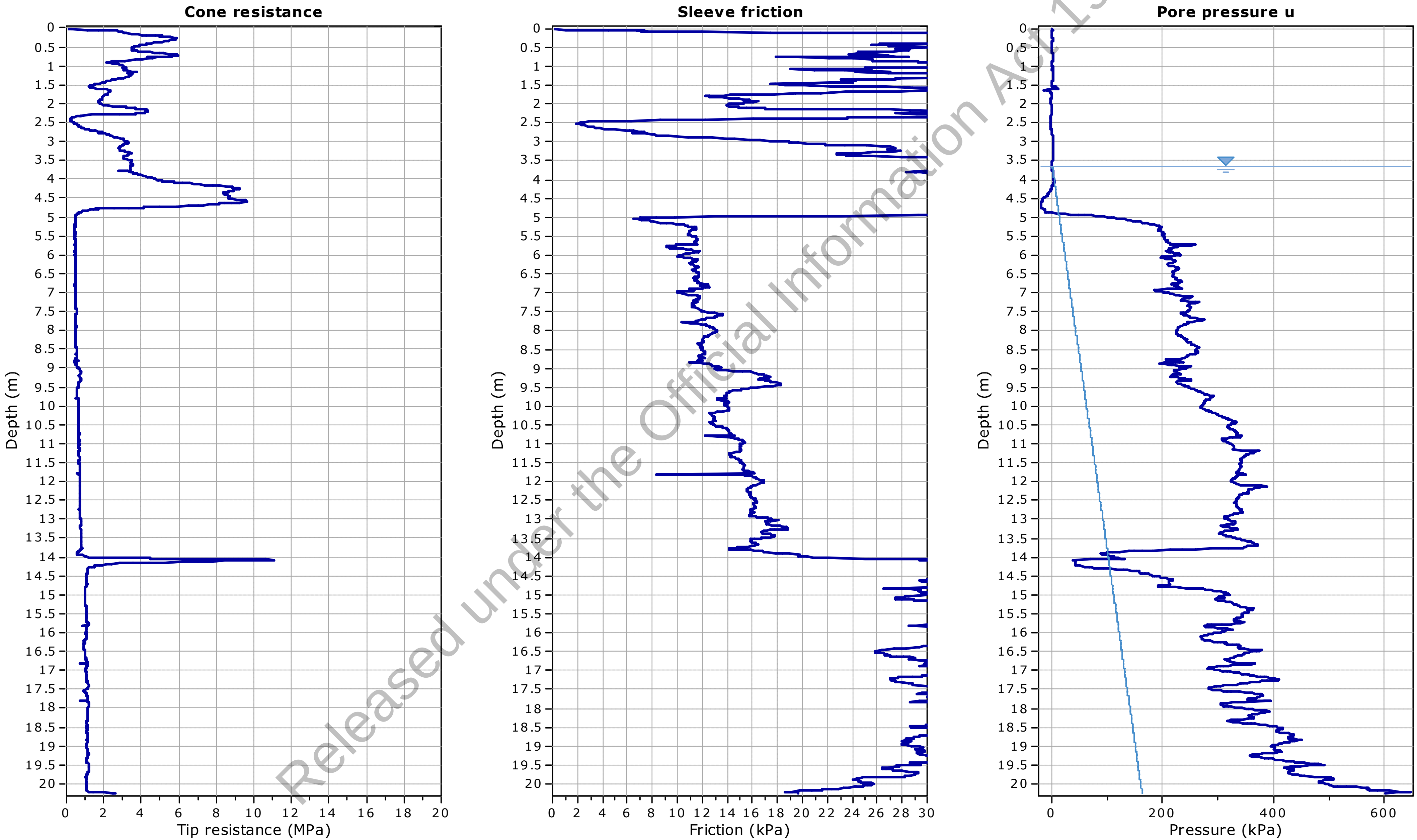
CPT: CPT-07A
Total depth: 20.25 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: piezo Cone
Cone Operator: Explore geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**



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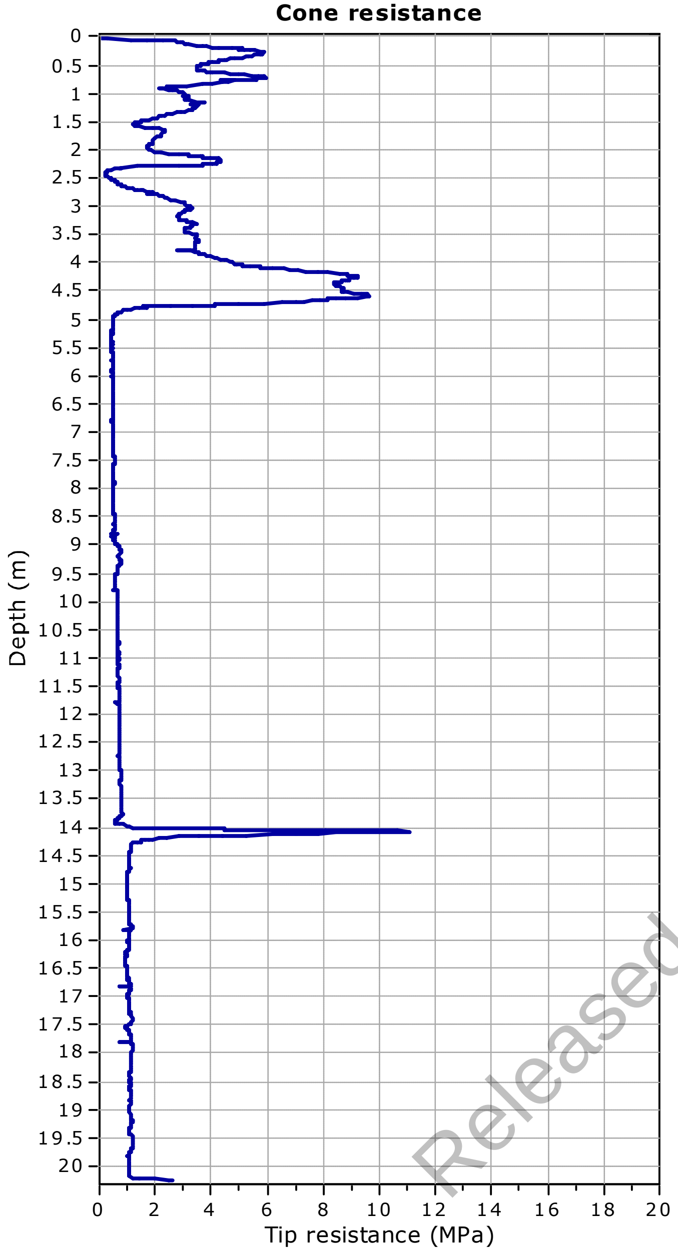
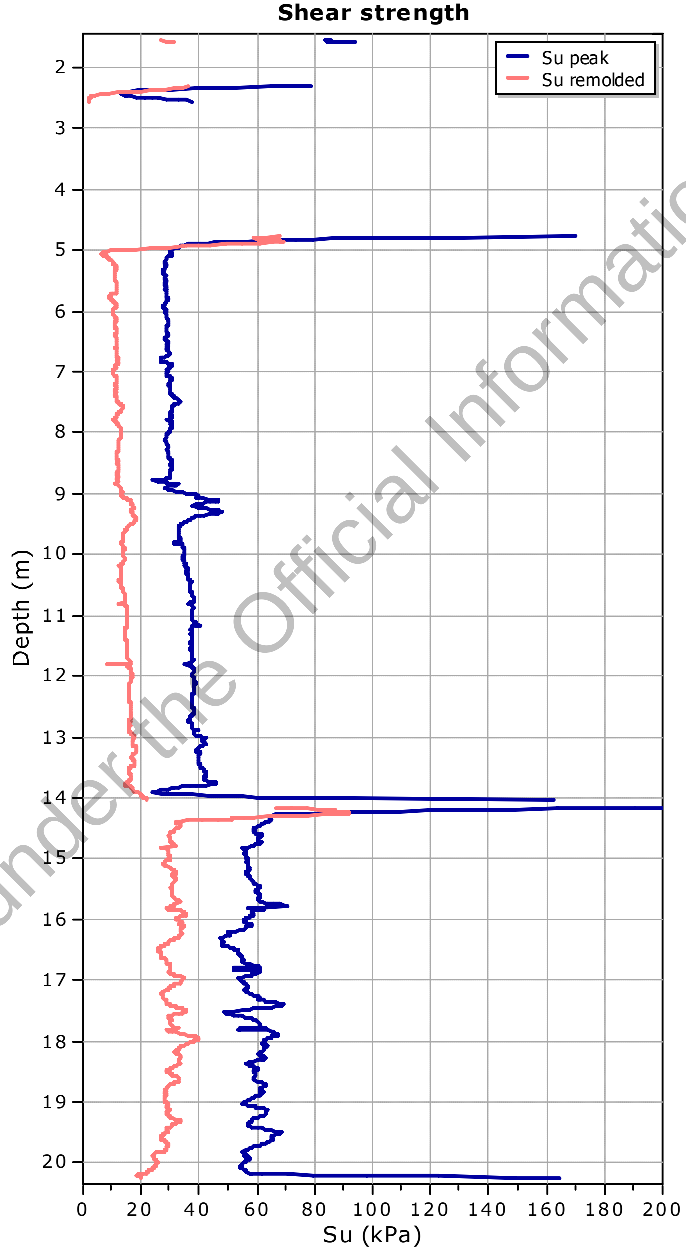
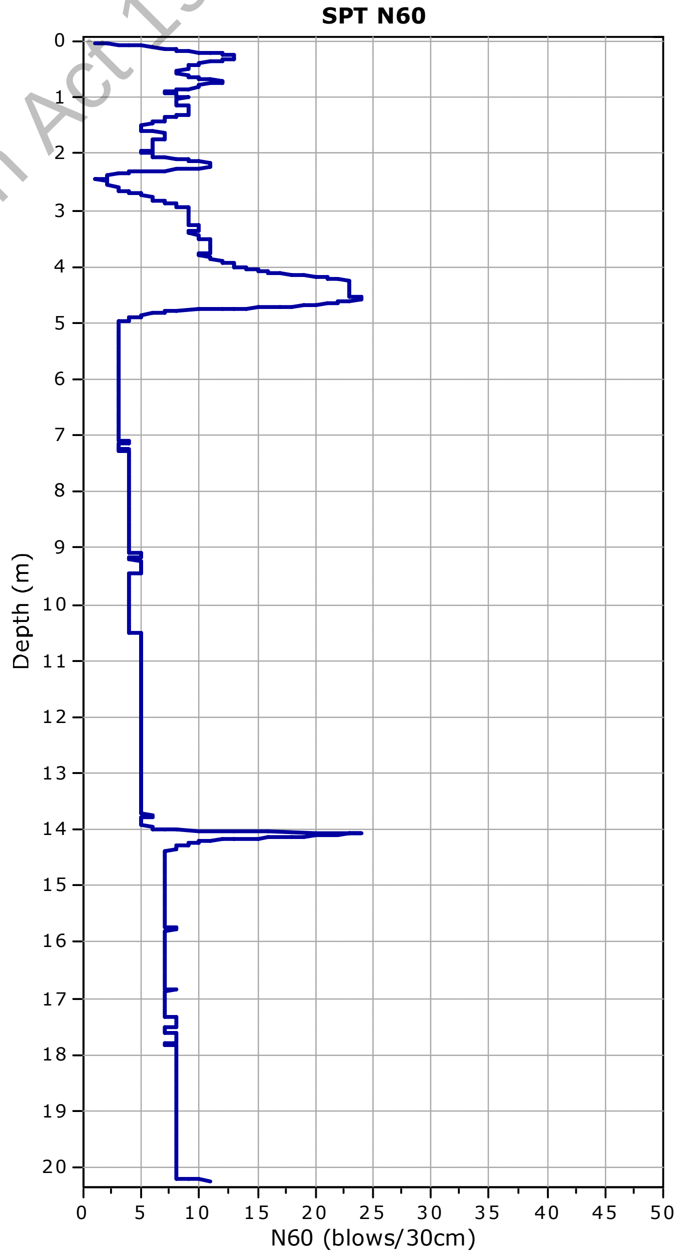
CPT: CPT-07A
Total depth: 20.25 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: piezo Cone
Cone Operator: Explore geotechnical

Project: **Geotecnical Investigation**
Location: **Mangapapa School**







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Project: **Geotecnical Investigation**
Location: **Mangapapa School**

CPT: CPT-07A
Total depth: 20.25 m, Date: 12/07/2019
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: piezo Cone
Cone Operator: Explore geotechnical