REPORT TO LAURUSCO DEVELOPMENTS (PTY) LTD ON A PRELIMINARY GEOTECHNICAL INVESTIGATION OF 53HA FOR COMMERCIAL DEVELOPMENT, MTUBATUBA

1. TERMS OF REFERENCE

Drennan Maud & Partners supplied a revised work proposal and cost estimate to Mr Neels Brink of Metallon Properties on the 24th April 2008 to carry out a preliminary geotechnical investigation of 53 ha over two sites near Mtubatuba. This was accepted by email on the 28th April 2008. The preliminary study was to comprise widely spaced inspection pits and penetrometer tests, supplemented by limited materials testing and two percolation tests. We visited the site on the 5th June 2008 to undertake the field testing. Our observations, conclusions and preliminary recommendations for development follow.

2. SITE DESCRIPTION

The main site is located at the northern Mtubatuba - N2 freeway intersection, on the eastern side of the freeway. The site is near level with a slight fall to the east. There are no obvious natural drainage channels across the site (although there are a couple of channels excavated to drain the freeway). The northern access road to Mtubatuba town traverses the site in the southern third. At present, the majority of the site in under commercial plantations, there also being a livestock yard just north of the Mtuba access road. Not shown on the orthophotograph site plan is the new bridge over the N2 with the associated on- and off-ramps that cross the western edge of the site.

The second site is immediately north of Mtubatuba town, also currently under formal plantations. This site is also near level with a slight fall to the south. No drainage lines cross the site.

3. FIELD WORK

Ten inspection pits, designated IP1 to IP10, were mechanically excavated by TLB at the approximate locations indicated on the site plans, Figures 1 & 2. Subsoil profiles are recorded in Appendix A. Depths of 1.9 to 2.7m were achieved with excavation slowing in places on the weathered rock at the N2 intersection site. No refusal was met on the Mtuba North site.

Eight dynamic cone penetrometer tests, named DCP1 to DCP8, were carried out at the approximate positions indicated on the site plans, Figures 1 & 2. Test results follow as Appendix B. Depths of 0.6 to 3.6m were achieved before refusal in each instance.

Five bulk disturbed material samples were collected for testing. These were returned to Thekwini Soils Laboratory to determine the indicators, Modified AASHTO density and CBR values. Results are summarised together with the materials analyses in Appendix C.

Two percolation tests, designated PT1 and PT2, were undertaken at the approximate positions indicated on the site plan, Figure 1. After saturation of the subsoils, both tests failed, that is, it took longer than 30 minutes for a 25mm drop in water level.

4. WESTERN N2 SITE

4.1 Geology

The western site adjacent to the N2 freeway is underlain by variably weathered basalt of the Letaba Formation and the associated clayey residual soils derived therefrom.

From the inspection pits it is evident that the colluvial cover is some 0.25 to 0.7m thick and comprises very slightly moist, dark grey brown, dark grey or dark red brown, firm to stiff, fissured, sandy clay, occasionally containing gravel. Below the colluvium, the residual soils are typically dark yellow, dark yellow orange or dark red orange, stiff, fissured to shattered, silty or gravelly clay. These soils are expected to be moderately active with fluctuating soil moisture contents.

The bedrock is recovered as a highly to completely weathered, dark yellow and dark grey speckled cream, close to medium jointed, very soft to soft rock basalt. As the basalt was placed in layers and as the material varies laterally and with depth (a function of the variable rates of cooling on placement) there is a potential for quite variable degrees of weathering (and rock hardness) over short distances.

4.2 Laboratory Test Results

4.2.1 *Colluvium*

The single sample tested proved to be a sandy clay with 47% clay content and a grading modulus of 0.43. The plasticity index was 18 and a linear shrinkage of 9% was recorded. In terms of the Revised US Classification, this is an A-6 (9) material.

The Modified AASHTO density was 1602kg/m^3 with an optimum moisture content of 20.3%. CBR values were 2 and 6.9 at 90 and 100% of Mod density. Due to the excessive CBR swell of 3.54%, this material cannot be classified in terms of the TRH14(1985).

4.2.2 *Residual Basalt*

Two samples were tested, grading as fine sandy clays with 24 and 54% clay content and grading moduli of 0.99 and 0.84 respectively. The plasticity indices were 18 and 19 while linear shrinkages of 9 and 10% were measured. In terms of the Revised US Classification, these are A-7-5 (5) and A-7-6(9) materials.

The Modified AASHTO densities were 1636 and 1710kg/m^3 and the optimum moisture contents 18.7 and 18.2% respectively. CBR values were limited to $\leq$ 1 at 90% and 1.9 to 3 at 100% of Mod density. The maximum CBR swell was an excessive 8.23% and hence these materials cannot be classified in terms of the TRH14(1985).

4.2.3 *Weathered Basalt*

The two weathered rock samples graded as clayey sands with 4 and 9% clay contents and grading moduli of 2.05 and 2.51. The plasticity indices were 14 and 15 and the linear shrinkage a uniform 7%. In terms of the Revised US Classification, these are A-2-7 class materials.

Modified AASHTO densities of 1810 and 1874kg/m^3 were determined with optimum moisture contents of 14.8 and 13.9% respectively. The CBR values were 4 and 12 at 90% of Mod density, increasing to 7 and 29 at 100% of Mod. Excessive maximum CBR swells of up to 2.73% were recorded preventing classification in terms of the TRH14(1985).

4.3 <u>Geotechnical Assessment and Recommendations for Development</u>

4.3.1 *Stability*

The near level site is considered inherently stable. Similarly, the cohesive underlying clayey soils should stand unsupported in near vertical trench sidewalls in the short term during construction.

4.3.2 *Excavatability and Earthworks*

Depths of at least three metres should be easily achieved mechanically in an open excavation and even in confined trenches with a reasonable TLB. Where (medium to coarse gravel size) blocky basalt is encountered in trenches, the interlocking of the blocks may make excavation challenging, but open excavation should still be straight forward.

Earthworks are expected to be limited to trimming of individual sub-divisions and installation of foundations and services as the overall site is already near level. No undue problems are expected in normal earthworks for this purpose. The insitu materials (both residual clays and weathered bedrock) do have a potential for heave if over compacted so all due caution must be exercised during earthworks to reduce heave (by limiting compaction to 90% of Mod density).

4.3.3 *Founding*

From the initial proposals it would appear that structures will include a petrol garage, a truck stop, a hotel and extensive parking areas. Before finalising design, site and structure specific founding assessments will be required.

Generally, where competent bedrock is encountered within 1.5m of platform level, strip footing foundations bearing into competent rock could be used. Where soil depths are greater than 1.5m, structures could be supported on ground beams spanning deeper, isolated footings into rock. Alternatively, if the structure is flexible and can accommodate some seasonal soil volume fluctuations, it may be founded through all fill into stiff residual clay at a minimum depth of 1.5m below final platform level. It is not expected that piling will be necessary on this site unless high bearing pressures were proposed in areas of deeper and softer soils.

4.3.4 *Site Drainage*

Due to the natural gentle gradient, surface drainage could be challenging across this site, however, this would be remedied during the creation of the platforms by the careful planning of levels and falls. Before the construction of the adjacent N2 bridge intersection, there was a problem with standing water at the western extent of the site but this appears to have been sorted out by the changing of levels during bridge construction.

There are a number of storm water drains from the N2 national road on the west of the site. These channels will have to be continued through the site or piped to discharge past this site, as the current layout will impose excessive surface storm water run-off on future development of this site.

No obvious subsurface seepage was noted during the site testing and subsoil seepage is not expected to be problematic. Any isolated areas of seepage as may be associated with the more deeply weathered basalt zones should be treated symptomatically. Subsoil cut-off drains will, however, be slow to take effect due to the high clay content of the soils. The plantations on the northern portion of the site will be felled for development. At present, these alien trees transpire copious quantities of water. There is a possibility that some subsoil seepage could only become evident once the trees have been removed and the natural system is back in balance.

4.3.5 *Erodibility*

The cohesive insitu clay soils are not particularly prone to erosion by wind or flowing water. The gentle slope gradient will further reduce the potential for erosion.

4.3.6 *On-site Sanitation*

Both percolation tests failed (north and south of the Mtuba access road) and hence, on-site waste water disposal by subsoil percolation is not considered feasible, even for limited volumes of effluent. The large volumes of effluent generated by a hotel and truck stop would certainly not be readily disposed of on site by french drain.

The development would either have to be connected to the municipal water borne sewer system in Mtubatuba (known to have limited capacity and not able to cope with the current load) or, alternatively, an on-site mini sewage treatment works could be considered. The treated, purified water could be re-used on site for washing trucks or irrigation of hotel lawn areas or discharged into the nearest natural water course.

4.3.7 *Construction Materials*

From the laboratory testing, it is evident that the insitu clays and weathered basalt bedrock are all potentially active and have a moderately high potential for heave when over consolidated. These materials are considered poor as subgrade below pavement layer works or surface beds and should be used with caution, or ideally, replaced. Most of the construction materials will have to be imported from a suitable local commercial source.

4.3.8 *Hydrogeological Assessment*

While a formal hydrogeological assessment was not carried out within the scope of this preliminary investigation, it is not expected that the development of a well maintained petrol station with the attendant subterranean fuel storage tanks would pose undue risk to the environment and ground water in particular. The low porosity clayey subsoils will limit passage of any spills while no evidence of shallow ground water was noted on this site at the time of the assessment. This should be better assessed by a detailed hydrogelogical assessment including a one kilometre radius around the site, at a later stage of the project.

5. MTUBATUBA NORTH SITE

5.1 Geology

The northern site is underlain by unconsolidated, wind deposited, fine grained sands of the Kwambonombi Formation to at least 2.6m depth. The upper soils are typically very slightly moist, very light grey brown or very light yellow brown, loose to medium dense, very slightly clayey fine grained sands. These sediments become increasingly clayey and stiff with depth, ultimately being underlain by basalts. The depth to bedrock was not confirmed within the scope of this assessment.

Mottled colouration in the Kwambonombi sands suggest a seasonal perched ground water table as shallow as about 1.5m in places.

5.2 Geotechnical Assessment and Recommendations for Development

5.2.1 *Stability*

The near level site is considered inherently stable in terms of overall slope. Conversely, the low cohesion, loose sandy soils underlying the site are considered to have a high potential for collapse from unsupported, near vertical trench sidewalls, even in the short term during construction. All trenches deeper than about 1.2m will require shoring for safety.

5.2.2 *Excavatability and Earthworks*

Excavation to at least three metres depth should not be difficult, manually or mechanically, apart from the need for shoring or otherwise supporting the low cohesion sands.

Again, earthworks are expected to be limited to the trimming of building platforms and installation of services and foundations. The sands are typically quite easy to work with and achieve required compaction, provided there is not an excessive moisture content. All backfills and embankments must be suitably compacted as the loose sands have a potential for consolidation and collapse settlement.

5.2.3 *Founding*

Due to the potential for both collapse and consolidation settlement in the low cohesion, loose sandy soils, deeper founding methods will likely be recommended when the site specific founding assessments are carried out. While small and/or flexible structures may be suited to lightly reinforced footings on an engineered soil raft, larger multi-storey structures may require piling or deeper footings. The detailed individual site investigations for multi-storey structures should attempt to better ascertain the depth of the sands, possibly by means of dutch probe testing or drilling of boreholes.

5.2.4 *Site Drainage*

While no shallow seepage was noted at the time of the investigation, Mtuba town is known to suffer from widespread shallow ground water conditions. Mottling in the soils exposed in the pits suggest seasonal moisture may rise to about 1.5m.

The plantations on this site will be felled for development. At present, these alien trees transpire copious quantities of water. There is a possibility that, in addition to the seasonal seepage, there will be a rebound in the ground water once the trees have been removed and the natural system is back in balance.

5.2.5 *Erodibility*

The low-cohesion sands are considered to be highly susceptible to erosion by wind and flowing water. While the sands will be exposed to wind by the removal of vegetation for development, the gentle slope gradient reduces the risk of erosion by fast flowing water.

All due precautions should be taken during and after construction to reduce the risk of erosion.

5.2.6 *On-site Sanitation*

While no percolation tests were carried out on this site, the fine grained sands are expected to have a reasonable percolation rate although this reduces in the very fine grained sands. Whether on-site waste water disposal by french drain is feasible on individual sites will depend on the site size and the nature of the proposed development. However, due to the porosity of the sands and the regional shallow ground water table, there is a moderately high risk of contaminating the ground water.

Ideally, the municipal sewer system should be upgraded to accommodate the effluent generated on this site. It is understood that the phased development will start with a community clinic. If this is the only development for some time, it would likely be feasible to run this on a septic tank and french drain for the near future. This can only be confirmed once the exact layout, site size and expected effluent volumes are known and a site specific percolation test is carried out.

5.2.7 *Construction Materials*

No materials sampling was undertaken on this site, however, from past work in the area, it is known that the slightly clayey sandy soils are typically suitable for use as general subgrade. The deeper, more clayey materials may exhibit a potential for activity but this is generally limited. The sands are typically too fine grained for use as plaster or mortar sand (although used extensively informally). All higher layer works and other construction materials will have to be imported from a suitable local commercial source.

6. CONCLUSION

Both the N2 and Mtuba North sites are considered suitable for the proposed commercial / community type development. No geotechnical fatal flaws were noted that would preclude development of these sites, however, both have geotechnical challenges that will have to be taken into account.

The N2 site is underlain by potentially active basalt and the associated residual clay soils. Appropriate founding could be footings into shallow rock or footings into stiff clays where structures can accommodate some movement. No obvious seepage was noted but the natural water levels could rebound once the plantations are removed for the development. On-site waste water disposal by subsoil percolation is not considered feasible on this site.

Mtuba North is underlain by loose to medium dense, low cohesion, slightly clayey sands which are expected to have a potential for both collapse and consolidation settlement. Here founding may have to be taken to depth by means of piling for multi-storey structures while smaller, flexible structures could be supported on reinforced footings. This will depend on the depth to competent rock which was not determined within the scope of this assessment. While the porous sands may be better suited to on-site sewage disposal in french drains, it is recommended that the municipal sewer system rather be upgraded to accept this effluent as there is a risk of contaminating the ground water by the large volumes of effluent generated by the full development. Initially, when only the community clinic is developed, it may be feasible to put this onto a septic tank.

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APPENDIX A

INSPECTION PIT PROFILES

(IP 1 - IP 10) & (PT 1 & PT 2)

APPENDIX B

DYNAMIC CONE PENETROMETER TEST RESULTS (DCP 1 - DCP 8)

APPENDIX C

LABORATORY TEST SUMMARY

MATERIALS ANALYSES

FIGURE 1

SITE PLAN

FIGURE 2

SITE PLAN