12 December 2008

Eureka 2 Project 8 Pty Ltd
BD (NSW) Project 24 Pty Ltd
PO Box 826
NEWCASTLE NSW 2300

Attention: Bob Hawes

Dear Bob

RE: HONEYSUCKLE CENTRAL

GEOTECHNICAL ASSESSMENT

INTRODUCTION

We refer to your instruction to provide a Geotechnical assessment of the abovementioned site in response the Director General's Requirement (DGR's) and your intentions to lodge a Project Application for development.

We note this site was formerly referred to as Stages 4 and 5 of the Lee Wharf Development. Moreover, we confirm we have undertaken directly and are aware of the existence of extensive and exhaustive reports conducted on this and adjacent sites in relation to Geotechnical investigations over the last ten years. Consequently, the conclusion of this assessment will note that the geotechnical conditions at this site have not changed and the conclusions and recommendations from these previous studies remain the same and can be applied to the future development of the site.

BACKGROUND

The DGR's propose the following scope in relation to the project and the site:

*"**Geotechnical Report** – **prepared by a recognised professional which assesses the risk of Geotechnical failure on the site and identifies design solutions and works to be carried out to ensure the stability of the land and structures and safety of persons**"*.

There have been a number of Geotechnical Reports which include data on the conditions at this site including;

- 'Report on Geotechnical & Mine Subsidence Investigation – Proposed Commercial Development, Civic Workshops, Honeysuckle' by D J Douglas & Partners Report 16670/1 November 1993;

- ‘Newcastle Newspapers Proposed Development, Lot 4 Honeysuckle, Geotechnical Investigation’ Coffey Partners International Report N6074/1-AB July 1997;
- ‘Honeysuckle Development, Geotechnical Investigation Report’ Golder Associates;
- ‘Lee Wharf Development, HB5B, HB7 and HB9 Honeysuckle Drive, Newcastle, Geotechnical Investigation” Coffey Geosciences Report N08459/01-AH 21 May 2003.

For the purposes of this project, we have reviewed these reports including the logs of the boreholes, results of laboratory testing and conclusions and recommendations of each report. These reports note that the conditions experienced on site will permit the construction of a multi storey building subject to detailed assessment and design towards construction certificate stage. Each of the reports indicated that numerous design solutions were available for support of multi storey development similar to the proposed Honeysuckle Central Development and with normal design and construction procedures such development would ensure the stability of the land and structures and safety of persons. The Conclusions and Recommendations from the most recent and relevant report for the site (formerly known as HB5B) include;

“FOUNDATION OPTIONS

1.1 General

Due to the presence of uncontrolled fill and deep, compressible alluvial soils at this site, the use of shallow footings is not recommended except for structures that are lightly loaded (i.e. bearing pressure not more than 10 kPa) and are not sensitive to total or differential settlements.

It is recommended that buildings at this site be supported on piles founded within the Unit 3 sand. If sufficient pile capacity cannot be achieved with a single pile, additional piles could be used to form a group. Settlement of pile groups can be greater than for a single pile and this possibility should be assessed if pile groups are required. Pile group interaction effects are expected to be minimal provided that piles are installed vertically and at least 2.5 maximum diameters apart, (2 helix diameters apart for steel screw piles).

1.2 Design Parameters

Table 2 presents a summary of ultimate design parameters that have been adopted for the relevant stratigraphic units. These parameters are simplifications of the real subsurface properties and are intended for foundation design purposes for downward loading. For uplift loading, reference should be made to Section 1.2.1.

TABLE 2 – SUMMARY OF ADOPTED ULTIMATE DESIGN PARAMETERS

UNIT	AREA	MATERIAL	q_c (MPa)	TIMBER PILES		ATLAS PILES		CFA PILES	
				f_s (kPa)	f_b (kPa)	f_s (kPa)	f_b (kPa)	f_s (kPa)	f_b (kPa)
1	Entire site	FILL, GRAVELLY SAND – loose to	3	Load bearing contribution ignored					

		medium dense							
2	Entire site	SANDY CLAY / CLAYEY SAND – very soft to stiff	0.25	Load bearing contribution ignored					
3	North of Honey - suckle Drive	SAND – medium dense to very dense with some loose lenses	15 at pile toe 20 average	150	7500	150	7500	150	2000
	South of Honey - suckle Drive		10 at pile toe 15 average	120	5000	150	5000	150	1500
5A	Entire site	SANDSTONE extremely to highly weathered, very low strength (Class V/IV)	-	**Bored cast *in-situ* concrete piles – Serviceability** [1] f_s = 100 kPa; f_b = 1000 kPa (for portion of shaft in Unit 3, use f_s for timber piles)					
5B	Entire site	SANDSTONE – low to medium strength (Class IV/III)	-	**Bored cast *in-situ* concrete piles – Serviceability** [1] f_s = 150 kPa; f_b = 1500 kPa (for portion of shaft in Unit 3, use f_s for timber piles)					

NOTES: q_c = cone resistance, f_s = skin friction, f_b = end bearing

1. Parameters are for serviceability conditions. No geotechnical reduction factor need be applied.

The above ultimate parameters should be used in limit state pile design in accordance with Australian Standard AS 2159-1995, *Piling – Design and Installation*. Appropriate geotechnical reduction factors should be applied as discussed in the following sections. See also Table 2, note No. 1.

1.2.1 Piles in Tension (Uplift Loading)

Timber, Atlas and CFA piles could be used to resist uplift loads as well as downward loads. As a preliminary guide, for piles of uniform cross section, the value of f_s may be taken as 0.6 times the value for downward loading. For tapered piles (non-uniform cross section), the value of f_s may be taken as 0.3

times the value for downward loading. This preliminary guide is expected to be very conservative, especially for Atlas and CFA piles that have shaft corrugations that develop significant interlock.

Alternatively, Steel Screw piles could be used to resist uplift loads. Steel screw piles are discussed in Section 1.9. It is recommended that pile uplift load testing be carried out no matter which pile type is proposed as the increased allowable pile capacity achieved is expected to significant.

1.3 Pile Types

Four piles types have been considered for this project. They are:

1. Driven Timber Piles – driven to a design refusal set into the Unit 3 sand.
2. Atlas Screw Piles - founded in the Unit 3 sand.
3. Grout Injected CFA Piles – founded in the Unit 3 sand.
4. Steel Screw Piles – founded in the Unit 3 sand. Refer to Section 1.9.

Driven Timber Piles

Driven piles provide resistance to load by both end bearing and shaft friction. The shaft friction developed along the pile length would be used to resist both axial compression and tension loads. The piles could be driven to a design refusal set. The ultimate bearing capacity of the pile may be determined from Hiley or wave equation analyses once hammer type, size and pile size and lengths are known.

Atlas Screw Piles

Atlas piles are a reinforced cast in-situ, concrete displacement piles with a sacrificial steel tip. They are constructed by Frankipile Australia Pty Ltd. The pile tip is screwed into the ground by a purpose built piling rig, once the design founding level is reached, concrete is poured down the inside of the drive shaft which is progressively retracted. The pile tip remains in the ground permanently forming the base of the pile. Typical pile diameters are 610 mm, 660 mm, 710 mm and 800 mm (maximum diameter). Atlas piles can be installed as singles or in groups.

Atlas pile shafts have corrugations along their length created by the screwing action of the steel tip. The lateral displacement of soil from the screwing action densifies the soil immediately surrounding the shaft. These two features allow the Atlas pile to generate significant friction resistance along its shaft. Load is also transferred to the soil by end bearing through the tip. Uplift loads are resisted by the self-weight of the pile as well as shaft friction.

Grout Injected CFA Piles

Grout injected piles are installed with a rig that screws in a continuos flight auger (CFA) to the design depth (a function of the pile size and subsurface condition). The auger is then withdrawn whilst grout is pumped through the auger to the hole below. A steel reinforcing cage in then inserted into the wet grout mix. A pile integrity test is recommended for each pile after installation to confirm the continuity of the section.

Unlike Atlas piles and driven piles, grout injected bored piles are not displacement piles and more conservative design parameters are appropriate.

1.4 Pile Capacities and Founding Levels

To maximise the allowable bearing capacity, the piles should not be founded too deep within the Unit 3 sand. This depth limitation is to avoid the zone of influence of the pile toe from being within the Unit 4a or 4b soil that has a lower ultimate bearing capacity than the Unit 3 sand. The extent of the zone of influence can be taken as being to three maximum pile diameters below the pile toe for a single pile. For pile groups, this zone of influence becomes larger. Hence, for a 0.3m diameter pile, the zone of influence can be taken as say 1m. The toe of such a pile should not therefore be any deeper than 1m above the base of Unit 3. For a 1m diameter pile, the zone of influence would extend about 3m below the toe of the pile.

The estimated RL of the base of Unit 3 is shown in Figure 7. This figure has been prepared by extrapolating data between points and its accuracy between such points cannot be guaranteed, as such it should be used with caution. Based on the depth of Unit 3 as shown in Figure 7, maximum pile toe depths are given in Table 3.

TABLE 3 – MAXIMUM PILE TOE DEPTHS

BUILDING	0.3m DIAMETER PILE	1.0m DIAMETER PILE
A1 and A2	RL -11m AHD	RL -9m AHD
B1 and B2	RL --10m AHD	RL -8m AHD
B7	RL -8.5m AHD	RL -6.5m AHD

For the maximum pile toe depths given in Table 3, it is expected that the maximum amount of pile shaft within Unit 3 will be between 3m to 8m. Driven piles may however reach their design set at shallower depths.

Based on the maximum pile toe depth given in Table 3, and assuming a basement depth of 3m, maximum pile lengths are expected to be between 5.5m and 10m.

Ultimate skin friction and end bearing pressure for the above pile types are given in Table 2. However, the following points should be borne in mind in the design process:

- Where the founding stratum is underlain by a weaker layer, the pile toe should be located at least three pile diameters above the top of the weaker layer as previously discussed;
- Settlement criteria may govern for structures sensitive to total and/or differential settlement (see further discussion below);
- Piles should be no closer than 2.5 pile diameters apart. If closer than this, interaction effects between piles should be taken into account and pile group settlement assessed;
- Pile design should be in accordance with Australian Standard AS 2159-1995, *Piling – Design and Installation.* Appropriate geotechnical reduction factors should be applied to the ultimate parameters provided in Table 2;
- More accurate ultimate bearing capacities and settlement estimates can be obtained by undertaking static

load tests on trial piles as discussed in Section 1.6.

- Where uplift is critical, it is recommended that uplift load tests be conducted.

Driven Timber Piles

Vibration limits may restrict the amount of energy per blow imparted on the pile when driving piles as discussed in Section 1.7.2.

Typical settlements are expected to be in the order of about 1% of the effective pile diameter for a single pile driven to a design pile refusal set.

Atlas Screw Piles

The surface area of the shaft (A_s) is based on an average of the minimum shaft diameter and the maximum diameter.

Settlement for a single pile of 0.8 m diameter with a load of 1000 kN has been estimated to be less than 10mm. For a 4700kN load the settlement is estimated at 30mm. Ideally, the deflection response of the piles should be evaluated by static load tests.

In addition to pile load testing, a useful indicator of achieved pile capacity may be derived from correlations with the torque required to turn the pile tip. The torque should be measured with a calibrated torque indicator that is capable of giving continuos read out. A relationship between the torque required and the axial capacity (correlation) should ideally be established by a series of load tests conducted on site. This relationship can then be used for other piles where load testing has not been conducted to confirm the achieved pile capacity.

Grout Injected CFA Piles

The calculation of surface areas A_s and A_b should be based on the nominal pile diameter minus 50 mm.

Settlement for a single pile of 0.8 m diameter with a load of 1000 kN has been estimated to be less than 10mm. For a 4700kN load the settlement is estimated at 30mm. Ideally, the deflection response of the piles should be evaluated by static load tests.

1.5 Pile Geotechnical Reduction Factors

In accordance with Australian Standard AS 2159-1995, *Piling – Design and Installation,* a geotechnical reduction factor (φ_g) of 0.5 is recommended at this stage. AS 2159 allows less conservative factors under certain conditions. For example, if the Hiley formula or wave equation analysis is used to determine the ultimate capacity (R_{ug}), A geotechnical reduction factor (φ_g) of 0.6 can be applied to the maximum load applied to determine the allowable pile capacity (R^*_g).

Table 4 provides factors and circumstances under which they are appropriate.

TABLE 4 – GEOTECHNICAL REDUCTION FACTORS

APPROPRIATE CIRCUMSTANCES	φ_g
Static analysis using CPT data	0.5
Static analysis using CPT data + Measurement during installation of propriety displacement piles using well established in-house formula (ATLAS AND DRIVEN PILES ONLY)	0.6
Static analysis using CPT data + Static load testing of 1% of piles	0.7
Static analysis using CPT data + Static load testing of 3% or more of piles	0.9

No geotechnical reduction factor has been applied to estimated settlements.

1.6 Static Pile Load Testing

Static pile load testing can be conducted on trial piles to ultimate failure or not to failure (proof). Where proof testing is conducted the geotechnical reduction factor should be applied to the maximum load applied during the test. Appropriate geotechnical reduction factor are given in Table 4. All load tests should be conducted on the site under conditions as similar as possible to the actual pile conditions anticipated.

The allowable capacity of a pile may be controlled by serviceability rather than ultimate load capacity and deflection and settlement of the piles should be measured during tests and recorded at different load increments. In addition, ultimate failure may not be obvious and load versus deflection plots should be made to assist in this determination.

Static pile load testing should be conducted in accordance with AS 2159-1995.

1.7 Pile Construction Issues

1.7.1 Pile Excavation / Driving Conditions

Augering in the Unit 1 fill is not expected to be a problem based on the drilling resistance encountered in the drilling of boreholes in 2003. Some resistance may be encountered north of Honeysuckle Drive where there may be buried railway sleepers and steel rail. The existence of other subsurface obstructions in other areas of fill cannot be precluded.

Pile driving resistance is expected to be high within Unit 1 fill around the locations CPT03-1 and CPT03-5, that is the northwest corner of Building A2 and the northeast corner of Building A1. The fill in these locations had to be pre-augered as the CPT refused within the fill. At CPT03-3, near the southwest corner of Building A2, a dense to very dense layer of sand was encounter within Unit 2. As discussed in Section 6.3, this sand is thought to be actually fill but has similar properties to Unit 3 sand. Pile termination within this material is not permissible and pre-augering to depths of about 4m may be required for driven piles to penetrate this layer. This depth is approximately 2m below the groundwater table in sand and so casing will be required if the pile hole is to be pre-augered to this depth.

1.7.2 Ground Vibrations

Driving piles will induce vibrations in the sand that may cause concern to occupiers of existing nearby buildings that are supported on high-level footings and / or unacceptable levels of noise to nearby commercial and residential establishments. If driven piles are intended for the foundations, we recommend that a dilapidation survey be conducted on surrounding buildings before and after piling and that ground vibration monitoring be carried out during piling. Coffey are able to provide vibration monitoring services if required.

Ground vibrations are expressed as peak particle velocity (ppv) and can be estimated using the method proposed by Massaisch (1993) Ref. 7. This method was used in the D.J. Douglas report (Ref. 1) for a pile impact energy of 25 000 joules and estimated the following:

- ppv = 2 mm/sec at a distance of 120m from the pile.
- ppv = 10 mm/sec at a distance of 25m from the pile.

An alternative method is proposed by Attewell *et al* (1990 & 1991) and Oliver and Selby (1991) as follows:

$$\nu = b\left(\frac{\sqrt{E}}{r}\right)^{x}$$

Where: ν = peak particle velocity (mm/s)

b = 1.33 for impact hammers and 1.18 for vibrodrivers

x = 0.73 for impact hammers and 0.98 for vibrodrivers

E = pile energy per blow (J)

r = surface distance (m)

It should be noted that the above method does not satisfactorily estimate vibrations less than 10m plan distance from the pile.

As shown in the following chart, an impact hammer imparting 25000 joules / blow is estimated to result in:

- ppv = 2 mm/sec at a distance of 90m from the pile.
- ppv = 10 mm/sec at a distance of 10m from the pile.

For a smaller impact energy of 15000 joules / blow, the vibrations are less, but still significant at:

- ppv = 2 mm/sec at a distance of 70m from the pile.
- ppv = 5 mm/sec at a distance of 20m from the pile.

The maximum allowable peak particle velocity will depend on the structures that are at risk and also the human occupancy in the affected area. Guidance as to what humans general perceive as tolerable levels of ground vibration and what certain classes of structures can tolerate are provided by Tynan (1973) Ref. 7 and are reproduced in Table 5.

TABLE 5 – GROUND VIBRATION LIMITS SUGGESTED BY TYNAN (1973)

STRUCTURE TYPE / HUMAN PERCEPTION	PEAK PARTICLE VELOCITY (mm/sec)
Historical and ancient buildings, ruins and monuments	2
Buildings visibly damaged, cracked	4
Structurally sound buildings (technically in good order)	8
Industrial buildings, concrete buildings – generally without plaster	10 to 40
Imperceptible to humans (at 5 Hz) [1]	< 0.3
Just perceptible to humans (at 5 Hz)[1]	0.3 to 0.9
Clearly perceptible to humans (at 5 Hz)[1]	0.9 to 3.0
Annoying to humans (at 5 Hz)[1]	3.0 to 14
Unpleasant or disturbing to humans (at 5 Hz)[1]	> 14

1 – Human sensitivities listed are for vertical vibrations (after Reiher and Meister 1931)

Based on the above information, a maximum allowable peak particle velocity of between 2 to 5 mm/sec is recommended for areas occupied by people and historic buildings.

Several measures could be used to reduce the ground vibrations near sensitive areas. Such measures might include reduced impact energies, pre-auguring the start of the pile (above the groundwater level) and perimeter trench around the sensitive area to dampen P- and S-waves. The effectiveness of such measures would need to be assessed by ground vibration monitoring.

1.8 Floor Slabs

1.8.1 General

Depending on the floor loading, settlement and differential settlement criteria, cast on grade floor slabs may not be feasible due to the presence of:

- Uncontrolled fill shown in Figure 5;
- Soft or loose sandy clay and clayey sand within Unit 2. This layer is assessed to be moderately to highly compressible.

Assuming these materials have been pre-consolidated with a pressure of up to 20 kPa, an assumed floor loading of 10 kPa is expected to result in settlements of between 10 mm to 20 mm.

To reduce settlement, compaction using a vibrating drum roller is feasible to improve the uniformity and density of the Unit 1 fill. However, improvement of the ground below Unit 1 is likely to require removal of the Unit 2 layer and replacement with compacted imported fill. Excavation to a depth of 5m will require dewatering as the groundwater level is at a depth of about 2m.

The use of vibrating compaction equipment should consider the risk of structural damage to nearby buildings from the generated ground vibrations, see Section 8.7.2.

Other options for supporting the ground floor slabs for this project are discussed in the following sections.

1.8.2 Piling the Floor Slab

The ground floor slab may be suspended on piles which also support the column loads. Design of the piles may be based on the recommendations given in Section 8.4 or Section 8.9. This approach is likely to result in significantly greater pile load capacity requirements as well as a substantial slab.

Alternatively, the floor slab may be suspended on closely spaced driven timber piles or steel screw piles founded in the Unit 3 sand. For a 10 kPa floor loading, treated timber piles having a toe diameter of 180mm and founded at RL-7m (ie about 9m depth), and at a grid spacing of 1.5m may be adopted. Settlement of such a floating pile system is assessed to be 5 mm to 10 mm. Steel screw piles could be used in a similar fashion with the expected advantage of an increased uplift capacity.

1.9 Steel Screw Piles

Steel screw piles are a preformed displacement pile, comprised of one or more helixes on a shaft. Because of the minimal soil displacement during installation, the pile is considered as a non-displacement pile for design purposes.

The piles are screwed into the ground, usually by a hydraulic head, mounted on an excavator. The diameter of the helix typically ranges from 350 mm to 600 mm although larger diameters are used. Screw piles can be installed vertical or raked and singular or in groups.

Screw piles in sand transfer axial compression and tension loads to the soil by end bearing of the helix or helixes as well as frictional resistance between the shaft and sand. Apart from the ultimate bearing capacity of the soil, the following factors may limit the ultimate capacity of the screw pile:

- Structural strength of the pile shaft, helixes and connections.
- Deflection of the helixes under load leading to a reduction in end bearing area and axial capacity, (higher deflections are expected for larger diameter piles).
- Eccentricity of the applied load and associated stresses induced in the pile shaft.
- Durability and corrosion resistance of the pile.

The design of steel screw piles should not only take the geotechnical and structural pile capacities into account separately but also the combined action when geotechnical and structural strengths are mobilised together through formation of a plastic hinge in the helix (Ref. 8). If the screw pile option is chosen, we recommend that design optimisation be conducted that includes pile load testing. As screw pile dimensions, configurations and installation procedures vary between piling contractors, pile design optimisation should be conducted in consultation with the piling contractor proposed to undertake the work. The piling contractors typically have established performance data from load testing and experience, specifically for their piles types and configurations.

As a preliminary guide, ultimate downward axial capacities of between about 500kN to 800kN are probably achievable with settlements of less than 10 mm, depending on the pile dimensions and configurations. In uplift, the ultimate axial capacities are expected to similar or perhaps slightly less if geotechnical strength is limiting. More accurate ultimate bearing capacities and settlement estimates can be obtained by undertaking static load tests of trial screw piles.

Multi helix piles can be used to increase pile bearing capacity. If for example, a two-helix pile is used, the pile capacity can be increased. The practice of doubling the capacity for a two-helix pile should, however, be used with caution and verified by static load tests on site. Settlement of a multi helix pile is also expected to be greater than two or more equivalent single helix piles.

As a general guide the following points are offered:

- Piles subject to uplift should have the upper most helix embedded at least five times the smallest helix diameter.
- The distance between helixes on a single pile shaft should be at least three times the helix diameter.
- All screw pile shafts should be filled with 40 MPa concrete for additional corrosion protection.

For pile groups, settlements can increase to levels higher than expected from individual piles due to interaction effects of the piles acting as a group rather than as a number of individual piles. In this case, the interaction effects are expected to be insignificant provided that the piles are installed vertically and at least two helix diameters apart.

In accordance with AS 2159-1995, no geotechnical reduction factor is applied to estimates of settlement.

EARTHWORKS

1.10 Stability of Excavations

Temporary excavations within the upper fill layer (Unit 1) above the groundwater level should be carried out no steeper than 1.5H:1V. Furthermore, if excavations to within 0.5m above the groundwater level is required, the floor of the excavation may be unstable under construction traffic. The placement of geofabric and granular rock fill to act as a working platform would be required in such situations.

If excavations to or below the groundwater level are required, dewatering will be required and excavation batters are likely to have to be made at slopes flatter than 2H:1V. Detailed slope stability analysis will be required for the specific excavation requirements at design stage. Alternatively, sheet pile walls could be used to support the sides of excavations. Again, detailed sheet pile wall design will be required for the specific excavation requirements.

1.11 Dewatering

Dewatering for excavations below the groundwater level is expected to be achievable with a series of spear points. Groundwater, soil permeability and dewatering studies have been presented in Ref. 1. As mentioned in Ref. 1, care should be exercised in dewatering around existing buildings as lowing of the groundwater table beneath the footings supporting such structures can lead to settlements that may damage cause structural and/or architectural damage.

A dewatering assessment presented in Ref. 1 concluded that a recharge system will be required to prevent the effect of dewatering (drawdown) extending considerable distances from the site. It should be noted that this assessment was conducted for a 40m and 120m long excavation to a depth of RL -3m AHD. It is recommended that a dewatering assessment be conducted specifically for the proposed excavations of this development.

1.12 Retaining Wall Design Parameters

For retaining walls to be constructed to support the Unit 1 fill, the following design parameters may be adopted:

- Bulk unit weight γ_b = 19 kN/m^3
- Coefficient of active earth pressure K_a = 0.35
- Coefficient of earth pressure at rest K_o = 0.5
- Coefficient of passive earth pressure K_a = 3.0

K_o should be used instead of K_a behind the retaining structures for walls which are relatively rigid and/or propped (e.g. basement walls).

Under serviceability conditions, it is recommended that a design water level of 1m below the existing ground level be adopted. This allows for approximately a 1m increase in the groundwater level measured during this and previous investigations, to allow for prolonged wet weather periods.

1.13 Ground Vibrations

Apart from pile driving, other sources of construction related ground vibrations have the potential to damage nearby structures. In particular, the historical Australian Wine Society Building. The vibration guidelines presented in Table 5 are generally relevant but do not take into account the higher frequency that vibrating rollers operate at.

It is recommended that ground vibration monitoring be conducted when there is a risk of structural damage to neighbouring property. The monitoring should comprise dilapidation surveys as well as pre-defined ground vibration alarm levels and associated reaction measures."

Importantly, these investigations provided Geotechnical data in sufficient detail for design of the Honeysuckle Central development for PA submission purposes.

We have also worked with the structural engineer in the approach to building foundation and footing design in order to gain an understanding of the requirements of the project subject to its approval and eventual construction.

DETAILED ASSESSMENT

Once the PA is approved more detailed investigations would be carried out to allow final detailed design of the approved development.

The field investigation program for the Geotechnical component will be designed to address the requirements of the structural engineers including support of foundations and retaining walls.

To supplement the existing geotechnical information we propose 15 Piezocone Penetrometer Probes pushed with a dedicated truck mounted penetrometer rig. The probes would be pushed through the dense sands into the underlying stiff clays.

In addition four boreholes would be drilled using a truck mounted rig equipped with continuous spiral flight augers. Mud drilling techniques would be used in sands / and soft alluvium. It is anticipated the boreholes will be drilled to depths of about 6m. Standard penetration tests would be carried out in sands and thin wall samples (U50 tubes) taken in clays and / or peats.

We propose to install standpipe piezometers in two boreholes to allow monitoring of ground water levels during the investigation.

Our report would also include a discussion of the geotechnical conditions found at the site and their significance to the proposed development. We would provide recommendations on the following:

- Site preparation;
- Excavation conditions;
- Alternative footing types and founding levels, including recommendations as to allowable bearing pressure and probable settlements;
- Special requirements for construction procedures and or site drainage;
- Groundwater aggressivity to buried structural elements;
- Acid sulfate soil conditions and requirements for an acid sulfate soil management plan.

CONCLUSION

The history of investigation and assessment on this site confirm the capacity and capability of the ground conditions subject to footing and foundation design to support multi storey development consistent with the current Honeysuckle Central proposal. These assessments also show the risks that may be present are capable of being mitigated to ensure the stability of the land and structures and safety of persons.

Detailed investigations in the next stage of design will include specific tests to respond to the structural building designs and we do not anticipate encountering circumstances that would compromise or prevent to structure from being erected owing to geotechnical conditions.

For and on behalf of Coffey Geotechnics Pty Ltd

Arthur Love

Principal Geotechnical Engineer

Attachment:

Important Information about your Coffey Report