Tonkin & Taylor

1959 - 2009 50 YEARS

T&T Ref : 23782.004
20 August 2009

Westfield (New Zealand) Ltd
PO Box 109-280
Newmarket
Auckland 1149

Attention: David Drew

Dear David

Westfield St Lukes Private Plan Change 8: Further Information Request

We refer to your email correspondence dated 17 July 2009 together with the accompanying letter from URS New Zealand Ltd (URS) dated 16 July 2009 requesting further information in relation to the above Plan Change application.

In response to the request, we provide the following information relating to section 4.0 of the URS letter dealing with Infrastructure.

4.1 Stormwater

4.1 (i) Rockbore soakholes

Additional soakage testing was recently carried out at the site to provide more information with respect to the Plan Change application. Three new test holes (DH1, DH2 and DH3) were percussion drilled and soakage tested. The location of the test holes is shown on Drawing 23782-01 Rev 0 included in Appendix A on which the hole locations have been hand marked. The three holes were located in the low level of the disused quarry area on the north-east side of the existing Shopping Centre.

The testing was carried out by Niederer Drilling Auckland Ltd, and their results are included in Appendix A.

Test hole DH1 was initially drilled at a 10 degree inclined angle to a depth of 5.5 m when highly fractured rock was encountered. A soakage rate of 17.5 l/s was recorded in this material. Because of hole collapse, the test hole was redrilled vertically to a depth of 14 m at which depth the drilling was terminated due to collapses within the fractured rock. Casing down to 6 m was installed through the upper fractured material. Fractured rock was encountered throughout the depth of the drill hole. Soakage testing of the hole below the

6 m casing was carried out using a fire hydrant and initially one water truck and then two. A flow rate of 47.8 l/s was recorded which was not the capacity of the hole but that of the hydrant and water trucks. The soakage capacity of the test hole is therefore at least this value.

Test hole DH2 was initially drilled at a 10 degree inclined angle to a depth of 9 m when rock fractures and cavities were encountered. Owing to collapses the hole was redrilled vertically to a depth of 20 m, with casing installed for the first 6 m. A soakage rate of 58 l/s was recorded for the lower zones of the hole which, again, was not the capacity of the hole but that of the fire hydrant and water trucks used for the test. The soakage capacity of the test hole is therefore at least this value.

Test hole DH3 was drilled vertically to a depth of 16.5 m (cased for the first 4.2 m) with fewer fractures encountered along its depth. A soakage rate of 10 l/s was recorded for the lower zone of the hole. The soakage capacity of the test hole is therefore taken to be 10 l/s.

The favourable soakage results obtained for drill holes DH1 and DH2 support the expectation described in our letter report dated 4 March 2009 that good soakage yield can be expected when permeable zones and joints in the rock are intercepted.

The additional soakage capacity required to be provided for the proposed expansion to meet 1 in 10 year design soakage is 460 l/s. It is possible that this soakage rate could be provided by 4 to 6 soakholes if conditions similar to those recorded at test holes DH1 and DH2 are encountered and if two drill holes are constructed for each soakhole.

Proposals

Given the favourable soakage results obtained, the preferred option is to provide soakage with a 1 in 100 year capacity without the need for detention storage. Other options proposed below comprise the provision of soakage between 1 in 10 year and 1 in 100 year capacity together with detention storage. All the options proposed are designed to manage a 1 in 100 year storm event.

- Option 1 Provide 1 in 100 year soakage capacity without any detention

  For this option, the effective additional soakage capacity required is 1350 l/s assuming that the catchment areas beyond the site have an effective 1 in 10 year soakage capacity. Allowing for an effective soakage rate during a critical storm of 80% of installed soakage capacity gives a required installed capacity of 1700 l/s. This could be provided by 15 to 20 soakholes if conditions similar to those recorded at test holes DH1 and DH2 are encountered and two drill holes are constructed for each soakhole.

- Option 2 Provide 1.5 times 1 in 10 year soakage capacity and detention by surface ponding

  For this option, the additional soakage capacity required to be installed is 700 l/s assuming that the catchment areas beyond the site have an effective 1 in 10 year soakage capacity. This option also allows for an 80% effective soakage rate. The

700 l/s soakage could be provided by 6 to 9 soakholes if conditions similar to those at test holes DH1 and DH2 are encountered and two drill holes are constructed for each soakhole.

From hydraulic modelling, the detention volume required is about 680 m^3 (see attached spreadsheet in Appendix C). It is proposed that this volume be provided by temporary ponding over a car park surface. We understand that the car park on the east side of the proposed expansion (see the marked up Development Plan drawing included in Appendix B) is likely to be a multi level car park extending down into the low area of the disused quarry. Ponding is proposed over the lowest level of this car park. Allowing for an available car park area of about 4500 m^2, a water depth of 0.15 m would be created, which is well below the maximum depth of 0.3m stated in overland flow hazard criteria.

- Option 3 Provide 1.5 times 1 in 10 year soakage capacity and detention by underground or above-surface stormwater chambers

  This option is as for Option 2, but the detention volume is provided by a tank or chamber beneath or on the surface of the lowest car park level. As stated in our letter report dated 4 March 2009, this detention could be achieved by constructing the lowest car park level as a suspended floor above the surface of the disused quarry or by installing one of several detention storage options structures available. This includes open bottomed stormwater chambers. The benefit of the latter is that, given their considerable area, stormwater discharge through the bottom of the chambers can be substantial which could significantly reduce the required soakage via soakholes.

In the event that the soakage capacities allowed for in the above options are not confirmed during the detailed design stage, greater detention volumes can be provided to supplement lower soakage capacities. Based on the proven results of the recent testing, this is not expected to be required.

4.1 (ii) Overland flows

Further to our letter report dated 4 March 2009, it is proposed that overland flow from catchments 1 - 3 (the expansion and residential area, area east of the Shopping Centre, and the northern car park respectively) be discharged into the former quarry area. Arrows indicating these overland flows are shown on the attached marked up Development Plan. As stated in the letter report, overland flow from catchment 4 (existing Shopping Centre main car parking area) runs southward towards and is managed within the area of the existing Shopping Centre. This overland flow will not be affected by the proposed expansion.

Preliminary design flow, runoff volume and storage volume calculations relating to the four catchment areas for 10, 50 and 100 year rainfall events are summarised in the spreadsheet and catchment area plan included in Appendix C. The ponding/detained volume results supersede those given in our letter report dated 4 March 2009 which were too high because they did not make adequate allowance for available soakage within the eastern catchment area. The updated results for the 1 in 100 year event are summarised in Table 1 below. The results for the 1 in 100 year event are summarised in Table 1 below. The analysis in each case assumes there is effective 1 in 10 year soakage within the catchment east of the site.

Table 1: Estimated 1 in 100 year overland flow ponding / detention volumes

Catchment	Temporary ponding/detained volumes (m^3)			
	Assuming 1 in 10 year effective soakage capacity within the Expansion	Assuming 50% of 1 in 10 year effective soakage capacity within the Expansion	Assuming 80% of 1 in 10 year effective soakage capacity within Expansion	Assuming 80% of 1.5 x 1 in 10 year effective soakage capacity within the Expansion
Catchments 1, 2 and 3 combined	1220 m^3	1990 m^3	1450 m^3	680 m^3

Allowing for regular and satisfactory maintenance of the soakholes, we consider that the provision of 80% effective soakage capacity is appropriate for the analysis.

As described in 4.1(i), the preferred option (Option 1) is the provision of sufficient soakage capacity that avoids the need for detention storage. Drilling during the detailed design stage would endeavour to confirm sufficient soakage capacity. If this is not found, then Options 2 and 3 would be proposed which require a ponding or detention volume of 680 m^3. In this case, the preferred solution is to allow this to pond with limited depth over the lowest car park. If subsurface storage is required, it is envisaged that it could be located below the proposed multi level car park on the east side of the expansion and shown on the marked up Development Plan. It could take the form of open bottomed stormwater chambers like the StormTech subsurface stormwater management system illustrated in the picture included in Appendix D. Alternatively it could take the form of a detention tank constructed on the surface of the lowest car park.

Whatever storage facility system is selected if required, it would include access and inspection manholes, vents if necessary, and would be such as to facilitate cleaning of the chambers and various elements using a vacuum truck and manual cleaning if necessary.

All three above overland flow management options are considered feasible. The inlet and outlet arrangements of Options 2 and 3 are indicated in the attached sketch nos. 1/1 Rev A and 1/2 Rev A included in Appendix D. The selected option will be confirmed during the detailed design phase when more detailed soakage testing will be carried out and the selected solution is finalised.

4.2 Wastewater

4.2 (i) Overflow point

As mentioned in our letter report dated 4 March 2009, we understand from Watercare that the nearest location of overflow from their combined trunk sewer is at Lyon Avenue from where the overflow enters Meola Creek. Other overflow points exist downstream of this location.

Overflow at Lyon Avenue from the Watercare sewer can be expected to contain wastewater originating from the proposed expansion. However, because of the extreme mixing and dilution that would take place within the trunk sewer flowing full, it can be expected that a significant portion of the wastewater would be conveyed downstream as part of the main flow contained within the sewer.

4.2 (ii) Overflow frequency and magnitude

Information published by Watercare in their Detailed Annual Sustainable Development Report, 2001, indicates an existing frequency of overflow at the Lyon Avenue location of 42 incidents per year. The maximum annual spilled flow indicated in the report is 3.42 m^3/s and an annual spilled volume of 111,000 m^3. We understand from Watercare that the flow capacity of the trunk sewer at this location is 7 - 8 m^3/s.

No increase in overflow frequency from the sewer is expected when taking the discharge from the proposed expansion into account. The estimated 7.0 l/s increase in peak wastewater flow from the proposed expansion is not more than 0.1% of the capacity of the trunk sewer. This would result in negligible increase in water height within the sewer and thus negligible risk of additional spillover at the Lyon Avenue sewer overflow weir.

We understand that this is the basis for Watercare's comment in their letter dated 11 March 2009, a copy of which is included in Appendix E, that the additional wastewater flow of 7.0 l/s would not exacerbate sewer overflows to Meola Creek.

4.2.(iii) Environmental effects

Because no additional overflow is expected from the Watercare sewer on account of the proposed expansion, no environmental effects associated with increased overflow frequency are therefore expected on the receiving waters.

Watercare's letter also lists measures being taken by Watercare to mitigate the effects of wastewater overflows. These include the approval of funds to proceed with the preliminary design of the Central Interceptor which will reduce wastewater overflows into the Meola Creek. A design contract has recently been placed by Watercare to proceed with the design process for the Interceptor.

Applicability

This report has been prepared for the benefit of Westfield (New Zealand) Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

TONKIN & TAYLOR LTD

Environmental and Engineering Consultants

Report prepared by: Authorised for Tonkin & Taylor by:

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Senior Civil Engineer Project Director

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