Fluent Infrastructure | Experience

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Ref: GL-260922ACS-1015 RevA

7 October 2026

Via Email: [redacted]
Taylor Planning

Attention: Louise Taylor

Dear Louise

Remarkables Skifield Fast Track Application

1.0 Reviewer: Water and Wastewater

My full name is Anthony Charles Steel.

I have a Bachelor of Civil Engineering from the University of Canterbury. I am a Chartered Professional Engineer and Chartered Member of Engineering New Zealand.

I have over 40 years' experience in the civil and infrastructure engineering field including 3Waters.

I am a senior civil engineer, project manager and Director of Fluent Infrastructure Solutions Ltd (Fluent) since its inception in March 2013.

I have worked on numerous three waters projects in Otago and throughout New Zealand and SE Asia. These works have included water supply intakes, treatment plants, pumping stations, pipelines and reservoirs, wastewater pipelines and pumping stations and stormwater conveyance systems such as channels, detention basins, culverts and pipelines. A significant proportion of by works have been on public three waters infrastructure but I have also been engaged by private developers to design and implement their three waters infrastructure.

Prior to working with Fluent Solutions I worked for Duffill Watts & King Ltd and its subsequent name changes for 20 years.

1.1 Statement of Compliance with Environment Court's Code of Conduct for Expert Witnesses

In the context of this application, which is made under the Fast-track Approvals Act 2024, and in relation to which there may be no hearing, I have been asked to confirm that this review has been prepared in accordance with the Environment Court's Code of Conduct for Expert Witnesses.

I confirm that I have read the Environment Court's Code of Conduct for Expert Witnesses, as contained in Section 9 of the Environment Court's Practice Note 2023 and agree to comply with it.

The data, information, facts and assumptions that I have considered in forming my opinions are set out in my advice. The reasons for the opinions expressed are also set out in the advice.

I confirm that the water and wastewater matters addressed in the technical report/spreadsheet are within my area of expertise, with the exception of where I confirm that I am relying on information provided by another person.

I have not omitted to consider material facts known to me that might alter or detract from my opinions expressed. I have specified where my opinion is based on limited or partial information, and I have identified any assumptions I have made in forming my opinion.

2.0 Reviewer: Stormwater

My full name is Gerhard Wilhelm Fourie.

I have a Bachelor of Civil Engineering from the University of Stellenbosch in South Africa. I am a Chartered Professional Engineer and Chartered Member of Engineering New Zealand.

I have over 20 years' experience in the civil and infrastructure engineering field including 3Waters.

I have been a senior civil engineer at Fluent Infrastructure Solutions Ltd (Fluent) since February 2024. Prior to working with Fluent Solutions I worked for Holmes ANZ in Queenstown for 4 years, and Burmeister & Partners in Windhoek, Namibia, for 13 years.

I have worked on numerous three waters projects in Otago and throughout New Zealand and Namibia. I have consultancy experience within the three waters / utilities / civil infrastructure engineering fields and have a diverse knowledge of three waters design and modelling.

I can confirm that my current focus area is surface water hydraulics and hydrology. I have worked on hydrological and hydraulic assessments for several bridge replacement projects post cyclone Gabrielle and have recently been involved with flood assessments and modelling for the Owhiro Stream in Mosgiel and Woolshed Creek in Hanley's Farm. I also have worked with several private clients and developers on stormwater management plans and flood assessments for their developments in the Otago region for resource consent and engineering approval.

2.1 Statement of Compliance with Environment Court's Code of Conduct for Expert Witnesses

In the context of this application, which is made under the Fast-track Approvals Act 2024, and in relation to which there may be no hearing, I have been asked to confirm that this review has been prepared in accordance with the Environment Court's Code of Conduct for Expert Witnesses.

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The data, information, facts and assumptions that I have considered in forming my opinions are set out in my advice. The reasons for the opinions expressed are also set out in the advice.

I confirm that the stormwater matters addressed in the technical report/spreadsheet are within my area of expertise, with the exception of where I confirm that I am relying on information provided by another person.

I have not omitted to consider material facts known to me that might alter or detract from my opinions expressed. I have specified where my opinion is based on limited or partial information, and I have identified any assumptions I have made in forming my opinion.

3.0 Conclusion

Fluent Solutions has reviewed the three waters aspects of the fast track application. On behalf of Central Otago District Council, we have formed a list of questions that have been responded to by the applicant and we are now in a position where we are comfortable with the responses. The remaining items in the correspondence that are still open can be addressed during a detailed design phase. If you have any questions please contact Anthony Steel on cell phone 021 838 661.

We have attached a copy of the latest spreadsheet with this letter for your information.

Yours faithfully
FLUENT INFRASTRUCTURE SOLUTIONS LTD
Per:





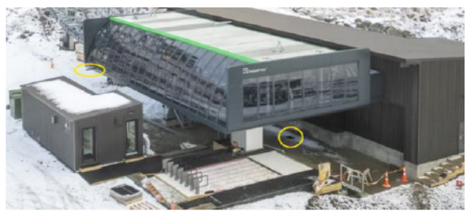
Anthony Steel
Director | Principal Civil-Water Engineer
BE (Civil), CPEng (1167007), CMEngNZ

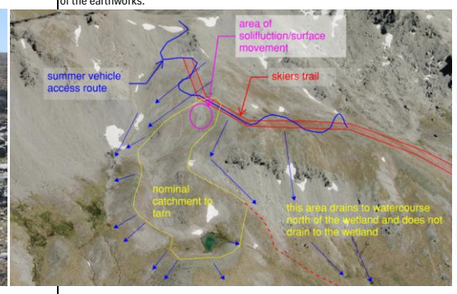
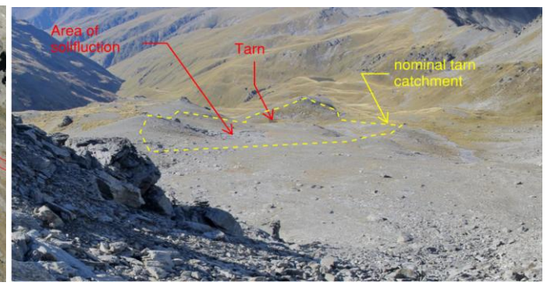
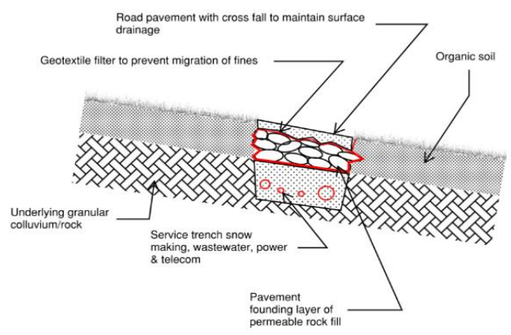
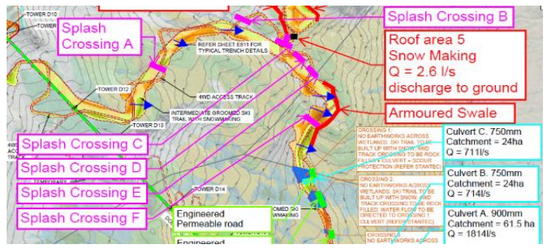
Gerhard Fourie
Senior Water Engineer
B.Eng (Civil), CPEng (1169203), CMEngNZ

Enclosure:

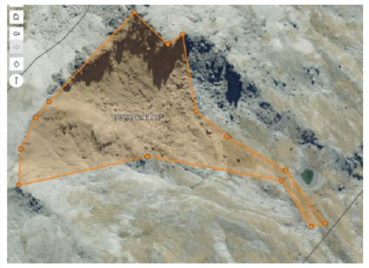
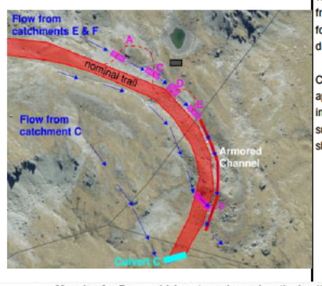
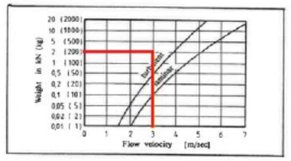
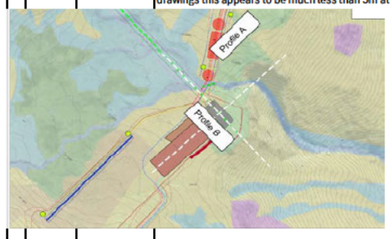
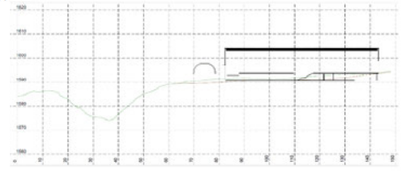
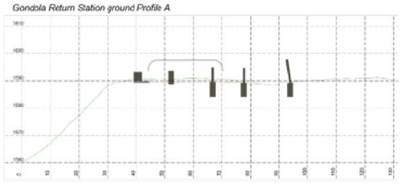
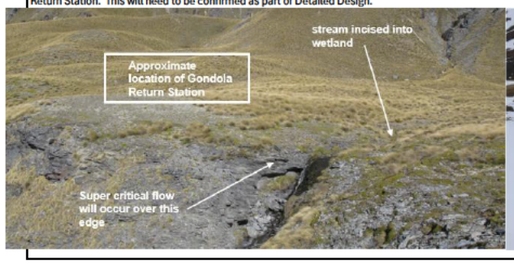
- Peer Review Spreadsheet

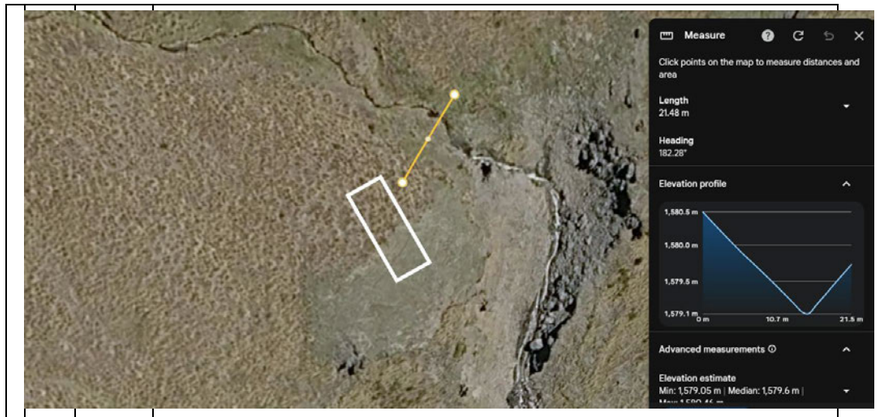
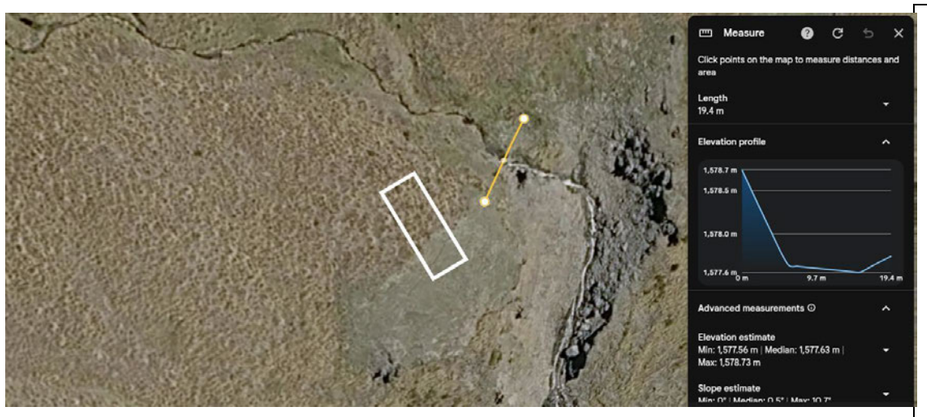
Project Number: 1015
Project Name: Peer Review - Remarkables Ski field Doolans Basin Expansion - Three Waters
Peer Review By: GF / ACS / EJ
Checked By: ACS
Date: 26/08/2026

A	B-16 Remarkables Ski Area Expansion Project Stormwater Concept Report Redacted				
ID	Page No	Section	Peer Review Comment	Response to Peer Review Comment	Peer Reviewer Comments #2 - 26 August 2026
1	3	3	Report states: *"The stormwater measures proposed are intended to minimise concentration and channelization of stormwater and to mimic existing stormwater flow routes. This includes promotion of overland sheet flow and maintaining existing stormwater channels and flow paths. Thus, following construction it is not anticipated that specific stormwater treatment facilities will be required. "* The introduction of roads and buildings will increase the peak runoff generated. This inevitable increase in runoff is not addressed in the report, neither is mitigation to manage increase in peak runoff. We recommend justifying why mitigation is not considered, or why this is considered to not have any adverse effect. If the existing flow paths and channels are to be maintained, this increase in runoff (volume and flow) will cause additional erosion. We propose that the above statement be carefully considered, or justified.	The Doolans Creek Right Branch basin is similar in both topography and geology to the adjacent developed Rastus Burn Basin, which is shown in figures 7,8, 10, 11,24, 25 and 26 in the stormwater concept report. The concept report describes the satisfactory stormwater performance of the Rastus Burn formed trails, constructed as part of the development of Sugar Bowl basin, Shadow Basin and Curvey basin. The trails in these areas are constructed within existing scree slopes and the philosophy employed in these areas has been to maintain the general route of the natural flow paths. Existing streams, ephemeral water courses and natural overland sheet flow have been maintained without significant artificial concentration of stormwater. This approach has been successful and no significant areas of erosion related to development have been identified. Diverting and concentrating natural sheet flow, ephemeral water courses and small streams, would involve a greater level of modification of the landscape to form channels and to armour these channels than maintaining existing flow paths. The increase in surface flow resulting from the formation of the trails in the upper part of the Doolans Creek Right Branch Catchment, above culvert C, is not substantial. The total basin area occupied by new trails is approximately 40,000m2 and the total catchment area is approximately 1km2. Thus the gravel surface trail area within the upper scree slopes is approximately 4% of the catchment area, and the permeability of the trail surface is not significantly different to the natural ground, because the trails are natural granular material formed from the surrounding material. Therefore pre vs post development stormwater flow will not be significantly different. The primary difference between the trails and the surrounding country material is that the trails form a bench which was not previously present, and the loose granular surface scree has been removed. Thus the rate of seepage of water though the pavement materials is not significantly different to the rate of seepage through the surrounding natural material. Capturing and concentrating surface water into side water channels and directing this to existing watercourses would alter the route of natural flow, which we wish to maintain in a practical manner. If stormwater from the trails were to be captured and concentrated into channels the net result would be to increase the quantity of stormwater in the downstream creeks and a reduction of the time to concentration. Maintaining sheet flow and minimising channelisation will maintain existing stormwater infiltration and flow times as much as is practical and mimic the existing flow regime. The existing scree slopes have become self armouring as the finer fractions have been washed from between the larger particles. This process also occurs as the access track pavements are constructed and the initial rainfall events will mobilise finer sediment from the surface. Thus the initial erosion and sediment control plan, which by necessity will be based on the construction sequence, will need to provide provision to capture the finer particles mobilised by initial rainfall, by settlement and/or filtration. Below culvert C, the access track traverses tussock slopes which are less steep than the scree slopes above. In this location there are a number of wetland and saturated areas. A permeable access track formation detail is proposed in order to allow natural migration of water within the surficial material. Similarly, along the access road between the wetland areas and the Doolans return station facilities, the summer surface water flow will be allowed to flow across the access track to maintain the existing flow regime. By necessity, this track formation will require granular pavement and will become self armoured after the initial erosion of the surface fines. During, and immediately following, construction this initial surface erosion will be managed by the erosion and sediment control plan.	Noted, and thank you for the well summarised concept for the stormwater management. This clarifies explanation clarifies and justifies the SW management approach.
				The roof area of the secondary buildings, the wastewater pump station sheds, the snow making pump station building and the raw water intake pump building, is insignificant compared to the overall catchment and discharge of roof water from secondary buildings will be to ground. This is the methodology used for virtually all alpine buildings in New Zealand. Examples of such buildings are shown in figure 32 and include Brewster, Whare Kea, McKinnon Pass, Centennial, Plateau and Empress huts. Stormwater disposal to ground from the roofs of these buildings has been without issue. The net increase in runoff resulting from the presence of these secondary buildings is insignificant compared to the size of the catchment. Stormwater flow from the Café/Carrier stacking building is greater than that from the secondary buildings as the building is larger in area. Concentrated stormwater discharge from this building may lead to erosion or nuisance. Thus stormwater flow from this building will be captured in surface sumps adjacent to the downpipes and piped directly to the adjacent creek in order to minimise nuisance and potential erosion. This is described in section 6.6.6 of the concept report. The surface sumps are also useful for draining spring melt to minimise standing melt water and resulting nuisance. As described in section 6.1.7 for the gondola return station, the operation of the carriers and the need for groomer access around the carrier path, negates the practical use of down pipes. Thus stormwater from the gondola return station will be allowed to fall from the machine deck level. This is shown in figure 33 of the concept report. This is the standard method by which virtually all similar gondola stations and chairlifts are drained. The ground will be armoured in these locations and stormwater collected in surface sumps adjacent to the discharge points and piped directly to the adjacent stream. Stormwater flow from both the gondola return station and the Café/carrier stacking building will increase the flow in the adjacent creek, but this increase is so small that it will not have a detectable effect compared to the overall catchment size. For short duration high intensity rainfall events the roof flow from these buildings will occur before flow from the upper section of the catchment arrives and thus there will be no net increase in flow. For longer duration lower intensity events the flow from the end of the catchment can coincide with flow from roofs of these buildings. In this situation the rainfall will be of a lower intensity over a longer time and no significant increase in flow resulting from the roof is likely to occur. The carrier stacking/Cafe roof area is approximately 1300m2 where as the catchment is approximately 546,000m2. The roof is approximately 0.25% of this total catchment area	
2	10	5.1	Provide concept design of how concentrated flow through shallow diagonal swales are to be re-integrated as sheet flow on the downstream side of the roads to mimic natural runoff. Will there be dispersals systems to avoid concentrated eroded channels forming? How will sheet flow be promoted, and downstream erosion mitigated?	This is a detail dependent on the specific nature of the location and will appropriately be developed on site as the channels are formed. In locations where the local shallow swale discharges to a shallow apron, limited work to promote flow dispersal or erosion will be required. In areas where the discharge is into steeper slopes specific armouring to promote both flow dispersal and to protect against erosion will be required. This will generally be achieved using a rip rap apron laid over geofabric. In many locations there are existing ephemeral water courses with appropriate existing natural armouring. An example of such a location is shown in the photograph below. Unfortunately due to the time of year the area is snow covered and we are not able to get a good photograph of a specific example. 	Noted. Erosion protection downstream of shallow diagonal swales shall be develop on a case by case basis during detailed design and during construction. Where required rip rap armouring and geofabric underlay should be appropriately sized for the runoff and velocities expected.
3	10	5.1	Clarify a popped system for the roof runoff from the Doolans Gondola Return Station and Carrier Building, as no further information is provided on this in the document.	The carriers will traverse around the machine deck in a similar manner to existing chairlift or 'Chondola' installations. A picture of a typical return station is shown below. Drainage from the roof structure is into a recessed gutter with cantilevered down pipes which discharge outside the footprint of the gondola path. Pictures of the cantilevered down pipes are shown in figure 33 of the stormwater concept report. The right photographs below shows the Shadow basin base station with the cantilevered down pipe circled and one of the surface water sumps beneath the building circled  	Noted, no additional comments.

4	11	5.1	Clarify how runoff from Gondola Mid Station and Gondola Return Station Roofs will be managed once discharged to ground? The document notes infiltration and surface flow, but no concept of how this will be managed as overland or secondary flow?	Please see the description above and the relevant section from the stormwater concept report below. Should the sumps become blocked the secondary flow path is over the ground surface to the creek as the ground falls to the creek. We note that during spring melt the ground surface will be covered by melting snow and the melt path will be generally via percolation through the melting snow to the creek. **6.1.7 Roof 4: Gondola Return Station** Similar to the Gondola Mid Station, runoff from the Gondola Return Station will be discharged from pipes cantilevered from the spouting to ground a safe distance from the carriers. The ground will be armoured with rip rap at the points of discharge and will drain to surface sumps from where it will be piped to a discharge adjacent to the nearby unnamed creek.	Noted, primary drainage and secondary flow paths details should be confirmed during detailed design for compliance with the appropriate guidelines.
5	10/11	5.2.2	The statement "*These identified cushion field and bog locations are consequently not at risk of either sediment or stormwater flow changes resulting from the trail and access road* ." is inaccurate. These wetlands are located upslope of the lower portion of trail shown on Figure 15, but there is a tarn located above what is defined as Catchment B in Figure 18. This tarn discharges to catchment B and has a catchment area that likely includes portions of the access road and trail. It is recommended that this be reconsidered in the report.   	We do not believe that detectable or significant levels of sediment originating from the trail will reach the wetlands. Only a small part of the proposed trail is within the catchment to the tarn as shown in the diagram below. This catchment area includes a significant natural area of solifluction (shown in the photographs below, person circled for scale) which is expected to generate significantly more sediment than the relatively small section of trail within the catchment. We have visited the tarn on a number of locations and we confirm that the tarn acts as a natural settlement pond for existing naturally occurring sediment and that flow from the tarn is effectively decanted out from the basin formed by the surrounding moraine. Thus we do not anticipate that any significant volume of additional sediment will be generated from the small area of trail within the catchment. The surface of the proposed trail is not significantly different to the surrounding scree. The tarn acts as a settling pond for the naturally occurring flow before the flow from the tarn is decanted to the ephemeral watercourse to the east. The risk of sediment generation during the construction of the trail will be managed by the erosion and sediment control plan. We anticipate that initial runoff from the trail formation will be captured, and directed to filtration or temporary sedimentation ponds along the route of the earthworks.	Noted. The bigger risk for erosion and sediment production appears to be during the construction phase, and careful and deliberate protection through well planned and approved erosion and sediment control would effectively manage this risk.
6	14	5.2.2	With reference to point 3 above, there is potential for increase in flow and sediment to the tarn from the associated upper catchment down to the southern wetland in "catchment B". The design of the trail, access road and temporary access track all cross through this upper catchment. The detailed design in this area should carefully consider that runoff from outside of this catchment is not inadvertently redirected into this catchment, which could occur along the steep trail and road sections even with crossfall, as this will impact on the wetland hydrology.	Please see the above description of the site. Detailed design will not direct any appreciable additional water to the existing catchment and wetland. This is one of the reasons to maintain existing flow paths within the area. Additionally the erosion and sediment control plan will address sediment generation during and immediately after construction.	
7	15	Figure 17	The lowest label on the sketch is cut off which describes the pavement formation.	Below is a complete version 	Noted, no additional comments.
8	16	5.3	We acknowledge that the NZBC E1/VM1 suggest the use for the Rational Method for catchments up to 100ha. The NZS4404:2010 limits this are to 50 ha, CCC limits it to 5 ha for hillside catchments, and QLDC to 10 ha. For this application we acknowledge that only one of the catchment areas (A) is larger than 50ha, and that the rational method would be appropriate if careful consideration is applied to the use of the runoff coefficient.	We agree.	No additional comment
9	17/19	Figure 18 and Table 3	Consider adjusting catchment areas for catchments A & B as discussed in Point 3 above.	Yes figure 18 is intended to be diagrammatic. The proposed works plans to maintain the existing flow regime and catchment areas.	No additional comment
10	18	Table 2	There is no figure or drawing relating the location of culverts and splash crossing to the table. The drawings in the Appendices does show location of culverts, but the naming conventions are not the same, and no splash crossing locations are shown on the drawings, only permeable road formations. Update drawings or include relevant appendices to reflect crossing locations / culverts etc.	Splash crossing locations as per appendix A as below. 	Noted, and thank you for the clarification.
11	18 & 19	5.3.1	There are two sections numbered 5.3.1	Acknowledged but the material nature of the report is not affected	No additional comment

12	19	5.3.1	A 50-year ARI (2% AEP) was selected for a design return period for culvert sizing. In section C4.3.2.4 of NZS 4404:2004 it is noted that roads should be passable to 4WD vehicles in the 1% AEP event. Would these culverts be passable as emergency egress under 1% AEP events? **Table 2.1:** Importance level and annual probabilities of exceedance for wind, snow, floodwater and earthquake actions for bridges Bridge categorisation: Bridges where failure would not be likely to endanger human life and the loss of which would not be detrimental to post-disaster recovery activities for an extended period. Importance level (as per AS/NZS 1170.0[NP]): 1 Bridge permanence*: Permanent; Temporary Annual probability of exceedance for the ultimate limit state: ULS for wind, snow and floodwater actions: Permanent 1/250, Temporary 1/50; DCL3* for earthquake actions: Permanent 1/250, Temporary 1/50 Annual probability of exceedance for the serviceability limit state: SLS 1 for wind, snow and floodwater actions: Permanent 1/25, Temporary -; SLS 2 for floodwater actions: Permanent 1/25, Temporary - b. Overall design of total waterway In the design of a waterway crossing, the total waterway shall be designed to pass an SLS 2 flood given in table 2.1 without significant damage to the road, the protection works and the waterway structure. Damage from a SLS 2 flood shall be minor, with no more than minimal disruption to traffic (eg temporary speed restrictions and temporary lane closures to facilitate repairs such as the reinstatement of rock rip rap) and be repairable within a period of one month.	This is an alpine environment and the proposed expansion is not one for which NZS 4404 is directly applicable. Emergency access in a 1% AEP event through this terrain is considered to be impracticaland potentially dangerous. We would not recommend trying to move in such an environment in such conditions, nor would NZSki's operational practices allow this. This is a steep alpine environment and navigation in such conditions would be inadvisable and crossing the splash crossings would be unsafe. Thus 2% AEP (50 year) is considered more than suitable, and some reviewers may consider this figure unnecessarily conservative. A more applicable approach is that given in the NZTA bridge manual. As noted this is an alpine environment, and during winter access will only be possible via the gondola, helicopter (should weather conditions allow) or by snow groomer/snow mobile. During summer the ski area will not be in operation and thus guests will not be present and staff will be present for maintenance activities or summer construction projects. In a 100 year ARI event, travel in this terrain is effectively impossible and would be perilous. Stream splash crossings will be impassible and in such conditions NZSki's operational safety procedures will not allow work within the basins. We also anticipate that in such a storm the wind speed would make movment unsafe or impossible. NZTA's bridge manual recommends a SLS 2 event with a 25 year return period and acceptance of some damage as below. Thus 2% AEP represents a conservative approach. The Mountain Safety Council recommends that back country users sit out such events and wait for rivers to drop before attempting to move in such alpine environments. https://www.mountainsafety.org.nz/learn/skills/river-safety	Noted, and thank you for the pragmatic clarification. These are practical approaches and sensible to this environment. From a consenting perspective deviation from the code of practice for subdivision will need to be clearly justified and documented in the detailed design documentation, and included as such in producer statements.
13	19	5.3.1	The runoff coefficient appears generally appropriate for the terrain of gravels, scree and tussock. However, there will be periods of ground frost where higher runoff coefficients can be expected. Additionally there could be large areas of bedrock below shallow gravels and scree that may temporarily delay runoff but not necessarily reduce intensity. We recommend consider the use of a sensitivity check using a higher runoff coefficient under these conditions to understand the implications on the design of the culverts and splash crossings.	Noted, but for the purposes of detailed design in this environment a conservative approach is appropriate rather than a sensitivity analysis. As there are only three culvert crossings on the access road between helicopter ridge and the Doolans base facilities a conservative design approach has minimal impact on overall cost. We propose that appropriately conservative sizing of the culverts and armouring is taken.	Noted, no additional comments.
14	19 & 20	Table 3 and Table 4	A runoff coefficient of 0.25 was selected and it is noted that this was increased by 0.1 as a slope correction factor. However the C - values in the tables are still 0.25, not 0.35? Please clarify.	noted. The value of the runoff coefficient will be specifically tabulated in the design attributes statement which will be prepared as part of detailed design. The 0.1 has not been added to 0.25 and this is an error in th econcept design report. Nevertheless culvert sizing at concept phase has been conservative and in detailed design appropriate runoff coefficients will be uses with allowance for slope.. We anticipate that there will not be significant change during detailed design. Note recommendations below further confirmation of the culvert diameter is required, but no significant changes are anticipated **Table 1: Run-off Coefficients** Paragraphs 2.6.1, 2.1.1, 2.1.3 Description of surface / C **Natural surface types** Bare impermeable clay with no interception channels or run-off control 0.70 Bare uncultivated soil of medium soakage 0.60 Heavy clay soil types: – pasture and grass cover 0.40 – bush and scrub cover 0.35 – cultivated 0.30 Amend 10 Jan 2017 Medium soakage soil types: – pasture and grass cover 0.30 – bush and scrub cover 0.25 – cultivated 0.20 Amend 10 Jan 2017 High soakage gravel, sandy and volcanic soil types: – pasture and grass cover 0.20 – bush and scrub cover 0.15 – cultivated 0.10 Parks, playgrounds and reserves: – mainly grassed 0.30 – predominantly bush 0.25 Gardens, lawns, etc. 0.25 **Table 2: Slope Correction for Run-off Coefficients** Paragraph 2.1.3 Ground slope / Adjust C by: 0-5% subtracting 0.05 5-10% no adjustment 10-20% adding 0.05 20% or steeper adding 0.10	Noted that this will be re-evaluated during detailed design.
15	19 & 20	Table 3 and Table 4	We recommend including the HIRDS V4 dataset used for this project in the Appendices. Without the information it is not possible to review correlation of intensity to storm to duration. We also note that the intensity remains consistent for varying durations, which typically require interpolation using the NIWA parameters. For example Table 3 has the following intensities for different durations (RCP4.5): - TOC 40 mins - 37 mm/hr - TOC 30 mins - 37 mm/hr - TOC 20 mins - 40 mm/hr - TOC 25 mins - 42.5 mm/hr This are inconsistent with what would be expected from an IDF curve.	Noted. Yes we agree with this this and a tabulated list of time to concentration vs intensity and area will form part of the detailed design and will be tabulated in the design attributes statement.	Noted, no additional comments.
16	19 & 20	Table 3 and Table 4	Clarify how the TOC was determined, and providing some evidence of establishing these numbers as this plays a crucial role in understanding the design intensity, to which the Rational Method is very sensitive.	Only three culvert crossings are proposed within Doolans Creek Right Branch basin between the Gondola return station and helicopter ridge. These are Culverts A, B & C. In these locations there is either permanent flow or a creek bed sufficiently incised that splash crossings would be difficult to implement. The times to concentration generally calculated in accordance with NZBC E1/VM1. this is a preliminary calculation for the purpose of concept development. For example for culvert C adjacent to the gondola return station the flow length is 900m . The fall is 100m (1720m to approximately 1820m). Thus the slope is 11%. The time to concentration identified in table 3 is 30 minutes. For the purpose of concept development this approach is sufficient. This is calculated as follows. **Figure 1: Times for Overland Flow** Paragraph 2.3.2 b) i) Bare Earth and Poor Pasture Pasture, Cultivated Land and Rock Fill Grassed Areas, Gravel Areas and Lawns Average Surface Roughness 0.8 0.4 0.2 0.1 0.05 60 50 40 30 20 10 5 4 3 2 1 5 10 20 50 100 200 500 1000 TIME OF TRAVEL OVER SURFACE - minutes LENGTH OF OVERLAND FLOW - metres	Noted, and the TOC will be formalised in more detail during detailed design.
17	20	Table 5	Justify what design guidance has been used or will be used during detail design to establish the culvert scour protection.	We suggest the following consent condtiions for the stormwater final design as follows; •HIRDS V4 design storm RCP 6 (Note this is a conservative approach, but as there are three culverts it is not significantly cost sensitive and thus this provides confidence in the solution) •ARI for design of culverts and crossings is 50 years •Primary design guidance that will be used for scour protection is Austroads guide to road design part 5b drainage - open channels, culverts and floodway's, •Methodogy for design as to be as per NZBC E1 Surface Water VM1	Noted, no additional comments.
18	21	Figure 19 & 20	Justification of what design standards are proposed to be used for the culverts as they are proposed to be designed without headwalls and standard aprons. Also justify what design standard will be applied for energy mitigation and erosion protection.	Specific scour protection design has not been undertaken at this concept stage. Scour protection will be designed in accordance with Austroads Guide to Road Design part 5B drainage - Open channels, culverts and floodway's. This methodology is the standard method for the design of such structures and is referenced in the NZTA Bride Manual.	Noted, no additional comments.
19	23	5.3.3	We recommend providing a concept design of pavement buildup for the splash crossings, and how downstream energy mitigation will be provided if there are drops. Also how will streambanks be protected and re-integrated into the original streambanks between splash crossing and the existing stream?	We agree that this is a worthwhile approach and will be incorporated in the detailed design. As part of detailed design a specific site visit to each of the locations is required in order to confirm the details of the profile at the crossing locations. This inspection is necessary to identify the immediate slope and the side slopes at the location. Also specific elements of the channel immediately downstream of splash crossing E requires a detailed specific site walkover due to the hummocky and complex nature of the terrain. This will form part of the detailed layout. However we reiterate that splash crossings are a normal and accepted detail for this use and terrain. For example the Wanaka Mt Aspiring road, which is owned and operated by the QLDC, has eight similar splash crossings, similarly the Dart valley road, the Rees valley road have splash crossings as does the Ohau ski area access road and the Lake Ohau road.	Noted, no additional comments.
20	24	Table 6	We recommend justifying what design guidance has been used or will be used during detail design to establish the splash crossing scour protection, including consideration for splash pool or downstream energy mitigation into creek bed.	Agreed. See note above	Noted, no additional comments.

21	24	Table 6	No associated flow estimations are included for Catchment E and F.   | | area | runoff coefficient C | RCP5 Q L/s | RCP 8.5 Q L/s | |---|---|---|---|---| | Catchment C | 24 | 0.35 | 854 | 980 | | catchments E & F | 18.5 | 0.35 | 658 | 755 | | sum | | | 1512 | 1735 | Manning for Trapezoidal, rectangular and vertical gutters | L (m) | 1:x | 1:y | Longitudinal gradient (1:z) | Manning n-value | Q (m^3/s) | d (m) | Q (l/s) | Velocity (m/s) | |---|---|---|---|---|---|---|---| | 1 | 2 | 2 | 20 | 0.03 | 1.599482 | 0.383213 | 1999.5 | 2.95 | Wetter X-sectional area (m^2) 0.676967	The existing survey data does no represent the hummocky nature of the area within the rock glacier. The area of the proposed armoured channel (shown in the appendices to the report) to convey flow from catchments E&F is hummocky and filled with large boulders. This is shown in the drawings in the appendix and elaborated on the diagrams at the left. The moraine in this area, east of the trail formation, in the rock glacier area is hillocky and contains numerous large boulders, and the concept is to provide an armoured open channel though this area to drain catchments E & F. Catchment C drains along a number of existing ephemeral water courses south and west of the trail and is conveyed over the trail over splash crossing F an Culvert C. Concept calculation of the flows from catchments E, F & C are shown at left. A runoff coefficient of 0.35 has been used in these calculations. The maximum flow from the combined catchments is approximately 1,700 L/S. We anticipate that a metre depth channel with 2H:1V side batters will be required and that the armouring rock will be 200 kg schist rocks laid in an interlocking bond on the invert and side. Concept design calculations indicate that this channel will have a capacity to convey 2000 L/S at a flow depth of approximately 400mm. Thus the estimated concept channel size is sufficient and will be confirmed as part of detailed design. Rock armouring will be designed in accordance with relevant documents as indicated above and laid in an interlocking mosaic patters as shown in the photograph below . The required minimum size of rocks for the riprap can be determined by referring to Diagram H-III.  Diagram H-III: Riprap sizing (1) 	Noted. This clarifies Catchments E & F and the proposed drainage approach. The detailed design should include the design for the outlet of the the proposed armoured channel , as this channel will convey high concentrated flows during bigger rainfall events. The outfall from the channel should ensure management of downstream erosion at the the concentrated outlet, and re-integration of the concentrated flow with natural flopaths/watercourses.
22	31	5.5.1	How is the runoff from the ephemeral watercourse to the tarn/reservoir intended to be split between the two trapezoidal channels? Or is the flow to be conveyed completely within the northeastern channel, with the western channel only collecting surrounding runoff?	The northern most channel will be designed for the total flow. The southern most channel will be designed to capture local flow.	Noted, no additional comments.
23	31	5.5.1	The conceptual dimensions of the channels differ between the text in the report and those given in Figure 31. Also what is the flattest gradient the concept dimensions are based upon?	The concept dimensions are at this stage nominal. The north east channel will fall approximately 2-3m over its length adjacent to the reservoir, from approximately 1709m to 1707m at the eastern end of the reservoir. However the details of the swale remain to be developed during detailed design and coordinated with the geometry of the reservoir. Thus the grade of the channel will fall approximately 2m over and 80m length, or approximately 2.5%	Noted, no additional comments.
24	31	5.5.1	The southwestern channel appears to discharge onto the access road to the reservoir. Would a culvert or splash crossing be provided here to prevent runoff washing away road or being channelled down the access road?	Yes this is correct and splash crossing will be used	Noted, no additional comments.
25	31	Figure 31	Is the reservoir going to have an emergency spillway in the case it overfills, or the channels fail during a large storm event and the runoff discharges into the reservoir?	Yes the reservoir will have an overflow weir and will be designed in accordance with the NSOLD guidelines. There is a separate reservoir concept report which covers these details	Noted, no additional comments.
26	33	6.1.1	Section 6.1.1 states that roof runoff is calculated using 50 yr ARI, Table 1 states that the 10 yr ARI will be used. Please clarify. Will 100 yr ARI be used for secondary flow calculations as per NZS 4404:2004 4.3.2.5.1?	All buildings other than the carrier stacking and gondola return station are secondary and discharge will be to ground without gutters as identified in section 6.1.2 of the report as gutters on steeper roofs are prone to snow damage. For the purposes of concept design 100 year ARI has been used for the Cafe/carrier stacking building. Snow build up in the gutters of the Cafe/carrier stacking building, may reduce the effectiveness of the gutters and thus the conservative figure of 100 year ARI has been used. The gondola return station will have recessed internal gutters and thus any surplus water can bypass the gutters over the cladding. This is shown in figure 33 of the stormwater concept report.	Noted, all design assumptions to be clearly clarified and justified during detailed design.
27	36	6.1.5	We recommend confirmation on whether the Gondola Mid Station roof runoff would discharge to the north (Rastus Burn catchment) or South (Upper Doolans Creek Right Branch Basin). It is recommended to maintain pre-development catchment distribution where possible.	The gondola mid station roof will discharge to the north.	Noted, no additional comments.
	B-15A Remarkables Ski Area Expansion Project Natural Hazards Assessment Redacted				
1	9	5.2	Has the flood risk been assessed for blockage of the culvert in the unnamed creek adjacent to the Gondola Return Terminal and Base Building? From the concept layouts is appears as if these buildings would be close to the creek, and that the platforms will be in cut, while the road will be filled over the creek. If the culvert block and the road overtops, what would be the potential flood extent and relationship to the buildings? On the drawings this appears to be much less than 5m at reported.   Gondola Return Station ground Profile A  Gondola Return Station Profile B	The risk of blockage of the culvert adjacent to the gondola return station resulting from rock or trees is considered to be low due to the nature of the catchment which does not contain either trees or other significant woody vegetation. During winter the creeks are entirely snow filled in a similar manner to occurs in all water courses above the snowline. During winter, precipitation occurs as snow and creek flow resulting from rainfall is attenuated by the overlying snow. There is approximately 2m between the building floor and the channel after the area has been cut down. In this area the channel is approximately 20m wide •The level difference between the platform level and the riverbed to the north-east is approximately 16m as shown in Profile A •The level difference between the platform level and the riverbed to the north is 2m •The level difference between the finished platform level and the wetland to the north-west is approximately 0.5m as shown in profile B While detailed design remains to be completed, for stormwater ponding in the wetland to reach the level of the ground at the Gondola Return Station, a channel up to approximately 2m deep and approximately 20 -25m wide with significant gradient would need to be flowing at the outlet to the wetland adjacent to the Gondola Return Station. We anticipate super critical flow will occur in the channel downstream of the Gondola Return Station. Thus, there is significant hydrological capacity available to drain the wetland before the water level in the wetland reaches the level of the Gondola Return Station. This will need to be confirmed as part of Detailed Design.  	Noted. However the critical condition at this location would not be the channel capacity dimension to the north, but the road elevation and culvert diameter where it crosses the watercourse. The proposed culvert diameter is 900mm, with say another 1m of fill over the culvert. Thus if the culvert is operating a full capacity, there will be headwater effect upstream of this road/culvert. Using inlet conditions for the culvert, and a flow of 1800 L/s, this could be as high as 1.5m or more, and this headwater relationship upstream of the road, to the building floor level should be addressed during detailed design to prove compliance with the NZBC. Blockage of the culvert could occur from debris movement of man made items and articles. We recommend a conservative approach here would be to compare the water level for peak flow over the road, compared to the building level as a sensitivity scenario during detailed design.

					
	B-14 Remarkables Ski Area Upgrade & Doolans Expansion - Tracks, Stream Crossings and Stormwater Freshwater Ecological and Hydrological Impact Assessment				
1	N/A		We recommend that the recommendations from this report be integrated into the concept stormwater report. There are clear guidelines and design suggestions that need to be incorporated into the design.	The points raised will be identified in the design attributes statement to accompany the final design documents.	Noted, no aditional comments.

Project Number: 1015
Project Name: Peer Review - Remarkables Ski field Doolans Basin Expansion - Three Waters
Peer Review By: GF / ACS / EJ
Checked By: ACS
Date: 26/08/2026

B	B-17A Remarkables Ski Area Upgrade and Doolans Basin Expansion -Doolans Wastewater Concept Report Redacted				
ID	Page No	Section	Peer Review Comment	Response to Peer Review Comment	Peer Reviewer Comments #2 - 26 August 2026
1			This peer review has not addressed treatment of the wastewater as it is located in the QLDC district.	noted	no further comment
2			Wastewater flows of 34 l/p/day have been based on measured flows and patronage numbers from the 2025 season. This is a good approach, however the potable water supply flows have been calculated using the same approach and have determined 50 l/p/day for water. The report does not identify where the 16 l/p/day difference ends up. Based on a peak patronage of 6000 people this equates to 96m3. Please explain. **Potable water flow rates for Remarkables Expansion** Maximum Building Occupancy (no. people) / **peak** hourly demand per occupant (litres) / Building peak hourly flow rate (m3/hour) / Building Buffer storage volume required for Cl contact @ 15mg/litre (m3) / approximate maximum building demand @ 50 litres/occupant/ day (m3/day) / RM16.115.01.2017 Existing Daily Abstraction Consent limit (m3/day) Doolans: 450 / 10 / 4.5 / 4.2 / 22.5 / Rastus Burn: 3052 / 10 / 30.5 / 28.3 / 152.6 / 320.4 sum: 3502 / / / / 175.1 / **Wastewater flow rates for Remarkables Expansion** maximum ski area occupancy (no. people) / wastewater generation L/person/day used in calculations / wastewater generation from historical measurements L/person/day / daily discharge based on design flow rates (@ 34 L/p/d) (m3/day) / daily discharge based on historic flow rates (@ 32L/P/d) (m3/day) / Existing Daily Consent discharge limit (m3/day) Doolans: 2500 Rastus Burn: 3500 sum: 6000 / 34 / 32.5 / 204 / 195 / 127.44	Two approaches have been taken to the sizing of water facilities. Wastewater daily flow rates have been based on records of measured wastewater flow from the Remarkables and other ski areas. Wastewater treatment at the Rastus Burn facilities area will include flow balancing tanks, thus the plant is designed for the daily wastewater volume rather than instantaneous flow rate. The design of the potable water supply to the buildings is based on peak hourly flow. This has been calculated on the basis of 10 L/h/person. Wastewater generation records show an average of 32.5 L of wastewater per visitor per day. This rate has been checked against ARC TP58 and ASNZS1547 standards where relevant. This has been increased to 34 L/visitor/d to allow a margin, and to allow accurate costs to be developed. The peak hour potable water demand has been based on 10 L/p/hour, based on 6 L toilet flushing and 4 L hand washing and other uses, and this has been applied to the building occupancy rather than total visitor numbers. A check of the existing consent for potable water abstraction was undertaken using a conservative demand figure of 50 L/d/p times 6000 people = 300 m3 daily. This check confirms that the existing daily consented water take (RM16.115.01.2017) of 320.4m3 from the Rastus Burn is sufficient for the predicted visitor numbers. The cost of the wastewater treatment plant is orders of magnitude greater per litre than for the potable water system. Thus the potable system has been designed for peak hourly rates and an upper bound check applied against the consent limits and daily flow. Whereas the wastewater treatment concept is sized for a more accurate figure of 34 L/p/d on the basis of flow records and checks against the above standards. These figures are summarised in the table at left.	Noted - Our concern with regards to the potable water supply system is related to the sizing of infrastructure based on 450 people vs. the potential 2500 using the facilities - see the water supply section
3			We agree with the preferred option to collect and pump the wastewater from the Doolans expansion to the existing wastewater treatment plant site on the other side of the hill assuming that the treatment plant is upgraded to receive the additional flows.	Noted. The preferred option identified by the Doolans Wastewater Concept Report is to transfer wastewater to the Rastus Burn catchment for treatment. A new wastewater treatment plant is required in the Rastus Burn. The Rastus Burn Feasibility Report assesses the treatment and disposal options required to receive these additional flows and concludes that feasible upgrade pathways exist. Refer to Sections 6 and 7 of the Rastus Burn Report. This will form part of the overall development. We confirm that the treatment plant in the Rastus Burn will be sized for the fully developed predicted visitor numbers.	Noted
4			We agree with the macerating pump followed by two progressive cavity pumps with chambers upstream of the pumps sized to allow the downstream rising main to drain back to their respective chambers in case of maintenance being required. We would want to see the rationale behind the sizing of the chambers to make sure that they also had sufficient operating storage in case of pump failure.	The length of each section of rising main between pump stations is approximately 1300m, the rising main diameter is 76mm (table 6 page 19) thus the storage volume provided at PS3 is approximately 6m3. At PS2, which is located adjacent to the Doolans Creek Right Branch facilities, includes a 75,000 L storage tank and the receiving/storage chambers are intended to accommodate full drain-down of the rising main plus 24 hours wastewater generation as described in section 10.1.1.1 of the wastewater concept report . The concept design includes duty/standby pumps at all pump stations. Accordingly, a single pump failure does not trigger utilisation of emergency storage. Final storage calculations will be confirmed during detailed design based on final pipeline volumes and operational requirements. Refer to section 10.1.1.1 of the wastewater concept report and note the 24hr emergency storage volume allowance.	Noted
5			No mention is made of the vapour pressure of water at these higher elevations. This may be important if the positive displacement pumps are sucking up out of the chambers upstream. Please address in the detailed design.	The progressive cavity pumps will not operate by suction from the well. Wastewater is boosted from the chambers by submerged macerating pumps which provide a positive feed to the progressive cavity pumps. These will be in a duty/standby arrangement. Thus suction is not an issue.	Noted, however be aware of hydraulic issues of pumping directly into the suction line of the next pump in series
6			Please discuss Peak Wet Weather flows and the associated peaking factors. At this stage the pipes and pumping stations appear to be designed for flushing flows and velocities but they are not compared to the estimated peak wet weather flows. Under CODC's standards an estimated peaking factor of 2x the peak dry weather flow should be considered.	The system is not a municipal system and the wet weather peaking factors are not applicable to this system as it is effectively a closed system. There are a three reasons for this: The first is that it is an alpine system and cannot use standard external gully traps which are the primary source of stormwater infiltration into a wastewater system. External gullies are susceptible to freezing and ingress of snow. Toilets will be plumbed directly to the underground wastewater system with venting, and kitchen and sinks will be plumbed into a vented external grease trap, which is sealed from surface water ingress. This avoids the risk of both freezing and ingress of surface water. The second reason is that when precipitation occurs during winter, the bulk of the precipitation will be as snow. When most spring thaw occurs the ski area will be closed. Finally, the third reason is that each pump station has a significant storage volume, and thus flows become significantly attenuated and the volume of wastewater in each 1300mm length of 75mm pipe is approximately the volume of daily wastewater generation. Traditional municipal wet-weather peaking factors have not been adopted for concept sizing as they are not considered representative of the proposed alpine wastewater system. The adopted wastewater generation rates have been derived from monitored wastewater flows and patronage data collected from the existing Rastus Burn facilities during ski field operations. These records reflect actual system operation, including the effects of seasonal use, by-wash, operational practices and any infiltration that may be present. The resulting flow rates are therefore considered more representative of anticipated performance than generic municipal peaking factors. The adopted design wastewater generation rates were also cross-checked against published guidance, other ski field facilities and AS/NZS 1547:2012.	Noted
7			Flushing cycles are discussed and should be incorporated into the design. It is recommended that water also be supplied to the pumping stations to allow the rising mains to be flushed with water and the chambers to be washed down. There may also be the need to supplement flows of water into the system during the lower demand summer months to prevent the wastewater going septic, generating malodours and adding additional load onto the treatment plant.	Yes we agree. Flushing cycles, septicity management, odour control measures and periodic high-velocity flushing are discussed in Sections 12.1.1 and 12.1.2 of the Doolans report. The concept report already identifies operational flushing strategies and, where required, chemical dosing options to control sulphide formation and odour. We anticipate that significant flushing of the line using untreated water from the storage tanks would be undertaken at the end of each season. The connection to allow flushing water into the system is a design detail.	No further comment
8		10.1	There appears to be a difference in rising main lengths quoted in this section. Section 10.1.1 quotes 2.6km while Table 6 quotes 3.61km	This is a typo, 3.6 should be 2.6km	Noted

9		10.1	There also appears to be a difference in the height to be pumped. Section 10.1.1 quotes 365m while Table 6 calculates to be 343m. Section 12 states 367m	The reviewer has correctly identified that different elevation figures are presented within the report. These values refer to different aspects of the concept design. Section 10.1.1.1 uses QLDC LiDAR data to estimate the cumulative elevation gain along the rising main alignment and provides context regarding the highly undulating nature of the proposed route. Because the alignment contains a number of rises and falls between pump stations, the cumulative elevation gain along the pipeline is not necessarily equal to the net vertical difference between the start and end points. Table 6 presents the point-to-point vertical lift between chambers and pump stations, which is the relevant parameter for concept pump sizing and hydraulic assessment. These values exclude intermediate undulations and represent the static lift that each pump station is required to overcome. If the rising main followed a continuously increasing grade with no intervening high and low points, the cumulative elevation gain and the overall static lift would be identical. However, the proposed alignment contains a number of undulating sections and therefore the cumulative elevation gain is greater than the net vertical rise. The values presented are based on concept-stage LiDAR information and are intended to provide order-of-magnitude assessment only. Final elevations, pump duties and total dynamic head calculations will be confirmed during detailed design using surveyed alignment data and the selected pipeline route. The report will be updated in the next revision to improve clarity and ensure the distinction between cumulative elevation gain and pump station lift requirements is clearly stated.	Thank you
10			No mention is made of how the pipeline alignment and trench details will cope with movement within the mobile scree slopes. Please advise.	Only the upper surface particles of the scree are sufficently mobile to be an issue, whereas deeper particles are not. Unless there is an actual landslide feature, or the services are within the depth affected by solifluction, ground movement is not an issue. Ski areas throughout New Zealand, including the Rastus Burn side of the Remarkables, have extensive underground services buried in scree through out the developed area. This includes data and communications, compressed air, snow making water, potable water and power. These services are laid at sufficient depth so that they are not affected by ground freezing and scree movement is not an issue at this depth. While a detailed geotechnical assessment remains a future design task, scree movement is not currently considered a significant constraint to the viability of the preferred option.	Noted
11			The wastewater treatment plant report (not being assessed as a part of this peer review) discusses by-wash water. Please confirm that this does not need to be considered in the design of the Doolans wastewater pumping stations and rising mains.	By-wash is discussed in Section 2.3 of the Rastus Burn Wastewater Feasibility Report. It is intended to provide facilities in the Doolans Creek Right Branch that are designed appropriately so that by wash will be minimised or eliminated. By wash is an ongoing operational nuisance in the Rastus Burn base facilities. NZSki is going through a significant programme of minimising, or eliminating, by wash by heating, insulation and operational practice in all areas possible. This programme of by wash reduction is one of the factors contributing to the ongoing monitoring of wastewater flows. With new facilities, such as the Doolans infrastructure, the infrastructure will be heated, insulated and operated in such a way that by wash is eliminated or reduced. Confirmation of expected operational by-wash volumes will nevertheless form part of subsequent design stages.	Noted
12		10.4.4	Table 11 of the report seems to have misquoted the expected wastewater quantities as 83,835 litres rather than 85,000 litres	Table 11 follows the procedure outlined in AS/NZS1547:2012 where peak occupancy is multiplied by L/Person/Day. This has not been rounded up to 85,000L/day to determine the minimum area requirement for land disposal. Due to this option not requiring flushing volumes like the rising main, (which has been allowed for in the 85,000L/day estimate) the 83,835L/day is correct. Typically during selection of a proprietary treatment plant these flows are increased and rounded to their nominal design size. As this has not been progressed further due to constraints discussed in this report, the estimated daily flow has not been rounded up thereby creating multiple rounding errors in the calculation. It is worth noting that there is a safety factor built into the Design Loading Rate DLR and as such, rounding up the peak daily flow (not average daily flow) and then applying via a lower loading rate (with a safety factor) can increase the size of the discharge field.	Noted
13		12	Section 12 describes the next steps. It is suggested that a surge analysis of the rising mains also be included as the height being pumped could result in significant surge pressures upon a sudden stop in flows	Agreed. Surge analysis is considered an appropriate component of the next design stage given the significant static head associated with the rising main system. This requirement does not materially alter the viability of the preferred transfer option and is expected to inform final pressure class selection, transient control measures and valve requirements.	No further comment
14		12	Please check the minimum scouring velocities against the WSA requirements for the pipeline duty. It is noted in Table 12 the velocities are quite low for the pipe selected.	Continuous self-cleansing velocities are not expected under all operating conditions and the concept design therefore adopts periodic flushing, filling the main and opening a drain-back valve to achieve a high velocity scouring flow back into the chamber and concurrent operation of duty/assist pumps as the primary means of maintaining the pipeline. The concept report acknowledges that normal operating velocities may be below self-cleansing velocities and therefore incorporates periodic flushing as a key operational requirement. Target flushing velocities, scour management and maintenance arrangements are discussed in the concept design recommendations. Given the unique operating characteristics of the system, including seasonal operation and pumped transfer, periodic flushing has been adopted as the preferred approach rather than relying solely on continuous self-cleansing velocities. The detailed design will further confirm velocity requirements and operational flushing practices.	Noted

Project Number: 1015
Project Name: Peer Review - Remarkables Ski field Doolans Basin Expansion - Three Waters
Peer Review By: GF / ACS / EJ
Checked By: ACS
Date: 26/08/2026

C	B-19A Remarkables Ski Area Upgrade and Doolans Expansion - Potable Water Supply Infrastructure Concept Report Redacted				
ID	Page No	Section	Peer Review Comment	Stantec Response to Peer Review Comment	Peer Reviewer Comments #2 - 26 Aug 2026
1			It is noted that the design uses raw water supply tanks to feed fire-fighting sprinklers in the building. Are the sprinkler suppliers and FENZ happy that the quality of the raw water will not impact the ability of the sprinklers to operate effectively without blocking?	The fire fighting reserve for the existing base building in the Rastus Burn is taken directly from the raw water tank farm, which is shown in figure 4 of the Potable Water Supply Infrastructure Concept Report. Fire fighting water in the Rastus Burn base building is untreated raw water but there are in line strainers as part of this fire fighting sprinkler system. The Doolans fire fighting system is proposed to be along the same lines. Fire fighting water supply will include inline strainers in a similar manner to the Rastus Burn base building system. The design is acceptable to FENZ. In addition, as noted below, the intake pump will be shut off at times when the Doolans Creek Right Branch is in flood and the water in the creek is turbid	Noted
2			It is noted that the water demand for snow making is not addressed significantly in this report as it focusses on the potable water supply. Water for snow making has more of an impact on the water take in Doolans Stream intake and pumping station - this was not reviewed significantly as part of this peer review	Snow making demand has been addressed in separate documents. Stantec has allowed for snow making flows in the Doolans Creek Right Branch intake structure and description of this is included in the intake structure concept report. Stantec has also provided a concept report for the snow making reservoir in the Doolans Creek Right Branch. The specific concept design of the snow making system and layout has been undertaken separately by TechnoAlpin	Noted
3		1.3.2	The treatment process for the Doolan's water supply has been based on only one water sample taken in May 2025. It is strongly recommended that more samples are taken over different seasons to help determine the appropriate treatment process. Our experience for the Cardrona water supply identified that where the water was being taken from the stream just below the Cardrona ski field it showed signs of algae at certain times of the year that needed to be considered in the treatment design. Stantec discounted this as a potential issue in one of the assumptions in Section 1.3.2 of the report.	Stantec agree that further water testing is appropriate prior to detailed design. However we consider that the design assumptions listed in 1.3.2 of the report are reasonable for the concept design and are based on the existing sample from Doolans Creek Right Branch, sampling of the Rastus Burn and our experience of raw water quality at both The Remarkables potable water system in the Rastus Burn and the potable water system at Coronet Peak. Thus we believe that assumptions upon the concept design is based on are appropriate for the location and reasonable. Nevertheless additional testing to confirm these assumptions is warranted prior to detailed design	Noted that additional testing and refinement of the treatment design will occur at the detailed design phase
4			Potable water demands have been based on an ultimate peak daily patronage of 6000 persons with 3500 using the facilities on the Rastus Burn side of the hill (QLDC) and 2500 persons using the facilities on the Doolans side of the hill (CODC). The peak daily water demand calculations are based on 50 l/p/day usage. This was determined based on measured water usage and patronage numbers from the 2025 season. This is reasonable and comparable to other similar water demand profiles such as schools and shopping centres. It was not really used in the design process as described below which seems odd.	Two approaches have been taken to the sizing of water facilities. Wastewater daily flow rates have been based on records of measured wastewater flow from the Remarkables and other ski areas. Wastewater treatment at the Rastus Burn facilities area will include flow balancing tanks, thus the plant is designed for the daily wastewater volume rather than instantaneous flow rate. The design of the potable water supply to the buildings is based on peak hourly flow. This has been calculated on the basis of 10 L/h/person. Wastewater generation records show an average of 32.5 L of wastewater per visitor per day. This rate has been checked against ARC TP58 and ASNZS1547 standards were relevant. This has been increased to 34 L/visitor/d to allow a margin, and to allow accurate costs to be developed. The peak hour potable water demand has been based on 10 L/p/hour, based on 6 L toilet flushing and 4 L hand washing and other uses, and this has been applied to the building occupancy rather than total visitor numbers. A check of the existing consent for potable water abstraction was undertaken using a conservative demand figure of 50 L/d/p times 6000 people = 300 m3 daily. This check confirms that the existing daily consented water take (RM16.115.01.2017) of 320.4m3 from the Rastus Burn is sufficient for the predicted visitor numbers. The cost of the wastewater treatment plant is orders of magnitude greater per litre than for the potable water system. Thus the potable system has been designed for peak hourly rates and an upper bound check applied against the consent limits and daily flow. Whereas the wastewater treatment concept is sized for a more accurate figure of 34 L/p/d on the basis of flow records and checks against the above standards. These figures are summarised in the table below.	Your comments are noted. Our concern with regards to the potable water supply system is related to the sizing of infrastructure based on 450 people vs. the potential 2500 using the facilities
5			It is noted that the wastewater demand has been calculated to be 34 l/p/day. Where is the remaining 16 l/p/day of water going from the measured 50 l/p/day determined for the water supply?	**Potable water flow rates for Remarkables Expansion** Maximum Building Occupancy (no. people) | **peak** hourly demand per occupant (litres) | Building peak hourly flow rate (m3/hour) | Building Buffer storage volume required for Cl contact @ 15mg/litre (m3) | approximate maximum building demand @ 50 litres/occupant/ day (m3/day) | RM16.115.01.2017 Existing Daily Abstraction Consent limit (m3/day) Doolans: 450 | 10 | 4.5 | 4.2 | 22.5 | Rastus Burn: 3052 | 10 | 30.5 | 28.3 | 152.6 | 320.4 sum: 3502 | 175.1 **Wastewater flow rates for Remarkables Expansion** maximum ski area occupancy (no. people) | wastewater generation L/person/day used in calculations | wastewater generation from historical measurements L/person/day | daily discharge based on design flow rates (@ 34 L/p/d) (m3/day) | daily discharge based on historic flow rates (@ 32L/P/d) (m3/day) | Existing Daily Consent discharge limit (m3/day) Doolans: 2500 | Rastus Burn: 3500 | sum: 6000 | 34 | 32.5 | 204 | 195 | 127.44	Our concern with regards to the potable water supply system is related to the sizing of infrastructure based on 450 people vs. the potential 2500 using the facilities

6			Albeit that the peak daily demand has been calculated as 50 l/p/day, subsequently peak daily and peak hourly flows have then been calculated based on building maximum occupancy rates which have then been used to calculate the sizing of the water supply infrastructure. Please explain the reasoning behind this because the maximum occupancy of the building does not reflect the maximum estimated number of people using the facilities. Please note that this applies to both the Rastus Burn and Doolans supplies. Please explain.	calculation for Rastus Burn potable water treatment based on occupancy as below; • Maximum WTP design flow rate: ○ Ultimate occupancy maximum Base Building occupancy: 3,052 people Peak hourly water demand: $\frac{3{,}052\ people}{1\ h} \times 10\frac{L}{person} \times \frac{1\ m^3}{1{,}000\ L} = 30.5\frac{m^3}{h}$ Where, 10 litres per capita estimate is based on 6 litres for toilet flushing and 4 litres for handwashing. calculation for Doolans potable water treatment based on occupancy as below; • Maximum WTP design flow rate: ○ Ultimate building occupancy: 450 people ○ Peak hourly water demand: $\frac{450\ people}{h} \times 10\frac{L}{person} \times \frac{1\ m^3}{1{,}000\ L} = 4.5\frac{m^3}{h}$ ○ Where, 10 litres per capita estimate is based on 6 litres for toilet flushing and 4 litres for handwashing.	We note that this approach assumes that 450 people may pass through the Doolans building in one hour albeit that there is the potential for 2500 people to be using the facilities at Doolans over a day. We assume that you are happy with this. Our concerns are that the amount of potable water storage and the infrstructure to deliver the daily water demand does not reflect the potential number of people using the facilities and how much potable water would be available to consumers if the treatment plant fails.
7		4.1	The sizing of the Doolan's water supply infrastructure has been based on the same approach taken for the Rastus Burn infrastructure on the other side of the hill. However the Rastus Burn infrastructure has assumed the full 6000 person patronage (by mistake?) Should it not have been 3500 persons? The peak daily demand calculated based on the building peak occupancy rate (305 m3/day) was very close to the calculated peak daily demand (300m3/day) by accident. This unfortunately did not apply to the Doolans calculations where the peak daily demand based on building peak occupancy was 45m3/day compared to 125m3/day when using 2500 x 50 l/p/day. This is a large variance and affects the sizing of all the water infrastructure on the Doolans side of the hill noted in the report. Please explain.	The 6,000 person patronage to the Rastus Burn building was selected as all visitors must access the Remarkables skifield (Rastus Burn or Doolans side) via this building. The approach to selecting the peak instantaneous flow rate for the Doolans Creek is the same as described above for the Rastus Burn.	See comment above
8			The Water Treatment Plants have been designed to treat the estimated peak hourly flows with the treated water storage only sized to give the ski field enough chlorine contact time. This applies to both the Rastus Burn and Doolan's supplies. **Notwithstanding that the treatment plant has been undersized for the Doolans supply as described above,** this means that if either treatment plant goes down there will be only a maximum of 1.7 hours storage (at peak demand) on the Rastus Burn side and 0.4 hours on the Doolans side of the hill based on an undersized WTP and sizing the reservoir based on chlorine contact time. This does not meet CODC's standards.	We do not believe that the Doolans Cafe/carrier stacking water supply and treatment system has been undersized. The water supply system in the Doolans facilities has been sized for the maximum occupancy of the building, which is 450 people, as outlined as above. This has been conservativel sized for a maximum flow of full occupancy and 10 L/person/hour. THere is a on site supply of 125,000 L of raw water. The treated water storage at Doolans is based on acheiveing sufficient chlorine contact time although currently the Doolans water supply system is not required to have chlorination, but chlorination has been provided as good practice and to allow for the drinking water standards to require this in the future. The recommended Doolans potable water system includes a single UV reactor (with the provision for an additional reactor)and a minimum of 5000 litres storage to ensure chlorine contact time. The facility is not a municipal supply and is intended only to serve day visitors and the facilities do not include overnight accommodation. If a significant power outage were to occur, the Doolans facilities would be closed and guests removed on the gondola using diesel backup operation. Should a significant failure of the potable water system occur, the facilities would be closed as the Cafe could not function. Ski area operations involve multiple environmental factors that result in closure of the facilities including wind, rainfall and avalanche control operations. The risk to operation from the potable water supply system is not considered significant to NZSki's operation compared to the other environmental factors. However, the layout will provide provision for the addition of a standby reactor, but a decision on whether to install this will be made following the initial operation and commissioning of the plant. Currently duty and standby reactors are provided at the Rastus Burn Base facilities, but the Doolans facilities are considerably smaller and the operation of the water treatment system in Doolans is not a critical element of the operation of the Doolans Creek Right branch ski facilities	Noted - you should probably also look at provisions for additional potable water storage as well
9			In Section 2.4.1 the report states: *"It is uncertain at this stage how the treatment systems will be maintained and how quickly they will be fixed if a fault occurs."* The report then goes on to state that no standby units (filters, UV or chlorination systems) will be installed in the treatment plants. This is questionable given the very low volumes of treated water storage being provided as noted above. It is recommended that this approach be revisited as once the treated water storage has been exhausted the skiers will not have access to any potable water.	see above comment. This is not a minicipal supply and should a significant failure of the water supply system occur the café will be closed. As noted this risk is considered to be less than the risk posed by other environmental factors such as wind. Should this be the case guests can ride the gondola back to the Rastus Burn facilities.	Noted. We assume that your comment about closing the café will also relate to the toilets and wash hand basins or are you using raw water for these?
10			Section 2.4 notes that 4-log removal of protozoa will be required to treat the surface water. We agree with this. The automatic backwashing disc filters however do not offer any log removal and the TrojanUV / Viqua Pro50 UV units are only rated to 3-log protozoa removal. Subsequent chlorination does not offer any log removal of protozoa. The treatment systems proposed for each side of the hill therefore do not comply with the Drinking Water Regulations. Please amend.	The TrojanUV / Viqua Pro50 UV provides a compliant dose rate as the Viqua Pro50 UV reactor is validated against the USEPA UVDGM 2006. Under this criteria, it was validated to deliver a dose of 40 mJ/cm2 at a flow rate of 189 L/min (3.15 L/s) at a UVT of 85%. As stated in the Concept Design Report, the installation of the UV reactor at both the Rastus Burn and Doolans Creek Right Branch water supply will be provided with an appropriately sized flow restrictor and UV sensor for dose monitoring. This meets the requirements of the current and revised (2026) Drinking Water Quality Assurance Rules to provide both a bacterial and protozoal treatment barrier. The UV dose targets, upon which the validation criteria is based (40 CFR 141.720(d)(1)), states that a dose of 22 mJ/cm2 provides 4-log inactivation of both cryptosporidium and giardia lamblia (protozoa). Nevertheless there are a variety of other similar UV disinfection units which provide 4 log protozoa deactivation across a variety of flow rates. Alternative acceptable alternatives are available. As part of the final design the design attributes staement will elaborate on the suitability of the system to achieve appropriate deactivation of protozoa.	Noted - please provide manufacturers certificates from the UV supplier during detailed design showing compliance
11		2.4.1	It is noted that the treated water reservoirs have been sized based on providing the necessary chlorine contact time. Proposed changes to the WSA Level 3 rules alluded to in Section 2.4.1 have now been released. Under the new rules (to be implemented in 1 July 2027), for a Level 3 supply you do not need to provide chlorine contact time if UV disinfection is being used and is effective. Reference to chlorine contact time is therefore not really relevant.	noted, see below	Noted
12		2.4.1	It is noted in Section 2.4.1 that the proposed treatment systems will provide a multi-barrier approach. Chlorine does not offer any protection against protozoa infection.	the provision of chlorination does not, and is not intended to, provide deactivation of protozoa. Chlorination is provide in the Doolans Creek Right Branch facilities to provide network protection from bacteria growth. Chlorination is not specifically required as the supply is for a single building. However the provision of chlorination may be required at a later date and providing provision for this is consistent with good practice. Chlorination is required at the Rastus Burn facilities as the system provides water supply to multiple buildings, being the ski patrol building and maintenance building in addition to the base facilities	Noted

13		5.2.2	Section 5.2.2 describes how the Doolans raw water rising main is sized. Based on the assumption that the Doolan's water demand is 2500 x 50 =125m3 needs to be delivered to the WTP over a 12 hour period (as there is some raw water storage) this equates to a flow rate of 2.9 l/sec and not 1.5 l/sec as quoted. This has an effect on the size of the pipeline and pumps. Please revisit this design.	It is anticipated that most visitors will use the toilet facilities at the Rastus Burn on their arrival. Nevertheless the potable water system is conservatively sized for 50 L per visitor. The Doolans Area facilities are sized for 450 people. This equates to a total daily water demand of 22500 litres. If this is pumped over 12 hours this equates to 0.5208 litres per second. In a 25mm ID pipe this equates to a flow velocity of 1.06 m/s, which is acceptable. If operated for 10 hours the flow velocity is approximately 1.3 m/s and over 8 hours operation, 1.6 m/s	You have stated that there is the potential of 2500 using the Doolans skiing facilities. This is significantly different to 450 people which is based on a building occupancy rate. At this stage the sizing of all the water facilities are based on the lower 450 persons figure.
14			The approach taken to size the raw water storage tanks seems to be confused. In the Executive Summary of the report it states that FENZ will most likely not be able to access the Doolan's Base building and as a result the building will need to be sprinklered. The report then goes on to size the sprinkler water storage tanks (raw water tanks) based on providing FW2 storage which is based on supplying hydrants outside a sprinklered building in this instance. Using this approach additional volume should also be added for sprinkler water storage. It is assumed that the Doolans Base Building will have a fire risk classification of at least OH1 with the potential of going higher in the adjacent carrier staking building. The water storage tanks should be sized based on the rules under NZS 4541 - Automatic Fire Sprinkler Systems and not NZS4509 - Fire Fighting Water Supplies Code of Practice - not NZS 4509 in this instance.	It will not be possible for the fire service to reach the Doolans base facilities in sufficient time to be effective at any time of the year. In winter wheeled vehicle access is completely impossible and in summer wheeled vehicle access is impractical and will take too long to be useful. The Carrier Stacking Building and the Cafe are one and the same structure. When the Cafe is in operation the carriers will be on the gondola rope line. When the Carriers are stacked in the building the Gondola will not be in use and there will be no guests and no room available for Cafe operations. During summer the carriers will be stored within the Carrier Stacking Building to avoid weather damage and to reduce potential wind loading on the towers greater than the serviceability wind speed. Thus, the purpose of the sprinkler system is primarily for protection of life when the Cafe is in operation during winter, however there is the added secondary benefit of protection of assets when the carriers are stacked within the building. Fire hydrants will not be provided at the Doolans base area for the reasons above as suitable access to the site for FENZ is not possible and in winter external hydrants are at risk of freezing and will be of no use. Raw water storage sufficient for sprinkler operation will be provided in accordance with NZS4541. The volume of 125,000 litres of storage indicated on the drawings in the form of five 25,000 litre tanks, is an upper bound estimate of storage anticipated. This figure needs to be specifically agreed with FENZ and the Territorial Authority as part of the development of the detailed design as this is an unusual situation. The minimum storage requirement is anticipated to be 72,000 litres which equates to 20 L/s for 60 minutes and equates to three 25,000 litre tanks. However for the purposes of indicating the maximum storage that may be required, the drawings show 125,000 litres which is made up from 72,000 Litres plus 45,000 potential allowance = 117,000. This figure has been rounded to 125,000 maximum on the basis of five 25,000 litre tanks. 125,000 litres this is consistent with the volume of raw water storage for the Rastus Burn base building. The Rastus Burn base building is sprinklered and has five 25,000 litre tanks as shown in Figure 4 of the report. The Rastus Burn base building is sprinklered but does not have complying external hydrants due to the risk of freezing.	Thank you - we note the removal of FW2 and NZS4509 from your response
15		5.2.1	It is noted in Figure 11 of the report that 125,000 l of raw water storage is to be provided adjacent to the Doolans Base Station Building whereas the words in Section 5.2.1 state 45,000 l is to be provided. Please confirm and correct the report.	125,000 litres is the correct figure. This is based on the maximum potential storage requirement as described above, but may be reduced in incriments of 25,000 following liason with FENZ and the TA during detailed design.	Noted
16		1.3.2	The report states: "When the source water becomes turbid the intake is closed and buffer storage used until the stream has cleared. This is an existing standard operating procedure." Is there a likelihood of such an event occurring during the ski season? How long can the storage sustain operations when the intake is closed?	During winter it is not anticipated that there is a significant risk of flooding causing turbidity as most winter precipetation is anticipated as snow rather than water. During heavy rain events, the ski area will be closed. Thus in such conditions it is not anticipated that there will be guests on site. It is also likely that in such circumstances the carriers will be removed from the rope and stored within the Carrier Stacking Building. Heavy rainfall leading to turbidity in the system is most likely to occur between November and March. Raw water will be fed from the dedicated pump at the intake structure to the raw water holding tanks. Should the source water become turbid this pump will be shut off. There will be a minimum of 8,000 litres of availale storage before depleating fire fighting reserve plus the 5,000 litres storage in the contact tank. This results in a minimum of 13,000 litres of available storage. This is approximately half a day supply at 50 l/person for full occupancy of 450 people which is considered to be more than sufficient in bad weather as most guests will have departed in such conditions.	See comment aove about the 450 persons vs. 2500.
17		2.5	The historic water demand calculations rely on data from the flow meter, but the flow meter accuracy has not been verified. We recommend the accuracy of the meter be checked and the flows be monitored to verify the historic flows.	noted and we agree	No further comment