REPORT TO

SCHOOL INFRASTRUCTURE NSW

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED FLOOD RECOVERY REBUILD

AT

LISMORE SOUTH PUBLIC SCHOOL,
69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Date: 6 June 2025
Ref: 36310LTrpt-rev3

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1 INTRODUCTION

1.1 Client Supplied Introduction

This geotechnical report has been prepared to support a Review of Environmental Factors (REF) for the rebuild of Lismore South Public School (the activity). We understand that the purpose of the REF is to assess the potential environmental impacts of the activity prescribed by State Environmental Planning Policy (Transport and Infrastructure) 2021 (T&I SEPP) as “development permitted without consent” on land carried out by or on behalf of a public authority under Part 5 of the Environmental Planning and Assessment Act 1979 (EP&A Act). The activity is to be undertaken pursuant to Chapter 3, Part 3.4, Section 3.37 of the T&I SEPP.

The activity will be carried out at Lismore South Public School (LSPS) located 69-79 Kyogle Street, South Lismore (the site).

The purpose of this report is to provide comments and recommendations on site classification, earthworks, footings and pavement design.

The site, located at 69-79 Kyogle Street, South Lismore, consists of two separate land parcels situated on either side of Wilson Street. The proposed activity will be undertaken on the eastern parcel, where most of the school's existing structures are located. The western parcel contains sports fields and temporary learning facilities. Plate 1 outlines the school’s boundary, covering approximately 2.5 hectares. Due to flood damage, the existing buildings on the eastern parcel are currently unused, and students are temporarily using facilities on the sports field and oval, located on the western side of Wilson Street, adjacent to the primary school.

Plate 1 Aerial image of site (Source: Nearmap)

1.2 Introduction

This report presents the results of a geotechnical investigation for the proposed flood recovery rebuild at Lismore South Public School (LSPS), 69-79 Kyogle Street, South Lismore, NSW. The location of the site is shown in Figure 1. The geotechnical investigation was commissioned by the NSW Department of Education.

The proposed activity comprises the rebuild of the LSPS on the eastern parcel of the existing site, in South Lismore, and will be delivered in a single stage. The eastern parcel is bound by Phyllis Street, Wilson Street and Kyogle Street to the north, west and south respectively. The western parcel is out of the scope of this activity. Any works required on the western parcel (such as removal of demountable classrooms) will be subject to separate approval (if required). A detailed description of the proposal is as follows:

1. Retention of the existing play equipment, Building K and covered outdoor learning area (COLA) on the western parcel.
2. Bulk earthworks, comprising fill and excavation and other site preparation works on the eastern parcel.
3. Construction of a new building on the eastern parcel for LSPS including:
    a. A one storey building (with undercroft areas below) fronting Kyogle Street containing a general learning space (GLS) hub, hall, library, support hub, administration, and pre-school.
    b. Undercroft outdoor learning areas as well as amenities and storage located on ground level.
4. Landscaping and public domain works, including tree planting, a games court in the northeast corner and an outdoor playing area adjacent to the preschool.
5. A car park on the eastern side of the site, with access from Kyogle Street.
6. Waste collection area access from Kyogle Street.
7. Multiple entrance points, including:
    a. Primary and secondary entries distributed on site frontages.
    b. Vehicular access point to provide access to waste collection/delivery areas and car parking.
8. Ancillary public domain mitigation measures.

Plate 2 below shows the scope of works.

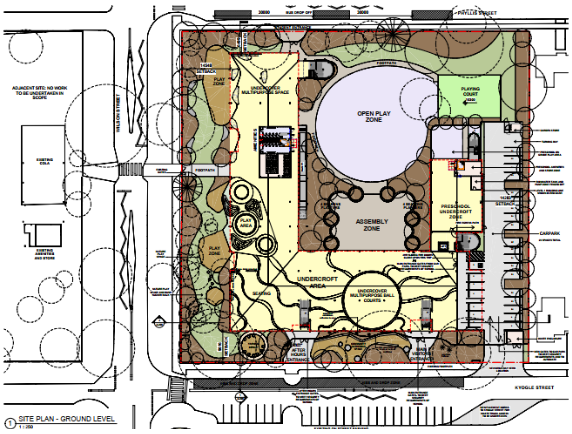

Plate 2 Proposed site plan (Source: EJE Architecture)

From the supplied civil markup prepared by TTW (Dated 4 October 2024) it appears that the proposed surface levels will be similar to the existing (i.e. the depth of cut or fill is not anticipated to generally be greater than about 0.5m from existing levels). From the structural drawings prepared by TTW, the single-storey building will be suspended above the ground surface with the undercroft areas to comprise pavements rather than ground floor slabs. We understand from discussion with TTW that column loads for the proposed building will be in the order of 800kN.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on site classification, earthworks, footings and pavement design.

This geotechnical investigation was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E36310PTrpt, for the results of the environmental site assessment.

2 INVESTIGATION PROCEDURE

The geotechnical investigation was carried out from 24 to 26 September 2024 and comprised the following:

- Seven Cone Penetration Tests (CPT3, CPT6, CPT15, CPT17, CPT18, CPT20 and CPT22), to depths ranging from 30.31m to 40.52m. The CPT probes were carried out to provide an assessment of the relative density and strength of the subsurface soils. CPT refusal occurred in five of the seven probes however CPT3 and CPT15 were terminated due to excessive inclination from the vertical and the limit of available rods being reached respectively.
- Six boreholes (BH2, BH11, BH15, BH19, BH21 and BH23) drilled using our track-mounted JK300 drilling rig to depths ranging from 3m to 6m. The boreholes were advanced using spiral auger techniques. Standard Penetration Tests (SPT) were carried out to assess the strength of the soils, augmented with hand penetrometer readings on cohesive soils recovered from the SPT split tube sampler.

The CPT method involves continuously pushing a 35mm diameter rod with a conical tip into the subsurface materials using hydraulic rams fitted to our truck mounted rig. Measurements of the end resistance on the cone tip and the frictional resistance on a separate sleeve, immediately behind the cone, are taken. A 'dummy cone' was initially used at each test location, to protect the test cone from damage due to obstructions in fill materials. The subsurface material identification, including the inferred 'possible' fill layer and the material strength/relative density, is by interpretation of the test results based on past experience, empirical correlations and correlations with geotechnical information from the geotechnical boreholes carried out nearby. We note that the CPT does not provide sample recovery. Piezocones were completed at CPT3, CPT15 and CPT22 to measure pore pressures within the soils to provide an indication of the likely groundwater level at the time of testing. At the remaining CPT locations, testing was completed using a standard cone which does not measure pore pressures.

The test locations, as shown on Figure 2, were measured using a differential GPS unit. The approximate surface reduced levels, as shown on the borehole and CPT logs, were interpolated from spot heights and contours on the survey plan prepared by Beveridge Williams (Project No. 2202780, Drawing No. DET-007, Revision A dated 10 March 2022). The height datum is the Australian Height Datum (AHD).

Groundwater levels were interpreted from the CPT results where a piezocone was used. Groundwater observations were made during and on completion of augering in the boreholes. Standpipes were installed in BH2, BH7 and BH23 to allow for longer term groundwater monitoring. Groundwater levels were measured on the completion of the fieldwork and during a return visit to site on 15 October 2024. Groundwater level measurements are shown on the borehole logs. Longer-term, continuous monitoring of groundwater levels with data loggers was outside the agreed scope of our engagement.

The investigation was carried out in the full-time presence of our geotechnical engineer, Mr Keagen Rousseau, who was present on-site full time during the CPT probing and borehole drilling, and nominated sampling and testing locations and prepared logs of the strata encountered. The borehole and CPT logs, which include field test results and groundwater measurements/inferred levels, are attached to the report together with our Report Explanation Notes, which further describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd (Envirolab), both NATA accredited laboratories, for laboratory testing. STS completed moisture content, Atterberg Limits, linear shrinkage, shrink-swell index and four-day soaked CBR tests which are summarised in the attached STS Tables A, B and C. Envirolab completed a suite of soil aggression testing comprising pH, sulphate content, chloride content and resistivity. The results of the soil aggression testing are presented in the attached Envirolab Certificate of Analysis No. 363781.

3 RESULTS OF INVESTIGATION

3.1 Site History

A review of the historical aerial imagery obtained by JKE, a selection of which are attached as Appendix A, and of historical records indicates that part of the site has been in use as a school since 1915. Two buildings, which are likely school buildings, are visible within the site in the 1942 image.

The 1958 image shows relatively similar conditions to 1942, except two residences appear to have been constructed in the north-eastern and south-eastern corners of the site. The 1967 image shows that a couple of additional buildings had been constructed within the eastern portion. The 1979 image shows several new school buildings, with most of the previous buildings appearing to have been demolished due to flooding in 1974, including the residence in the north-eastern corner of the site. The 1979 image correlates with the establishment date on the existing buildings, which is 1978. From 1979, no new buildings appear to have been constructed until 2009, where a new building is visible within the south-eastern corner of the site. After 2009, no significant changes appear in the imagery until 2023 when several buildings are present within the southern half of the western block of the school, which correlate with the temporary school buildings following flooding in 2022.

3.2 Site Description

The site is located within relatively level topography associated with a floodplain bound by Leycester Creek, Wilsons River and Hollingworth Creek. Surface levels within the site are relatively level.

The site comprises the eastern block of Lismore South Public School. Buildings within the site generally comprise two storey buildings, primarily of brick construction, although some weatherboard buildings are also present. In the south-eastern corner of the site is a single-storey brick building. The north-eastern corner of the eastern portion school comprises a grassed play area. Asphalt surfaced carparks are present within the western and southern portions of the site. The buildings and surfaced pavements appeared to be in good condition based on cursory inspection. Medium to large trees are interspersed throughout the site and along the boundaries.

East of the site are residential properties, which contain single storey houses suspended above under crofts and generally surrounded by lawns. Surface levels across the boundaries with these properties are similar to those within the subject site.

The site is bound to the north, west and south by Phyllis Street, Wilson Street and Kyogle Street, respectively. Wilson Street separates the eastern and western blocks of the school. The pavements along Kyogle Street and Wilson Street appear to be in poor condition with rutting and extensive cracking of the pavements. The portion of Phyllis Street on the northern side of the site appears to be in fair condition.

3.3 Subsurface Conditions

The NSW Seamless Geology Version 2.4 indicates that the site is underlain by Quaternary alluvial floodplain deposits comprising *"silt, very fine- to medium-grained lithic to quartz-rich sand, clay"*. The boreholes and CPT probes encountered a generalised profile comprising relatively shallow fill and a deep alluvial soil profile. A summary of the subsurface conditions is provided below. For further specific details of the conditions encountered at each location, reference should be made to the attached borehole logs and CPT test results sheets.

Pavements and Fill

Sprayed seals were encountered at BH2 and BH15 drilled in the existing carparks within the western and southern portions of the site. Underlying the sprayed seal, silty sandy gravel fill was encountered to depths of 0.2m and 0.6m in BH2 and BH15 respectively. The gravel fill is anticipated to comprise an unbound granular 'road base' layer within the car park pavement.

In the remaining four boreholes silty sand fill extended to depths of 0.2m and 0.4m. The silty sand fill contained inclusions of igneous gravel.

Alluvial Soils

Alluvial soils, assessed as predominantly comprising silty clay, were encountered below the fill in all boreholes and in each of the CPT probes. The silty clay was assessed as being of high plasticity from initial contact to the termination depths of the boreholes.

For the purposes of this report we have divided the alluvial clay profile into four layers:

- Unit 1: Upper stiff clay which extended to depths ranging from 2.2m to 4.2m, with some firm strength clay encountered in the upper 2m of this profile.
- Unit 2: Upper very stiff clay which contained occasional hard bands and which extended to depths ranging from 14.5m to 16.5m. In the basal 1m to 1.5m of this stratum layers of silty sand and clayey silt were encountered. The silty sand was generally assessed as being of medium dense relative density, although some loose to medium dense sands were encountered in CPT15.
- Unit 3: Slightly over-consolidated to normally consolidated clay of stiff to very stiff strength which extended to depths ranging from 25.4m to 36.5m.
- Unit 4: Lower very stiff to hard clay which was encountered within the basal portion of each of the CPT probes, except CPT3. The clay contained bands of sandy silt, clayey silt and silty sand. The silts were generally assessed as being of hard strength and the sands of medium dense relative density. CPT refusal occurred within this deeper, stiffer stratum at depths ranging from 32.3m to 38.8m.

A summary of the depth to and surface level at the top of these strata is summarised in the table below:

CPT	Unit 1		Unit 2		Unit 3		Unit 4	
	Depth (m)	Level (mAHD)	Depth (m)	Level (mAHD)	Depth (m)	Level (mAHD)	Depth (m)	Level (mAHD)
3	0.6	10.1	2.2	8.5	16.5	-5.8	Not encountered	
6	0.2	10.4	3.5	7.1	15.5	-4.9	35..6	-25.0

15	0.6	10.2	4.2	6.4	15.8	-5.2	36.5	-25.9
17	0.4	10.3	3.2	7.5	14.5	-3.8	30.3	-19.6
18	0.6	10.2	3.9	6.9	14.5	-3.7	28.2	-17.4
20	0.7	9.9	2.7	7.9	15.7	-5.1	36.1	-25.5
22	0.4	10.2	2.2	8.4	14.5	-3.9	25.4	-14.8

Groundwater

All boreholes were 'dry' during and on completion of drilling. During a return visit to site on 15 October the monitoring wells in BH7 and BH23 were 'dry' however groundwater was measured at a depth of 5.3m in BH2, correlating with a reduced level at approximately RL5.4m.

From the piezocone pore pressure measurements, groundwater appears to have been encountered at depths of approximately 1.7m, 3.8m and 2.2m in CPT3, CPT15 and CPT22 respectively. These measurements do not appear to correlate with the groundwater observations within the monitoring wells which indicate that the groundwater level is currently at greater depths.

From river height data from the Bureau of Meteorology it appears that the steady-state level of water within Wilsons River and Leycester Creek near Lismore is in the range of RL0 to RL0.8m. Due to the location of the site in relatively close proximity to both of these water bodies we anticipate that the groundwater level is close to these levels. During and following flood events, the groundwater level is anticipated to rise closer to the ground surface.

3.4 Laboratory Test Results

The Atterberg Limit testing completed on the alluvial clay indicate they are of high plasticity. The moisture contents of the clays were above their respective plastic limits. The linear shrinkage and shrink-swell index test results generally indicate a very high potential for shrink-swell movements with changes in moisture content in the alluvial clay.

The four-day soaked CBR tests on the alluvial clay returned soaked CBR values of 1% and 0.5%. During soaking the samples swelling ranging from 4.5% to 6.5% was measured. The soaked CBR values are very low and the swelling indicates a high reactivity to variations in moisture content which correlates with the classification and shrink-swell index test results.

The results of the pH, sulphate, chloride and resistivity tests are summarised in Table 4 below.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
BH2	3.0-3.45	ALLUVIAL Silty Clay	7.7	53	180	5,300
BH7	4.5-4.95	ALLUVIAL Silty Clay	7.4	41	710	1,700
BH15	0.8-1.0	ALLUVIAL Silty Clay	5.1	490	140	2,700

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
BH19	1.5-1.95	ALLUVIAL Silty Clay	5.3	87	350	3,100
BH21	1.5-1.95	ALLUVIAL Silty Clay	5.3	180	250	3,200
BH23	3.0-3.45	ALLUVIAL Silty Clay	5.8	210	710	1,500

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Considerations

From a geotechnical perspective, the site will be challenging to develop due to the presence of the deep alluvial clay profile which is also highly reactive. We consider that the main geotechnical considerations relating to the design and construction of the proposed activity will be as follows:

- The alluvial clays are highly reactive and therefore footings will need to consider the potential for large shrink-swell movements with changes in moisture content within the design, particularly considering the possibility of periodic flooding. Additional consideration will need to be given to detailing of services, vegetation etc. which may affect the future performance of structures.
- The alluvial clays will likely undergo substantial strength loss when wet and they have very low CBR values. Although major earthworks are not anticipated, a working platform will be required to facilitate trafficability of the site for plant and construction of pavements and floor slabs. Development will require the use of relatively thick pavements, with some form of subgrade treatment to improve the subgrade quality, or bound subbases for concrete pavements.
- The alluvial clays are generally of stiff to very stiff strength to depths greater than 30m and appear to be normally consolidated or at best slightly over-consolidated below depths of approximately 15m. Due to the depth of the soil profile footings will need to be founded within the clays which, depending on the footing system adopted and the founding depth, will need to consider the potential for consolidation and possibly some creep settlement. Further in-situ and laboratory testing to assess stiffness and consolidation parameters for the clays is recommended
- The total depth of the soil profile is unknown. Although CPT refusal occurred in five of the seven probes this may have occurred on dense/hand layers within the alluvial profile rather than the surface of the underlying bedrock. For earthquake design we recommend that the site sub-soil classification be Class D_e unless additional investigation confirming the total soil profile depth is carried out.
- The site is located on a floodplain bound by Wilsons River, Leycester Creek and Hollingworth Creek. Design of the proposed structures must take into account the effect that fluctuations in groundwater levels will have on the performance of structures.
- Following demolition of the existing buildings, additional investigation should be completed to confirm the subsurface conditions in those areas which cannot currently be accessed.

Further comments on these issues are provided within the following sections of this report.

4.2 Site Classification

We note that in the strictest sense, AS2870-2011 does not apply to developments such as this, however it provides a useful guide for footing design on reactive clay sites. Reference may also be made to AS2870 for design, construction, performance criteria and maintenance precautions on reactive clay sites.

Assessment of the site classification for this site has been completed with reference to the results of the shrink-swell index, Atterberg Limits and linear shrinkage testing completed on the alluvial clays. The soils encountered were found to be of such reactivity that, even assuming no earthworks, and that the alluvial clays are not subject to any adverse moisture conditions (such as from flooding, trees, buildings, pavements etc.), the site would classify as Class ‘H2’ in accordance with AS2870-2011.

However, the site will almost certainly be subject to adverse moisture conditions, and probably also some cut and fill earthworks, where more onerous soil shrink-swell conditions can occur. Therefore, considering the site conditions, our recommendation is that structures be designed to accommodate shrink-swell movements normally associated with a Class ‘E’ site.

Apart from the characteristics of the soil and the presence of fill, there are many factors that affect the actual surface movements that occur. Such factors include:

- The depth of the soil profile;
- The likelihood of flooding;
- The presence of trees, past, present and future;
- The previous presence of structures and concrete slabs;
- The control and maintenance of drainage; and
- The installation of underground services.

The designers of structures on Class ‘E’ sites must consider the additional requirements of such sites as defined in Sections 5.6 and 6.6 of AS2870-2011. Owners of such lots must be made aware of the foundation maintenance requirements as stated in Appendix B of AS2870-2011. The landscape designers, structural and civil engineers should also be made aware of potential reactive soil issues. Particular attention is drawn to the installation of buried services and service penetrations to allow for accommodation of the anticipated shrink-swell movements as detailed in Sections 5.6.3 and 5.6.4 of AS2870-2011.

Reduced shrink-swell soil movements could be achieved by the use of inert (non-reactive) fill. As a guide, if the existing reactive alluvial clays were excavated out to a depth of not less than 0.5m and replaced with non-reactive fill we expect that the shrink-swell movements would likely be within the range of Class ‘H2’. For localised areas, for small structures, this would need to extend at least 1.5m beyond the edge of the proposed footings.

4.3 Earthworks

All earthworks recommendations provided below should be complemented by reference to AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments'.

4.3.1 Site Drainage

The alluvial clay subgrade at the site is expected to undergo substantial loss in strength when wet as evident from the low CBR values. Furthermore, the clay has a very high shrink-swell potential. Therefore, it is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principal aim of the drainage is to promote run-off and reduce ponding. A poorly drained clayey subgrade may become un-trafficable when wet, and consideration should be given to providing a crushed rock or crushed concrete working platform to minimise delays following rainfall. The earthworks should be carefully planned and scheduled to maintain good crossfalls during construction. Good surface and subsurface drainage must also be provided post construction to improve the long-term performance of the external paved areas.

4.3.2 Site Preparation

Following demolition of the existing buildings and pavements, and removal of trees (including their root balls), all grass, topsoil, root affected soils and any deleterious fill or contaminated soil should be stripped. The topsoil is not considered suitable for reuse as engineered fill however may be reused for landscaping purposes.

Care must be taken not to undermine or remove support from the site boundaries during stripping and subsequent bulk excavation works.

4.3.3 Excavation

Excavation for the proposed activity is generally not anticipated to extend below depths of 0.5m however locally deeper excavations may be required for footings or services. Excavations will encounter the existing fill and alluvial clay. This material can be excavated using hydraulic excavators.

Where slab on-grade construction is proposed then all existing fill should be stripped to the surface of the alluvial clay. Due to the limited depths of excavations and the results of our limited monitoring, we do not anticipate that excavations will encounter the groundwater table. As the site is located within an area prone to flooding following flood/heavy rainfall events, we anticipate that the groundwater table will be elevated for a period of time after the flood/heavy rainfall. In this regard, excavations may become inundated with water for a period following flooding/heavy rainfall and if this occurs then sump and pump techniques may be required to dewater excavations to allow construction to proceed.

4.3.4 Subgrade Preparation

Following stripping and any minor bulk excavation, the exposed subgrade should be proof rolled with at least six passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes deadweight. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas.

Subgrade heaving during proof-rolling is anticipated to occur in areas where the clays have become 'saturated' and/or are of firm to stiff strength. The CPT probes and boreholes indicate that firm to stiff clays may be encountered across a large proportion the subgrade. In this regard, bridging layer support using appropriately sized well graded durable crushed rock, and possibly high tensile geogrids, could be considered to facilitate construction of pavements and trafficability of the site during construction. If only small areas require improvement, then this may be achieved by locally removing the heaving/soft material to a stable base and replacing with engineered fill, as outlined below.

If the area requiring subgrade improvement is large, then a minimum 300mm thick bridging layer comprising well graded, coarse grained, durable crushed rock or crushed concrete of nominal 40-70mm size, with a dense grade non-woven geotextile filter fabric placed on the surface of the subgrade to control subsoil erosion, may be required. We forewarn that if crushed concrete is used, then it must contain less than 10% brick and tile fragments. Brick and tile fragments break down during compaction of the bridging layer, and have the propensity to absorb moisture, thus potentially negating the performance of the layer. Options and detailed design of subgrade improvement works must be provided by the geotechnical engineer following the proof rolling inspection.

If soil softening occurs after rainfall periods or flooding, the clay subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the clay subgrade exhibits shrinkage cracking, then the surface must be moistened with a water cart and rolled until the shrinkage cracks are no longer evident. Care must be taken not to over-water the subgrade as this will result in softening.

Engineered fill must be used to raise site levels.

4.3.5 Engineered Fill

General

From a geotechnical perspective, due to the relatively limited height of filling proposed, the reactivity of compacted clays and issues with moisture control of clay fill our preference is for the alluvial clay soils not be reused as engineered fill.

Engineered fill should preferably comprise well-graded, non-reactive granular material such as crushed basalt. The fill material should be tentatively compacted in maximum 300mm thick loose layers using a large static (non-vibratory) smooth-drum roller (say, at least 12 tonnes deadweight) to a density ratio strictly between 98% and 100% of Standard Maximum Dry Density (SMDD).

Service Trenches
Backfilling of service trenches must be carried out using engineered fill in order to reduce post-construction settlements. Due to the reduced energy output of compaction plant that can be placed in trenches, backfilling should be carried out in maximum 150mm thick loose layers and compacted using a trench roller, a pad-foot roller attachment fitted to an excavator, and/or a vertical rammer compactor (also known as a 'Wacker Packer'). Due to the reduced loose layer thickness, the maximum particle size of the backfill material should also reduce to 50mm. The compaction specification provided above is applicable. Alternatively, consideration could be given to backfilling service trenches with stabilised sand which does not require compactive effort.

Earthworks Inspection and Testing
Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved, as outlined below:

- The frequency of density testing for general engineered fill should be at least one test per layer per $2500m^2$ or one test per $500m^3$ distributed reasonably evenly throughout the full depth and area, or 3 tests per lot (as defined in Clause 1.2.8 of AS3798-2007), whichever requires the most tests (assumes maximum 350mm thick loose layers);
- The frequency of density testing for trench backfill should be at least one test per two layers per 40 linear metres (assumes maximum 150mm thick loose layers);
- The frequency of density testing for retaining wall backfill should be at least one test per two layers per $50m^2$ (assumes maximum 150mm thick loose layers).

Density testing should be regularly carried out on any engineered fill to confirm that the project specification has been met. Supervision and regular density testing in accordance with Level 1 requirements of AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments' is recommended if engineered fill is required to support structural loads from buildings. In pavement or landscaped areas, or where fill is placed as form fill below buildings, Level 2 testing may be carried out.

4.4 Footings

The site is located on a floodplain and is bounded by Wilsons River, Leycester Creek and Hollingworth Creek. The design of structures must take into account the potential for fluctuations in groundwater levels, flooding, and the effect this will have on the performance of structures. Without any longer-term groundwater monitoring data, and based on the historic flooding that has occurred at the site, we recommend that structural designs consider groundwater levels being present at, or above, surface levels when assessing the potential for uplift pressures on any buried structures.

4.4.1 Main Building

Due to the depth of alluvial soil underlying the site, footings for the proposed structures will be founded within the alluvial silty clay which is generally of stiff to very stiff strength. For the main building, based on the advised column loads, we anticipate that either piled footings or a stiffened raft foundation (which may include piles integrated with the stiffened raft) will be required.

We understand from the current design, that no ground floor slabs are proposed for the main building and therefore adoption of a stiffened raft foundation will require more extensive earthworks, (such as stripping all existing fill) then is likely currently proposed for the fully suspended structure. Should a stiffened raft foundation, with or without piles, be preferred then further specific design and analysis will be required.

4.4.1.1 Piled Footings

For piled foundations we consider that either deep single piles or shallower, pile groups will be required to support the main structure. Single pile foundations will likely need to extend very deep or have very large diameters. Due to the high reactivity of the alluvial clays we recommend pile caps be embedded to a minimum 1.5m depth to reduce the risk of swelling pressures adversely affecting pile groups. Any pile caps founded above 1.5m would need to be underlain by void formers to reduce the risk of uplift pressures on the underside of the pile cap.

Due to the presence of groundwater, unless piles can be founded above the groundwater table, then we recommend that either steel screw piles or grout-injected, continuous flight auger (CFA) piles be adopted. Driven piles, although feasible, have a higher risk of damage to nearby structures due to ground-borne vibrations. Due to the proximity of adjoining residential buildings we do not recommend the use of driven piles.

The table in Section 3.3 provides our assessment of the depth and reduced levels for the various clay units encountered within the boreholes and CPT probes. Based on this assessment, the following table presents our recommendations on allowable and ultimate end bearing pressures, allowable and ultimate shaft adhesions, elastic modulus and Poisson's ratio values for Units 2 and 3. For the anticipated loads we do not consider that the clays within Unit 1 will be sufficiently stiff and therefore we have not provided parameters for piles within this stratum. Similarly, values have not been provided for Unit 4 as piles would be excessively deep to reach this stratum are considered uneconomical. The end bearing values in the table below assume a length to diameter (L/D) ratio of at least 4 is achieved for the piles and that the piles are founded within Unit 2 on at least very stiff clay (with a minimum undrained shear strength of 130kPa), or within Unit 3 on at least stiff to very stiff clay (with a minimum undrained shear strength of 100kPa).

Unit	Allowable End Bearing (kPa)	Ultimate End Bearing Pressure (kPa	Allowable Shaft Adhesion (kPa)	Ultimate Shaft Adhesion (kPa)	Elastic Modulus (MPa)	Poisson's Ratio
2	400	1,200	20	60	15	0.25
3	300	900	15	50	10	0.25

For the design of piles in tension, a Factor of Safety of 2 should be applied to the shaft adhesion values in the table above. For steel screw piles, due to the disturbance to the ground around the shaft caused by penetration of the helix, we recommend that shaft adhesion be ignored in the design of these piles. For CFA piles the shaft adhesion in the upper 1.5m should be ignored due to the potential for shrinkage resulting in material moving away from the pile shaft. However, for assessing the effect of swelling of the soils on piles,

The values in the table above are for single piles and where pile groups are adopted, with a pile cap founded within the alluvial clays below the ground surface, the capacity should be taken as the lesser of:

1. The sum of the capacities of the piles in the group and the pile cap, acting independently, with the area of the pile cap bearing on the soil, calculated as the pile cap area less the area of the piles. Consideration will need to be given to strain compatibility between the pile cap and the piles in the group. For the pile cap an allowable bearing pressure of 80kPa may be adopted for the stiff clay encountered within Unit 1.
2. The bearing capacity for a rectangular block containing the piles and the soil between them plus the portion of the pile cap outside the perimeter of the block.

Generally pile spacings less than 2.5D for friction piles is not recommended unless an analysis of interaction effects indicates that the overall pile group performance is not adversely impacted. For piles deriving their resistance mainly from end-bearing (such as steel screw piles), the spacing should not be less than 2D, unless interaction effects for such groups is analysed. Where pile groups are adopted due to the increased width of the block acting at the base of the piles the load will be distributed over a greater depth than individual piles which may lead to an increased potential for consolidation settlements.

Where allowable bearing pressures and skin friction values are adopted, elastic settlement of piles will typically be less than 1% of the pile diameter at the toe of the pile. However, additional consolidation and possibly creep settlement may also occur due to the underlying normally consolidated/slightly over-consolidated clays within Unit 3. As a guide, based on empirical correlations from the existing information, we estimate that consolidation settlements from the advised loading may be in the order of 35mm to 45mm. Further investigation to assess the stiffness and consolidation characteristics of the alluvial clays must be carried out to refine the potential settlements. Additional investigation should include dilatometer testing, which will provide more refined measurements for parameters such as Elastic Modulus, coefficient of consolidation and undrained cohesion. Additional laboratory testing should include oedometer tests on the clay from Units 2 and 3 to assess the consolidation characteristics. Some dissipation testing using the CPT rig would also be beneficial to assess consolidation characteristics. The feasibility of footing designs will not be able to be confirmed until the recommended additional investigations are carried out.

Where ultimate end bearing and skin friction values are adopted, settlements will be greater. Once column loads are known, detailed settlement analysis of the foundation system is recommended to check that predicted settlements are within acceptable limits.

Where ultimate end bearing and skin friction values are adopted, then the ultimate values recommended in the table above must be reduced by an appropriate geotechnical reduction factor. The geotechnical reduction factor should be based on the risk assessment procedure set out in Table 4.3.2 (A) of AS2159-2009, but should not be greater than 0.4, unless the risk factors producing a higher geotechnical reduction factor can be fully justified. Consideration should also be given to the pile testing requirements when determining a suitable geotechnical strength reduction factor. The use of ultimate values will result in higher settlements and therefore specific analysis of the footing settlements must be carried out to confirm that it is consistent with the required structural performance.

Where piling rigs are set up at bulk excavation level, we anticipate that a working platform with a minimum thickness of 0.3m will be required to protect the subgrade from deterioration during inclement weather. The specific requirements for any working platforms should be determined once the piling rig and the loading conditions are known and a thickness of more than 0.3m may be required. Where a bridging layer is required for earthworks then this may be included in the working platform assessment. A proof roll inspection of the subgrade, as recommended in Section 4.3.4, should be completed by the geotechnical engineer to confirm the suitability of the material and identify any soft spots requiring remediation. As a guide, the wearing surface material could comprise a DGB20 or similar granular material, such as recycled crushed concrete. The DGB20 material must be compacted using a medium sized static roller to at least 98% SMDD. The compacted wearing surface should extend at least 2m outside the working area of the pilling rig.

For suspended slabs founded on piers, the portion of slabs above a clay subgrade will need to be underlain by degradable void formers of at least 100mm thickness to reduce the risk of swelling soils 'jacking' the slabs off the piles.

4.4.1.2 Stiffened Raft Footing Systems

To distribute the load over a larger area then is achievable with piled foundations, stiffened raft slabs may be considered for the footing system of the main building. Due to the larger foundation footprint of a stiffened raft, the depth of influence of additional loading will be considerably deeper than piled foundations and will include the slightly over-consolidated to normally consolidated clays within Unit 3.

Further geotechnical investigations involving deeper CPT testing and dilatometer testing should be completed to obtain a continuous subsurface profile and assess the extent of any weaker subsurface conditions. The dilatometer, is particularly useful as it provides a direct measurement of the soil stiffness characteristics (elastic modulus) for incorporating within settlement analysis.

Once building loads are known, preliminary analysis could assess the potential settlements, slab reactions and contact pressures for the raft slab using elastic modulus parameters derived from the current testing of the soils. JK Geotechnics can assist with detailed raft slab analysis using software such as Plaxis 3D, which would be required to estimate the settlements (including consolidation) and the contact pressures below the raft. The design of large stiffened rafts is complex and requires detailed analysis procedures for soil/structure interaction. Therefore, we expect that the design of a raft foundation will be an iterative procedure with both the geotechnical and structural engineers having input to the process. The first pass of the analysis would be to demonstrate the feasibility of the concept and identify the parameters critical to the design. The parameters will then need refinement and would require further investigation and testing to justify the key assumptions and enable the design to be refined. As a guide only, elastic settlements from the advised structural loads may be in the order of 30mm to 40mm whilst consolidation settlements within Unit 3 are estimated to be of a similar order of magnitude as those from pile groups i.e. 35mm to 45mm.

For construction of stiffened raft slabs the subgrade below the slabs should be prepared in accordance with the recommendations in Section 4.3. The stiffened raft slab will need to be designed for shrink-swell movements for the appropriate shrink-swell movements which should be determined following confirmation of bulk excavation levels, material used as fill and the presence of nearby trees.

4.4.2 Minor Structures

Lightly loaded structures may be designed to be supported on either high-level footings or steel screw piles founded within the alluvial clay (Unit 1).

Shallow footings may be designed based on an allowable bearing pressure of 80kPa for alluvial clay of at least stiff strength. For high-level footings founded within reactive alluvial clay, these will need to be designed to accommodate the shrink-swell movements which will depend on the material used as fill, the excavations carried out and the presence of nearby trees. All of these factors will need to be taken into account to determine the appropriate shrink-swell movements for each structure as it may vary in different areas of the site. Reference should be made to Section 4.2 above on likely shrink-swell movements. Particular consideration will also need to be given to the effect of reactive engineered fill as greater surface movements may apply. Reference should also be made to Appendix B of AS2870-2011 which provides further guidance on foundation performance and maintenance for structures on reactive silty clay soils.

Steel screw piles for lightly-loaded structures may be designed using the same parameters as those for the main building.

4.4.3 Footing Inspections

We recommend that the geotechnical engineers inspect high-level footing excavations, including beams for stiffened raft slabs, to confirm the above recommended bearing pressures and skin frictions are being achieved. If steel screw piles are adopted then we note that inspection of the material at the base of the piles will not be possible. Steel screw piles are generally completed on a design and construct basis and where adopted we recommend that installation of screw piles be carried out as close as possible to test locations to allow for calibration of torque readings and resistance to drilling with known subsurface conditions. Inspection of the drilling of CFA piles will only be able to assess the depth is consistent with the borehole and CPT results, possibly with inspection of material at the base of the augers upon extraction.

Prior to pouring concrete, high-level footings will need to be dewatered, cleaned of all loose debris from the base, inspected and approved by the geotechnical engineers. High-level footings will need to be poured as soon as possible after excavation.

4.5 Earthquake Design

Based upon AS1170.4:2024 "Structural Design Actions, Part 4: Earthquake actions in Australia", the following design parameters may be adopted:

- Hazard Factor (Z) = 0.08;
- Class D_e – Deep or soft soil site

We note that the current testing completed has confirmed that soil extends to depths of at least 40.5m below existing surface levels however the total depth of soil has not been confirmed. It may be possible to revise the site sub-soil classification to Class C_e if the depth of soil can be proven to be less than 60m. However, further investigation with CPT testing and possibly a couple of deep cored boreholes will be required to confirm the depth of the soil profile.

4.6 Exposure Classification

Based on the soil aggression test results, the alluvial clays are classified as having a 'Mild' exposure classification for concrete and steel piles in accordance with Tables 6.4.2(C) and 6.5.2(C) of AS2159-2009 'Piling – Design and installation'. For concrete structures in contact with the alluvial clay an exposure classification of 'A2' would apply in accordance with Table 4.8.1 of AS3600-2018.

4.7 Pavements

Following completion of bulk earthworks, we anticipate that the subgrade for pavements will predominantly comprise alluvial silty clay. The subgrade will need to be prepared in accordance with the requirements of Section 4.3 above. We consider that a design CBR of 0.5% should be adopted for the alluvial silty clay.

Due to the very low design CBR value we consider that construction of pavements without improvement to the subgrade will be difficult to achieve. In this regard we consider that some options for pavement design and construction are as follows:

1. Provide an appropriate select fill layer as part of the overall pavement thickness. The select fill should preferably comprise a well-graded, good quality ripped or crushed basalt with a minimum soaked CBR value greater than 10%. The pavement sections where imported fill is used to raise site levels may be designed taking into account the thickness and soaked CBR value of the imported fill material.

OR

2. Stabilise the subgrade to a depth of about 300mm by the addition of lime. When thoroughly mixed and re-compacted to a minimum of 98% of SMDD, a reduction in reactivity along with substantial increase in strength will be achieved. As a guide, the addition of approximately 4% lime by dry weight of clay should result in a soaked CBR value of around 5% or an equivalent subgrade reaction modulus of 35kPa/mm. This should, however, be confirmed by laboratory testing. If lime stabilisation is undertaken, an experienced contractor with appropriate equipment should complete it, and a number of lime demand tests will need to be undertaken prior to commencement to confirm the percentage

of lime required to achieve the required soaked CBR value. We note that use of lime close to residential areas is generally not preferred unless an acceptable method of dust suppression can be adopted.

OR

3. If rigid pavements are preferred a 150mm lean-mix concrete subbase should be placed below the concrete base course such that an effective subgrade strength of 5% may be adopted. Alternatively, if granular materials for working platform, capping or select fill layers are adopted then the CBR of these materials can be used to determine a higher equivalent design CBR value. For example for a 0.3m thick layer of granular material with a CBR≥30% an equivalent design CBR of 3% could be achieved, based on Equation 55 of Austroads' *Guide to Pavement Technology Part 2: Pavement Structural Design*.

Where a working platform and/or bridging layer is adopted then this layer may be included within the pavement design. Due to the highly reactive nature of the clay, a low-permeability capping layer should be incorporated within the pavement profile to limit moisture related movement. The capping layer should comprise a select fill or subbase material with a minimum thickness of the greater of 150mm or 2.5 times the maximum particle size. The capping layer should extend at least 500mm past the edge of the pavement, including kerb and gutter.

To further reduce the potential for moisture variations below the pavements, consideration should be given to including sealed shoulders and impermeable verge materials with a minimum width of 1m from the pavement edges.

All unbound granular materials for pavements should comprise good quality, fine crushed rock in accordance with TfNSW QA Specification 3051. The pavement materials should be compacted using a large static smooth drum roller to at least 100% of SMDD. Adequate moisture conditioning to within 2% of Standard Optimum Moisture Content (SOMC) should be provided during placement.

For rigid pavements slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

Density tests should be carried out on the unbound granular pavement materials to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 50m length of lane or 1 per 500m2 layer, whichever requires the most tests. At least Level 2 testing should be completed on pavement layers and the geotechnical inspection and testing authority (GITA) should be directly engaged by the client or their representative.

In order to protect pavements, subsoil drains should be provided along the perimeter of all proposed pavement areas. Subsoil drains should not extend into the reactive alluvial clay subgrade to reduce the potential for variations in moisture content. The drainage trenches should be excavated with a continuous longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. from the subsoil drains should be piped to the stormwater system for disposal.

4.8 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Further site investigation and laboratory testing including dilatometer and oedometer testing, and dissipation tests to characterise the stiffness, consolidation characteristics and depth of the alluvial clays. Additional investigation should also be completed following demolition to confirm site conditions in those areas currently inaccessible to the drilling rigs.
- Detailed settlement analysis for the foundation system. This is required to further assess the potential and magnitude of any consolidation settlement that will occur as a result of the additional stresses place don the lower normally consolidated clay layer (Unit 3).
- Proof-rolling inspections and further advice on subgrade treatment such as bridging layers and/or lime stabilisation.
- Lime-demand and lime-stabilised CBR testing, if such an approach is preferred for pavement construction.
- In-situ density testing of all materials placed as engineered fill to confirm that it complies with the earthworks specification.
- Design of working platforms for the specific piling rigs proposed.
- Inspection of footing excavations and piling.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the detailed design and construction phase of the project. In the event that any of the advice presented in this report is not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long-term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Due Diligence - Flood Recovery
Location: 69-79 Kyogle Street, South Lismore, NSW

Report No.: 36310LT - A
Report Date: 17/10/2024

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
2	0.50 - 0.95	50.7	90	31	59	18.5*
7	0.50 - 0.95	33.7	64	29	35	16.0**
15	1.50 - 1.95	43.6	92	30	62	13.0**
19	0.50 - 0.95	39.5	94	32	62	18.0 * **
21	0.50 - 0.95	44.8	67	28	39	15.0**
23	0.50 - 0.95	34.7	88	27	61	20.0 *

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 02/10/2024.
- Sampled and supplied by client. Samples tested as received.
- * Denotes Linear Shrinkage curled.
- ** Denotes Linear Shrinkage cracked.

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17/10/2024
Authorised Signature / Date
(D. Treweek)

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Report No.:**	36310LT - B
Project:	Due Diligence - Flood Recovery	**Report Date:**	16/10/2024
Location:	69-79 Kyogle Street, South Lismore, NSW	**Page 1 of 1**	

BOREHOLE NUMBER		BH 19	BH 21	BH 23
DEPTH (m)		0.50 - 0.95	0.50 - 1.10	0.50 - 0.95
Surcharge (kg)		4.5	4.5	4.5
Maximum Dry Density (t/m^3)		1.38 STD	1.35 STD	1.31 STD
Optimum Moisture Content (%)		32.2	31.5	28.5
Moulded Dry Density (t/m^3)		1.34	1.32	1.29
Sample Density Ratio (%)		98	98	98
Sample Moisture Ratio (%)		104	100	98
Moisture Contents				
Insitu (%)		35.2	37.1	38.7
Moulded (%)		33.5	31.5	27.8
After soaking and				
After Test, Top 30mm(%)		53.9	55.1	60.7
Remaining Depth (%)		38.4	36.9	44.6
Material Retained on 19mm Sieve (%)		0	0	0
Swell (%)		4.5	5.0	6.5
C.B.R. value:	@2.5mm penetration	1.0	1.0	0.5

NOTES: Sampled and supplied by client.

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 02/10/2024.
- All samples dried back prior to testing as it was too saturated.

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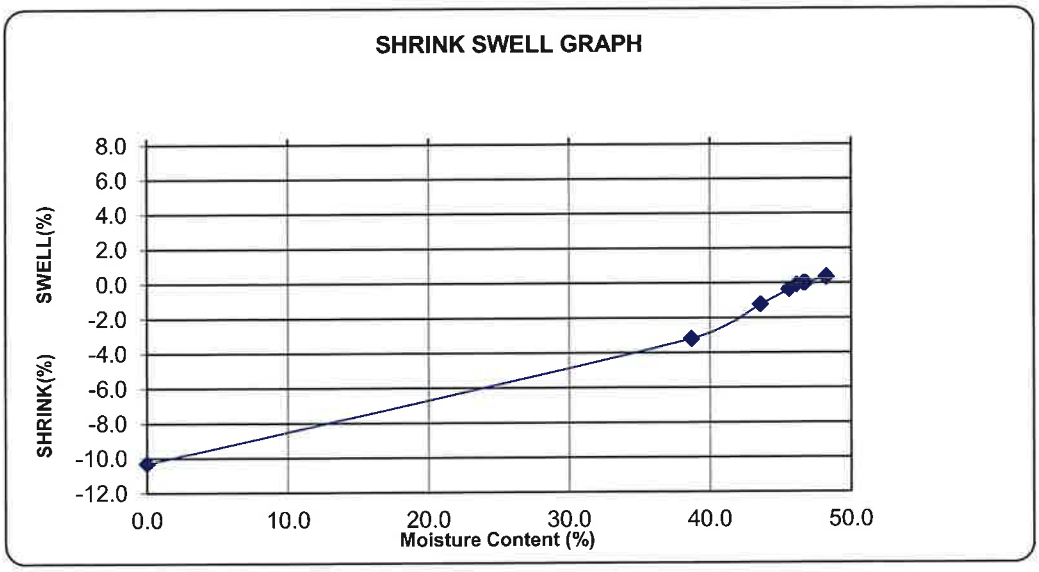
SOIL TEST SERVICES

ABN 43 002 145 173

TABLE C
SHRINK - SWELL TEST REPORT
TEST METHOD: AS1289 7.1.1

Client:	JK Geotechnics	**Report No.:**	36310LT - C
Project:	Due Diligence - Flood Recovery	**Report Date:**	16/10/2024
Location:	69-79 Kyogle Street, South Lismore, NSW	**Page**	1 of 3

Borehole No.: 2	Depth: 1.00 - 1.40m			
MOISTURE CONTENT (SWELL)		ESTIMATED UNCONFINED COMPRESSIVE STRENGTH		
BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST	
46.6%	48.2%	110 kPa	85.95 kPa	
LOAD	SETTLEMENT UNDER LOAD BEFORE SATURATION		SWELL ON SATURATION	SHRINKAGE
25	-0.5%		0.3%	10.1%



SHRINK SWELL INDEX
5.68 %/pF

Notes: Sampled and supplied by client. Sample tested as received.
- Suction Value used in calculation = 1.8pF
- Volume Change Coefficient (α) was assumed = 2
- Visually estimated inclusions by volume = 0-5%
- Shrinkage Cracking = Moderate
- Soil Crumbling = none
- Date of receipt of sample: 02/10/2024.

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16/10/24

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SOIL TEST SERVICES

ABN 43 002 145 173

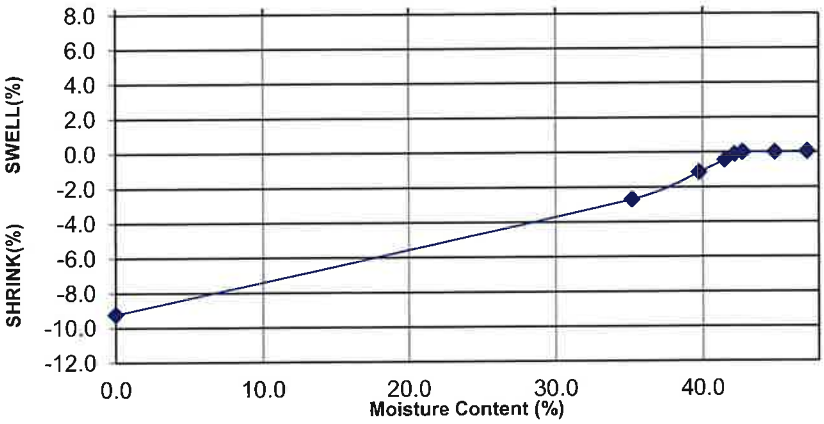
TABLE C
SHRINK - SWELL TEST REPORT
TEST METHOD: AS1289 7.1.1

Client: JK Geotechnics
Project: Due Diligence - Flood Recovery
Location: 69-79 Kyogle Street, South Lismore, NSW

Report No.: 36310LT - C
Report Date: 16/10/2024
Page 2 of 3

Borehole No.: 19		Depth: 1.10 - 1.42m	
MOISTURE CONTENT (SWELL)		ESTIMATED UNCONFINED COMPRESSIVE STRENGTH	
BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST
45.0%	47.2%	130 kPa	100 kPa
LOAD	SETTLEMENT UNDER LOAD BEFORE SATURATION	SWELL ON SATURATION	SHRINKAGE
25	-0.7%	0.0%	9.0%

SHRINK SWELL GRAPH



SHRINK SWELL INDEX
4.99 %/pF

Notes: Sampled and supplied by client. Sample tested as received.

- Suction Value used in calculation = 1.8pF
- Volume Change Coefficient (α) was assumed = 2
- Visually estimated inclusions by volume = 0-5%
- Shrinkage Cracking = Moderate
- Soil Crumbling = none
- Date of receipt of sample: 02/10/2024.

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16/10/24

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SOIL TEST SERVICES
ABN 43 002 145 173

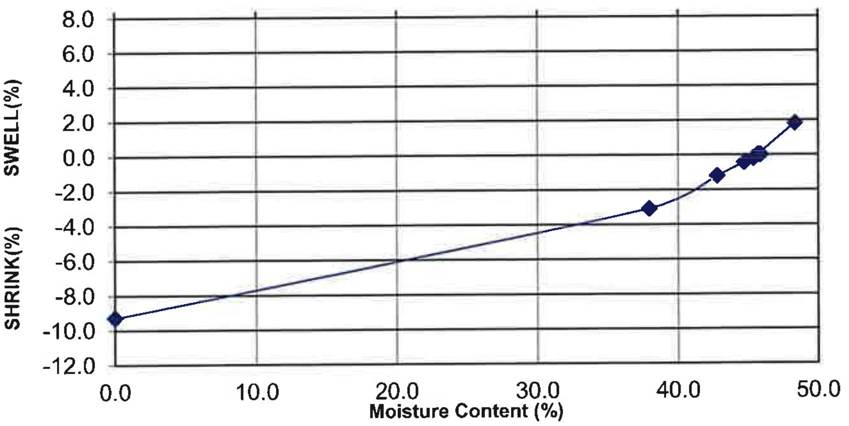
TABLE C
SHRINK - SWELL TEST REPORT
TEST METHOD: AS1289 7.1.1

Client: JK Geotechnics
Project: Due Diligence - Flood Recovery
Location: 69-79 Kyogle Street, South Lismore, NSW

Report No.: 36310LT - C
Report Date: 16/10/2024

Borehole No.: 23		Depth: 1.00 - 1.37m		
MOISTURE CONTENT (SWELL)		ESTIMATED UNCONFINED COMPRESSIVE STRENGTH		
BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST	
45.7%	48.4%	150 kPa	90 kPa	
LOAD	SETTLEMENT UNDER LOAD BEFORE SATURATION		SWELL ON SATURATION	SHRINKAGE
25	-1.1%		1.8%	9.0%

SHRINK SWELL GRAPH



SHRINK SWELL INDEX
5.52 %/pF

Notes: Sampled and supplied by client. Sample tested as received.
- Suction Value used in calculation = 1.8pF
- Volume Change Coefficient (α) was assumed = 2
- Visually estimated inclusions by volume = 0-5%
- Shrinkage Cracking = Moderate
- Soil Crumbling = none
- Date of receipt of sample: 02/10/2024.

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CERTIFICATE OF ANALYSIS 363781

Client Details

Client	JK Geotechnics
Attention	Keagan Rousseau
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**36310LT, 69-79 Kyogle St, South Lismore NSW**
Number of Samples	6 Soil
Date samples received	11/10/2024
Date completed instructions received	11/10/2024

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	18/10/2024
Date of Issue	16/10/2024

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Results Approved By
Diego Bigolin, Inorganics Supervisor

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil						
Our Reference		363781-1	363781-2	363781-3	363781-4	363781-5
Your Reference	UNITS	BH2	BH7	BH15	BH19	BH21
Depth		3-3.45	4.5-4.95	0.8-1.0	1.5-1.95	1.5-1.95
Date Sampled		25/09/2024	25/09/2024	25/09/2024	25/09/2024	25/09/2024
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	15/10/2024	15/10/2024	15/10/2024	15/10/2024	15/10/2024
Date analysed	-	15/10/2024	15/10/2024	15/10/2024	15/10/2024	15/10/2024
pH 1:5 soil:water	pH Units	7.7	7.4	5.1	5.3	5.8
Chloride, Cl 1:5 soil:water	mg/kg	180	710	140	350	250
Sulphate, SO4 1:5 soil:water	mg/kg	53	41	490	87	180
Resistivity in soil*	ohm m	53	17	27	31	32

Misc Inorg - Soil		
Our Reference		363781-6
Your Reference	UNITS	BH23
Depth		3-3.45
Date Sampled		25/09/2024
Type of sample		Soil
Date prepared	-	15/10/2024
Date analysed	-	15/10/2024
pH 1:5 soil:water	pH Units	6.8
Chloride, Cl 1:5 soil:water	mg/kg	710
Sulphate, SO4 1:5 soil:water	mg/kg	210
Resistivity in soil*	ohm m	15

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			15/10/2024	1	15/10/2024	15/10/2024		15/10/2024	[NT]
Date analysed	-			15/10/2024	1	15/10/2024	15/10/2024		15/10/2024	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	7.7	7.5	3	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	180	180	0	104	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	53	55	4	106	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	53	54	2	[NT]	[NT]

Result Definitions	
NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments
Samples received in good order: Holding time exceedance

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BOREHOLE LOG

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **Method:** SPIRAL AUGER **R.L. Surface:** ~10.5 m
Date: 24/9/24 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** K.R./A.B.

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION				0		-	BITUMINOUS SURFACE: 3mm.t	w>PL			
							FILL: Silty sandy gravel, fine to medium grained, dark grey, igneous, angular, fine to medium grained sand.	w>PL			
			10			CH	Silty CLAY: high plasticity, dark grey, brown and light brown, trace of fine to medium grained rounded gravel, and root fibres.	w>PL	St		ALLUVIAL
	DS	N = 4 1,2,2								110 110 120	
	U50			1							
	DS	N = 6 1,2,4	9							130 110 110	
			8	2							
							Silty CLAY: high plasticity, grey mottled brown, trace of fine to medium grained rounded gravel.	w~PL	VSt		
	DS	N = 15 5,8,7		3						360 340 370	
			7								
			6	4							GROUNDWATER MONITORING WELL INSTALLED TO 6.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 1.5m TO 6.0m. CASING 0.1m TO 1.5m. 2mm SAND FILTER PACK 1.4m TO 6.0m. BENTONITE SEAL 0.3m TO 1.4m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
ON 15/10/24			5	5							
				6			END OF BOREHOLE AT 6.00 m				
			4								

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BOREHOLE LOG

Borehole No.
11
1 / 1

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **Method:** SPIRAL AUGER **R.L. Surface:** ~10.8 m
Date: 24/9/24 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** K.R./A.B.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION AND ON 15/10/24				0			FILL: Silty sand, fine to medium grained, brown, trace of fine to medium grained igneous gravel, metal fragments and root fibres.	M			GRASS COVER
				0.4		CH	Silty CLAY: high plasticity, dark brown, trace of fine to medium grained sand.	w>PL	St		ALLUVIAL
	DS	N = 8 2,4,4	10							120 130 130	NO SPT SAMPLE RECOVERY
				1							
				1.3			Silty CLAY: high plasticity, grey and dark brown, trace of ash and root fibres.	w~PL	VSt		
	DS	N = 5 2,2,3	9							215 220 220	
				2							
			8	3							
	DS	N = 14 5,7,7								350 380 390	
			7	4							
				4.5			as above, but brown.				GROUNDWATER MONITORING WELL INSTALLED TO 6.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.0m TO 6.0m. CASING 0.04m TO 2.0m. 2mm SAND FILTER PACK 1.5m TO 6.0m. BENTONITE SEAL 0.3m TO 1.5m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
	DS		6	5							
			5	6			END OF BOREHOLE AT 6.00 m				
			4								

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BOREHOLE LOG

Borehole No.
15
1 / 1

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **Method:** SPIRAL AUGER **R.L. Surface:** ~10.6 m
Date: 24/9/24 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** K.R./A.B.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION							0		-	BITUMINOUS SURFACE: 3mm.t	M			
										FILL: Silty sandy gravel, fine to medium grained, dark grey, igneous, angular, fine to medium grained sand, trace of brick fragments.				
					N=SPT 5/ 0mm REFUSAL	10			CH	Silty CLAY: high plasticity, grey mottled brown and light brown, trace of fine to medium grained rounded gravel, and root fibres.	w>PL	St - VSt		ALLUVIAL
				■			1							
				■	N = 8 3,4,4	9							190 230 200	
							2							
						8								
							3			END OF BOREHOLE AT 3.00 m				
						7								
							4							
						6								
							5							
						5								
							6							
						4								

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BOREHOLE LOG

Borehole No.
19

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **Method:** SPIRAL AUGER **R.L. Surface:** ~10.7 m
Date: 24/9/24 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** K.R./A.B.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			FILL: Silty sand, fine to medium grained, brown, trace of fine grained igneous gravel, plastic fragments and root fibers.				GRASS COVER
	N = 6 2,3,3	CH	SILTY CLAY: high plasticity, dark brown, trace of root fibres.	w>PL	St	130 100 110	ALLUVIAL
	N = 9 3,5,4		SILTY CLAY: high plasticity, grey mottled brown.			120 120 120	
	N = 14 5,7,7		SILTY CLAY: high plasticity, grey mottled red brown and brown, trace of fine to medium grained rounded gravel, and ash.	w~PL	VSt	370 360 300	
			END OF BOREHOLE AT 3.45 m				

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BOREHOLE LOG

Borehole No. 21

1 / 1

Client: SCHOOL INFRASTRUCTURE NSW

Project: PROPOSED SCHOOL REDEVELOPMENT

Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **Method:** SPIRAL AUGER **R.L. Surface:** ~10.6 m

Date: 24/9/24 **Datum:** AHD

Plant Type: JK300 **Logged/Checked By:** K.R./A.B.

Groundwater Record	SAMPLES ES / U50 / DB / DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION				0			FILL: Silty sand, fine to medium grained, brown, trace of fine to medium grained igneous gravel, clay nodules and root fibres.	M			GRASS COVER
						CH	Silty CLAY: high plasticity, grey mottled brown, trace of root fibres.	w>PL	(St)		ALLUVIAL
									VSt		
	DB	N = 7 2,4,3	10							280 310 300	
				1							
	DS	N = 9 3,4,5	9	2						370 370 380	
			8	3							
	DS	N = 17 6,9,8									
			7	3.45			END OF BOREHOLE AT 3.45 m				
			6	4							
			5	5							
			4	6							

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BOREHOLE LOG

Borehole No. **23**

1 / 1

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **Method:** SPIRAL AUGER **R.L. Surface:** ~10.8 m
Date: 24/9/24 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** K.R./A.B.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION AND ON 15/10/24				0			FILL: Silty sand, fine to medium grained, brown and light grey, trace of fine to medium grained igneous gravel, roots and root fibres.	w>PL			GRASS COVER
	DS	N = 7 2,3,4	10	1		CH	Silty CLAY: high plasticity, dark grey and brown, trace of root fibres.	w>PL	St	130 110 140	ALLUVIAL
	U50						as above, but grey mottled brown.				
	DS	N = 6 2,2,4	9	2						150 140 150	
	DS	N = 12 4,6,6	8	3							
	DS		7	4				w~PL	VSt	220 210 220	
			6	5							GROUNDWATER MONITORING WELL INSTALLED TO 6.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.0m TO 6.0m. CASING 0.11m TO 2.0m. 2mm SAND FILTER PACK 1.5m TO 6.0m. BENTONITE SEAL 0.3m TO 1.5m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
			5	6			END OF BOREHOLE AT 6.00 m				
			4								

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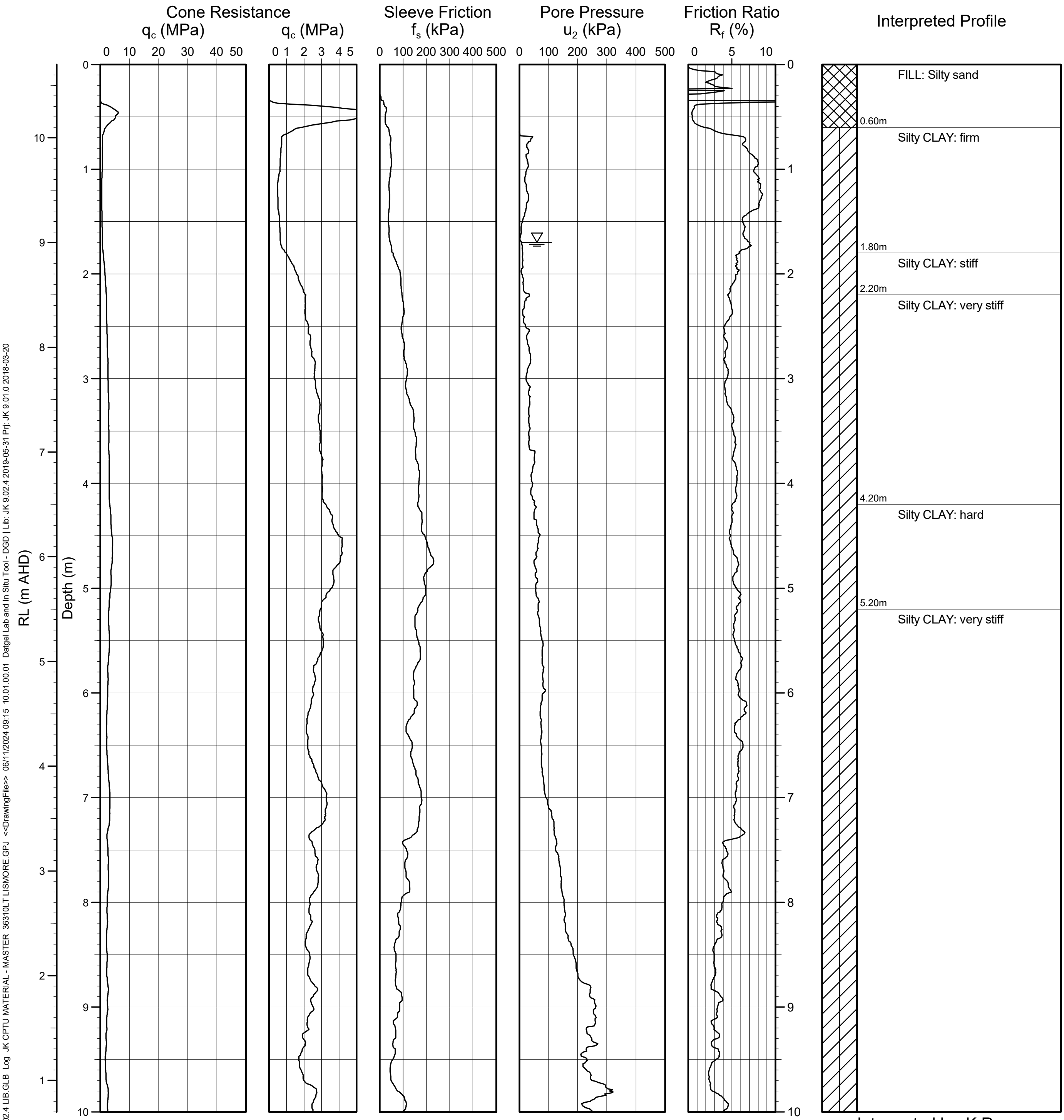
CONE PENETROMETER TEST RESULTS

CPT No. CPT3

1 / 4

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.7 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 36310LT LISMORE.GPJ <<DrawingFile>> 06/11/2024 09:15 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

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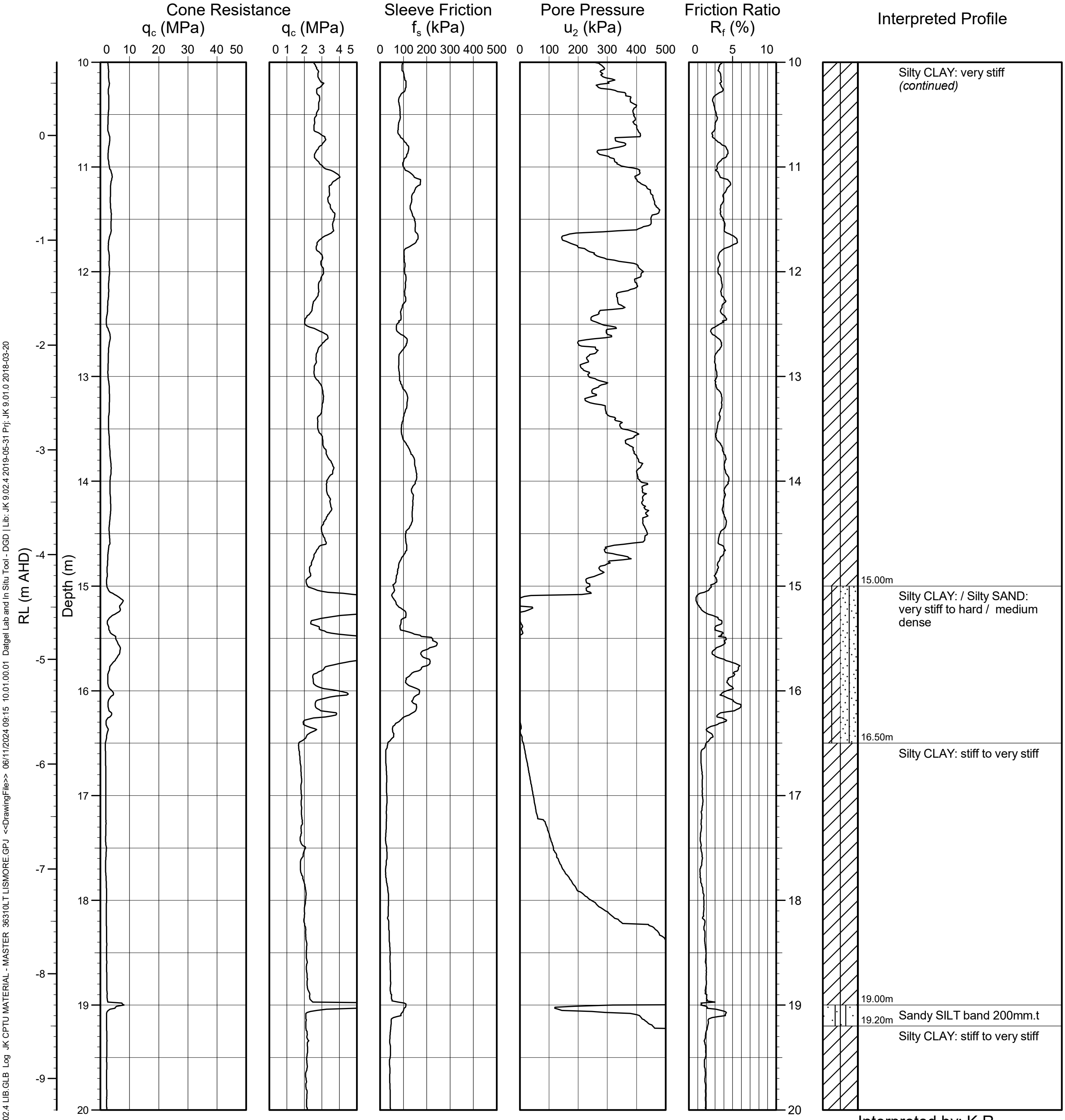
CONE PENETROMETER TEST RESULTS

CPT No.
CPT3

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.7 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

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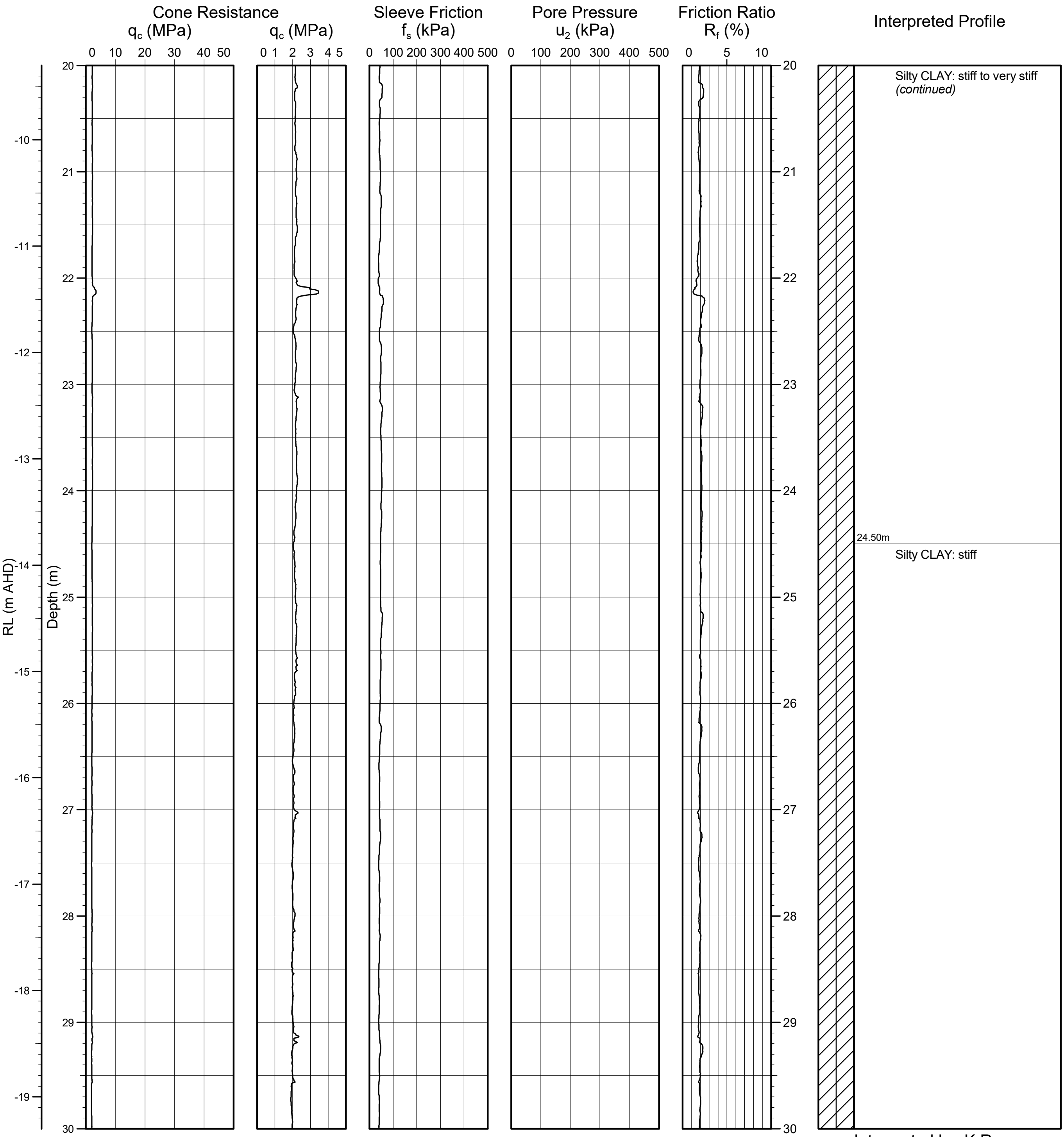
CONE PENETROMETER TEST RESULTS

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.7 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

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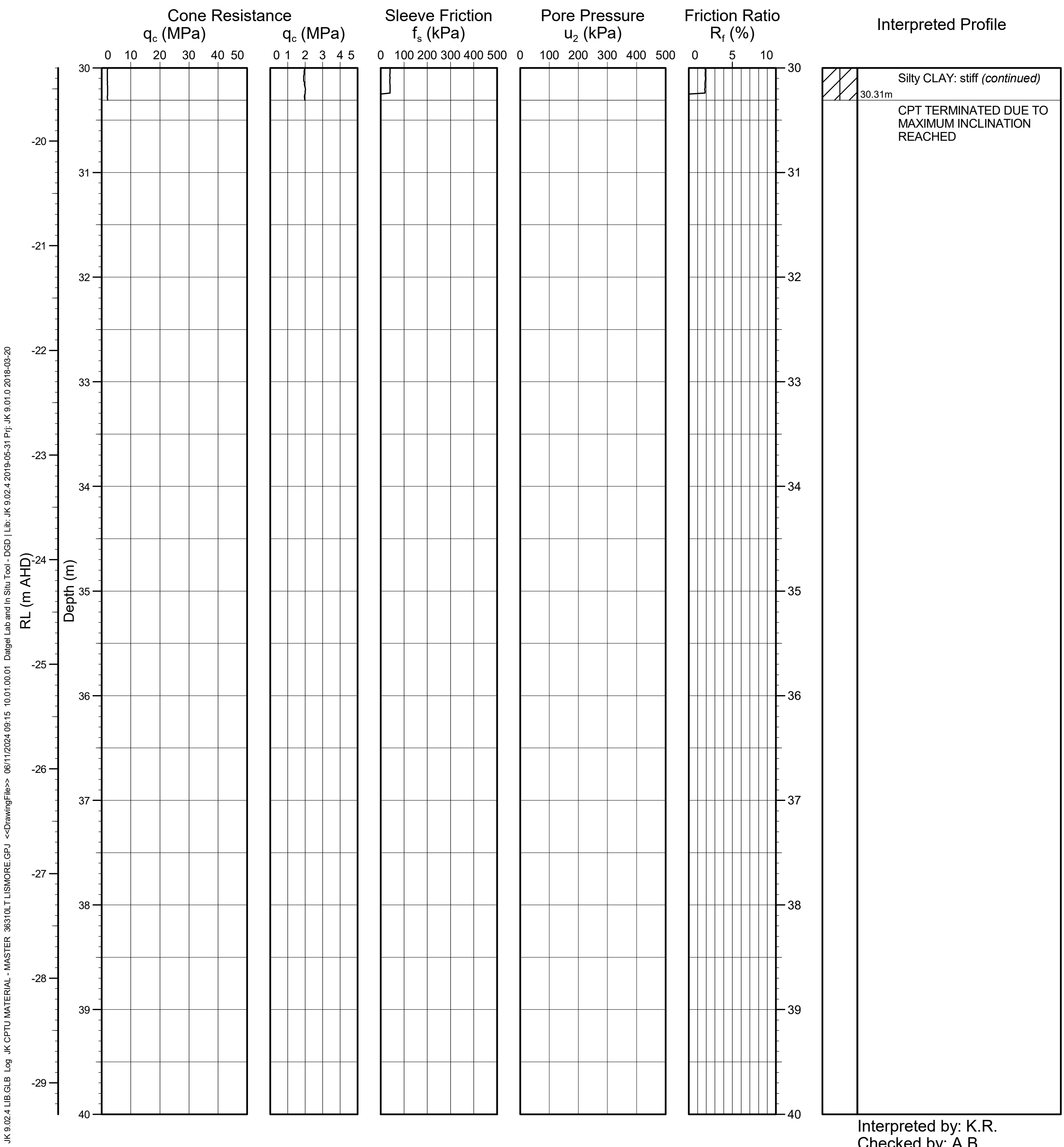
CONE PENETROMETER TEST RESULTS

CPT No. CPT3

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **R.L. Surface:** ~10.7 m **Data File:** 36310BT South Lismore
Date: 26/9/24 **Datum:** AHD **Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

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CONE PENETROMETER TEST RESULTS

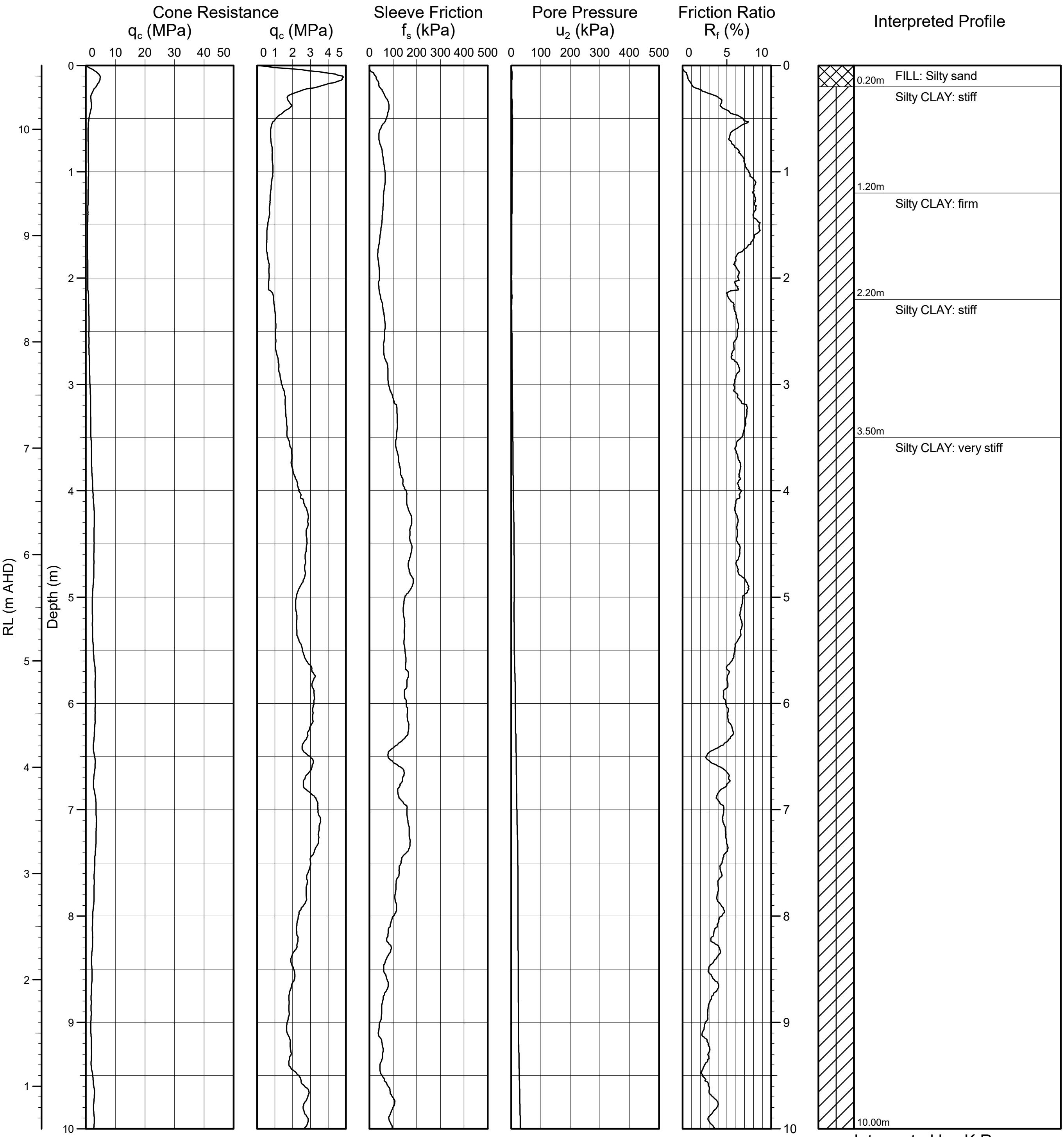
CPT No.
CPT6
1 / 4

Client: SCHOOL INFRASTRUCTURE NSW

Project: PROPOSED SCHOOL REDEVELOPMENT

Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

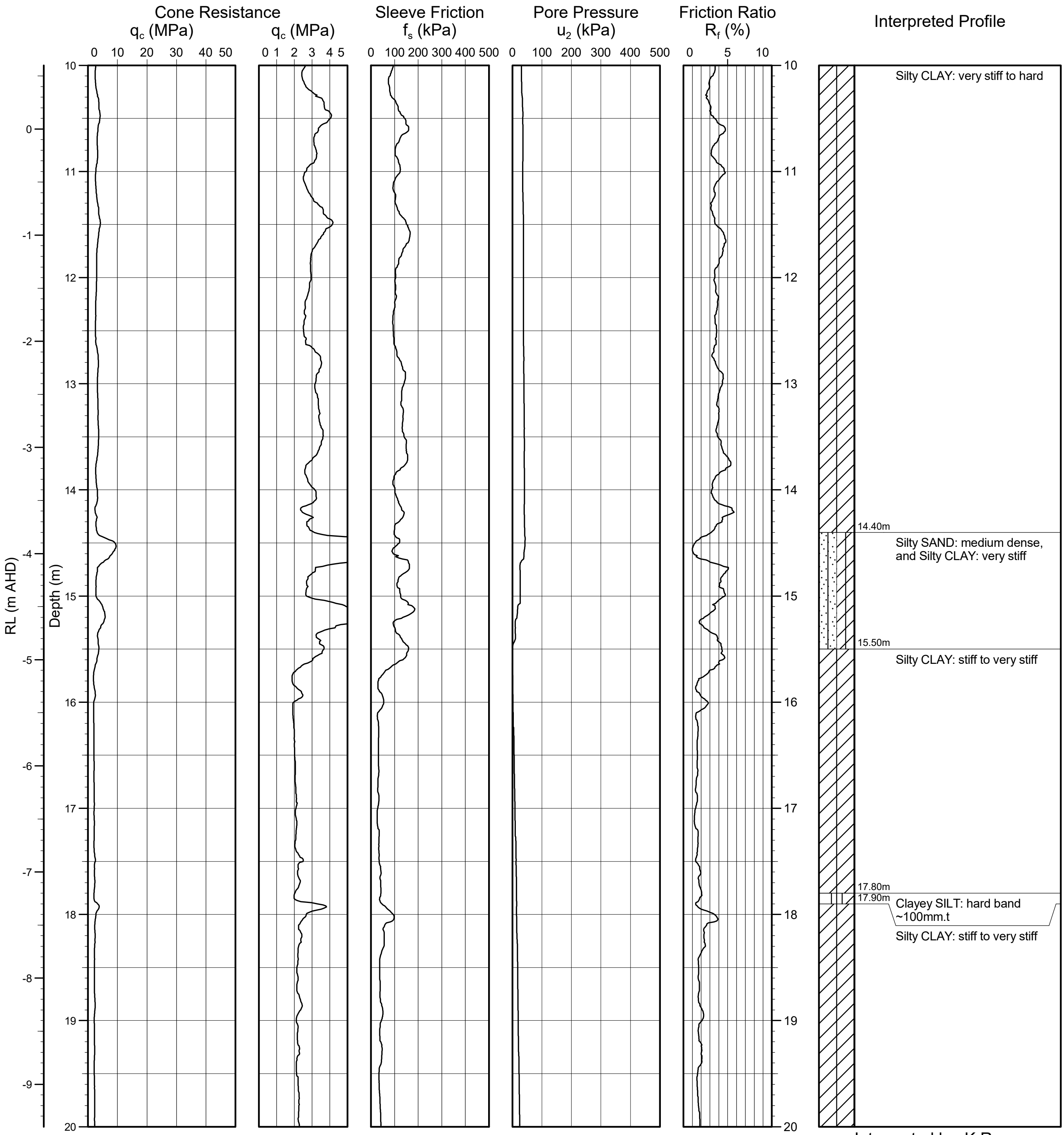
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT6
2 / 4

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

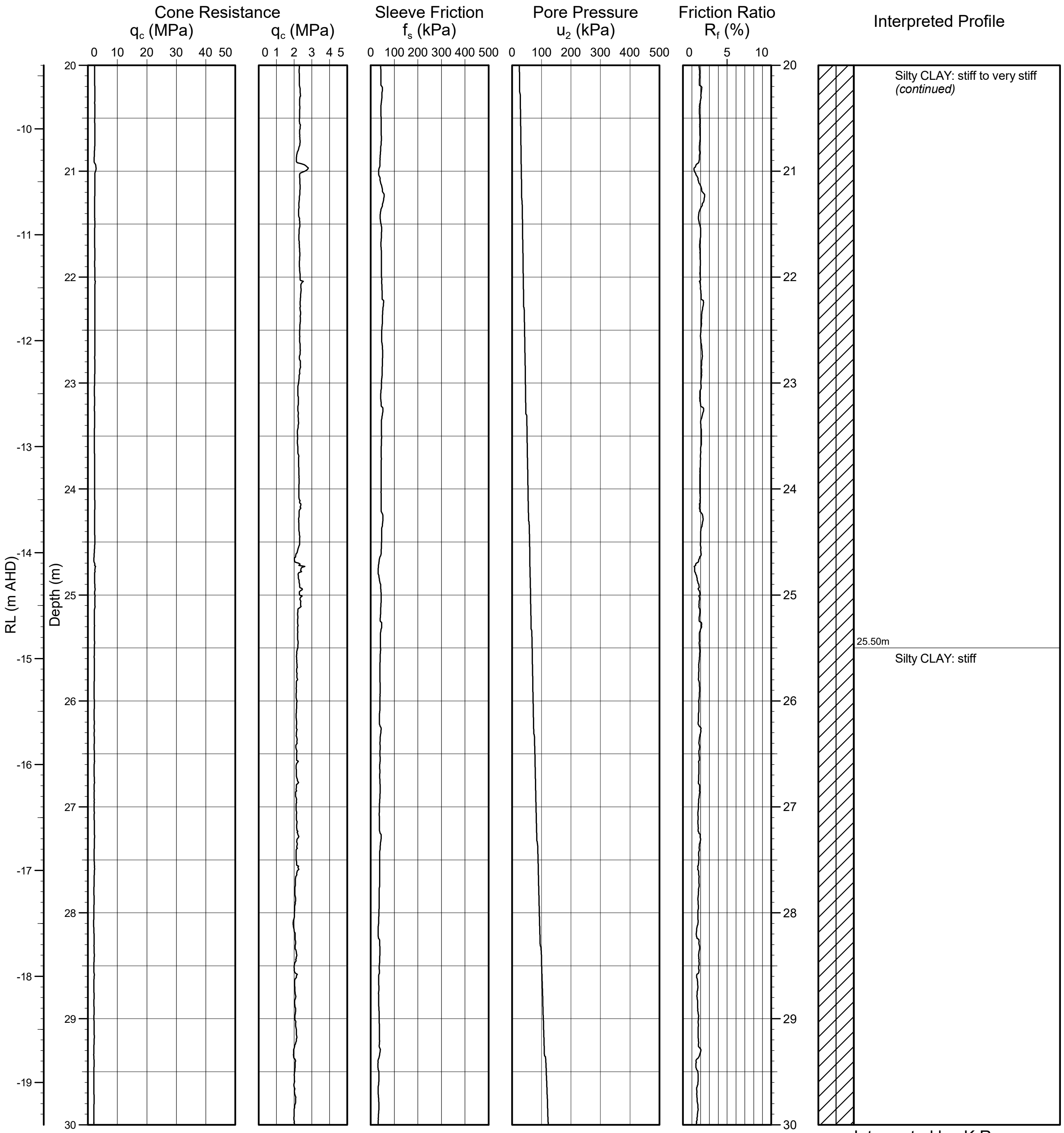
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT6

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



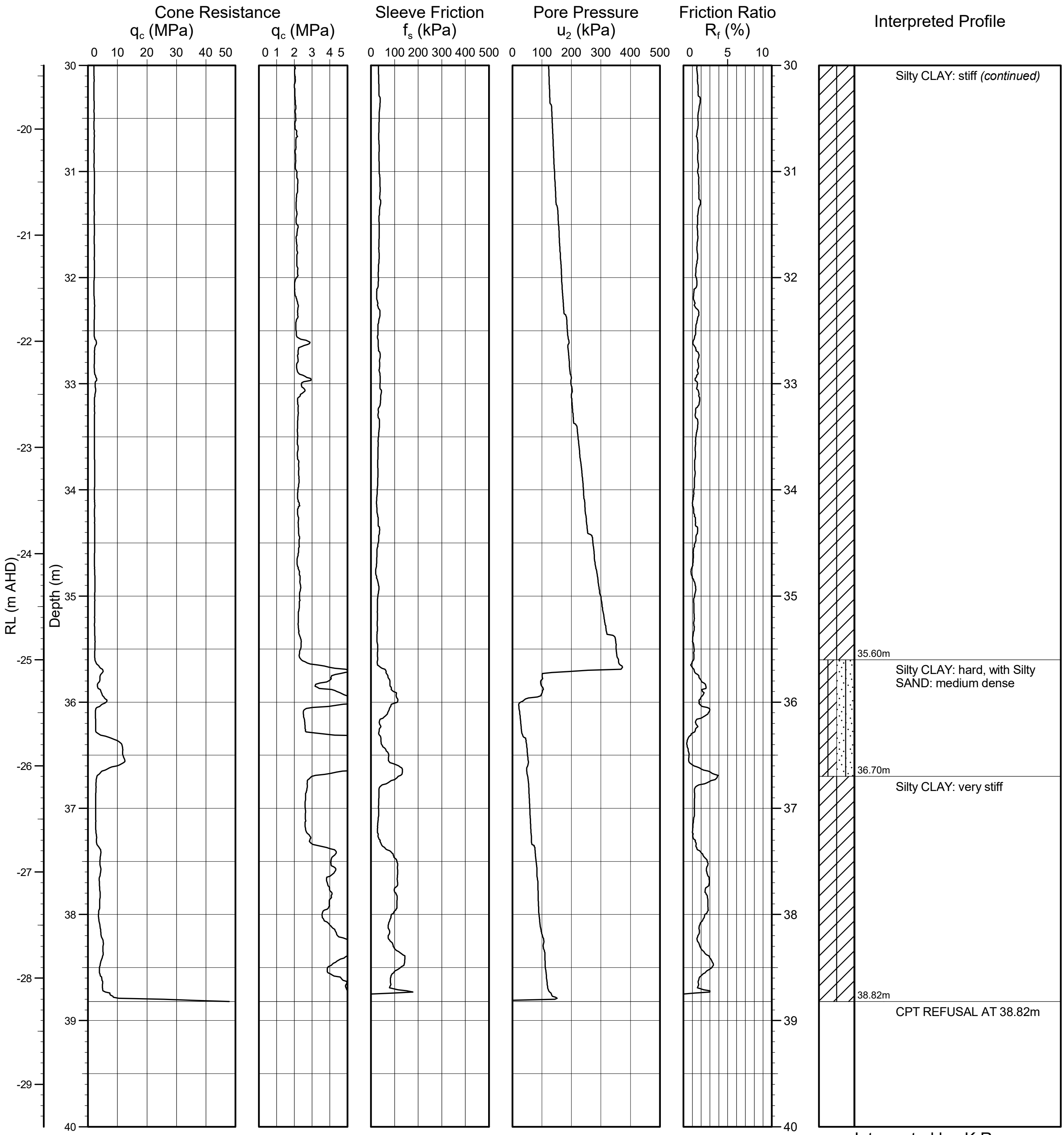
Interpreted by: K.R.
Checked by: A.B.

JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT6

Client:	SCHOOL INFRASTRUCTURE NSW	
Project:	PROPOSED SCHOOL REDEVELOPMENT	
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW	
Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

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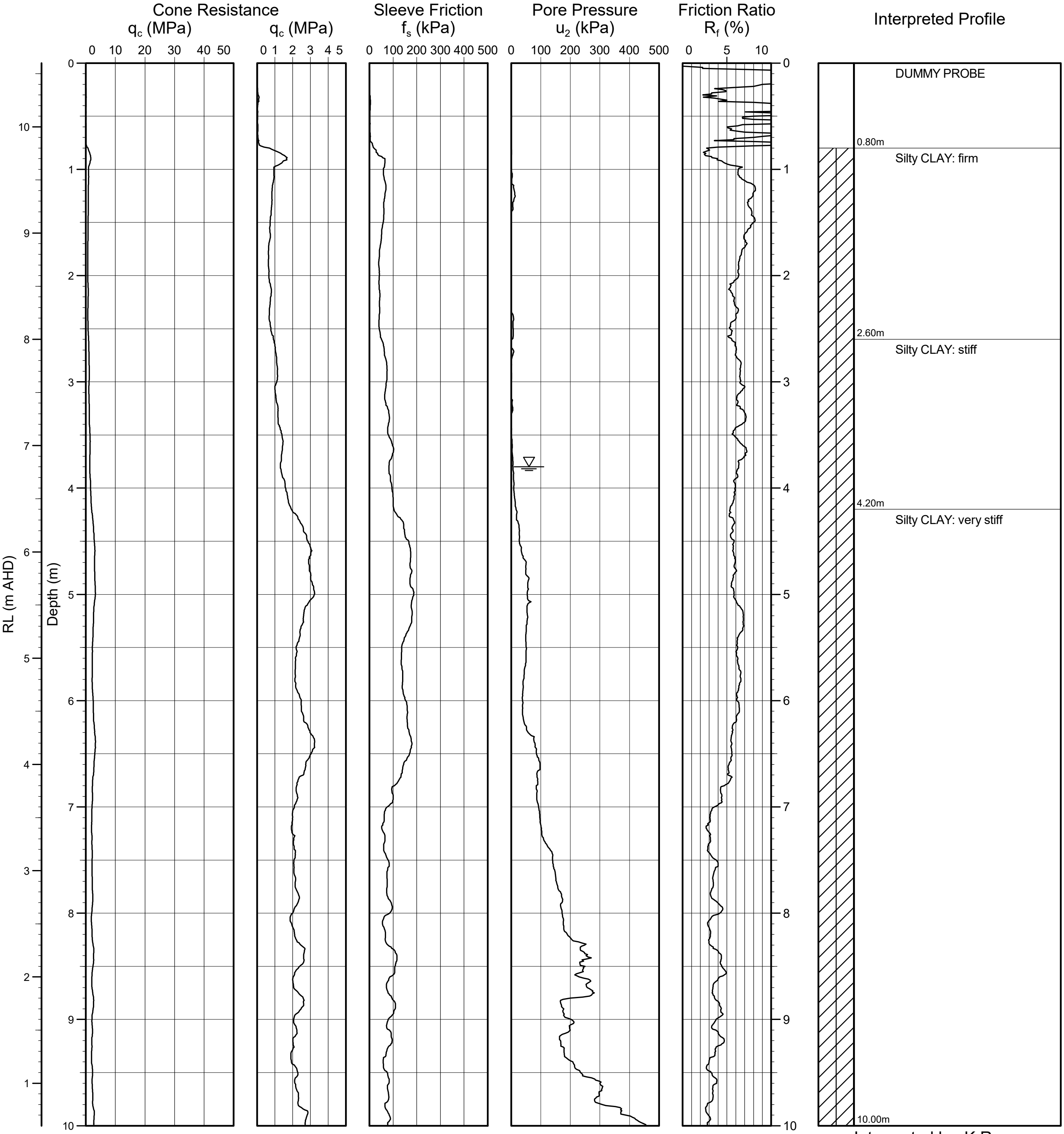
CONE PENETROMETER TEST RESULTS

CPT No.
CPT15
1 / 5

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

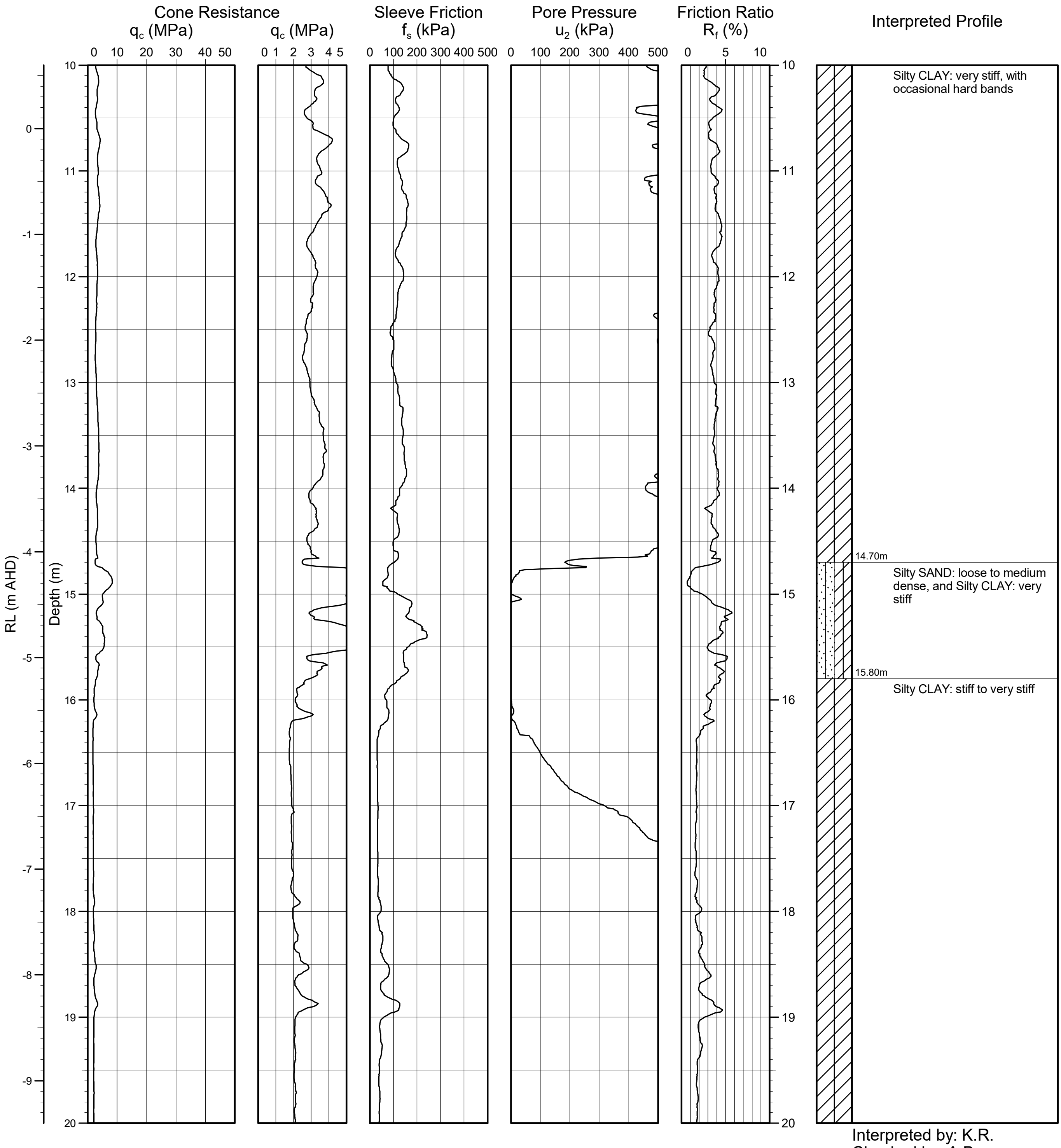
JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 36310LT LISMORE.GPJ <<DrawingFile>> 06/11/2024 09:15 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

CONE PENETROMETER TEST RESULTS

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

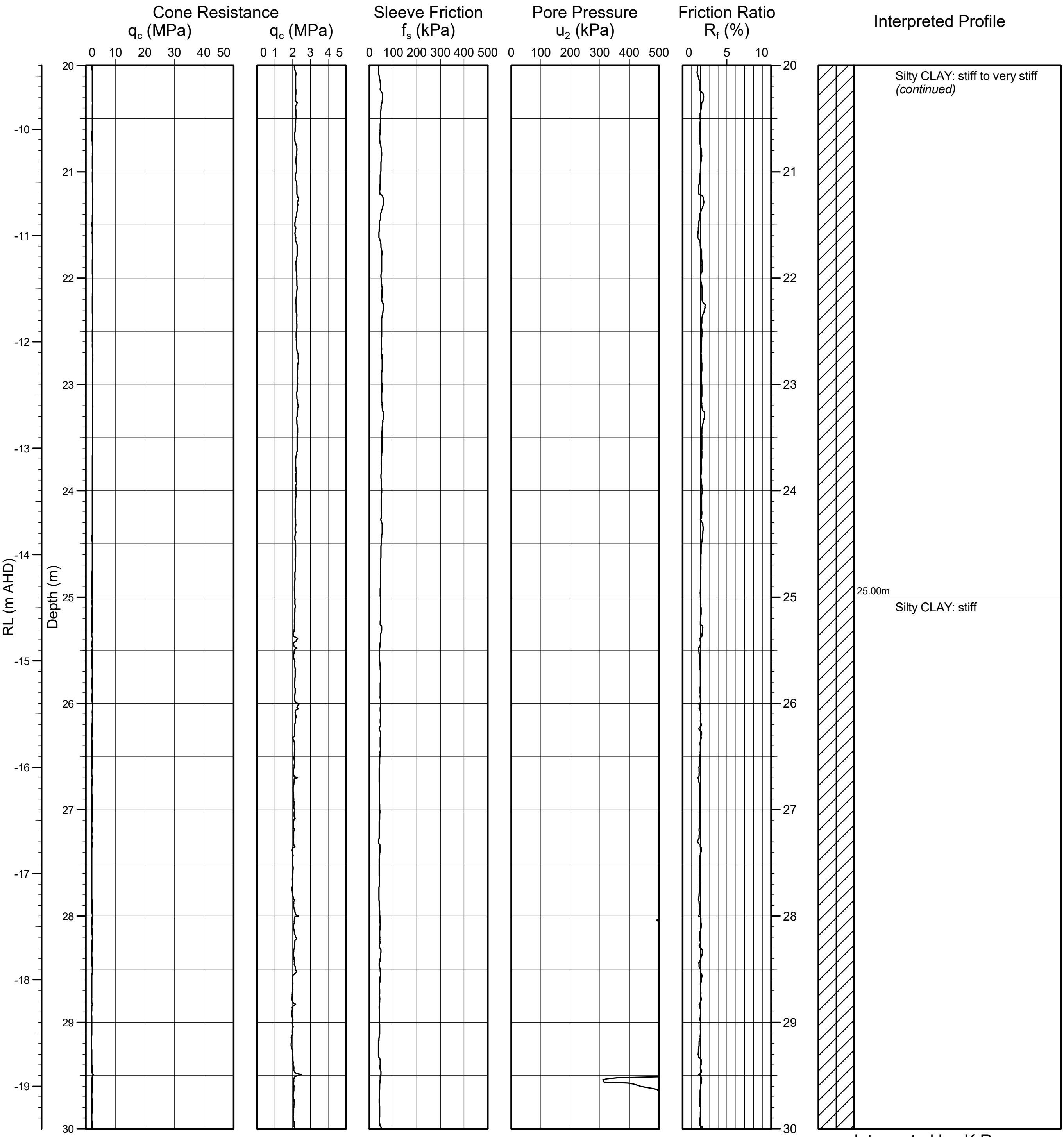
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT15

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

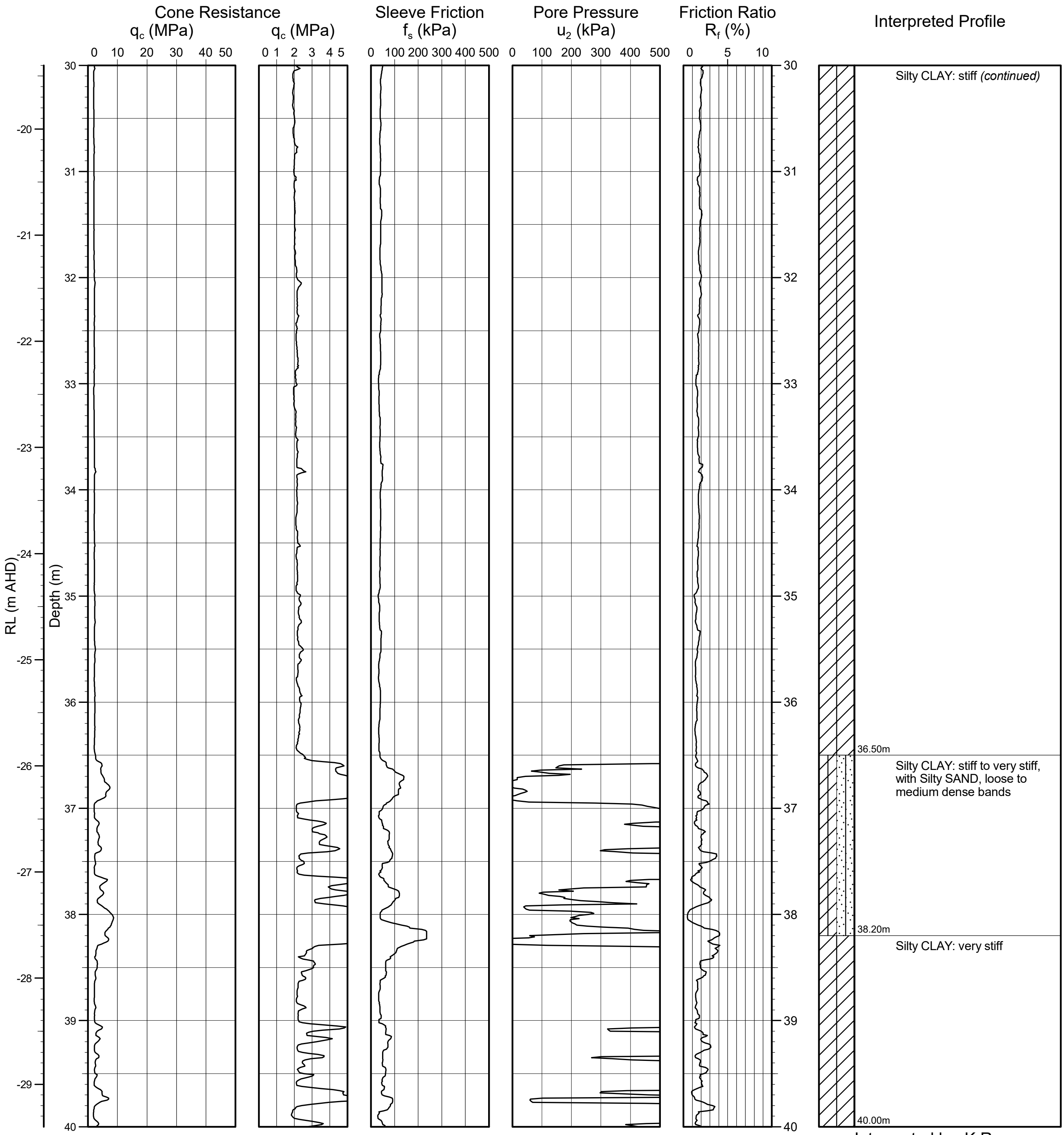
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT15

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

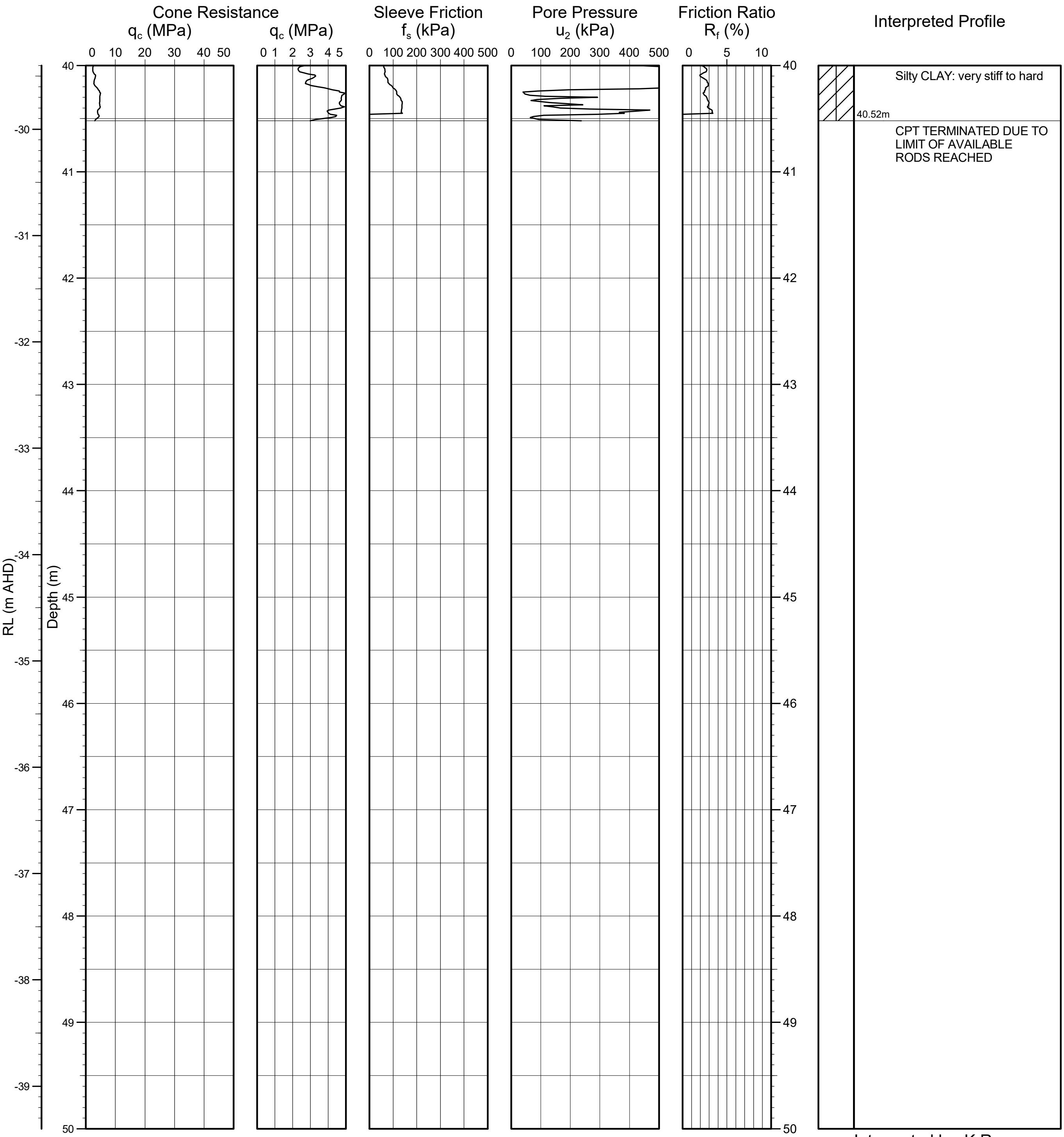
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT15
5 / 5

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

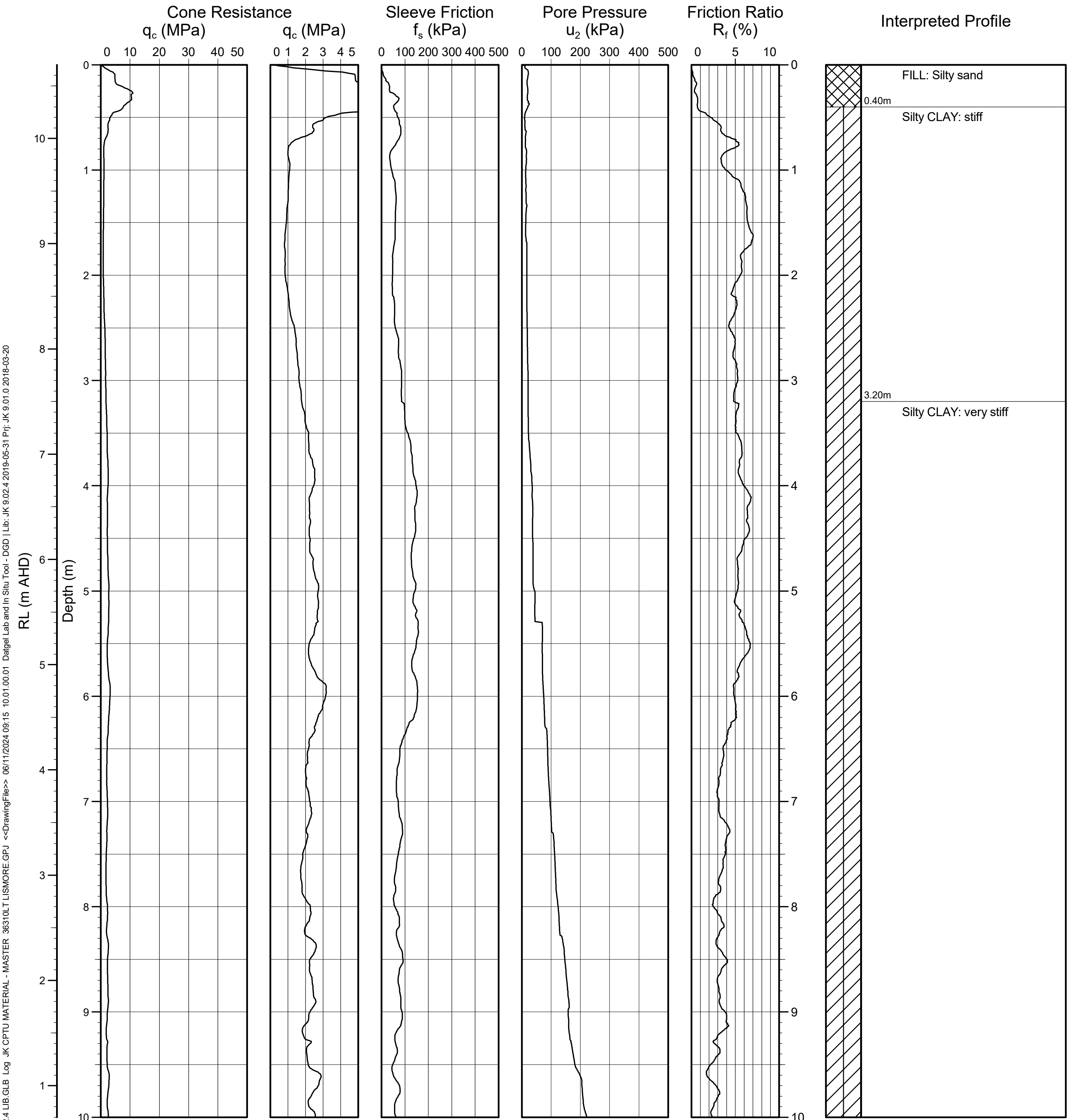
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT17
1 / 4

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT | **R.L. Surface:** ~10.7 m | **Data File:** 36310BT South Lismore
Date: 25/9/24 | **Datum:** AHD | **Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 36310LT LISMORE.GPJ <<DrawingFile>> 06/11/2024 09:15 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

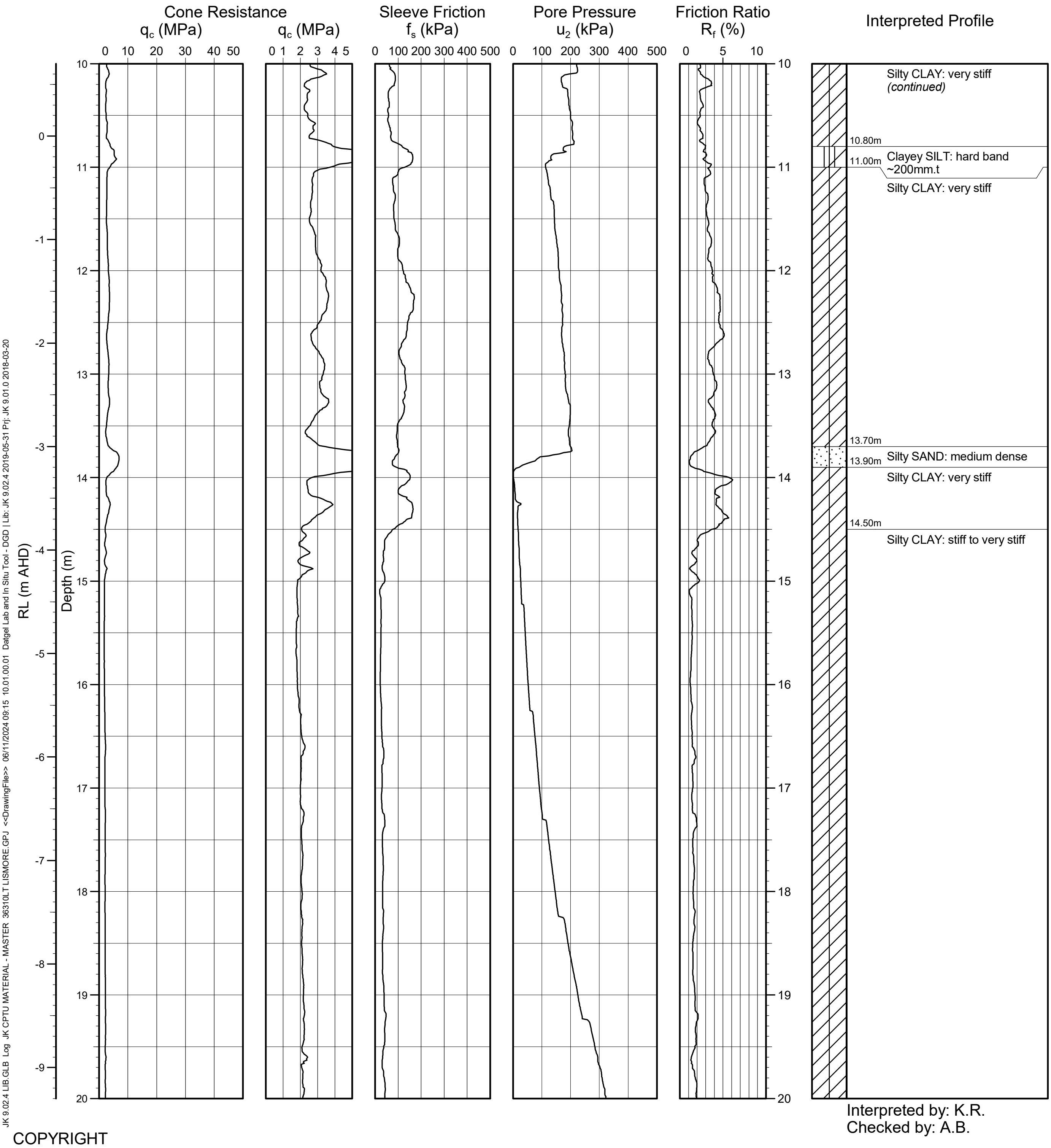
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT17
2 / 4

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.7 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

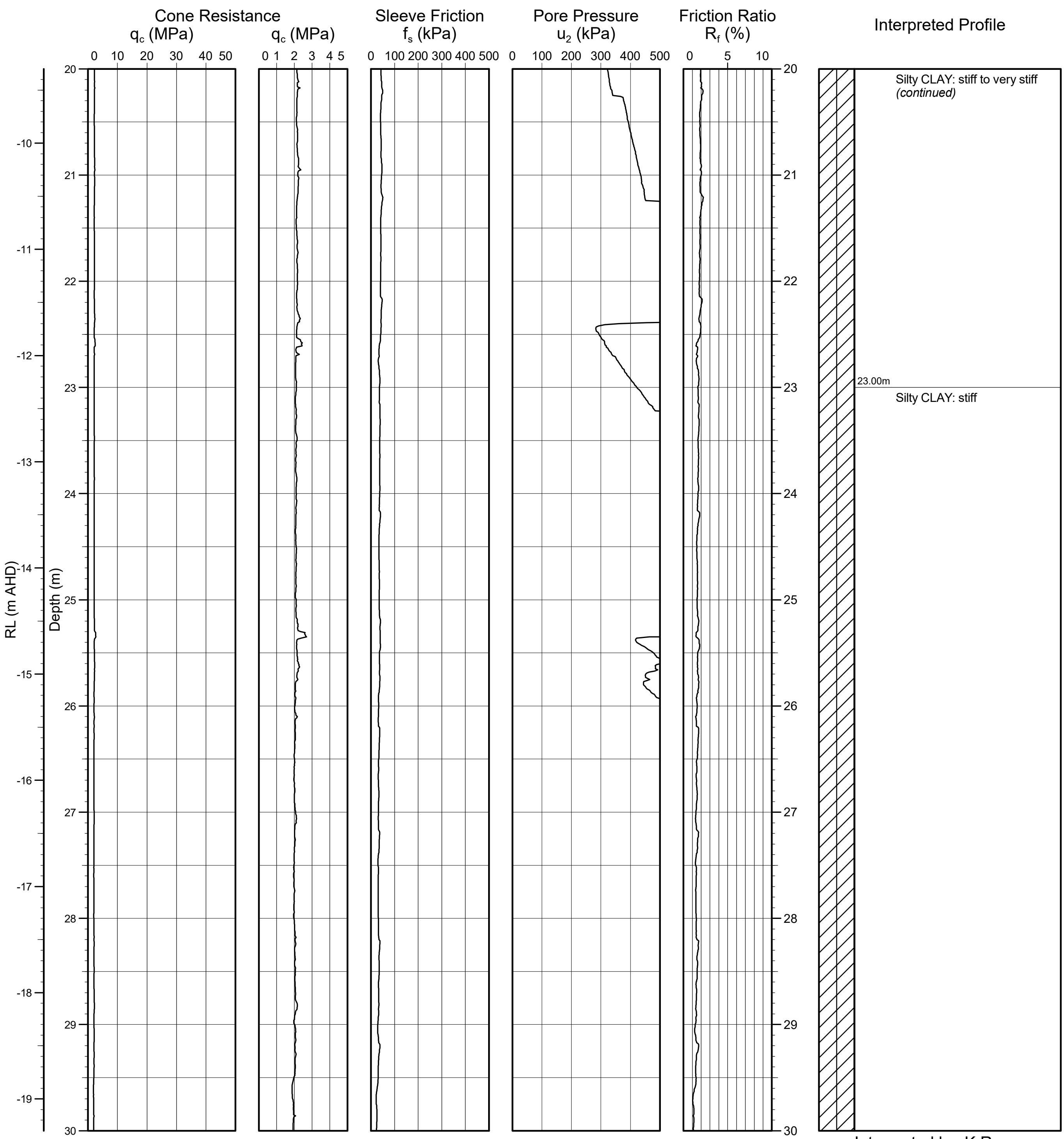
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT17
3 / 4

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.7 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

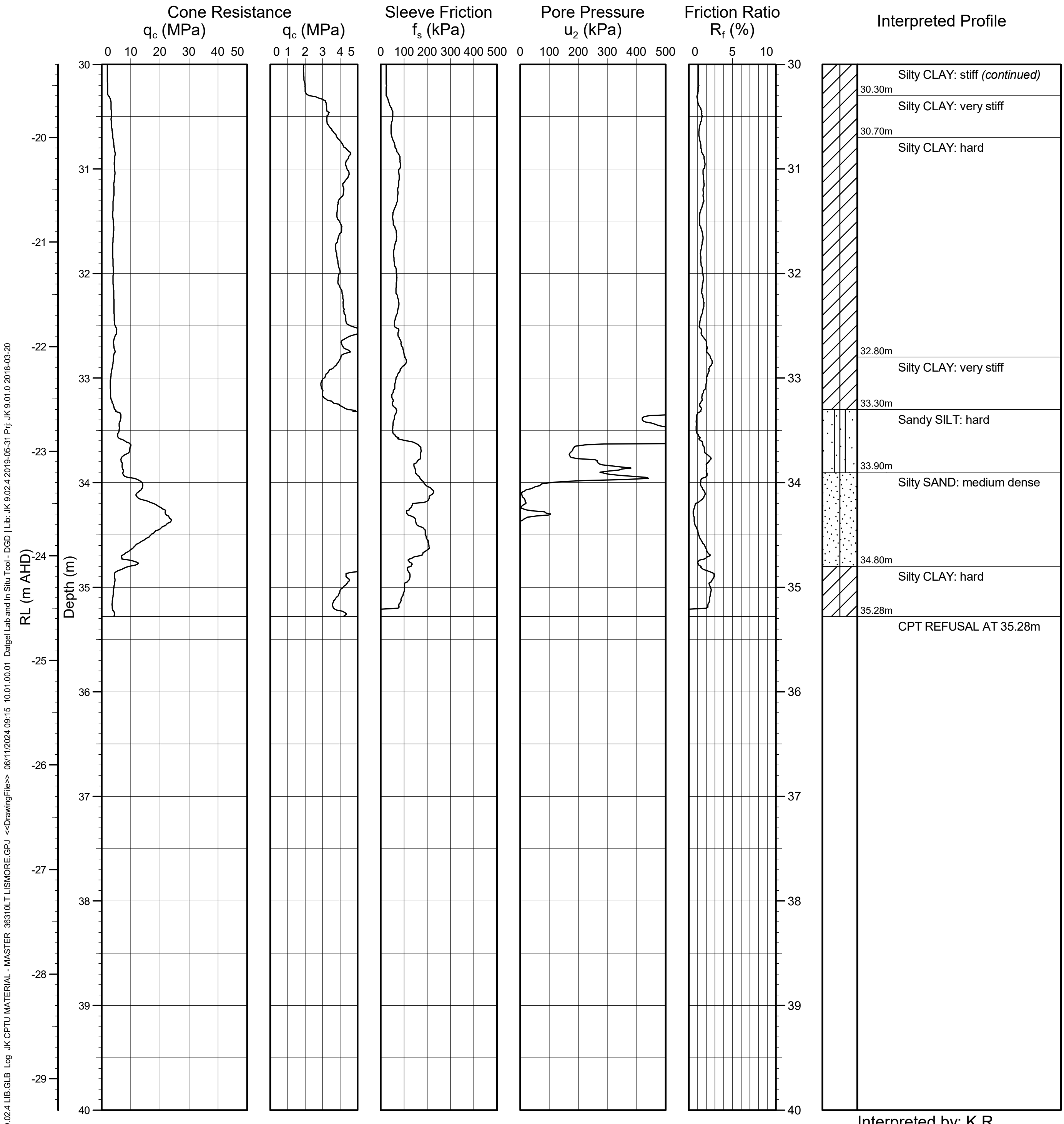
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT17
4 / 4

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.7 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

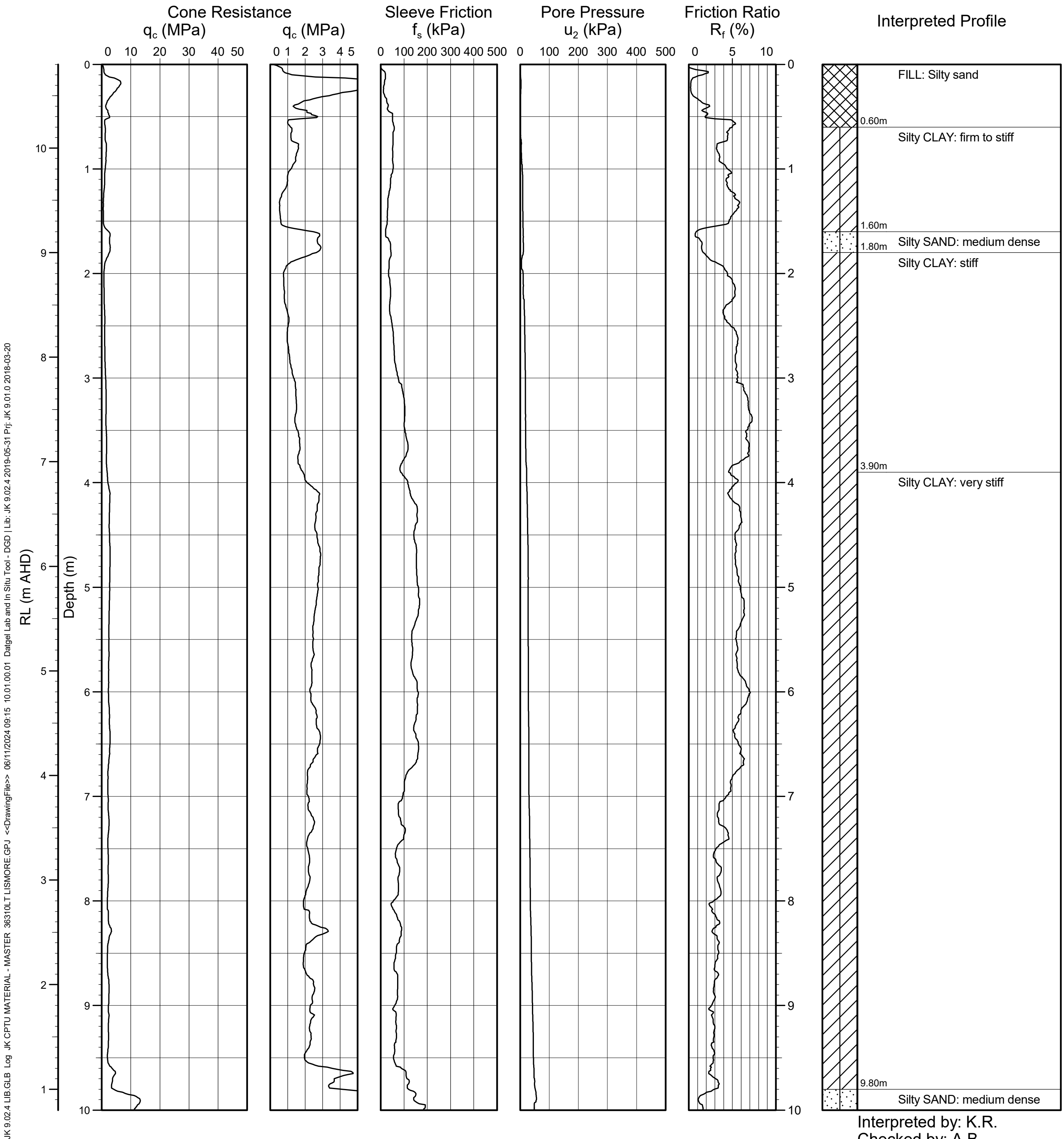
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT18

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.8 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

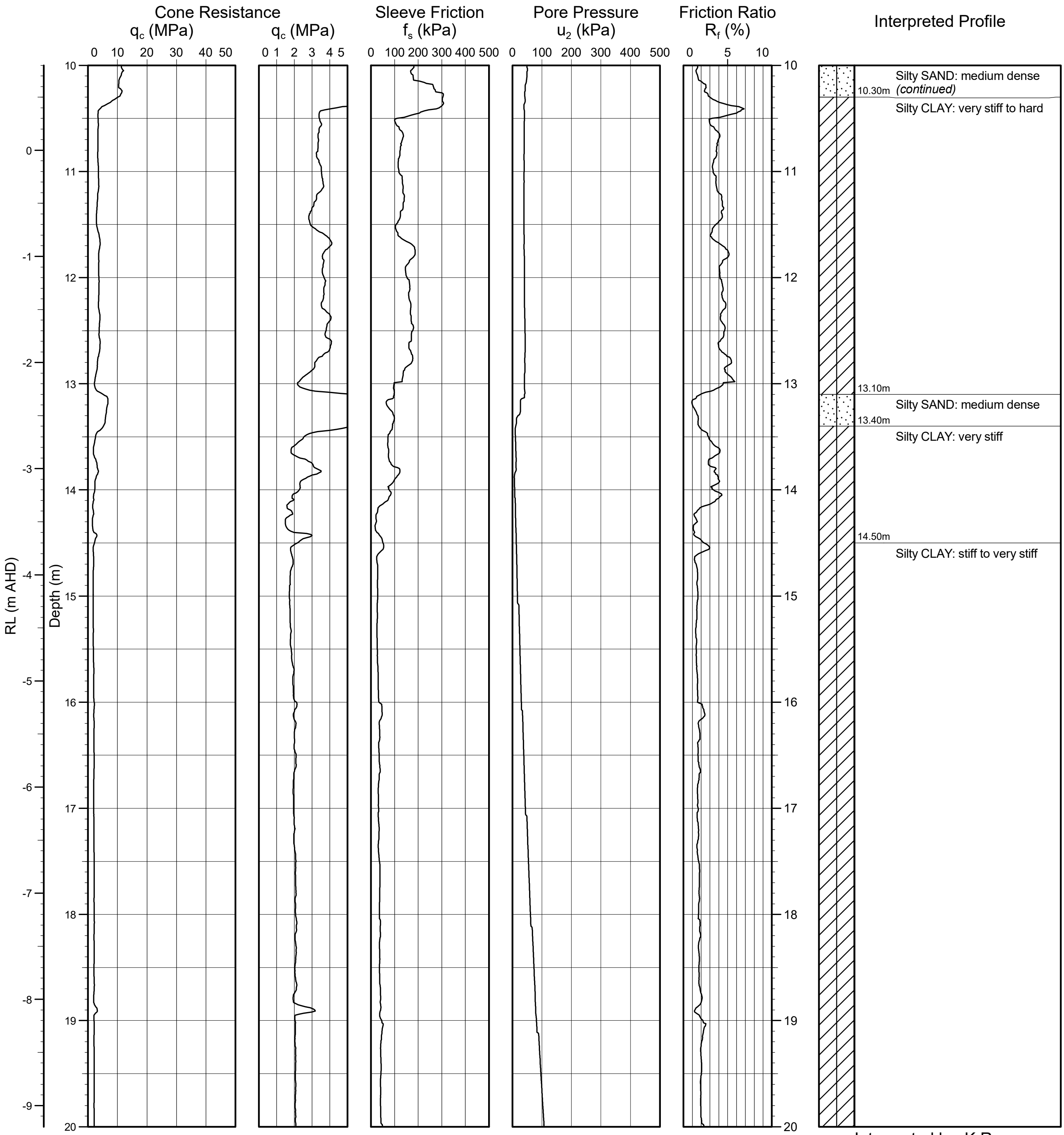
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT18
2 / 4

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.8 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 36310LT LISMORE.GPJ <<DrawingFile>> 06/11/2024 09:15 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

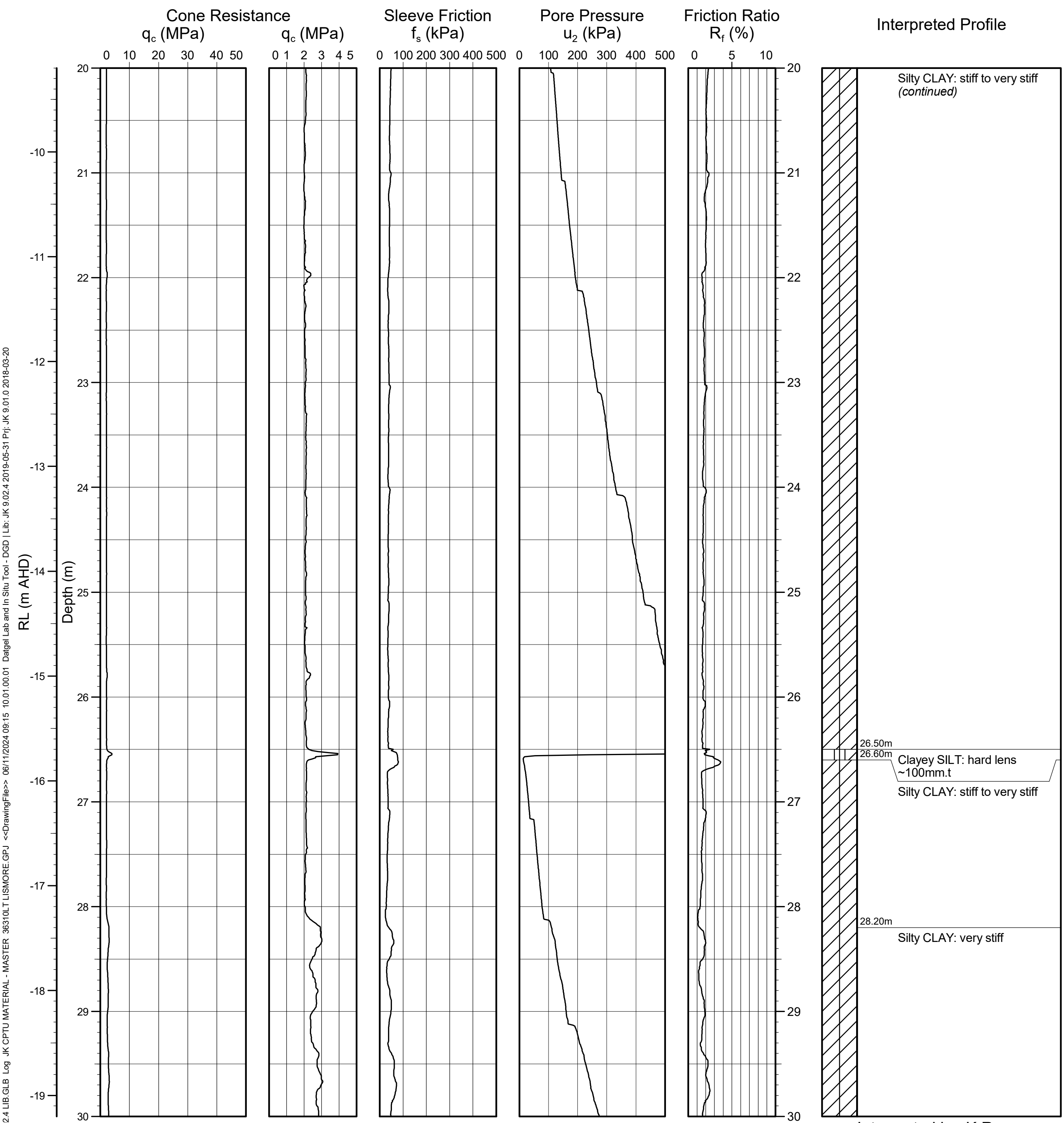
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT18
3 / 4

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.8 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

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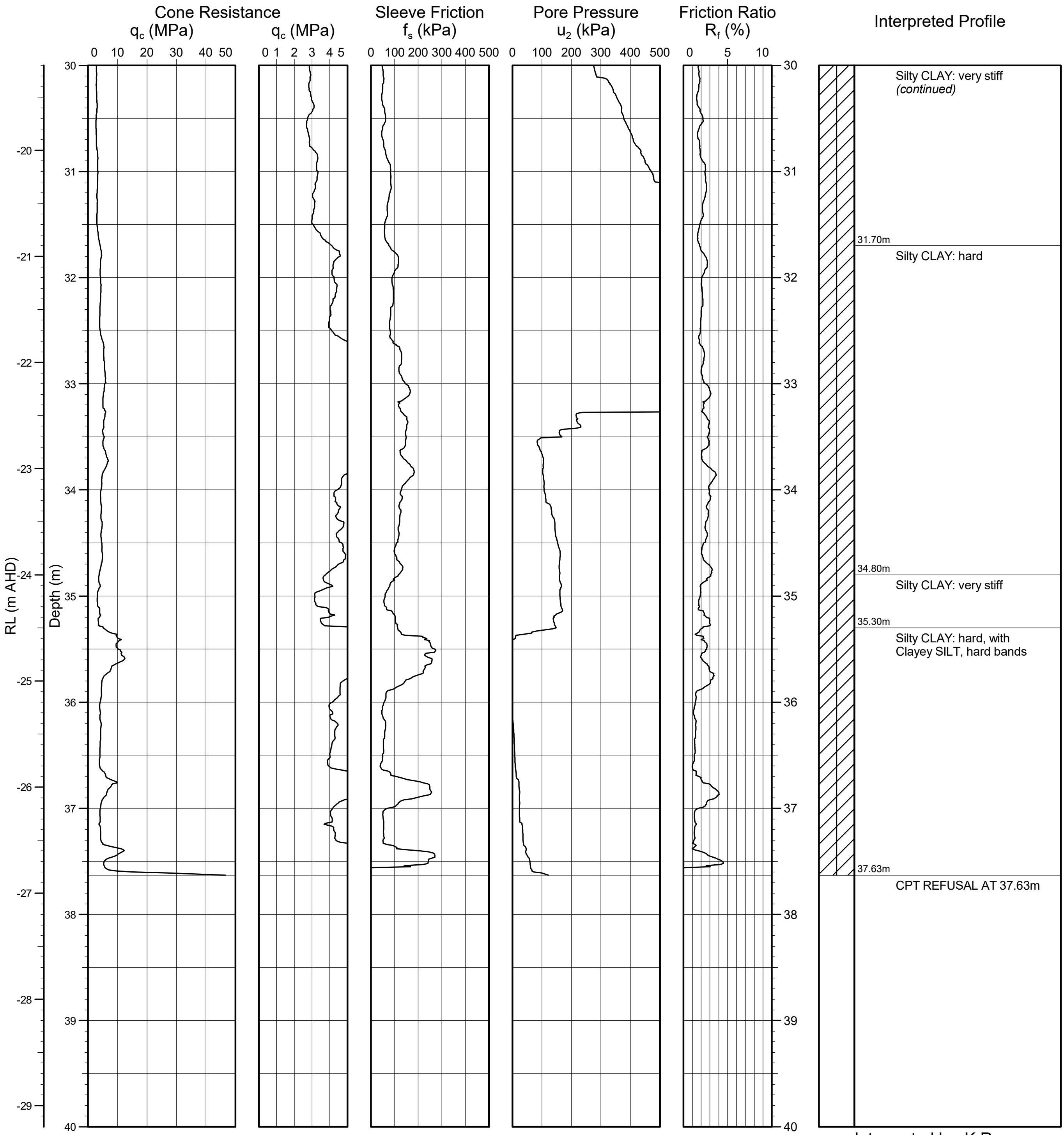
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT18

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.8 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

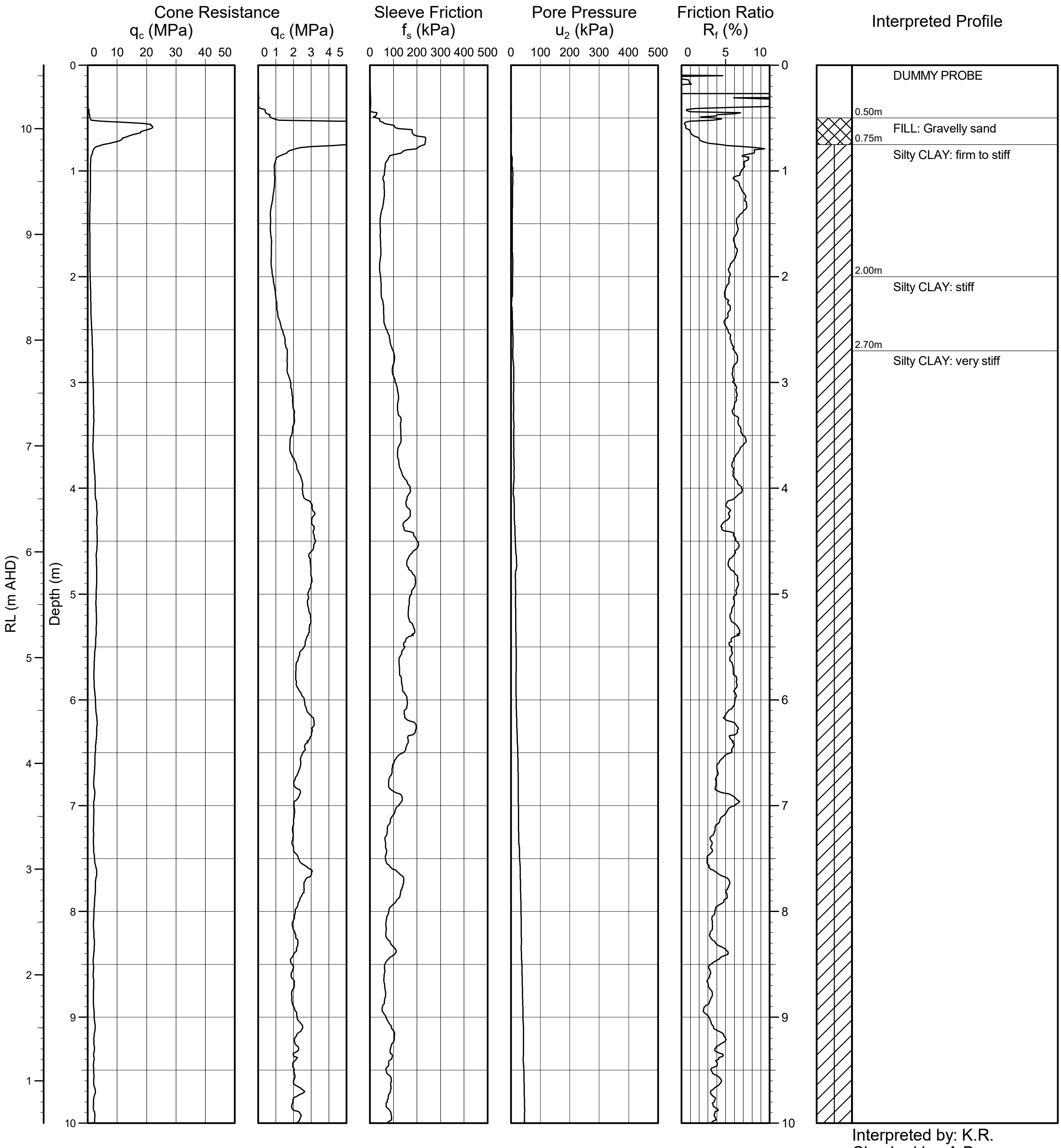
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT20

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



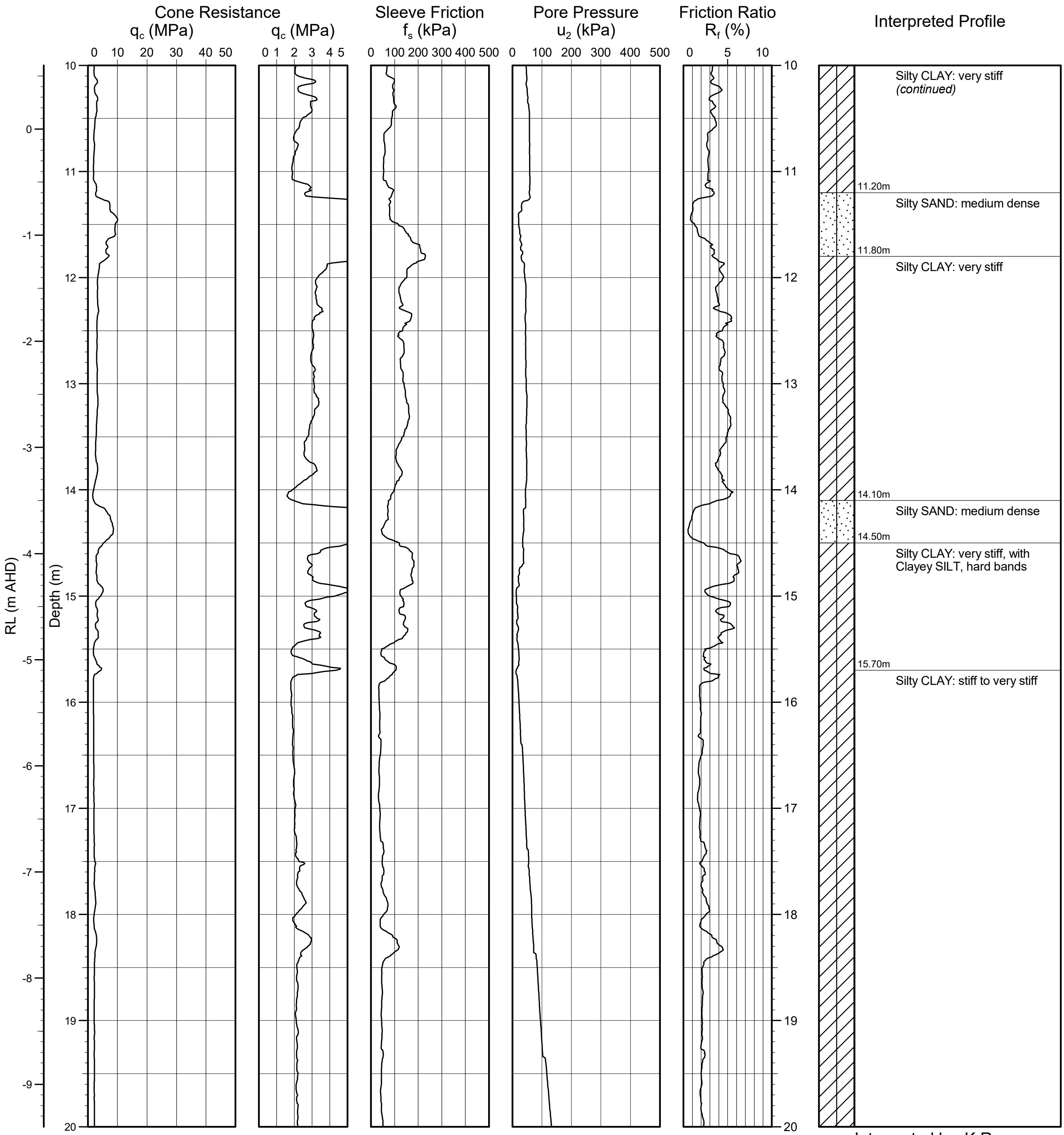
Interpreted by: K.R.
Checked by: A.B.

JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT20
2 / 4

Client: SCHOOL INFRASTRUCTURE NSW		
Project: PROPOSED SCHOOL REDEVELOPMENT		
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW		
Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

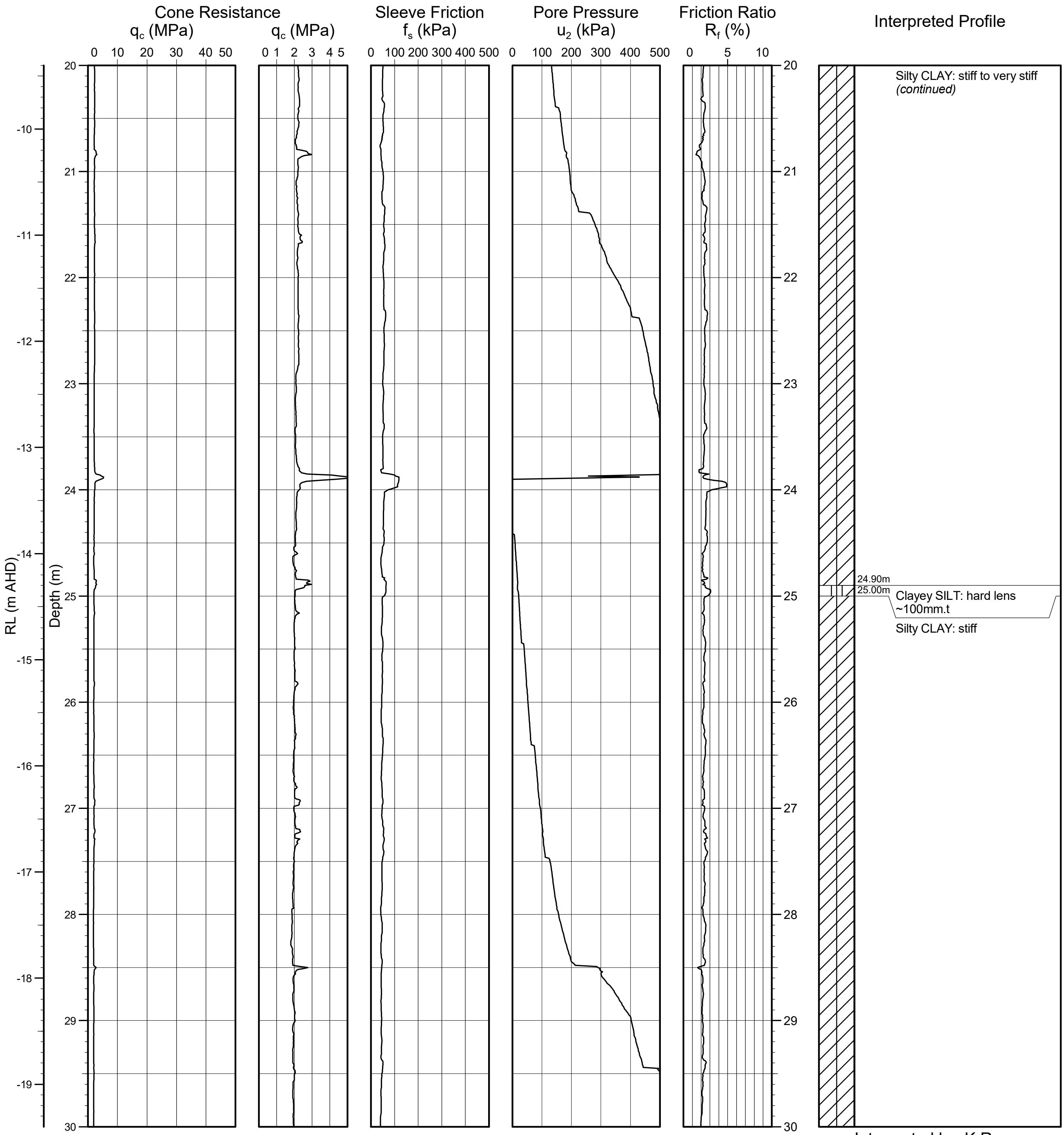
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT20

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT | **R.L. Surface:** ~10.6 m | **Data File:** 36310BT South Lismore
Date: 26/9/24 | **Datum:** AHD | **Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

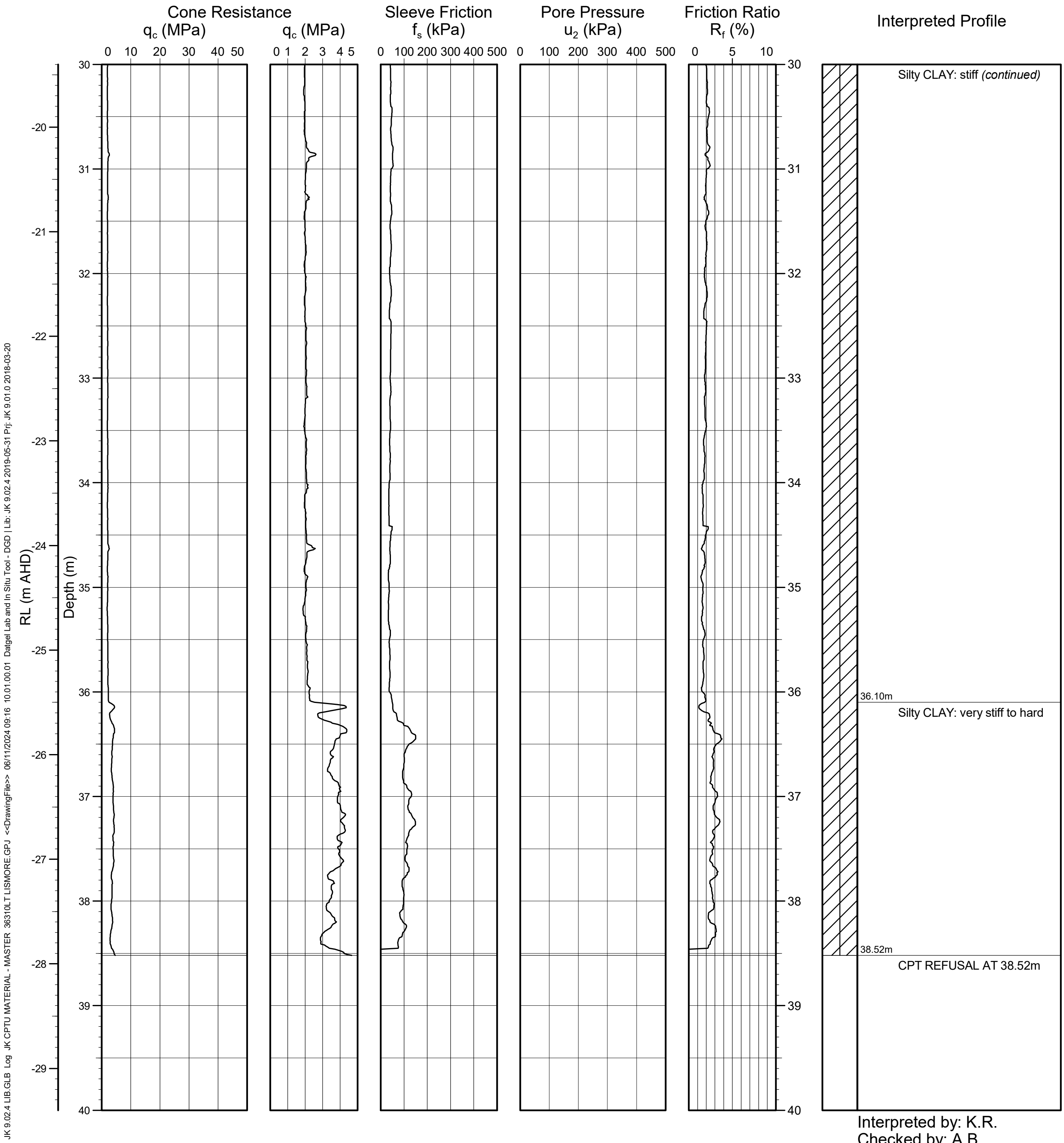
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT20
4 / 4

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 26/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

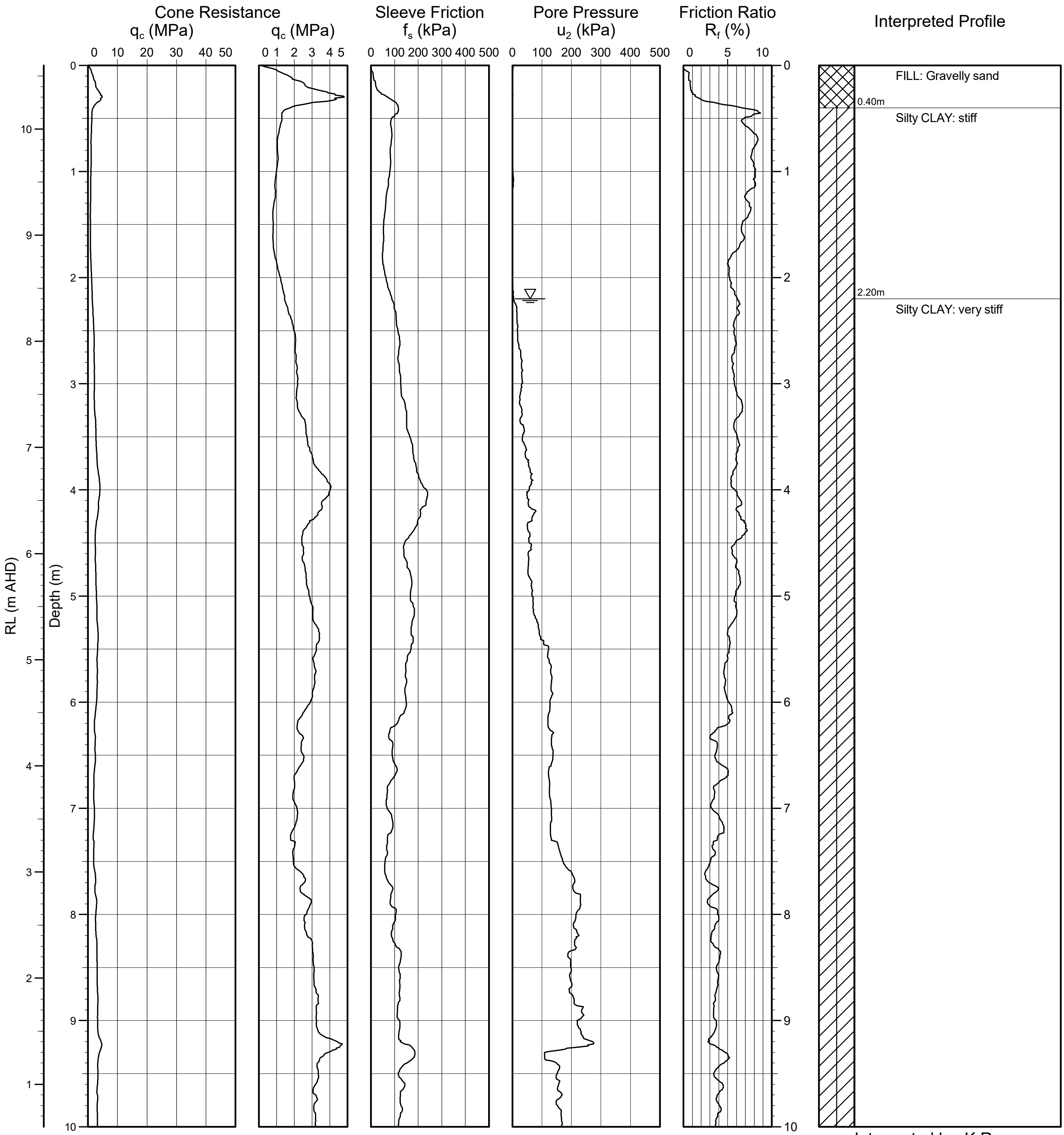
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT22

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 36310LT LISMORE.GPJ <<DrawingFile>> 06/11/2024 09:16 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

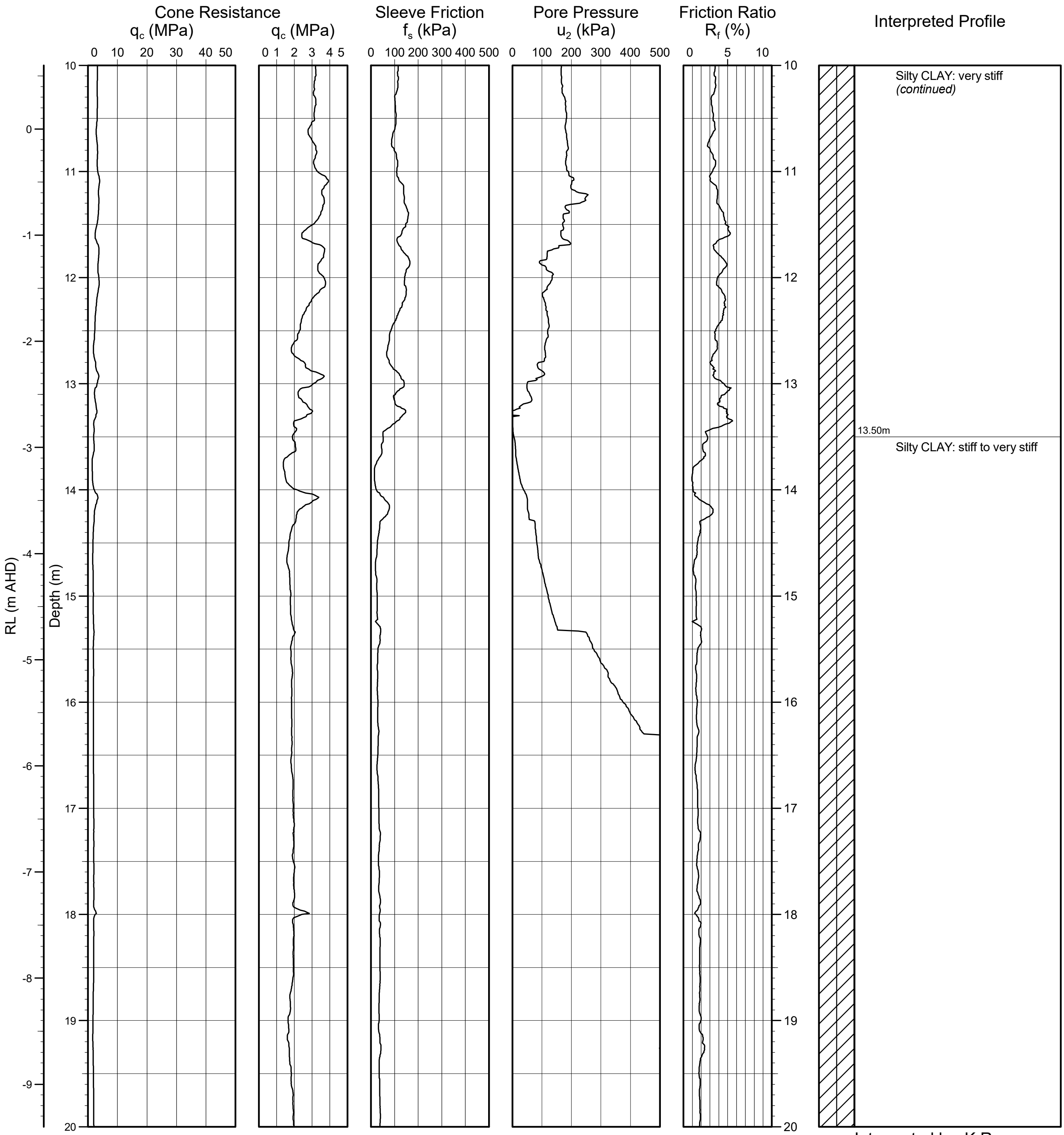
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT22

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **R.L. Surface:** ~10.6 m **Data File:** 36310BT South Lismore
Date: 25/9/24 **Datum:** AHD **Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

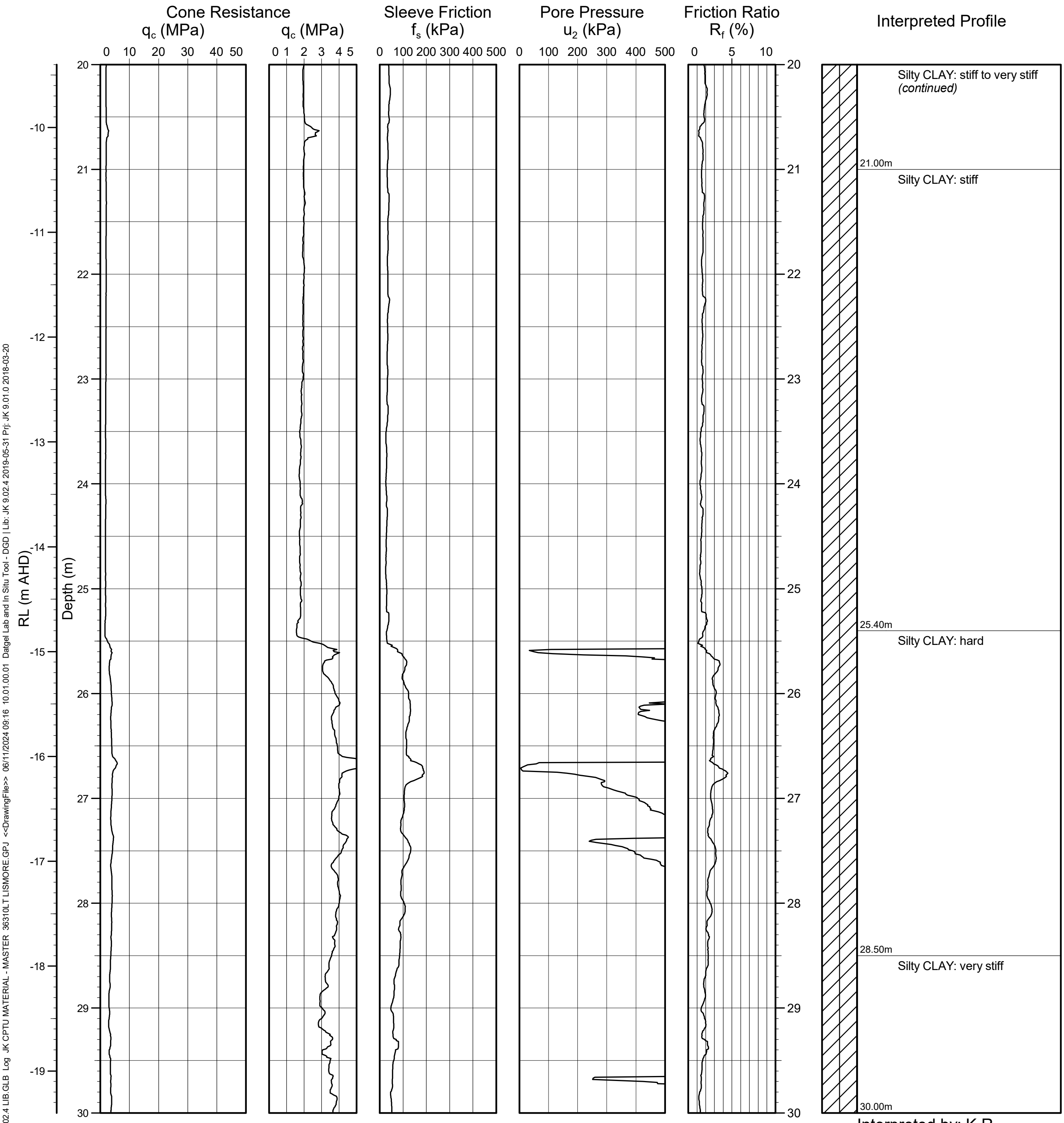
JKGeotechnics

CONE PENETROMETER TEST RESULTS

CPT No.
CPT22

Client: SCHOOL INFRASTRUCTURE NSW
Project: PROPOSED SCHOOL REDEVELOPMENT
Location: LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT **R.L. Surface:** ~10.6 m **Data File:** 36310BT South Lismore
Date: 25/9/24 **Datum:** AHD **Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.

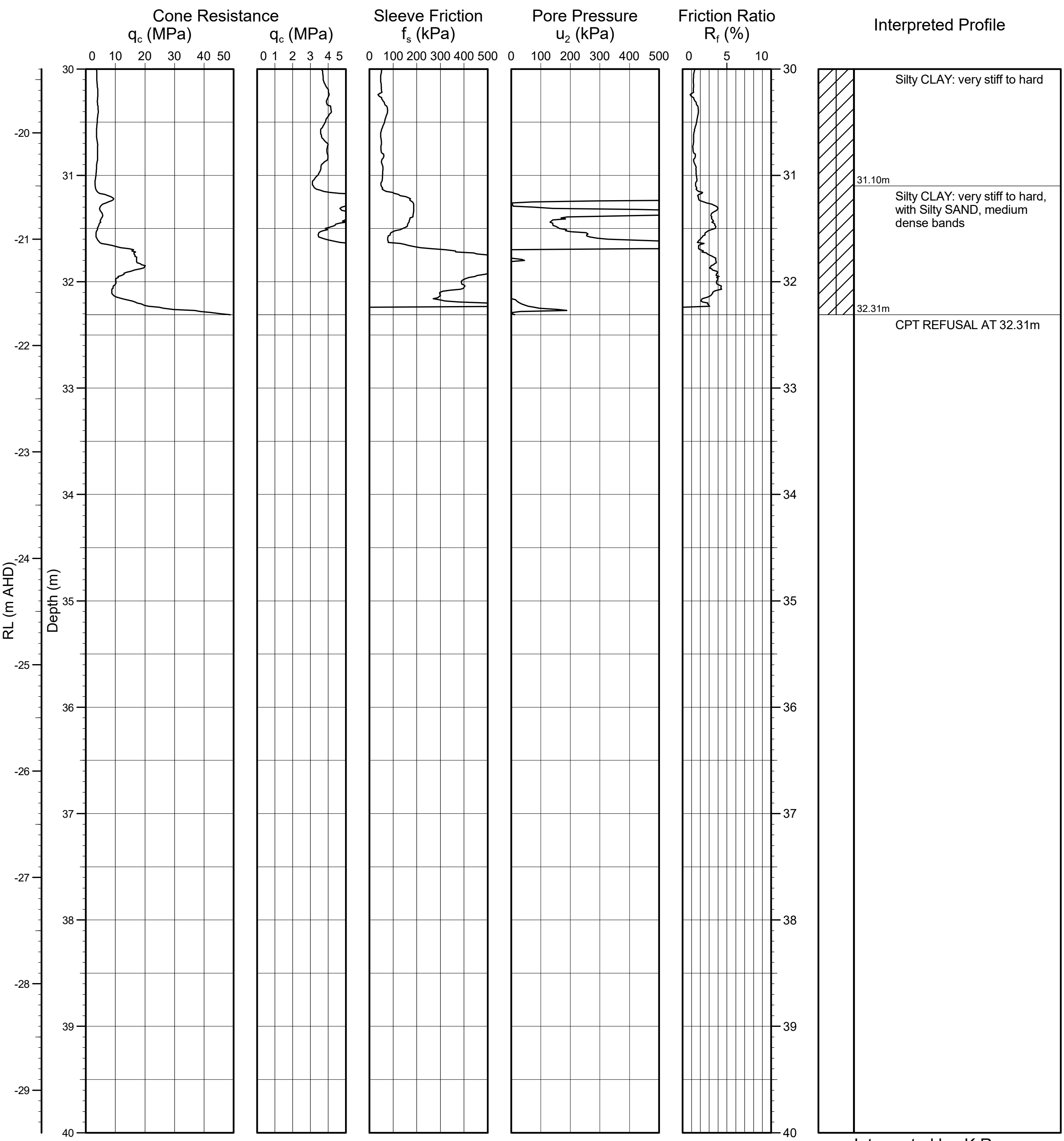
JKGeotechnics

CONE PENETROMETER TEST RESULTS

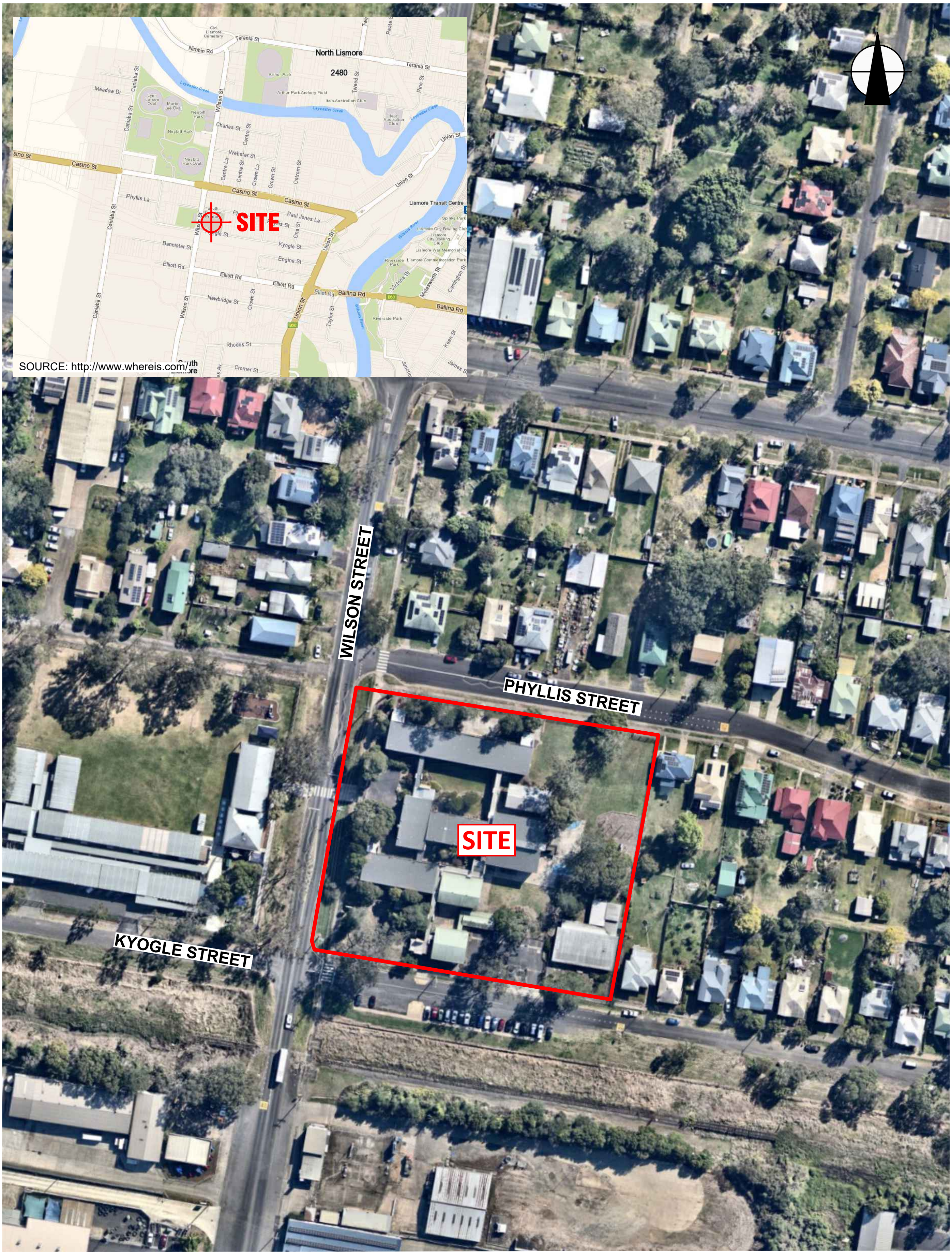
CPT No.
CPT22

Client:	SCHOOL INFRASTRUCTURE NSW
Project:	PROPOSED SCHOOL REDEVELOPMENT
Location:	LISMORE SOUTH PUBLIC SCHOOL, 69-79 KYOGLE STREET, SOUTH LISMORE, NSW

Job No.: 36310LT	**R.L. Surface:** ~10.6 m	**Data File:** 36310BT South Lismore
Date: 25/9/24	**Datum:** AHD	**Operator:** B.J.



Interpreted by: K.R.
Checked by: A.B.



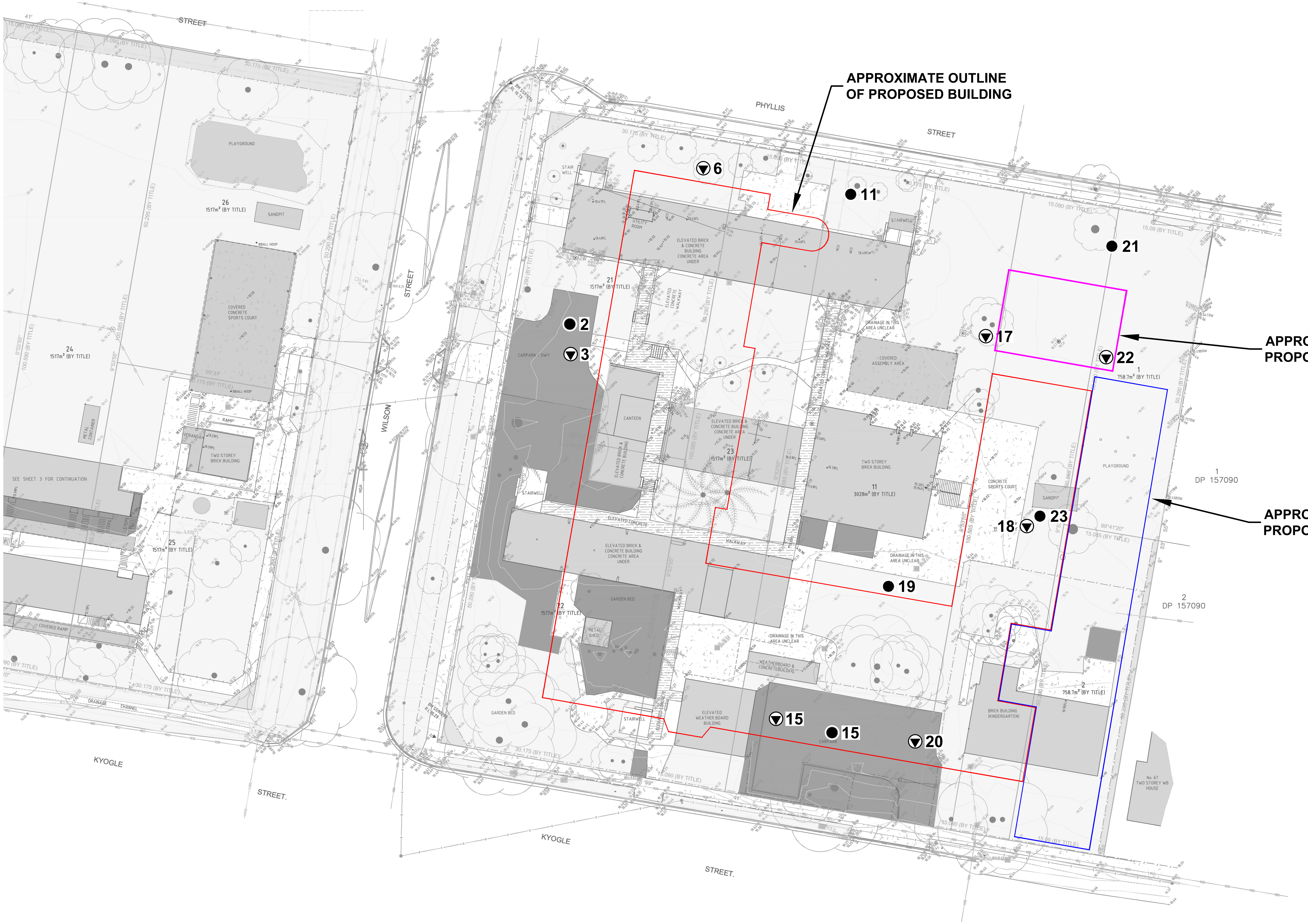
PLOT DATE: 16/10/2024 3:34:20 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36310BT SOUTH LISMORE\CAD\36310LT.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the JK Geotechnics report.

Title:	**SITE LOCATION PLAN**
Location:	69-79 KYOGLE STREET, LISMORE, NSW
Report No: 36310LT	Figure No: 1

JKGeotechnics



LEGEND

- ● BOREHOLE
- ▼ CONE PENETROMETER TEST

0 6 12 18 24 30
SCALE 1:600 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:	**INVESTIGATION LOCATION PLAN**
Location:	69-79 KYOGLE STREET, LISMORE, NSW
Report No:	36310LT
Figure No:	2

JKGeotechnics

PLOT DATE: 5/06/2025 2:19:24 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\36000'S\36310LT SOUTH LISMORE\CAD\36310LT.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\, D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

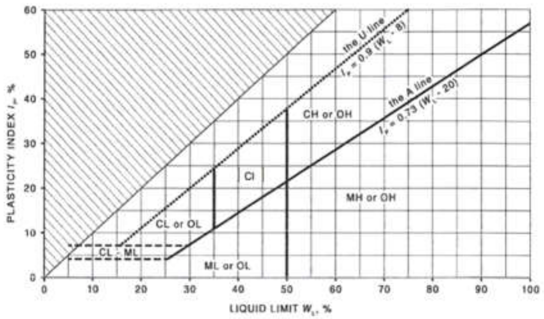
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL w ≈ PL w < PL w ≈ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)	 VL L MD D VD ( )	**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)** VERY LOOSE ≤ 15 0 – 4 LOOSE > 15 and ≤ 35 4 – 10 MEDIUM DENSE > 35 and ≤ 65 10 – 30 DENSE > 65 and ≤ 85 30 – 50 VERY DENSE > 85 > 50 Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as: RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock. EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock. ALLUVIAL – soil deposited by creeks and rivers. ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents. MARINE – soil deposited in a marine environment. AEOLIAN – soil carried and deposited by wind. COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits. LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength: Point Load Strength Index $Is_{(50)}$ (MPa)	Guide to Strength: Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

APPENDIX A

Selected Historical Aerial Imagery

Aerial Imagery 2023

69-79 Kyogle Street, South Lismore, NSW 2480

Legend

Site Boundary

Buffer 150m

Scale: 0 30 60 90 120 Meters	Data Source Aerial Imagery: © Aerometrex Pty Ltd	Coordinate System: GDA 1994 MGA Zone 56	Date: 10 October 2023

Aerial Imagery 2009

69-79 Kyogle Street, South Lismore, NSW 2480

Scale: 0 30 60 90 120 Meters	Data Source Aerial Imagery: © 2023 Google Inc, used with permission. Google and the Google logo are registered trademarks of Google Inc.	Coordinate System: GDA 1994 MGA Zone 56	Date: 05 October 2023

Aerial Imagery 1979

69-79 Kyogle Street, South Lismore, NSW 2480

Scale: 0 30 60 90 120 Meters	Data Source Aerial Imagery: © NSW Department of Customer Service	Coordinate System: GDA 1994 MGA Zone 56	Date: 05 October 2023

Aerial Imagery 1942

69-79 Kyogle Street, South Lismore, NSW 2480

Scale: 0 30 60 90 120 Meters

Data Source Aerial Imagery: © 2023 Geoscience Australia

Coordinate System: GDA 1994 MGA Zone 56

Date: 05 October 2023