GEOTECHNICAL INVESTIGATION REPORT

No. 19-21 Banks Street
Padstow, NSW

Prepared for

HL Australia Investments P/L
C/- CDArchitects

Reference No. ESWN-PR-2025-2890

15th March 2025

Geotechnical Engineering Services

- *Geotechnical investigation*
- *Geotechnical design*
- *Excavation methodology and monitoring plans*
- *Footing inspections*
- *Slope stability and landslide risk assessment*
- *Permeability test*
- *Lot classification*
- *Finite Element Analysis(FEA)*

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DISTRIBUTION AND REVISION REGISTER

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Author: Jiameng Li..

Signed: .....JLi..

Date: 15/03/2025 ....

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REFERENCES

1. Australian Standard – AS 1726-2017 Geotechnical Site Investigation.
2. Australian Standard AS 1289.6.3.2 – Determination of the penetration resistance of a soil – 9 kg dynamic cone penetrometer test.
3. Australian Standard – AS 2870-2011 Residential Slabs and Footings.
4. Australian Standard – AS 2159-2009 Piling - Design and Installation.
5. Australian Standard – AS 3798-2007 Guidelines on Earthworks for Commercial and Residential Developments.
6. Australian Standard – AS 1170.4-2007 Structural Design Actions – Part 4: Earthquake actions in Australia.
7. Australian Standard – AS 4678-2002 Earth-retaining Structures.
8. Austroads – "Pavement Design – A Guide to the Structural Design of Road Pavements", 2004.
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10. Pells, P.J.N, Mostyn, G. & Walker B.F., "Foundations on Sandstone and Shale in the Sydney Region", Australian Geomechanics Journal, 1998.
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1. INTRODUCTION

ESWNMAN Pty Ltd (ESWNMAN) was commissioned by HL Australia Investments Pty Ltd c/- CDArchitects to undertake a geotechnical investigation for a proposed development at No.19-21 Banks Street, Padstow, NSW 2211. The fieldwork was completed on 27th February 2025 by ESWNMAN staff supervised by an experienced Geotechnical Engineer.

The purpose of the investigation was to assess feasibility of the site in geotechnical prospective for a proposed mixed use development.

This report presents results of geotechnical investigation and in-situ tests, interpretation, geotechnical assessment, and provides comments on geotechnical related issues and recommendations.

1.1 Available Information

The following information was provided to ESWNMAN prior to fieldwork:

- Architectural drawings titled “Proposed Mixed Use Development, 19-21 Street, Padstow, NSW 2211” prepared by CDArchitects, referenced J24628D, including drawing nos. DA1101 to DA7002 (6 sheets), and dated February 2025.

1.2 Proposed Development

The design drawings provided as referenced in Section 1.1 indicated the proposed development will comprise the demolition of existing site structures and construction of a four(4) storey building above a basement level.

Based on design plans provided, approximate excavation between 2.5m and 3.0m deep would be required for proposed basement level. An approximate setback of 2.0m from north-eastern boundary and 5.0m from south-western side boundary was proposed for the basement level.

In addition, the following excavation, cut/fill and earthworks are also likely required during construction:

- Excavation of driveway ramp from street entry to basement area;
- Excavation of proposed lift shaft and OSD tank;
- Excavation within structural footing areas (such as, pad/strip footings/piles);
- Trench excavation/backfilling for installation of water/sewer/stormwater pipes;
- Subgrade preparation for pavement & footpath; and

- Landscaping.

1.3 Scope of Work

The fieldwork of geotechnical investigation was undertaken by ESWNMAN staff supervised by an experienced Geotechnical Engineer, including the following:

- Desktop study on local geology and our in-house dataset near the subject site;
- Collection and review of Before-You-Dig-Australia (BYDA) plans;
- A site walkover to assess site accessibility and surface conditions, identify relevant site features, and nominate borehole and testing locations;
- Augering of boreholes to check thickness of fill and property of natural soils;
- Performing of Dynamic Cone Penetrometer (DCP) Test to assess the strength of soils with depth and rock profile;
- Geotechnical logging of materials retrieved from boreholes by an experienced Geotechnical Engineer;
- Reinstatement of site upon completion of fieldwork;
- Interpretation of investigation data obtained; and
- Preparation of a geotechnical report.

The approximate locations of boreholes and DCP test completed are shown on Figure 1 – “Site location Plan” as included in Appendix A of this report.

2. SITE DESCRIPTION

The site is located within the Canterbury-Bankstown Council area, approximately 18km to the southwest of Sydney CBD, 320m to the northeast of Padstow Railway Station and 570m to the west of Salt Pan Creek.

The site is a rectangular-shaped land, identified as Lots F & G in Deposited Plan (DP)21715, with an approximate total area of 1393.5m^2. At time of fieldwork, the site was occupied by a single storey brick & clad house.

Based on plans provided and our observations during a site walkover, the existing ground surface is characterized by a gentle sloping ground towards the Salt Pan Creek in southeast.

Selected site photographs recorded during site investigation are provided in Appendix B.

3. LOCAL GEOLOGY

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 (Edition 1), dated 1983, by the Geological Survey of New South Wales, Department of Mineral Resources, indicates the site is located within an area underlain by Triassic Age Ashfield Shale Formation (Rwa) of the Wianamatta Group. The Ashfield Shale is described as "Black to dark grey shale and laminite".

Results of the investigation, as provided in Section 5.2 confirmed the published geology.

4. METHODOLOGY OF INVESTIGATION

4.1 Pre-fieldwork

Prior to commencement of fieldwork, a desktop study on local geology and review of our in-house dataset near the subject site was undertaken.

BYDA underground services search was also conducted with plans reviewed on-site prior to in-situ tests.

4.2 Borehole Drilling

At time of fieldwork, portable geotechnical investigation was undertaken involved augering of boreholes to check the thickness of fill and property of natural soils using a hand operated equipment assisted with in-situ tests.

The borehole locations are shown on Figure 1 attached in Appendix A. Engineering logs of boreholes processed using Bentley gINT software together with borehole explanatory notes are presented in Appendix C.

4.3 Dynamic Cone Penetrometer (DCP) Test

The Dynamic Cone Penetrometer (DCP) Test involves hammering cone tipped rods using a standard weight and drop height. The number of blows required to penetrate each 100 mm is recorded (Reference 2). The DCP test is used to assess in-situ strength of undisturbed soil and/or compacted materials. The penetration rate of the 9-kg DCP can be used to estimate in-situ CBR (California Bearing Ratio) and to identify strata thickness and other material characteristics.

A total of four(4) DCP tests, denoted as DCPs 1 to 4 inclusive, were completed during site investigation. DCP test reached refusal depth and bounce of DCP hammer occurred at 1.4m, 1.1m, 1.5m and 1.4m below existing ground level (BGL) at location of DCPs 1 to 4 respectively.

The location of DCP test is shown on Figure 1 attached in Appendix A. The record of DCP test results is presented in Appendix D.

All fieldwork was supervised on a full time basis by a Geotechnical Engineer who was responsible for nominating locations of boreholes and DCP tests, preparing field engineering logs of subsurface strata encountered in accordance with AS 1726 for Geotechnical Site Investigation (Reference 1), conducting in-situ tests & taking site photographs.

5. INVESTIGATION RESULTS

5.1 Surface Conditions

At time of fieldwork, apart from existing dwellings, an inground swimming pool, a garage, metal sheds, concrete driveway and surface, the remainder of outdoor area was covered with grass and lawn.

5.2 Subsurface Conditions

Based on borehole information and interpreted results of DCP test, subsurface conditions at testing locations consist of the following:

- **Fill** (Unit 1): Silty CLAY, low plasticity, grey, some topsoil near surface, some sand, trace gravel, variable compaction, mostly fairly compacted, approximately extending to 0.3m, 0.4m, 0.6m and 0.6m BGL at location of BH1 to BH4 respectively; overlying
- **Residual Soils** (Unit 2): Silty CLAY, low plasticity, brown with red mottling, moist, varying from “stiff” to “hard” consistency, extending to inferred top of rock below 1.4m, 1.1m, 1.5m and 1.4m BGL at DCPs 1 to 4 respectively; overlying
- **Weathered Shale** (Unit 3): Class V SHALE, grey, some laminite, extremely weathered, extremely low and low strength, based on interpreted results of DCP test and local geology. The classification of rock was carried out in accordance with Pells et al (Reference 10).

A generalised ground profile based on the above is provided in Table 1 overleaf.

Table 1 – Subsurface Conditions at Testing Locations

Geotechnical Unit and Description			**Inferred Depth to Top of Unit** (m, BGL)			
			BH1/ DCP1	BH2/ DCP2	BH3/ DCP3	BH4/ DCP4
Fill (Unit 1)		Silty CLAY, fairly compacted	0	0	0	0
Residual Soils (Unit 2)	Unit 2a	Silt CLAY, stiff	0.3	0.4	0.6	0.6
	Unit 2b	Silty CLAY, very stiff & hard	0.6	1.0	0.9	1.0
Weathered Shale (Unit 3)		Class V SHALE, extremely low strength	1.4	1.1	1.5	1.4

5.3 Groundwater

No groundwater was encountered during augering of any boreholes. No indication of water seepage/inflow and no wet soil materials were observed on DCP tools when DCP accessories were extracted onto ground surface upon completion of DCP tests.

6. GEOTECHNICAL ASSESSMENT

The main geotechnical aspects associated with the proposed development are assessed to include the following:

- Site classifications;
- Excavation conditions and methods;
- Stability of excavation/shoring measures;
- Earth retaining structures;
- Foundations;
- Subgrade/foundation preparation;
- Earthworks and material use;
- Water/seepage management; and
- Preliminary comments on pavement design.

The assessment of listed above and recommendations for proposed development are presented in the following sections.

6.1 Site Classifications

(a) Site reactivity classification

Based on ground profile of the site and the criteria specified in AS 2870 (Reference 2), the site is assessed as Class M – "Moderately reactive clay or silt sites", which may experience moderate ground movement from moisture changes, provided that our recommendations in Section 6.5 are adopted during design and construction.

The above classification and footing recommendations are provided on the basis that the performance expectations set out in Appendix B of AS2870 are accepted.

Design, construction and maintenance of plumbing, ground drainage, protection of building perimeter, the garden, etc. should be carried out in accordance with CSIRO BTF18 (Reference 11) to avoid any water related problems or significant changes of moisture in building foundations, which may contribute to surface movement.

(b) Site earthquake classification

The results of the site investigation indicate the presence of fill and residual soils underlain by Class V Shale or stronger rock. In accordance with Australian Standard AS 1170.4 (Reference 6), the site may be classified as a “Shallow soil site” (Class C_e) for design of foundations and retaining walls embedded in the soils or classified as a “Rock site” (Class B_e) for design of foundations and retaining walls embedded in the underlying Shale. The Hazard Factor (Z) for Padstow in accordance with AS 1170.4 is considered to be 0.08.

6.2 Excavation Conditions and Methods

Based on proposed development as provided in Section 1.1, variable excavation between 2.5m and 3.0m deep for basement level would be required during construction. Other minor excavation, including excavation of driveway ramp, lift shaft, footing excavation for building and retaining walls (such as, pad/strip footings/piles), trench excavation for installation of water/sewer/stormwater pipes and landscaping, would be also required during construction.

Results of geotechnical investigation in Section 5.2 indicate the presence of Fill(Unit 1), Residual Soils(Unit 2) and Weathered Shale(Unit 3) underlying the site.

Excavation of the soils and low strength Class V Shale would be typically feasible using conventional earthmoving equipment. Excavation of low strength Class IV Shale or shale with lamination may be feasible with conventional earthmoving equipment and ripping equipment. Heavy ripping and rock breaking equipment or vibratory rock breaking equipment is typically required for excavation in medium strength Class IV Shale or stronger rock.

The temporary excavation shoring measures should be constructed in accordance with our recommendation provided in Section 6.3.

Any fill and deleterious materials, including old footings/buried structures, concrete slabs, plant/tree roots, redundant services, timber/brick material, and boulders, are expected to be stripped and removed from development area to spoils.

Based on groundwater conditions in Section 5.3, **we assessed the excavation of proposed basement level and associated works will not encounter groundwater.**

6.3 Excavation Support / Stability of Excavation

For shallow excavation (i.e. <1.5m in depth), it should be carried out in accordance with the 'NSW WorkCover: Code of Practice – Excavation' (Reference 8).

Temporary excavations using safe excavation batter through the underlying residual soils to a maximum depth of 1.5m provided that:

- They do not encroach ZOI(Zone of Influence, defined as 45° angle of draw from nearest edge of footing underside) of any site or adjoining structures;
- They are barricaded when not in use;
- They are not left open for more than 24 hours;
- No surcharge loading is applied within 1.5m from edge of excavation/footings;
- No groundwater flows are encountered; and
- They are not used for access by a worker.

Where access is required for workers, the temporary excavation batters should be re-graded to no steeper than 2 Horizontal (H) to 1 Vertical (V) for the soils above the natural groundwater level.

We recommend a permanent safe batter of 3H:1V or flatter can be adopted for soil materials within the site for re-battering and landscaping.

Any permanent excavation (or filling) greater than 0.6m in height should be retained by a permanent retaining wall to be designed by a qualified Engineer based on the recommendation provided in Section 6.4 of this report.

For deep excavation (i.e. >1.5m deep) and excavation away from boundaries, temporary safe batters for excavated slopes in Table 2 overleaf can be adopted under dry conditions:

Table 2 - Recommended Safe Excavation Batters[1]

Geotechnical Unit		Maximum Batter Angle	
		Temporary[2]	Permanent
Fill (Unit 1)		2.0H:1V	To be retained
Residual Soils (Unit 2)	Unit 2a	1.75H:1V	To be retained
	Unit 2b	1.5H:1V	To be retained
Class V Shale (Unit 3)		1H:2.5V to sub-vertical Self-supporting	Reinforced shotcrete or To be retained

Notes:

[1] - Typical temporary batters of excavated slopes (Hoerner, 1990).

[2] – Reinforced shotcrete with drainage, inclined/raking shores/braces or earth berms/formworks can be considered as temporary support/shoring measures for excavation over short period of time.

Based on proposed setbacks and depth of excavation, we assessed excavation using temporary safe batters recommended in Table 2 would be typically feasible for excavation of proposed basement level.

However, it may not be possible or impractical using safe batters recommended in Table 2 for excavation of basement level along site boundaries due to inadequate setback proposed, or to control lateral ground movement and consideration of safe work in front of excavation/cut, the following temporary measures to shore and support the excavation should be adopted **prior to excavation**:

- Soldier pile wall shoring system; or
- A line of closely spaced piles (spacing not more than 1m C.T.C).

Other alternative shoring options may be considered subject to an assessment by the project Structural Engineer in consultation with the project Geotechnical Engineer.

Dilapidation survey should be undertaken for the adjoining properties, roads and public infrastructure prior to commencement of construction excavation.

Earth retention structures can be designed using the recommended parameters provided in Section 6.4.

If our recommendations on the above are adopted during design and construction, the construction of proposed development and excavation will have no impacts on adjoining properties, road and public infrastructure.

6.4 Earth Retaining Structures

The earth retaining structure should be designed to withstand the applied lateral pressures of the subsurface layers, the surcharges in their zone of influence, including loading from existing structures, construction machines, traffic and construction related activities. The design of retaining structures should also take into consideration hydrostatic pressures and lateral earthquake loads as appropriate. Filter type geofabric should be considered to be installed between retaining wall backfill area and surrounding soils to prevent the fines from entering the wall drainage system.

Earth retention structures can be designed in accordance with AS 4678 (Reference 7).

The recommended preliminary parameters for design of retaining structures are presented in Tables 3 and 4 below. The coefficients provided are based on drained conditions.

Table 3 - Preliminary Geotechnical Design Parameters for Retaining Walls

Geotechnical Unit	Unit Weight (kN/m^3)	Effective Cohesion c′ (kPa)	Angle of Effective Internal Friction ϕ'(°)	Modulus of Elasticity E′ (MPa)	Poisson Ratio ν
Fill (Unit 1)	17	2	27	10	0.35
Residual Soils (Unit 2)	18	5	28	30	0.35
Class V Shale[1] (Unit 3)	23	50	30	80	0.25

Table 4 - Preliminary Coefficients of Lateral Earth Pressure

Geotechnical Unit	Coefficient of Active Lateral Earth Pressure (Ka)	Coefficient of Active Lateral Earth Pressure at Rest (Ko)	Coefficient of Passive Lateral Earth Pressure (Kp)
Fill (Unit 1)	0.38	0.55	2.7
Residual Soils (Unit 2)	0.36	0.53	2.8
Class V Shale[1] (Unit 3)	0.33	0.50	3.0

[1] - Classification of the rock in accordance with Pells et al (Reference 10).

The coefficients of lateral earth pressure should be verified by the project Structural Engineer for design of retaining walls. Simplified calculations of lateral active (or at rest) and passive earth pressures can be carried out using Rankine’s equation shown below:

$Pa = K\,\gamma\,H - 2c\sqrt{K}$ For calculation of Lateral Active or At Rest Earth Pressure
$Pp = K_p\,\gamma\,H + 2c\sqrt{K_p}$ For calculation of Passive Earth Pressure
Where:

P_a = Active (or at rest) Earth Pressure (kN/m^2)
P_p = Passive Earth Pressure (kN/m^2)
γ = Bulk density (kN/m^3)
K = Coefficient of Earth Pressure (K_a or K_o)

Kp = Coefficient of Passive Earth Pressure
H = Retained height (m)
c = Effective Cohesion (kN/m^2)

6.5 Foundations

Results of geotechnical investigation and assessments indicate the ground conditions at this site are suitable for proposed development and associated works.

It is noted that after excavation of proposed basement level to proposed FFL, the materials at bulk excavation level are likely occupied by Unit 3 – "Class V Shale" or stronger rock. Therefore, **cast in-situ reinforced concrete shallow foundations,** such as pad/strip footings under columns and walls/stiffened raft slab, are applicable for structures within basement area. **Piers/piles** founded in Unit 3 – "Class V Shale" can be considered for any structures proposed outside the footprint of basement level (if any). Bored piles can be adopted.

We recommend the suitable founding materials should be Unit 3 – "Class V Shale" or stronger rock for all building & retaining wall structures.

The preliminary geotechnical parameters recommended for design of foundations are provided in Table 5 below.

Table 5 - Preliminary Geotechnical Foundation Design Capacities and Parameters

Geotechnical Unit		Allowable End Bearing Pressure (kPa[1])	Allowable Shaft Adhesion Compression[2] (kPa)	Modulus of Elasticity (Es,v, MPa)
Fill (Unit 1)		N/A[3]	N/A[3]	15
Residual Soils (Unit 2)	Unit 2a	150 (Shallow footings)	10	30
	Unit 2b	250 (Shallow footings/piles)	20	50
Class V Shale[4] (Unit 3)		500 (Shallow footings/piles)	40	100

Note:
[1] Applying a multiplier of 3.0 for ultimate capacities.
[2] Shaft Adhesion applicable to piles only. Applying a multiplier of 0.5 for skin friction under tension.
[3] N/A, being excavated or Not Applicable or not recommended for supporting structures unless re-compacted.
[4] If a higher allowable bearing capacity than recommended adopted for the design, a Geotechnical Engineer should be consulted and bearing capacity should be verified during footing excavation.

Design of shallow and piled foundations should be carried out in accordance with Australian Standards AS2870 (Reference 3) and AS2159 (Reference 4).

To minimise the potential effects of differential settlement under the buildings loads, it is recommended all foundations of the proposed building should be founded on consistent materials of similar properties or rock of similar class.

Any water, debris, loose and wet materials should be removed from excavations prior to placement of reinforcement and pouring of concrete.

A Geotechnical Engineer should be engaged to inspect footing excavations to ensure foundation bases have suitable materials with adequate bearing capacity, and to check the adequacy of footing embedment/socket depth if **unexpected ground conditions** are encountered or **a geotechnical certificate** is required.

6.6 Foundation/subgrade Preparation

For service pipes or slabs to fully or partially rely on soils underneath (either existing fill or new fill), to achieve an allowable bearing capacity of 150kPa, the following is recommended:

- Excavate and re-compact existing Fill (Unit 1);
- Remove roots/timber and organic matters and oversized materials(if any);
- Level off the existing natural ground surface and provide proof rolling;
- Place fill materials (preferably granular materials) at loose layer of not exceeding 150mm in thickness for cohesive soils and 200mm for cohesionless materials;
- Densify the fill mechanically, using a suitable roller (as guidance, at least 10 passes of 5-8 tonne deadweight roller) or compaction equipment;
- Repeat the above till proposed FLL is reached.

For compaction over a small area or inside a trench, a vibrating plate compactor is commonly used to compact and densify the subgrade/foundation areas.

The fill and compaction for different engineering purposes should be carried out in accordance with recommendations provided in Section 6.7 below.

6.7 Earthworks and Material Use

The excavated materials from excavation are assessed to be generally suitable for landscaping provided they are free of any contaminants.

The suitability of the site won materials or imported materials for use as engineering fill should be subject to satisfying the following criteria:

- The materials should be Virgin Excavated Natural Material (VNEM) and clean (i.e. free of contaminants, deleterious or organic material), free of inclusions of >75mm in size, high plasticity material be removed and suitably conditioned to meet the design assumptions where fill material is proposed to be used.
- The materials should satisfy the Australian Standard AS 3798 Guidelines on Earthworks for Commercial and Residential Developments (Reference 5).

As a guidance for the fill construction, the following compaction targets can be adopted:

- Moisture content of ±2% of OMC (Optimal Moisture Content);
- Minimum density ratio of 100% of the Maximum Dry Density (MDD) for filling within building/structural foundation areas;
- Minimum density ratio of 98% of MDD for filling surrounding pipes within trenches or behind retaining walls (unless otherwise specified in design drawings);
- The loose thickness of layer should not exceed 200mm for cohesionless soils; and
- For the driveway/footpath/pavement areas, minimum density ratio of 95% of MDD for general fill and 98% for the subgrade to 0.5m depth.

Design and construction of earthworks should be carried out in accordance with Australian Standard AS 3798 (Reference 5).

6.8 Water/seepage Management

The groundwater conditions and observations summarised in Section 5.3 indicate the construction excavation of proposed basement level will not encounter groundwater.

Based on our experience and observation of excavation in surrounding areas, it is possible that localised minor seepage/inflow may occur along interface between soils and underlying rock, fractures/defects in the rock, including apertures, joints or other natural defects within the underlying shale/claystone, in particular, when it encounters an intense and prolonged rainfall event.

Nevertheless, it would be prudent at this stage to allow for precautionary drainage measures in the design and construction of the proposed development. As a guidance, the following measures can be considered during design and construction:

- Strip drains or drainage materials to be installed behind shoring/retaining walls.
- Collection trenches or pipes and pits connected to the building stormwater system. A stormwater storage tank and pump system may be required.

- A filter type geofabric membrane to be installed between the retaining wall and backfilling soils to prevent loss of fines from surrounding soils.
- "Sump and pump method" can be typically adopted for any stormwater/localised seepage occurs in foundation pit during intense and prolonged rainfall period.

6.9 Preliminary Comments on Pavement Subgrade

It is recommended that pavement be designed on a CBR value of 3% minimum on stiff silty CLAY or medium dense sand subgrade, based on the results of DCP tests.

Topsoil and any deleterious material should be removed prior to pavement construction. Loose/soft fill materials as indicated by low DCP test results (DCP value<3) may present within the site, as confirmed by a site inspection and in-situ testing, should be improved by compaction in order to increase the strength of the material.

If require, pavement design should be carried out in accordance with "Pavement Design – A Guide to the Structural Design of Road Pavements" (Reference 8) and should be complemented by the provision of adequate surface and subsurface drainage.

7. CONCLUSIONS AND RECOMMENDATIONS

- Results of geotechnical investigation and assessment indicate the ground conditions at this site are suitable for proposed development and associated works.
- Based on proposed development and subsurface conditions, we assessed a footing system consisting of **cast in-situ reinforced concrete shallow foundations** for structures within basement area and **piers/piled foundations** for those structures proposed outside the basement footprint(if any), are applicable for proposed development at this site. **We recommend the suitable founding materials should be Unit 3 – "Class V Shale"** for any footing systems adopted. The details of applicable footing systems and recommended geotechnical parameters for foundations are provided in Section 6.5.
- For service pipes and slabs to fully or partially rely on soils underneath (either existing fill or new fill), "Foundation/subgrade Preparation" in accordance with Section 6.6 should be implemented during construction.
- The construction, including fill and compaction, excavation methods, safe excavation batters, excavation support/shoring measures, footing systems, foundation/subgrade preparation, retaining walls and earthworks, water/seepage management and pavement subgrade should be implemented in accordance with the recommendations provided in Section 6.

- A Geotechnical Engineer should be engaged to inspect footing excavations to ensure the foundation base have been taken to suitable materials of appropriate bearing capacity and adequate embedment depth/socket length if unexpected ground conditions are encountered or a geotechnical certificate is required.

8. LIMITATIONS

This report should be read in conjunction with the "Limitations of Geotechnical Investigation Statement" attached as Appendix E, which provides important information regarding geotechnical investigation, assessment and reporting. If the actual subsurface conditions exposed during construction vary significantly from those discussed in this report, this report should be reviewed and the undersigned should be contacted for further advices.

For and on behalf of
ESWNMAN Pty Ltd

JLi

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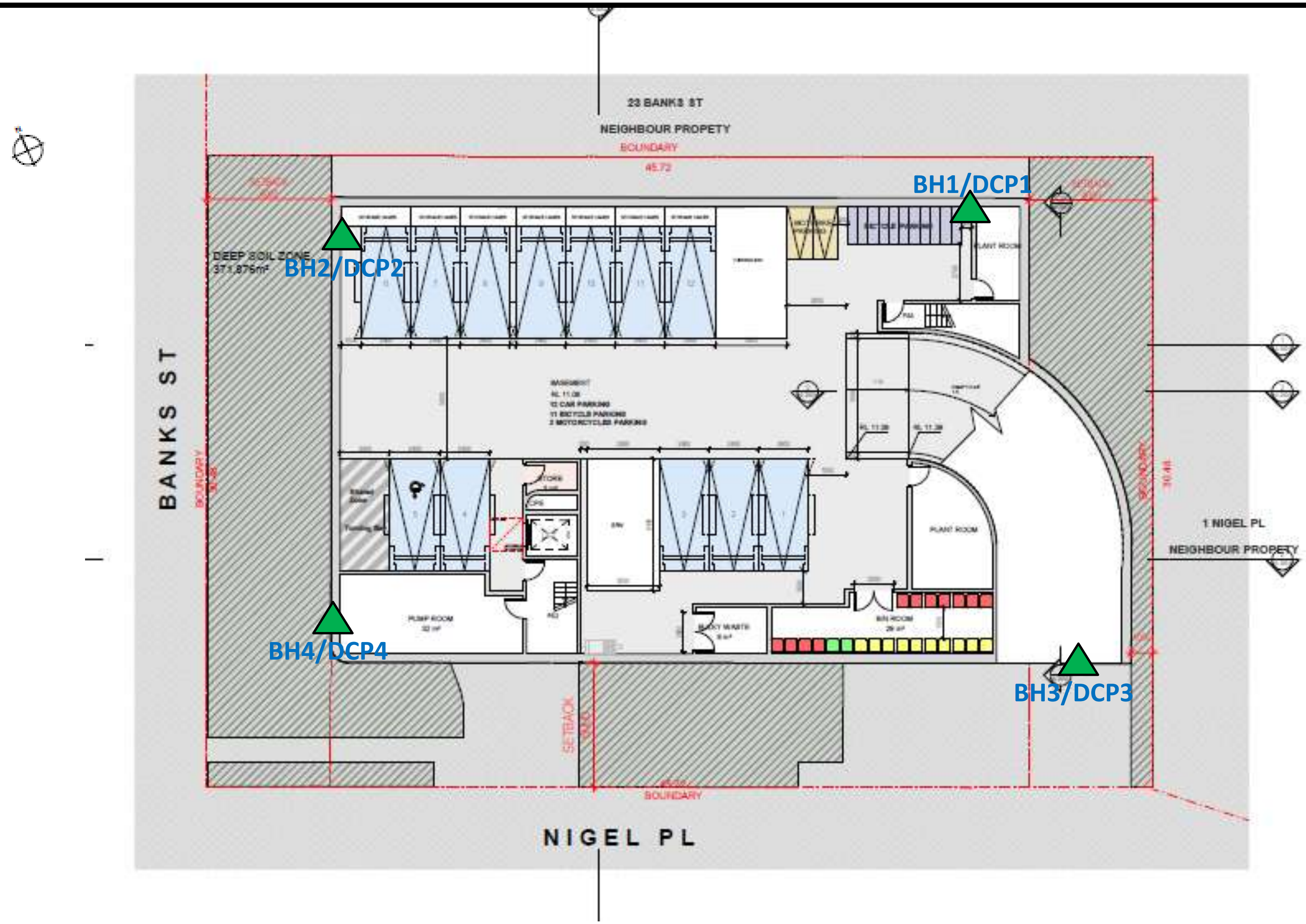
APPENDIX A

SITE LOCATION PLAN

Image source: An architectural drawing prepared by CDArchitects.



LEGEND ▲ Approximate Location of Dynamic Cone Penetrometer (DCP) Test & Borehole (BH)

PROJECT: 19-21 Banks Street, Padstow, NSW 2211	**DRAWN BY:** J.L.	ESWNMAN 25 YEARS EXPERIENCE	**CLIENT:** HL Australia Investments P/L	
PROJECT NO: ESWN-PR-2025-2890	**DATE:** 15th March 2025		**TITLE:** Site Location Plan	FIGURE 1

APPENDIX B

SITE PHOTOGRAPHS

	
Photograph 1 Dynamic Cone Penetrometer(DCP) Test in progress at location of DCP1 within rear portion	**Photograph 2** DCP test in progress at location of DCP2 within front portion
	
Photograph 3 DCP test in progress at DCP3 within rear portion portion	**Photograph 4** DCP test in progress at DCP3 within front yard

Appendix B Site Photographs

APPENDIX C

ENGINEERING BOREHOLE LOGS AND EXPLANATORY NOTES

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BOREHOLE NUMBER BH1

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CLIENT HL Australia Investments P/L
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER ESWN-PR-2025-2890
PROJECT LOCATION 19-21 Banks Street, Padstow, NSW

DATE STARTED 27/2/25
COMPLETED 27/2/25
R.L. SURFACE
DATUM
DRILLING CONTRACTOR ESWNMAN Pty Ltd
SLOPE 90°
BEARING ---
EQUIPMENT Hand Auger & DCP Test
HOLE LOCATION Refer to Figure 1
HOLE SIZE 70mm Diameter
LOGGED BY W.L.
CHECKED BY J.L.
NOTES Rear portion

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
HA	NOT ENCOUNTERED			▩	CL	Silty CLAY, low plasticity, grey, some sand, trace gravel, moist, fairly compacted.		FILL
			0.5	▨	CL	Silty CLAY, low plasticity, brown, moist, stiff.		RESIDUAL SOILS
			1.0	▨	CL	Silty CLAY, low plasticity, brown, moist, very stiff.		
				▨	CL	Silty CLAY, low plasticity, brown-reddish brown, moist, hard.		DCP test indicates top of rock below 1.4m depth
			1.5			Borehole BH1 terminated at 1.4m		
			2.0					

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BOREHOLE NUMBER BH2

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CLIENT HL Australia Investments P/L

PROJECT NAME Geotechnical Investigation

PROJECT NUMBER ESWN-PR-2025-2890

PROJECT LOCATION 19-21 Banks Street, Padstow, NSW

DATE STARTED 27/2/25 **COMPLETED** 27/2/25

R.L. SURFACE

DATUM

DRILLING CONTRACTOR ESWNMAN Pty Ltd

SLOPE 90°

BEARING ---

EQUIPMENT Hand Auger & DCP Test

HOLE LOCATION Refer to Figure 1

HOLE SIZE 70mm Diameter

LOGGED BY W.L.

CHECKED BY J.L.

NOTES Front portion

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
HA	NOT ENCOUNTERED			▩	CL	Silty CLAY, low plasticity, grey, some sand, trace gravel, moist, fairly-well compacted.		FILL
			0.5	▨	CL	Silty CLAY, low plasticity, brown, moist, stiff.		RESIDUAL SOILS
			1.0	▨	CL	Silty CLAY, low plasticity, brown with red mottling, moist, hard.		DCP test indicates top of rock below 1.1m depth
			1.5 2.0			Borehole BH2 terminated at 1.1m		

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Ashfield, NSW 1800

BOREHOLE NUMBER BH3

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CLIENT HL Australia Investments P/L

PROJECT NAME Geotechnical Investigation

PROJECT NUMBER ESWN-PR-2025-2890

PROJECT LOCATION 19-21 Banks Street, Padstow, NSW

DATE STARTED 27/2/25 **COMPLETED** 27/2/25

R.L. SURFACE

DATUM

DRILLING CONTRACTOR ESWNMAN Pty Ltd

SLOPE 90°

BEARING ---

EQUIPMENT Hand Auger & DCP Test

HOLE LOCATION Refer to Figure 1

HOLE SIZE 70mm Diameter

LOGGED BY W.L.

CHECKED BY J.L.

NOTES Rear portion

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
HA	NOT ENCOUNTERED		0.5	▩	CL	Silty CLAY, low plasticity, grey, some gravel, moist, fairly compacted.		FILL
				▨	CL	Silty CLAY, low plasticity, brown, moist, stiff.		RESIDUAL SOILS
			1.0	▨	CL	Silty CLAY, low plasticity, brown, some ironstone, moist, very stiff.		
			1.5	▨	CL	Silty CLAY, low plasticity, reddish brown, moist, hard.		DCP test indicates top of rock below 1.5m depth
			2.0			Borehole BH3 terminated at 1.5m		

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BOREHOLE NUMBER BH4

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CLIENT HL Australia Investments P/L

PROJECT NAME Geotechnical Investigation

PROJECT NUMBER ESWN-PR-2025-2890

PROJECT LOCATION 19-21 Banks Street, Padstow, NSW

DATE STARTED 27/2/25

COMPLETED 27/2/25

R.L. SURFACE

DATUM

DRILLING CONTRACTOR ESWNMAN Pty Ltd

SLOPE 90°

BEARING ---

EQUIPMENT Hand Auger & DCP Test

HOLE LOCATION Refer to Figure 1

HOLE SIZE 70mm Diameter

LOGGED BY W.L.

CHECKED BY J.L.

NOTES Front portion

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
HA	NOT ENCOUNTERED		0.5	▩	CL	Silty CLAY, low plasticity, grey, some topsoil near surface, trace sand, moist, poorly compacted.		FILL
			1.0	▨	CL	Silty CLAY, low plasticity, brown, moist, stiff.		RESIDUAL SOILS
				▨	CL	Silty CLAY, low plasticity, brown, moist, very stiff.		
				▨	CL	Silty CLAY, low plasticity, reddish brown, moist, hard.		DCP test indicates top of rock below 1.4m depth
			1.5 2.0			Borehole BH4 terminated at 1.4m		

Explanatory Notes – Description for Soil

In engineering terms soil includes every type of uncemented or partially cemented inorganic material found in the ground. In practice, if the material can be remoulded by hand in its field condition or in water it is described as a soil. The dominant soil constituent is given in capital letters, with secondary textures in lower case. The dominant feature is assessed from the Unified Soil Classification system and a soil symbol is used to define a soil layer .

METHOD

Method	Description
AS	Auger Screwing
BH	Backhoe
CT	Cable Tool Rig
EE	Existing Excavation/Cutting
EX	Excavator
HA	Hand Auger
HQ	Diamond Core-63mm
JET	Jetting
NMLC	Diamond Core –52mm
NQ	Diamond Core –47mm
PT	Push Tube
RAB	Rotary Air Blast
RB	Rotary Blade
RT	Rotary Tricone Bit
TC	Auger TC Bit
V	Auger V Bit
WB	Washbore
DT	Diatube

WATER

▽	Water level at date shown	◁	Partial water loss
▷	Water inflow	◀	Complete water loss

NFGWO: The observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit.

NFGWE: The borehole/test pit was dry soon after excavation. Inflow may have been observed had the borehole/test pit been left open for a longer period.

SAMPLING

Sample	Description
B	Bulk Disturbed Sample
D	Disturbed Sample
Jar	Jar Sample
SPT	Standard Penetration Test
U50	Undisturbed Sample –50mm
U75	Undisturbed Sample –75mm

UNIFIED SOIL CLASSIFICATION

The appropriate symbols are selected on the result of visual examination, field tests and available laboratory tests, such as, sieve analysis, liquid limit and plasticity index.

USC Symbol	Description
GW	Well graded gravel
GP	Poorly graded gravel
GM	Silty gravel
GC	Clayey gravel
SW	Well graded sand
SP	Poorly graded sand
SM	Silty sand
SC	Clayey sand
ML	Silt of low plasticity
CL	Clay of low plasticity
OL	Organic soil of low plasticity
MH	Silt of high plasticity
CH	Clay of high plasticity
OH	Organic soil of high plasticity
Pt	Peaty Soil

MOISTURE CONDITION

Dry - Cohesive soils are friable or powdery
Cohesionless soil grains are free-running

Moist - Soil feels cool, darkened in colour
Cohesive soils can be moulded
Cohesionless soil grains tend to adhere

Wet - Cohesive soils usually weakened
Free water forms on hands when handling

For cohesive soils the following codes may also be used:

MC>PL	Moisture Content greater than the Plastic Limit.
MC~PL	Moisture Content near the Plastic Limit.
MC<PL	Moisture Content less than the Plastic Limit.

PLASTICITY

The potential for soil to undergo change in volume with moisture change is assessed from its degree of plasticity. The classification of the degree of plasticity in terms of the Liquid Limit (LL) is as follows:

Description of Plasticity	LL (%)
Low	<35
Medium	35 to 50
High	>50

COHESIVE SOILS - CONSISTENCY

The consistency of a cohesive soil is defined by descriptive terminology such as very soft, soft, firm, stiff, very stiff and hard. These terms are assessed by the shear strength of the soil as observed visually, by hand penetrometer values and by resistance to deformation to hand moulding.

A Hand Penetrometer may be used in the field or the laboratory to provide an approximate assessment of the unconfined compressive strength (UCS) of cohesive soils. The undrained shear strength of cohesive soils is approximately half the UCS. The values are recorded in kPa as follows:

Strength	Symbol	Undrained Shear Strength, C_u (kPa)
Very Soft	VS	< 12
Soft	S	12 to 25
Firm	F	25 to 50
Stiff	St	50 to 100
Very Stiff	VSt	100 to 200
Hard	H	> 200

COHESIONLESS SOILS - RELATIVE DENSITY

Relative density terms such as very loose, loose, medium, dense and very dense are used to describe silty and sandy material, and these are usually based on resistance to drilling penetration or the Standard Penetration Test (SPT) ‘N’ values. Other condition terms, such as friable, powdery or crumbly may also be used.

Term	Symbol	Density Index	N Value (blows/0.3 m)
Very Loose	VL	0 to 15	0 to 4
Loose	L	15 to 35	4 to 10
Medium Dense	MD	35 to 65	10 to 30
Dense	D	65 to 85	30 to 50
Very Dense	VD	>85	>50

COHESIONLESS SOILS PARTICLE SIZE DESCRIPTIVE TERMS

Name	Subdivision	Size
Boulders		>200 mm
Cobbles		63 mm to 200 mm
Gravel	coarse	20 mm to 63 mm
	medium	6 mm to 20 mm
	fine	2.36 mm to 6 mm
Sand	coarse	600 µm to 2.36 mm
	medium	200 µm to 600 µm
	fine	75 µm to 200 µm

Description for Rock

The rock is described with strength and weathering symbols as shown below. Other features such as bedding and dip angle are given.

METHOD

Refer soil description sheet

WATER

Refer soil description sheet

ROCK QUALITY

The fracture spacing is shown where applicable and the Rock Quality Designation (RQD) or Total Core Recovery (TCR) is given where:

$$\text{TCR (\%)} = \frac{\text{length of core recovered}}{\text{length of core run}}$$

$$\text{RQD (\%)} = \frac{\text{Sum of Axial lengths of core > 100mm long}}{\text{length of core run}}$$

ROCK MATERIAL WEATHERING

Rock weathering is described using the abbreviations and definitions used in AS1726. AS1726 suggests the term "Distinctly Weathered" (DW) to cover the range of substance weathering conditions between (but not including) XW and SW. For projects where it is not practical to delineate between HW and MW or it is deemed that there is no advantage in making such a distinction, DW may be used with the definition given in AS1726.

Symbol	Term	Definition
RS	Residual Soil	Soil definition on extremely weathered rock; the mass structure and substance are no longer evident; there is a large change in volume but the soil has not been significantly transported
XW	Extremely Weathered	Rock is weathered to such an extent that it has 'soil' properties, ie. It either disintegrates or can be remoulded in water
HW	Highly Weathered	The rock substance is affected by weathering to the extent that limonite staining or bleaching affects the whole rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength is usually decreased compared to the fresh rock. The colour and strength of the fresh rock is no longer recognisable.
DW	*Distinctly Weathered (see AS1726 Definition below)*	
MW	Moderately Weathered	The whole of the rock substance is discoloured, usually by iron staining or bleaching, to the extent that the colour of the fresh rock is no longer recognisable
SW	Slightly Weathered	Rock is slightly discoloured but shows little or no change of strength from fresh rock
FR	Fresh	Rock shows no sign of decomposition or staining

"Distinctly Weathered: 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 the deposition of weathering products in pores." (AS1726)

ROCK STRENGTH

Rock strength is described using AS1726 and ISRM - Commission on Standardisation of Laboratory and Field Tests, "Suggested method of determining the Uniaxial Compressive Strength of Rock materials and the Point Load Index", as follows:

Term	Symbol	Point Load Index $Is_{(50)}$ (MPa)
Extremely Low	EL	<0.03
Very Low	VL	0.03 to 0.1
Low	L	0.1 to 0.3
Medium	M	0.3 to 1
High	H	1 to 3
Very High	VH	3 to 10
Extremely High	EH	>10

●	Diametral Point Load Index test
■	Axial Point Load Index test

DEFECT SPACING/BEDDING THICKNESS

Measured at right angles to defects of same set or bedding.

Term	Defect Spacing	Bedding
Extremely closely spaced	<6 mm	Thinly Laminated
	6 to 20 mm	Laminated
Very closely spaced	20 to 60 mm	Very Thin
Closely spaced	0.06 to 0.2 m	Thin
Moderately widely spaced	0.2 to 0.6 m	Medium
Widely spaced	0.6 to 2 m	Thick
Very widely spaced	>2 m	Very Thick

DEFECT DESCRIPTION

Type:	Definition:
B	Bedding
BP	Bedding Parting
F	Fault
C	Cleavage
J	Joint
SZ	Shear Zone
CZ	Crushed Zone
DB	Drill Break

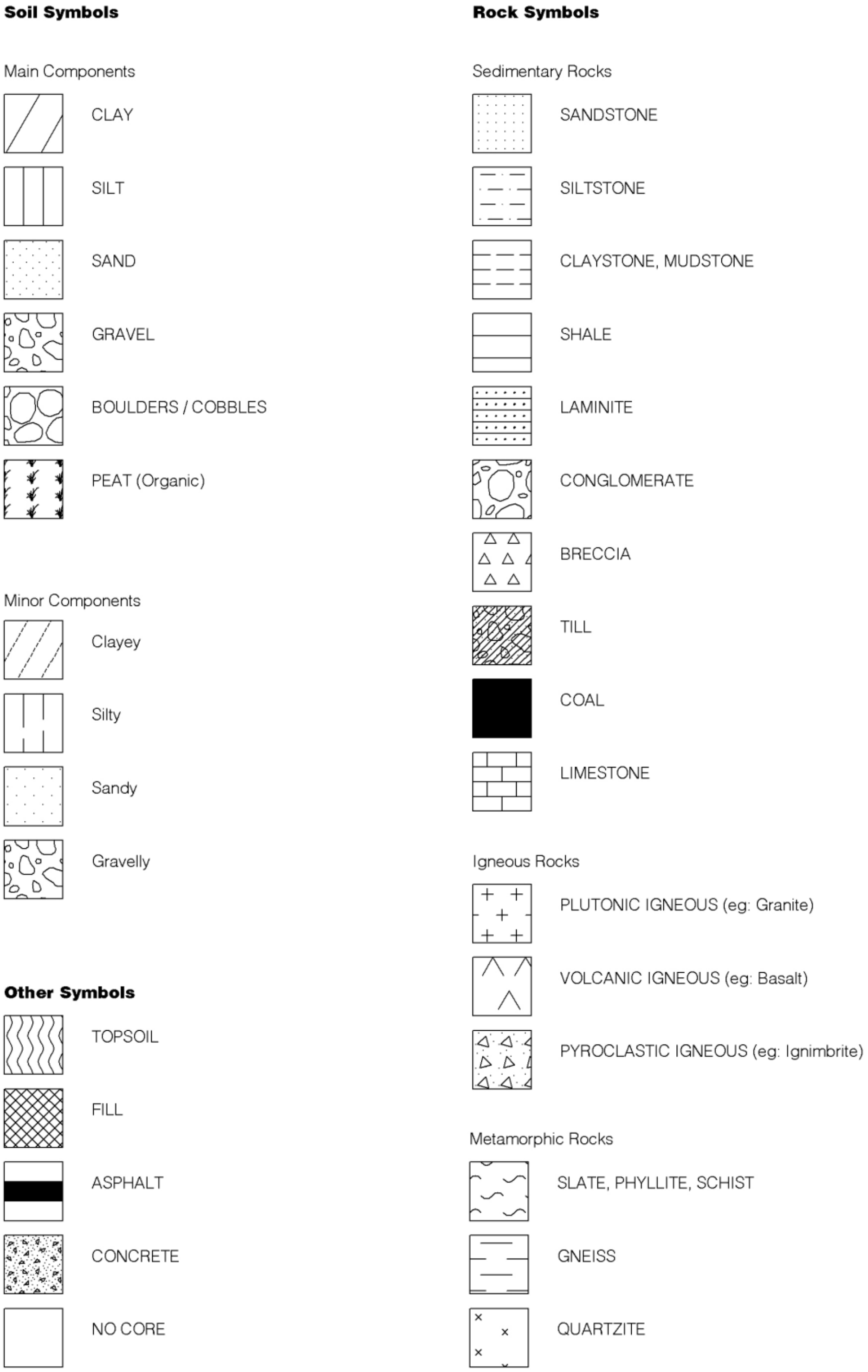
Planarity:	Roughness:
P – Planar	R – Rough
Ir – Irregular	S – Smooth
St – Stepped	Sl – Slickensides
U – Undulating	Po – Polished

Coating or Infill:	Description
Clean	No visible coating or infilling
Stain	No visible coating or infilling but surfaces are discoloured by mineral staining
Veneer	A visible coating or infilling of soil or mineral substance but usually unable to be measured (<1mm). If discontinuous over the plane, patchy veneer
Coating	A visible coating or infilling of soil or mineral substance, >1mm thick. Describe composition and thickness

The inclinations of defects are measured from perpendicular to the core axis.

Graphic Symbols for Soil and Rock

Graphic symbols used on borehole and test pit reports for soil and rock are as follows. Combinations of these symbols may be used to indicate mixed materials such as clayey sand.



Engineering classification of shales and sandstones in the Sydney Region - A summary guide

The Sydney Rock Class classification system is based on rock strength, defect spacing and allowable seams as set out below. All three factors must be satisfied.

CLASSIFICATION FOR SANDSTONE

Class	Uniaxial Compressive Strength (MPa)	Defect Spacing (mm)	Allowable Seams (%)
I	>24	>600	<1.5
II	>12	>600	<3
III	>7	>200	<5
IV	>2	>60	<10
V	>1	N.A.	N.A.

CLASSIFICATION FOR SHALE

Class	Uniaxial Compressive Strength (MPa)	Defect Spacing (mm)	Allowable Seams (%)
I	>16	>600	<2
II	>7	>200	<4
III	>2	>60	<8
IV	>1	>20	<25
V	>1	N.A.	N.A.

1. ROCK STRENGTH

For expedience in field/construction situations the uniaxial (unconfined) compressive strength of the rock is often inferred, or assessed using the point load strength index (Is_{50}) test (AS 4133.4.1 - 1993). For Sydney Basin sedimentary rocks the uniaxial compressive strength is typically about 20 x (Is_{50}) but the multiplier may range from about 10 to 30 depending on the rock type and characteristics. In the absence of UCS tests, the assigned Sydney Rock Class classification may therefore include rock strengths outside the nominated UCS range.

2. DEFECT SPACING

The terms relate to spacing of natural fractures in NMLC, NQ and HQ diamond drill cores and have the following definitions:

Defect Spacing (mm)	Terms Used to Describe Defect Spacing[1]
>2000	Very widely spaced
600 – 2000	Widely spaced
200 – 600	Moderately spaced
60 – 200	Closely spaced
20 – 60	Very closely spaced
<20	Extremely closely spaced

[1] After ISO/CD14689 and ISRM.

3. ALLOWABLE SEAMS

Seams include clay, fragmented, highly weathered or similar zones, usually sub-parallel to the loaded surface. The limits suggested in the tables relate to a defined zone of influence. For pad footings, the zone of influence is defined as 1.5 times the least footing dimension. For socketed footings, the zone includes the length of the socket plus a further depth equal to the width of the footing. For tunnel or excavation assessment purposes the defects are assessed over a length of core of similar characteristics.

Source: Based on Pells, P.J.N, Mostyn, G. and Walker, B.F. (1998) – Foundations on sandstone and shale in the Sydney region. Australian Geomechanics Journal, No 33 Part 3

APPENDIX D

RESULTS OF DYNAMIC CONE PENETROMETER(DCP) TEST

RESULTS OF DYNAMIC CONE PENETROMETER TEST

ESWNMAN
25 YEARS EXPERIENCE

Client:	HL Australia Investments P/L	Ref No:	ESWN-PR-2025-2890
Project:	Geotechnical Investigation	Date Tested:	27/02/2025
Location:	19-21 Banks Street, Padstow, NSW 2211	Tested By:	W.L./J.L.

Depth (mm)	DCP No.				Depth (mm)	DCP No.			
	DCP1	DCP2	DCP3	DCP4		5	6	7	8
0-100	3	0	2	0	0-100				
100-200	6	6	6	1	100-200				
200-300	8	9	9	1	200-300				
300-400	6	5	7	1	300-400				
400-500	6	3	6	3	400-500				
500-600	7	4	10	1	500-600				
600-700	8	5	7	3	600-700				
700-800	8	4	8	3	700-800				
800-900	9	7	6	4	800-900				
900-1000	10	6	11	7	900-1000				
1000-1100	10	5/20mm	8	10	1000-1100				
1100-1200	9	Bounce	9	10	1100-1200				
1200-1300	9		11	12	1200-1300				
1300-1400	12/90mm		11	13	1300-1400				
1400-1500	Bounce		4/10mm	Bounce	1400-1500				
1500-1600			Bounce		1500-1600				
1600-1700					1600-1700				
1700-1800					1700-1800				
1800-1900					1800-1900				
1900-2000					1900-2000				
2000-2100					2000-2100				
2100-2200					2100-2200				
2200-2300					2200-2300				
2300-2400					2300-2400				
2400-2500					2400-2500				
2500-2600					2500-2600				
2600-2700					2600-2700				
2700-2800					2700-2800				
2800-2900					2800-2900				
2900-3000					2900-3000				
3000-3100					3000-3100				
3100-3200					3100-3200				
3200-3300					3200-3300				
3300-3400					3300-3400				
3400-3500					3400-3500				
3500-3600					3500-3600				
3600-3700					3600-3700				
3700-3800					3700-3800				
3800-3900					3800-3900				
3900-4000					3900-4000				
RL(m)					**RL(m)**				

Notes:

DCP testing equipment designed and conducted in accordance with AS1289.6.3.2

APPENDIX E

LIMITATIONS OF GEOTECHNICAL INVESTIGATION

General

In making an assessment of a site from a limited number of boreholes or test pits there is the possibility that variations may occur between testing locations. Site exploration identifies specific subsurface conditions only at those points from which samples have been taken. The risk that variations will not be detected can be reduced by increasing the frequency of testing locations. The investigation program undertaken is a professional estimate of the scope of investigation required to provide a general profile of the subsurface conditions. The data derived from the site investigation program and subsequent laboratory testing are extrapolated across the site to form an inferred geological model and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

The borehole/test pit logs are the subjective interpretation of subsurface conditions at a particular location, made by trained personnel. The interpretation may be limited by the method of investigation, and cannot always be definitive.

Subsurface conditions

Subsurface conditions may be modified by changing natural forces or man-made influences. A geotechnical report is based on conditions which existed at the time of subsurface exploration.

Construction operations at or adjacent to the site, and natural events such as rainfall events, floods, or groundwater fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept appraised of any such events, and should be consulted to determine if additional tests are necessary.

Assessment and interpretation

A geotechnical engineer should be retained to work with other appropriate design professionals explaining relevant geotechnical findings and in reviewing the adequacy of their drawings/plans and specifications relative to geotechnical issues.

Information and documentations

Final logs are developed by geotechnical engineers based upon their interpretation of field description and laboratory results of field samples. Customarily, only the final logs are included in geotechnical engineering reports. These logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings. To minimise the likelihood of bore/profile log misinterpretation, contractors should be given access to the complete geotechnical engineering report prepared or authorised for their use. Providing the best available information to contractors helps prevent costly construction problems.

Construction phase service (CPS)

During construction, excavation is frequently undertaken which exposes the actual subsurface conditions. For this reason geotechnical consultants should be retained through the construction stage, to identify variations if they are exposed and to conduct additional tests which may be required and to deal quickly with geotechnical problems if they arise.

ESWNMAN does not accept any liability for site conditions not observed during the time of the construction or inspection.

Report

The report has been prepared for the benefit of the client and no other parties. ESWNMAN PTY LTD assumes no responsibility and will not be liable to any other person or organisation for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisation arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of ESWNMAN PTY LTD or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own enquiries and obtain independent advice in relation to such matters.

Other limitations

ESWNMAN PTY LTD will not be liable to update or revise the report to take into account any events or emergent circumstances or facts occurring or becoming apparent after the date of the report.