Preliminary Lake Assessment

Corbridge Golf Course - Stormwater

SIO No12 Limited

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Contents

1. Introduction 4
2. Background 5
3. Sources of Information 5
4. Hydrological Analysis 6
   - 4.1 Purpose 6
   - 4.2 Assessment Process 6
     - 4.2.1 Catchment Delineation 6
     - 4.2.2 Determination of Sub-Catchment Storage Effects 8
     - 4.2.3 Runoff 9
5. Lake Level Assessment 11
   - 5.1 Process 11
   - 5.2 Soakage Rate 12
     - 5.2.1 Previous Geotechnical Reporting 12
     - 5.2.2 Soakage Rate Reduction 13
   - 5.3 Lake Level Fluctuation 13
6. Next Steps 15

1. Introduction

CGW Consulting Engineers have been engaged by SIO No12 Limited to undertake a preliminary stormwater assessment for the proposed Corbridge Golf Course development in support of a plan change application to Queenstown Lakes District Council (QLDC). The assessment is intended to establish the anticipated stormwater response of the proposed central lake and its contributing catchments, identify the likely range of lake levels during design rainfall events, and provide an initial appraisal of the feasibility and capacity of the proposed soakage-based discharge system. The scope of work comprises the following:

- Assessment of the pre- and post-development contributing catchments and flows to the proposed central lake, including preparation of catchment plans,
- Assessment of likely flood water levels within the central lake,
- Assessment of stormwater disposal from the lake via soakage, including preliminary sizing and feasibility of the proposed soakage area located in the north-west corner of the site, and options for additional storage basins to the west of the main lake area.

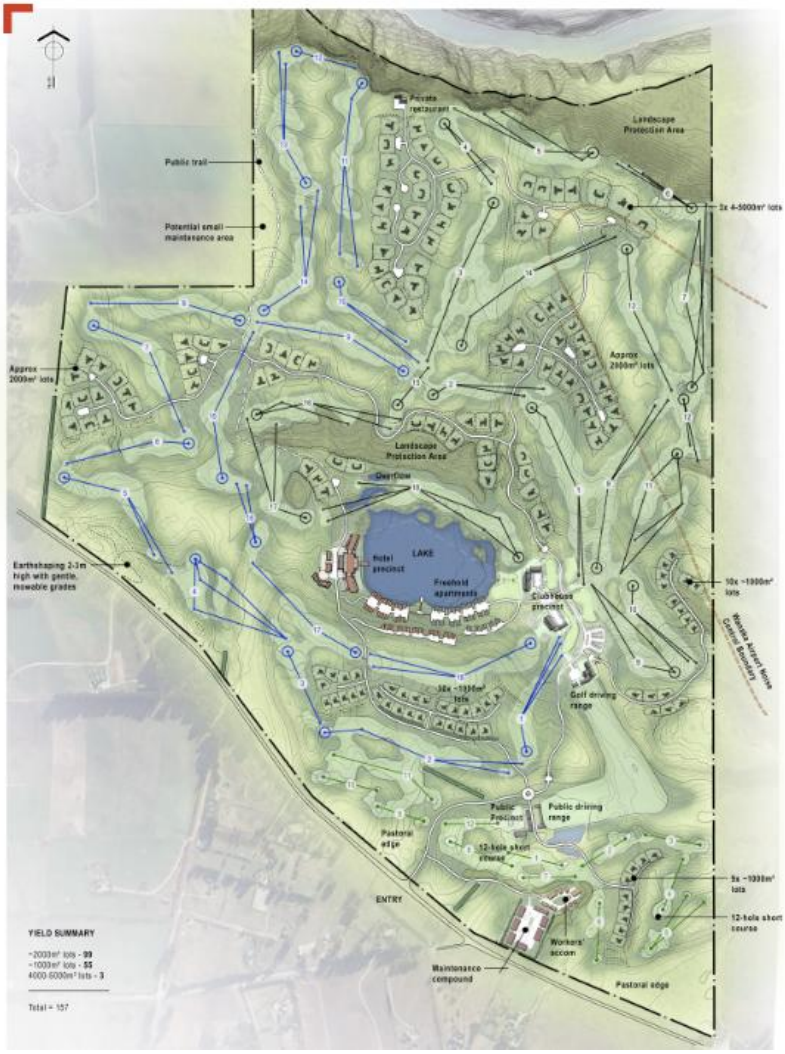


Figure 1. Baxter Design's indicative site masterplan showing central lake

2. Background

We understand that the project involves the development of the Corbridge site with the following:

- Two 18-hole championship golf courses,
- Public clubhouse,
- Members-only clubhouse,
- Members' restaurant,
- Maintenance compound,
- Boutique hotel and restaurant,
- Local residential lots,
- Staff accommodation,
- Lakeside apartment precinct,
- Plateau apartment precinct,
- Resort-style and fairway-style residential lots.

To support the proposed development, a preliminary stormwater assessment has been undertaken to estimate likely lake-level fluctuations during significant rainfall events and inform consideration of potential effects on the surrounding proposed buildings and development.

This scope does not include detailed stormwater design, final infrastructure sizing, construction documentation, or resource consent engineering design.

This report outlines the analysis undertaken. The assessment has been prepared with reference to the Queenstown Lakes District Council Land Development and Subdivision Code of Practice 2025.

3. Sources of Information

The following information has been used in the preparation of the preliminary lake assessment:

- The Baxter Design concept landscape architecture and structure plans have been referenced for the proposed site layout.
- Topographical surface information has been sourced from LINZ data service, and then Otago LIDAR 1.0m 2016 DEM has been utilised for catchment modelling as this is highest resolution available LIDAR data set that covers the site and full upstream catchment extents.
- Rainfall intensity data has been sourced from Earth Sciences New Zealand High Intensity Rainfall Design Statistic (HIRDS) Tables, using the RCP 8.5 (2081-2100) climate change scenario, in accordance with the 2025 update to the QLDC LDSCoP.

- Various geotechnical reports have been undertaken for the site, and the following geotechnical reports have been reviewed and referenced:
    - Tonkin and Taylor, Geotechnical Investigation Corbridge Downs Subdivision, March 2012,
    - GeoSolve, Preliminary Geotechnical Report for Resource Consent, July 2021,
    - ENGEO, Stage A Geotechnical Assessment, November 2022,
    - GeoSolve, Geotechnical Report for Plan Change, June 2026.

4. Hydrological Analysis

4.1 Purpose

The purpose of this assessment is to quantify the probable rainfall runoff entering the proposed lake during the selected design rainfall events and to estimate the resulting variations in lake water level.

Lake level response is governed by the interrelationship between the lake's stage–area–storage characteristics and the capacity of the associated discharge and soakage areas. Accordingly, a flexible preliminary model has been developed so that these inputs can be updated as options are refined during the preliminary design phase.

The assessment is limited to stormwater quantity and lake level response. It does not assess lake water quality or the treatment of stormwater before it enters the lake.

It has also been assumed that the lake will be topped up from other sources as necessary to maintain the minimum water level required for firefighting, irrigation, and aesthetic purposes. The source, volume, and infrastructure requirements associated with this top-up supply have not been assessed.

This preliminary assessment should be read in conjunction with Patersons' Infrastructure Assessment Report.

4.2 Assessment Process

4.2.1 Catchment Delineation

The contributing catchments to the proposed central lake were delineated in HEC-HMS using the Otago 2016 digital elevation model (DEM). Sub-catchment boundaries were generated using a nominal delineation threshold of 0.25 km², with the resulting model indicating a total contributing catchment area of approximately 709.28 ha draining to the existing depression within which the lake is proposed.

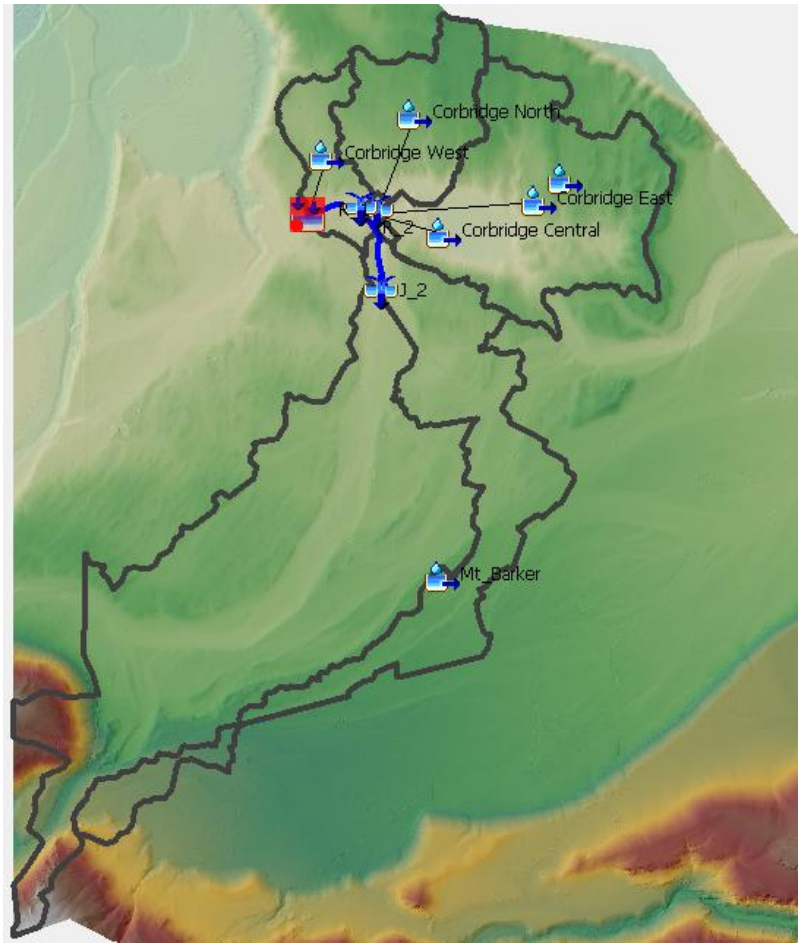


Figure 2. HEC-HMS catchment delineation

To carry out the catchment delineation, the DEM was modified to represent the existing culvert beneath State Highway 6 (SH6), by burning in a channel. This adjustment represents the runoff from the catchments south of SH6, including the upper areas extending towards Mt Barker, passing beneath the highway and discharge into the Corbridge site in accordance with the understood existing drainage arrangement.

Catchment response times were derived from the HEC-HMS sub-basin characteristics using the TP108 time-of-concentration equation. A basin lag equal to 60% of the calculated time of concentration was then adopted for the SCS unit hydrograph transform, together with the standard SCS unit hydrograph peak rate factor of 484. Rainfall losses were represented using the SCS Curve Number method, with a curve number of 61, an initial abstraction of 5 mm, Hydrologic Soil Group B, and 0% directly connected impervious area.

For this preliminary assessment, it has been assumed that runoff generated from existing and future impervious surfaces has been, and will be managed locally through disposal to ground, attenuation, or other stormwater controls. On this basis, the effective discharge from each modelled sub-catchment has been represented using the pre-development curve number of 61. This assumption has been applied consistently to the wider upstream catchment and to the proposed development areas within the Corbridge Golf Course site.

As the design progresses, the extent and connectivity of proposed impervious areas, together with the performance of stormwater management devices located throughout the site, should be incorporated into the hydrological model. The curve numbers for the developed areas should also be refined based on the proposed ground shaping and any drainage alterations as part of the golf course formation. The current assessment assumes that individual development sub-catchments will be managed so that their post-development runoff response generally replicates pre-development hydrological conditions, consistent with the LDSCoP requirements applicable to individual sites. This assumption should be tested and refined once the site layout, earthworks, drainage strategy, and proposed treatment and attenuation measures are sufficiently developed.

4.2.2 Determination of Sub-Catchment Storage Effects

An initial hydrological assessment was undertaken on the conservative assumption that all natural depressions and sinks within the contributing catchments were already full at the commencement of the design event. Under this approach, no additional depression storage was available and runoff generated within each sub-catchment was assumed to discharge directly to the downstream drainage network. This represents a conservative scenario comparable to a design storm occurring immediately after an antecedent rainfall event that had already filled the available surface storage.

As anticipated, excluding the available depression storage produced substantially higher runoff volumes and peak flow rates than those understood to have occurred during smaller observed rainfall events. Although anecdotal observations are not sufficient for formal model calibration, the difference indicated that the initial modelling approach was unlikely to represent the typical catchment response and that the effects of upstream storage warranted further consideration.

Review of the terrain data within HEC-HMS identified numerous low-lying areas and closed depressions across the upstream catchment. These features provide potentially significant temporary ponding storage, which can retain runoff and delay or reduce the volume and peak rate of flow reaching the downstream outlet. An example of the principal depressions identified from the terrain model are shown in the figure below.

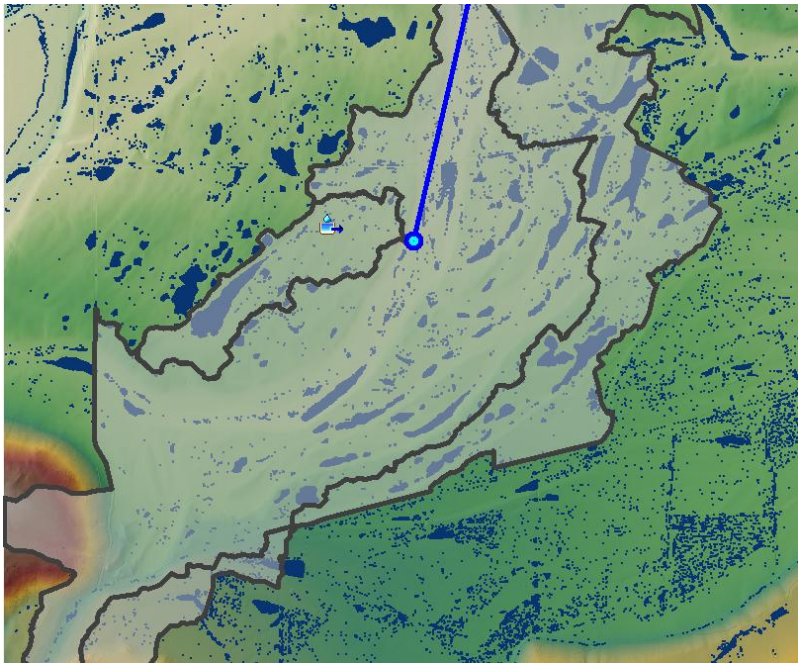


Figure 3. Excerpt of terrain data in HEC-HMS showing ponded sinks in blue.

To represent this storage more realistically, two-dimensional hydraulic modelling was undertaken in HEC-HMS for the contributing catchments south of SH6. The unmodified LiDAR terrain-based model was used to assess how runoff would collect within the identified depressions, move between storage areas, and ultimately discharge towards the SH6 culvert and the Corbridge site. The resulting attenuation and retained storage effects were then used to inform more representative runoff volumes and flow rates for these upstream catchments.

The catchments north of SH6 comprise the proposed development area and are expected to be substantially modified through future earthworks, grading, landscaping, lake formation, and stormwater infrastructure. Existing terrain depressions within this area may therefore be removed, reshaped, or replaced by purpose-designed storage and attenuation measures. For this reason, storage based on the existing northern terrain was not incorporated into the lake assessment. The northern catchments were retained as lumped catchment inflow hydrographs, representing the conservative assumption that runoff is not attenuated by the existing depressions. Their post-development storage characteristics should be reassessed once the proposed landform and stormwater design are sufficiently developed.

The current analysis accounts for initial rainfall losses but does not include ongoing infiltration from ponded water within the upstream depressions. If the soils retain appreciable infiltration capacity under saturated conditions, this process would reduce the modelled inflow from the southern catchments.

4.2.3 Runoff

No sustained baseflow was observed within the on-site catchments during the assessment. Accordingly, no baseflow component was assigned to the modelled reaches, and the simulated flows represent direct runoff generated by the adopted design rainfall events.

Design rainfall temporal patterns were developed in HydroCAD as 24-hour nested storm hyetographs using the adopted HIRDS rainfall data. Hyetographs were prepared for the 50%, 10%, 5%, 2%, 1%, and 0.4% annual exceedance probability (AEP) events, corresponding approximately to the 2-, 10-, 20-, 50-, 100-, and 250-year average recurrence interval events. Three-point smoothing was applied to the hyetographs to moderate abrupt changes between successive rainfall increments while retaining the overall storm depth and temporal distribution.

The resulting 24-hour incremental rainfall-depth hyetographs were then imported into HEC-HMS. For each design event, the corresponding 24-hour rainfall depth for the RCP8.5 climate-change scenario for the period 2081–2100 was applied, consistent with the rainfall basis adopted for this preliminary assessment.

HEC-HMS simulations were subsequently undertaken for each selected design event to calculate the runoff response from the contributing catchments. The model applied the adopted catchment loss, transform, and routing parameters to generate runoff hydrographs representing the variation in discharge over the duration of each 24-hour storm.

The resulting 24-hour catchment runoff hydrographs were exported from HEC-HMS and imported into HydroCAD to assess the interaction between storm inflow, available lake storage, soakage discharge, and the resulting fluctuation in lake level.

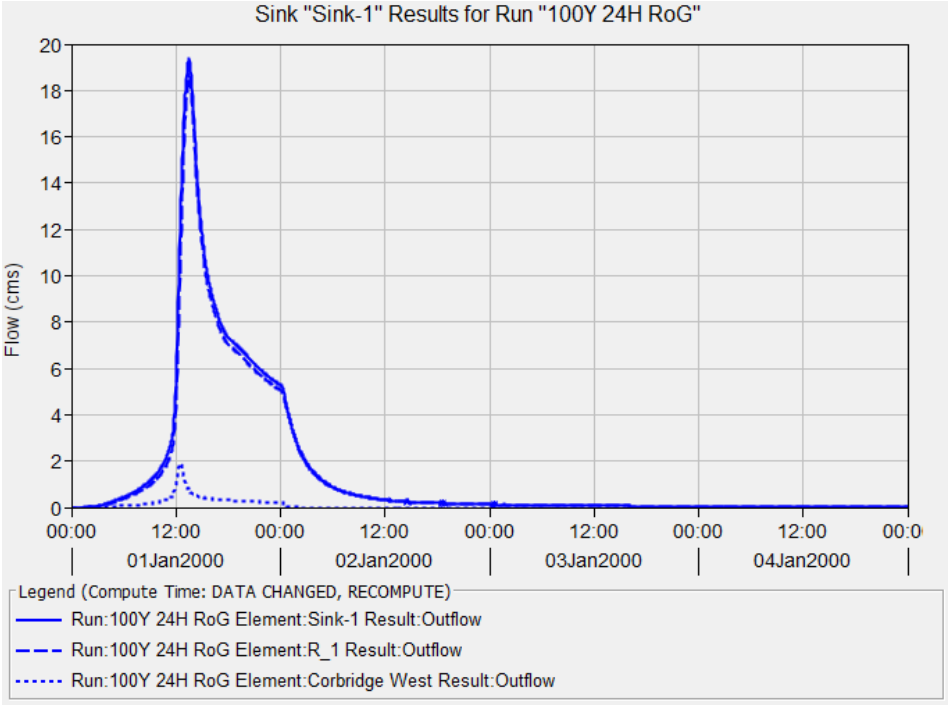


Figure 4. Modelled 1% AEP hydrograph from HEC-HMS for the combined inflow to the Lake

5. Lake Level Assessment

5.1 Process

A pond node was established in HydroCAD to represent the proposed central lake. The pond geometry was derived from the available existing contour information, and the catchment runoff hydrographs generated in HEC-HMS were applied directly to the lake node as inflows.

A preliminary stage–storage relationship was developed using the assumed lake footprint and available terrain data. The relationship defines the lake surface area and storage volume at successive modelled depths and forms the basis for calculating lake-level fluctuations during each design event. The adopted relationship is presented in the figure below. As the lake geometry remains conceptual, the stage–storage curve will require review and updating during subsequent design stages as the proposed lake footprint, side slopes, operating level, and surrounding landform are refined.

The stage–storage curve was extended above the anticipated lake operating level to represent temporary flood storage across the surrounding low-lying land. This allows the model to account for progressive inundation beyond the normal lake footprint when storm inflows exceed the available storage and soakage discharge capacity.

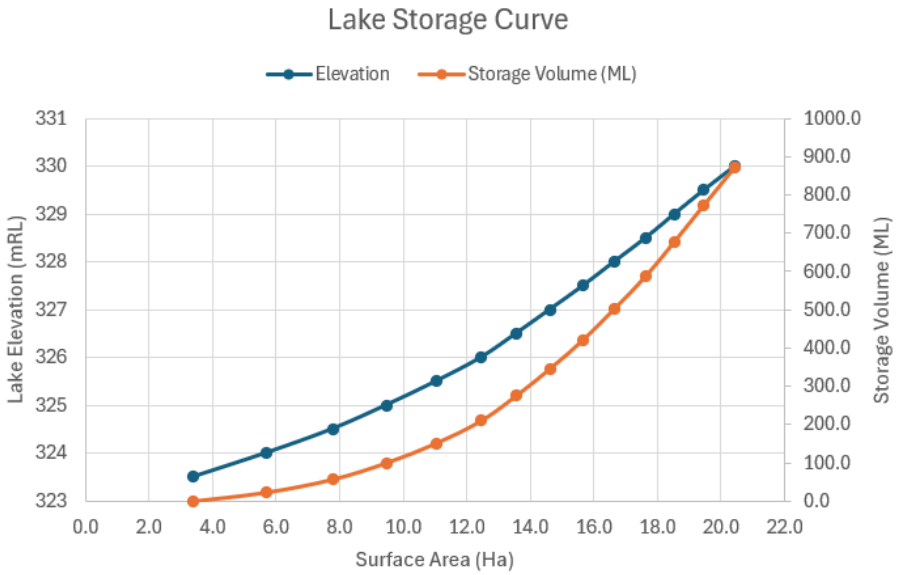


Figure 5. Preliminary Lake Stage Storage Curve

No allowance has been made for structures, landscaping elements, internal embankments, or other features that could displace water and reduce the effective storage volume within the lake or its surrounding flood-storage area. Evapotranspiration losses from the lake surface have also been excluded, as these losses are expected to be negligible over the duration of the modelled 24-hour design storms.

The lake node was routed to a separate HydroCAD node representing the proposed soakage field. Separating these components enables the assumed soakage rate and disposal area to

be varied independently, allowing the sensitivity of peak lake levels and the duration of elevated water levels to changes in soakage performance to be assessed.

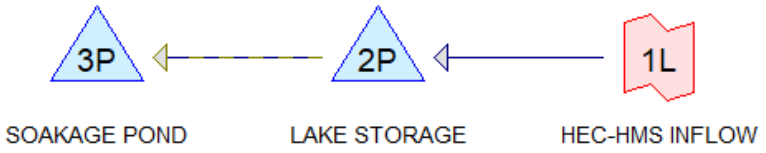


Figure 6. HydroCAD model arrangement

For the preliminary calculations, a direct hydraulic connection was assumed between the lake and the soakage field. This represents an operating arrangement in which the lake is maintained at a nominal normal water level and storm inflows above that level are conveyed to the soakage area for discharge to the underlying ground. The model also includes the temporary storage available above the soakage field, allowing runoff volumes that temporarily exceed the instantaneous infiltration capacity to be detained before infiltrating.

5.2 Soakage Rate

5.2.1 Previous Geotechnical Reporting

The available geotechnical reports contain a range of estimated soakage rates for soils encountered in and around the proposed lake area. These values are summarised below, together with the reported assessment method, location, and relevant soil type.

- Tonkin & Taylor (March 2012) visually estimated an infiltration rate of approximately 4×10^{-4} m/s (1,440 mm/hr) from test-pit observations near the western edge of the proposed lake. The material was described as permeable outwash gravel.
- GeoSolve (July 2021) visually estimated infiltration rates ranging from approximately 1×10^{-5} to 1×10^{-4} m/s (36 to 360 mm/hr) from test pits near the western edge of the proposed lake. The tested material was identified as outwash gravel.
- ENGEO (November 2022) estimated infiltration rates of approximately 800 to 1,000 mm/hr for outwash gravel and approximately 300 to 500 mm/hr for outwash sand. These values were estimates rather than the results of site-specific soakage testing.
- GeoSolve (June 2026) described the outwash gravels as having moderate to high soakage potential; however, no numerical infiltration rates were reported.

Overall, the geotechnical reports are consistent in indicating that glacial till and fluvial sediments are likely to have relatively low infiltration capacity, whereas the outwash gravels identified around the western edge of the proposed lake may provide substantially higher soakage rates. However, the lateral extent, thickness, continuity, and depth of these more permeable gravel deposits have not yet been confirmed. The reported values should therefore be treated as preliminary estimates until targeted field investigations and soakage testing are completed within the proposed disposal area.

5.2.2 Soakage Rate Reduction

In accordance with the 2025 QLDC LDSCoP, the measured or assessed soil infiltration rate must be reduced by an appropriate combined safety factor, $F_{(total)}$, to derive the design infiltration rate used for sizing and performance assessment. This adjustment allows for uncertainty in the field-test results and the potential for soakage performance to reduce over the design life of the system. The design factored soil infiltration rate, K, has therefore been calculated from the tested soakage rate, k, using the following relationship:

$$\mathbf{K = k / F_{(total)}}$$

The appropriate combined safety factor should be confirmed during subsequent design stages with reference to the consequence of failure, the quality and representativeness of the infiltration testing, anticipated variability in the receiving soils, the proposed system configuration, and the ability to inspect and maintain the soakage area. These matters cannot be fully resolved until targeted testing has been completed and the lake outlet and soakage system have been developed in greater detail.

For this preliminary assessment, a combined safety factor of 2.0 has been adopted for a design soakage rate of 700 mm/hr. This provides an initial allowance for uncertainty while enabling the practical implications of the proposed lake and soakage arrangement to be assessed. As soakage is currently assumed to be the sole means of lake outflow, a higher safety factor may ultimately be warranted to reflect the consequences of reduced infiltration performance or progressive clogging. The adopted value should therefore be reviewed following site-specific soakage testing, confirmation of the extent and continuity of the permeable outwash gravels, and development of the proposed outlet, redundancy, maintenance, and overflow provisions.

5.3 Lake Level Fluctuation

As expected, the size of the soakage field directly affects the calculated lake-level fluctuation. The figure below shows the relationship between the soakage area provided and the calculated maximum lake-level fluctuation.

No allowance for the storage volume within the soakage field media has been included

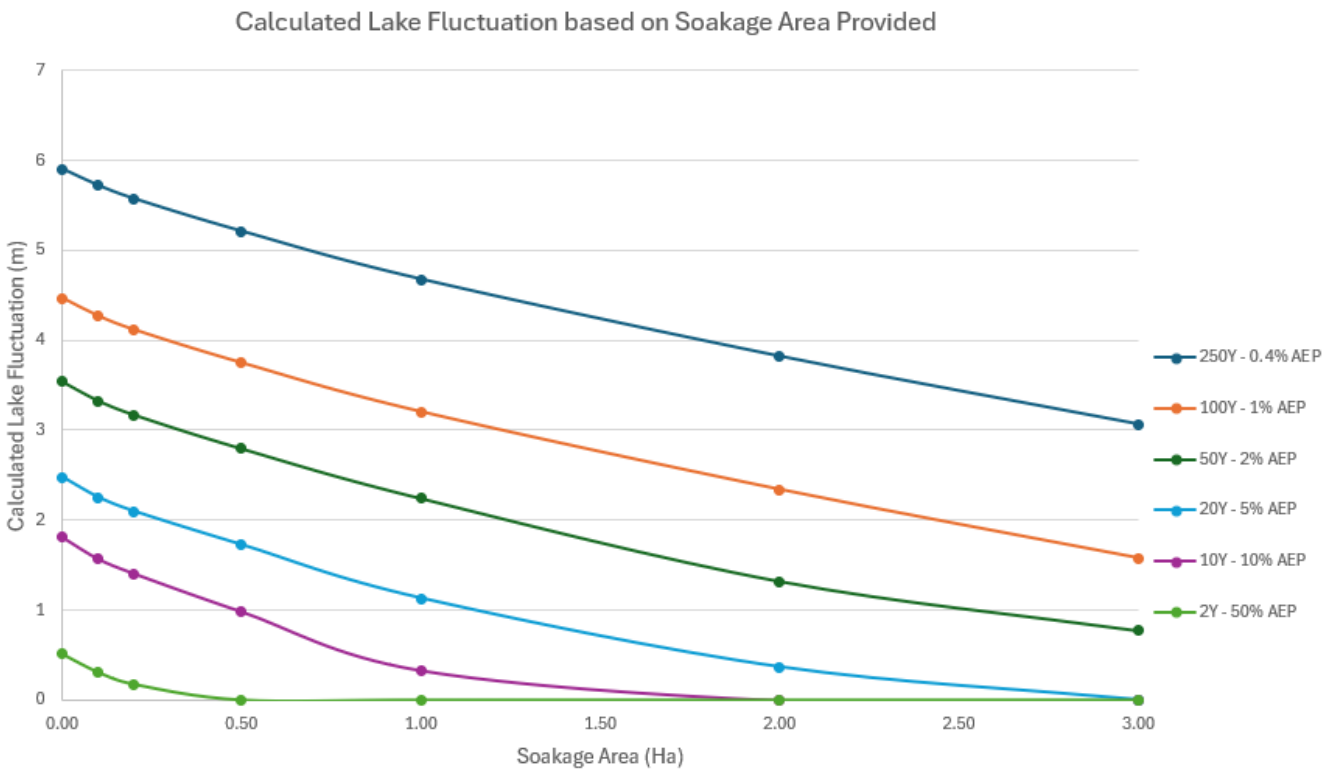


Figure 7. Calculated preliminary lake fluctuation based on soakage area provided

For the modelled 1.0 ha soakage field, the calculated maximum lake-level fluctuation is approximately 3.2 m during the 1% AEP event and 4.7 m during the 0.4% AEP event. These results represent modelled depths above the adopted stage–storage base and are preliminary because the lake geometry, operating level, soakage performance, and surrounding landform have not yet been confirmed.

The preliminary calculations indicate that approximately 3.0 ha of soakage area would be required to limit the modelled lake-level fluctuation to less than 1.5 m during the 1% AEP event. With this soakage area, the calculated fluctuation during the 0.4% AEP event remains approximately 3.0 m.

These results are provided for preliminary assessment purposes only. Their interpretation is subject to the following assumptions and limitations:

- The modelled lake is based on the existing basin in the topography on the site. Modifying this to reduce the footprint and available storage area will lead to a corresponding increase in the lake level fluctuation.
- The runoff assessment adopts Hydrologic Soil Group B and a curve number of 61. We are not aware of local modelling that calibrates the curve number for soils within the Upper Clutha Basin. Further assessment is required to confirm the appropriate hydrological soil group and curve numbers, as changes to these parameters would affect the calculated lake-level fluctuations.

- Detailed assessment of soakage rates and any potential groundwater interactions are still to be undertaken. The capacity of the receiving soils may have significant effects on the stormwater/lake soakage volume.

6. Next Steps

This preliminary assessment estimates runoff to the proposed lake, the resulting range of modelled lake-level fluctuations, and the indicative soakage areas required for disposal to ground. The results are intended to inform the plan change and subsequent design development; they do not constitute detailed design or confirmation of final lake, soakage, or overflow levels.

Further detailed assessments will be required to develop the design of the proposed central lake, which should include the following aspects:

1. Quantify the storage volumes and runoff-routing effects within the post-development Corbridge site catchments.
2. Include any impervious areas that discharge directly to the lake, as well as any effects of stormwater management and disposal infrastructure elsewhere on the site.
3. Review the adopted hydrological soil groups and curve numbers against available site information and observed catchment responses, where suitable monitoring data are available.
4. Undertake soakage testing in identified soakage disposal areas to confirm previous visual assessment of soakage rates.
5. Confirm the extent and depth of the outwash gravels on the western edge of the proposed lake area.
6. Adjust the lake model to account for the development in the size, shape, slope etc of the proposed lake and surrounding areas.

Appendix A: Limitations

This report has been prepared solely for the benefit of our client, SIO No12 Limited, as per our brief and an agreed consultancy agreement. The reliance by any other parties on the information or opinions contained in this report shall, without our prior agreement in writing, be at such parties' sole risk.

The conclusions and recommendations contained within this report are based on the investigations as described in detail above. The nature and continuity of subsoil conditions are inferred and it must be appreciated that actual conditions could vary considerably. Defects and unforeseen ground conditions may remain undetected which might adversely affect the stability of the site and the recommendation made herein.

This report has been prepared solely to address the issues raised in our brief, and shall not be relied on for any other purpose.

Where we have provided comments on aesthetic issues these need to be confirmed by an architect or other expert in the field.

In the event the third party investigation data has been provided to us, the client acknowledges that we have placed reliance on this information to produce our report and CGW will accept no liability resulting from any errors or defect in the third party data provided to us.