4415
20 November 2020

GEOTECH consulting ltd

Selwyn District Council
PO Box 90
Rolleston

Attention: J. Lewes

Dear Ms Lewes,

RE: Plan Change 69
Rolleston Industrial Developments Ltd,
1491 Springs Road, Lincoln

Geotechnical Report Peer Review

1 Introduction

Geotech Consulting has been asked to carry out a peer review on the geotechnical report for the proposed Change to the Selwyn District Plan. The site is on the south side of Lincoln township and to the east and west of Springs Road. The Plan Change Request is to rezone the 186 ha area for residential use (Living LX & LZ) with a small zone for a local commercial centre (B1). The bulk of the area would be subdivided for a minimum of 12 households per hectare, with smaller areas of medium density (15 households per hectare) or large lots. The area could support about 2,000 new residential lots.

The report reviewed is titled *Geotechnical Assessment Report, Rolleston Industrial Developments Ltd, 1491 Springs Road, Lincoln, Rev 1,* by Coffey Services (NZ) Ltd dated 20 October 2020, for Rolleston Industrial Developments Ltd. In particular the peer review is to ensure compliance with the MBIE guidelines for the geotechnical assessment of subdivisions.

Geotech Consulting has also peer reviewed geotechnical reporting for the large subdivisions which adjoin the plan change area along most of the length of the north side. Reference has been made to these reviews to check consistency in conclusions.

2 Site Testing

The site testing has consisted of 20 cone penetration tests (CPTs) to between 2.2m and 12.0m (average 5.5m) depth and three MASW geophysical investigation lines totaling 2.6 km. The report states that other data from around the site area has also been considered, but no details are included.

> *Comment:* The MBIE guidance suggests 0.2 to 0.5 deep tests per hectare at plan change stage to characterize the soil profile to a depth of at least 15m. This gives a range 35 to 89 tests for the 178 ha area as given in the Coffey report, or about twice the number actually made. The western part west of Springs Road has only six tests with spacing up to 0.7 km apart. The MASW surveys help, but they are along part of one side of the site and in the eastern quarter. The number and depth of testing is questionable (refer to comments in (3), below). More testing is essential at subdivision consent stage, if the plan change proceeds.

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3 Subsurface Conditions

The tests show 0.3 – 0.4m of topsoil over interbedded silty sand, sandy silt and silt of variable thickness, on sandy gravel. The gravel is at a depth of 1.0 – 2.2m depth west of Springs Rd, and 3.5 – 5.5m depth east of Springs Road (Table 2 of the report). It is noted that the ground profile includes highly interfingered layering of silty and sand/gravel alluvium and the eastern edge potentially has organic soils. The MASW profiles are interpreted as confirming the CPT tests stopped on dense non-liquefiable soils with shear wave velocities greater than 200m/s. Groundwater was measured at between 0.5m and 3.4m in 17 of the 20 CPT holes, deepening to the west. A groundwater level of 1.0m has been assumed for the eastern part and 2.0 – 2.5m for the western part.

> *Comment:* The MASW profiles do not correlate particularly well with the stratigraphy inferred from the CPT tests. Our experience with MASW profiling on other sites in the Christchurch area has also highlighted a need for caution with their interpretation. The report does not refer to any geotechnical information other than the CPTs and MASW made as part of this investigation, and therefore there is no confirmation of soil types below the depth of the CPT tests, many of which are relatively shallow and with an average depth of only 5.5m. We have checked several bores on the Ecan well data base. The four looked at do show gravel soils from a depth similar to that shown in the closest CPT tests, and it does appear that the soils below about 5m are dense enough and of a grading such that liquefaction is not an issue. However, we recommend that Coffey research publicly available borehole information (Ecan well data base and NZ Geotechnical Database) to verify the deeper profile. This will probably also increase the number of locations where ground conditions are known, particularly along the northern side, and thus enhance confidence in the overall geotechnical model.
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> The soil profile as described is generally consistent with that determined for the subdivisions along the north side. We note that the area to the northeast does contain significant amounts of organics in places, such that careful consideration had to be given to how these more compressible soils would respond to filling and building loads. Without any sampling by test pit or borehole in this plan change area, there is a possibility that organic soils will be more widespread than anticipated.

4 Liquefaction potential

A liquefaction analysis of the CPT data suggests index settlements of less than the TC1 limit of 15mm in all but one of the 19 tests analysed at SLS levels of shaking, and less than the 25mm limit in all but two tests at ULS levels of shaking. The analysis includes transition layer adjustment. The report considers that with refinement of ground water levels, the estimated settlements will be reduced, and concludes that the majority of the site is equivalent TC1 with some small areas of equivalent TC2 land

> *Comment:* The analysis is by the MBIE standard procedure with appropriate input parameters. The use of a 1m water table depth for the eastern part is probably conservative. As no liquefaction outputs are provided, it is not known at what depths the liquefaction is predicted to occur. There is no discussion of evidence of ground damage in the 2010-11 earthquakes. It is noted that the site has certainly been well tested to in excess of SLS shaking and probably in excess of ULS shaking in the September 2010 earthquake, yet the closest residential land at the time of the earthquakes – further north with generally more sandy soils - was all classified Foundation Technical Category TC1 by MBIE, suggesting little to no ground damage.
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> The recent subdivisions adjacent to the north side also considered liquefaction. The land north of the subject land and west of Springs Road was concluded to be mostly TC1 with two small areas of equivalent TC2, similar to the conclusions in this report. The Te Whariki subdivision has had numerous reports comp0lied for it and the various stages. For one stage on the east side of Springs Road, an

early report designated the whole area as requiring TC2 foundations, to address both peat consolidation issues as well as some areas of higher liquefaction hazard. A later report by another consultant amended this to TC1 for most of the area with TC2 restricted to only 6% of the lots where proximity to natural springs or detention basins increased lateral spread hazard. Therefore, the current report is consistent in general conclusion with the work done on adjacent areas, which are on very similar ground conditions.

Lateral spread has not been assessed. This will need to be addressed at subdivision consent stage for land along all waterways, either natural or formed, and around stormwater detention ponds and the like.

Our conclusion is that the analysis and conclusions are probably appropriate, but that Coffey need to comment on lateral spread as a potential hazard..

5 Buildings

The report makes no comment on implications for buildings, foundations or infrastructure design.

6 RMA section 106 matters

The only natural hazards identified for the site are related to liquefaction, which is judged to be a low risk and thus can be mitigated by appropriate foundations, settlement over organic soils, which may be present in the area likely to be used for stormwater basins, slippage, which is applicable only along stream banks and mitigated by appropriate bank design.

Comment: it appears that there are no geotechnical hazards that prevent this site being used for use in terms of section 106.

7 Conclusion

The report concludes that the site is suitable for development subject to further investigation and design at subdivision consent stage. The bulk of the area is deemed to be equivalent TC1 with some small areas of TC2. No geotechnical hazards that prevent this site being used for use in terms of RMA section 106 have been identified. Although the overall soil model and conclusions appear to appropriate for the Plan Change area, we recommend that additional information is requested to bring the geotechnical investigation more in line with the MBIE Guidance and to enhance confidence in the conclusions.

a) research publicly available borehole information (Ecan well data base and NZ Geotechnical Database) to verify the deeper profile which is only inferred as shear wave velocity profile in parts of the site, and increase the number of locations where ground conditions are known, particularly along the northern side, and thus enhance confidence in the overall geotechnical model.

b) comment on lateral spread as a potential hazard

It is noted that further testing is essential at subdivision consent stage.

Yours faithfully
Geotech Consulting Limited

Ian McCahon

4415
22 February 2021

Selwyn District Council
PO Box 90
Rolleston

Attention: J. Lewes

Dear Ms Lewes,

RE: Plan Change 69
Rolleston Industrial Developments Ltd,
1491 Springs Road, Lincoln

Geotechnical Report Peer Review

1 Introduction

Geotech Consulting has been asked to carry out a peer review on the geotechnical report for the proposed Change to the Selwyn District Plan. The site is on the south side of Lincoln township and to the east and west of Springs Road. The Plan Change Request is to rezone the 186 ha area for residential use (Living LX & LZ) with a small zone for a local commercial centre (B1). The bulk of the area would be subdivided for a minimum of 12 households per hectare, with smaller areas of medium density (15 households per hectare) or large lots. The area could support about 2,000 new residential lots.

The report as provided as part of the plan change application was titled *Geotechnical Assessment Report, Rolleston Industrial Developments Ltd, 1491 Springs Road, Lincoln, Rev 1,* by Coffey Services (NZ) Ltd dated 20 October 2020, for Rolleston Industrial Developments Ltd. That report was reviewed (our letter dated 20 November 2020) and additional information was requested. Those aspects have been included in the Revision 2 report, dated 28 January 2021. The Revised report contains little change. The major additions are Appendix D (entirely new) with logs of 16 test pits on an area of proposed borrow, and Appendix E (also new) with logs of 18 tests from on or around the site from other sources. This additional data is referenced in section 2 (Scope), 4 (site investigation with new tables 2 & 3), 6.3, 7.3.2 and 8. There is also slightly expanded comment on the MASW data in 4.1. The more useful comments on the review questions are in the Coffey *Design Review Sheet*, appended to the revised report but not otherwise referenced, but which are included in clauses 130 – 139 of the Novagroup RFI response of 18 February 2021..

This letter is a review of the revised report, the Coffey comments in the Design Review Sheet and includes the relevant parts of the first report review. It supersedes our letter of 20 November 2020. The reference for the review is the MBIE guideline for the geotechnical assessment of subdivisions.

It is noted that Geotech Consulting has also peer reviewed geotechnical reporting for the large subdivisions which adjoin the plan change area along most of the length of the north side. Reference has been made to these reviews to check consistency in conclusions.

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2 Site Testing

The site testing has consisted of 20 cone penetration tests (CPTs) to between 2.2m and 12.0m (average 5.5m) depth, sixteen test pits to between 0.2m and 4.45m deep and three MASW geophysical investigation lines totaling 2.6 km. The report states that other data from around the site area has also been considered, and details of 12 CPT tests (1.7 to 8.3m deep) and 6 boreholes (8.8m to 30.2m deep) are included in Appendix E.

> *Comment:* The MBIE guidance suggests 0.2 to 0.5 deep tests per hectare at plan change stage to characterize the soil profile to a depth of at least 15m. Coffey claim that with the additional tests referenced from the NZGD and Ecan well database, the number of tests is now 54, but this includes the 16 test pits, six of which are less than 2m deep. However the number of deeper tests, combined with the MASW profiles does now meet the intent of the MBIE guidance for testing at plan change. Coffey note that the western part appears to be geologically consistent and the lower test density is considered adequate.

3 Subsurface Conditions

The tests show 0.3 – 0.4m of topsoil over interbedded silty sand, sandy silt and silt of variable thickness, on sandy gravel. The gravel is at a depth of 1.0 – 2.2m depth west of Springs Rd, and 3.5 – 5.5m depth east of Springs Road (Table 2 of the report). It is noted that the ground profile includes highly interfingered layering of silty and sand/gravel alluvium and the eastern edge potentially has organic soils. The MASW profiles are interpreted as confirming the CPT tests stopped on dense non-liquefiable soils with shear wave velocities greater than 200m/s. Groundwater was measured at between 0.5m and 3.4m in 17 of the 20 CPT holes, deepening to the west. A groundwater level of 1.0m has been assumed for the eastern part and 2.0 – 2.5m for the western part.

> *Comment:* The additional data in the new Appendix E does confirm the deeper soils are dense. The correlation of the MASW profiles with the stratigraphy inferred from the CPT tests and other boreholes will be checked at subdivision stage, when additional deep drilling is also intended (Coffey DRS). The poorer ground in the northeast is to be low density residential and stormwater management areas and this reduces the risks of building over peaty soils.
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> We note that the test pits in the proposed borrow area show a marked change in depth to gravel. Six tests on the west side contacted gravel at 0.4 – 0.9m depth, while 9 of the other 10 tests contacted gravel at between 2.1m and 4.3m. This is consistent with the description of interfingered alluvium, but does illustrate how quickly the shallow soil profile can change over short distances.
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> Three properties included in the plan change area are not covered in the geotechnical report. Coffey (DRS) state that the soil conditions are likely to be consistent with that of the surrounding land. 36 Collins Rd has 4 existing CPT tests on it, now incorporated into Appendix E. These show dense gravel(?) at 3.5m depth, consistent with the overall soil profile in this area. We concur with Coffey's expectation of similar soil conditions for these areas.

4 Liquefaction potential

A liquefaction analysis of the CPT data suggests index settlements of less than the TC1 limit of 15mm in all but one of the 19 tests analysed at SLS levels of shaking, and less than the 25mm limit in all but two tests at ULS levels of shaking. The analysis includes transition layer adjustment. The report considers that with refinement of ground water levels, the estimated settlements will be reduced, and concludes that the majority of the site is equivalent TC1 with some small areas of equivalent TC2 land

Comment: The analysis is by the MBIE standard procedure with appropriate input parameters. The use of a 1m water table depth for the eastern part is probably conservative. As no liquefaction outputs are provided, it is not known at what depths the liquefaction is predicted to occur. There is no discussion of evidence of ground damage in the 2010-11 earthquakes. It is noted that the site has certainly been well tested to in excess of SLS shaking and probably in excess of ULS shaking in the September 2010 earthquake, yet the closest residential land at the time of the earthquakes – further north with generally more sandy soils - was all classified Foundation Technical Category TC1 by MBIE, suggesting little to no ground damage.

The recent subdivisions adjacent to the north side also considered liquefaction. The land north of the subject land and west of Springs Road was concluded to be mostly TC1 with two small areas of equivalent TC2, similar to the conclusions in this report. The Te Whariki subdivision has had numerous reports comp0lied for it and the various stages. For one stage on the east side of Springs Road, an early report designated the whole area as requiring TC2 foundations, to address both peat consolidation issues as well as some areas of higher liquefaction hazard. A later report by another consultant amended this to TC1 for most of the area with TC2 restricted to only 6% of the lots where proximity to natural springs or detention basins increased lateral spread hazard. Therefore, the current report is consistent in general conclusion with the work done on adjacent areas, which are on very similar ground conditions.

Lateral spread of land along all waterways, either natural or formed, and around stormwater detention ponds and the like, has not been assessed, beyond a comment in the DRS which acknowledges that further assessment is needed once the subdivision planning is further advanced, but that the use of TC2 foundations may be sufficient mitigation. This will need to be addressed at subdivision consent stage

Our conclusion is that the analysis and conclusions are appropriate for plan change.

5 Buildings

The report makes no comment on implications for buildings, foundations or infrastructure design.

6 RMA section 106 matters

The only natural hazards identified for the site are related to liquefaction, which is judged to be a low risk and thus can be mitigated by appropriate foundations, settlement over organic soils, which may be present in the area likely to be used for stormwater basins, slippage, which is applicable only along stream banks and mitigated by appropriate bank design.

Comment: it appears that there are no geotechnical hazards that prevent this site being used for use in terms of section 106.

7 Conclusion

The report concludes that the site is suitable for development subject to further investigation and design at subdivision consent stage. The bulk of the area is deemed to be equivalent TC1 with some small areas of TC2.

No geotechnical hazards that prevent this site being used for use in terms of RMA section 106 have been identified. The overall soil model and conclusions appear to appropriate for the Plan Change area, and the additional information now included int the report enhances confidence in the conclusions.

It is noted that further testing is essential at subdivision consent stage.

Yours faithfully
Geotech Consulting Limited

JF McCahon

Ian McCahon