APPENDIX 5: Wastewater Recommending Report (Holmes Consulting)

Te Kowhai Airpark Development
Wastewater Recommending Report

Limmer Road (SH6)
Te Kowhai
Hamilton

Holmes Consulting

Revision 2
27 June 2017
131928.00

Holmes Consulting

Report

Te Kowhai Airpark Development –Wastewater Recommending Report

Prepared For: TK Airfield Limited Partnership

Date: 27 June 2017
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Holmes

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Holmes

EXECUTIVE SUMMARY

The Te Kowhai Airfield subdivision is located 5 km from the nearest branch of the Hamilton City reticulated wastewater network. After consideration of a number of solutions, it is considered that on-site treatment and dispersal of wastewater is the most appropriate solution for this site.

Peak wastewater flows of approximately 70 m^3/day are expected and will consist of a 90/10 spilt between residential and commercial sources. Due to the dilution of commercial flows it is likely that wastewater strength will be close to that expected from a standard residential source.

Various constraints were considered in terms of environmental sensitivity, capital expenditure, operating costs, and potential for staging of construction. The recommendation for on-site wastewater treatment, after considering these constraints, is a Septic Tank Effluent Pumping (STEP) system feeding to a recirculating textile packed bed reactor (PBR). This form of treatment is able to meet likely Waikato Regional Council effluent quality requirements.

Land dispersal of treated effluent is considered the most environmentally sustainable approach for this site. Trench disposal is recommended due to the reduced land area required over a dripline bed. However, due to high groundwater levels and the free draining nature of the underlying soils, the ground level will need to be raised by 1.05 m using approximately 1,470 m^3 of Category 3 soils that will likely need to be imported to site.

It is recommended that a staged treatment plant installation is undertaken to reduce initial capital expenditure, however it is likely that the entire disposal field will need to be installed in its entirety at the outset.

1 INTRODUCTION

Te Kowhai Airpark development is a proposed airpark with some commercial facilities located adjacent to Te Kowhai Airfield. Once complete the development will contain 133 residential dwellings, 11,124 m^2 of commercial floor area, and 75 public aircraft hangers.

1.1 Scope of Work

The scope of work for this portion of the project includes the following:

1. Complete an options assessment with regard to on-site wastewater treatment/disposal and provide recommendations in this regard.

1.2 Limitations

Findings presented as a part of this project are for the sole use of TK Airfield Limited Partnership in their evaluation of the subject property. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses. Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

2 SITE CONDITIONS

2.1 General

The site is located on flat ground adjoining Te Kowhai airfield. It is bordered on the south by State Highways 39a and to the north by the Te Kowhai airfield runway. The site is located approximately 3 km from the Waipa River to the west and 6.5 km from the Waikato River to the east.

2.2 Surface Water

As noted above, the Waipa River lies approximately 3 km from the boundary of the subject site. A number of small tributaries of the Waipa River drain the land surrounding the site. The largest of these is Ohote Stream which drains Lake Rotokauri to the south. The closest tributaries are located 400-600m from the proposed development.

2.3 Vegetation Cover

The site vegetation is largely grassed pasture, with isolated areas of hedgerows and trees.

2.4 Permeability Testing

Permeability testing has been carried out by Bloxam, Burnett & Olliver Limited (BBO) and is included as Appendix B. Soakage rates in both boreholes tested was approximately 350 mm/h. It is noted that standard test procedures were not followed and therefore this testing is only seen as indicative of soakage rates. However 350 mm/hr is well above the 125 mm/hr threshold for category 1 soils within AS/NZS 1547:2012 and therefore this categorisation is likely. Further permeability testing should be undertaken prior to any detailed design.

2.5 Groundwater

As outlined in the BBO geotechnical report, groundwater was generally encountered at 1.3-2.5 m below ground. However, this was during summer months and higher groundwater levels are expected through winter months, including minor areas of low land flooding.

3 WASTEWATER CHARACTERISTICS

3.1 Wastewater Design Flows

The proposed subdivision development is split into five main areas or precincts. The layout of these areas is as per the proposed development plan attached as Appendix A.

Precinct A Runway and Operations Precinct
It is anticipated that the runway and operations precinct will have minimal wastewater demand.

It is noted that no allowance has been made for aeroplane wash down. Aircraft wash down has the potential to adversely impact an onsite wastewater treatment system due to oils/grease and large hydraulic loads. In this regard it is assumed that a separate system will be established for treatment and disposal from a dedicated wash down area. This will likely be in the form of a proprietary oil and grit interceptor followed by a bio retention swale and soakage to ground. Treatment in this manner is deemed feasible and further consideration will occur under detailed design.

Precinct B Public Hangers
The public hanger precinct will contain 75 aircraft hangers. It is understood that these hangars will be serviced by a number of centralised bathrooms. It is conservatively assumed that on the busiest day of the year half the hangers would be in use. Assuming a maximum 4 persons per aircraft, and all persons using bathroom facilities prior to or after flight, this would equate to 150 bathroom uses.

A single toilet use will on average produce 6-7 litres of wastewater from the toilet itself and 2-3 litres from hand washing. Conservatively it is estimated a bathroom use will produce 10 litres of wastewater. This is consistent with Table H4 of NZS1547:2012 'Community Halls – Meetings' which is the closest applicable source within the standard. On the busiest day of the year it is therefore anticipated that the 75 public hangers will produce a maximum 1,500 litres/day of wastewater.

Precinct C Aeronautical Commercial
This precinct will contain light commercial activities. In the absence of specific supportable design data Figure 5.1 of The Hamilton City Design Manual requires commercial flows to be calculated at a density of 30 persons per hectare. The total land area within the commercial precinct is approximately 3.5 hectares and therefore 105 persons are anticipated based on the Design Manual requirements. Table 6.2 of ARC TP58 recommends 40 l/day for 'Day Staff' for standard facilities. Based on these numbers the commercial precinct anticipated to produce 4,200 litres/day of wastewater.

Precinct D General Residential
Table H3 of AS/NZS1547:2012 recommends 180 l/day/person for a household with standard fixtures and on-site roof water supply. NZS4404:2010 recommends an average 2.5-3.5 persons per dwelling and therefore an average 3 persons is anticipated per dwelling. Precinct D is proposed to contain 88 residential dwellings. Based on NZS1547 and NZS4404 this equates to a maximum 47,520 litres/day of wastewater.

Based on the above volumes the peak day total wastewater production from the development as a whole is approximately 70,000 litres. In reality, the daily wastewater generation may be less than this. However, the wastewater treatment and disposal facilities must be designed to meet the maximum potential demand.

3.2 Wastewater Strength

The wastewater on the site will come from a mix of both residential and commercial premises. Due to the lack of dilution from showers, baths, washing machines and other similar facilities, the wastewater strength from the commercial premises is normally expected to be much higher than that from the typical domestic strength residential effluent, a comparison is as shown in Table 3-2 below.

Table 1: Wastewater Strength Comparison

Constituent (mg/l)	Typical Domestic Strength [1]	Typical Commercial Premises
BOD_5	250-350	800
Total Suspended Solids	300-400	800
Total Nitrogen	varies	120-180

[1] ARC TP58

As the commercial flows only make up a small portion of the wastewater flows from the total development it is therefore anticipated that commercial flows will be diluted substantially. Wastewater strength will therefore be close to that normally expected from standard residential development.

3.3 Nitrogen Reduction

Any discharge of treated wastewater or effluent onto or into land within the Waikato Regional Council's jurisdiction needs to comply with the maximum loadings outlined in the Waikato Regional Plan. At present this limit is noted as 150 kg N/ha/year. As the total land area for the greater site is very large (at approximately 45 ha), the overall nitrogen loading easily complies with this limit. However, as the intention is to dispose of the treated wastewater over as small a disposal field as possible, the possibility of a plume forming, able to concentrate nitrogen into the groundwater is possible and therefore some nitrogen reduction is recommended. It is also recommended that grass or other vegetation is grown over the disposal field, and that the grass clippings are removed from this area when the lawn is mowed to allow uptake of some of the nitrogen by the vegetation.

The level of nitrogen reduction possible through wastewater treatment is a product of a number of factors. The main factors are influent strength, carbon availability, alkalinity availability, temperature and toxicity.

Two main steps are involved in nitrogen reduction. The first step is nitrification, which is the oxidisation of ammonia to nitrate, followed by denitrification, the reduction of nitrate to nitrogen gas. These two biological processes require very different conditions for effective nitrogen reduction.

Nitrification is carried out by aerobic bacteria and therefore needs to occur in the presence of abundant oxygen. BOD_5 reduction also requires free oxygen, and due to the more aggressive organisms involved in organic decomposition, the nitrification process will generally follow BOD_5 reduction. Partly for this reason, it can take 1-3 months for an adequate population of nitrifying microbes to populate a wastewater treatment system. The nitrification process also creates acid, thereby lowering the pH of the biological population. As the growth rate of nitrifying microbes slows with decreasing pH, and nitrification stops completely below a pH of 6, adequate alkalinity (7.1 mg/L as $CaCO_3$ for each mg/L of ammonia nitrogen to be oxidised) is required. Dosing of alkalinity into the wastewater system is often required to assist nitrification where challenging conditions exist.

Denitrification occurs in the absence of oxygen, as the microbes involved can obtain their oxygen requirement from either dissolved oxygen or by taking it off nitrate molecules. In anoxic conditions, with dissolved oxygen below 0.5 mg/L, nitrate becomes the primary oxygen source for these microbes, and denitrification occurs. The bacteria are also heterotrophic and require a carbon source as food. Dosing of a carbon source such as molasses is often required to enhance denitrification where an adequate carbon source is not naturally present in the wastewater to be treated.

Both biological processes are temperature dependent. Although nitrification will occur at a reduced rate below temperatures of 10°C, the initial establishment or re-establishment of nitrification requires temperatures above 10°C. Denitrification will occur between 5 and 30°C.

The processes are also very sensitive to toxicity, especially nitrification, and careful consideration will need to be paid to cleaning products and other toxins able to enter the system. In particular any products containing Quaternary Ammonium Compounds (QAC) should be avoided.

For these reasons, it is recommended that the wastewater treatment plant is chosen carefully to ensure the likely future conditions of consent will be met.

4 TREATMENT AND DISPOSAL OPTIONS

There are a number of options generally available for dealing with wastewater of the type expected from the Te Kowhai Airpark development, however, due to the specifics of the site, some of these options are not considered economically or technically viable.

4.1 Discharge to Council Sewer

Te Kowhai township located 1 km to the north has a small localised wastewater scheme serving approximately 10 houses. This scheme is not considered to have capacity for connection of this development.

The Te Kowhai Airpark development is approximately 5 kilometres from the nearest branch of the Hamilton City Council reticulated wastewater network located at Te Rapa Park to the east. The option to construct a conventional gravity sewer has been considered and discounted due to cost associated with intermediary pump stations and political difficulties with inter council wastewater connections between Waikato District and Hamilton City.

An alternative to the conventional gravity sewer is a Septic Tank Effluent Pumping (STEP) system. With this system large volume septic tanks for the commercial developments and individual septic tanks for residential lots receive all of the wastewater from the development, where solids settlement and decomposition occurs. Screened pump vaults take effluent from the clear zone within the septic tank and pump the primary treated wastewater into a small bore effluent sewer. As the solids are retained within the tanks (for pump out every 5-10 years), the effluent sewer/rising main can be smaller than would normally be required for a traditional sewage pumping station and rising main, and a maximum total design head (TDH) of up to 60 m can be accommodated without intermediate pump stations. For this site, a small bore sewer system associated with a STEP system is technically feasible, however, the associated costs are likely to be higher than for on-site treatment and disposal. It is also possible that additional treatment will be required prior to discharge to the sewer or treatment plant, for example aeration to improve dissolved oxygen concentration. Typically the costs for installing the small bore sewer system are around $75/m, accounting for a capital cost of approximately $375,000 for the sewer main, excluding the on-site primary treatment tanks and pumps. Development contributions to council will be payable per lot and for the commercial development. These are estimated to be approximately $8,000/lot ($944,000), and a comparable pro-rata contribution for commercial activities of an assumed $85,000. Including development contributions and pipe work the total developer cost is estimated at approximately $1,400,000. Noting this excludes the cost of STEP tanks and pumps.

While the ongoing maintenance associated with this option is lower than an on-site system, the initial installation costs for approximately 5 kilometres of pipeline are higher and far more challenging, as easements for services within road reserves and interaction with existing services will need consideration. This option also suffers from political difficulties with inter council wastewater connections between Waikato District and Hamilton City. As such, this option has not been investigated further.

4.2 On-site Wastewater Treatment and Dispersal

A number of options are available for the conveyance, treatment, and dispersal of wastewater close to its source of generation.

4.2.1 Wastewater Conveyance

Gravity Sewer

The option of a conventional gravity wastewater pipe network feeding to a centralised wastewater treatment plant within the proposed airpark developmetn has been considered and dismissed due to treatment costs associated with the requirement by Hamilton City Council for a 2x infiltration factor to be added to peak wastewater volumes from conventional gravity pipe networks.

STEP/Pressure Sewer

Due to the potential staging of development and available areas of green space, the subject site is ideally suited to a localised low diameter Septic Tank Effluent Pumping (STEP) system, or alternatively a standard pressure sewer. A sealed sewer system should result in no significant inflows and infiltration and therefore wet weather peaking factors can be removed when sizing the on-site wastewater treatment and effluent disposal field. This system also has the added benefit of being more resilient through the use of polyethylene pipe.

4.2.2 Primary Treatment

A STEP system (effluent sewer) undertakes primary treatment within a septic tank located within each residential/commercial lot and resultant effluent is pumped to a centralised secondary treatment and dispersal area. The primary tanks retain the majority of solids and the sludge is pumped out every 5-10 years. The cost of the tank, effluent pump, sludge removal, and operational costs is generally the responsibility of the future lot owner.

A grinder pump pressure sewer system utilises individual pump stations installed within each residential/commercial lot and macerated raw sewage is conveyed to a centralised treatment plant where both primary and secondary treatment occurs. The cost of the wet well, macerating pump, and ongoing operational costs is generally the responsibility of the future lot owner. The developer is still required to install primary septic settling tanks at the centralised plant and undertake periodic removal of sludge.

4.2.3 Secondary Treatment

A number of options exist within New Zealand for on-site secondary wastewater treatment at the scale required for this project. These include:

- Recirculating Textile Packed Bed Reactors (PBR)
- Submerged Aerated Filtration (SAF)
- Sequencing Batch Reactor (SBR)
- Membrane Bioreactors (MBR)

All the above systems can result in treated effluent that will meet the likely required secondary treatment standards of <20 mg/ltr BOD_5 and TSS, with total nitrogen to meet the permitted activity threshold of <150 kgN/ha/year, averaged across the development.

4.2.3.1 Recirculating Textile Packed Bed Reactor (PBR)

The PBR is a biological treatment process that involves the application of small, regular doses of primary treated effluent onto a textile bed from a recirculation tank. The textile bed has a large effective surface area and is well ventilated, promoting the growth of a wide range of naturally occurring organisms. These organisms break down the wastewater, providing a high level of treatment. The treated wastewater then passes back into the recirculation tank for further treatment, with a portion of the highly treated wastewater passing out into a final tank before being pumped out into drip irrigation for final dispersal.

Enhanced nitrogen reduction in the PBR is achieved through recirculating a portion of the partially treated and oxygenated wastewater into an anoxic tank (often the primary treatment tank) for denitrification.

4.2.3.2 Submerged Aerated Filter (SAF)

A SAF is an activated sludge system where aerated wastewater is made to flow through a submerged filter, which acts as a media to support a population of microorganisms. The process of producing aerated wastewater involves forcing air into primary treated wastewater through the use of an aerator. After passing through the submerged filter, the partially treated wastewater moves into a secondary clarifier before entering the final chamber and being pumped out into the dispersal field.

4.2.3.3 Sequencing Batch Reactor (SBR)

The SBR process is a form of activated sludge system, with the wastewater undergoing sequential stages of mixing, aeration and settlement, with the treated wastewater decanted off from the clear zone for final disposal. The aeration is required to support the population of microorganisms and fine tuning of the system is often required to ensure optimal wastewater treatment.

4.2.3.4 Membrane Bioreactor (MBR)

The MBR process is a form of suspended growth activated sludge system, with micro or ultra-filtration used in place of secondary clarifiers. The primary effluent enters an anoxic zone, and follows through into an aerobic zone which also houses the membrane equipment. The membranes provide a physical barrier to particulate matter, a large proportion of pathogens including Giardia and Cryptosporidium, and much of the microorganism population responsible for providing biological treatment.

As in the PBR, enhanced nitrogen reduction is achieved by recycling a portion of the mixed liquor from the aerobic zone back through anoxic zones for denitrification.

4.2.3.5 Comparison of Options

The nature of the site and the wastewater generated, with fluctuating loads, low temperatures, and other factors, results in a list of benefits of the various systems, as indicated in table 2 below.

Table 2: Treatment System Performance

Treatment System	Sludge Generation	Nitrogen Reduction Capacity	Ability to handle peak loads	Maintenance Input Required	Capital Cost	Biological Treatment Level	Power Use
PBR	Low	Moderate	Good	Moderate	Moderate	Good	Low
MBR	High	Good	Moderate	High	High	Excellent	High
SAF	Moderate	Moderate	Poor	Moderate	Low	Moderate	High
SBR	High	Low	Poor	High	Low	Moderate	High

Based on the above comparison and considered benefits the recommended treatment system would be a PBR. The PBR option is likely to have capital costs of approximately $420,000 for the plant and $100,000 for the associated land disposal. An additional $50,000 would also be required for install of the pressure effluent/sewer pipe and fittings (1400 m @ $30/m) to each lot. Total developer capital cost is estimated at approximately $570,000 for a pressure system with PBR treatment and effluent disposal to ground.

4.2.4 Treated Effluent Dispersal

Disposal to Water

The discharge of wastewater into water is a discretionary activity under the Waikato Regional Council's Water Plan, and a resource consent for this discharge where there is the ability to discharge onto or into land is unlikely to be granted. For this reason, the discharge of treated wastewater into any of the water courses in the area has not been further investigated.

Disposal to Land
Dispersal into or onto land, including land treatment, can be carried out in a number of ways, including drip irrigation, infiltration trenches or beds, mounds or surface irrigation. The method of disposal is determined by the soils on the site and the sensitivity of the receiving environment. AS/NZS 1547:2012 requires a minimum set back of 0.6-1.5 m between the base of the disposal trench and groundwater.

Two main disposal options have been considered for the Te Kowhai Airpark Development. These are drip irrigation and traditional trench disposal. With winter groundwater levels close to current surface both disposal option will require modification of the existing ground to achieve the required minimum 0.6 m unsaturated soil below and thereby achieve appropriate levels of treatment prior to effluent entering the water table.

4.2.4.1 Drip Irrigation

To achieve the required 0.6 m minimum separation to ground water, drip irrigation laid within the top 100 mm of soil will require the disposal area to be raised by 0.6 m. This can be achieved by scarifying/ploughing the existing ground surface and then placing appropriate free draining fill to 0.6 m above. Dripline can then be placed and 100 mm of topsoil placed over and grassed.

For Category 1 soils as identified in the soils report, the following design loading rates have been determined from AS/NZS1547:2012.

Table 3: Dripline Design Loading Rate

Disposal Methodology	Design Loading Rate (DLR)	Area required
Drip irrigation	5 $l/m^2/day$	14,000 m^2

Due to the large area required for disposal the use of drip irrigation at this site has been discounted.

4.2.4.2 Conventional Trench

Traditional trench disposal is installed at a depth of 450-600 mm below ground and as per Figure L5 of AS/NZS1547:2012. To allow for a minimum 0.6 m unsaturated soil below the base of the trench, the ground surface in the area of the trenches will need to be locally raised by at least 1.05 m. This can be achieved by scarifying/ploughing the ground surface prior to installation of the base of the trench within the top 100 mm of soil. The standard trench profile can then be constructed with the final topsoil layer being graded at a 1V:3H line from the edge of the trench.

In order to achieve suitable contact times and nutrient retention Table L1 of AS/NZS1547:2012 identifies that conventional trenches within category 1 & 2 soils require importation of a suitable filtration media to achieve a discharge control trench as per Figure L4 of the standard. In this regard the material utilised to raise the local ground level shall be Category 3 (or below) to ensure appropriate contact times.

For moderately structured Category 3 soils, the following design loading rates have been determined from AS/NZS1547:2012.

Table 4: Trench Design Loading Rates

Disposal Methodology	Design Loading Rate (DLR)	Area required
Conventional Trenches	50 $l/m^2/day$	1,400 m^2

It is recommended that conventional trench disposal systems be utilised for the airpark development due to the smaller area required when compared to drip irrigation. This would reduce the costs to prepare the disposal beds as well as on-going maintenance of the area.

4.2.5 Location of Wastewater Facilities

It is recommended that the treatment and disposal facilities for the airpark's wastewater be located adjacent to each other for ease of access and maintenance. The exact location for these facilities would be finalised during the detailed design process, however it is noted that the most appropriate location for these facilities would be to the north of the runway in Area A of the airpark development. This location is removed from areas of easy public access and has sufficient space to provide for the required trench disposal beds.

5 RECOMMENDATIONS

Due to a lower capital cost and the added benefits in staging the installation and therefore deferring some of the capital expenditure, on-site treatment and disposal of wastewater is recommended as the preferred option over a gravity or STEP pumping system to the Hamilton City reticulated wastewater network at Te Rapa Park to the east.

5.1 Wastewater Conveyance

To convey wastewater within the site a conventional gravity wastewater pipe network feeding to a centralised wastewater treatment plant would require a 2x infiltration factor to be added to peak wastewater volumes. A sealed pressure sewer system is therefore recommended as this results in no infiltration and a more resilient pipe network.

5.2 Primary Treatment

A STEP system (effluent sewer) undertakes primary treatment within a septic tank located within each residential/commercial lot and conveys effluent to a centralised secondary treatment plant and dispersal area. This system allows the cost of primary treatment to be transferred to the future owner of the lot and results in a reduced capital outlay for the developer. Due to reduced cost and initial capital outlay it is recommended that a Septic Tank Effluent Pumping (STEP) system is installed at time of development and primary treatment tanks installed by future lot owners at time of building construction.

5.3 Secondary Treatment

From sections 3 and 4 above, the PBR is the preferred secondary treatment option for the development.

The benefits of the PBR include:

- Ability to handle peak loads. Diurnal peaks are buffered out in the septic tanks and recirculation tank stages, with small controlled volumes of wastewater dosed onto the textile media.
- Lower power requirements
- Low maintenance requirements with limited local operator knowledge required
- Low sludge production
- Low odour and noise production
- High quality treated effluent
- Modular installation allowing treatment plant size to be installed in stages

The PBR has the lowest OPEX costs and advantages in staging of the treatment plant, in that the treatment upgrades can be carried out in a number of stages, as growth occurs. It is possible that the total wastewater volume or strength may be less than anticipated, resulting in fewer upgrades than originally intended.

5.4 Land disposal

Although drip irrigation would likely represent a lesser impact on the environment and a lower capital cost associated with install than a trench disposal system, the fill material volumes required to raise ground levels over the 14,000m2 of drip disposal area would be considerably more than that required for the 1,400m2 area of trench disposal. Likewise, the large loss of land associated with the drip disposal system is a further negative against that option. The use of conventional trench disposal would represent a lower overall total cost and is therefore the recommended method of disposal.

5.5 Staging

It is recommended that the option to install a staged PBR is considered due to the advantages in reduced initial capital outlay. However, it is recommended that any Resource Consent Application applies for the ultimate volume to simplify future upgrades.

6 REFERENCES

1. *Auckland Regional Council Technical Publication No. 58 (TP58) On-site Wastewater Systems: Design and Management Manual Third Edition 2004*, A W Ormiston & R E Floyd

2. *AS/NZS1547:2012 On-site domestic-wastewater management*, Standards Australia/Standards New Zealand

3. *Hamilton City Council Development Manual 2006*

7 APPENDICIES

LEGEND

- Airside Lot - 1000m² -1200m² (13 total)
- Airside Lot - 1200m²-2500m² (21 total)
- Airside Lot - 2500m²-2600m² (36 total)
- Large Lot 2600m²+ (17 total)
- Small Lot - Typically 600m²-1000m2 (46 total)
- Street Trees
- Planted Acoustic Bund
- Taxiway (live aircraft movements)
- Existing Gas Main
- Existing Contours (0.5m intervals)
- Precinct Boundary
- Site Boundary

PRECINCTS

Precinct A - Runway and Operations
Precinct B - Commercial Precinct
Precinct C - Medium Density Residential
Precinct D - General Residential

NOTES

1. Aircraft storage hangers for general public use
2. General commercial hangers
3. Potential aeronautical themed cafe and playground
4. Existing building to be retained / renovated
5. Re-fueling / service hub
6. Public carpark
7. New airside commercial building
8. Aircraft operations area
9. Potential landscape buffer / SW storage area
10. Pedestrian / cyclist / golf cart access only
11. Entry feature / potential gate to residential area
12. Entry / gateway landscape feature
13. Existing gas lines to run underneath road
14. Potential helipad location
15. Existing gas line infrastructure

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INDICATIVE LAYOUT FOR DISCUSSION PURPOSES ONLY

TE KOWHAI AIRFIELD MASTERPLAN

Concept Masterplan

| Date: 13 April 2017 | Revision: B |

Plan prepared for Foster Develop Ltd. by Boffa Miskell Limited

Project Manager: Jonathanb@boffamiskell.co.nz | *Drawn:* SFo | *Checked:* JBr

TE KOWHAI AIRFIELD

PROPOSED SUBDIVISION

GEOTECHNICAL REPORT

BLOXAM BURNETT & OLLIVER

ENGINEERS • PLANNERS • SURVEYORS

MARCH 2008

TE KOWHAI AIRFIELD

PROPOSED SUBDIVISION

GEOTECHNICAL REPORT

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1. INTRODUCTION

This geotechnical report has been prepared for Te Kowhai Airfield and Micro Aviation to assist in their application for subdivision consent. The proposed subdivision is located between Limmer Road and the Te Kowhai airfield, as shown on the site plan in Appendix A, and is proposed to be rural/residential in nature, with airfield access to all lots.

The purpose of the investigation is to determine the suitability of the in situ soils to found standard timber framed buildings and for the disposal of stormwater and wastewater from each new lot. Characteristic information that is indicative of the site in general has been obtained, and further investigations will be required to allow detailed design work to proceed.

This report presents the results of the geotechnical investigation carried out at the site and makes general development recommendations.

2. SITE DESCRIPTION

The site is located between Limmer Road and the Te Kowhai airfield, to the south of Te Kowhai. The site is currently in pasture. The topography is generally flat, except for an area on the north (airfield) boundary, which has been used as a sand quarry and fill site, and contains both excavated and raised areas. The extent of fill in this area is indicated on the site plan.

3. SITE GEOLOGY

The Institute of Geological and Nuclear Sciences geological map "Geology of the Waikato Area" (2005) shows Hinuera Formation alluvium of late Quaternary age in the site area. This material is described as "cross-bedded pumice sand, silt and gravel with interbedded peat".

Seismicity and volcanism are not considered to be significant issues at this site. There are no known or observed active faults or recent volcanic activity in the area. No reason has been identified to depart from normal properties for seismic load.

4. FIELDWORK

Fieldwork was undertaken at the site on the 22nd of February 2008. This consisted of six machine augered boreholes located as shown on the site plan. The boreholes were excavated to depths ranging from 2.5 to 4.95 m, and Standard Penetration Tests (SPT's) were undertaken at 1.5 m intervals. Detailed field descriptions of the soils encountered are shown on the exploratory borehole logs attached in Appendix B. Two falling head soakage tests were undertaken adjacent to boreholes 1 and 6. Soakage test 1 was 0.94 m deep, and soakage test 6 was 0.8 m deep. These tests were not presoaked and testing was only undertaken during drilling of the adjacent boreholes.

5. SUBSOIL MATERIALS AND CONDITIONS

5.1 Fill

Fill was encountered throughout boreholes 4 and 5, these boreholes ended at 2.5 and 3.45 m depth respectively. These boreholes were not extended further due to auger damage that occurred in the drilling of borehole 4, where something grinded the auger shaft down, and concerns that further damage may occur in the drilling of borehole 5. Fill typically consisted of silt and sand layers, with medium dense/very stiff strengths (SPT's of 18 to 26). Apart from whatever caused the auger damage, the fill was relatively clean, with occasional topsoil streaks, dark grey and dark brown layers, and sub angular gravels that were inconsistent with Hinuera Formation materials.

The indicative extent of fill at the site is shown on the site plan in Appendix A. This information has been provided by the client.

5.1 Topsoil

Topsoil was typically encountered at the surface of the boreholes, down to depths ranging from 0.05 to 0.2 m. Exceptions to this are boreholes 4, 5, and 6. Boreholes 4 and 5 encountered fill at the surface, however borehole 5 has topsoil fill in the top 0.2 m. borehole 6 encountered topsoil down to 0.4 m depth.

5.2 Silt and Sand Layers

Hinuera Formation alluvial sand with occasional silt layers underlie the site, these were encountered in all boreholes except boreholes 4 and 5. Textures ranged from silt through to coarse grained sand, and gradings ranged from well graded (such as fine grained sand), through to poorly graded (such as silty fine to coarse grained sand). The most common texture was fine grained sand, or silt with fine grained sand. This material was dilatant and liquefied when vibrated.

Soil strength in the top 0.5 m of the boreholes appeared to be affected by the current soil moisture deficit, with dry, stiff descriptions in some boreholes underlain by moist, firm soils. The higher strengths observed near the surface are likely to be significantly reduced in winter when the watertable is raised.

The SPT test results were typically loose near the surface. Boreholes 3 and 6 also contained loose material at depth, and borehole 2 contained very loose material at depth (at 4.5 m). Medium dense results were obtained at depth in boreholes 1 and 2, and near the surface in borehole 6.

A silty peat layer was encountered in borehole 6 from approximately 4.2 to 4.5 m depth. This layer was described as dark brown, amorphous, and firm.

5.6 Groundwater

Material in the boreholes was described as being wet below 1.3 to 2.5 m depth, this is expected to be the depth of the current summer groundwater level. The groundwater level is expected to be much higher in winter, and according to local advice, low-lying areas of the site flood during prolonged periods of rainfall in winter.

5.4 Soakage

The soakage test results are included in Appendix C. As standard test procedures were not followed this testing is only indicative of soakage rates, and these rates may be optimistic. The final soakage rates in both boreholes was approximately 350 mm/h. This soakage rate falls within Category A of NZS 4610:1982, and is considered to be rapid to very rapid draining.

6. DESIGN CONSIDERATIONS

6.1 Floor Foundations

The proposed light weight residential building foundations are expected to have static design bearing pressures of 100 kPa under standard residential foundations. If design pressures exceed this please refer the matter back to Bloxam, Burnett & Olliver Ltd.

The in situ loose sands typically encountered near the surface of the site are not expected to have adequate allowable bearing capacity to support standard light weight timber frame structures (NZS 3604). Excavation and replacement of these materials, or piled foundations, may be possible where medium dense sands underlie these materials. Where loose material extends with depth, specially designed raft foundations will be required. Further geotechnical investigations will be required at each building site to determine the most appropriate foundation type and design.

The medium dense fill material typically encountered at the site is generally expected to have adequate allowable bearing capacity to support standard light weight timber frame structures (NZS 3604). Some minor excavation and replacement of loose/soft materials near the surface may be required, and a thorough investigation will be required at each building site to ensure strength consistency. Shallow spread or strip footings are expected to be sufficient to found standard light weight structures over this material. Where the required bearing capacity is not met either specially designed raft or piled foundations are expected to be acceptable. Piled foundations will require the presence of an acceptable founding layer.

Any especially weak or organic layers (such as those encountered in boreholes 2 and 6) underlying proposed buildings should be assessed for potential settlement. The relatively low organic content of the peat in borehole 6 is such that significant settlement due to organic decomposition is considered unlikely. The depth of this organic layer, and the very loose layer encountered in borehole 2, is such that pressures from typical building loadings are unlikely to have a significant effect at this depth. If layers with high peat contents, or peat/weak soils, occur close to the surface then their potential settlement should be considered in detail. If potential settlements are greater than acceptable then specially designed foundations will be required above these soils. If similar materials are found in other areas, they will require similar assessments and foundation designs.

Geotechnical investigations will be required at each building site to determine the most appropriate foundation type and design. Structure foundations are to be inspected by an Engineer to ensure the required minimum allowable bearing capacity for each structure is met.

6.2 Dilatant Materials

The extensive deposits of dilatant silts and fine grained sands at the site are expected to be unsuitable for earthworks, and should be left in place where possible. Unnecessary vibration of this material should also be avoided. The liquefaction potential of these materials should be recognised when designing foundations, and the risk and consequences of liquefaction specifically accommodated in design proposals. Further detailed investigations are required to examine the effects of liquefaction.

6.3 Stormwater Disposal

It is intended that stormwater from this site will be disposed of to ground. The site investigations indicate that soil soakage is typically rapid to very rapid with possible slight limitation only. However the advice that the groundwater level is at or near the surface in places during winter indicates that ground soakage will not be possible at all times. Additional stormwater control measures likely to be required at this site include ponding and tanks to detain and attenuate stormwater flows.

Additional soakage testing at each proposed disposal field, and winter watertable monitoring over the site, should be undertaken to allow the stormwater design to proceed.

6.4 Wastewater Disposal

The high winter watertable level will also affect the wastewater disposal design. The final design may include features such as a large holding tank or above surface low impact systems. The soakage testing and piezometers recommended in section 6.3 above will be required to allow the wastewater disposal design to proceed.

7. FURTHER INVESTIGATIONS

Further geotechnical investigation is required at individual building platforms to allow foundation design to proceed. Settlement assessments are required for building platforms on any weak silt and peat deposits. Liquefaction assessments are required to examine the effects of liquefaction on the proposed structures.

Piezometers are required over the site to monitor the winter groundwater levels. Soakage tests are required at each site where stormwater and wastewater disposal to ground is proposed to allow the soakage and wastewater design to proceed.

8. CONCLUSIONS

The surfical in situ soils are considered to generally be unsuitable to found light weight structures. In some areas the excavation of weak materials and replaced with engineered fill will be possible, however some areas are expected to require specially designed raft or piled foundations. The potential for liquefaction should be considered in the design of foundations above dilatant materials. Building platforms over weak or organic layers will require settlement assessments.

The high winter watertable level is expected to affect stormwater and wastewater disposal, requiring specially designed disposal systems.

9. LIMITATION

The recommendations and options contained in this report are based on our visual reconnaissance of the site, information from geological maps, data from the field investigation, and the results of in situ testing of soil samples from the site. Inferences about the nature and continuity of the subsoils away from and beyond the boreholes are made, but cannot be guaranteed.

During construction a geotechnical engineer competent to judge whether the conditions are compatible with the assumptions made in this report should examine the site. In all circumstances, if variations in the subsoil occur which differ from those described or assumed to exist, then the matter should be referred back to Bloxam, Burnett and Olliver Ltd.

This report has been prepared for the particular project described in the report and no responsibility is accepted for the use of any part of this report in any other context or for any other purposes.