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Report on Preliminary Geotechnical Assessment

Proposed Northbank Enterprise Hub
Lot 1001 DP 1127780, 365 Tomago Road
Tomago

Prepared for
ADW Johnson Pty Ltd
on behalf of
Northbank Enterprise Hub Pty Ltd

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Report on Preliminary Geotechnical Assessment
Proposed Northbank Enterprise Hub
Lot 1001 DP 1127780, 365 Tomago Road, Tomago

1. Introduction

This report presents the findings of a preliminary geotechnical assessment of the site of a proposed industrial subdivision at Tomago Road, Tomago. The geotechnical assessment was undertaken at the request of Mr Craig Marler of ADW Johnson Pty Ltd (ADW) on behalf of Northbank Enterprise Hub Pty Ltd with reference to our proposal NCL100153 dated 31 March 2010.

It is understood that the site is proposed to be developed for industrial/commercial purposes and portions of the site will need the surface level raised using imported fill materials.

It is further understood that the site is currently zoned IN1 General Industrial under the provisions of SEPP (Major Development) 2005. The current zoning encourages industrial development to proceed.

The geotechnical assessment comprised a review of documents held by Douglas Partners, including in-house reports prepared for the proposed Austeel Development, Transport Corridor, Hunter River North Arm, Tomago Aluminium Smelter, Tomago Sandbeds and others. A site visit be a senior geotechnical engineer was also undertaken.

The site occupies approximately 239 ha, located between Tomago Road and the north arm of the Hunter River. It is proposed to raise the surface level of about half of the site by filling, to create an industrial subdivision.

The principal aim of the review was to assess geotechnical constraints to development, and provide comment on potential foundation treatment solutions, settlements and earthworks.

A Preliminary Contamination Assessment (PCA) report (Ref 10) was prepared concurrently with this geotechnical assessment report as a separate document.

2. Site Description

The site is identified as Lot 1001 DP 1127780, 365 Tomago Road, Tomago, New South Wales, and is shown in Drawing 1, Appendix A.

The subject site is situated on the south-eastern side of Tomago Road, Tomago and is bordered by the north arm of the Hunter River on the south-west, and low-lying grazing land to the south and south-east. The area to the south-east of the site is an environmental conservation zone.

The site has a eastern frontage of about 1300 m to Tomago Road, and comprises an irregular shaped area of about 239 ha. Photos 1 to 3 show typical views of the site.

Figure 1 - View of the site, looking north-east along Power Transmission Easement

Figure 2 - View looking south-east showing tractor ruts and exposed clayey topsoils

Figure 3 - Saturated ground surface along access track and adjacent paddock

A detailed description of the current site condition and more photographs of the site are presented in the Preliminary Contamination Assessment report (Ref 10).

3. Proposed Development

It is understood that an industrial subdivision is proposed for the site, comprising industrial-sized allotments and roads to service the allotments. Concept plans for the proposed development were not available at the time of preparing this report. It is likely that the fill surface levels will need to be above the 1:100 flood levels The design finished surface level of the fill platform would be expected to be about RL 2.5 to 3.10 AHD, therefore requiring at least 2 m of fill.

The site will ultimately require a substantial amount of fill material to raise the ground to the design levels. The source of fill material has not yet been determined.

4. Data Review

4.1 Published Data

4.1.1 Regional Geology

Reference to the 1:250,000 Newcastle Geology map indicates that the site is underlain by Quaternary alluvium, which typically comprises gravel, sand, silt and clay. The underlying bedrock comprises siltstone and sandstone of the Permian aged Tomago Coal Measures.

4.1.2 DPI NSW Coastal Quaternary Geology Map

The NSW Department of Primary Industries (DPI) NSW Coastal Quaternary Geology Data Package (Ref 3) provides more detail on sedimentary deposits than Geology maps. This data indicates that a variety of Quaternary Pleistocene and Holocene units are likely to be present on the site. Drawing 1, attached, shows the approximate DPI-mapped Quaternary units overlaid on the site plan. Table 1 below describes these units.

Table 1 – DPI Quaternary Alluvium Units (see Drawing 1)

Geological Symbol	Age	Unit	Lithology
Qhap	Holocene	Floodplain	Silt, fluvial sand, clay
Qhas	Holocene	Backswamp	Organic mud, peat, silt, clay
Qhea	Holocene	Estuarine paleochannel fill	Organic mud, peat, clay, silt, marine sand
Qhem	Holocene	Estuarine basin and bay	Clay, silt, shell, fluvial or marine sand
Qhemd	Holocene	Fluvial delta front	Fluvial sand, silt, clay, shell
Qher	Holocene	Estuarine shoreline ridge and dune	Fluvial sand, marine sand, gravel, silt, clay, shell
Qhes	Holocene	Saline swamp	Organic mud, peat, clay, silt, marine sand, fluvial sand
Qpb	Pleistocene	Undifferentiated barrier unit	Marine sand, indurated sand
Qpbd	Pleistocene	Barrier dune	Marine sand, indurated sand

4.1.3 Acid Sulphate Risk Map

The NSW Department of Land and Water Conservation (now part of the NSW Department of Natural Resources) published maps indicating the risk of encountering for acid sulphate soils. The Acid Sulphate Soil Risk Map for Beresfield (Ref 9232N3) indicates that the site covers two categories:

- the northern part of the site, comprising a strip alongside Tomago road of 120 m to 200 m width, lies in an area designated Wa4(p). This indicates an alluvial sandplain of Pleistocene age and an elevation exceeding 4 m. This zone has a low probability of acid sulphate soils at a depth or more than 3 m below ground surface.
- The remainder of the site lies within an area designated Ap2. This indicates an alluvial floodplain with an elevation between 2 m and 4 m. This area has a high probability of acid sulphate soil conditions occurring between 1 m and 3 m below the ground surface.

It is noted that the elevation of the site is lower than the ranges indicated on the Acid Sulphate Risk map, and is generally less than 1 m over a large portion of the site, rising to around 2 m to 3 m in the vicinity of Tomago Road. This is discussed further in Section 5.11.

4.2 Previous Geotechnical Investigations at the Site

4.2.1 Data Quantity and Quality

The two main geotechnical investigations carried out at the site to date (Refs 1 & 2) have comprised test bores, cone penetration tests, test pits and laboratory tests. Tables 2 and 3 summarise the tests carried out by Soil & Rock Engineering (Refs 1 & 2) and comments on the quantity and quality of data available. It is noted that the available copy of Ref 2 did not include Appendices B through F, and Appendix A may also be incomplete.

Table 2 - Geotechnical Field Data Quantity and Quality

Test Type	Total Number of Tests	Range of Termination Depths (m)	Comment
Borehole	25	20.7– 81.25	Fair to good quality data on stratification plus the benefit of disturbed and undisturbed soil samples
CPT [(1)]	135	5.7 – 60.0	Very good quality data on stratification, including the strength, thickness and layering of soils to some depth.
Test Pit	50	1.0 – 3.5	Provides visual confirmation of soil conditions and samples within the upper few metres of soil.

Notes: 1. CPT = cone penetration test (including piezocone tests where undertaken)

The overall total of 160 ‘deep’ tests (excluding the test pits) carried out within Lot 1001 represents an average test density of one per 29,860 m^2 (equivalent to a 173 m by 173 m grid). The tests, however were more concentrated in the northern parts of the site, and if this area only is considered, the density of ‘deep’ tests increases to one per 20,910 m^2 (145 m by 145 m grid). This number is an overall average as the Stage 2 investigation was concentrated in the area of the proposed steel mill and had a higher test density than the Stage 1 investigation.

Despite the large number of tests, the test density is relatively sparse due to the size of the site. While there is sufficient data to provide a preliminary assessment of the subsurface conditions, there remains the likelihood of variations in subsurface conditions between test locations.

Table 3 - In-situ Tests

Test Type	Total Number of Tests	Range of Results	Comment
SPT	29	0.5 to >50	Commonly used test, but relatively crude and can be inaccurate.
Shear Vane	142	Peak: 6-31 kPa Residual: 2-8 kPa	Provides peak and residual undrained shear strength of clays.
CPTu Dissipation (2)	36	2.1 - 6800 m^2/yr	Useful in-situ data on consolidation rates, however there are a wide range of results.
SCPT shear wave velocity	3	V_s profiles not provided	Results converted to G_{max} and G values
Pressuremeter Modulus	41	48 -11053 MPa	All except one test carried out in rock strata; provides estimate of drained Young's modulus.

Table 4 – Laboratory Testing Data Quantity and Quality

Test Type	Total Number of Tests	Result Range	Comment
Grading to 75μm	36	% passing 75μm: 3 - 98 (fines)	Wide range of soils tested, from silty sand to clayey silt, with resulting wide range of fines contents.
Grading plus hydrometer	21	% passing 2μm: 7 - 71(clay fraction)	Range of clayey soils tested, with resulting wide range of clay contents.
Atterberg Limits	4	LL: 18 - 78% PI: 1 - 47%	Results indicate generally low to medium plasticity with some high plasticity results.
Oedometer Consolidation	5	m_v: 0.04-2.05 m^2/MN C_v: 0.30 - 3.0 m^2/yr	Higher m_v and lower c_v values are generally related to the main compressible soils.
CBR	2	CBR: 8%	Tests undertaken on clayey sand / sandy clay only. The CBR of imported fill will be more relevant for pavement design.
Acid Sulphate	177	S_{cr}: 0.002-0.670%	Alluvial soils to 3-4 m depth in the southern zone are potential acid sulphate soils and if exposed by excavation or dewatering, will require a management plan.

The laboratory testing program was reasonably thorough for the size of the site and proposed development, and was concentrated in the area of the proposed steel mill in the north-western part of the site. The quality and quality of laboratory test data is enhanced by the large amount of in situ test data (CPT, SPT).

4.2.2 Sub-Surface Conditions

The most pertinent features of the sub-surface conditions encountered by the investigations to date are:

- The site is underlain by alluvial and fluvial sediments of Quaternary age, associated with the meandering river valley of the Hunter River.
- The soil profile comprises mixtures of sand, silt, clay and gravel.
- The upper strata comprise compressible soils, including very soft to soft clayey soils (silty clay, sandy clay, clayey silt), and very loose to loose sands (silty sand, clayey sand).
- The upper compressible soils are underlain by medium dense to dense sand and very stiff sandy clay / silty clay, then bedrock.

A geotechnical model of the site and the engineering properties of the soil and rock strata are discussed in more detail in Section 5.

Groundwater was encountered at levels in the range 0.0 AHD to 1.0 AHD, and was typically around 0.5 AHD.

Some of the water levels were recorded at levels below AHD and hence probably do not represent a stabilised level. Measurements are typically made upon the completion of testing and in low permeability soils there is insufficient time for water to enter the borehole, CPT hole or test pit prior to backfilling/collapse. In order to obtain accurate water levels, standpipes or piezometers installed in boreholes are required, monitored once levels have had sufficient time to stabilise.

4.3 Relevant Data from Nearby Sites

4.3.1 Tomago Sandbeds

DP has previously carried out numerous studies of the Tomago Sandbeds aquifer, which lies to the north of the site. This work, undertaken between 1983 and 2001, included assessment of hydro-geological characteristics, groundwater chemistry and the production of groundwater contours (Refs 4 & 5). The Tomago Sandbeds is a large unconfined aquifer harvested by the Hunter Water Corporation for town water, via a network of pumping stations. The sandbeds is a Pleistocene inner barrier formation which extends into the northern fringe of the subject site.

The general groundwater flow direction is from the sandbeds towards Fullerton Cove and the Hunter River (i.e. generally south, but with local variations). The horizontal rate of flow is typically in the range 6 m/yr to 40 m/yr through the sand strata (Ref 5).

The subject site is down-gradient of the sandbeds and therefore the site development would have no adverse effect on this important water supply aquifer. The hydro-geological characteristics of the southern fringe of the sandbeds are however of relevance to the site.

4.3.2 Tomago Aluminium

The Tomago Aluminium smelter is situated to the north-west of the site. Previous studies by DP have included the original geotechnical investigations for the development in 1980, and a hydro-geological assessment of the groundwater regime in 1990 (Ref 6) which included a statistical analysis of maximum likely water levels during extreme wet periods.

The soil profile at the smelter site comprises sand, however the site includes a ridge or hill in the buried bedrock profile that results in a ‘mound’ in the groundwater levels and locally affects the flow rate and direction of groundwater movement.

The subject site is also within the buffer zone of the smelter and potentially subject to atmospheric fallout of contaminants associated with the smelter. Further discussion of this is provided in the Preliminary Contamination Assessment report (Ref 10).

4.3.3 Hunter River North Arm

DP has carried out preliminary investigations within the North Arm of the Hunter River for the NSW Premier’s Department (Ref 7). The main purpose was to consider the amount of sand which might be available from dredging the river. The previously proposed steel mill development would have required large volumes of fill material to be placed over a relatively short period, hence dredging was proposed.

The bores drilled in the river adjacent to the subject site indicated that there are substantial sand layers present, but they are ‘patchy’ and inter-bedded with silt and clay layers. This means that dredging would produce significant fine material (silt and clay) that would have to be carefully managed using sedimentation ponds and possibly treated for acid sulphate conditions (see Section 5.11). The overall sand content from all bores drilled in the North Arm was about 45%. This result, however, would probably vary depending on the number, location and depth of test bores.

It is expected that the proposed industrial development will require large quantities of fill material to raise site levels. In the event that dredging of the North Arm of the river is considered, further investigation would be required in the river to better define the extent and quality of the sand, and to address regulatory requirements for dredging approval.

4.3.4 WesTrac Site Preload Trial

The WesTrac Stage 1 development (also owned by Northbank Enterprise Hub Pty Ltd) lies to the east of the subject site and is likely to have similar soils in terms of engineering properties. The main difference is that the compressible silt and clay strata at the Westrac Site were thinner than at the subject site.

A preload trial pad was constructed at the WesTrac site at the southern boundary of Stage 1, in a location where the compressible silts and clays were approximately 5 m thick, being the thickest identified from tests. The trial pad measured 40 m by 40 m at the base and applied a ground pressure of approximately 90 kPa. The pad was instrumented so that the settlement and ground behaviour could be monitored. Figure 4 shows the settlement profile recorded from the nine settlement monitoring plates (SMP).

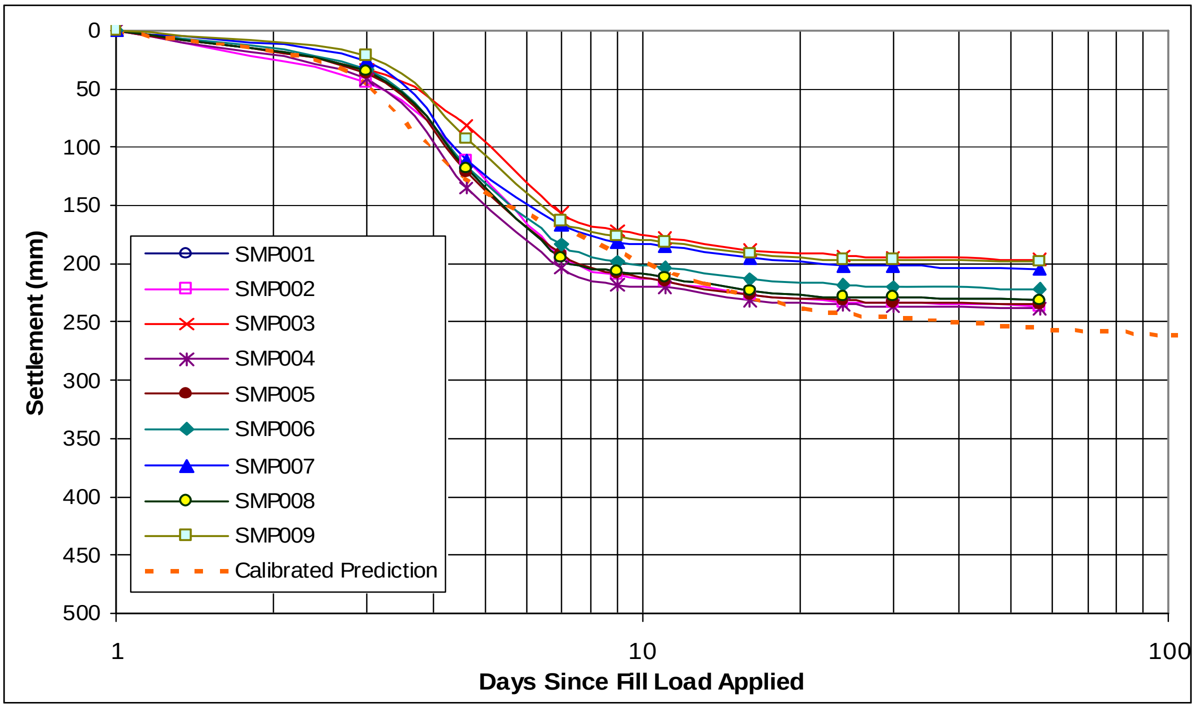

Figure 4 - Results of Preload Trial at WesTrac Stage 1

The trial pad settled approximately 200 mm to 240 mm during the monitoring period. A significant finding of the trial pad was that the rate of consolidation was faster than was expected on the basis of laboratory consolidation tests.

5. Discussion

5.1 Geotechnical Model of the Site

The available data indicates that the soil profiles at the site can be split into two broad categories:

North-Western Zone

The North-western Zone comprises a strip of land that adjoins Tomago Road and is around 200 m wide. This zone generally coincides with the southern fringe of the Pleistocene barrier dune formation (mapped as Qpbd and Qpb on Drawing 1). The subsurface conditions may be generalised as shown in Table 5.

Table 5: General Subsurface Conditions - North-Western Zone

Depth (m)		Description	Unit
From	To		
Ground level	0.15 / 0.25	TOPSOIL – Silty sand / sandy silt, organic matter	1
0.15 / 0.25	4.2 / 5.3	SAND – very loose to loose, silty in part	3a
4.2 / 5.3	9.2 / 14.4	SAND – dense to very dense	3b
9.2 / 14.4	10.7 / 15.7	CLAY – very stiff to hard	4
10.7 / 15.7	-	BEDROCK – extremely low to low strength interbedded Siltstone / Sandstone	5

South-Eastern Zone
The South-eastern Zone comprises the remainder of the site, and the generalised subsurface conditions are shown in Table 6.

Table 6: General Subsurface Conditions - Southern Zone

Depth (m)		Description	Unit
From	To		
Ground level	0.15 / 0.25	TOPSOIL – Silty sand / sandy silt, organic matter	1
0.15 / 0.25	0.5 / 1.5	SILTY CLAY - firm, medium to high plasticity	2a
0.5 / 1.5	3.5 / 9.6	SILTY CLAY & CLAYEY SAND - sensitive fine grained soil, very soft to soft, low plasticity	2b
3.5 / 9.6	7.1 / 13.5	SAND – very loose to loose, silty in part	3a
7.1 / 13.5	10.8 / 25.2	SAND – dense to very dense	3b
10.8 / 25.2	29.1 / 38.7	CLAY – very stiff to hard	4
29.1 / 38.7	-	BEDROCK – very low strength Siltstone / Sandstone	5

It is noted that further south, adjacent to the Hunter River the depth of very soft to soft silty clay (Unit 2b) was found to exceed 60 m. The bedrock in this area was encountered from 76.4 m depth.

The design soil parameters were assessed for the main soil layers based on the data review, and are shown in Table 7. These are ‘typical’ rather than lower or upper bound values and hence could vary either way.

Table 7 - Design Parameters for Settlement Analysis

Unit	Description	Average q_c (MPa)	m_v (m^2/MN)	c_v (m^2/yr)	$C_{\alpha\varepsilon}$
1	Topsoil - silty sand	1.5	0.15	20	0.002
2a	Silty Clay - firm	0.5	0.40	5	0.015
2b	Silty Clay / Clayey Sand	0.2	1.40	10	0.020
3a	Sand - very loose to loose	5.0	0.02	1000	-
3b	Sand - dense to very dense	20.0	0.005	1000	-
4	Clay - very stiff to hard	5.0	0.04	5	0.001

Notes: 1. q_c CPT cone tip resistance
2. m_v coefficient of volume compressibility
3. c_v coefficient of consolidation
4. $C_{\alpha\varepsilon}$ coefficient of secondary consolidation (creep strain per log cycle)

The time for primary consolidation to be substantially complete is usually taken as 90% of pore water pressure dissipation. Secondary consolidation (also called creep) continues beyond this indefinitely at a diminishing rate with time. Figure 5 illustrates the relationship between the thickness of the soft clay layer (Unit 2b), the coefficient of consolidation (c_v) and the time for 90% primary consolidation, assuming two-way (top and bottom) drainage.

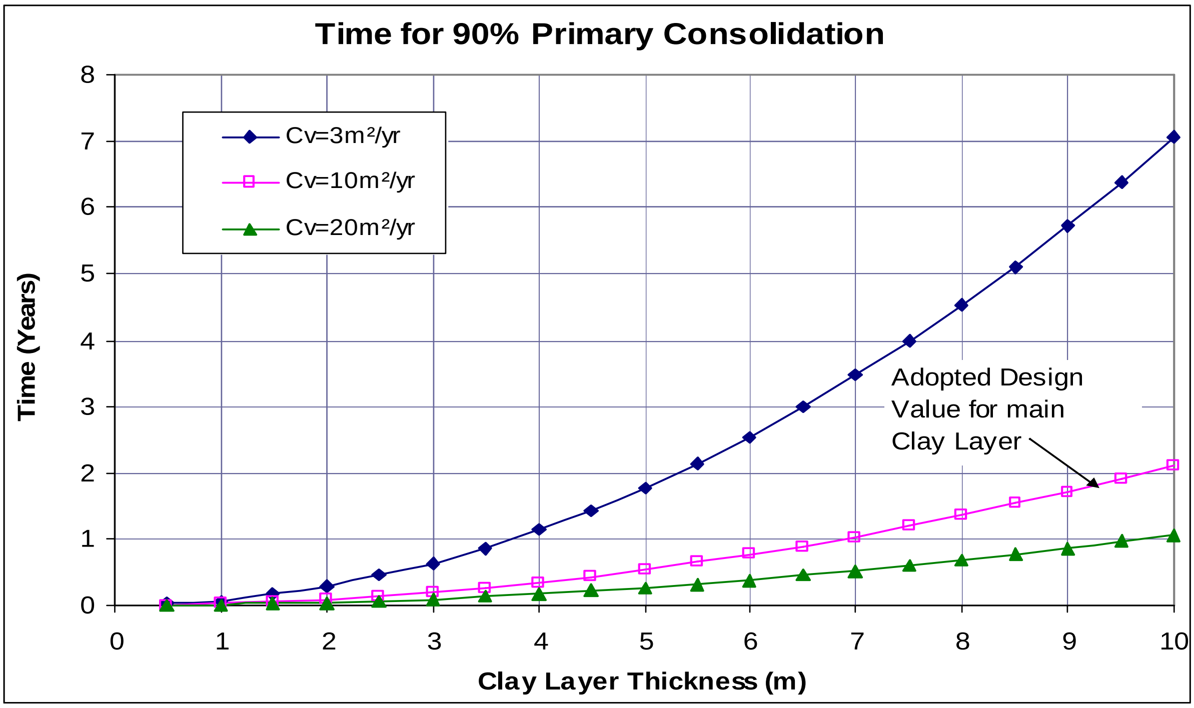

Figure 5 – Relationship between Clay Thickness, c_v and Primary Consolidation

The time for consolidation to occur increases with greater clay thickness and lower c_v values. Given the range in clay thickness recorded at the site, and the large range of possible c_v values, it is evident that settlement times will range from a few months to several years.

5.2 Implications of Geotechnical Conditions

The main implications of the geotechnical conditions at the site arise due to the upper weak alluvial strata. Units 2a and 2b are compressible soils that will consolidate under the loads imposed by the development. The contours of soft/loose soil thickness (isopachs), as presented in Ref 1, have been overlaid on the proposed development as shown on Drawing 2. The main geotechnical considerations are as follows:

- The placement of about 2 m of fill to raise site levels will impose a widely distributed pressure of about 40 kPa on the foundation soils. This will cause time-dependent settlement due to consolidation of the clays.
- The time-dependent settlements will be mainly associated with Units 2a and 2b. Units 3a and 3b consist of sandy strata and will consolidate rapidly (i.e. mostly during construction).
- Structures founded in the fill will impose additional pressures where shallow footings are used, such as raft slabs, strip footings and pad footings. This will cause additional settlement beneath the buildings, dependent on the foundation pressure imposed by the structure. Structure-induced settlement will not occur where buildings are supported on piles, however this introduces different design issues such as pile downdrag and effects on services connections (see further comment in Section 5.10).
- There will be a time lag between fill-induced settlement and structure-induced settlement, such that the combined settlement will be influenced by the delay between completing earthworks and construction of buildings.
- Differential settlements will occur, exacerbated by differing heights of fill required and variation in the thickness of Units 2a, 2b and 3a. Predicting and managing differential settlement presents the major challenge of developing this site.

5.3 Hydrogeology and Potential Effects on Wetlands

The existing hydrogeological regime comprises water-logged low-lying land with groundwater levels generally in the range 0 to 1 AHD, rising to around 2 AHD within the more elevated land adjacent to Tomago Road. A review of the groundwater contour plans produced in the 1990s by DP for the Tomago Sandbeds (Ref 4) and Tomago Aluminium (Ref 6), indicates that the regional groundwater flow would be from the north and north-east of the site to the Hunter River along the south-west of the site.

The above plans indicate that the groundwater levels on the northern side of Tomago Road are typically around 3 AHD. This implies a hydraulic gradient of approximately 1:600 and a flow rate in the order of 20 m/yr to 30 m/yr (based on a sand permeability of 1.5 to 2.0 x 10^{-4} m/s and an effective porosity of 0.35).

Raising the site ground levels will have some impact on water levels and flow rates. The exact nature of the impact will depend on the type of fill used and staging of the fill. In broad terms the higher ground levels would be expected to initially lead to slightly higher water levels within the filled area, particularly if sand is used. This in turn would result in slightly steeper gradients and hence higher flows along the southern margins on the filled area. Given that the water levels are presently close to the surface, there may be an increase in the incidence of water-logging immediately downstream of the filled area. The use of low-permeability fill and/or the subsequent construction of pavements and buildings will lead to an increase in runoff and a reduction of groundwater levels beneath the filled area. It is expected these effects would be relatively localised and the overall impact on the wetlands is likely to be minor.

There could also be seepage from the toe of the fill embankments and the potential effects on batter stability should be considered in design.

Further assessment, and possibly some groundwater flow modelling, should be carried out during detailed civil design of the subdivision.

5.4 Preliminary Settlement Estimates - Untreated Site

Preliminary settlement estimates have been carried out based on one-dimensional consolidation theory, using an in-house method where compressibility is estimated from CPT cone resistance. The adopted soil parameters for the compressible strata were as shown in Table 7. The load applied comprised a 2 m fill platform (40 kPa uniform load) and the results are shown in Table 8 below. A design period of 50 years was adopted for the analysis.

Table 8 - Estimated Settlement due to 2 m of Fill over 50 Years

Thickness of Unit 2 Clays	Primary Consolidation (mm)	Secondary Consolidation (Creep) (mm)	Total Settlement after 50 Years (mm)
2	140	130	290
4	250	175	460
6	360	205	615
8	470	215	755

Notes: 1. Pressure based on a unit weight of 20 kN/m^3 for compacted Fill.

The estimated time-settlement curves are shown plotted in Figure 6 below.

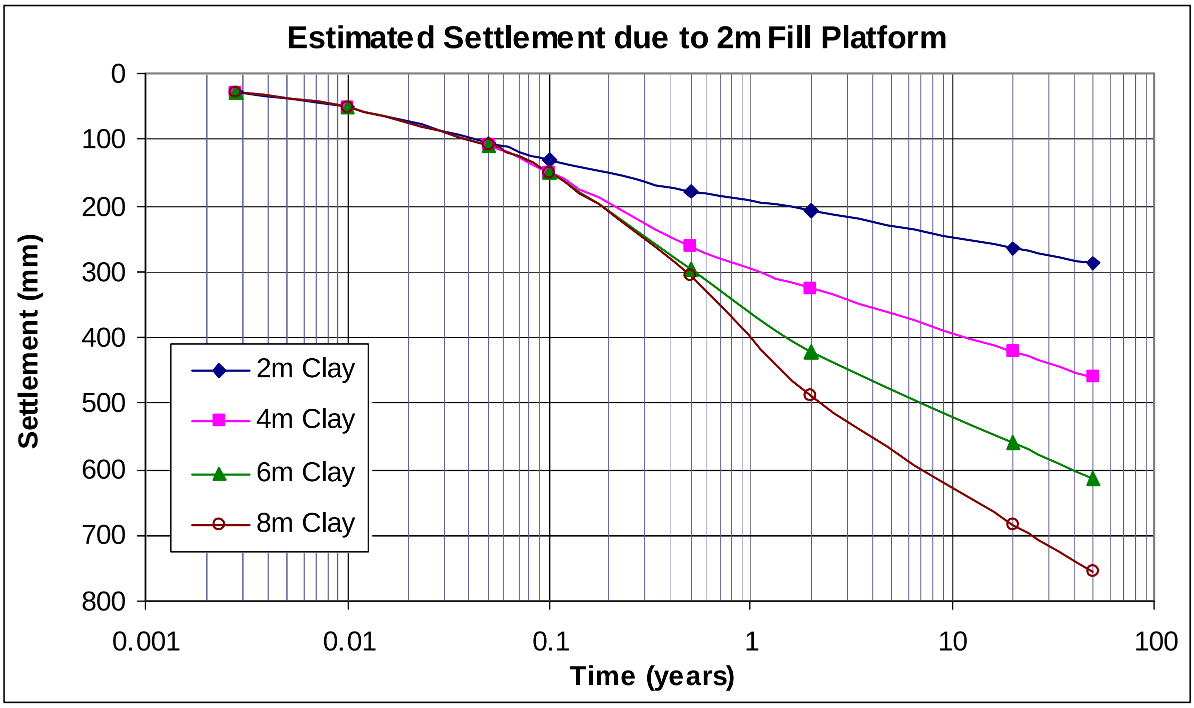

Figure 6 - Estimated Time-Settlement Curves for 40 kPa Fill Load

The loads due to structures founded on shallow footings will subsequently add to settlement. The time lag between completing the fill platform and construction of a building will influence the magnitude of settlement experienced by the building. This concept is illustrated in Figure 7 below for a structure built three months after completing the fill platform. The assumed slab load is 15 kPa and the adopted Unit 2 clay thickness is 4 m in this example.

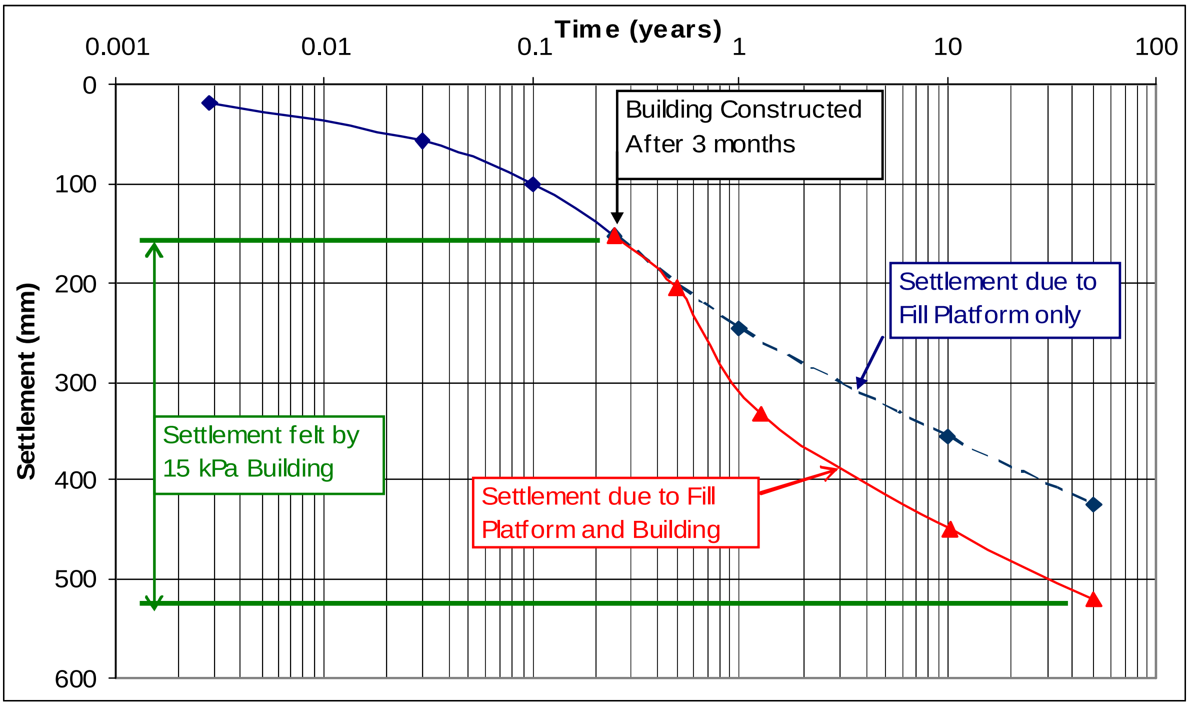

Figure 7 –Settlement if Building Completed 3 Months after Fill Placed

It is recommended that prior to final design an instrumented trial pad would be beneficial to gain field data on the rate and magnitude of settlement, as consolidation parameters can vary widely.

The foregoing analyses are based on no ground treatment and represent the starting point for considering the need for ground treatment. It is likely that the estimated settlements cannot be tolerated and some form of ground treatment (or use of piles) will be required over the major southern portion of the site. The same analytical process can be applied to the post-treatment site to estimate long-term differential settlements for specific structures or parts of the site for design purposes.

5.5 Ground Treatment Options

A number of possible ground treatment options have been considered for development of the site. These are listed in Table 9 below, ranked in order of *geotechnical* preference for the site and considering the proposed staged development. The rank may change based on other considerations such as cost, schedule, environmental and/or regulatory requirements.

Table 9 – Ground Treatment Options

Rank	Technique	Principle	Comment
1	Preload	Application of a load to the foundation which is usually greater to the final loads after construction. The fill over and above the final fill level is called	The load is usually applied in the form of additional fill (surcharge) which is later removed. This method is one of the simplest ground treatment methods but requires sufficient time and excess fill

		'surcharge'.	material.
2	Wick Drains with Preload	Installation of vertical drains to accelerate consolidation by providing a shorter drainage path for the expulsion of water.	Requires preload as above but can significantly shorten the construction time, particularly where the clay is thick. May only be required if developing the thick clay areas.
3	Dynamic Replacement	Formation of granular columns by dynamically forcing granular material through the clay into the lower sand stratum.	The granular material can be sand or imported gravel. The reduction in settlement is not as effective as preload. The process is only suited to soft clay depths of $\leq$ 6 m, hence would not be practical for the eastern portion of the site.
4	Stone Columns	Columns of compacted stone installed by vibro-replacement through the clay into the lower sand stratum.	The granular material is usually imported gravel or ballast. The reduction in settlement is not as effective as preload.
5	Deep Soil Mixing	Mixing of dry unslaked lime and/or cement with the soft clay to form a column of treated soil.	Reduces the plasticity and compressibility of the soil. Highly specialised equipment is required to effectively mix the lime and cement with the soil.
6	Excavate and Replace	Removal of some or all compressible soils and replacement with granular soils.	Requires treatment of acid sulphate soils, disposal of treated soils and importation of replacement granular material. Only practical or economical for shallow removal (say < 1m such as near the sand/clay transition zone).
7	Compression Grouting	Grout is pumped into the soil at regular spacing to displace and therefore consolidate the clays	As with dynamic compaction this process can be difficult to control and is more suited to sands.
8	Dynamic Compaction	Dynamic compression of the soils by dropping large weights on a grid pattern.	Can improve clays by shearing (and subsequent consolidation) but is difficult to control; this technique is more suited to densifying loose sands.
9	Vacuum Consolidation	Use of atmospheric pressure by applying suction to clay soils via vacuum tubes installed on a grid.	This method is more suited to deep deposits of clay and is unlikely to be feasible or economic for this site.

It should be noted that the above techniques do not eliminate post-construction settlement; they merely aim to reduce settlement to manageable/tolerable levels. Excavate and replace has the greatest effect on reducing settlement as the most compressible layers are removed.

It is considered that preload and/or preload with wick drains are likely to be the most effective ground treatment methods for the site. This is predicated on there being enough time and fill material to surcharge the site. Where there can be a substantial delay between filling and building on the site, the use of wick drains would not be required. If surcharge fill (additional fill to the finished level) is not placed, the post-construction settlements can still be significant, even if there is a substantial delay between filling and construction, but the magnitude will depend on the structural loads.

5.6 Preload Design

If preload is adopted as the ground improvement method, the following guidance is provided for preliminary design purposes. For effective reduction of post-construction settlement the preload should include a surcharge above final earthworks level. The post-construction settlement will take the form of re-compression and creep, with the latter reduced by the effects of over-consolidation. Upon reaching the target degree of consolidation or settlement (determined by monitoring), the surcharge would be removed.

Based on the assessment to date, and experience at the nearby WesTrac site, it is expected that the surcharge heights will need to be at least 2 m and possibly up to 3 m (i.e. in addition to the permanent fill). Following the suggested ground treatment the total long-term post-construction settlement is estimated to be in the order of 50 mm to 100 mm where the Unit 2a clays are the thickest. It is noted, however, that the ground treatment can be designed to target the required residual settlement criteria.

The differential settlements could approach the total settlements where structures extend over varying clay thickness. The preload surcharge height or target degree of consolidation could be reduced if and where larger settlements are acceptable, such as hardstand areas.

The appropriate permanent batter slopes at the edges of the fill platform will depend on the type of fill material used, but would typically be about 2H:1V. The batter slope of the temporary surcharge layer could possibly be steeper, however there may need to be a ‘stability berm’ (i.e. a set-back) between the edge of permanent fill and toe of the surcharge batter slope, in order to prevent slope failure.

Further comments on earthworks procedures and preload monitoring are presented in Sections 5.8 and 5.9 respectively.

5.7 Preload with Wick Drains

Wick drains could be used in conjunction with preload to increase the rate of settlement and thereby speed up construction time. While wick drains were not required at the WesTrac site, they could be required at this site if the parts of the site with thick clays (i.e. > 5m thick) are developed and/or the soils are less permeable than at WesTrac. Modern practice is to use prefabricated vertical drains (PVD) comprising a corrugated plastic strip drain covered in a woven geotextile. PVDs are typically 100 mm wide and 4 mm thick.

The rate of consolidation is controlled by both the vertical and radial (horizontal) permeability of the clay layer under consolidation, with the latter dominating. Consequently the main parameter in wick drain design is the spacing of the drains, with factors such as clay layer thickness, geometric layout and drain flow capacity being contributory. The most efficient layout is an equilateral triangle.

The following design values for the coefficients of consolidation were adopted for analysis:

- $c_v = 10\ m^2/yr$
- $c_h = 30\ m^2/yr$

The ratio $c_h/c_v = 3$ was adopted based on Soil and Rock Engineering's recommendation in Ref 2, which is understood to have originated from pumping test results at the site. The target degree of consolidation for preload should be 90%.

The variation in consolidation times for a range of clay thicknesses and wick drain spacing (triangular pattern) is shown in Figure 8 below. For example, wick drains installed at 2 m spacing would be expected to achieve 90% consolidation after 6 to 16 weeks, depending on clay thickness.

The requirement for wick drains and the optimal spacing will depend on timing / scheduling requirements for particular parts of the development site. It is noted however that wick drains should be installed *prior* to any substantial filling of the site.

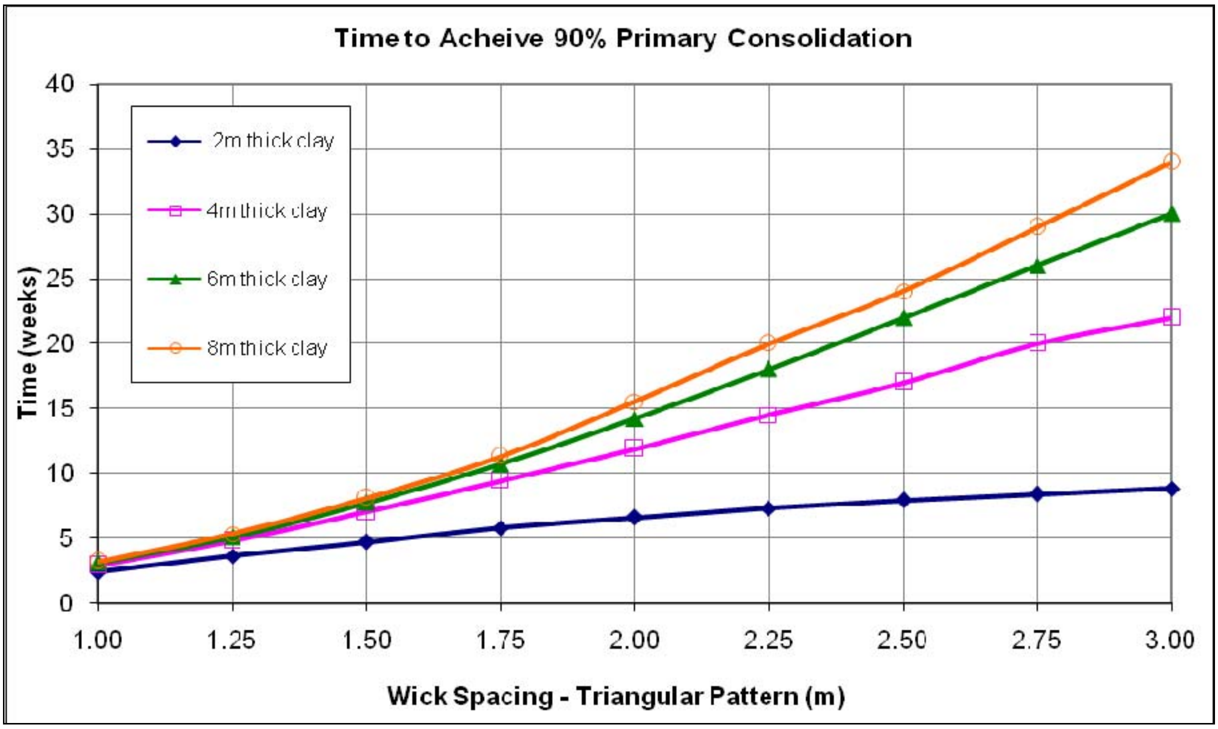

Figure 8 - Consolidation Time vs Wick Drain Spacing

5.8 Earthworks

For preliminary planning purposes the following typical process for earthworks, preload and surcharge could be considered:

- Strip the site leaving grass and topsoil to act as reinforcement. Where there is little or no vegetation, it may be necessary to use a geofabric to provide a separation and filtration function.
- Place approximately 0.5 m to 0.8 m thick layer of granular fill over the low-lying part of the site, spreading out using tracked machinery only to form a construction platform (preliminary earthworks level). The actual thickness will depend on the type of fill. It is noted that if material coarser than sand is used, it may impair the ability to punch wick drains through to the underlying soft clays.
- If preloading, install geotechnical instrumentation and (if required) wick drains (see Sections 5.7 and 5.9 for indicative information).
- Fill the site to design finished level plus allowance for settlement, preferably using granular material, compacted to an appropriate specification for final use (i.e. construction of buildings pavements).
- The batter slopes at the edges of the fill platform should be formed at 2H:1V after compaction and trimming.
- Place surcharge material (preferably sand) in accordance with the detailed preload plan yet to be developed (see Section 6). The surcharge only requires nominal compaction to maintain form and trafficability. A surcharge height of 2.5 m should be considered for preliminary planning. By the time the surcharge is placed in a given area there may be sufficient strength gain in the clays to avoid a wide stability berm.
- It is anticipated that the surcharge could be placed in successive areas to minimise the amount of excess material needed (preload sequencing), however the number of ‘moves’ will depend on the rate of developing the subdivision and the material available.
- Remove surcharge when target settlements are achieved and commence construction.

It should be noted that the amount of imported fill required to maintain the design finished surface level will need to take account of the expected settlement during and following preloading.

A review of available suitable fill materials should be carried out to assess geotechnical properties and implications in relation to ground treatment measures, foundations for structures, effect on groundwater flows and drainage.

5.9 Preload Instrumentation and Monitoring

5.9.1 Geotechnical Instruments

It is important that adequate geotechnical monitoring of the preload be undertaken. This should take the form of instrumentation installed in the ground immediately following preliminary earthworks and wick drain installation (where required), and prior to significant filling of the site.

The details of geotechnical instrumentation and monitoring would be determined at the detailed design stage to suit the proposed ground treatment programme. The components of the preload instrumentation are likely to comprise some or all of the following:

- ***Settlement Monitoring Plates (SMP):*** An SMP comprises a steel base plate, buried in the ground (at preliminary earthworks level), with vertical steel pipe attached to the plate. The steel pipe should be protected by PVC casing. A level is taken by survey on the pipe prior to placement of preload. As the fill height increases around the pipe, extension pipes are added, enabling the settlement of the foundation to be measured by survey.
- ***Piezometers:*** Vibrating wire piezometers should be installed within the upper soft clay strata, in order to monitor the excess pore water pressures generated by placement of the preload.
- ***Inclinometers:*** Lateral deflection and/or rotation of the preload batter slopes should be monitored by inclinometers. These provide a warning if embankment instability is imminent, and allow earthworks to be adjusted accordingly.
- ***Earth Pressure Cells:*** Earth pressure cells could be installed at preliminary earthworks level to confirm the pressure applied by the preload fill. This would allow adjustment of the preload height (if necessary) to achieve the required applied pressure.
- ***Hydrostatic Profile Gauges:*** Hydrostatic Profile Gauges (HPG) provide additional information on settlement. They comprise a tube and pulley box system which is installed horizontally in a trench under the area to be loaded. The access points can be located at the edge of the fill platform and therefore are out of the way.

SMPs generally provide the most valuable data on the magnitude and rate of settlement, and are the least expensive instrument. Although not as accurate as SMPs, HPGs are less susceptible to disturbance during the placement and moving of preload material. They can also be used to monitor settlement during and after construction of buildings and pavements, whereas SMPs are inevitably destroyed during removal of the surcharge.

5.9.2 Geotechnical Monitoring and Verification

In the case of preload, all instruments should be read upon installation to obtain baseline readings. Upon placement of permanent fill and the surcharge, the instruments should initially be read twice weekly, then weekly for a period which will depend on the rate of consolidation. In areas where consolidation is expected to occur more slowly, the reading interval may be increased.

If alternative ground treatment methods are used, a different form of verification will be required to confirm the performance of the treatment. The type of testing will depend on the type of ground treatment being used.

5.10 Piles

Heavily loaded structures or concentrated point loads may need to be supported on piles if the anticipated post-construction settlements cannot be accommodated by the structure. It is anticipated that this will be the responsibility of the purchaser of specific allotments, and the design may depend on how long after site filling the construction is proposed. The target founding stratum for piles would be the Unit 3b dense sand.

It is noted that piles installed through Unit 2 clays could attract negative skin friction ('downdrag') as these soils consolidate over time (i.e. after pile installation). The estimated additional load induced by downdrag should be accounted of in design of piles. Some methods that could be considered to reduce or eliminate downdrag include the use of low-friction coatings or sleeving, delaying construction, or steel screw piles (due to small shaft diameter).

Piled structures will experience minimal settlement, hence the differential settlement between the building and connecting services and peripheral pavements will be exaggerated. This will need to be accounted for in the design and detailing of services, pavements and drainage.

The amount of downdrag on piles and differential settlement across structures and services will reduce as the delay between filling and construction increases.

Further geotechnical advice should be sought on pile design at the appropriate design stages. General guidance on piles and typical capacities could be developed to assist intending purchaser of allotments. Once development details are known for a specific allotment, the proponent should obtain specific design advice for piled structures.

5.11 Acid Sulphate Soils

The upper 3 m to 4 m of soil in the south-eastern zone (south and east of the zone mapped as Qpbd or Qpb on Drawing 1) includes potential acid sulphate soils. These soils could generate acid if exposed to air, either be excavation or dewatering activities.

The proposed development involves raising the ground level by filling and hence is unlikely to disturb the potential acid sulphate soils within the site. Nevertheless, the details of future site development and usage are not known at this stage, and certain activities could oxidise the acid sulphate soils. These include:

- any basement excavations that extend below the fill into the natural soils;
- trenching for installation of deep services, if penetrating the natural soils;
- dewatering for any purpose, including the above activities
- pumping for water abstraction.

If dredging of the river is proposed the sand and silts brought to the surface may also require treatment for acid-generating conditions. Previous experience with dredging from the river further downstream indicates that the saline water can buffer acid generation and reduce or obviate the need for treatment. Nevertheless the requirement for treatment of dredged sediments cannot be ruled out and will need to be further investigated if dredging is proposed.

In the event that any of the above (or similar) activities are proposed, the works would need to be carried out in accordance with an Acid Sulphate Soil Management Plan developed for the activity in question. Usually, these activities are readily managed through appropriate monitoring and liming of soils/water during construction, as set out in the Management Plan.

5.12 Seismicity and Liquefaction

The new earthquake code, released in October 2007 (AS1170.4-2007, Ref 8), has a rating system for soil profiles that differs from its predecessor. The design of earthworks and structures should take into account potential seismic loading.

Based on the review of the existing data it is assessed that the appropriate 'site sub-soil class' for the proposed development site, as defined in Section 4.1 of Ref 8, is **Class D_e** (deep or soft soil site). The Hazard Factor Z is 0.11, corresponding to the acceleration coefficient for the Newcastle area.

The subsurface soils include zones of loose sands / silty sands, and these soils can be susceptible to liquefaction during a significant seismic event.

A preliminary assessment of potential liquefaction has been carried out for the site conditions following filling. Selected CPT profiles were analysed using the method of Juang et al (Ref 9); these CPTs were tests carried out by Douglas Partners on behalf of Soil & Rock Engineering so the original CPT data files were available in-house.

The results indicate that the Unit 2b and 3a very loose to loose sands / silty sands, present to depths of around 4 m to 5 m could undergo liquefaction in a Magnitude 5.5 earthquake (approximately a 1 in 500 year event and similar to the 1989 Newcastle earthquake). The analysis indicated that the probability of liquefaction ranges from 80% to 90% for the site in its present condition.

The addition of fill increases overburden pressure and strengthens the soil thorough consolidation. The analysis for the site after placement of 2 m of fill material indicates that the probability of liquefaction reduces to 25% to 60%. This suggests that liquefaction would be marginally unlikely during a Magnitude 5.5 earthquake, following site development.

Further specific analysis should be carried during detailed geotechnical investigation of the site.

6. Further Investigation And Design

While there is sufficient geotechnical data for preliminary or concept design purposes, there will need to be further geotechnical investigation and design to proceed with detailed earthworks and ground treatment design.

The next stages of investigation and design, from a geotechnical / geo-environmental perspective, are set out in Table 10 below. The list of activities is not exhaustive but they provide general guidance on requirements for the next stages of the project. The items are listed in approximate chronological order, although some items could proceed concurrently.

Table 10 - Further Investigation and Design Tasks

Item	Task
1	Undertake additional geotechnical investigation to fill in the data gaps and target the proposed development area. This would comprise a combination of bores and CPTs and include revision of clay thickness contours and design parameters.
2	Undertake a Stage 2 Contamination Assessment in accordance with NSW DECCW guidelines. This would include sampling and chemical testing, targeting the areas of interest identified in the Preliminary Contamination Assessment (Ref 10).
3	Assess alternative ground improvement measures for each part of the site and recommend most suitable options This should include preload design, optimal wick drain spacing and typical requirements for other suitable ground improvement methods.
4	Undertake a geotechnical review of suitable fill materials and sources. Proposed materials should be classified in accordance with DECCW Waste Classification Guidelines as ENM or VENM material, or obtain specific DECCW exemption / approvals.
5	Assess potential effects of the development on the local groundwater regime, taking account of the type of fill material that forms the fill platform and staging of the filling works. If necessary, carry out groundwater flow modelling
6	If preload is adopted, refine the preload design, comprising heights, batter slopes and staging (sequencing) to minimise the amount of imported fill required, and prepare preload and earthworks plans.
7	If preload is adopted, prepare a plan of geotechnical instrumentation - numbers, locations and depths. For other ground treatment measures, prepare inspection and test plan (ITP) and verification procedures.
8	For proposed pavements and structures, determine the likely applied foundation pressures and tolerable total and differential settlements. This could include generic geotechnical pile designs for typical structural loads to assist prospective purchasers of allotments.
9	Prepare pavement thickness designs for internal roads, including material quality and compaction specifications.

The geotechnical investigation could be combined with the detailed contamination assessment (refer Ref 10) for a more efficient cost-effective programme of investigation.

7. References

1. Soil & Rock Engineering Pty Ltd, "Preliminary Geotechnical Investigation Proposed Steel Mill and Port Development, Tomago, New South Wales, Australia", Report dcb5264_1_01rep", 13 July 2001.
2. Soil & Rock Engineering Pty Ltd, "Stage 2 Geotechnical Investigation Proposed Steel Mill, Tomago, New South Wales, Australia", Report dcb5264_3_01rep", 28 December 2001.
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4. Douglas Partners Pty Ltd, "The 1994/95 Annual Review of Mineral Sands Mining at Tomago, NSW", Project 9000-22, 22 December 1995.
5. M J Thom & S R Jones, "Geotechnical and Geophysical Characteristics of an Unconfined Coastal Aquifer at Tomago, NSW, Australia", Proc International Conference on Environmental Management, Geo-Water and Engineering Aspects, Wollongong, February 1993, A A Balkema.
6. Douglas Partners Pty Ltd, "Report on Prediction of Maximum Water Levels at Tomago Aluminium, Tomago", Project 13918, 30 July 1990.
7. Douglas Partners Pty Ltd, "Report on Geotechnical Investigation, Hunter River North Arm, Newcastle, Project 31295, 29 June 2001.
8. AS1170.4-2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia", Standards Australia.
9. Lee D, Juang, C.H. and Ku, C.(2001), "Liquefaction Performance at the Site of a Partially Completed Ground Improvement Project During the 1999 Chi-Chi Earthquake in Taiwan", *Canadian Geotechnical Journal,* No 38, pp 1241-1245.
10. Douglas Partners Pty Ltd, "Report on Preliminary Contamination Assessment, Proposed Northbank Enterprise Park, Lot 1001 DP 1127780, 365 Tomago Road, Tomago, Project 49608-PCA, July 2010.

8. Limitations

Douglas Partners (DP) has prepared this report for this project at 365 Tomago Road, Tomago, in accordance with DP's proposal NCL100153 dated 31 March 2010. The work was carried out under DP Conditions of Engagement. This report is provided for the exclusive use of ADW Johnson Pty Ltd and Northbank Enterprise Hub Pty Ltd for the specific project and purpose as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party.

The results provided in the report are considered to be indicative of the sub-surface conditions on the site only to the depths investigated at the specific sampling and/or testing locations, and only at the time the work was carried out. DP's advice may be based on observations, measurements, tests or derived interpretations. The accuracy of the advice provided by DP in this report is limited by unobserved features and variations in ground conditions across the site in areas between test locations and beyond the site boundaries or by variations with time. The advice may be limited by restrictions in the sampling and testing which was able to be carried out, as well as by the amount of data that could be collected given the project and site constraints. Actual ground conditions and materials behaviour observed or inferred at the test locations may differ from those which may be encountered elsewhere on the site. Should variations in subsurface conditions be encountered, then additional advice should be sought from DP and, if required, amendments made.

This report must be read in conjunction with the attached "Notes Relating to This Report" and any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions from review by others of this report or test data, which are not otherwise supported by an expressed statement, interpretation, outcome or conclusion stated in this report. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

Douglas Partners Pty Ltd

Appendix A

About this Report

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

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

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

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

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

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

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

Information for Contractual Purposes

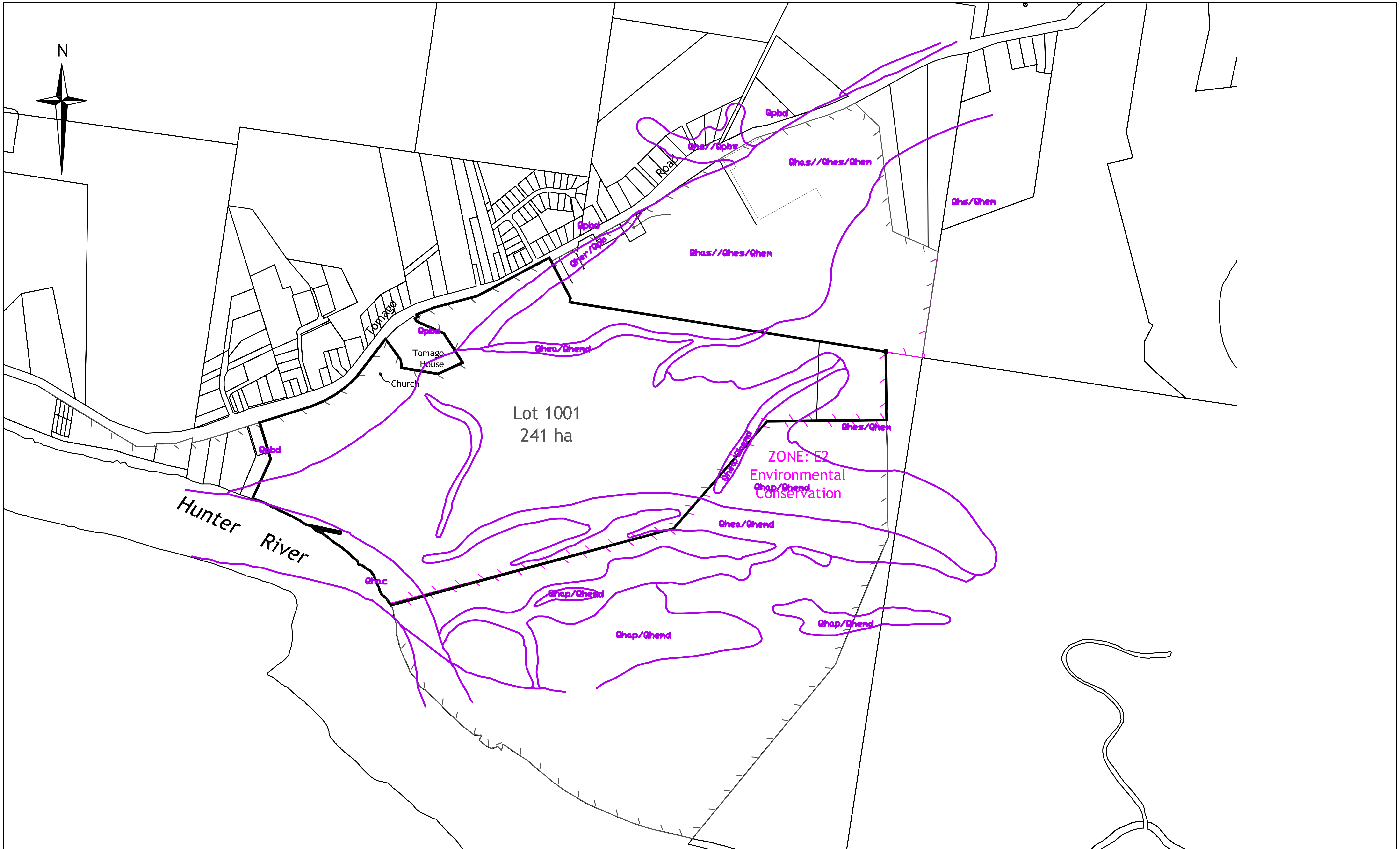
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix B

Drawing 1 – Mapped Quaternary Sediments
Drawing 2 – Proposed Site Development and Clay Isopachs



LEGEND

BOUNDARY

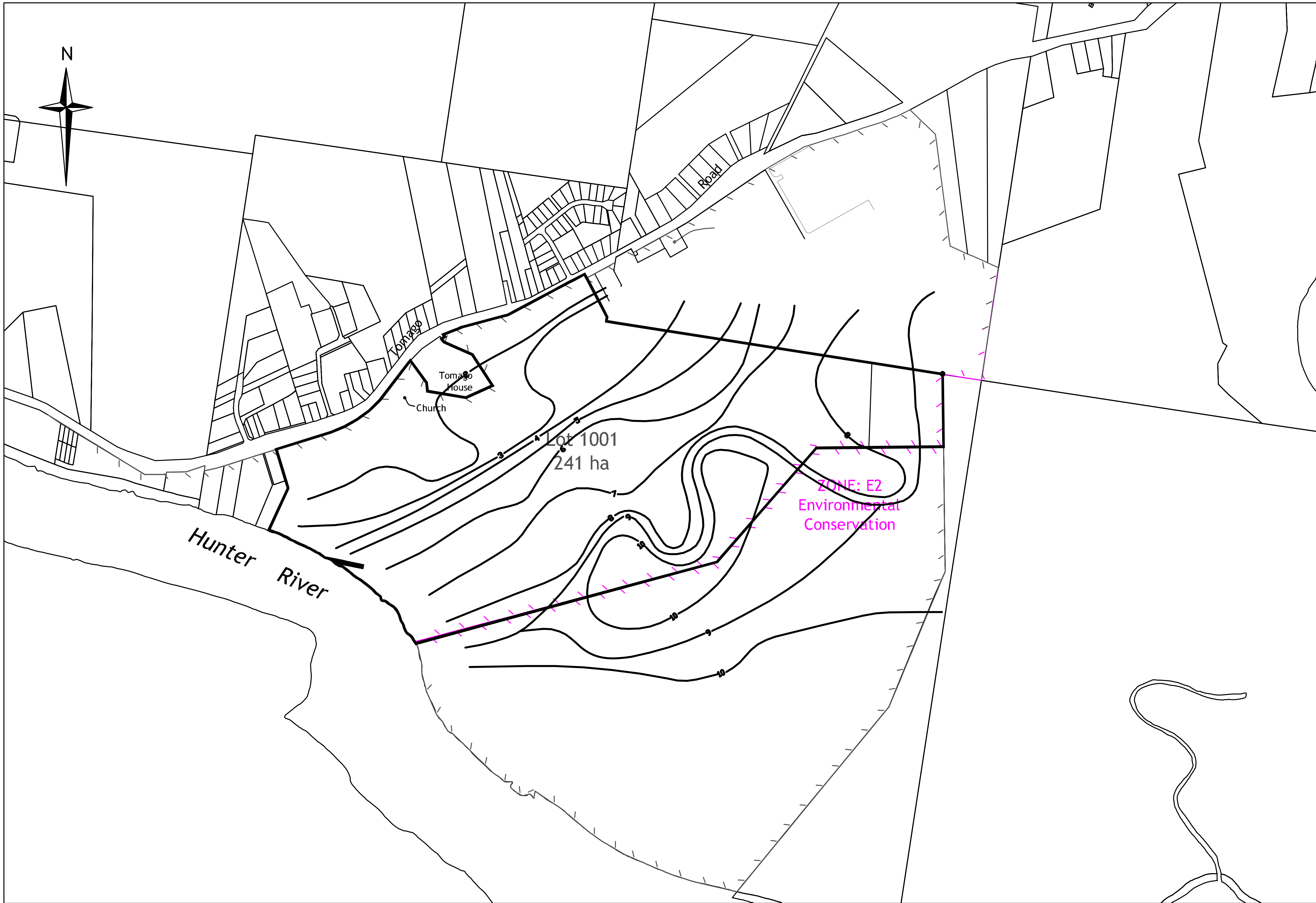
QUATERNARY SEDIMENTS (REFER TO TABLE 1 OF REPORT FOR EXPLANATION

0 250 500 750m

SCALE 1:15000 (A3 SHEET)

DRAWING ADAPTED FROM PLAN SUPPLIED BY CLIENT, REF 37672 VERSION A DATED 22/3/10

Douglas Partners Geotechnics Environment Groundwater Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong, Darwin	TITLE: SITE LOCATION PLAN & MAPPED QUATERNARY SEDIMENTS PROPOSED NORTHBANK ENTERPRISE PARK LOT 1001, DP1127780 TOMAGO ROAD, TOMAGO	CLIENT: ADW JOHNSON PTY LTD DRAWN BY: PLH SCALE: 1:15000 APPROVED BY:	REF:P:\49608\DRAWINGS\49608 DRAWING G1 PROJECT No: 49608 OFFICE: NEWCASTLE DATE: DRAWING No: G1



LEGEND

BOUNDARY

CONTOURS OF SOFT/LOOSE SEDIMENT THICKNESS (m), AS PRESENTED BY SOIL AND ROCK ENGINEERING

0 250 500 750m

SCALE 1:15000 (A3 SHEET)

DRAWING ADAPTED FROM PLAN SUPPLIED BY CLIENT, REF 37672 VERSION A DATED 22/3/10

Douglas Partners
Geotechnics Environment Groundwater

Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong, Darwin

TITLE: PROPOSED DEVELOPMENT & ISOPACHS OF SOFT/LOOSE SEDIMENTS PROPOSED NORTHBANK ENTERPRISE PARK LOT 1001, DP1127780 TOMAGO ROAD, TOMAGO	CLIENT: ADW JOHNSON PTY LTD	REF:P:\49608\DRAWINGS\49608 DRAWING G1	
	DRAWN BY: PLH SCALE: 1:15000	PROJECT No: 49608	OFFICE: NEWCASTLE
	APPROVED BY:	DATE:	DRAWING No: G2