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CADDENS ESTATE
DEVELOPMENT PTY LTD

Geotechnical Investigation

Caddens Corner, 80 O'Connell Street, Kingswood NSW

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1. Introduction

1.1 Background

At the request of Mr Marvin Huang of Caddens Estate Development Pty Ltd (the Client), EI Australia (EI) has carried out a Geotechnical Investigation (GI) for the proposed development at Caddens Corner, 80 O'Connell Street, Kingswood NSW (the Site).

This GI report has been prepared to provide advice and recommendations to assist in the preparation of designs for the proposed development. The investigation has been carried out in accordance with the agreed scope of works outlined in EI's proposal referenced P20860.3_Rev2, dated 24 October 2022, and with the Client's signed authorisation to proceed, dated 25 October 2022.

1.2 Proposed Development

The following documents, supplied by the Client, were used to assist with the preparation of this GI report:

- Architectural drawings prepared by Turner – Project No. 20096, Drawing No. DA-110-006 Rev 10, DA-110-008 Rev 10, DA-112-005 Rev 09, DA-112-005 Rev 9, DA-113-005 Rev 9, DA-115-008 Rev 9, DA-116-008 Rev 9, DA-117-008 Rev 9, dated on 20 February 2024;
- Site survey plan prepared by Ramsay Surveyors Pty Ltd – Drawing Ref No. 8966, Sheet 1 to 5, dated on 5 August 2022; and
- Request for Geotechnical Proposal prepared by Northrop, dated 19 October 2022.

Based on the provided documents, EI understands that the proposed development involves the construction of five mixed-use residential and retail building blocks (2 to 8 storeys high) with park areas and local roads overlying one to two-level basements. The building blocks consisted of one building or set of several buildings together. The details are shown below:

- **Building A** – multi-storey residential development overlying a single-level of basement. The lowest basement level is proposed to have a Finished Floor Level (FFL) of RL 53.9m. A Bulk Excavation Level (BEL) of 53.6m is assumed, with excavation depths range approximately between 0.6m to 2.1m, which includes allowance for the construction of the basement slab.
- **Buildings D, E, F & G** – multi-storey residential development overlying a two-level of basement. The lowest basement level is proposed to have a Finished Floor Level (FFL) of RL 48.1m. A Bulk Excavation Level (BEL) of 47.8m is assumed, with excavation depths range between 4.9m to 6.89m, which includes allowance for the construction of the basement slab.
- **Buildings B, C, H & J** – multi-storey residential development overlying a two-level of basement. The lowest basement level is proposed to have a Finished Floor Level (FFL) of RL 51.7m. A Bulk Excavation Level (BEL) of 51.4m is assumed, with excavation depths range approximately between 3.3m to 8.8m,which includes allowance for the construction of the basement slab.
- **Buildings K, L, M & N** – multi-storey residential development overlying a two-level of basement. The lowest basement level is proposed to have a Finished Floor Level (FFL) of RL 56.7m. A Bulk Excavation Level (BEL) of 56.4m is assumed, with excavation depths range approximately between 0m to 9.5m, which includes allowance for the construction of the basement slab.

- **Buildings P, Q, R, S, T & U**– multi-storey residential development overlying a two-level of basement. The lowest basement level is proposed to have a Finished Floor Levels (FFL) of RL 42.05m - 42.5m . A Bulk Excavation Level (BEL) of 41.75 - 42.2m is assumed, with excavation depths range approximately between 7.55m to 19.95m, which includes allowance for the construction of the basement slab.

Locally deeper excavations may be required for footings, lift overrun pits, crane pads, and service trenches.

1.3 Objectives

The objective of the GI was to assess the existing site surface and subsurface conditions at nine borehole locations, and to provide preliminary geotechnical advice and recommendations addressing the following:

- Dilapidation Surveys;
- Excavation methodologies and monitoring requirements;
- Groundwater considerations;
- Vibration considerations;
- Excavation support requirements, including preliminary geotechnical design parameters for retaining walls and shoring systems;
- Building foundation options, including;
    - Preliminary design parameters.
    - Earthquake loading factor in accordance with AS1170.4:2007.
- The requirement for additional geotechnical works.

1.4 Scope of Works

The scope of works for the GI included:

- Preparation of a Work Health and Safety Plan;
- Review of relevant geological maps for the project area;
- Site walkover inspection by a Geotechnical Engineer to assess topographical features and site conditions;
- Scanning of proposed borehole locations for buried conductive services using a licensed service locator with reference to Dial Before You Dig (DBYD) plans;
- Auger drilling of nine boreholes (BHA1M, BHA4, BHA7M, BHC1, BHC7, BHD4M, BHG1M, BHH4 and BHH7M) by a track-mounted drill rig using solid flight augers equipped with a ‘Tungsten-Carbide’ (T-C) bit. The boreholes were auger drilled to depths as shown in **Table1-1** below:

Table 1-1 Augering and Rock Coring Depths

Borehole ID	Augering		Rock Coring	
	Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)
BHA1M	4.70	44.30	10.10	38.90
BHA4	4.60	44.40	10.48	38.52
BHA7M	4.25	45.25	10.85	38.65
BHC1	9.90	43.40	14.70	38.60
BHC6	1.60	52.70	15.20	39.10
BHD4M	6.15	47.35	10.10	43.40
BHG1M	4.60	50.50	9.75	45.35
BHH4	2.70	56.30	7.80	51.20
BHH7M	2.30	60.70	10.22	52.78

- Standard Penetration Testing (SPT) was carried out (as per AS 1289.6.3.1-2004), where possible, during auger drilling of the boreholes to assess soil strength/relative densities.
- Measurements of groundwater seepage/levels, where possible, in the augered sections of the boreholes during and shortly after completion of auger drilling;
- The strength of the bedrock in the augered sections of the boreholes was assessed by observation of the auger penetration resistance using a T-C drill bit and examination of the recovered rock cuttings. It should be noted that rock strengths assessed from augered boreholes are approximate and strength variances can be expected.
- The approximate surface levels shown on the borehole logs were interpolated from spot levels shown on the supplied survey plan. Approximate borehole locations are shown on **Figure 2**;
- Continuation of all nine boreholes using NMLC diamond coring techniques to termination depths shown above in **Table 1-1**. The rock core photographs are presented in **Appendix A**;
- Borehole BHA1M, BHA7M, BHD4M, BHG1M and BHH7M were converted into groundwater monitoring wells with depths of 8.0m BEGL (RL 41.0m), 9.0m BEGL (RL 40.5m), 8.8m BEGL (RL 44.7m), 4.0m BEGL (RL 51.1m) and 8.5m BEGL (RL 54.5m) to allow for long-term groundwater monitoring.
- Boreholes BHA4, BHC1, BHC6 and BHH4 were backfilled with drilling spoils;
- Soil and rock samples were sent to STS Geotechnics Pty Ltd (STS) and SGS Australia (SGS), which are National Australian Testing Authority (NATA) accredited laboratories, for testing and storage; and
- Preparation of this GI report.

EI's Geotechnical Engineer was present full-time onsite to set out the borehole locations, direct the testing and sampling, log the subsurface conditions and record groundwater levels.

1.5 Constraints

The GI was limited by the intent of the investigation. The discussions and advice presented in this report are preliminary and intended to assist in the preparation of initial designs for the proposed development. Further additional investigation in the form of boreholes of the site is required. Further geotechnical inspections should be carried out during construction to confirm the geotechnical and groundwater models, and the preliminary design parameters provided in this report.

2. Site Description

2.1 Site Description and Identification

The site identification details and associated information are presented in **Table 2-1** below while the site locality is shown on **Figure 1.** An aerial photograph of the site is presented in **Plate 1** below.

Table 2-1 Summary of Site Information

Information	Detail
Street Address	Caddens Corner, 80 O'Connell Street, Kingswood NSW
Lot and Deposited Plan (DP) Identification	Lot 1 and 2 in DP 1268507
Brief Site Description	At the time of our investigation, the site was occupied by carpark area with asphalt pavement at the western portion and vacant grassy area on the eastern portion. There is an existing asphalt paved road that runs through the middle eastern portion of the site.
Site Area	The site area is approximately 54614m^2.

Plate 1: Aerial photograph of the site (source: Nearmap, accessed October 2022)

2.2 Local Land Use

The site is situated within an area of residential use. Current uses on surrounding land at the time of our presence on site are described in **Table 2-2** below. For the sake of this report, the site boundary adjacent to O'Connell Street shall be adopted as the southern site boundary.

Table 2-2 Summary of Local Land Use

Direction Relative to Site	Land Use Description
North	Vacant, grassy land.
East	Vacant, grassy land.
South	O'Connell Street, a two-lane asphalt paved road, beyond this road are one to two storey residential dwellings.
West	Caddens Corner Shopping Centre, a single-storey commercial building with no basements.

2.3 Regional Setting

The site topography and geological information for the locality is summarised in **Table 2-3** below**.**

Table 2-3 Topographic and Geological Information

Attribute	Description
Topography	The site is located on the low north side of the road within gently (3° to 4°) northwest dipping topography with site levels varying from R.L. 48.5m at the northwest site corner to R.L. 66.2m at the southeast site corner.
Regional Geology	Information on regional sub-surface conditions, referenced from the Department of Mineral Resources Geological Map Penrith 1:100,000 Geological Series Sheet 9030 (Edition 1) 1991, indicates the site to be underlain by Bringelly Shale (Rwb), which consists of shale, carbonaceous claystone, laminite, fine to medium-grained lithic sandstone, rare coal and tuff.

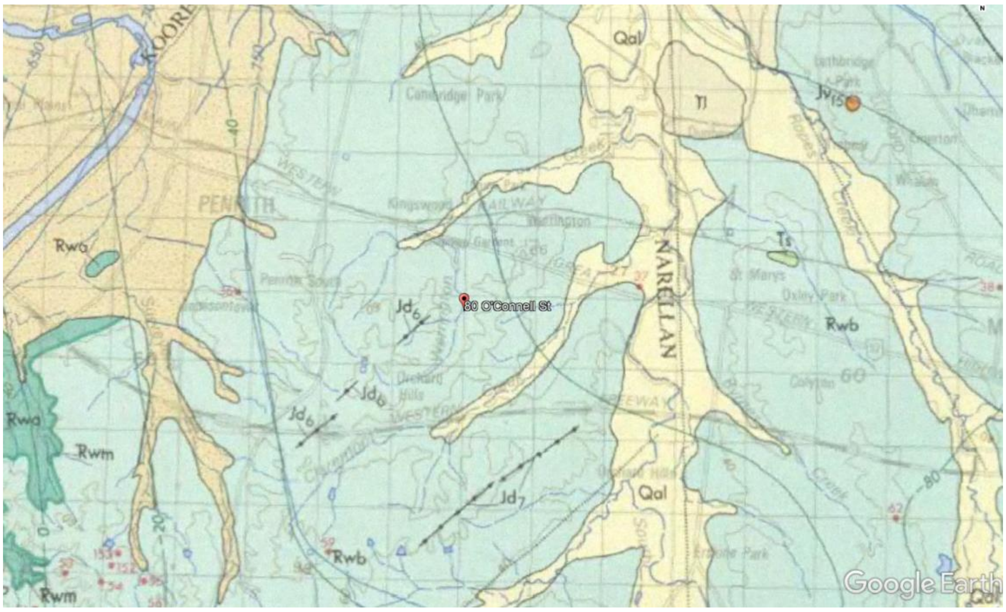

Plate 2: Excerpt of geological map showing location of site.

3. Investigation Results

3.1 Stratigraphy

For the development of a site-specific geotechnical model, the stratigraphy observed in the GI has been grouped into four geotechnical units. A summary of the subsurface conditions across the site, interpreted from the assessment results, is presented in **Table 3-1** below. More detailed descriptions of subsurface conditions at each borehole location are available on the borehole logs presented in **Appendix A**. The details of the methods of soil and rock classifications, explanatory notes and abbreviations adopted on the borehole logs are also presented in **Appendix A.**

Table 3-1 Summary of Subsurface Conditions

Unit	Material[2]	Depth to Top of Unit (m BEGL)[1]	RL of Top of Unit (m AHD)[1]	Observed Thickness (m)	Comments
1	**Topsoil/Fill**	0.00	49.0 to 63.0	0.1 to 3.3	Sandy clay, sandy gravel, gravelly sand, gravelly sandy clay and silty clay with rootlets. 50mm asphalt pavement was observed in the carpark area.
2	**Residual Soil[3]**	0.1 to 3.3	48.4 to 62.9	0.8 to 6.0	Medium to high plasticity, stiff to very stiff silty clay and sandy clay with trace ironstone gravels, grading into extremely weathered materials with depth. SPT values ranged from 8 to 25.
3	**Very Low Strength Bedrock (Siltstone / Sandstone)**	0.2 to 9.3	44.0 to 62.1	1.4 to 7.3	Distinctly weathered very low strength sandstone / siltstone bedrock. Core loss observed from depths between 5.0m to 5.3m in BHA4, from depths between 2.3m to 3.4m in BHC6 and from depths between 4.3m to 4.8m in BHC6. The bedrock generally consisted of very close to close spacing defects consisting of joints, bedding parts and fractured zones and extremely weathered zones.
4	**Low Strength Bedrock (Siltstone / Sandstone)**	2.7 to 12.0	41.3 to 56.3	2.1 to 2.8	Distinctly to Slightly weathered, low strength sandstone / siltstone bedrock. Occasional medium strength bands were encountered in BHA1M, BHC1, BHC6, BHD4M, BHG1M and BHH4. Core loss was observed from depths between 14.0m to 14.2m in BHC1 and from depths between 3.05m to 4.00m in BHH4. The bedrock generally consisted of close to medium spacing defects consisting of joints, bedding parts and fractured zones and extremely weathered zones.
5	**Medium Strength**	2.3 to 10.3	41.0 to 60.7	-[3]	Slightly weathered to fresh, medium strength sandstone/siltstone. Encountered in BHA4, BHA7M, BHC6

Unit	Material[2]	Depth to Top of Unit (m BEGL)[1]	RL of Top of Unit (m AHD)[1]	Observed Thickness (m)	Comments
	Bedrock (Siltstone / Sandstone)				and BHH7M only.

Note 1 Approximate depth and level at the time of our assessment. Depths and levels may vary across the site.
Note 2 For more detailed descriptions of the subsurface conditions, reference should be made to the borehole logs attached to **Appendix A.**
Note 3 Observed in boreholes BHA4, BHA7M, BHC6 and BHH7M only.

3.2 Groundwater Observations

Groundwater seepage was observed during auger drilling of BHC1, BHD4M and BHG1M only. Following their completion, groundwater monitoring wells were installed in BHA1M, BHA7M, BHD4M, BHG1M and BHH7M and bailed dry. The groundwater levels were then measured within the monitoring wells as per **Table 3-2** below:

Table 3-2 Groundwater Levels

Borehole ID	Groundwater Seepage Level During Augering		Groundwater Level After Well Development		Time elapsed (approx.)
	m BEGL	RL (m AHD)	m BEGL	RL (m AHD)	
BHA1M	Not encountered	Not encountered	2.6	46.4	24 hours
BHA4	Not encountered	Not encountered	N/A	N/A	N/A
BHA7M	Not encountered	Not encountered	3.25	46.25	24 hours
BHC1	5.6	47.7	N/A	N/A	N/A
BHC6	Not encountered	Not encountered	N/A	N/A	N/A
BHD4M	3.2	50.3	5.6	47.9	2 hours
BHG1M	3.0	52.1	1.8	53.3	24 hours
BHH4	Not encountered	Not encountered	N/A	N/A	N/A
BHH7M	Not encountered	Not encountered	4.55	58.45	24 hours

Note 1 N/A – Not available

Water circulation due to coring within the boreholes prevented further observations of groundwater levels within BHA1M, BHA7M, BHD4M and BHG1M. We note that the groundwater levels may not have become evident or stabilised in the augered boreholes within the limited observation period. No long term groundwater monitoring was carried out.

3.3 Test Results

Ten soil and three bulk samples were selected for laboratory testing to assess the following:

- Atterberg Limits and Linear Shrinkage
- Soil aggressivity (pH, chloride and sulfate content and electrical conductivity).
- California Bearing Ratio (CBR);

A summary of the soil test results is provided in **Table 3-3 and Table 3-4** below. Laboratory test certificates are presented in **Appendix B**.

Table 3-3 Summary of Soil Laboratory Test Results

Test/ Sample ID		BHA1M_1.5-1.95	BHA4_1.5-1.95	BHA4_3.0-3.45	BHC1_4.5-4.95	BHC1_7.5-7.95
Unit		2	2	2	2	2
Material Description [1]		Silty CLAY	Sandy CLAY	Sandy CLAY	Silty CLAY	Silty CLAY
Aggressivity	Chloride Cl (ppm)	68	150	-	590	-
	Sulfate SO_4 (ppm)	100	32	-	54	-
	pH	9.6	9.9	-	8.3	-
	Electrical Conductivity (µS/cm)	440	550	-	490	-
	Moisture Content (%)	16.7	18.0	22.9	14.6	17.8
Atterberg Limits	Liquid Limit (%)	-	-	66	-	42
	Plastic Limit (%)	-	-	21	-	15
	Plasticity Index (%)	-	-	45	-	27
	Linear Shrinkage (%)	-	-	16.0	-	12.0

Test/ Sample ID		BHD4M_3.0-3.45	BHG1M_0.5-0.95	BHG1M_1.5-1.95	BHH4_0.5-0.6	BHH7M_0.5-0.9
Unit		2	2	2	2	2
Material Description [1]		Silty CLAY	Silty CLAY	Silty CLAY	Silty CLAY	Silty CLAY
Aggressivity	Chloride Cl (ppm)	-	27	-	59	2.8
	Sulfate SO_4 (ppm)	-	67	-	71	26
	pH	-	8.9	-	7.2	6.6
	Electrical Conductivity (µS/cm)	-	300	-	90	22
	Moisture Content (%)	12.4	20.1	20.9	14.5	13.0
Atterberg Limits	Liquid Limit (%)	42	-	49	-	-
	Plastic Limit (%)	19	-	15	-	-
	Plasticity Index (%)	27	-	34	-	-
	Linear Shrinkage (%)	11.0	-	15.0	-	-

Note 1 More detailed descriptions of the subsurface conditions at each borehole location are available on the borehole logs presented in **Appendix A**.

The Atterberg Limits result on the selected clay sample indicated clays to be of medium to high plasticity and of moderate shrink-swell potential.

The assessment indicated low permeability soil was present above and below the groundwater table. In accordance with Tables 6.4.2(C) and 6.5.2(C) of AS 2159:2009 'Piling – Design and Installation', the results of the pH, chloride and sulfate content and electrical conductivity of the soil provided the following exposure classifications:

- 'Non-aggressive' for buried concrete structural elements; and
- 'Mild' for buried steel structural elements.

Table 3-4 Summary of CBR Test Results

Test/ Sample ID	BHC1_CBR	BHD4M_CBR	BHG1M_CBR
Depth (m BEGL)	3.5-4.5	2.6 to 3.5	0.5 to 1.5
Unit	2	2	2
Material Description [1]	Silty CLAY	Silty CLAY	Silty CLAY
CBR (4-day Soaked) (%)	6.0%	3.0%	4.5%
Maximum Dry Density (t/m^3)	1.69	1.74	1.78
Optimum Moisture Content (%)	20.5	18.8	17.9

Note 1 More detailed descriptions of the subsurface conditions at each borehole location are available on the borehole logs presented in **Appendix A**.

Bulk samples of the Unit 2 material from BHC1, BHD4M and BHG1M were tested for compaction and four day soaked CBR, resulted in values of 3% to 6% when compaction to 100% of Standard Maximum Dry Density (SMDD) and surcharged with 9kg.

59 selected rock core samples were tested by STS to estimate the Point Load Strength Index (Is_{50}) values to assist with rock strength assessment. The results of the testing are summarised on the attached borehole logs.

The point load strength index tests correlated reasonably well with our field assessments of rock strength. The approximate Unconfined Compressive Strength (UCS) of the rock core, estimated from correlations with the point load strength index test results, varied from <1 MPa to 66 MPa.

4. Recommendations

4.1 Geotechnical Issues

Based on the results of the assessment, we consider the following to be the main geotechnical issues for the proposed development:

- Basement excavation and retention to limit lateral deflections and ground loss as a result of excavations, resulting in damage to nearby structures;
- Rock excavation and vibrations;
- Groundwater within the depth of the excavation; and
- Foundation design for building loads.

4.2 Dilapidation Surveys

Prior to excavation and construction, we recommend that detailed dilapidation surveys be carried out on all structures and infrastructures surrounding the site that falls within the zone of influence of the excavation to allow assessment of the recommended vibration limits. The zone of influence of the excavation is defined by a distance back from the excavation perimeter of twice the total depth of the excavation. The reports would provide a record of existing conditions prior to commencement of the work. A copy of each report should be provided to the adjoining property owner who should be asked to confirm that it represents a fair assessment of existing conditions. The reports should be carefully reviewed prior to demolition and construction.

4.3 Excavation Methodology

4.3.1 Excavation Assessment

Prior to any excavation commencing, we recommend that reference be made to the Safe Work Australia Excavation Work Code of Practice, dated January 2020.

EI assumes that the proposed development will require a BEL of between RL 42.2m and 56.4m for the basements, or an excavation depth of between about 0.0m and 19.5m BEGL. Locally deeper excavations for footings, service trenches, crane pads and lifts overrun pits may be required.

Based on the borehole logs, the proposed basement excavations will therefore extend through all units as outlined in **Table 3-1** above. As such, an engineered retention system must be installed prior to excavation commencing.

Units 1 and 2 could be excavated using buckets of large earthmoving Hydraulic Excavators, particularly if fitted with ‘Tiger Teeth’. Excavation of Units 3, 4 and 5 (where encountered) may present hard or heavy ripping, or “hard rock” excavation conditions. Ripping would require a high capacity and heavy bulldozer for effective production. Wear and tear should also be allowed for. The use of a smaller size bulldozer will result in lower productivity and higher wear and tear, and this should be allowed for. Alternatively, hydraulic rock breakers, rock saws, ripping hooks or rotary grinders could be used, though productivity would be lower and equipment wear increased, and this should be allowed for.

Should rock hammers be used for the excavation of the bedrock, excavation should commence away from the adjoining structures and the transmitted vibrations monitored to assess how close the hammer can operate to the adjoining structures while maintaining transmitted vibrations within acceptable limits. To fall within these limits, we recommend that the size of rock hammers do not exceed a medium sized rock hammer, say 900 kg, such as a Krupp 580, and be trialled prior to use. The transmitted vibrations from rock hammers should be measured to determine how close each individual hammer can operate to the adjoining buildings.

The vibration measurements can be carried out using either an attended or an unattended vibration monitoring system. An unattended vibration monitoring system must be fitted with an alarm in the form of a strobe light or siren or alerts sent directly to the site supervisor to make the plant operator aware immediately when the vibration limit is exceeded. The vibration monitor must be set to trigger the alarm when the overall Peak Particle Velocity (PPV) exceeds set limits outlined by a vibration monitoring plan. Reference should be made to **Appendix C** for a guide to acceptable limits of transmitted vibrations.

If it is found that the transmitted vibrations by the use of rock hammers are unacceptable, then it would be necessary to change to a smaller excavator with a smaller rock hammer, or to a rotary grinder, rock saws, jackhammers, ripping hooks, chemical rock splitting and milling machines. Although these are likely to be less productive, they would reduce or possibly eliminate risks of damage to adjoining properties through vibration effects transmitted via the ground. Such equipment would also be required for detailed excavation, such as footings or service trenches, and for trimming of faces. Final trimming of faces may also be completed using a grinder attachment rather than a rock breaker in order to assist in limiting vibrations. The use of rotary grinders generally generates dust and this may be supressed by spraying with water.

To assist in reducing vibrations and over-break of the bedrock, we recommend that initial saw cutting of the excavation perimeters through the bedrock may be provided using rock saw attachments fitted to the excavator. Rock sawing of the excavation perimeter has several advantages as it often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers. However, the effectiveness of such approach must be confirmed by the results of vibration monitoring.

Groundwater seepage monitoring should be carried out during bulk excavation works and prior to finalising the design of a pump out facility. Outlets into the stormwater system will require Council approval.

Furthermore, any existing buried services, which run below the site, will require diversion prior to the commencement of excavation or alternatively be temporarily supported during excavation, subject to permission or other instructions from the relevant service authorities. Enquiries should also be made for further information and details, such as invert levels, on the buried services.

4.3.2 Excavation Monitoring

Consideration should be made to the impact of the proposed development upon neighbouring structures, roadways and services. Basement excavation retention systems should be designed so as to limit lateral deflections.

Contractors should also consider the following limits associated with carrying out excavation and construction activities:

- Limit lateral deflection of temporary or permanent retaining structures;
- Limit vertical settlements of ground surface at common property boundaries and services easement; and
- Limit Peak Particle Velocities (PPV) from vibrations, caused by construction equipment or excavation, experienced by any nearby structures and services.

Monitoring of deflections of retaining structures and surface settlements should be carried out by a registered surveyor at agreed points along the excavation boundaries and along existing building foundations / services/ pavements and other structures located within or near the zone of influence of the excavation. Owners of existing services adjacent to the site should be consulted to assess appropriate deflection limits for their infrastructures. Measurements should be taken in the following sequence:

- Before commencing installation of retaining structures where appropriate to determine the baseline readings. Two independent sets of measurements must be taken confirming measurement consistency;
- After installation of the retaining structures, but before commencement of excavation;
- After excavation to the first row of supports or anchors, but prior to installation of these supports or anchors;
- After excavation to any subsequent rows of supports or anchors, but prior to installation of these supports or anchors;
- After excavation to the base of the excavation;
- After de-stressing and removal of any rows of supports or anchors; and
- One month after completion of the permanent retaining structure or after three consecutive measurements not less than a week apart showing no further movements, whichever is the latter.

4.4 Groundwater Considerations

Groundwater was observed in all monitoring wells as detailed in **Table 3-2**, all of which are above the assumed BEL.

Due to the low permeability of the bedrock profile any groundwater inflows into the excavation should not have an adverse impact on the proposed development or on the neighbouring sites and should be manageable. However, we expect that some groundwater inflows into the excavation along the soil/rock interface and through any defects within the bedrock (such as jointing, and bedding planes, etc.) particularly following a period of heavy rainfall. The initial flows into the excavation may be locally high, but would be expected to decrease considerably with time as the bedding seams/joints are drained. We recommend that monitoring of seepage be implemented during the excavation works to confirm the capacity of the drainage system.

We expect that any seepage that does occur will be able to be controlled by a conventional sump and pump system. We recommend that a sump-and-pump system be used both during construction and for permanent groundwater control below the basement floor slab.

In the long term, drainage should be provided behind all basement retaining walls, around the perimeter of the basement and below the basement slab. The completed excavation should be inspected by the hydraulic engineer to confirm that adequate drainage has been allowed for. Drainage should be connected to the sump-and-pump system and discharging into the stormwater system. The permanent groundwater control system should take into account any possible soluble substances in the groundwater which may dictate whether or not groundwater can be pumped into the stormwater system.

The design of drainage and pump systems should take the above issues into account along with careful ongoing inspections and maintenance programs.

4.5 Excavation Retention

4.5.1 Support Systems

From a geotechnical perspective, it is critical to maintain the stability of all adjacent structures and infrastructures during demolition, excavation and construction works. The setback distances of each building are detailed as below:

- **Basement of Building A**: The proposed basement outline has a minimum setback of approximately 6.5m from the northern site boundary, 2m to 3m from the eastern site boundary and 40m from the northern basement boundary of Buildings B, C, H & J.
- **Basement of Buildings D, E, F & G and Buildings B, C, H & J**: The proposed basement outline has a minimum setback of approximately 52m to 77m from the northern site boundary, 40m from the southern basement boundary of Building A, 23m to 27m from the eastern site boundary, 29m to 44m from the northern basement boundary of Buildings K, L, M & N and 22m from the eastern basement boundary of Buildings P, Q, R, S, T & U.
- **Basement of Buildings K, L, M & N**: The proposed basement outline has a minimum setback of approximately 32m from the eastern boundary, 29m to 44m from the southern basement boundary of Buildings D, E, F & G and Buildings B, C, H & J, 11m from the eastern basement boundary of Buildings P, Q, R, S, T & U and no setback from the southern site boundary.
- **Basement of Building P, Q, R, S, T & U**: The proposed basement outline has a minimum setback of approximately 1.4m to 3.5m from the western site boundary, 27m from the northern site boundary, 22m from the western basement boundary of Buildings D, E, F & G and Buildings B, C, H & J, 11m from the western basement boundary of Buildings K, L, M & N and no setback from the southern site boundary.

Based on the above, the encountered subsurface conditions, the depth of excavation, temporary batters of no steeper than a safe angle of 1 Vertical (V) to 1 Horizontal (H) may be feasible where space allows for the fill and residual soil profile. The above temporary batters should remain stable provided that all surcharge loads, including construction loads, are kept at a distance of at least 2h (where 'h' is the height of the batter in metres) from the crest of the batter. If steeper batters are to be used, then these must be supported by shotcrete and soil nail system designed by a suitable structural or geotechnical engineer. The stability of these batters can be assessed using computer slope stability analysis software such as Slope/W. we can complete such analysis, if commissioned to do so.

Where batters are used, the space between the batters and the permanent retaining walls will need to be carefully backfilled to reduce future settlement of the backfill. Only light compaction equipment should be used for compaction behind retaining walls so that excessive lateral pressures are not placed on the walls. This will require the backfill to be placed in thin layers, say 100mm loose thickness, appropriate to the compaction equipment being used. The compaction specification for the backfill will depend on whether paving or structures are to be supported on the fill. If the fill is to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular fill materials, but if it is only to support landscaped areas of lower compaction specification, say 95% of SMDD, may be appropriate, provided the risk of future settlement and maintenance can be accepted. An alternative for backfill would also be to use a uniform granular material, wrapped in a geofabric.

Where space for temporary batters is not available, especially towards western and southern site boundaries and part of eastern site boundary near Building A, a suitable retention system will be required for the support of the entire depth of the excavation. For this site, we consider that an anchored and/or propped soldier pile wall with shotcrete in between the piles installed to below BEL to be the most suitable. Anchors/props and mass concrete must be installed progressively as excavation proceeds. However, an option of contiguous pile wall may be considered if required by the designer.

A suitable retention system will be required for the support all units. For this site, EI recommends an anchored and/or propped soldier pile wall with mass concrete in between the piles be founded into low to medium strength bedrock (Unit 4). Consideration may be made for some piers, which are not supporting the vertical structural loads of the building, to be terminated at least 0.5m, into Unit 4 material or better, above the base of the bulk excavation levels.

Bored piles are considered to be the most suitable for this site. Tremie pumps may be required where high groundwater seepage inflows are present during the drilling of the bored piles. However, relatively large capacity piling rigs will be required for drilling through the bedrock. The proposed pile locations should take into account the presence of buried services. Further advice should be sought from prospective piling contractors who should be provided with a copy of this report.

4.5.2 Retaining Wall Design Parameters

The following parameters may be used for static design of temporary and permanent retaining walls at the subject site:

- Conventional free-standing cantilever walls which support areas where movement is of little concern (i.e. where only gardens or open areas are to be retained), may be designed using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a, as shown in **Table 4-1;**
- Cantilevered walls, where the tops of which are restrained by the floor slabs of the permanent structure or which support movement sensitive elements, should be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient, Ko, as shown in Table 4-1 below.
- For progressively anchored or propped walls where minor movements can be tolerated (provided there are no buried movement sensitive services), we recommend the use of a trapezoidal earth pressure distribution of 5H kPa for soil, where H is the retained height in meters. These pressures should be assumed to be uniform over the central 50% of the support system, tapering to nil at top and bottom;
- For progressively anchored or propped walls which support areas which are highly sensitive to movement (such as areas where movement sensitive structures or infrastructures or buried services are located in close proximity), we recommend the use of a trapezoidal earth pressure distribution of 8H kPa for soil, where 'H' is the retained height in meters. These pressures should be assumed to be uniform over the central 50% of the support system, tapering to nil at top and bottom;
- All surcharge loading affecting the walls (including from construction equipment, construction loads, adjacent high level footings, etc.) should be adopted in the retaining wall design as an additional surcharge using an 'at rest' earth pressure coefficient, Ko.
- The retaining walls should be designed as drained and measures are to be taken to provide complete and permanent drainage behind the walls. Strip drains protected with a non-woven geotextile fabric should be used behind the shotcrete infill panels for soldier pile walls.
- For piles embedded into Unit 5 or better, the allowable lateral toe resistance values outlined in **Table 4-1** below may be adopted. These values assume excavation is not carried out within the zone of influence of the wall toe and the rock does not contain adverse defects etc. The upper 0.3m depth of the socket should not be taken into account to allow for tolerance and disturbance effects during excavation.
- If temporary anchors extend beyond the site boundaries, then permission from the neighbouring properties would need to be obtained prior to installation. Also, the presence of neighbouring basements and/or services and their levels must be confirmed prior to finalising anchor design**.**
- Anchors should have their bond length within Unit 4 or better. For the design of anchors bonded into Unit 4 or better, the allowable bond stress value outlined in **Table 4-1** below may be used, subject to the following conditions:

1. Anchor bond lengths of at least 3m behind the 'active' zone of the excavation (taken as a 45 degree zone above the base of the excavation) is provided;
2. Overall stability, including anchor group interaction, is satisfied;
3. All anchors should be proof loaded to at least 1.33 times the design working load before locked off at working load. Such proof loading is to be witnessed by and engineer independent of the anchoring contractor. We recommend that only experienced contractors be considered for anchor installation with appropriate insurances;
4. If permanent anchors are to be used, these must have appropriate corrosion provisions for longevity.

Table 4-1 Geotechnical Design Parameters

Material [1]		Unit 1 Fill	Unit 2 Residual Soil	Unit 3 Very Low Strength Bedrock (Sandstone / Siltstone)	Unit 4 Low Strength Bedrock (Sandstone / Siltstone)	Unit 5 Medium Strength Bedrock (Sandstone / Siltstone)
RL of Top of Unit (m AHD) [2]		**49.0 to 63.0**	**48.4 to 62.9**	**44.0 to 62.1**	**41.3 to 56.3**	**41.0 to 60.7**
Bulk Unit Weight (kN/m^3)		**18**	**20**	**23**	**24**	**24**
Friction Angle, ϕ' (°)		**25**	**25**	**28**	**30**	**36**
Earth Pressure Coefficients	At rest, K_o [3]	0.58	0.58	0.53	0.50	0.41
	Active, K_a [3]	0.41	0.41	0.36	0.33	0.26
	Passive, K_p [3]	-	-	2.77	3.00	3.85
Young's Modulus (E, MPa)		2	10	35	80	300
Cohesion, c (kPa)		-	5	35	50	150
Poison's Ratio		-	0.35	0.3	0.3	0.25
Allowable Bearing Pressure (kPa) [5]		-	-	700	1200	3000
Allowable Shaft Adhesion (kPa) [4, 5]	in Compression	-	-	70	120	300
	in Uplift	-	-	35	60	150
Allowable Toe Resistance (kPa)		-	-	-	60	300
Allowable Bond Stress (kPa)		-	-	70	100	250
Earthquake Site Risk Classification		▪ AS 1170.4:2007 indicates an earthquake subsoil class of Class C_e.(Shallow Soil) ▪ AS 1170.4:2007 indicates that the hazard factor (z) for Sydney is 0.09.				

Notes:

1 More detailed descriptions of subsurface conditions are available on the borehole logs presented in **Appendix A.**
2 Approximate levels of top of unit at the time of our investigation. Levels may vary across the site.
3 Earth pressures are provided on the assumption that the ground behind the retaining walls is horizontal.
4 Side adhesion values given assume there is intimate contact between the pile and foundation material and should achieve a clean socket roughness category R2 or better. Design engineer to check both 'piston pull-out' and 'cone liftout' mechanics in accordance with AS4678-2002 Earth Retaining Structures.
5 To adopt these parameters we have assumed that:
- Footings have a nominal socket of at least 0.3m, into the relevant founding material;
- For piles, there is intimate contact between the pile and foundation material (a clean socket roughness category of R2 or better);
- Potential soil and groundwater aggressivity will be considered in the design of piles and footings;
- Piles should be drilled in the presence of a Geotechnical Engineer prior to pile construction to verify that ground conditions meet design assumptions. Where groundwater ingress is encountered during pile excavation, concrete is to be placed as soon as possible upon completion of pile excavation. Pile excavations should be pumped dry of water prior to pouring concrete, or alternatively a tremmie system could be used;
- The bases of all pile, pad and strip footing excavations are cleaned of loose and softened material and water is pumped out prior to placement of concrete;
- The concrete is poured on the same day as drilling, inspection and cleaning.
- The allowable bearing pressures given above are based on serviceability criteria of settlements at the footing base/pile toe of less than or equal to 1% of the minimum footing dimension (or pile diameter).

4.6 Foundations

Following bulk excavation of each building, we expect Unit 2, Unit 3 and Unit 4 to be exposed at BEL of the basements

It is recommended that all footings for the building be founded within medium strength bedrock (Unit 5) or better to provide uniform support and reduce the potential for differential settlements.

4.6.1 Shallow Footings

For the areas where the Unit 5 to be exposed at BEL, pad or strip footings founded within Unit 5 may be preliminarily designed for an allowable bearing capacity of 3000kPa, based on serviceability. However, we note that all basement excavations are expected variable strength bedrock to be exposed at BEL. Deep footings in the form of piles will be required to ensure all foundations are founded into similar strength material.

Due to limited geotechnical boreholes, medium strength bedrock (Unit 5) is established in a few boreholes. EI recommends to complete additional boreholes prior to construction stage to establish a comprehensive sub-surface profile across the site.

Geotechnical inspections of foundations are recommended to determine that the required bearing capacity has been achieved and to determine any variations that may occur between the boreholes and inspected locations.

4.6.2 Pile Footings

Alternatively, for the area where the Unit 2, Unit 3 and Unit 4 to be exposed at BEL, the proposed development may be supported on deep foundations, such as piles, founded into Unit 5.

For piles founded Unit 5 bedrock, these must be embedded a minimum of 0.5m into Unit 5, and can be designed for a maximum allowable bearing pressure of 3000kPa. The allowable shaft adhesion in Unit 5 bedrock may be designed as 10% of the allowable bearing pressure (or 5% for uplift) for the socket length in excess of 0.5m.

At least the initial drilling of piles should be completed in the presence of a geotechnical engineer to verify that ground conditions meet design assumptions.

Where groundwater ingress is encountered during pile excavation, concrete is to be placed as soon as possible upon completion of pile excavation. Pile excavations should be pumped dry of water prior to pouring concrete, or alternatively a tremmie system could be used. Concrete must be poured on the same day as drilling, inspection and drilling.

The aggressivity of natural soils and groundwater (if encountered) should be taken into consideration in the design.

4.7 Basement Floor Slab

Following bulk excavations for the proposed basement, residual soil or bedrock is expected to be exposed at the basement floor BEL.

Following the removal of all loose and softened materials, we recommend that underfloor drainage be provided and should comprise a strong, durable, single sized washed aggregate such as ‘blue metal gravel’. Joints in the concrete floor slab should be designed to accommodate shear forces but not bending moments by using dowelled and keyed joints. The basement floor slab should be isolated from columns. The completed excavation should be inspected by the hydraulic engineer to confirm the extent of the drainage required.

In addition, a system of sub-soil drains comprising a durable single sized aggregate with perforated drains/pipes leading to sumps should be provided. The basement floor slab should be isolated from columns.

Permission may need to be obtained from the NSW Department of Primary Industries (DPI) and possibly Council for any permanent discharge of seepage into the drainage system. Given the subsurface conditions, we expect that seepage volumes would be low and within the DPI limits. However, if permission for discharge is not obtained, the basement may need to be designed as a tanked basement.

4.8 Subgrade Preparation and Engineered Fill

Earthworks recommendations provided in this report should be complemented by reference to AS3798.

1. Fill should be fully excavated down to surface of the residual soils, and stockpiled separately since these materials are not suitable for re-use as engineered fill. Such excavation may need to be carried out with the excavation sides battered at an angle of no steeper than 1 Vertical to 1 Horizontal. The new fill must be 'keyed-in' the sides of these batters.
2. The exposed subgrade at the base of the excavation should be proof rolled with a smooth drum roller (say 12 tonne) used in static or non-vibratory mode of operation. Caution is required when proof rolling near existing infrastructures and utilities (where present). The purpose of the proof rolling is to detect any soft or heaving areas, and to allow for some further improvement in strength or compaction.
3. The final pass should be undertaken in the presence of an experienced geotechnician or geotechnical engineer, to detect any unstable or soft subgrade areas, and to allow for some further improvement in strength/compaction.
4. If dry conditions prevail at the time of construction then any exposed residual clay subgrade may become desiccated or have shrinkage cracks prior to pouring any concrete slabs. If this occurs, the subgrade must be watered and rolled until the cracks disappear.
5. Unstable subgrade detected during proof rolling should be locally excavated down to a sound base and replaced with engineered fill or further advice should be sought. Any fill placed to raise site levels should also be engineered fill, as per the specifications below.

If suspended floor slabs and pavement are designed, then it would be unnecessary to complete any particular subgrade preparation other than stripping of root affected soils from the footprint of the proposed building structures and replaced with surface levelling compacted fill for the floor slab formwork.

4.9 Pavement Design

The design of new pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of fill excavated or imported to the site, as well as vehicle loadings and use. Various alternative types of construction could be used for the pavements. Concrete construction would undoubtedly be the best in areas where heavy vehicles manoeuvre such as trucks turning and manoeuvring. Flexible pavements may have a lower initial cost, but maintenance will be higher. These factors should be considered when making the final choice.

Based on the laboratory test results, the samples collected from the proposed road alignments return the CBR value 3.0% to 6.0%. We recommend that pavement design may be based on the CBR value of 3.0%.

The 3.0% CBR value is low but it may be increased by stabilising the subgrade to a depth of 200mm to 300mm by the addition of lime. When thoroughly mixed and re-compacted to a minimum of 90% of SMDD, a reduction in reactivity along with substantial increase in strength will be achieved.

Alternatively, an appropriate select fill layer comprising of good quality, well graded granular material (such as unbound base or ripped, crushed sandstone with CBR greater than 10%, a maximum particle size of 60mm, well graded and Plastic Index less than 10, compacted to at least 98% of SMDD) may be used below the pavement.

Further soaked CBR tests may be carried out on representative samples of the subgrade to obtain a large population of values to enable a proper statistical analysis to be performed and possibly an increase in the design CBR value. However, it should be borne in mind that even with more test values being obtained there will still be isolated pavement areas where the risk of potential failure and higher maintenance will occur due to the subgrade having a lower CBR value than the statistical characteristic value opted for design purposes.

We recommend that in situ density tests be completed on the proof rolled and prepared subgrade to confirm that at least 98% Standard Maximum Dry Density (SMDD) has been achieved. If the existing fill is removed and replaced with imported fill, the CBR of the imported material may be taken into account. These design values should be confirmed by inspection and Dynamic Cone Penetration (DCP) testing of the subgrade following proof rolling.

All upper (base) course should be crushed rock to RMS QA specification 3051 (2013) unbound base and compacted to at least 100% of SMDD. All lower (sub-base) course should be crushed rock to RMS QA specification 3051 (2013) unbound base or ripped/crushed sandstone with CBR greater than 40%, maximum particle size of 60mm, well graded and Plastic Index less than 10. All lower course material should be compacted to an average of no less than 100% of SMDD, but with a minimum acceptance value of 98% of SMDD.

Concrete pavements should have a sub-base layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (2013) unbound base material (or equivalent good quality and durable fine crushed rock) which is compacted to at least 100% SMDD. Concrete pavements should be designed with an effective shear transmission of all joints by way of either doweled or keyed joints.

Careful attention to subsurface and surface drainage is required in view of the effect of moisture on the clay soils. Pavement levels will need to be graded to promote rapid removal of surface water so ponding does not occur on the surface of pavements. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The capacity of the stormwater collection system from the pavement should be checked and upgraded if necessary. In order to protect the pavement edge, subsoil drains should be provided along the perimeter of all proposed new external pavement areas, particularly in those areas of cut, with invert levels of at least 200mm below subgrade level.

The long-term successful performance of the pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance programme should not be limited to routine compaction density testing only. Other important factors associated with the earthworks includes subgrade preparation, selection of fill materials, control of moisture content and drainage, etc.

5. Further Geotechnical Inputs

Below is a summary of the previously recommended additional work that needs to be carried out:

- Additional Geotechnical Investigation in the form of cored boreholes to confirm the depth and quality of Unit 4 bedrock or better across each building blocks;
- Long term groundwater monitoring, pump out testing and seepage modelling;
- Stability assessment of temporary batters using computer modelling, if required;
- Dilapidation surveys;
- Design of working platforms (if required) for construction plant by an experienced and qualified geotechnical engineer;
- Classification of all excavated material transported off site;
- Witnessing installation of support measures and proof-testing of anchors (if required).
- Geotechnical inspections of unsupported excavations in where betters would be adopted;
- Geotechnical inspections of all new footings/piles by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata; and
- Ongoing monitoring of groundwater inflows into the bulk excavation;

We recommend that a meeting be held after initial structural design has been completed to confirm that our recommendations have been correctly interpreted. We also recommend a meeting at the commencement of construction to discuss the primary geotechnical issues and inspection requirements.

6. Statement of Limitations

This report has been prepared for the exclusive use of Marvin Huang and Caddens Estate Development Pty Ltd who is the only intended beneficiary of EI's work. The scope of the assessment carried out for the purpose of this report is limited to those agreed with Marvin Huang and Caddens Estate Development Pty Ltd

No other party should rely on the document without the prior written consent of EI, and EI undertakes no duty, or accepts any responsibility or liability, to any third party who purports to rely upon this document without EI's approval.

EI has used a degree of care and skill ordinarily exercised in similar investigations by reputable members of the geotechnical industry in Australia as at the date of this document. No other warranty, expressed or implied, is made or intended. Each section of this report must be read in conjunction with the whole of this report, including its appendices and attachments.

The conclusions presented in this report are based on a limited investigation of conditions, with specific sampling and test locations chosen to be as representative as possible under the given circumstances.

EI's professional opinions are reasonable and based on its professional judgment, experience, training and results from analytical data. EI may also have relied upon information provided by the Client and other third parties to prepare this document, some of which may not have been verified by EI.

EI's professional opinions contained in this document are subject to modification if additional information is obtained through further investigation, observations, or validation testing and analysis during construction. In some cases, further testing and analysis may be required, which may result in a further report with different conclusions.

We draw your attention to the document "Important Information", which is included in **Appendix D** of this report. The statements presented in this document are intended to advise you of what your realistic expectations of this report should be. The document is not intended to reduce the level of responsibility accepted by EI, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.

Should you have any queries regarding this report, please do not hesitate to contact EI.

References

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AS1726:2017, *Geotechnical Site Investigations*, Standards Australia.

AS2159:2009, *Piling – Design and Installation*, Standards Australia.

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Abbreviations

AHD	Australian Height Datum
AS	Australian Standard
BEL	Bulk Excavation Level
BEGL	Below Existing Ground Level
BH	Borehole
DBYD	Dial Before You Dig
DP	Deposited Plan
EI	EI Australia
GI	Geotechnical Investigation
NATA	National Association of Testing Authorities, Australia
RL	Reduced Level
SPT	Standard Penetration Test
T-C	Tungsten-Carbide
UCS	Unconfined Compressive Strength

Figures

Figure 1	Site Locality Plan
Figure 2	Borehole Location Plan
Figure 3	Section A-A’
Figure 4	Section B-B’
Figure 5	Section C-C’
Figure 6	Section D-D’

Appendix A – Borehole Logs And Explanatory Notes

Appendix B – Laboratory Certificates

Appendix C – Vibration Limits

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally considered to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) directions, in the plane of the uppermost floor), are summarised in **Table A** below.

It should be noted that peak vibration velocities higher than the minimum figures in **Table A** for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual conditions of the structures.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table A DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity (mm/s)			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10 Hz	10 Hz to 50 Hz	50 Hz to 100 Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (e.g. buildings that are under a preservation order)	3	3 to 8	8 to 10	8

Note: For frequencies above 100 Hz, the higher values in the 50 Hz to 100 Hz column should be used.

Appendix D – Important Information