BEFORE THE HEARINGS PANEL

IN THE MATTER	of the Resource Management Act 1991
AND	
IN THE MATTER	of proposed Plan Change G: Aokautere Urban Growth to the Palmerston North City Council District Plan

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SECTION 42A TECHNICAL REPORT OF ALLISON REIKO BAUGHAM AND TONY MILLER ON BEHALF OF PALMERSTON NORTH CITY COUNCIL

TECHNICAL – STORMWATER

Dated 15 September 2023

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A. EXECUTIVE SUMMARY

1. Provided the recommendations set out in this s 42A report are implemented, we consider that stormwater effects from proposed Plan Change G (**PCG**) will be managed by the proposed mitigations, which are intended to achieve:

    (a) Hydraulic neutrality;

    (b) A match of pre-development erosion threshold exceedance; and

    (c) Treatment of the 90th percentile rainfall volume from impervious developed areas.

2. In our opinion these design criteria will ensure an acceptable level of stormwater effect from PCG, and will benefit the receiving environment, through improved management of stormwater, including from existing Aokautere development, and gully restoration.

B. INTRODUCTION

Author 1 – Allison Reiko Baugham

3. My full name is Allison Reiko Baugham. I hold a dual degree in Civil Engineering and Engineering & Public Policy from Carnegie Mellon University (USA) received in 2008, and a Master of Engineering degree in Environmental Engineering & Water Resource Systems from Cornell University (USA) received in 2009.

4. I am a Chartered Professional Engineer (CPEng) of Water engineering, with my practice area being investigation, design and construction management of water, wastewater and stormwater reticulation systems. I am also a licensed Professional Engineer (PE) in the state of New York, USA.

5. I am a Senior Water Engineer with 14 years' of experience in the planning and design of three waters infrastructure. In addition to infrastructure design, my experience also includes hydraulic / hydrological modelling of stormwater for local government. I transferred to New Zealand in 2013 and have mainly been involved in projects for local councils in the Manawatū-Whanganui region.

6. I have served as a Consultant Engineer for Palmerston North City Council (**Council**) since 2017, assisting in a myriad of capital works projects relating to stormwater and planning across the city. As part of my role, I have assisted in preparing the stormwater servicing assessments for multiple Plan Changes and providing expert evidence on behalf of the Council. I have also provided assistance to the Stormwater Services Manager and Development Team since 2016 reviewing resource consents, subdivision plans and stormwater management plans.

7. In addition to my work at Council, I have also been involved in developing the Stormwater Master Plan for Whanganui District Council, various structure plans and servicing assessments for Whanganui and Manawatū District Councils, and have undertaken impact assessments for Whanganui, Manawatū and Horowhenua District Councils as they relate to growth and its effects on stormwater management.

8. I have been engaged by Council in relation to PCG, which seeks to rezone a new greenfield growth area in Aokautere for residential development and insert an accompanying Structure Plan and provisions (objectives, policies and rules) into the District Plan.

9. I have been involved with PCG since 2020. My role has involved providing technical advice in the area of stormwater management and mitigation strategies.

10. As part of my role I authored, co-authored, and/or served as Project Director for the following reports:

    (a) *Stormwater Management Strategy: Plan Change G - Aokautere* (GHD, 23 May 2022) (the "**Stormwater Management Strategy**")[1] (**Appendix A**). This report documents the stormwater analysis undertaken to determine the effects of PCG and the proposed stormwater management strategy to enable development.

    (b) *Stormwater Expert Evidence – Stream Erosion Assessment Summary* (Rev 1, 22 June 2023) (**Appendix B**). This technical memorandum summarises the erosion in the gullies and associated risks, and the implications this has on PCG and

[1] Appendix 11, Section 32 Report, July 2022.

existing development. Preliminary recommendations for mitigation were proposed in this memorandum, but have since been updated through a multi-disciplinary process involving the PCG technical expert team.

(c) *Proposed Stormwater and Stream Erosion Mitigation* (28 August 2023) (**Appendix C**). This memorandum followed a number of workshops and discussions between the Council team of experts including Dr Adam Forbes (ecology), Mr Eric Bird (geotechnical), Mr Andrew Burns (urban design) and Ms Copplestone (planning) to discuss the preliminary stormwater controls identified. This led to further refinement and update of stormwater management controls within the gullies to ensure the effects mitigation hierarchy was applied to its fullest extent possible.

(d) *Technical Memorandum - Results of additional modelling incorporating the proposed mitigation measures* (September 2023) (**Appendix D**). This memorandum documents the details added to the hydrologic and hydraulic models developed to inform the updated Stormwater Management Strategy. The modelling has informed updated sizing and locations of the detention ponds and sizing of bioretention devices in order to meet the design criteria.

11. I am familiar with the site for PCG, having visited it on 7 November 2022 and 4 July 2023. These site visits focused on the gullies and various receiving systems from PCG, taking note of the existing water features and evidence of erosion.

Author 2 – Tony Miller

12. My name is Anthony Thomas Miller. I hold a degree in Engineering (Civil) from the University of Auckland received in 1980.

13. I was a Chartered Professional Engineer CPEng from 2002. My membership has recently lapsed as I am near retirement. My practice area involves investigation, design and construction management of rivers and coastal protection, surface waters as well as water, wastewater, and stormwater reticulation systems. I am also a licensed International Professional Engineer (IntPE).

14. I am currently employed by GHD (and have been since 1990). My current role is Technical Director (3 Waters).

15. I have 50 plus total years of experience with 40 plus years in the civil engineering field. My background is in farming and forestry just north of this site and as such I am familiar with the topography, geology, soil types and rainfall patterns of the area.

16. My experience as a civil engineer has spanned both construction and consulting engineering with an emphasis around 3 Waters. I have a specific interest in surface water, from investigation and modelling, river scour, coastal erosion and revetment systems, stream rehabilitation, to dam and hydro engineering including stormwater and irrigation pumps and their conveyance pipelines. In addition to infrastructure design my experience also includes hydraulic / hydrological modelling of stormwater for local government. I have worked in the UK, North America, Australia, and the Pacific, with most of my career in New Zealand.

17. As stated above, I grew up in the Manawatū and since 2015, I have been involved in several Palmerston North based projects, including for the Council.

18. I have been involved with PCG since 2021. I was originally the mentor for the author of the Stormwater Management Strategy notified with PCG, Mr David Arseneau. Following Mr Arseneau's departure from GHD in late 2022, I have continued as the Stormwater Discipline Lead, alongside Ms Baugham on the plan change.

19. As part of my role, I authored or co-authored the reports referred to in paragraph 9(b), (c) and (d) above. I have also reviewed the Stormwater Management Strategy, before completing the additional work, alongside Ms Baugham, described in this report.

20. I am familiar with the site for PCG, having visited it on three separate occasions since becoming the stormwater discipline lead in March 2023. These site visits focused on the PCG and adjacent catchment areas, the gullies to which these catchments discharge to and various receiving systems from PCG, taking note of the existing water features and evidence of erosion. I have also been employed through GHD to provide advice to the Council development engineer(s) on a number of individual stage developments currently under design/construction within (and outside of) the PCG area.

C. CODE OF CONDUCT

21. Both authors make the confirmation below.

I confirm that I have read and agree to comply with the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023. This technical report has been prepared in accordance with that Code. I confirm that I have not omitted to consider material facts that might alter or detract from the opinions that I express. The opinions I express are within my area of expertise, except where I state I am relying on the opinions of other reporting officers.

D. SCOPE

22. This s 42A report addresses the stormwater management strategy in relation to PCG, including the following issues:

    (a) The effects of development in relation to stormwater runoff, flooding and erosion; and

    (b) The relevant planning framework.

23. In preparing this report, we have had regard to the *Engineering Standards for Land Development*, Palmerston North City Council, Fourth Edition, March 2023.

24. In addition to our own observations, we have relied on the following reports:

    (a) *Aokautere Structure Plan: Ecological Features, Constraints and Restoration* by Forbes Ecology, July 2021. See Appendix 7, Section 32 Report, for PCG.

    (b) Aokautere Slope Stability: considerations for consenting, Tonkin+Taylor, May 2022. See Appendix 9, Section 32 Report, for PCG.

    (c) The s 42A technical reports prepared by Dr Adam Forbes, Mr Eric Bird, Mr Andrew Burns and Ms Anita Copplestone.

25. We have reviewed submissions and further submissions on PCG. Of particular note when considering our combined fields of expertise are submissions relating to:

    (a) Natural hazards – stormwater, flooding and erosion; and

(b) Climate change mitigation and adaptation.

E. BACKGROUND

26. PCG seeks to rezone a new greenfield growth area to the south-east of Palmerston North for residential development and inserts an accompanying structure plan and provisions (objectives, policies, and rules) into the District Plan. The plan change will provide for additional housing supply in Aokautere (and the City), to help meet growth projections for Palmerston North over the medium to long term, while addressing the specific topography and environmental issues in Aokautere.

27. Aokautere, the plan change area, exhibits a natural hilly terrain with a network of gullies and plateaus. The gullies ultimately discharge into larger streams that originate in the Tararua Range, specifically the Moonshine Valley Stream and the Turitea Stream, which flows around the plan change area to the Manawatū River.

28. Although the plan change area is largely used for agricultural purposes, there are several areas of existing residential developments dating from the 1980s and onwards. Several small gullies have also been filled during construction of the existing developments. This existing development (and existing natural conditions) have led to poor stormwater outcomes, in terms of both downstream flooding and erosion.

29. Given these conditions, stormwater management has been understood as a key issue in planning for development of the plan change area. The Stormwater Management Strategy was one of the key inputs into the Aokautere Masterplan process.

30. Following the receipt of submissions and further submissions on PCG, we have revisited the Stormwater Management Strategy (and underlying modelling) and undertaken some further analysis with regard to:

    (a) The highly erodible geology of the area, which has been subject to deposition and recent uplift (recent in the geological timeframe context);

    (b) The changing climate and potential for larger and more frequent intense storms;

(c) The combined effects of urbanisation and whether the design criteria within the Stormwater Management Strategy, the accompanying proposed PCG provisions (including assessment criteria) and the Council's Engineering design standards are appropriate to mitigate the effects of development; and

(d) The effectiveness of proposed mitigation measures to control stormwater discharge to the receiving environment as set out in the PCG application documentation, including the cumulative effect of the pond discharges (when considered as one system) within the gully network (particularly Gully 1 and Gully 3), and whether further measures were required to meet the design criteria within the Stormwater Management Strategy.

31. The generalised implications of urban development from a stormwater perspective are:

(a) Increased areas of impermeable surface leading to increased volume of stormwater runoff from the land over a shorter period;

(b) Reduced volume of stormwater infiltrating to the ground;

(c) Downstream effects such as higher flows from comparable storm events leading to faster and higher flows with more erosive power; and

(d) Upon reaching the flatter and lower areas of the catchment system, increased depth and ponding of this runoff leading to flooding. (We note that there are no habitable areas / dwellings in this lower portion of the catchment that would be susceptible to an increase in flood depths).

32. The specific implications of urbanisation for the PCG area and adjacent catchments are:

(a) The runoff from the catchment even prior to (and without) the proposed development will cause the gully floor to continue to down cut and erode;

(b) Left unmitigated, increased runoff from the catchment will accelerate the rate of erosion; and

(c) As down cutting continues, this will lead to valley sides/valley walls to become over steep leading to failure and slips. This slip debris will eventually reach the valley floor where this previously mobilised material will be removed by subsequent runoff flow in the streams.

33. The mitigation measures proposed as part of the Stormwater Management Strategy, supported by the further measures identified following our (and other experts') further assessment, are intended to address the above effects within the PCG area and adjacent catchments. We discuss these in greater detail below.

F. INITIAL STORMWATER ASSESSMENT

34. The Stormwater Management Strategy evaluates the risks to and impacts of development, as well as recommends the mitigation measures to appropriately service PCG. The stormwater assessment informing the Stormwater Management Strategy considered the flood risk, erosion risk and water quality requirements for the proposed development and recommended the following design criteria:

(a) Control of runoff peak flows to pre-development levels[2] for the 2-year, 5-year, 10-year, 20-year, 50-year and 100-year ARI flows, to control flood risk (i.e., hydraulic neutrality).

(b) Further control of peak flows as needed to match the pre-development erosion threshold exceedance, measured using a cumulative effective work index, in the Aokautere Church Stream (Gully 1), Moonshine Valley Reserve Stream (Gully 3), and Tutukiwi Reserve Stream.[3]

(c) Treatment of the 90th percentile rainfall volume from impervious developed areas through a stormwater treatment device or multi-device system.

35. The assumptions and methodology around the initial modelling carried out to inform the original Stormwater Management Strategy are outlined in the 2022 report.[4] Notably, pre-development conditions were established as the land conditions prior to any residential development within PCG area, including those limited areas of development constructed over the previous 20 to 30 years. As we discuss later in our

2 Pre-development levels are described in paragraph 36.
3 The 'Cumulative Effective Work Index' and its use in measuring erosion threshold exceedance is described in the Stormwater Management Strategy at 4.2.2.
4 At 1.4 and 3.

report, existing development represents approximately 52% of the PCG area. The scope of the strategy reflects the intention "*…to effectively address all stormwater runoff in the study area and avoid the "grandfathering" of existing areas which would then incur a disproportionately high impact to the receiving environment*."[5]

36. The concept design for the Aokautere stormwater management system was developed to mitigate the flood, erosion and water quality impacts identified and quantified in the Stormwater Management Strategy as a result of both future development and residential development in the last 20 to 30 years. In addition to meeting the proposed design criteria (including addressing the effects of existing and new development within the Aokautere catchments), the stormwater mitigation measures also needed to be responsive to design and ecological/environmental constraints. Relevant factors informing the nature and location of stormwater management measures, including the location of stormwater detention ponds, were (in summary):

(a) The presence of highly erosive soils within the gullies, making them susceptible to erosion and sensitive to any changes in flow;

(b) Slope stability, which impacted setback requirements and necessitated the capture of runoff along the gully slopes;

(c) Perceived stream degradation;[6] and

(d) Existing vegetation, streams and wetland areas including sensitive environmental areas (i.e., areas of high vegetation or ecological value), including those identified for PCG by Council's ecologist, Dr Adam Forbes.

37. The concept design for the mitigation works was completed by our former colleague, Mr David Arseneau, and was included in the Stormwater Management Strategy. The mitigation proposed to meet the design criteria described above, included:[7]

(a) stormwater detention facilities to mitigate both flood and erosion risk;

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5 Stormwater Management Strategy, at 3.1.

6 When we say the stream degradation is 'perceived', we are referring to the attribution of that stream degradation to the development enabled by PCG – its *cause* is perceived. Our intent in using this wording is to convey that even if the effects of the development were mitigated, erosion would still continue.

7 Stormwater Management Strategy, at 5.3

(b) roadside bioretention facilities;

(c) a perimeter swale drain / buffer area at the top of the gully slopes to intercept surface runoff and protect the gully slopes from erosion;

(d) stormwater reticulation designed for the 10-year ARI climate change flow (primary system); and the overland flow network[8] designed for the 100-year ARI climate change flow (secondary system);

(e) controlled discharge for the 100-year ARI climate change flow to the gully floor (i.e., no direct overland discharge down the gully slope); and

(f) consideration of stream stabilisation to mitigate perceived impacts from the development[9] and enhance aquatic habitat and community amenity.

38. The above measures have informed the master plan process for PCG, including the structure plans and accompanying planning framework. Ms Copplestone addresses how these matters have been provided for within PCG, including the plan provisions. Our evidence discusses specific measures, like the stormwater buffer area, in response to submissions below. However, in our opinion, subject to the additional recommendations set out within this report and amendments to the plan provisions highlighted by Ms Copplestone, we consider that PCG gives effect to the Stormwater Management Strategy, and that the Structure Plan and plan provisions are appropriate.

G. FURTHER ASSESSMENT

39. Following notification of PCG and receipt of submissions, concerns were raised around the erosive nature of the land including the gully systems and the effects of the development within the PCG area, and the function and location of the stormwater ponds. Submitters sought further assessment to better understand and provide further detail on the risks posed to the receiving system and mitigation required to address any effects.

8 Piped networks are considered the primary system with a defined level of service (the pipes are sized for a certain event). In order to cater for those "over design" events, best practice is to include a secondary system in the form of designated overland flow paths.

9 The Stormwater Management Strategy acknowledges that Aokautere Church Stream and Moonshine Valley Reserve Stream will remain highly sensitive to erosion in the future regardless of upstream development.

40. These submissions are addressed individually at Section H of this report– however, in summary, we have considered the following issues through further assessment/analysis:

(a) Confirmation of the modelling assumptions informing the initial Stormwater Management Strategy, to ensure that effects were managed;

(b) The potential for an over design event (i.e., the potential for storm events of greater magnitude with either higher intensity rainfall rate, or greater volume of total rain than that allowed for in the design) and any consequences of those events on the Stormwater Management Strategy and mitigation measures;

(c) The relevant assessment period. This has been adopted as being the likely impacts over the next 100 years, with allowance for climate change;

(d) Whether the original modelling was sufficiently conservative, given the highly erosive soils in the area, and the potential for scour and erosion to have an impact on gully floors over the next 100-years;

(e) Review of the original Stormwater Management Strategy and proposed mitigation measures and determine its/their effectiveness including an assessment of whether further and/or different mitigation measures are required (including review of instream mitigation measures) to manage and limit the existing and future development impacts over the design period; and

(f) Controls to ensure the proposed stormwater buffer / perimeter swale is protected at all times and that the runoff intercepted by this collection system will be conveyed to the base of the valley floors and released in a controlled manner with suitable energy dissipation.

41. The methodology and modelling for the above analysis is described in **Appendix D**.

Rainfall

42. In response to the submissions on rainfall and the changing climate, further analysis has been undertaken on rainfall. This involved review of rain gauges near the PCG area, including Scott's Road gauge to the south and two further gauges in the Ruahine ranges

to the north. The period of assessment includes 2004, 2015 (in which there were significant rain events) and the recent Cyclone Gabrielle in 2023.

43. In consideration of these significant rain events, the peak intensity adopted for the 1% AEP event used in the modelling for the Stormwater Management Strategy (i.e., 100-year ARI including climate change) was reviewed and is confirmed to have a higher total rainfall and higher peak intensity than the local historic rainfall reviewed.

44. We are satisfied that the 100-year storm (including an allowance for climate change in accordance with PNCC's Engineering standards) in the modelling reflect the <u>likely</u> maximum depth and intensity over the next 100-year design period. This is considered accepted best practice.

Geology

45. A further review of the geological setting of the area was also undertaken to understand the effects currently being observed (as raised in submissions) in the gullies and valley walls. The gully material is geologically young consisting of recently laid and recently uplifted material to form the river terraces seen today. The material has not undertaken any cementation or binding or metamorphic process. As such the material on the valley walls and gully floor is highly susceptible to erosion, with multiple instances of this erosion being apparent to the authors during their respective site visit(s).

46. As such, the Stormwater Management Strategy identified controls to not only consider the flood risks, but the erosive risks as well. The strategy has been set to match pre-development flows within Aokautere as it relates to the erosive forces of the flow; that is, the rate of erosion is not accelerated as a result of development. It is also important to note that erosion within the gullies will continue (that is, erosion is occurring regardless of development). Submissions have raised this continuing erosion within the gullies as a concern, and the stream erosion risk has been further considered by the authors of this report for the purpose of informing responses to submissions.

47. This subsequent assessment identified the following matters:

(a) While the proposed stormwater controls proposed as part of PCG mitigate the effects of each sub-catchment, their cumulative discharge (i.e., all of the proposed ponds operating at once) needed to be considered further;

(b) The extent of erosion likely to occur regardless of development will have effects that are more than minor; and

(c) The consequence of the continuing erosion regardless of development, is likely to cause slope instability and erosion over the design period of 100 years.

48. As erosion continues, it is our opinion that the streams will experience further downcutting. This will occur regardless of development within the plan change area; development will only affect the rate at which downcutting occurs. As such, the consequences of any downcutting was reviewed by Mr Miller to understand the implications it may have on not just the PCG area, but the wider area as well. This assessment estimated the predicted range of downcutting expected over the next 100 years, with the results set out in Appendix D.

49. Following on, the change in the gully floor and impact on the toe of the gully slope was evaluated by geotechnical experts. Mr Bird, Council's geotechnical expert, addresses downcutting and geotechnical matters further in his s 42A report.

50. Based on the predicted range of downcutting, Mr Miller reviewed the model to confirm the design criteria used and to check whether the effects were being appropriately managed. As with the original 2022 modelling, the pre-development scenario (i.e., land use to the year 2000) was confirmed to be prior to residential development, with the land assumed as being farmed. In addition, the pre-development scenario has used historical rainfall data, with post-development accounting for climate change. This not only provides a certain level of conservatism, but also helps mitigate any effects observed within the gullies arising from storm event size increases attributable to climate change as well as managing the effects of residential development within the past 20 years.

51. The modelling has assumed (as was the case originally) that the proposed stormwater mitigation will limit peak flows up to the 90th percentile storm to pre-development levels while also limiting flows as needed to match the pre-development erosion

threshold exceedance (measured by a cumulative effective work index). This has been evaluated not only for the individual design events nominated in the design criteria set out above, but also using a continuous five-year rainfall simulation. The effects of a fully developed catchment during large rain events, with all stormwater ponds within a catchment having reached full capacity and discharging in unison (i.e., the cumulative effects during large rain events) have also been assessed and refined further since the original modelling to better represent the outlet controls. In doing so, the proposed controls are tested under realistic conditions where back-to-back rainfall events are likely to occur. The modelling results demonstrated the need for additional storage capacity within the ponds, and the storage volumes have subsequently been updated.

52. As stated above for the 90th percentile storm, appropriate attenuation and graduated release of stored water over time to match pre-development conditions has been achieved. However, in larger events (e.g., 2-year, 10-year and 100-year events) the ponds are filled, and substantial flows occur through the dam service and emergency spillways. The impact of the flow exceedances in uncommon and rare events means that the rate of spill and flow will increase, depth and velocity will also increase substantially, however, the duration and frequency of these infrequent and rare events will be for limited periods of time. The revised Stormwater Management Strategy (which we discuss below) seeks to address these increases in all events up to the 100-year event using a series of mitigation measures.

53. As we have noted, refinements and additions to the stormwater controls proposed in the Stormwater Management Strategy have been proposed to supplement the original stormwater concept design. These changes will ensure that the design criteria and objectives set out in the Stormwater Management Strategy are met,[10] while accounting for the updated modelling (including the predicted range of downcutting).

54. The selection and design of these mitigation measures has been the consequence of a collaborative multi-disciplinary process. Through collective exploration and review of potential stormwater mitigation options, avoidance and nature-based solutions have been prioritised as far as possible.[11] The agreed approach focuses on first avoiding, then

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[10] Stormwater Management Strategy, at 5.1.

[11] These mitigations were designed following planning and ecological input from Ms Copplestone and Dr Forbes, respectively.

reducing and minimising the generation of adverse effects, through on-site control of stormwater contaminants and flows, planning controls, restoration of natural systems, and infrastructure works. The stormwater management measures attempt to minimise the need for watercourse stabilisation and restoration, in recognition of the important terrestrial and aquatic values in these ecosystems. The stormwater controls also minimise any loss of stream length to where it is only functionally necessary.

55. The proposed stormwater management controls include:

(a) Revegetating the gullies to help reduce the risk of erosion;

(b) Avoiding direct discharge of stormwater runoff over the gully slopes through a stormwater perimeter swale prescribed within the District Plan provisions;

(c) Reducing the impervious area limit as much as practicable to be 40% minimum permeable area for suburban areas and 25% for medium density. Based on the densities proposed in the Structure Plan as notified, as well as the existing level of development, the overall permeable area is estimated to be 37%, which is higher than the operative standard currently set in the District Plan of 30%;

(d) Implementing water sensitive design elements to retain / reduce stormwater runoff on the plateaus, as well as incorporating additional storage within them to reduce the size of detention ponds (refer below);

(e) Incorporating larger detention ponds that avoid areas of moderate to very high vegetative constraints and are located offline as far as practicable; and

(f) Implementing in-stream stabilisation and erosion protection measures in limited reaches to reduce steep gradients and slow flow velocities.

56. The impact on the upper gully systems are small and no further mitigation has been considered necessary with the exception of updated pond volumes. However, in the lower reaches of the larger catchments, namely Gully 1 and Gully 3, some additional instream stabilisation and erosion protection measures have been further considered. These measures include:

(a) Below Ground Dams (BGD). These have been included as an option to keep the low flow and high flow within the gully controlled to discharge at a particular and constant location and elevation within the gully floor. The BGD prevents downcutting from occurring upstream from its constructed location. The BGD allows for downcutting to continue on the downstream side of the dam through naturally occurring processes and this structure can include a spillway and fish passage to control the flow of the water.

(b) Cascade Weirs (CW). These engineered structures are constructed in the bed of the valley and accommodate both low flow but also high and extreme flow rates. They generally include a series of pools stepping down the valley and include for low flow passage of fish between each step up in the pool sequence. Each structure would fully contain a 10-year flow. In event of higher flows some maintenance is expected.

(c) Attenuation/Detention Dams (DD). In addition to SW treatment, attenuation and detention dams have been included as mitigation options within the assessment. These DD receive low, medium and high inflows of runoff, store the accumulated water, then release the flow over an extended period. They work well until the dam is full in this regard.

57. The above engineering works (BGD's, CW's and DD's) are designed to reduce velocities and sediment transport during frequent rainfall events. The proposed stormwater controls within the gullies, and potential stream impacts, are set out in Appendix C.

58. Care has been taken to avoid and limit works and structures in and around the waterways in the gully systems as far as possible. All proposed works are (conceptually) designed to ensure surface water flows are achieved and maintained and fish passage will be provided for and maintained. Where in-stream works are required, Dr Forbes has assessed freshwater offsets (at a high level) with regard to any residual adverse effects, and it appears there is sufficient freshwater habitat to achieve a no-net-loss, or net-gain, position for freshwater biodiversity within the plan change area .[12]

12 Section 42A Technical Report - Dr Adam Forbes dated 15 September 2023, at paragraph 1(d).

59. For completeness, we note that as part of the options analysis described above, stormwater controls that were discounted include:

(a) Rainwater tanks. While rainwater tanks are considered a sustainable solution in terms of water management, the correct use of rainwater tanks for onsite detention is difficult to regulate and for this reason are not recommended as a central stormwater control method for large greenfield development. For example, rainwater tanks are often used for non-potable water purposes, so individual landowners may modify the rainwater tanks to store water and therefore will not have any capacity during rainfall events. Further, since these will be privately owned it is difficult for Council to ensure they are kept empty in order to receive and store runoff in the next rain event.

(b) Permeable pavements. Large areas used to retain water onsite through the act of infiltration (e.g., permeable pavers) could result in super-saturated soils. Given the known geotechnical risks, this was not a recommended site-wide management strategy. However, this could still be implemented to reduce the amount of storage required if supported with sufficient geotechnical advice.

60. The Stormwater Management Strategy manages the entire plan change area, including existing residential development, and provides for water quality treatment and attenuation of runoff prior to discharge. It will directly lead to the vesting and protection of the natural stream systems within the gullies over the long term for conservation and amenity purposes. The need for stream restoration/stabilisation work has been considered in greater detail following submissions, with further recommendations incorporated into the Structure Plan (with a revised slope setback, as discussed in Mr Bird's s 42A report) and the Stormwater Management Strategy to address the ongoing existing erosion issues in the Aokautere area.

61. Once the mitigation is adopted, it is our opinion that the Stormwater Management Strategy will work to ensure the original design criteria are met.[13]

[13] As set out above at paragraph 34.

H. SUBMISSIONS

62. We have identified a number of issues raised in submissions and further submissions, which we address in detail below.

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
(1) Increase in flooding and erosion in Moonshine Valley Stream • Inga Hunter, S17.001 • R & J Stevens, S34.001 • Nathan Meyer, S36.001 • A & R Gear, S39.005 and 006 • R & A Gear, FS15.004 • Gill Welch, S49.004 • Larry Harrison, S53.001 • Barry Scott, S54.002 • Elizabeth Fisher, S80.001 • Gaylene Tiffin, S85.001 • Colin Perrin, S90.002 • Sara Burgess, S98.003	The addition of housing above Moonshine Valley Road will result in additional stormwater runoff to an area that is already vulnerable to erosion. S17 expressed concerns around the functionality of the detention ponds, especially during winter. Some submitters also expressed concern around the multi-unit dwellings proposed and the increase in stormwater runoff. S39 raised particular concern regarding the effects of additional runoff and erosion to Bryant's Bridge on Aokautere Drive. Decision requested is to prevent housing between the last gully before the start of the downhill slope and the hill, and for this area to be made a reserve instead. FS15, S39, S80, S90 and S98 requested a minimum section size of 1ha; S53 requested a minimum area of 0.5ha.	The Moonshine Valley Stream has highly erosive soils, and is therefore vulnerable to increases in flows. While there have been requests for increases in lot size to help reduce the downstream effects, it should be noted that downcutting is anticipated to occur regardless of development. This is because of the properties of the soils present. In our opinion, limiting further development will not accelerate the erosion, but it also will not prevent future erosion. The Stormwater Management Strategy proposes to help mitigate the effects of existing development, thereby improving the erosion currently being witnessed by residents off Moonshine Valley Road. Based on the lot pattern and density maps as notified (Map 7A.4B), existing development represents approximately 52% of the PCG area. The Stormwater Management Strategy proposes to mitigate the effect of existing development upstream of Moonshine Valley Road. Risks posed to existing infrastructure (i.e., existing culverts) have since been highlighted and

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
		added to the Stormwater Management Strategy through the updated stormwater concept design/mitigation.
(2) Setback of homes along the top edge of the gully embankments • Inga Hunter, S17.001 • A & R Gear, S39.005 • Brett Guthrie, S41.008 • Barry Scott, S54.002 • Steve Welch, S65.001 • S & Yann Le Moigne, S71.003 • Wayne Phillips, S78.001 • Elizabeth Fisher, S80.001 • Colin Perrin, S90.002 • Sara Burgess, S98.003	The increase in housing density close to the edge of the gully embankment will increase the risk of landslides. S17 expressed particular concern to both new and existing homes on Moonshine Valley Road. S39 noted that soil saturation will increase the risk of landslides. Decision requested by S17 is to require more space be left between housing and the edge of the hill to Moonshine Valley Road, and using the last gully as the edge to the housing with water drainage to the main road and large stormwater drains so that there is no possibility of water coming over the edge. S39 and S98 requested 15m setback and limit of two storeys only, as well as 1ha minimum lot size in the land adjacent to Moonshine Valley. S54, S80 and S90 requested a setback of 15m in the land adjacent to Moonshine Valley, whilst S71 requested 30m setback and S78 requested 10m with a further 5m setback to the building line.	S65 discusses buildings being located just outside the 5m stormwater setback. However, the 5m stormwater setback is only part of the total setback which will be required for each lot within the plan change area. The setbacks presented in the Structure Plan are comprised of several different inputs, which include stormwater controls, geotechnical risks and visual impact. Increasing the buffer to 10m or 15m is therefore unnecessary for stormwater purposes. The issue of setbacks, particularly in relation to non-stormwater related effects, is more fully discussed by Ms Copplestone. The 5m setback identified in the Stormwater Management Strategy is to provide a laneway / buffer to intercept, collect and convey stormwater runoff and discharge it directly to the bottom of the gully in a controlled manner. This will aid in reducing the erosion of the gully embankments currently observed by submitters due to surface runoff discharging along the tops of the gullies, in addition to managing the

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
		additional runoff generated by development. Equally, the full stormwater buffer width set out in the Stormwater Management Strategy may not be required as long as it provides sufficient width for access and maintenance. That is, an effective cutoff drain sized for the 1% AEP (plus climate change) can be designed to fit within a smaller corridor. As discussed above, the purpose of the 5m setback is to collect and convey the stormwater runoff to a controlled discharge point. As such, it is not expected that this area will retain any water, and therefore risk of soil saturation should be low. This will need to be considered as part of the subdivision design, as it is likely that these 5m setbacks will also be located within Class D land (as defined in Map 10.1A).
(3) Multi-unit dwellings • R & J Stevens, S34.001 • Nathan Meyer, S36.001 • Marie Thompson, S38.002 • Brett Guthrie, S41.008 • Sonya Park, S44.001 • Russell Poole, S68.004	Multi-unit dwellings adjacent to rural land will impact properties already suffering from erosion. Decision requested is to remove multi-unit dwellings on the promontories. S38 requested single storey housing on larger sections. S44, S68, S72 and S80 submissions related specifically to the area adjacent to Moonshine Valley Road, and	Areas zoned for multi-unit dwellings may have a greater volume of runoff depending on the site coverage requirements. In general, building “up” rather than “out” will not increase stormwater runoff. As the lot sizes for medium density areas will be smaller (minimum $150m^2$), site coverage will be higher. This will have implications for

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
• Kerry Park, S72.001 • Elizabeth Endres, S74.001 • Wayne Phillips, S78.001 • Elizabeth Fisher, S80/001	suggested shifting the multi-unit housing away from the reserve areas.	permeability coverage, which will need to be considered at the time of development. Minimum permeable area requirements are being recommended for both medium density areas and suburban low density areas. We understand that Ms Copplestone has recommended that the promontories are developed for suburban low density housing (with an option retained for multi-unit). If the promontories are developed for suburban low density housing, it is likely they will have more permeable areas than previously envisaged.
(4) Lack of available information • Heritage Estates 2000 Ltd, FS18.016 through 019, 021 through 024, 026 through 029 and 031 through 033 • Steve Welch, S65.003	The technical information presented in relation to erosion, geotechnical and stormwater is insufficient to enable the submitter to peer review the evidence. Furthermore, the parameters and inputs into the flood modelling have not been made available to the submitter. S65 stated that they were unable to understand the strategy, and therefore opposed the strategy as it relates to “water storage tanks” and their positioning.	A memorandum detailing the methodology adopted for the stormwater modelling is at Appendix D. In our view, the Stormwater Management Strategy is robust and sound. Additional work to support that Strategy has been undertaken in light of submissions, including further modelling. The Stormwater Management Strategy retains its original approach, but additional controls have been proposed to ensure the design criteria are met, and anticipated outcomes achieved.

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
(5) Effects on adjacent land • Prasika Reddy, S21.001 • Gill Welch, S49.004 and 005 • Waka Kotahi NZ Transport Agency, FS16.001 • Steve Welch, S65.005 • Russell Poole, S68.006 • Colin Perrin, S90.002	The stormwater assessment does not address the effects on the land adjacent to the PCG area. S49 specifically raised concerns around the stormwater ponds and the effect this will have on areas normally dry. S65 raised similar concerns relating to their forested areas and how the proposed ponds will affect the root system. Decision requested is to have a stormwater assessment carried out for the residents of Moonshine Valley and Whiskey Way who border the development. FS16 asks that further investigations be carried out to better understand the potential flooding and stormwater hazards on existing infrastructure, such as State Highway 57.	The Stormwater Management Strategy included an erosion threshold analysis, which looked at the erosion potential within the surrounding gullies. The controls proposed in the notified Stormwater Management Strategy sought to control peak flows from both existing residential and future development as part of PCG so as to maintain the erosion exceedance to pre-development levels (prior to existing residential development). This was evaluated using actual rainfall data over a 5-year period. Further work was undertaken to consider the issues raised in submissions. This included modelling the effects of a fully developed catchment during large rain events, with all stormwater ponds within a catchment having reached full capacity and discharging in unison (i.e., the cumulative effects during large rain events). The results of this modelling demonstrated the need for additional storage capacity within the ponds. The storage volumes have subsequently been updated and the

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
		findings of this further assessment included in the Stormwater Management Strategy. However, as is standard practice, it should be noted that the updated volumes will still need to be confirmed as part of subdivision based on the subdivision layout, catchments, lot sizing, etc.
(6) Practicality of having detention ponds on the plateaus • Ee Khen Ang, S30.001 • A & R Gear, S39.007 • Colin Perrin, S90.002	The detention ponds will reach capacity by mid-winter and overflow to the gullies, creating erosion. In addition, there is risk should these ponds fail and release a large volume of water down the gully embankment. S30 raised concerns around the specific siting of a pond located along the top of their property.	Conceptual locations and footprints for stormwater detention areas to mitigate flood and erosion risk have been proposed, having regard to a range of factors identified in the Stormwater Management Strategy. Where possible (i.e., when not constrained by other factors), stormwater controls were located in those gullies which are less ecologically sensitive. This means selecting the more modified gullies with low sensitivity in terms of vegetation types and along ephemeral stream reaches, rather than permanent or intermittent stream reaches. In some areas this is unavoidable, and therefore ponds have been proposed on the plateaus themselves. In response to the submitters concerns, the risk of failure is low because:

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
		- The proposed detention ponds are sized for the 1% AEP event including climate change. The number of times the pond will be full to the emergency spillway level is very low. - The ponds will be designed to continuously drain over time. Once the ponds fill during heavy rain, they will discharge over an extended time until empty. In the winter and in summer the ponds will always completely drain. However, during back-to-back large rain events and in over design events the ponds may not be completely drained at the commencement of the subsequent rain event. A long term 5 year simulation has been carried out based upon historic rainfall from the 2011 to 2016 period. This showed that the 1% AEP event was a worse event with higher pond storage levels. It is conceivable that even higher pond storage volumes could occur in an over design event. It is recommended that a long-term simulation be carried out during the detailed design of the ponds to reduce the likelihood of a pond reaching the emergency

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
		spillway level. This is similar to what was carried out under the Stormwater Management Strategy. If necessary, modifications to the pond size could then be undertaken at the detailed design stage to account for the results of these simulations.
(7) Increase in stormwater runoff • Chris Teo-Sherrell, S43.001 • Michael Poulsen, S46.001 • Christine Scott, S55.002 • Horizons Regional Council, S60.005 • Gareth Orme, S75.001 • Manawatū Branch of Forest & Bird, S97.003	Development and increase in hard surface area will increase stormwater runoff. S75 noted the need for a whole-of-catchment approach. Decision requested is to limit the impervious areas, to consider on-site rainwater detention features, require rain gardens and other water sensitive design features, and the detention ponds to attenuate flow and detain sediment. S46 specifically highlighted the need to break up and minimise large, paved areas to avoid significant volumes of runoff. S55 requested that the size of lots be increased and density reduced.	It is recommended that the following be required to limit the increase in stormwater runoff generated by future development: - Impervious area be limited. That is, medium density areas to have a minimum 25% permeable area and low density to have a minimum 40% permeable area. - Road corridors be designed in accordance with the Structure Plan, which includes provisions for rain gardens and other low impact water design facilities. - The gullies be planted to both reduce surface runoff and erosion risk. Consideration of other water sensitive design features should also be incorporated into development. As noted earlier, onsite storage in

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
		the form of rainwater tanks and stormwater retention (the act of retaining water onsite) have not been recommended as part of the Stormwater Management Strategy generally.
(8) Stream erosion • Horizons Regional Council, S60.003 • Manawatū Branch of Forest & Bird, S97.006	Due to risk of erosion in waterways, consenting requirements need to be determined prior to development. S60 states that information sought will include stream stabilisation measures, discharge locations and sizes, etc. A maintenance strategy may also be required. S97 recommends that river corridors be designated to ensure streams aren't channelised.	Stormwater mitigation measures need to be carefully considered in relation to staging and consenting. It is recommended that the stormwater ponds within Gullies 1 and 3 that service multiple developments be Council-led projects, and that these be in place prior to any new upstream development. In addition, it will need to be understood that any subsequent development within the catchment must occur in accordance with the relevant Stormwater Management Plan. It is recommended that the stormwater management plans developed for development include a detailed maintenance programme. Concern was also raised around stream modification. We have discussed in-stream works above. The Stormwater Management Strategy will need to be updated to identify some of the proposed stream restoration measures, which includes simple stabilisation to reduce the gradient of the stream and isolated areas of bank

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
		stabilisation at bends that are prone to erosion. There is no intention to confine the stream within the existing channel. Surface water flows will be maintained and fish passage will be provided and maintained.
(9) Stormwater management • Horizons Regional Council, S60.005 • Wayne Phillips, S78.001 • Ben Somerton, S83.005 • Manawatū Branch of Forest & Bird, S97.002 through 004	The Conclusions and Recommendations from the Stormwater Management Strategy report must be stringently followed, especially in the northeast portion of PCG area. S60 requested that the mitigation measures are completed prior to inhabitation. S78 requested the detention ponds be replaced with a fully piped system across the rear of the properties that discharge directly to the four discharge locations. S83 commented that the gully crossings will need to cater for high flows and therefore may need to have a large diameter. S97 requested that provisions be in place to limit the volume of sediment during construction. That is, the area being worked on is limited. Furthermore, stormwater management systems	In order to properly manage the effects of development, it is recommended that all stormwater management controls/mitigation are in place prior to development. Note that this will require controls during construction of any subdivision to protect any downstream stormwater controls. Due to the storage volumes required to manage the effects of development, ponds are the only economical stormwater control. This could be coupled with piped reticulation, but are unlikely to replace the need for ponds altogether. Cutoff swales at the top of the gullies are required to prevent stormwater runoff eroding the sides of the gullies. Piping stormwater discharge down the gullies was previously considered, however ecological

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	be installed prior to development. S97 also requested that consent conditions be put in place to manage the effects of development. This includes proactive monitoring to improve water quality and river habitat rather than waiting for degradation to occur.	and environmental concerns have discounted this option. Due to the sensitivity of the gullies and erosivity of the soils, proactive monitoring of the gullies will be essential. In addition to the ponds and impervious limits, we also recommend that Council consider utilising the raingardens required as part of development to also provide attenuation. That is, the raingardens be lowered to enable ponding up to 300mm deep during rainfall events, to provide for slower release of stormwater into the gullies.
(10) Prescribed management • Heritage Estates 2000 Ltd, FS18.042 and 034 • Heritage Estates 2000 Limited, S51.028 through 031, 036 • CTS Investments Ltd, Woodgate Ltd and Terra Civil Ltd, S58.004	FS18 and S51 state that specific design solutions imposed through the Structure Plan will make it difficult to develop should the effects of PCG be different in nature to those envisaged by the masterplan process. The submitters believe that it prevents or restricts innovative alternatives and flexibility. The added wording in Policies 4.6 through 4.9 and 5.4 relating to this is opposed by FS18 and S51 in relation to the words around the Structure Plan.	The ultimate stormwater infrastructure will be dependent on the nature of the development within the catchment, and is influenced by factors such as density, typologies and roading layout. It will also be dependent on specific site constraints, which relate to hydrology, geology and ecology. The level of direction within the Structure Plan is considered appropriate. The Structure Plan and provisions need to adequately highlight the risks associated with developing the plan change area and provide the necessary controls to ensure development occurs

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	S58 opposes the proposed wetland area on the terrace as they believe adequate provision can be provided in the gullies, thereby not taking up developable land.	responsibly. The Stormwater Management Strategy is an example of how development could occur to show the feasibility of developing the land in accordance with the Structure Plan (in a manner that appropriately manages effects), but is a conceptual design of stormwater detention and water quality treatment facilities and may be subject to refinement over time.
(11) Vestment of gullies to PNCC • PN Industrial and Residential Developments Ltd, S45.010, 011	PCG requires that gullies are vested at the earliest subdivision opportunity, with little to no regard for how these will be accessed by Council for maintenance, enhancement and installation of public access infrastructure. Decision requested is that the gullies be vested where they are contiguous to an area of land sought to be developed.	We can only comment on this submission within our area of expertise. We agree the gullies need to be vested early on and with sufficient access for construction of stormwater controls and for ongoing maintenance.
(12) Level of service requirements • PNCC, S50.007 • Jill White, S67.001 • Rangitāne O Manawatū, S77.010 • Bruce Wilson, S105.022	"ARI" is used in the proposed provisions but not defined anywhere. Decision requested by S50 is that the following definition be provided: "Average Recurrence Interval (ARI) means the average time period between floods of a certain	Whilst the definition of the ARI is technically correct, we do not recommend using it as it could be misleading in terms of understanding probability and risk. It is important to remember that the rainfall event used to inform stormwater designs is based on the probability of an event

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• Manawatū Branch of Forest & Bird, S97.001 and 005	size. For example, a 100-year ARI flow will occur on average once every 100 years.” S67 and S105 requested that climate change considerations be brought to the fore when making stormwater management decisions. S97 requested that the plan allows for the most up to date information be used as climate change projections change. S77 only commented that flood mitigation ponds are not to be confused with water quality treatment devices. S97 requested that an assessment against NPSFM needs to be undertaken.	occurring at any given time, and not strictly the frequency at which it occurs. Our recommendation would be to move away from using the term “ARI” and instead switch to “AEP”, or “Annual Exceedance Probability”. This is given as a percentage, and typically defined as the probability of an event occurring in any given year. As such, the 100-year ARI is the same as the 1% AEP, and the 50-year ARI is the same as the 2% AEP. Further a 1% AEP has a 1% chance of occurring in any 1 year but a 63% chance of occurring in a 100 year period. We agree that climate change needs to be considered. In order to ensure that the latest information is considered with regards to climate change, we would recommend referencing PNCC’s Engineering Standards, which are regularly updated by the Council. In our opinion, stormwater quality controls and quantity controls can be provided together. In saying that, it is important to note that stormwater quantity controls should not adversely affect the quality controls, which may be why S77 has requested this delineation.

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
		Under item (9) we recommend that Council consider utilising rain gardens to also provide attenuation. The conceptual design of the stormwater management controls have had regard to the NPS-FM, and in particular, the recent proposal has resulted from an approach of avoiding, then reducing and minimising the generation of adverse effects, through on-site control of stormwater contaminants and flows, planning controls, restoration of natural systems, and infrastructure works . In-stream works have been limited as far as possible, and interventions have been sensitively designed, with an offset package considered feasible on the advice of Dr Forbes. Ms Copplestone comments further on the NPS-FM in their section 42A reports.
(13) Rule 7.15.2.1 • Heritage Estates 2000 Limited, S51.014	Performance Standard (c) is not supported in its current form. The submitter seeks more appropriate text.	We do not recommend any further changes beyond what has been discussed and agreed with the Council's planning expert Ms Copplestone, and as presented in her evidence.

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
(14) Section 7A Objective 3 • PNCC, S50.008	Objective 3 does not explicitly address natural hazards, but the proposed policies underneath it do. Decision requested is to amend Objective 3.	While this references natural hazards, we consider that this is a planning matter that Ms Copplestone is best placed to address.
(15) Section 7A, Policy 4.6, 4.8, 4.9, 5.15, 6.6 • CTS Investments Ltd, Woodgate Ltd and Terra Civil Ltd, S058.014 through 016, 024, 025 • Rangitāne O Manawatū, S77.010	S058 are opposed to the entire directive approach of the Plan Change and the implications for the Plan provisions. Conversely, S77 supports Policy 4.6, 4.8 and 6.6 as notified. S77 however requests that Policy 4.9 clarify to ensure that flood control devices are differentiated from water quality treatment devices.	No specific amendments or clarifications on their concerns is provided. Regarding Policy 4.9, the policy could be revised so as to highlight that stormwater quantity control must not adversely affect the operation of the stormwater quality controls.
(16) Rule 7A.5.2.2 Performance Standards (g) and Rule 7A5.2.3 Performance Standard (d) • Heritage Estates 2000 Limited, S51.065 • Rangitāne O Manawatū, S77.028	The submitter has concerns in relation to the technical reports and that the requirements for both flood modelling and transport network, including climate change effects, are unclear. Conversely, S77 supports R7A.5.2.2.	The Stormwater Management Strategy is to be updated to reflect the changes recommended following the review of submissions and further assessment/analysis. Further, the plan provisions have been reviewed in conjunction with Council's planning expert. The updated provisions are presented as part of Ms Copplestone's evidence.
(17) Provisions for flood management	Horizons flood modelling shows most of the modelled flood risk is within or near the	We consider that the additional work described in this report addresses the matters in Policy 9-2

Topic & Submission Point Reference	Submitter Concerns/ Requested Mitigation	Comment
• Horizons Regional Council, S60.002 and 004	waterways and within the gully system, however Council's flood modelling should take precedence. Decision requested is that provision for flood managements is included that gives effect to One Plan Policy 9-2, as well as stormwater management is demonstrated to achieve an outcome that is consistent with One Plan Rule 14-18.	and Rule 14-18 of the One Plan . Particular attention has been paid to the management of erosion and flooding arising from development, and expected outcomes are described in section H above.

I. RECOMMENDATIONS

63. For the reasons we set out above, we recommend the following changes to the Stormwater Management Strategy and the PCG Structure Plan/plan provisions:

(a) Any consequential impacts on the 20 and 30 degree no build lines arising from the changes in erosion projections for the gullies described in Appendix D**;** (these changes are to be addressed by Mr Bird in his s 42A report);

(b) Updates to the mitigation measures to reduce runoff, as described in Appendix D, including:

(i) The potential for an increased number of ponds and total combined storage;

(ii) Further recommendations for top of terrace ponds and rain gardens; and

(iii) Higher permeability requirements.

(c) Further detail should be included in the Stormwater Management Strategy on potential mitigation measures in the major gullies to prevent downcutting over the design life, including the following potential options:

(i) Cascading weirs;

(ii) Additional detention and attenuation ponds both online and off-line;

(iii) Bottom of pond oversized habitable culverts and spiral fish ladders; and

(iv) Below ground dams.

(d) Top of terrace works, including:

(i) A fully reticulated primary stormwater collection system (as before);

(ii) A secondary overland flow path collection and conveyance to intercept all flow up to the 1% AEP event and an ability to convey or pipe this flow either to the top of terrace pond or to a let-down structure to discharge the captured flow to the gully floor in a near tranquil state (as before); and

(iii) Perimeter swales / collection system to intercept runoff from the rear of each lot, with a piped system to the pond and a let-down mechanism to allow discharge of water to the gully floor in a near tranquil state (as before).

64. In terms of the PCG provisions, we recommend the following amendments:

(a) Perimeter stormwater swale: We recommend that there is flexibility on whether this should vest in the Council, or whether an easement would be sufficient. The width, depth and geometry of the drain will be dependent on the size of the catchment. We make the following recommendations:

(i) The swale should be designed to accommodate the 1% AEP storm event (including climate change).

(ii) An easement width ('setback') of 5m should be anticipated, but the final width should be confirmed as part of the subdivision design when the lot layout and catchments are determined.

(iii) We have prepared a cross-section that demonstrates the potential geometry of this perimeter swale (see below). In general, our example swale assumes 1:5 side slopes and a 1 metre base width. The swale will be grassed and this side slope angle provides for the swale to be safely mowed.

(iv) The swale should be positioned within a 5 metre setback commencing at the 25 degree or 1:4 maximum slope of any gully edge, where this adjoins residential or commercial lots.

(v) The swale is not necessary where roads adjoin the gully edges, as the roading corridor provides for stormwater collection and conveyance. The swale is intended to intercept, collect and convey stormwater from rear and side yards to a central discharge point. Runoff from the front of lots will be conveyed to the roading stormwater network and roading corridor. Checks will need to be undertaken at individual subdivision stages to ensure the intent of this provision is captured prior to stage approval being given.

(vi) If the swale is provided within a private lot, restrictions on use and the requirement for unrestricted access for maintenance will need to be recorded on a consent notice. No structures, earthworks or landscaping will be permitted within the 5 metre setback, or any other activities or works that would obstruct or prevent the functioning of the swale. The swale will be maintained by Council.

(vii) We recommend that the cross-section of the swale set out in Appendix E is inserted in the District Plan, alongside further policy direction on the function, purpose and restrictions on use of the swale corridor. We understand that Ms Copplestone has recommended plan provisions to achieve this.

(b) Key design criteria: The design specifications in the Stormwater Management Strategy remain valid and are specified in the policy framework and performance standards for stormwater management plans. We have recommended some minor changes to the structure of the plan provisions to make these clearer, and to ensure that a stormwater plan is prepared, which is consistent with the stormwater reporting requirements, for any development proposal that is brought forward (i.e., regardless of whether it is preceded by a subdivision application).

(c) Restriction on development: No development should commence in the upstream catchment until the necessary receiving infrastructure is in place. We have recommended some minor changes to the provisions to make it clear that residential development should not commence until the stormwater mitigation works, including those in the gullies, are in place to receive and manage runoff.

(d) Permeable surfaces: We recommend that a performance standard is added for minimum permeable surfaces. We have discussed with the Council experts a workable performance standard tailored for medium density (25% net site area) and low density (40% net site area) sites. This compares with the operative District Plan permeable standard of 30% net site area for standard suburban sites.

(e) Stormwater ponds: We have reviewed the locations and volumes of the proposed stormwater ponds, in conjunction with Mr Burns, and on the basis of geotechnical advice from Mr Bird and ecological advice from Dr Forbes. The revised Structure Plan provided with Mr Burn's S42A report shows the revised locations, and indicative sizing for these ponds, noting that final siting and sizing will be dependent on the subdivisions layout.

Allison Reiko Baugham and Tony Miller

15 September 2023

J. APPENDICES

- **Appendix A:** *Stormwater Management Strategy: Plan Change G - Aokautere* (GHD, 23 May 2022)
- **Appendix B:** *Stormwater Expert Evidence – Stream Erosion Assessment Summary* (Rev 4, Reiko Baugham and Tony Miller, 22 June 2023)
- **Appendix C:** *Proposed Stormwater and Stream Erosion Mitigation* (Reiko Baugham and Tony Miller, 28 August 2023)
- **Appendix D:** *Model Update – Technical Memo: Aokautere Plan Change G* (Sarah Irwyn and Jeff Doucette, 30 August 2023)
- **Appendix E**: Cross-section of Perimeter Stormwater Swale

Stormwater Management Strategy

Plan Change G – Aokautere

Palmerston North City Council

23 May 2022

→ The Power of Commitment

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S4	Rev 2	D. Arseneau	R. Baugham	SR. Baugham	R. Baugham	SR. Baugham	20/05/22

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1. Introduction

1.1 Background

GHD Limited (GHD) has been retained by Palmerston North City Council (PNCC) to undertake a stormwater management servicing assessment and strategy for the Aokautere Plan Change (Plan Change G). PNCC is undertaking the Aokautere Plan Change in response to ongoing existing and anticipated future development in the Aokautere area, in order to ensure that development proceeds in a coordinated, thoughtful, and effective manner that protects existing development and sensitive ecological features.

Aokautere is a rural community area located immediately southeast of the Manawatū River across from the Hokowhitu suburb of Palmerston North. Perched among the foothills of the Tararua Range, Aokautere exhibits a natural hilly terrain with a network of gullies and plateaus supporting significant recreational and ecological amenities. The location of the Aokautere Plan Change area (referred to as the site, study area) is presented on Figure 1.1.

Although the site is largely used for agricultural purposes, there are several areas of existing residential developments dating from the 1980s and onwards, and several other residential developments have recently begun to be constructed along the study area's plateaus. Several small gullies have also been filled during construction of the existing developments, and development pressure in this area is expected to increase in the future.

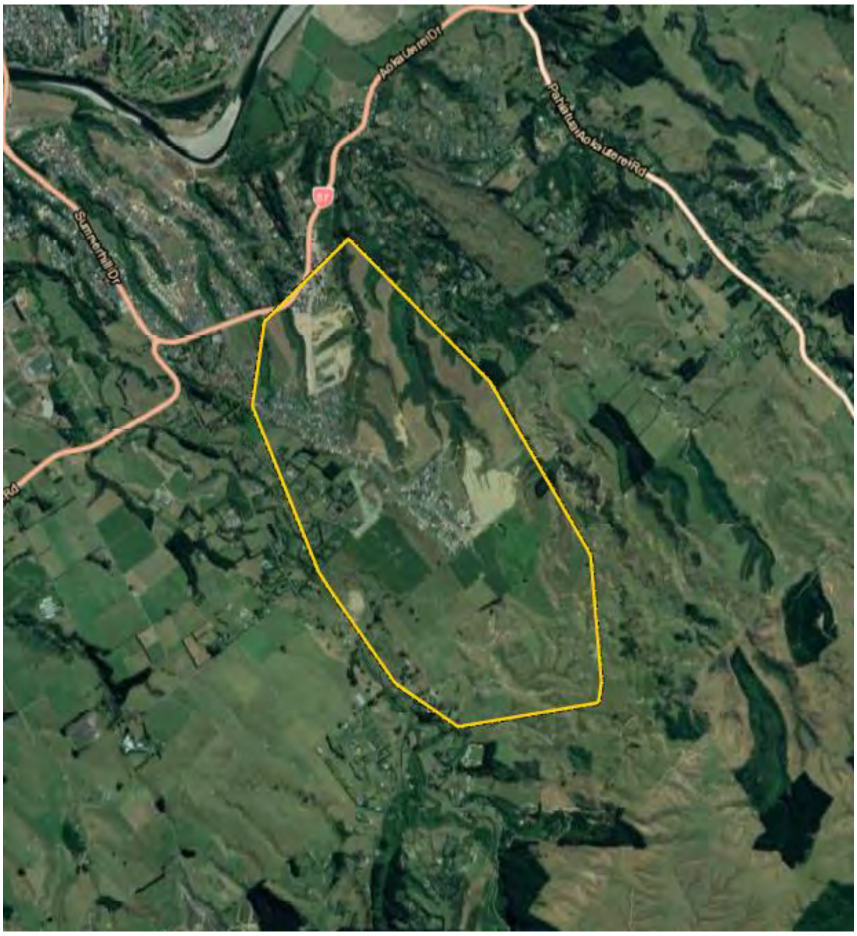

Figure 1.1 Aokautere Plan Change stormwater assessment study area

1.2 Purpose of this report

The purpose of this report is to present the outcomes of the stormwater management analysis and recommend a stormwater management strategy for the Aokautere Plan Change. This report will contribute to and form part of the Plan Change structure plan for implementation during detailed design, resource consenting and construction of the proposed developments.

1.3 Scope and limitations

The scope of the stormwater management analysis is to estimate and assess the flooding, erosion and water quality impacts of the development on the receiving watercourses and downstream areas. The analysis completed for this project includes hydrologic and hydraulic modelling of the study area (detailed in section 3) and quantitative assessment of the stormwater impacts based on the modelling results and available industry standards and guidelines (described in section 4). There are three components of the stormwater management analysis, including:

– Flood assessment to predict the impact of the development on peak flow rates and flood potential of the receiving environment.
– Erosion assessment to predict the impact of the development on the frequency and duration of flow events that exceed the erosion threshold of the receiving gully system.
– Water quality assessment to identify contaminant profiles typically associated with runoff from residential development and identify potential impacts to the receiving environment.

The stormwater management strategy (described in section 5) proposes design criteria and conceptual design alternatives for stormwater controls to mitigate the assessed impacts. The existing developments that are already in place are assessed in conjunction with the proposed development areas in order to recommend stormwater management controls that collectively achieve the established design criteria for the entire Structure Plan area.

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1.4 Assumptions

The analysis is based on the publicly available GIS data for the existing site conditions and the preliminary subdivision layout of the proposed development provided by McIndoe Urban (received 16 October, 2019, Appendix A) and subsequent revisions (final dated 11 May 2022, Appendix A). All grades and elevations for proposed conditions are assumed to generally follow the existing topography, noting that some modification of these grades is likely to occur during development. Additional modelling assumptions are outlined in section 3 of the report.

2. Existing Conditions

The development area is primarily located on a series of plateaus that form the headwaters of smaller tributaries of the natural drainage system, located in a series of gullies that separate the plateaus into distinct areas. The gullies within the study area ultimately discharge into larger streams that originate in the Tararua Range, specifically the Moonshine Valley Stream and the Turitea Stream, which flow around the development area to the Manawatū River.

A map of the total catchment area studied in the stormwater management analysis is presented on Figure 2.1, and includes the contributory drainage areas of the Moonshine Valley and Turitea Streams as well as the catchments within the study area itself. The preliminary proposed subdivision layout and the existing storm main reticulation are presented on Figure 2.2 (as of October 2019). The data for the existing storm main reticulation was obtained from PNCC Open Data. It is GHD's understanding that as of late 2019 there is currently only one identified stormwater management device (a gross pollutant trap) in the study area, located at the end of Silicon Way. However, development around Atlantic Drive within the Plan Change area is currently ongoing and will include installation of rain gardens and attenuation ponds. These facilities have not been incorporated into this stormwater assessment.

Potential discharge locations from the development area were identified based on a review of the existing storm main reticulation, the preliminary subdivision layout, and the existing site topography (contours). Site contours were generated from a 2018 Digital Elevation Model (DEM) obtained from PNCC Open Data, the most recent elevation data available at the time of this analysis.

Major discharge locations were identified where stormwater runoff ultimately exits the study area to the natural drainage system. Minor discharge locations were identified where stormwater is expected to discharge from the engineered stormwater collection and treatment system to the gullies within the study area. Figure 2.3 shows the major and minor discharge locations and their respective contributing drainage areas, which form the basis for the stormwater modelling and analysis described in the remainder of this report. Actual minor stormwater discharge locations from developed areas into the receiving environment will ultimately depend on site-specific design to be completed at the subdivision consent stage.

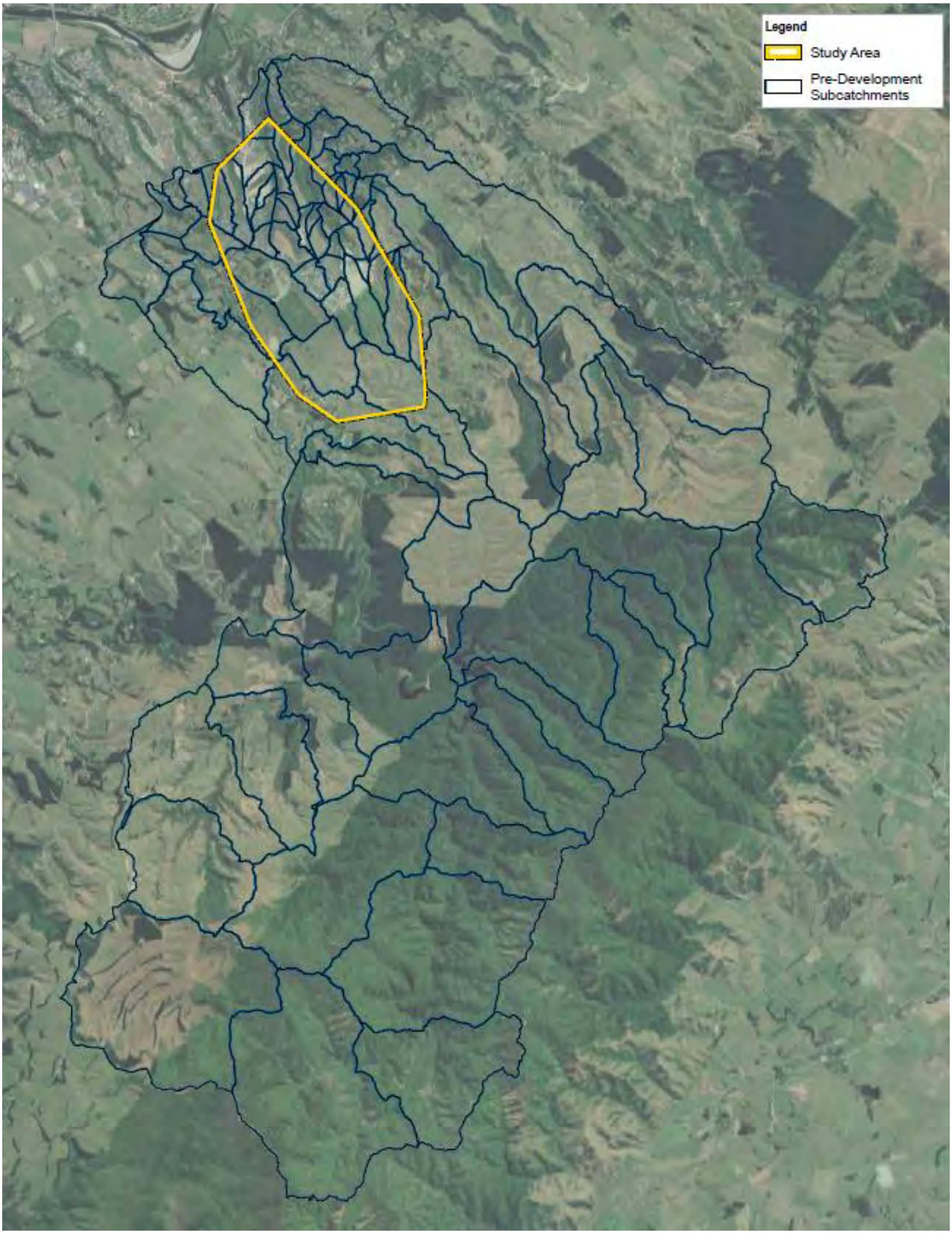


Figure 2.1 Aokautere stormwater assessment catchment area

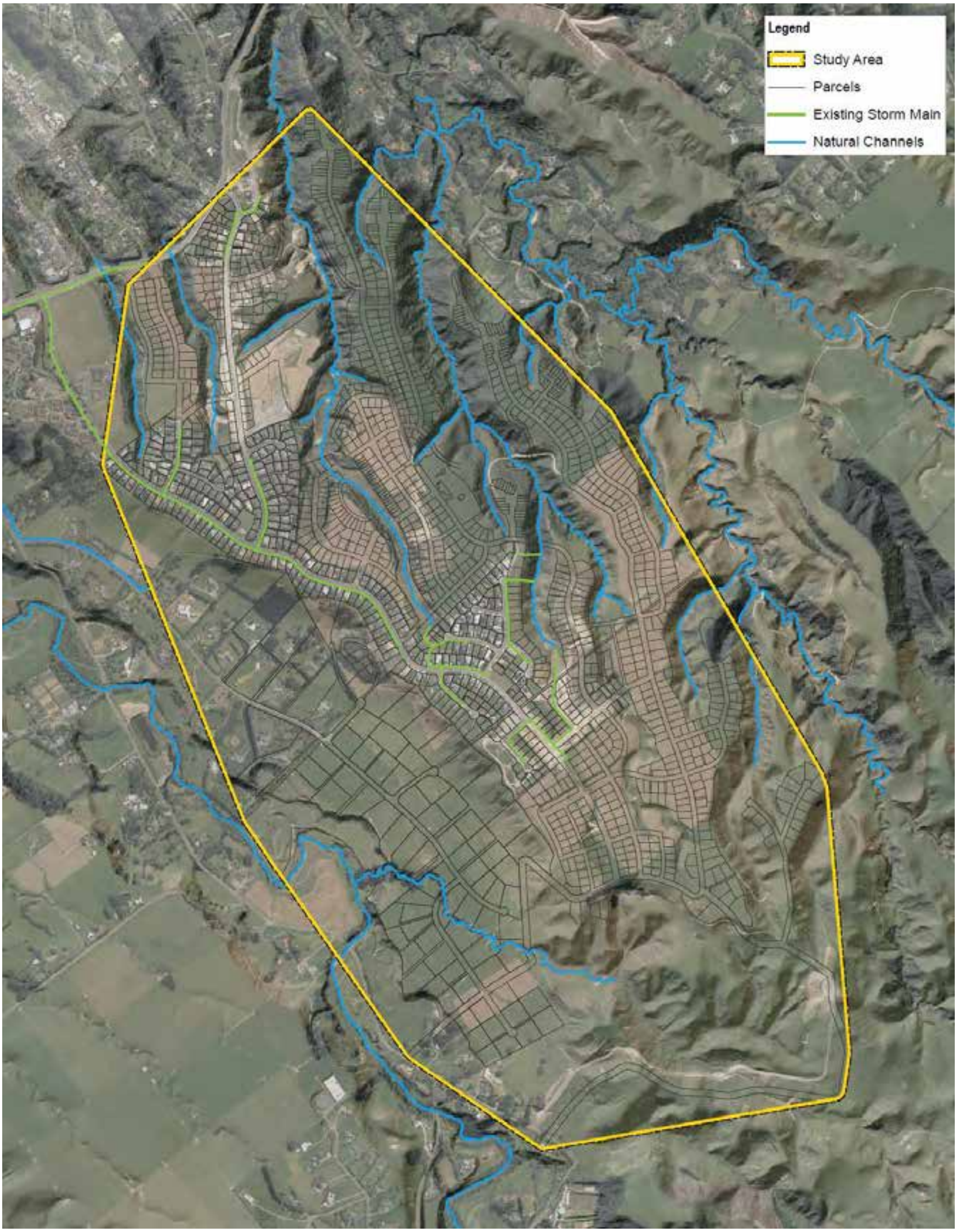


Figure 2.2 ***Preliminary development layout (Oct 2019) and existing stormwater reticulation***

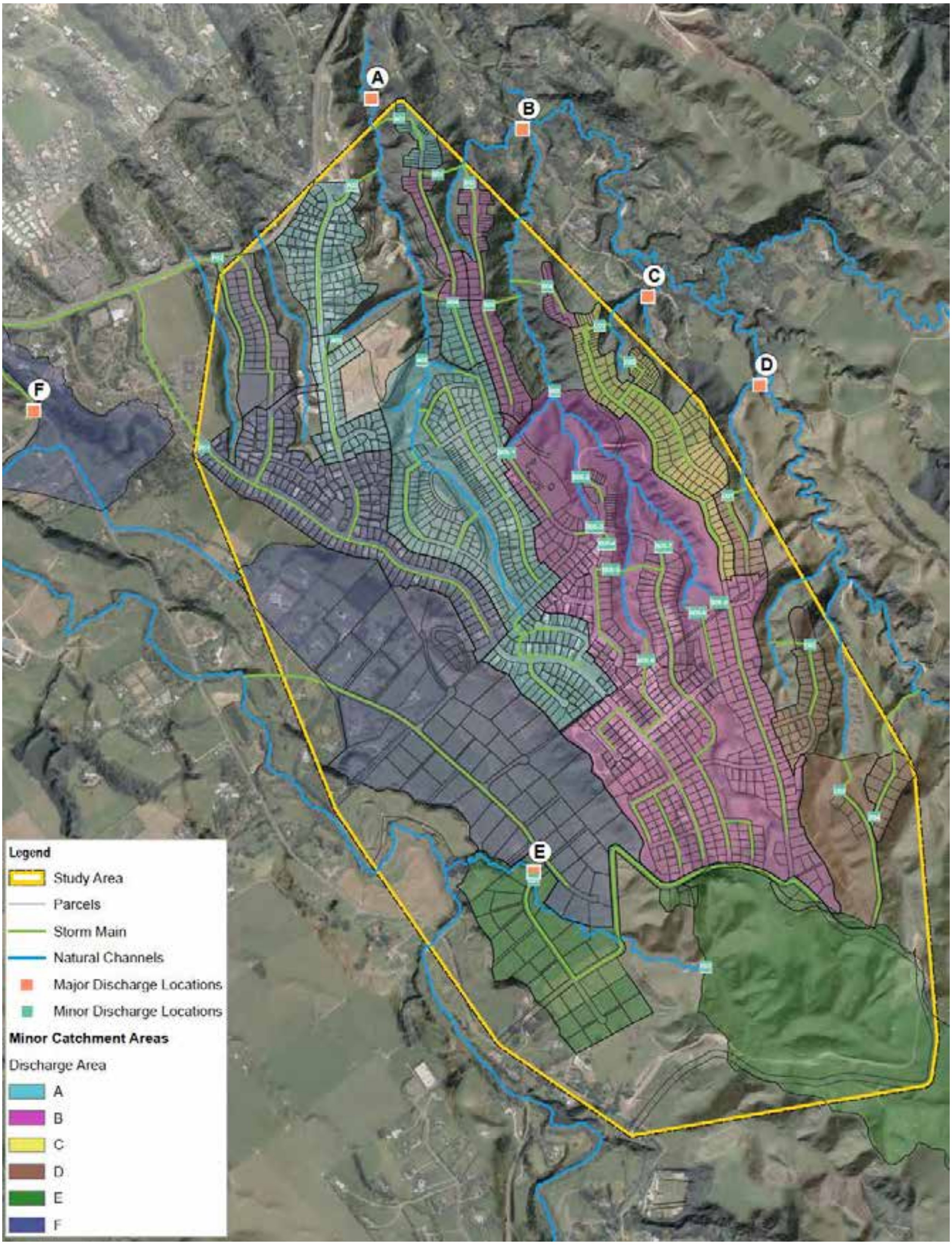


Figure 2.3 *Major and minor stormwater discharge locations*

3. Hydrologic and Hydraulic Modelling

Hydrologic and hydraulic modelling of the study area was undertaken to characterise stormwater runoff conditions in support of the flood and erosion impact assessments, including the quantification of pre-development and post-development (uncontrolled) runoff flows and volumes at the site discharge locations. The models described in this section are further employed in the impact assessment and conceptual design of stormwater management controls, described in sections 4 and 5.

The modelling was performed using the PCSWMM software (Computational Hydraulics International, 2017). PCSWMM is a spatial decision support system for the U.S. Environmental Protection Agency SWMM 5 software. The model requires input of topographical features (catchment area, flow length, slope, hydraulic roughness), ground cover conditions (land use, vegetation cover), infiltration parameters (infiltration capacity, drainage time), rainfall (hyetograph), and drainage paths (channels, channel lengths, roughness) in order to effectively simulate the stormwater runoff conditions of a subject site.

The setup of the pre- and post-development stormwater models are described in sections 3.1 and 3.2, respectively.

3.1 Pre-Development Model

For the purpose of this analysis, pre-development conditions were established as the land conditions prior to any residential development within the study area, including those limited areas of development constructed over the previous 20 to 30 years. This reflects the intention of the stormwater strategy to effectively address all stormwater runoff in the study area and avoid the “grandfathering” of existing areas which would then incur a disproportionately high impact to the receiving environment.

3.1.1 Subcatchment Parameters

Pre-development subcatchments were delineated using the Watershed Delineation Tool (WDT) in PCSWMM. The WDT employs flow direction, flow accumulation, stream definition, and watershed delineation calculations to create subcatchments from a DEM. The 2018 DEM was used to delineate subcatchments in the development area, and a 2015 DEM was used to delineate subcatchments in the upstream drainage area, as the more recent 2018 DEM did not extend to the full catchment limits. The 2015 DEM was obtained from the Land Information New Zealand (LINZ) Data Service. A map of the pre-development subcatchments is presented on Figure 3.1.

Flow lengths and subcatchment slopes were calculated manually using the site contours and measuring tool in PCSWMM. Flow lengths of large subcatchments (primarily located upstream of the study area) were calculated as the longest flow path to main channel, which include long sections of concentrated channel flow. Flow lengths of small subcatchments were calculated as the maximum overland flow path to the main channel, assuming that only limited concentrated flow occurs. Higher Manning’s ‘n’ values were assigned to the small subcatchments to represent the higher frictional forces associated with overland flow compared to concentrated flow.

Subcatchment parameters including percent impervious area, Manning’s ‘n’ values, and depression storage values were assigned based on land cover type, spatially averaged over the subcatchments. Land cover data was obtained from the Land Cover Database (LCDB) v40 layer of the Landcare Research Portal, added June 27, 2014.

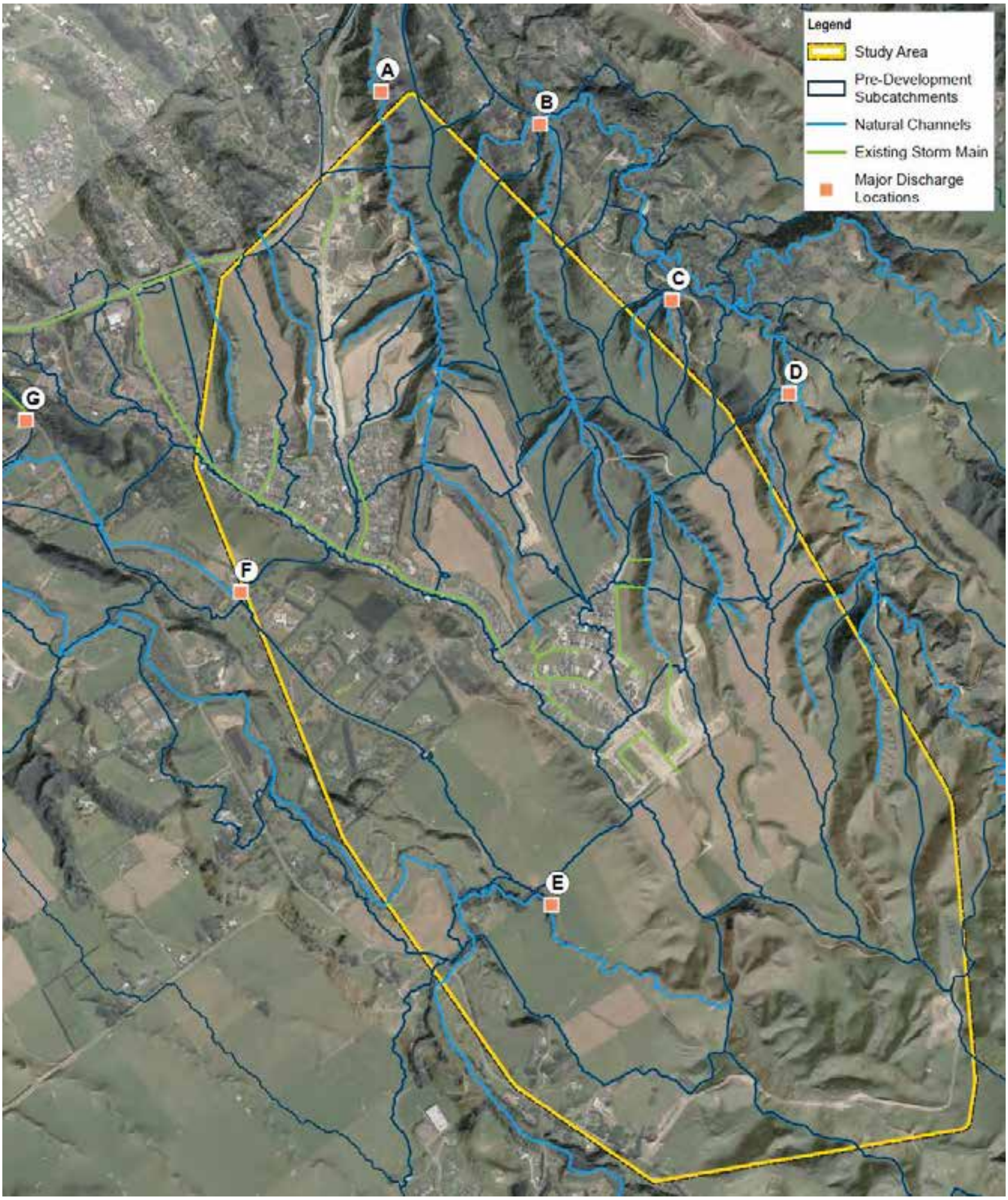


Figure 3.1 *Pre-development stormwater catchments*

Table 3.1 presents a summary of the Manning's 'n' values and depression storage values based on land cover type (Chow, 1959; US Environmental Protection Agency, 2015).

Table 3.1 Manning's 'n' and depression storage values based on land cover for pre-development conditions

No.	Land Cover Classification	Manning's 'n' (Channelised Flow)	Manning's 'n' (Overland Flow)	Depression Storage (mm)
1	Built-up Area	0.013	0.013	2
2	Urban Parkland/Open Space	0.030	0.030	5
5	Transport Infrastructure	0.013	0.013	2
6	Surface Mine/Dump	0.024	0.024	2
16	Gravel/Rock	0.024	0.024	2
20	Lake/Pond	0.040	0.040	0
21	River	0.040	0.040	0
30	Short-rotation Cropland	0.045	0.170	5
33	Orchard/Vineyard/Other Perennial	0.050	0.170	5
40	High Producing Exotic Grassland	0.050	0.130	5
41	Low Producing Grassland	0.050	0.130	5
45	Herbaceous Freshwater Vegetation	0.070	0.400	8
51	Gorse/Broom	0.160	0.400	8
52	Manuka/Kanuka	0.160	0.400	8
54	Broadleaved Indigenous Hardwoods	0.160	0.400	8
64	Mixed Exotic Shrubland	0.160	0.400	8
68	Forest-Harvested	0.160	0.400	8
69	Deciduous Hardwoods	0.160	0.400	8
71	Indigenous Forest	0.160	0.400	8

3.1.2 Infiltration Model

The Soil Conservation Service (SCS) curve number method was used to calculate infiltration and other hydrologic losses in the flood and erosion assessments. Curve numbers represent the average antecedent runoff/infiltration conditions of the subcatchments, and were selected from the "Guidelines for Stormwater Runoff Modelling in Auckland Region (TP 108)" and "Urban Hydrology for Small Watersheds (TR-55)" based on the hydrologic soil group and the land cover type of the subcatchments.

The hydrologic soil group was assumed based on the particle size distribution from the surficial soil map (FSL Particle Size Classification layer from the Land Resource Information System (LRIS) portal, Landcare Research, added on June 7, 2010). Sand, silt, loam, and clay designations from the LRIS data were assigned to hydrologic soil groups A, B, C, and D, respectively. The predominant soil type within the study area is classified as loam and was assigned to the C soil group. The soil types used for the hydrologic assessment are shown in Figure 3.2.

A curve number polygon shapefile was created by performing a spatial intersection of the surficial soil map (that provides information on the particle size distribution and hydrologic soil group), and the land cover map. The resultant polygons were assigned curve numbers using a site-specific look up table that was created for the development area. Curve numbers were then spatially averaged over the subcatchments. Table 3.2 presents the site's curve number lookup table.

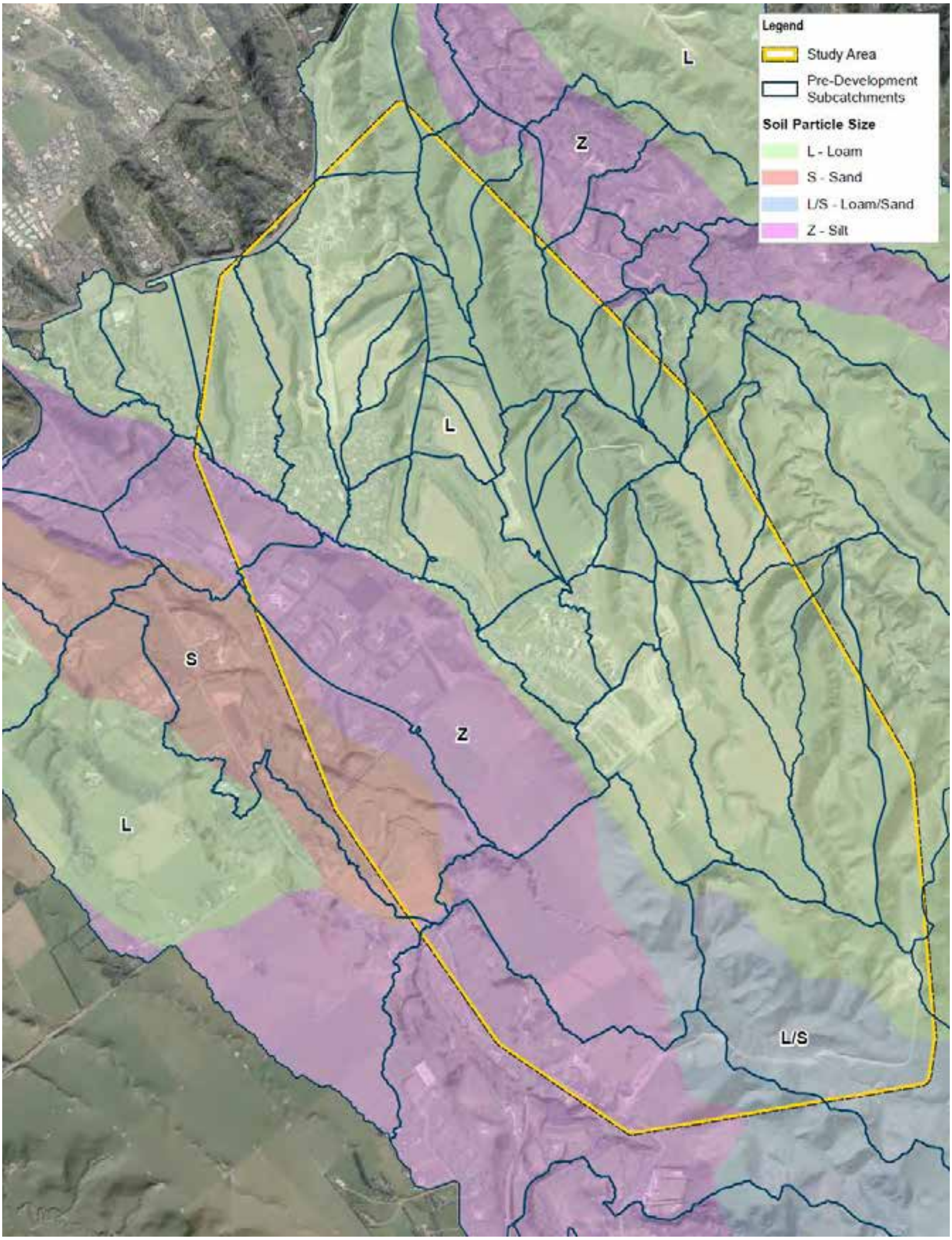


Figure 3.2 Study area soil types

Table 3.2 Curve number lookup table based on land cover and hydrologic soil group

Land Cover Data		Hydrologic Soil Group			
		A	B	C	D
1	Built-up Area	61	75	83	87
2	Urban Parkland/Open Space	39	61	74	80
5	Transport Infrastructure	98	98	98	98
6	Surface Mine/Dump	98	98	98	98
16	Gravel/Rock	77	86	91	94
20	Lake/Pond	98	98	98	98
21	River	98	98	98	98
30	Short-rotation Cropland	72	81	88	91
33	Orchard/Vineyard/Other Perennial	32	58	72	79
40	High Producing Exotic Grassland	39	61	74	80
41	Low Producing Grassland	39	61	74	80
45	Herbaceous Freshwater Vegetation	68	79	86	89
51	Gorse/Broom	48	67	77	83
52	Manuka/Kanuka	48	67	77	83
54	Broadleaved Indigenous Hardwoods	48	67	77	83
64	Mixed Exotic Shrubland	48	67	77	83
68	Forest-Harvested	30	55	70	77
69	Deciduous Hardwoods	30	55	70	77
71	Indigenous Forest	30	55	70	77
-	Proposed Residential (1/4 Acre Lots)	61	75	83	87
-	Proposed Residential (1 Acre Lots)	51	68	79	84
-	Proposed Residential (2 Acre Lots)	46	65	77	82

3.1.3 Rainfall Input

Rainfall was modelled using a design storm approach for the flood assessment and was based on the requirements outlined in the PNCC Engineering Standards for Land Development (ESLD) (2019). Rainfall hyetographs were created using a Normalised 24-hour Design Storm distribution, and rainfall intensities for the 2-year, 10-year, 50-year and 100-year 24-hour return period events. Historical rainfall was used to form the hyetographs for the pre-development model, and the climate change projected rainfall was used to form hyetographs for the post-development model using the RCP 6.0 climate change scenario for the 2081 to 2100 time period (as per PNCC ESLD). The return period rainfall intensities used in the analysis are summarized in Table 3.3. Rainfall intensities were obtained from the National Institute of Water and Atmospheric Research (NIWA) High Intensity Rainfall Design System (HIRDS). The Normalised 24-hour design storm distribution was acquired from TP 108 and is summarised in Table 3.4. The RCP 6.0 2081-2100 rainfall intensities are generally 11% to 14% larger than historical values.

Table 3.3 Summary of 24-hour ARI rainfall intensities for the historical and RCP 6.0 climate change scenario for 2081 – 2100

Scenario	Rainfall Intensity (mm/hour)			
	2-year ARI	10-year ARI	50-year ARI	100-year ARI
Historical	2.34	3.50	4.70	5.25
Projected (RCP 6.0)	2.62	3.96	5.35	5.98

Table 3.4 24-hour nested design storm rainfall intensities for pre- and post-development conditions

Begin Time	End Time	I/I24[1]	Historical (Pre-Development) [mm/hr]				RCP 6.0 2081-2100 (Post-Development) [mm/hr]			
			2-year	10-year	50-year	100-year	2-year	10-year	50-year	100-year
00:00	06:00	0.34	0.80	1.19	1.60	1.79	0.89	1.35	1.82	2.03
06:00	09:00	0.74	1.73	2.59	3.48	3.89	1.94	2.93	3.96	4.43
09:00	10:00	0.96	2.25	3.36	4.51	5.04	2.52	3.80	5.14	5.74
10:00	11:00	1.4	3.28	4.90	6.58	7.35	3.67	5.54	7.49	8.37
11:00	11:30	2.2	5.15	7.70	10.34	11.55	5.76	8.71	11.77	13.16
11:30	11:40	3.8	8.89	13.30	17.86	19.95	9.96	15.05	20.33	22.72
11:40	11:50	4.8	11.23	16.80	22.56	25.20	12.58	19.01	25.68	28.70
11:50	12:00	8.7	20.36	30.45	40.89	45.68	22.79	34.45	46.55	52.03
12:00	12:10	16.2	37.91	56.70	76.14	85.05	42.44	64.15	86.67	96.88
12:10	12:20	5.9	13.81	20.65	27.73	30.98	15.46	23.36	31.57	35.28
12:20	12:30	4.2	9.83	14.70	19.74	22.05	11.00	16.63	22.47	25.12
12:30	13:00	2.9	6.79	10.15	13.63	15.23	7.60	11.48	15.52	17.34
13:00	14:00	1.7	3.98	5.95	7.99	8.93	4.45	6.73	9.10	10.17
14:00	15:00	1.2	2.81	4.20	5.64	6.30	3.14	4.75	6.42	7.18
15:00	18:00	0.75	1.76	2.63	3.53	3.94	1.97	2.97	4.01	4.49
18:00	00:00	0.4	0.94	1.40	1.88	2.10	1.05	1.58	2.14	2.39

1. Note, I/I24 represents the nested design storm ratio of intensity for the specific time step to the 24-hour intensity for the appropriate ARI.

A continuous modelling approach was used to perform the erosion assessment in order to assess the total erosive forces imposed on the receiving watercourses across a representative range of flow events. This modelling approach requires continuous hourly rainfall data as input. The hourly rainfall time series was obtained from the National Climate Database (NIWA) for the Palmerston North Ews gauge for the 2012 to 2016 time period.

3.1.4 Drainage System

The natural drainage system was generated by the WDT in PCSWMM using the DEM. Figure 3.1 shows the delineated drainage system. The natural channels were represented by a stepped cross-section estimated based on the available elevation contours and visual observations. The channel portion of the cross-section consists of a trapezoidal geometry with a 1 m depth, 1 m bottom width, and 2H:1V side slopes.

Channel slopes were calculated as the change in elevation over the channel lengths, where elevations of the upstream/downstream ends of the channel were assigned based on the DEM. Manning’s ‘n’ values were selected for the channels according to a review of the aerial imagery of the catchment area. A Manning’s ‘n’ value of 0.030 was assigned to the channel portions of the cross-section. Manning’s ‘n’ values of 0.100 and 0.035 were assigned to floodplain areas that were more representative of forest and grassland cover, respectively (Chow, 1959).

3.2 Post-Development Uncontrolled Model

An intermediary but critical step in the development of an effective stormwater management strategy for the study area is the quantification of runoff impacts related to the proposed development. To accomplish this, a post-development model was developed that included the full extent of residential development within the study area but excluded stormwater management controls. The setup of this uncontrolled model is described in the following sections.

3.2.1 Subcatchment Parameters

The post-development model was based primarily on the pre-development model to ensure that the results could be considered comparable. Post-development subcatchment areas were modified from the pre-development catchments in the study area to represent the preliminary subdivision layout. The lots were assumed to be graded toward the roads, where minor flows are captured and conveyed in a reticulated storm sewer system and major flows are conveyed along the road. Upstream subcatchments that are outside of the development area remain unchanged from pre-development conditions. Figure 3.3 presents a map of the post-development catchments, which show a marked increase in the number of catchments within the study area in order to represent the complexity of urban development drainage pathways.

Post-development subcatchments were characterized in a similar manner to the pre-development subcatchments. The land cover shapefile was modified to include the roads and lots of the preliminary subdivision layout. Roads were included in the Transportation Infrastructure land cover classification, which has a Manning's 'n' value of 0.013, and a depression storage value of 2 mm. Lots were assigned a land cover classification called Proposed Residential (developed by GHD for the purpose of this study). The majority of the Proposed Residential land cover is assumed to be 70 percent impervious (based on typical PNCC practice for recent structure plans), with a weighted Manning's 'n' value of 0.018, and depression storage value of 3 mm for impervious/pervious areas. Some residential lot sizes are notably larger in the south and west extents of the study area. In these areas the land cover is assumed to be 30 percent impervious, with a weighted Manning's 'n' value of 0.025, and a depression storage value of 4 mm for the impervious/pervious areas.

3.2.2 Infiltration Model

The curve number shapefile for the flood assessment was modified to reflect proposed conditions. The roads were included in the Transportation Infrastructure land cover classification that has a curve number of 98. The lots were assigned to the GHD-defined Proposed Residential land cover classification and assigned a curve number based on the Residential District land use type with an average lot size of one-quarter acre (approximately 1,000 m^2), 1 acre (4,000 m^2), or 2 acres (8,000 m^2) from the TR-55 document. Although the average lot size has not been confirmed, the selected curve number represents an impervious cover percentage that is expected to be appropriate to the development, regardless of lot size.

3.2.3 Rainfall Input

Similar to the pre-development model, rainfall was modelled using a design storm approach for the flood assessment. Rainfall hyetographs were created using a Normalised 24-hour Design Storm distribution, and rainfall intensities for the 2-year, 10-year, 50-year and 100-year 24-hour return period events. Climate change projected rainfall was used to form hyetographs for the post-development model using the RCP 6.0 climate change scenario for the 2081 to 2100 time period. The return period rainfall intensities used in the analysis are summarized in Table 3.3 in section 3.1.3. Rainfall intensities were obtained from the National Institute of Water and Atmospheric Research (NIWA) High Intensity Rainfall Design System (HIRDS). The Normalised 24-hour design storm distribution was acquired from TP 108.

3.2.4 Drainage System

The natural drainage channels were modelled using stepped cross sections as in the pre-development model. In the development area, the minor and major drainage networks were represented by stormwater mains and road cross-sections, respectively. The stormwater mains were sized to convey the 10-year peak flow rate under climate change conditions using the Manning's equation, as per the *PNCC ESLD* (2019). Surface elevations of catch basins and manholes were determined from existing topography. Invert elevations were assumed and set to ensure a minimum pipe slope of 0.3 to 0.5% (depending on the pipe size, as per PNCC standards), a minimum depth of cover of 1.0 m and maximum depth to invert of 2.9 m below grade. The road cross-sections were classified as local, local-collector, or collector based on the preliminary subdivision layout provided by McIndoe Urban. The travelled portions of the local, local-collector, and collector roads were assumed to be 5.5 m, 6.5 m, and 9.5 m wide, respectively, with 0.15 m high curbs. The remainder of the right-of-way was modelled as a grassed surface with 4 percent slope toward the road, on either side of the roadway, to ensure that all flow is contained within this corridor.

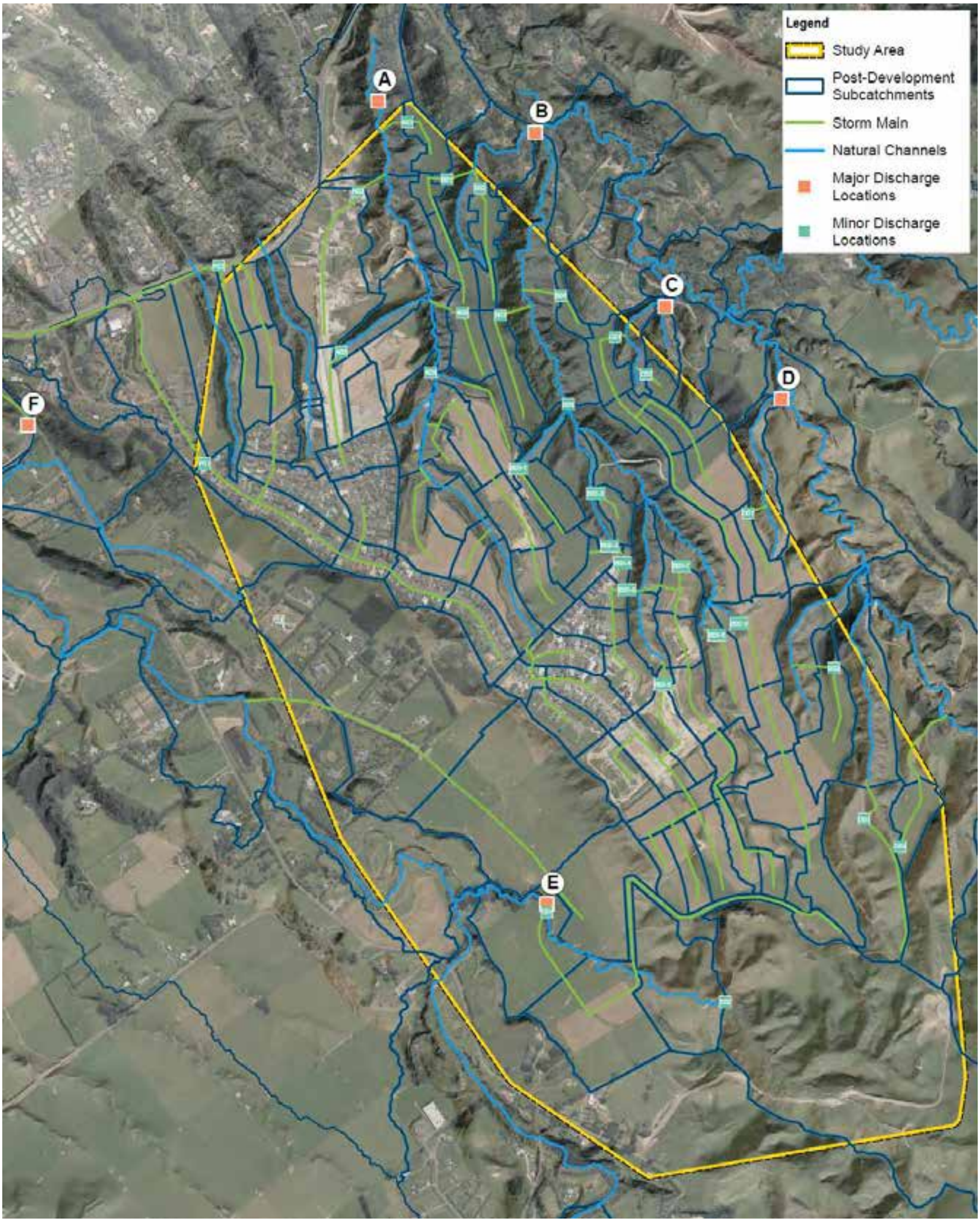


Figure 3.3 Post-development stormwater catchments

The drainage system employed in the post-development model is conceptual only for the purpose of this study. The network configuration and pipe diameters must be confirmed during detailed design for each development area to ensure the appropriate engineering standards and design criteria are met.

3.3 Sensitivity Analysis

Calibration is an important component of the hydrologic modelling process to ensure the model provides meaningful results. Calibration involves tuning model parameters to match modelled output to measured streamflow records. However, since streamflow records are not available for any of the site discharge locations, model calibration could not be performed. As such, an analysis was performed to assess the sensitivity of the model to the selected hydrologic parameters in order to understand the level of uncertainty associated with the model output. The results of the sensitivity analysis are important to consider when using the model output to support decision making processes such as for the number/size/type of stormwater management controls required for the development area.

The sensitivity analysis was conducted using the pre-development model. The analysis was performed by varying subcatchment and routing parameters by 75% and 125% of the initial value as per the *Technical Guidelines for Flood Hazard Mapping* (Environmental Water Resources Group Ltd., 2017). The analysis was performed for subcatchment parameters including flow length, subcatchment slope, Manning's 'n' (impervious and pervious), depression storage (impervious and pervious), and curve number. The analysis was also performed for the Manning's 'n' values of the natural channels.

Table 3.5 and Table 3.6 present the sensitivity of modelled peak runoff to a 25% change in subcatchment and routing parameter values. The resultant percent change in peak runoff was calculated for each subcatchment (and conduit for the Manning's 'n' of the channels) for the full range of ARI design flows included in the model, and the statistics (maximum and average) of percent change values were reported.

Similarly, Table 3.7 and Table 3.8 present the sensitivity of the modelled runoff volume to a 25% change in subcatchment parameter values. The percent change in runoff volume was calculated for each subcatchment area for the full range of ARI design flows included in the model, and the maximum and average percent change values were reported.

Table 3.5 Sensitivity of Modelled Peak Runoff to Flow Length, Subcatchment Slope, and Manning's 'n'

Statistic	Flow Length		Subcatchment Slope		Manning's 'n' Pervious	
	-25%	+25%	-25%	+25%	-25%	+25%
Max. Change in Peak Runoff (%)	14.7%	-15.0%	-10.0%	8.6%	15.3%	-15.0%
Avg. Change in Peak Runoff (%)	11.2%	-10.7%	-5.5%	5.5%	11.2%	-10.7%

Table 3.6 Sensitivity of Modelled Peak Runoff to Depression Storage, Curve Number, and Manning's 'n' for Channelized Flow

Statistics	Depression Storage Pervious		Curve Number		Manning's 'n' Channel	
	-25%	+25%	-25%	+25%	-25%	+25%
Max. Change in Peak Runoff (%)	7.0%	-10.0%	-45.0%	83.9%	6.2%	-8.1%
Avg. Change in Peak Runoff (%)	1.8%	-2.8%	-40.6%	62.6%	1.4%	-1.5%

Table 3.7 Sensitivity of Modelled Runoff Volume to Flow Length, Subcatchment Slope, and Manning's 'n'

Statistics	Flow Length		Subcatchment Slope		Manning's 'n' Pervious	
	-25%	+25%	-25%	+25%	-25%	+25%
Max. Change in Runoff Volume (%)	1.5%	-1.7%	-0.8%	0.8%	1.5%	-1.7%
Avg. Change in Runoff Volume (%)	0.5%	-0.6%	-0.3%	0.3%	0.5%	-0.6%

Table 3.8 Sensitivity of Modelled Runoff Volume to Depression Storage and Curve Number

Statistics	Depression Storage Pervious		Curve Number	
	-25%	+25%	-25%	+25%
Max. Change in Runoff Volume (%)	3.2	-4.0	-33.4	53.2
Avg. Change in Runoff Volume (%)	1.7	-2.1	-30.2	44.6

The sensitivity analysis results demonstrate that peak runoff flow is generally more sensitive than runoff volumes to the selected model parameters. Peak runoff is most sensitive to curve number with a maximum percent increase in peak runoff of approximately 84% due to a 25% increase in the curve number. This indicates that the model results are highly sensitive to the overall level of impervious cover that will ultimately be constructed in the study area; any modifications to the development plan (i.e., densification) need to be considered in the context of their potential stormwater impacts.

4. Stormwater Runoff Assessment

This section provides the results of the flood assessment, erosion assessment and water quality assessment which were undertaken to define the targets for the design of stormwater management controls in the development area. Development of the stormwater concept design to meet these design criteria is discussed in section 5.

4.1 Flood Assessment

The purpose of the flood assessment is to understand the impact of the proposed development on the peak runoff flows where the site discharges to the receiving environment. Higher peak flow rates create higher water levels in channels, which increases the risk of flooding for adjacent people, property, infrastructure, and natural habitat. Generally, paving of roads and construction of residential areas results in an increase in impervious area, which causes an increase in peak flows and runoff volumes as less water infiltrates into the soil or evaporates.

The proposed stormwater management design criterion is to control the post-development peak flow rates to pre-development levels. It is recommended to provide flood control for the full range of ARI events from the 2-year to the 100-year events to ensure a robust level of runoff management.

Pre-development and uncontrolled post-development peak flow rates were established at all site discharge locations using the PCSWMM models described in section 3. Stormwater management controls must be designed to reduce the pre-development flow rates such that they are equal to or less than post-development levels at each of these locations.

Typically, stormwater management measures are used to attenuate post-development peak flow rates through the provision of storage. Wetlands, dry ponds, and wet ponds are examples of stormwater management measures that may be considered to attenuate peak flow rates for flood control. The selection and conceptual design of these controls is further assessed in section 5 of the report.

Table 4.1 presents the pre-development and uncontrolled post-development peak flow rates for the 50-year and 100-year return period events for each discharge location, previously identified in Figure 2.3. It also presents the estimated minimum storage volume that is required to attenuate the post-development flow rates to pre-development levels for the maximum design storm event, based on analysis using the Storage Calculator Tool in PCSWMM (which provides a useful but conceptual indication of required storage volume). A full summary of the results for all ARI events is included in Appendix B. In all cases the 100-year ARI storage volume is the largest value of the ARI events assessed.

Table 4.1 Flood assessment results and criteria

Discharge Point[1]	50-year ARI			100-year ARI		
	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required
	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)
A01	0.09	0.23	288	0.11	0.27	323
A02	0.49	1.49	2,129	0.61	1.71	2,288
A03	0.27	0.86	1,159	0.33	0.99	1,267
A04	0.12	0.40	506	0.15	0.46	546
A05	1.88	5.65	7,948	2.33	6.45	8,604
B01	0.14	0.37	630	0.17	0.42	696
B02	0.12	0.36	480	0.15	0.42	521
B03	0.16	0.55	741	0.20	0.63	793
B04	0.05	0.13	147	0.06	0.15	168
B05-1	0.14	0.48	654	0.17	0.55	715
B05-2-3-4[2]	0.24	0.66	871	0.30	0.77	947
B05-5-6-7[2]	1.36	4.51	6,220	1.69	5.02	6,732
B05-8	0.81	2.68	3,649	1.00	2.99	3,986

Discharge Point[1]	50-year ARI			100-year ARI		
	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required
	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)
B05-9	0.68	1.97	2,917	0.84	2.18	3,175
C01	0.59	1.77	2,519	0.76	1.95	2,639
C02	0.05	0.12	131	0.06	0.15	149
D01	0.05	0.33	699	0.06	0.38	783
D02	0.17	0.91	1,949	0.22	1.07	2,141
D03	0.09	0.52	1,113	0.11	0.61	1,252
D04	0.06	0.43	851	0.08	0.50	912
E01	0.58	2.47	4,392	0.78	2.95	4,725
E02	1.84	2.94	5,826	2.45	3.73	6,362
F01	0.55	2.87	6,348	0.73	3.25	6,599
F02	0.24	1.41	2,572	0.31	1.53	2,728

1. *The discharge point may include smaller upstream discharge points, as indicated in the naming of the catchment.*

2. *Discharge points B05-2, B05-3 and B05-4 as well as B05-5, B05-6, B05-7 have the potential to be combined into centralized storage areas; as such, the required storage volumes for these groups of outlets have been lumped together.*

A screen capture of the storage calculator tool is provided in Figure 4.1. The post-development uncontrolled hydrograph is presented in yellow, and the controlled hydrograph defined by the target peak flow rate is presented in blue. The storage volume required to attenuate the post to pre-development peak flow rates is estimated as the blue shaded area between the two hydrographs.

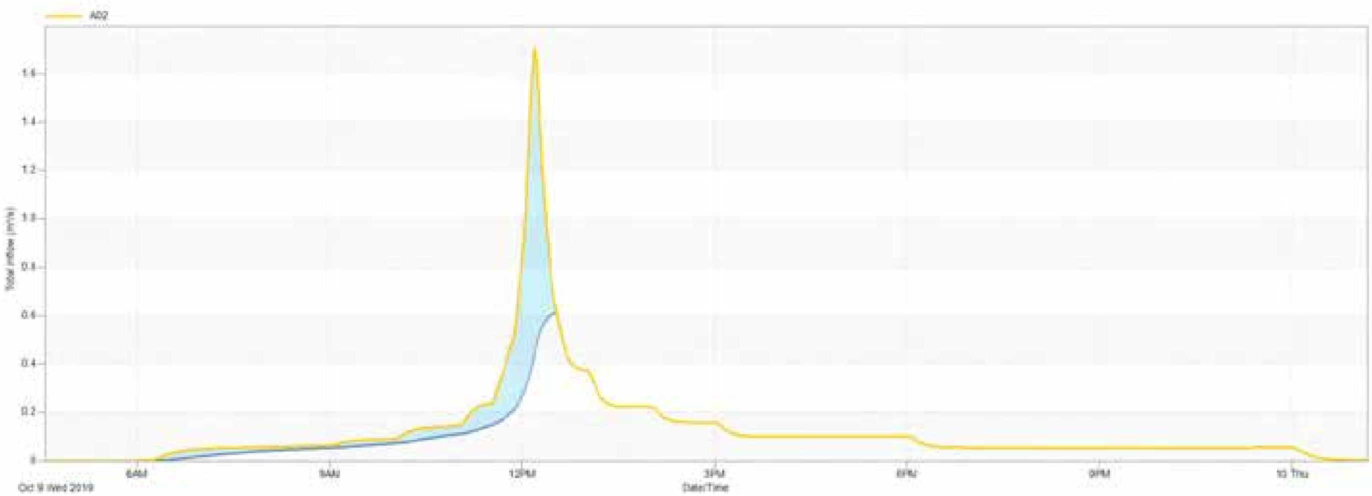


Figure 4.1 PCSWMM storage calculator tool output (sample)

4.2 Erosion Assessment

The purpose of the erosion assessment is to understand the impact of the proposed development on the degree and rate of erosion in the receiving watercourses, which is determined both by the erosivity of the watercourse bed and bank soils, and the magnitude and duration of flows within the watercourse.

The first factor, the erosivity of the watercourse bed and bank soils, is quantified by an erosion threshold, representing the flow level at which the soil particles will erode. The determination of erosion thresholds for the selected Aokautere receiving watercourses is further discussed in section 4.2.1.

The second factor, the magnitude and duration of flows within the watercourse, is assessed through a continuous hydrologic and hydraulic model using a long-term rainfall time series. The resulting time series of watercourse flows is then used to calculate the frequency and duration of erosion threshold exceedance, as well as the cumulative effective work exerted on the watercourse by the flows. Pre- and post-development modelled flow time series can be compared in this way to assess the expected erosion impacts related to the development. The evaluation of erosion threshold exceedance is described in section 4.2.2.

4.2.1 Determination of Erosion Thresholds

An understanding of the potential geomorphic response of a receiving watercourse to changes in flow regime allows the sensitivity of the channels to be assessed. It also allows quantification of their potential to assimilate flows without exacerbating or increasing instream erosion beyond natural rates. In support of this approach a target flow is usually defined for comparison between pre- and post-development conditions. This target flow is usually defined as an erosion threshold, which is the flow that theoretically can entrain bed or bank sediments within the most sensitive reach (i.e., section) of the watercourse.

An erosion threshold is assessed by identifying the most sensitive reaches along the network of channels that could potentially be impacted. From this reach, a target critical velocity, or critical shear stress for the bed or bank materials, is defined. Once the critical shear stress or velocity is known, the equivalent discharge can be determined from detailed measurements of the watercourse geometry.

For the Aokautere Structure Plan area, the watercourses located in the gullies upstream of discharge points A and B were assessed as a significant portion of the proposed development would discharge stormwater runoff into these gullies. As well, there is a desire to minimise the need for additional watercourse stabilisation or restoration in response to stormwater flows, as the gullies have been identified as having significant terrestrial and aquatic features. The stream located downstream of discharge points C and D was assessed as an indication of a larger watercourse in similar conditions as the gully streams. For the purpose of this study, these three watercourses were designated as Aokautere Church Stream (upstream of discharge point A, close to the Aokautere Community Church), the Moonshine Valley Reserve Stream (upstream of discharge point B, located in the Moonshine Valley Reserve) and the Tutukiwi Reserve Stream (downstream of discharge points C and D, located in the Tutukiwi Reserve).

The Aokautere Church Stream in the study reach was observed to be a narrow, grass-lined watercourse with a bed largely composed of fine silt and clay (Photo 1 below). However, in one location at the downstream end of the study reach, a large scour pool had formed (Photo 2 below), exposing the layer of sand, gravel and cobbles underlying the thick silt and clay layer. The stream was observed to have very little active flow, consisting mostly of remnant pools from the previous rainfall event.

↑ Photo 1: Aokautere Church Stream (typical)

↑ Photo 2: Scour pool in Aokautere Church Stream

The Moonshine Valley Reserve Stream flows through the densely vegetated Reserve in two distinct reaches, located upstream and downstream of a significant scour pool and “knick point” in the stream bed (Photo 3 below). Upstream of the scour pool, the stream is roughly rectangular in section and relatively narrow (Photo 4 below), with a bed composed of soft silt and clay similar to the Aokautere Church Stream. Downstream of the scour pool, significant erosion has widened and deepened the stream, exposing a layer of sand, gravel and cobbles underlying the silt and clay.

↑ Photo 3: Scour pool in Moonshine Valley Reserve Stream

↑ Photo 4: Moonshine Valley Reserve Stream upstream of scour pool (typical)

The Tutukiwi Reserve Stream receives drainage from several of the eastern slopes of the Structure Plan area as well as rural lands southeast of the study area, and is significantly larger than both the Aokautere Church Stream and Moonshine Valley Reserve Stream. Although some reaches of the stream are exhibiting signs of erosion and adjustment (Photo 5 below), the stream appears to be generally in stable condition where it flows through the established bush (Photo 6 below).

↑ Photo 5: Eroded meander bend in Tutukiwi Reserve Stream

↑ Photo 6: Tutukiwi Reserve Stream (typical)

The threshold discharge in the study streams is limited by the shear stress resistance of the bed and bank material. Critical shear stress is the shear acting on the bed that can potentially entrain a given characteristic sediment size. To compare the sensitivity of the bed and banks and examine initial entrainment and full bed mobilization, shear stress thresholds for bank materials, the median bed materials (D50) and larger bed materials (defined by the D84) are provided. The critical shear stress for the median bed materials (D50) and larger bed materials (D84) are based on critical shear for non-cohesive sediments from the Miller et al. (1977) model. The calculation of critical shear stress for bank materials was based on cohesive sediments from tables and plots from Chow (1959). The shear stress on the bank material was estimated as 75% of the average bed shear stress since bank materials are not subject to the same level of shear as the bed (Chow, 1959). The resulting erosion thresholds were determined based on the sensitivity of both channel bed and banks. The determination was aided by field observations at the time of survey.

Table 4.2 summarises the key stream characteristics used in the assessment of erosion thresholds, as observed and/or measured on site.

Table 4.2 Summary of stream characteristics for erosion threshold assessment

Parameter	Aokautere Church Stream	Moonshine Valley Reserve Stream	Tutukiwi Reserve Stream
Bankfull Width (m)	2.2	2.5	4.1
Bankfull Depth (m)	0.4	0.6	0.6
Channel Gradient (%)	3.24	1.87	0.94
Bed Material D50 (mm)	1	1	15
Bed Material D84 (mm)	2	2	32
Manning's n[1]	0.045	0.04	0.04
Bankfull Velocity (m/s)	2.02	2.14	1.72
Bankfull Discharge (m^3/s)	1.90	3.01	3.99
Flow competence[2] (m/s) for D50	0.2	0.2	0.7
Flow competence[2] (m/s) for D84	0.3	0.3	1.0
Critical Shear (Nm-2)[3] for D50	0.73	0.73	10.93
Critical Shear (Nm-2)[3] for D84	1.46	1.46	23.31
Critical Shear (Nm-2)[4] for bank material	5	5	5
Parameters for Typical Channel Cross Section at the Erosion Threshold			
Max Critical depth (m) for entrainment	0.024	0.029	0.083
Critical discharge (m^3/s) for entrainment	0.001	0.003	0.038
Critical velocity (m/s) for entrainment	0.20	0.23	0.37

1. *based on visual estimate (2019) and checked using technique outlined in Chow (1959)*
2. *according to Komar (1987)*
3. *according to Miller et al. (1977)*
4. *from tables in Chow (1959)*

For both the Aokautere Church Stream and the Moonshine Valley Reserve Stream, the erosion threshold was determined to be very low due to the sensitivity of the existing fine silt and clay bed and bank materials. This is compared to the more mature/established Tutukiwi Stream which has a coarser bed material and thus higher thresholds for erosion. It is likely that these streams erode easily in response to flow, which is considered reasonable given the location of the streams within gullies that have been historically shaped by erosive processes. This also makes the streams highly sensitive to changes in flow and presents a significant challenge to stormwater management in mitigating erosion impacts from development. To assess the sensitivity of the streams to changes in this threshold, a higher threshold of 0.05 m^3/s was also analysed, representing a potential higher end threshold for more cohesive fine-grained materials.

An assessment of the erosion threshold exceedance at both streams is presented in the following section.

4.2.2 Erosion Threshold Exceedance

Erosion thresholds are regularly exceeded in natural systems with defined watercourses; in fact, it is erosion that forms the watercourse out of the surrounding soils. In order to minimize potential impacts and maintain the natural channel function, post-development erosive flows should match as closely as possible the pre-development erosive flows in magnitude and duration. Comparisons of exceedance of the erosion threshold by the pre- and post-development flows can be determined using several criteria including frequency of exceedance and cumulative effective work.

Exceedance criteria are determined through the use of a continuous hydrologic model to compare time series of discharge over several years. The frequency or cumulative time of exceedance provides a simple comparison of the amount of time the discharge is above the erosion threshold, and thus the amount of time in which channel erosion is more likely to occur. It does not however account for the excess work above the erosion threshold, which represents the accumulation of active force applied against the stream bed and banks from the flows. The excess work is dependent on the magnitude and duration of the flow exceedance. It can be represented by the

Cumulative Effective Work Index or the total amount of stream power above the erosion threshold as defined by the threshold shear stress. It is calculated following the method described in Rowney and MacRae (1992):

$$PWR = \sum (\tau_o - \tau_{thr}) V \Delta t$$

Where PWR is the cumulative stream energy expended above a threshold value (J/m^2)

τ_o is the instantaneous shear stress at the stream reach (N/m^2)

τ_{thr} is the threshold shear at the stream reach (N/m^2)

Δt time step

V velocity (m/s)

The Cumulative Effective Work Index is typically a better indicator of potential impacts to watercourse geomorphology than the time of exceedance, since it incorporates both the frequency and magnitude of the exceedance.

The Cumulative Effective Work Index was determined for the uncontrolled post-development conditions in Aokautere using a custom MATLAB program developed for this analysis and compared to the pre-development conditions to determine potential impacts on the local watercourse geomorphology. Table 4.3 and Table 4.4 summarise the results of the erosion threshold exceedance analysis for the lower and higher thresholds, respectively, for both the pre-development and uncontrolled post-development scenarios. Graphs of the exceedance analysis results are included in Appendix C.

The parameters summarised in Table 4.3 and Table 4.4 demonstrate the complex interaction between the magnitude and duration of stream flows and the erosive potential of those flows. An overview of the significance of each parameter reported in the tables is included below:

– Total Exceedance Time represents the total number of hours over the five-year simulation period (total of approximately 43,848 hours) when the modelled stream flow exceeds the erosion threshold.
– % Exceedance Time is the total exceedance time represented as a percentage of the total simulation time, to illustrate the significance of the total values.
– Number of Exceedance Events shows the number of discrete occasions where the flow exceeded the erosion threshold and then returned to a level below the threshold, which could indicate a higher potential for erosion.
– Cumulative Effective Discharge sums the total modelled flow volume that was above the erosion threshold for the stream, indicating the raw volume of erosive flow that will act on the stream.
– Cumulative Effective Work Index sums the total amount of “work” exerted on the stream above the erosion threshold, indicating the total amount of erosive energy over the simulation period.

Table 4.3 Summary of erosion threshold exceedance analysis – uncontrolled post-development (lower threshold)

Parameter	Aokautere Church Stream (0.001 m^3/s threshold)			Moonshine Valley Reserve Stream (0.003 m^3/s threshold)			Tutukiwi Reserve Stream (0.038 m^3/s threshold)		
	Pre-Dev.	Post-Dev. Uncontrolled	% Change	Pre-Dev.	Post-Dev. Uncontrolled	% Change	Pre-Dev.	Post-Dev. Uncontrolled	% Change
Total Exceedance Time (hrs)	5,131	3,924	-24%	3,336	3,140	-6%	3,466	3,432	-1%
% Exceedance Time	11.7	8.9	-24%	7.6	7.2	-5%	7.9	7.8	-1%
Number of Exceedance Events	169	263	56%	184	287	56%	100	149	49%
Cumulative Effective Discharge (m^3)	890,800	918,800	3%	738,700	982,400	33%	606,700	585,000	-4%
Cumulative Effective Work Index (J/m^2)	173,700,000	176,100,000	1%	84,360,000	109,900,000	30%	149,400,000	143,900,000	-4%

Note that the total simulation time extends over 5 years, totalling approximately 43,848 hours.

Table 4.4 Summary of erosion threshold exceedance analysis – uncontrolled post-development (higher threshold)

Parameter	Aokautere Church Stream (0.05 m^3/s threshold)			Moonshine Valley Reserve Stream (0.05 m^3/s threshold)			Tutukiwi Reserve Stream (0.05 m^3/s threshold)		
	Pre-Dev.	Post-Dev. Uncontrolled	% Change	Pre-Dev.	Post-Dev. Uncontrolled	% Change	Pre-Dev.	Post-Dev. Uncontrolled	% Change
Total Exceedance Time (hrs)	1,207	1,112	-8%	997	1,131	13%	3,171	3,110	-2%
% Exceedance Time	2.8	2.5	-11%	2.3	2.6	13%	7.2	7.1	-1%
Number of Exceedance Events	188	292	55%	177	286	62%	102	144	41%
Cumulative. Effective Discharge (m^3)	531,000	621,300	17%	468,300	710,900	52%	592,400	570,900	-4%
Cumulative Effective Work Index (J/m^2)	83,250,000	97,010,000	17%	44,400,000	66,940,000	51%	143,600,000	138,300,000	-4%

Note that the total simulation time extends over 5 years, totalling approximately 43,848 hours.

The results shown in Table 4.3 and Table 4.4 indicate that the uncontrolled post-development stormwater runoff flows have the potential to incur a significant amount of additional "work" on the smaller streams in the study area, representing a similarly significant increase in the potential for erosion and stream degradation. Although post-development conditions in the Tutukiwi Reserve Stream show a significant increase in the number of exceedance events, this does not translate into a direct increase in erosive potential. This is due to the much larger catchment of this stream compared to the Aokautere Church Stream and Moonshine Valley Reserve Stream, as well as the more mature and stable channel configuration observed in the Tutukiwi Reserve Stream where it has accessed the granular/cobble glacial till that underlies the finer deposits in the study area.

To mitigate erosion impacts from urban runoff, stormwater controls need to mitigate this additional work through reducing the shear stress, velocity, and duration of flow events in the stream that exceed the erosion threshold. This is accomplished in similar ways to the mitigation of flood risk: detention to reduce peak flows and infiltration to reduce total flow volume.

For the Aokautere Structure Plan area, infiltration is not considered an effective means of stormwater management due to the widespread presence of cohesive soils with low hydraulic conductivity, as well as to the high levels of maintenance and pre-treatment required to sustain large-scale infiltration systems. Therefore, stormwater detention was investigated for erosion risk mitigation. The continuous hydrologic model used to assess the uncontrolled post-development erosion impacts was revised to incorporate the recommended stormwater detention volumes established in Section 4.1 to mitigate flood risk. The erosion threshold exceedance analysis was then completed for these controlled post-development runoff results.

For the purpose of evaluating the effectiveness of proposed stormwater management controls, the higher erosion threshold was deemed more appropriate as it better accommodates flow changes that result from urbanisation and stormwater management. As well, the higher threshold appears to provide a more conservative assessment of stream erosion impact due to stormwater runoff, leading to more robust and resilient recommendations for the Structure Plan.

Erosion threshold analysis results for the preliminary controlled post-development conditions are summarised in Table 4.5, Table 4.6 and Table 4.7.

Table 4.5 Summary of erosion threshold exceedance analysis – controlled post-development – Aokautere Church Stream

Parameter	Aokautere Church Stream (0.05 m^3/s threshold)				
	Pre-Dev.	Post-Dev. Uncontrolled	% Change[1]	Post-Dev. Controlled	% Change[1]
Total Exceedance Time (hrs)	1,207	1,112	-8%	1,382	14%
% Exceedance Time	2.8	2.5	-11%	3.2	14%
Number of Exceedance Events	188	292	55%	179	-5%
Cumul. Effective Discharge (m^3)	531,000	621,300	17%	466,100	-12%
Cumul. Effective Work Index (J/m^2)	83,250,000	97,010,000	17%	72,390,000	-13%

1. The % Change figure is calculated for uncontrolled and controlled post-development conditions relative to pre-development conditions.

Table 4.6 Summary of erosion threshold exceedance analysis – controlled post-development – Moonshine Valley Reserve Stream

Parameter	Moonshine Valley Reserve Stream (0.05 m^3/s threshold)				
	Pre-Dev.	Post-Dev. Uncontrolled	% Change[1]	Post-Dev. Controlled	% Change[1]
Total Exceedance Time (hrs)	997	1,131	13%	1,605	61%
% Exceedance Time	2.3	2.6	13%	3.7	61%
Number of Exceedance Events	177	286	62%	147	-17%
Cumul. Effective Discharge (m^3)	468,300	710,900	52%	489,700	5%
Cumul. Effective Work Index (J/m^2)	44,400,000	66,940,000	51%	45,930,000	3%

1. *The % Change figure is calculated for uncontrolled and controlled post-development conditions relative to pre-development conditions.*

Table 4.7 Summary of erosion threshold exceedance analysis – controlled post-development – Tutukiwi Reserve Stream

Parameter	Tutukiwi Reserve Stream (0.05 m^3/s threshold)				
	Pre-Dev.	Post-Dev. Uncontrolled	% Change[1]	Post-Dev. Controlled	% Change[1]
Total Exceedance Time (hrs)	3,171	3,110	-2%	3,065	-3%
% Exceedance Time	7.2	7.1	-1%	7.0	-3%
Number of Exceedance Events	102	144	41%	103	1%
Cumul. Effective Discharge (m^3)	592,400	570,900	-4%	568,100	-4%
Cumul. Effective Work Index (J/m^2)	143,600,000	138,300,000	-4%	137,800,000	-4%

1. *The % Change figure is calculated for uncontrolled and controlled post-development conditions relative to pre-development conditions.*

The erosion threshold assessment of the preliminary controlled post-development conditions indicate that the proposed stormwater detention volumes required for flood risk mitigation are largely adequate to mitigate erosion risk in the study area streams. However, this is in comparison only to the current pre-development conditions. The Aokautere Church Stream and Moonshine Valley Reserve Stream will remain highly sensitive to erosion in the future regardless of upstream development and will continue to erode and degrade in a manner that may create slope stability risk or water quality impacts.

4.2.3 Gully Slope Erosion Impacts

In addition to the potential for erosion impacts to the receiving watercourses of the study area, there are potential impacts to the gully slopes from uncontrolled overland flows. There are numerous existing rills and channels that have formed along the gully slopes due to overland flow from the plateaus, which are equally as or more sensitive than the primary gully watercourses and represent a potential slope stability and erosion risk if stormwater flows from development are permitted uncontrolled access to these areas. As well, encroachment of development on these critical slopes can cause further disturbance and impact to vegetation and slope stability. Formalised stormwater collection could be considered along the tops of the gully slopes (e.g., swales) with controlled slope drains to safely discharge the runoff to the gully floor. Further, a buffer/setback from the top of slope, particularly along the rear of residential properties, could be considered to minimise encroachment and provide a corridor for the proposed stormwater collection system. This is further evaluated in section 5. It should be noted that this assessment does not consider any geotechnical risks that may require further setback.

4.3 Water Quality Assessment

Stormwater quality parameters of concern due to urbanization include suspended solids, metals (i.e., copper, zinc, lead, etc.), petroleum hydrocarbons, nutrients, and other organic compounds. Urban catchments can also increase runoff temperature, which may impact habitat for sensitive cold-water species. These contaminants have the potential to impact aquatic life and habitat in receiving streams. Impacts can be particularly high during construction of the proposed development if not managed through effective erosion and sediment controls.

Enhanced approaches to water quality control include provision for retention and disposal of the “first flush” runoff from urban catchments, i.e., the initial runoff from a previously dry catchment. The first flush typically carries the highest proportion of contaminants as particles, oils, and other urban contaminants are washed from the impervious surfaces. A typical first flush management target would be to retain and dispose of the first 10 to 25 mm of a rainfall event on site through infiltration, detention, evaporation, etc. This equates to approximately the 90th percentile rainfall depth over a 24-hour period, capturing the vast majority of smaller, more frequent events during a typical year. A treatment train approach may be required, where multiple water quality devices are implemented throughout the drainage system to achieve the water quality criteria.

Figure 4.2 (from GD2017/01, Table 15) summarizes the effectiveness of various stormwater management devices in terms of their water quantity control and water quality treatment performance. The table shows that bioretention devices, including swales, rain gardens, tree pits, and planter boxes, provide effective water quality treatment for all listed contaminants. Constructed wetlands provide effective water quality control for most contaminants with the exception of indicator bacteria and temperature, for which they achieve partially effective control. Wet ponds provide effective water quality control for sediments and gross pollutants, and partially effective control for all other parameters, with the exception of temperature. Constructed wetlands and wet ponds also provide the benefit of effective water quantity control.

Key ● Effective ○ Partially effective - Not effective	Quantity control					Quality control								
	1% AEP	Detention of 50% and 10% AEP	90th & 95th percentile detention	Groundwater recharge	Retention	Sediment	Gross pollutants	Heavy metals	Oils and grease	Nutrients	Organics	Hydrocarbons	Indicator bacteria	Temperature
Pervious pavement - unlined	-	-	●	○	●	●	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]
Pervious pavement - lined	-	-	●	-	-	●	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]
Living roof	-	-	●[a]	-	●	○	NA	○	NA	○	○	NA	○	●
Rainwater tank (no reuse)	-	○	●	-	-	●	NA	○	NA	○	○	NA	○	○
Rainwater tank (with reuse)	-	○	●	-	●	●	NA	○	NA	○	○	NA	○	○
Infiltration device	-	○	●[a]	●	●	-	-	-	-	-	-	-	-	●
Swale (lined)	-	-	-	-	-	●	○	○	○	○	○	○	○	●
Bioretention swale (unlined)	-	-	●	●	●	●	●	●	●	●	●	●	●	●
Rain garden	-	-	●	●	●	●	●	●	●	●	●	●	●	●
Stormwater tree pit[c]	-	-	○	○	●	●	●	●	●	●	●	●	●	●
Planter box	-	-	○	○	●	●	●	●	●	●	●	●	●	●
Constructed wetland	-[d]	●	●	-	○	●	●	●	●	●	●	●	○	○
Wet pond	●	●	●	-	-	●	●	○	○	○	○	○	○	-
Dry pond (detention basin)	●	●	●	-	-	-	-	-	-	-	-	-	-	●

Notes:

NB: Assumes sizing, construction and maintenance are compliant with this guideline's requirements

NA: Not applicable, does not treat this pollutant because it is generally not present in the drainage area

●[a]: Assumes retention of up to the 90th and 95th percentile events

-[b]: Assumes limited water quality treatment for active pervious paving systems. Passive pervious paving is assumed to have some treatment effectiveness if maintained correctly

[c]: Stormwater tree pits are different to street tree pits in that they are specifically designed for stormwater management and must be sized accordingly.

-[d] Wetlands designs should bypass large storm events to protect vegetation and ensure sediments are not resuspended

Figure 4.2 Summary of Effectiveness of Stormwater Management Devices in terms of Water Quantity Control and Water Quality Treatment (GD2017/01, Table 15)

Bioretention devices can temporarily store, treat, and infiltrate runoff at the source. A number of bioretention devices may be implemented as part of a multi-device treatment approach. The Water Quality Flow (WQF) event is used to size bioretention devices for water quality treatment purposes, where WQF is calculated with a rainfall intensity of 10 mm/hour (an approximate 90th percentile annual rainfall intensity for the Auckland area) for all impervious areas. The minimum area for a bioretention bed is calculated as the WQF divided by the infiltration rate of the engineered filter media, which is typically 1000 mm/hr or less. A safety factor of '0.5' is applied to the infiltration rate to account for clogging of the filter media, similar to the sizing of soakage devices which are commonly used in certain parts of Palmerston North.

Wetlands and wet ponds are designed to provide water quality treatment through the provision of a permanent water quality volume and/or minimum 24-hour detention of the water quality volume. The permanent water volume (or water quality volume, WQV), is equivalent to the runoff volume generated by the 90th percentile storm event from all impervious areas. Wetlands and wet ponds are typically located at the base of a catchment at the end of a treatment train. Wetlands and wet ponds should have a minimum catchment area of 5 ha and a preferred catchment area of 10 ha in order to maintain volumetric turnover of the permanent pool volume (Ontario Ministry of the Environment, 2003).

Figure 4.3 shows that the 90th percentile rainfall depth over a 24-hour duration is approximately 15 mm based on the Palmerston North AWS record over the 1991 to 2019 period. It may also be desired to design water quality controls for a 25 mm daily rainfall depth, a typical “first flush” target in other jurisdictions, as a more conservative approach.

A summary of WQF and WQV calculations for the Aokautere discharge locations is shown in Table 4.8.

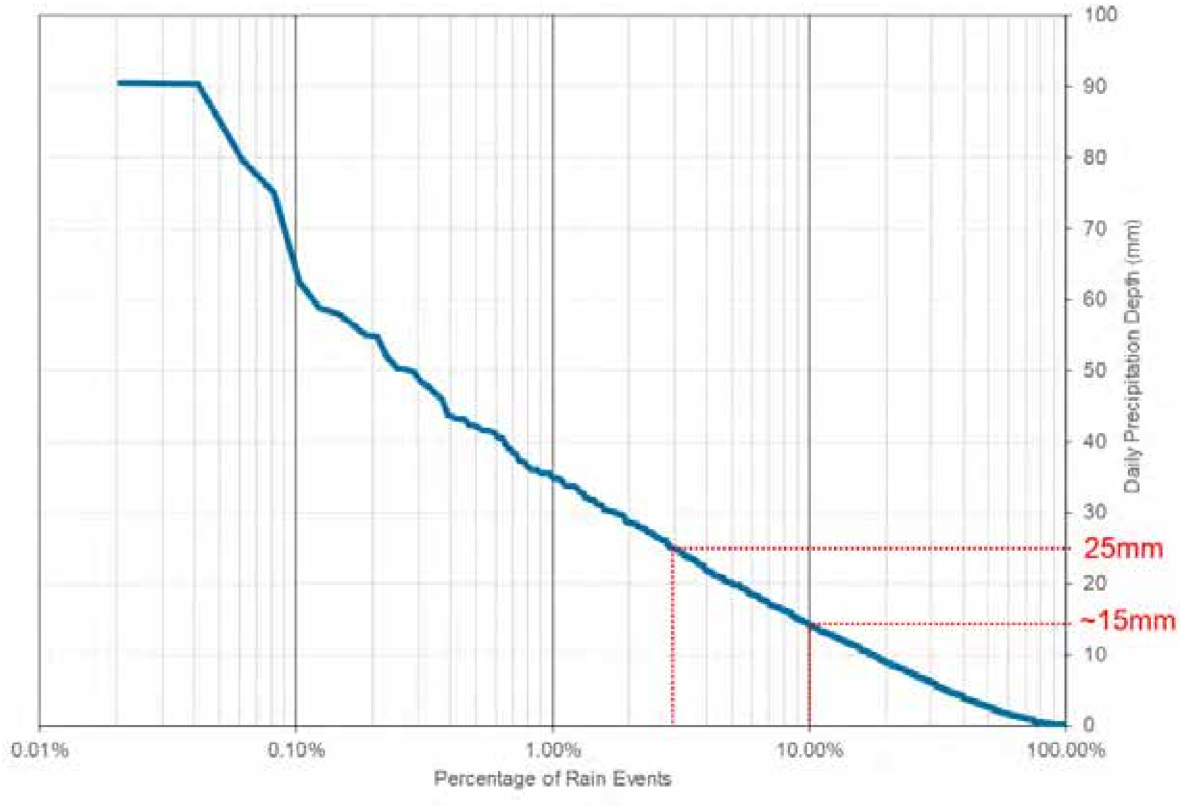


Figure 4.3 Ranked Daily Rainfall Depth at Palmerston North AWS (1991-2019)

Table 4.8 Summary of WQF and WQV values by discharge location

Discharge Location	Area (ha)	Impervious Area (ha)	Proportion of Impervious Area	Runoff Coefficient[1]	WQF (m^3/hr)	WQV (m^3)[2]
A01	1.6	0.7	0.4	0.4	67.3	99.2
A02	8.5	6.1	0.7	0.7	592.6	916.8
A03	4.6	3.6	0.8	0.7	342.7	532.7
A04	2.0	1.5	0.8	0.7	149.2	231.8
A05	32.6	22.4	0.7	0.7	2,178.0	3,358.7
B01	2.4	1.8	0.7	0.7	174.0	269.9
B02	2.2	1.2	0.6	0.5	118.3	179.3
B03	2.8	2.2	0.8	0.8	210.3	327.2
B04	0.8	0.3	0.4	0.4	34.5	50.4
B05-1	2.4	1.9	0.8	0.8	187.0	291.3
B05-2-3-4	4.3	1.9	0.4	0.5	193.9	287.4
B05-5-6-7	24.1	18.5	0.8	0.7	1,783.1	2,771.0

Discharge Location	Area (ha)	Impervious Area (ha)	Proportion of Impervious Area	Runoff Coefficient[1]	WQF (m^3/hr)	WQV (m^3)[2]
B05-8	14.3	10.6	0.7	0.7	1,029.2	1,596.2
B05-9	12.0	8.7	0.7	0.7	845.9	1,309.7
C01	9.8	7.5	0.8	0.7	726.7	1,129.7
C02	0.8	0.4	0.5	0.5	36.9	55.1
D01	1.7	1.3	0.8	0.8	129.7	202.2
D02	6.0	2.9	0.5	0.5	288.4	430.7
D03	3.1	1.2	0.4	0.4	120.1	174.3
D04	2.2	1.6	0.7	0.7	155.2	240.6
E01	19.6	7.6	0.4	0.4	778.9	1,134.9
E02	61.8	6.2	0.1	0.1	865.2	927.0
F01	18.8	14.5	0.8	0.7	1,395.1	2,168.6
F02	8.0	5.9	0.7	0.7	570.9	884.9

1. *The runoff coefficient (c) is derived from the proportion of impervious area (i) using the equation $c = 0.05 + 0.9i$, based on typical industry guidance*

2. *The water quality volume is determined using the 90th percentile rainfall depth of 15 mm, as shown in Figure 4.3*

Oil grit separator (OGS) units are another type of water quality control (not listed in Table 15 of GD2017/01) that are used to collect oils and sediment from urban runoff. OGS units may be used to provide pre-treatment in a multi-device approach, or they may provide adequate treatment at the base of a small catchment area that is typically less than 2 ha in size (Ontario Ministry of the Environment, 2003). Suppliers typically assist in the selection of an appropriate OGS model, based on site rainfall characteristics, impervious area, and a representative sediment particle size distribution.

A single device (wetland, wet pond) or multi-device treatment approach may be employed to achieve the desired water quality treatment performance at each outfall location. Opportunities to integrate stormwater management features such as rain gardens or wetlands into public amenity spaces should be identified; these opportunities may in turn suggest alternative methods of stormwater treatment that will better integrate into these public spaces. For example, community gardens could be located in rain garden areas, or a stormwater wetland could be the focus of a recreational park or pathway.

5. Stormwater Concept Design

The concept design for the Aokautere stormwater management system has been developed to mitigate the flood, erosion and water quality impacts identified and quantified in section 4. A variety of stormwater controls can be considered to develop the overall treatment and mitigation approach for the study area, assuming that the selected controls satisfy the design criteria based on the analyses established in section 4. The selected controls will need to be evaluated for their constructability, capital costs, maintenance requirements, and impact on the surrounding development (in terms of space and amenity).

The concept design has been based on a residential land use throughout the Plan Change area. A proposal was put forward in early 2022 from one of the major landowners (Aokautere Land Holdings) to place a retirement village close to the proposed Town Centre, just off Pacific Drive and within the catchment of Pond A05, replacing an area previously identified for smaller-lot, medium density residential use. A proposed layout for the retirement village is included in Appendix A. The stormwater impact of the retirement village is considered to be comparable to the previously identified medium-density residential use, and so no specific additional controls are considered to be required; the retirement village design will need to incorporate bioretention/rain garden treatment of stormwater prior to discharge to external areas in accordance with the overall stormwater management strategy.

5.1 Design Criteria

For the Aokautere Plan Change the following design criteria are recommended to be adopted:

- Control of runoff peak flows to pre-development levels for the 2-year, 5-year, 10-year, 20-year, 50-year and 100-year ARI flows, to control flood risk.
- Further control of peak flows as needed to match the pre-development erosion threshold exceedance cumulative effective work index in the Aokautere Church Stream, Moonshine Valley Reserve Stream, and Tutukiwi Reserve Stream. The Erosion Assessment included in Section 4.2 indicates that erosion risk can be largely mitigated through the recommended detention volumes for flood control.
- Treatment of the 90th percentile rainfall volume (i.e., 15 mm) from impervious developed areas through a stormwater treatment device or multi-device system.

Table 5.1 summarises the requirements for flood and erosion control and water quality treatment for the study area discharge locations, based on the proposed location of stormwater detention (i.e., on the plateau or in the gullies).

Table 5.1 Required detention storage, footprints and bioretention footprints by discharge location

Discharge Location	Total Area (ha)	Impervious Area (ha)	Bioretention Footprint (m^2) [1, 2]	100-Year ARI Storage Footprint for Flood Storage on Plateaus (m^2) [3]	100-Year ARI Storage Volumes for Flood Storage in Gullies (m^2)
A01	1.6	0.7	140	500	
A02	8.5	6.1	1,190	1,980	
A03	4.6	3.6	690	1,220	
A04	2.0	1.5	300	650	
A05 [4]	32.6	22.4	4,360		8,610
B01	2.4	1.8	350	770	
B02	2.2	1.2	240	630	
B03	2.8	2.2	430	850	
B04	0.8	0.3	70	320	
B05-1	2.4	1.9	380	790	
B05-2-3-4	4.3	1.9	390		950
B05-5-6-7	24.1	18.5	3,570		6,740
B05-8	14.3	10.6	2,060		3,990

Discharge Location	Total Area (ha)	Impervious Area (ha)	Bioretention Footprint (m^2) [1, 2]	100-Year ARI Storage Footprint for Flood Storage on Plateaus (m^2) [3]	100-Year ARI Storage Volumes for Flood Storage in Gullies (m^2)
B05-9	12.0	8.7	1,700	2,600	
C01	9.8	7.5	1,460	2,230	
C02	0.8	0.4	80	290	
D01	1.7	1.3	260	850	
D02	6.0	2.9	580		2,150
D03	3.1	1.2	250	1,210	
D04	2.2	1.6	320	950	
E01	19.6	7.6	1,560	3,670	
E02	61.8	6.2	1,740	4,760	
F01	18.8	14.5	2,800	4,920	
F02	8.0	5.9	1,150	2,290	
TOTAL	**246**	**130**	**26,070**	**31,480**	**22,440**

Note: bioretention, and flood storage footprint and volume values are rounded up to the nearest 10.

1. Bioretention footprint and permanent pool volumes reflect alternate approaches to achieving water quality treatment targets and can be considered mutually exclusive.

2. Bioretention footprint was determined using an assumed infiltration rate for engineered filter media of 1,000 mm/hr and a safety factor of 0.5 to account for potential clogging.

3. 100-Year ARI storage footprint was determined assuming a 2 metre maximum storage depth for the 100-year ARI flow, including a 0.3 m freeboard, and accounts for flood control only, not water quality treatment. Required volumes have been listed previously in Section 4.1.

4. Total values do not include the minor discharge locations that contribute to downstream discharge points (i.e., A05-1, A05-2, etc. are counted in A05).

5.2 Stormwater Constraints

The location and design of stormwater management controls needs to be considered in the context of a number of factors, including (but not limited to):

– the proposed development layout;
– sensitive environmental areas (i.e., areas of high vegetation or ecological value);
– the topography of the site;
– constructability; and
– ongoing maintenance requirements.

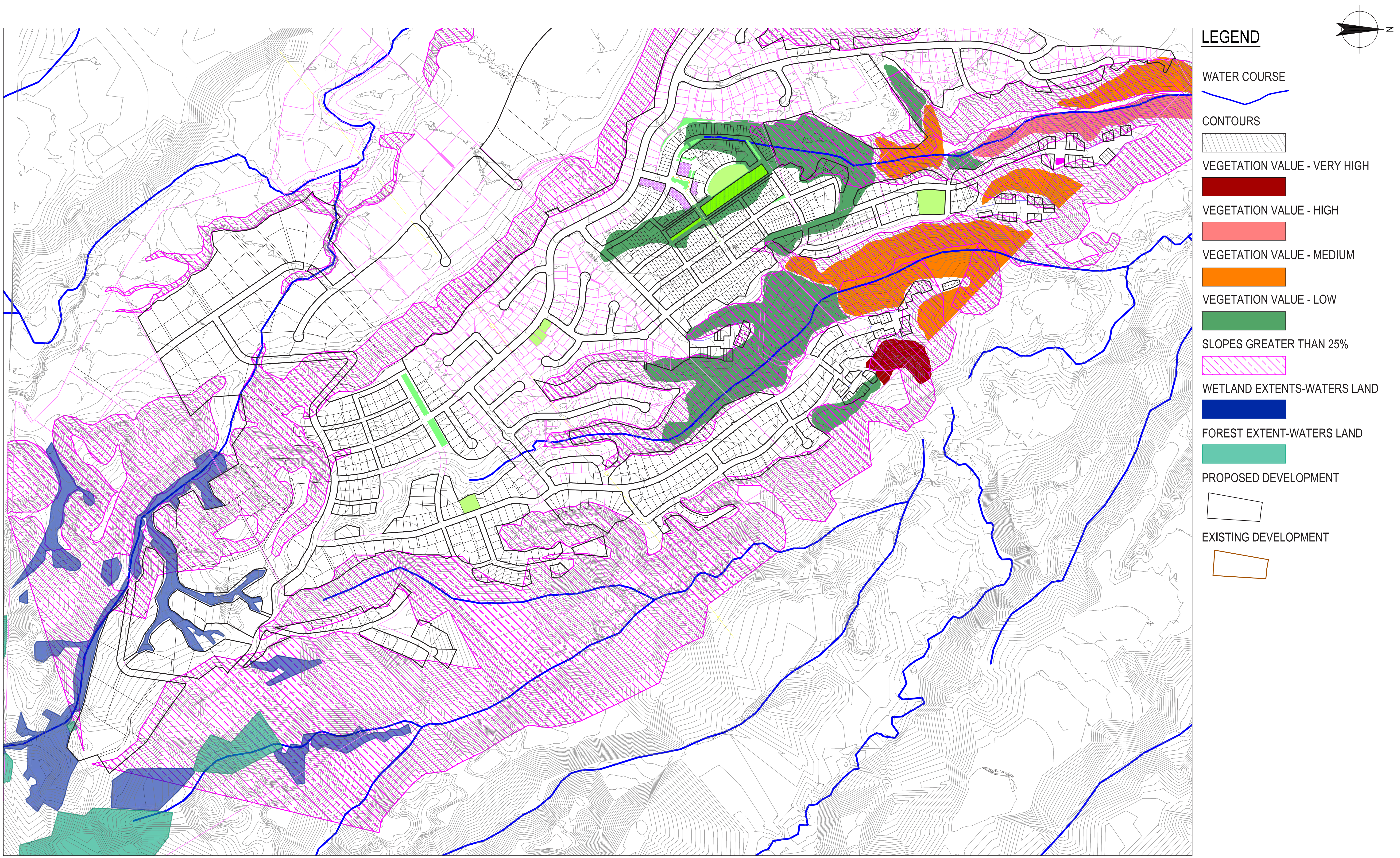
The incorporation of the recommended stormwater controls into the development layout will be completed by McIndoe Urban as part of the Structure Plan, and for the purpose of this concept design no modifications to the development layout were made. Where possible (i.e., when not constrained by other factors), stormwater controls were located in the gully system to minimise impact on developable area. As well, lots that have already been subdivided within the study area were avoided when placing stormwater detention features.

An environmental constraint study for the Plan Change area was completed by Forbes Ecology in parallel with this stormwater assessment, which identified areas of terrestrial/vegetation and aquatic ecological value; these areas are shown on Figure 5.1. Note that the aquatic potential of the Aokautere Church Stream and Moonshine Valley Reserve Stream were assessed to be higher downstream of the “low” value vegetation areas, where the streams transition from intermittent to permanent. As well, several wetland and forest areas have been identified by Forbes Ecology in the “Waters Land” property of the Structure Plan area (catchment E02); these areas are also shown on Figure 5.1 and are avoided in the placement of conceptual stormwater facilities.

The site topography constraints assessed for this study include the gully slopes and the suitability of the gullies to contain stormwater management features. For the assessment of gully slopes, an elevation analysis was completed to identify those areas where the ground slope was equal to or steeper than 25% (i.e., 4:1 horizontal to vertical), forming a perimeter around the tops of the gullies and approximately delineating a threshold beyond which some manner of runoff control is recommended. This 4:1 slope line is shown on Figure 5.1 and is intended only to evaluate the potential impact on developable land resulting from this type of control/buffer. A setback or buffer along the rear limits of residential properties roughly in alignment with the 4:1 slope line is recommended to both prevent development encroachment and serve as a stormwater collection corridor to protect the sensitive gully slopes. The specific width or location of the buffer should be modified to suit specific development needs if sufficient allowance for stormwater conveyance and future maintenance is made, and as permitted by geotechnical slope hazard concerns, using the 4:1 slope line as a baseline/reference. Figure 5.2 illustrates the Structure Plan implementation of a recommended stormwater setback along the rear property lines of residences, developed in collaboration with McIndoe Urban.

The suitability of the gullies to contain stormwater management features was assessed based on the vegetation and aquatic ecology constraints (e.g., no stormwater controls were recommended for areas of “medium” or greater vegetation value), constructability and access, as well as the topography of the gullies themselves. A conceptual assessment of gully grading was undertaken to identify if the required storage volume for stormwater detention could be accommodated on the gully floors (specifically at discharge points A05 and B05), allowing one centralised facility to manage runoff from several contributing upstream discharge points. Discharge point A05 appears to be suitable for such an installation, whereas point B05 is located in a narrow, steep portion of gully with limited opportunities for stormwater detention. However, several smaller upstream gullies have been used to locate stormwater detention for various subsets of the total B05 catchment area.

Property and topography constraints along Pacific Drive in the F01 catchment has limited the available locations for a detention pond to service this catchment. However, this catchment is already connected to a consented section of Council reticulation and is largely fully developed. There is an opportunity to install a new detention pond at the outlet of this reticulation system near the intersection of Old W Rd and Turitea Rd, but this is located outside of the Plan Change area and is therefore not included in the stormwater concept design.



Rev	Description	Checked	Approved	Date
B	LAYOUT CHANGES	DA*		22/04/22

Author: C. O'DONNELL — Drafting Check
Designer — Design Check

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Project No. 12554906

Client: PALMERSTON NORTH CITY COUNCIL

Project: STUDY AREA STORMWATER CONSTRAINTS

Status: FINAL

Status Code

Drawing Title: FIGURE 5.1

Size: A3

Drawing No. 12554906-FIGURE 5.1

Rev: B

Plot Date: 30 May 2022 - 12:47 PM   Plotted by: Danny Dao   File Name: \\ghdnet\ghd\NZ\Palmerston North\Projects\51\12518392\CADD\Drawings\11204663-Figure 5.1.dwg

5.3 Stormwater Concept Design

The proposed stormwater concept design for the Aokautere structure plan area is recommended to be comprised of the elements identified below.

1. Detention ponds

Indicative conceptual locations and footprints for stormwater detention areas to mitigate flood and erosion risk are shown on Figure 5.2 for consideration in the development layout. The figure shows the footprints required for storage only; additional area would need to be added for treatment (as discussed further below).

2. Buffer strip

A 5 m buffer along the rear property lines of residential lots is also recommended to be incorporated into the development layout, in which no buildings would be recommended to be placed. This excludes any setback required for risk associated with landslips or other geotechnical hazard.

3. Stormwater reticulation

Stormwater reticulation is to be designed to accommodate the 10-year ARI flow and overland flow paths designed to accommodate the 100-year ARI flow, as per PNCC engineering standards. All stormwater outlets into the gully system must be designed as a piped system to safely convey the 100-year runoff to the gully floor, with appropriately designed erosion control measures at the ultimate outlet in the gully. Direct overland discharge down the gully slope is not recommended, nor is it permitted in the PNCC ESLD.

4. Water quality treatment

The incorporation of water quality controls is recommended to be completed when finalising the development layout as follows:

– Incorporate bioretention (rain gardens) into the road reserve in such a way that all discharge from impervious surfaces (roofs, driveways, road surface) is directed into the rain gardens for filtration. Underdrains located under the rain garden filter media will then direct the runoff into the reticulation network. Ensure that the minimum required bioretention footprint summarised in section 5.1 is achieved.

– If it is desired to utilise wet ponds or wetlands for stormwater quality treatment instead of (or in combination with) bioretention, the required permanent pool volume must be incorporated into the detention pond *in addition to* the required storage volume for flood control. In this case the footprints of the detention facilities would need to be increased to accommodate the shallower detention depth (assuming a maximum pond depth of 2 m is maintained, including 0.3 m of freeboard). Although the detention facilities would occupy a larger space, the centralisation of treatment has several benefits for operations and maintenance, as well as potential savings from reduced need for distributed bioretention facilities. An indicative estimate of wetland footprint for each discharge location is shown in Table 5.2, assuming an average 0.5 m deep wetland permanent pool and 1.1 m active storage detention area. Wetland footprints are indicated only for those facilities with a minimum catchment area of 5 ha to promote volumetric turnover of the permanent pool.

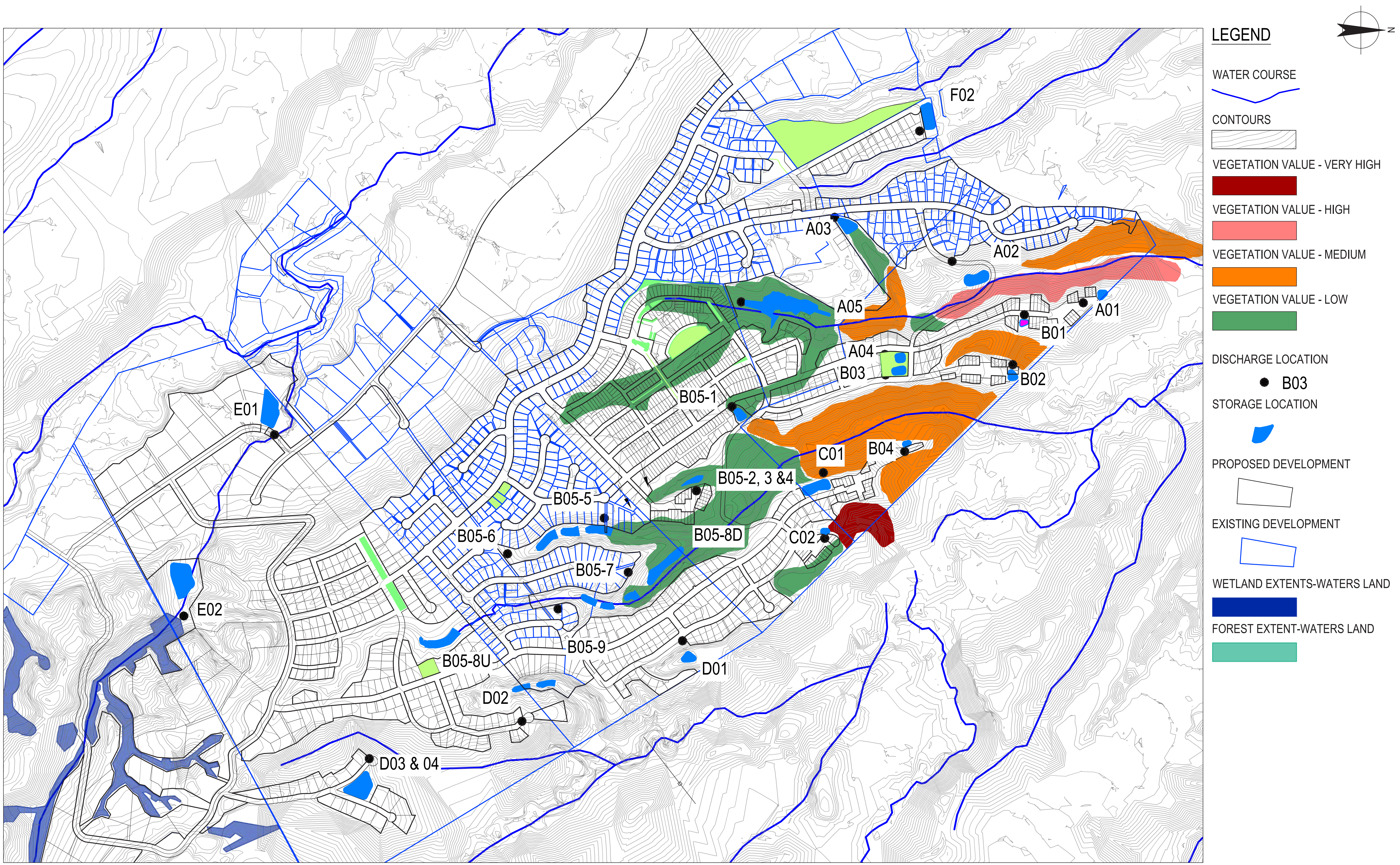
Table 5.2 Stormwater wetland footprints for flood control and water quality treatment on plateaus (alternative option)

Discharge Location[1]	Total Area (m^2)	Permanent Pool Volume (m^3)	100-Year ARI Storage Footprint with Permanent Pool (m^2)[2]
A02	8.5	920	2,770
B05-9	12.0	1,310	3,720
C01	9.8	1,130	3,290
E01	19.6	1,140	3,300
E02	61.8	930	2,790
F01	18.8	2,170	5,730
F02	8.0	890	2,690

Note: permanent pool and flood storage footprint and volume values are rounded up to the nearest 10.
1. Only discharge points with minimum 5 ha catchment area were assessed.
2. Permanent pool volume is equivalent to Water Quality Volume (WQV) summarized in Table 4.5.

5. Stream stabilisation

Based on the erosion assessment and site visits carried out, the proposed stormwater detention volumes required for flood risk mitigation are largely adequate to mitigate erosion risk in the study area streams when considering pre-development and post-development flows. However, the Aokautere Church Stream and Moonshine Valley Reserve Stream will remain highly sensitive to erosion in the future regardless of upstream development. Therefore, stream stabilisation within the gullies may be prudent to offset any perceived impacts from the development.



Rev	Description	Checked	Approved	Date
C	LAYOUT CHANGE	DA*		22/04/21

Author C. O'DONNELL Drafting Check

Designer D. ARSENEAU Design Check

0 100 200 300m

SCALE 1:10,000 AT ORIGINAL SIZE

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Project No. 12554906

Client PALMERSTON NORTH CITY COUNCIL

Project STORMWATER CONCEPT DESIGN OVERVIEW

Status PRELIMINARY - SHARED Status Code S4

Drawing Title FIGURE 5.2

Size A3

Drawing No. 12554906-FIGURE 5.2 Rev C

Plot Date: 30 May 2022 - 1:06 PM Plotted by: Danny Dao File Name: \\ghdnet\ghd\NZ\Palmerston North\Projects\51\12518392\CADD\Drawings\11204663-Figure 5.2.dwg

5.4 Review of Draft Plan Change Provisions

The Aokautere Plan Change (Plan Change G) will involve updates/revisions to existing District Plan provisions across a number of current chapters/sections. Those sections with most relevance to stormwater management include: Section 7 Subdivision, Section 7A Greenfield Residential, Section 10 Residential, and Section 15 Recreation. Draft versions of proposed District Plan revisions were reviewed in March 2022 to assess whether the updates would achieve the desired stormwater management outcomes for Aokautere.

In general, the provisions establish a restricted discretionary activity status for most stormwater-relevant items related to subdivision and residential development, including the management of effects on the existing gully network, which is considered appropriate. These are reflected both in the high-level objectives/policies as well as more detailed rules for specific activities. Existing provisions are expanded/strengthened to require a wider focus for stormwater management assessments beyond the immediate site boundaries (i.e., flooding and erosion in downstream watercourses) for a more robust approach to management of effects. New provisions require a Stormwater Management Plan (as part of a Comprehensive Development Plan) prepared by a suitably qualified stormwater design consultant to demonstrate the management of flooding, erosion and water quality effects from the development. New provisions are also included to establish the 5 metre no-build setback along the backs of residential lots to protect the gully slopes, as recommended in this report, and provide for the long-protection of the gully systems through vesting of those areas to Council for conservation and amenity purposes. Vesting of any other community-scale stormwater management measures should also be included in the District Plan to ensure adequate maintenance and operation of the facilities in the future.

5.5 Horizons Regional Council One Plan Considerations

Horizons Regional Council (Horizons) provided feedback on a draft circulation of the Plan Change in March 2021 and indicated several aspects of the proposed residential development related to stormwater that may require consenting or further inspection under the One Plan provisions. A summary of the feedback received and how the Aokautere stormwater management strategy addresses the feedback is included below.

1. Flooding – Horizons notes the existing waterways that traverse the site, primarily in a north-westerly direction through the gully systems, with potential flooding hazard to any new development. The proposed structure plan does not place any residential development within the gullies, effectively avoiding this hazard. Proposed development along Valley Views appears to be outside of the floodable area from the Turitea Stream, and therefore incurs little flood risk for future housing.
2. Stormwater Management – Horizons notes several broad concerns related to the management of stormwater, including water quality issues, the potential for exacerbation of existing erosion issues, and issues related to the discharge of stormwater across sloping land. The Aokautere stormwater management strategy explicitly requires water quality treatment of runoff prior to discharge, and has presented an in-depth erosion assessment to identify potential future effects. As well, the strategy includes provision for a slope setback to manage runoff that would otherwise flow directly down the steep gully slopes.
3. Natural Hazards – The structure plan avoids the placement of residential development in areas that could potentially be impacted by flooding hazards.
4. Climate Change – Stormwater quantity and quality controls have been assessed in consideration of future climate change, using best available guidance from NIWA (via the HIRDS v4 system).
5. National Policy Statement on Freshwater Management – The stormwater strategy provides for water quality treatment and attenuation of runoff prior to discharge, and will directly lead to the vesting and protection of the natural stream systems within the gullies. Stream restoration/stabilisation work is also recommended as part of the strategy to mitigate the ongoing existing erosion issues in these areas.

In general, the Aokautere stormwater management strategy is considered to address Horizons One Plan requirements in a comprehensive manner.

6. Conclusions and Recommendations

GHD completed a stormwater management analysis for the Aokautere Structure Plan area, including assessment of flood risk, erosion risk, and water quality requirements for the proposed development with an assumed level of post-development impervious cover of 70%. Based on this analysis, a recommended stormwater management strategy was developed including a conceptual design of stormwater detention and water quality treatment facilities; note that if post-development impervious cover is higher than 70%, additional stormwater management areas may be required. This report will contribute to and form part of the Structure Plan for implementation during detailed design, resource consenting and construction of the proposed developments.

Ultimately, the following stormwater design criteria are recommended to be adopted:

– Control of runoff peak flows to pre-development levels for the 2-year, 5-year, 10-year, 20-year, 50-year and 100-year ARI flows, to control flood risk.
– Further control of peak flows as needed to match post-development erosion threshold exceedance cumulative effective work index in the Aokautere Church Stream, Moonshine Valley Reserve Stream, and Tutukiwi Reserve Stream.
– Treatment of the 90th percentile rainfall volume (i.e., 15 mm) from impervious developed areas through a stormwater treatment device or multi-device system.

The key constraints identified for the area include:

– Erosion
– Slope stability
– Perceived stream degradation
– Existing vegetation and wetland areas

The stormwater management concept design recommended to be included in the Aokautere Structure Plan includes the following components:

– Stormwater detention facilities to mitigate flood and erosion risk.
– Roadside bioretention facilities (rain gardens) to provide water quality treatment, with underdrains connecting to the stormwater reticulation.
– An indicative top-of-slope setback of 5 m from the 4:1 grade line to safely convey stormwater runoff to designated discharge points and protect gully slopes from erosion.
– Stormwater reticulation to accommodate the 10-year ARI climate change flow, with major overland flow network to accommodate the 100-year ARI climate change flow, as per PNCC ESLD.
– Discharge of stormwater runoff downstream of detention facilities into the receiving gullies through a pipe installed down the gully slopes, sized for the 100-year ARI climate change flow. No overland discharge down the gully slopes should be permitted.
– Consideration of stream stabilisation in the Aokautere Church Stream and Moonshine Valley Reserve Stream to mitigate perceived impacts from the development and enhance both aquatic habitat and community amenity.

Indicative footprints have been provided for the various detention and bioretention facilities for the structure plan area as a feasibility test; these locations and footprints are to be confirmed through subdivision design stage to assess site-specific requirements and constructability. Prior to the approval of any engineering plans for the proposed developments, the developer should be required to provide a stormwater management report and design demonstrating compliance with these design criteria and concepts.

Generally, the draft District Plan provisions developed for this Plan Change appear to be suitable to achieve the desired stormwater outcomes.

7. References

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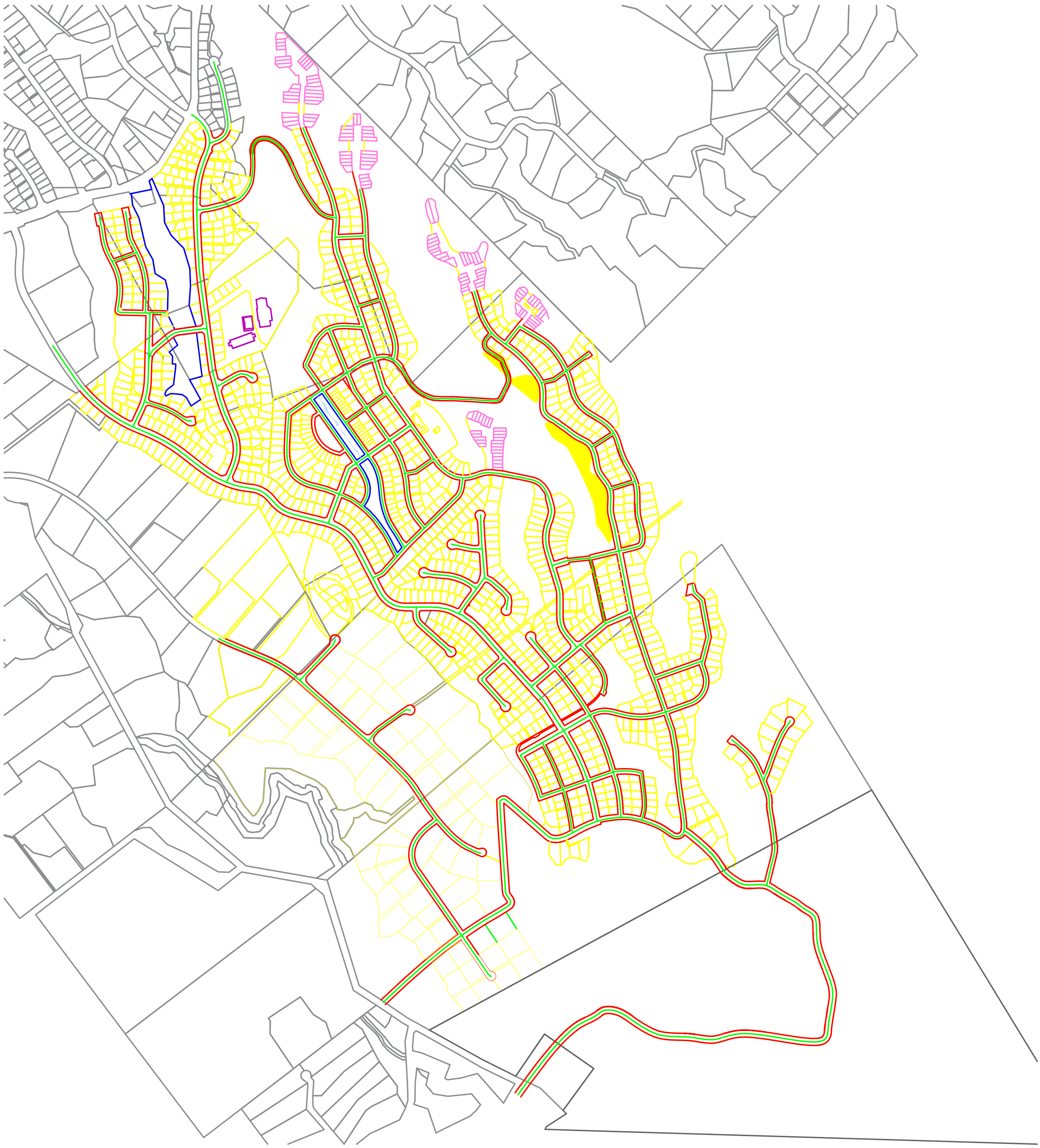
Appendices

Appendix A

Aokautere Structure Plan Development Layout (McIndoe Urban, October 2019, July 2021 and March 2022)

Draft Aokautere Structure Plan Development Layout

Received from McIndoe Urban
16 October 2019



Aokautere Structure Plan: Final Structure Plan

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09.07.2021 McIndoe Urban

Aokautere - Structure Plan

McIndoe Urban
11.05.2022

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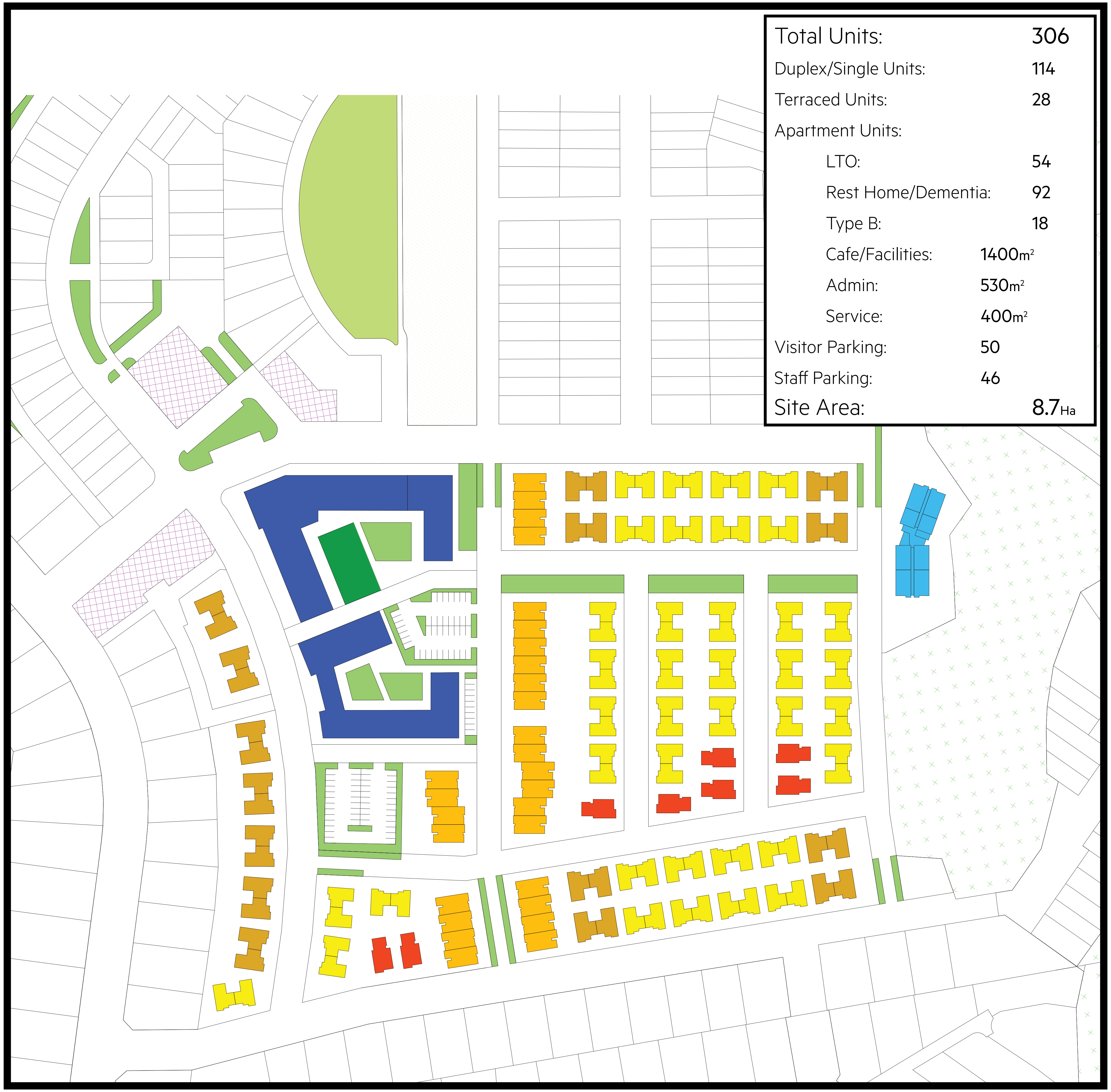

Total Units:	306
Duplex/Single Units:	114
Terraced Units:	28
Apartment Units:	
LTO:	54
Rest Home/Dementia:	92
Type B:	18
Cafe/Facilities:	1400m²
Admin:	530m²
Service:	400m²
Visitor Parking:	50
Staff Parking:	46
Site Area:	8.7Ha

Metlifecare Gulf Rise Retirement Homes / Warren and Mahoney

DRAFT

project name
Aokautere Retirement Village Options

project no.
2038c

client
PNCC

scale
1:2000@A3

N

date
10.05.2022

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Appendix B

Summary of Flood Assessment Results and Criteria

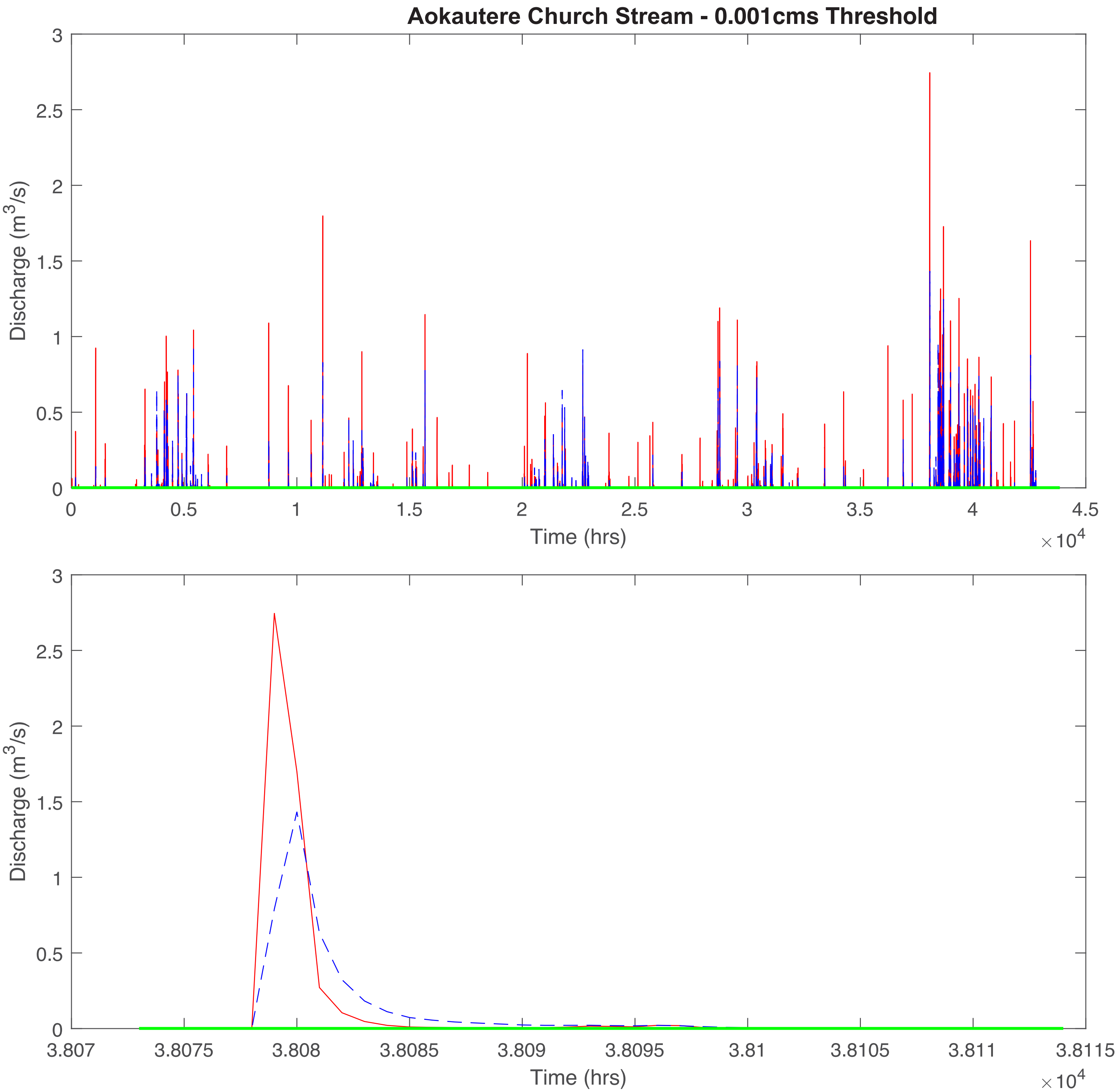
Appendix B - Flood assessment results and criteria

Discharge Point	2-year ARI			5-year ARI			10-year ARI			20-year ARI			50-year ARI			100-year ARI		
	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required
	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)
A01	0.02	0.07	117	0.03	0.11	195	0.05	0.15	208	0.07	0.18	230	0.09	0.23	288	0.11	0.27	323
A02	0.09	0.56	1,188	0.18	0.81	1,498	0.27	1.00	1,652	0.36	1.20	1,849	0.49	1.49	2,129	0.61	1.71	2,288
A03	0.05	0.32	656	0.10	0.47	819	0.15	0.58	896	0.20	0.70	1,003	0.27	0.86	1,159	0.33	0.99	1,267
A04	0.02	0.15	297	0.04	0.22	379	0.06	0.28	417	0.09	0.33	434	0.12	0.40	506	0.15	0.46	546
A05	0.33	2.00	4,527	0.70	3.02	5,563	1.03	3.78	6,173	1.38	4.57	6,900	1.88	5.65	7,948	2.33	6.45	8,604
B01	0.02	0.19	391	0.04	0.28	506	0.07	0.30	513	0.10	0.32	549	0.14	0.37	630	0.17	0.42	696
B02	0.02	0.12	253	0.04	0.18	340	0.06	0.23	383	0.09	0.29	403	0.12	0.36	480	0.15	0.42	521
B03	0.02	0.21	505	0.05	0.30	580	0.08	0.37	613	0.11	0.44	676	0.16	0.55	741	0.20	0.63	793
B04	0.01	0.04	64	0.02	0.06	88	0.02	0.08	136	0.03	0.10	146	0.05	0.13	147	0.06	0.15	168
B05-1	0.02	0.18	433	0.04	0.26	541	0.07	0.32	547	0.10	0.39	584	0.14	0.48	654	0.17	0.55	715
B05-2-3-4	0.03	0.20	542	0.08	0.32	608	0.12	0.42	695	0.17	0.52	756	0.24	0.66	871	0.30	0.77	947
B05-5-6-7	0.19	1.69	3,991	0.44	2.46	4,723	0.68	3.06	5,104	0.96	3.69	5,543	1.36	4.51	6,220	1.69	5.02	6,732
B05-8	0.11	1.02	2,351	0.26	1.50	2,772	0.41	1.86	2,966	0.57	2.23	3,256	0.81	2.68	3,649	1.00	2.99	3,986
B05-9	0.09	0.69	1,937	0.22	1.04	2,240	0.34	1.30	2,424	0.48	1.58	2,606	0.68	1.97	2,918	0.84	2.18	3,175
C01	0.06	0.73	1,914	0.16	1.05	2,126	0.27	1.30	2,202	0.39	1.54	2,351	0.59	1.77	2,519	0.76	1.95	2,639
C02	0.00	0.04	91	0.01	0.06	125	0.02	0.08	124	0.03	0.10	132	0.05	0.12	131	0.06	0.15	149
D01	0.01	0.13	354	0.01	0.18	576	0.02	0.22	598	0.03	0.27	651	0.05	0.33	699	0.06	0.38	783
D02	0.02	0.27	1,055	0.05	0.44	1,300	0.08	0.57	1,481	0.11	0.71	1,702	0.17	0.91	1,949	0.22	1.07	2,141
D03	0.01	0.18	634	0.03	0.27	715	0.04	0.35	879	0.06	0.42	963	0.09	0.52	1,113	0.11	0.61	1,252
D04	0.01	0.17	423	0.02	0.24	558	0.03	0.30	646	0.04	0.35	736	0.06	0.43	851	0.08	0.50	912
E01	0.06	0.65	2,179	0.15	1.14	2,837	0.24	1.51	3,345	0.37	1.89	3,790	0.58	2.47	4,392	0.78	2.93	4,725
E02	0.19	0.36	2,783	0.47	0.84	3,747	0.76	1.36	4,477	1.17	1.97	4,920	1.84	2.94	5,826	2.45	3.73	6,362
F01	0.06	1.12	4,069	0.14	1.62	4,955	0.23	2.00	5,453	0.36	2.37	5,721	0.55	2.87	6,348	0.73	3.25	6,599
F02	0.02	0.52	1,648	0.06	0.76	1,899	0.10	0.94	2,111	0.15	1.12	2,330	0.24	1.41	2,572	0.31	1.53	2,728

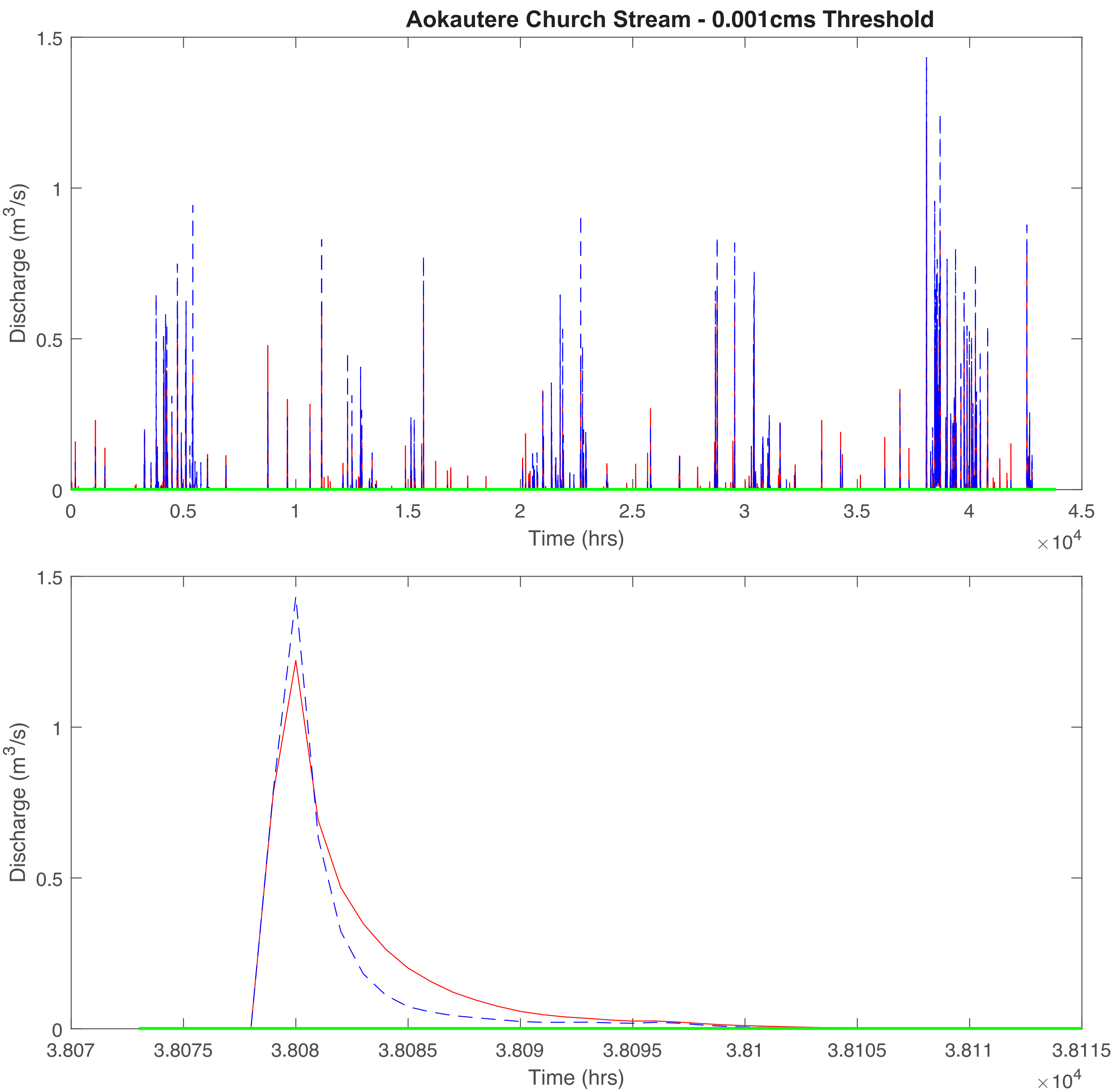
Appendix C

Erosion Threshold Exceedance Analysis

Results Graphs



Pre-Development (Blue)	**Post-Development (Red)**
Total Exceedance Time = 5131 hrs	Total Exceedance Time = 3924 hrs
% Exceedance Time = 11.7	% Exceedance Time = 8.9
Number of Exceedance Events = 169	Number of Exceedance Events = 263
Cumul. Effective Discharge(m^3) = 8.908E+05	Cumul. Effective Discharge(m^3) = 9.188E+05
Cumul. Effective Work Index(J/m^2) = 1.737E+08	Cumul. Effective Work Index(J/m^2) = 1.761E+08



Pre-Development (Blue)

Total Exceedance Time = 5131 hrs

% Exceedance Time = 11.7

Number of Exceedance Events = 169

Cumul. Effective Discharge(m^3) = 8.908E+05

Cumul. Effective Work Index(J/m^2) = 1.737E+08

Post-Development - with SWM Control (Red)

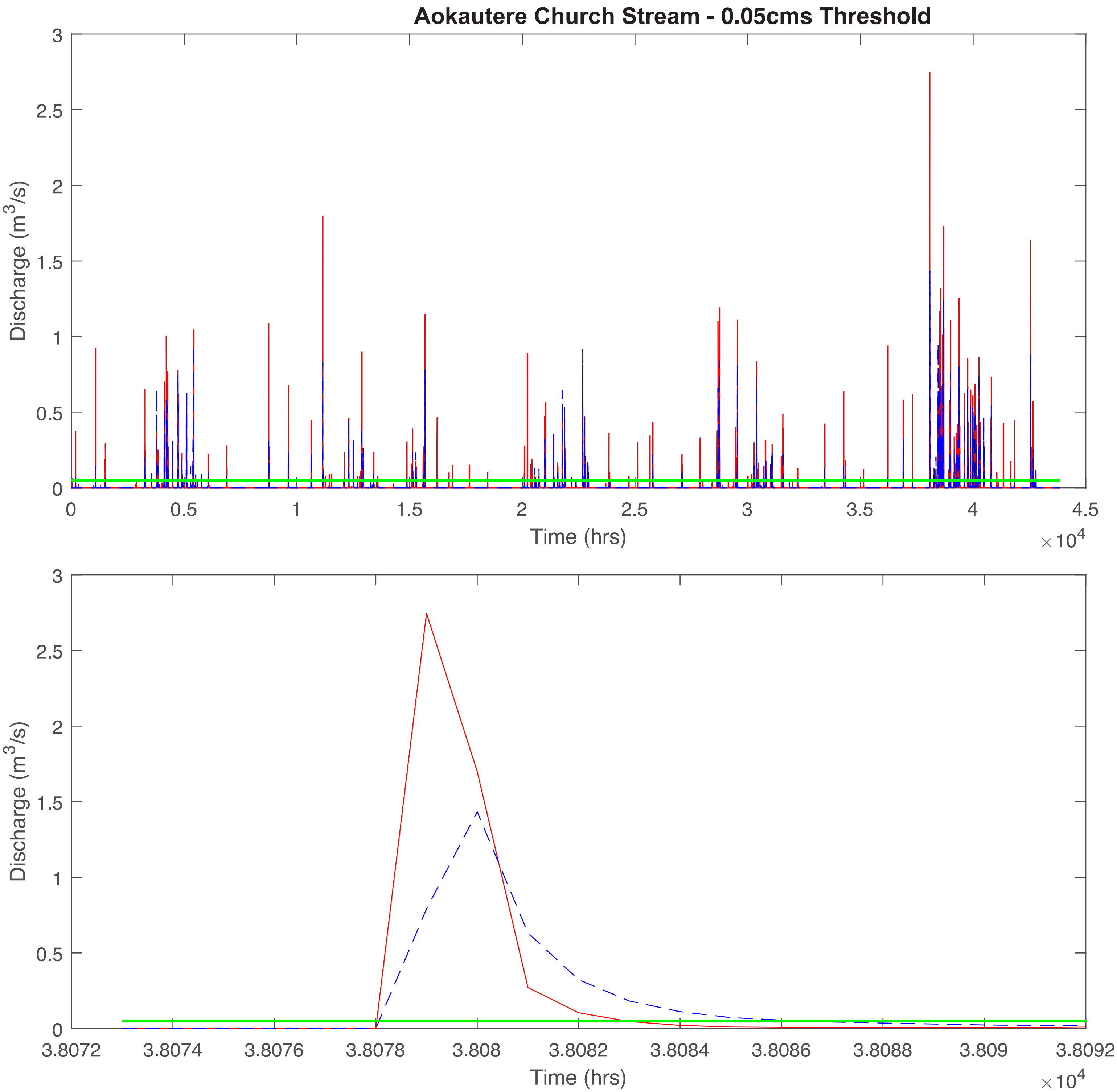
Total Exceedance Time = 5557 hrs

% Exceedance Time = 12.7

Number of Exceedance Events = 187

Cumul. Effective Discharge(m^3) = 8.907E+05 Cumul.

Effective Work Index(J/m^2) = 1.755E+08



Pre-Development (Blue)

Total Exceedance Time = 1207 hrs

% Exceedance Time = 2.8

Number of Exceedance Events = 188

Cumul. Effective Discharge(m^3) = 5.310E+05

Cumul. Effective Work Index(J/m^2) = 8.325E+07

Post-Development (Red)

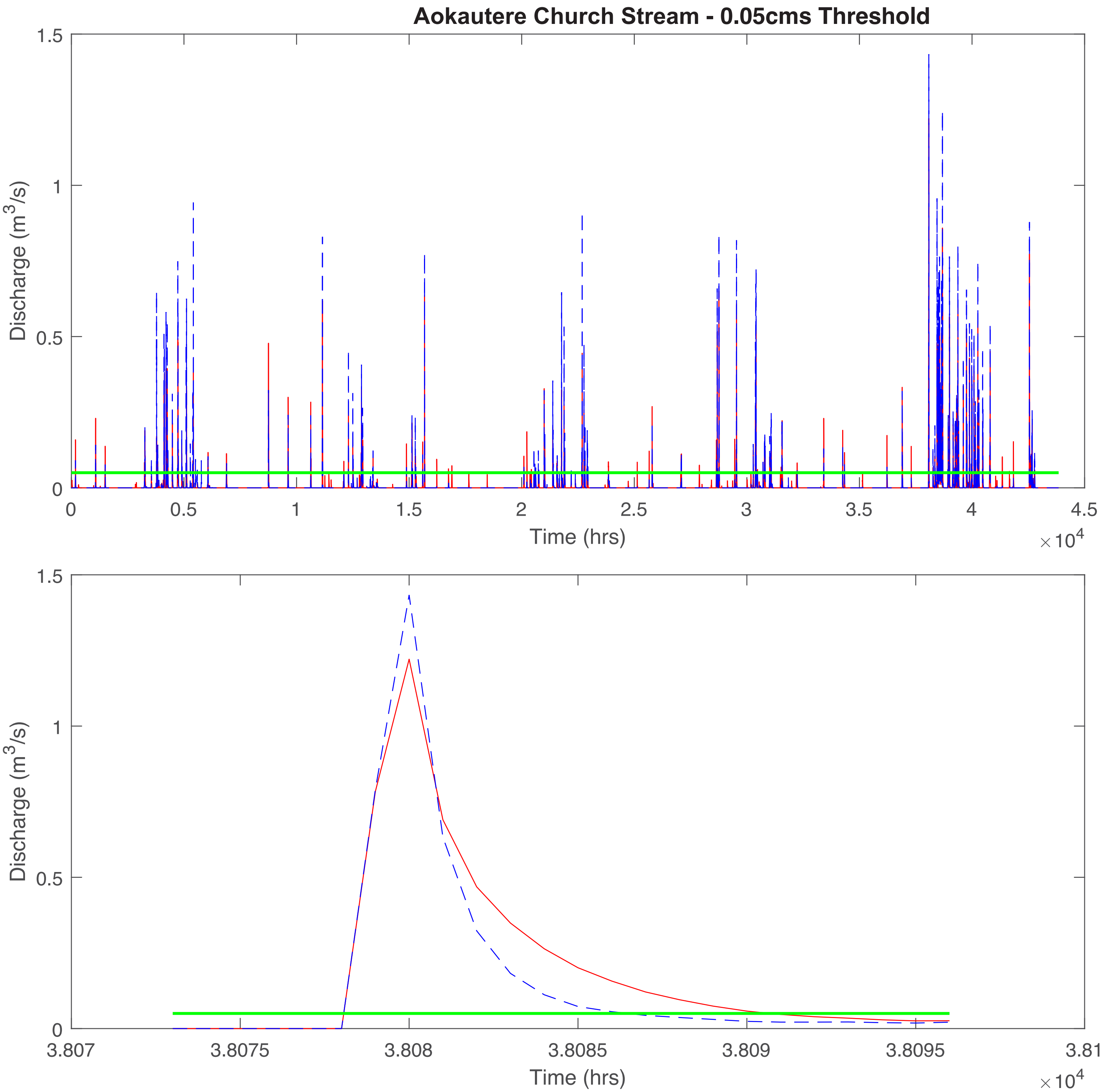
Total Exceedance Time = 1112 hrs

% Exceedance Time = 2.5

Number of Exceedance Events = 292

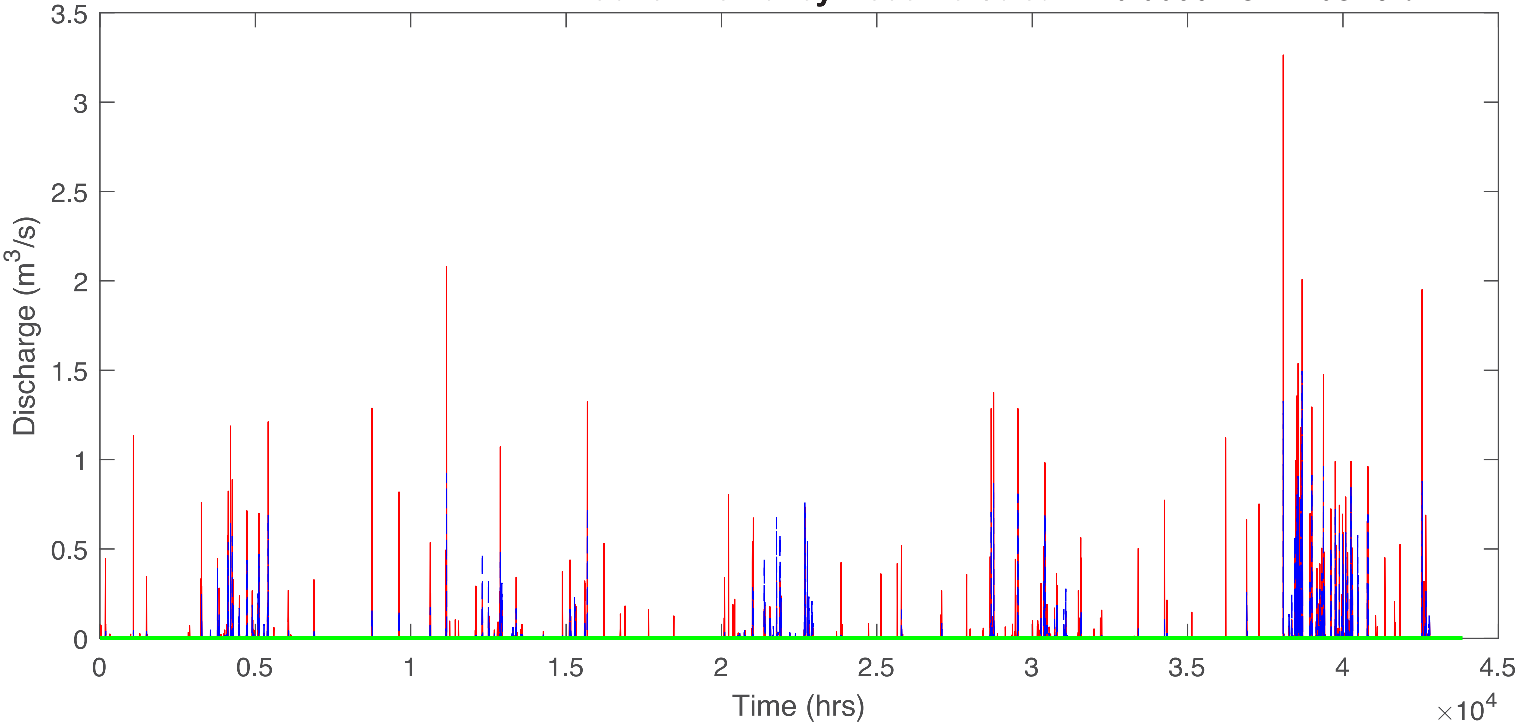
Cumul. Effective Discharge(m^3) = 6.213E+05

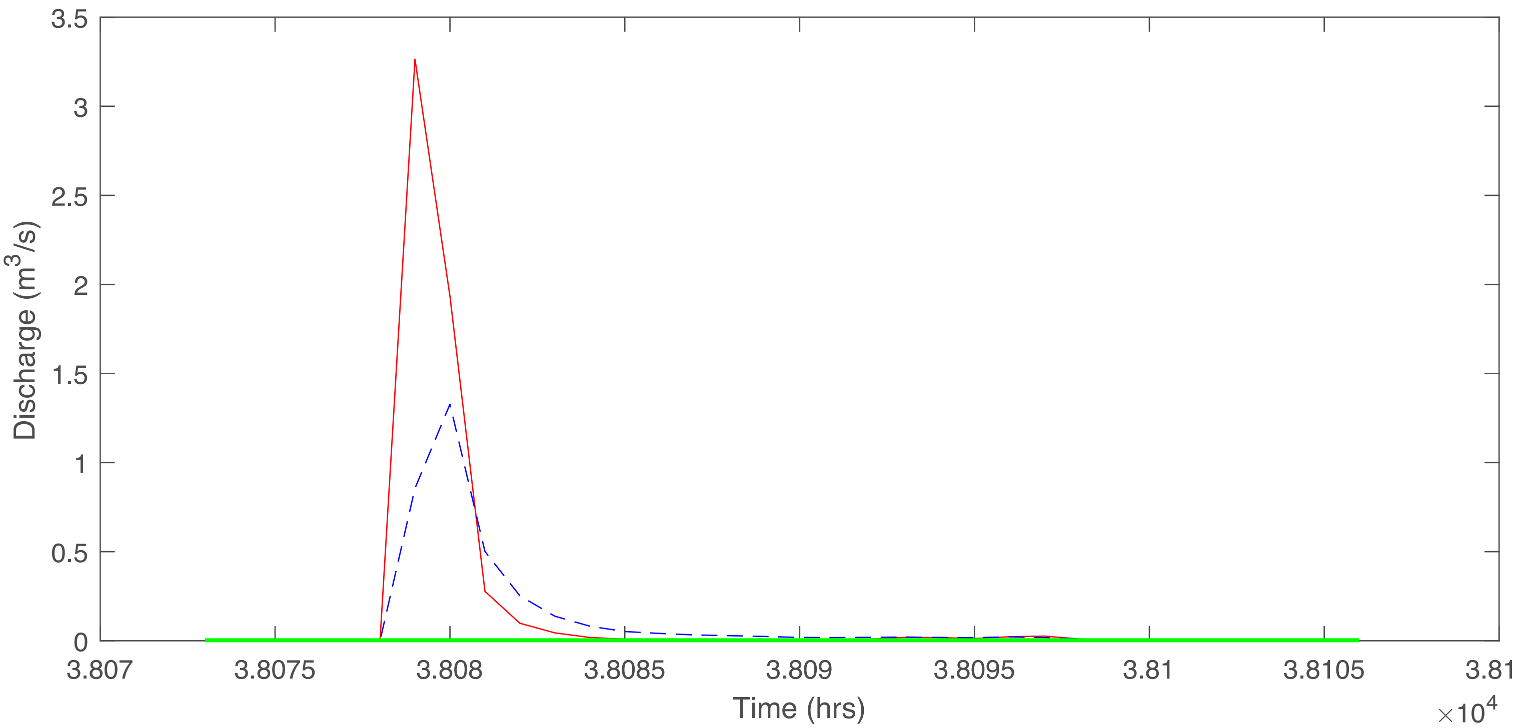
Cumul. Effective Work Index(J/m^2) = 9.701E+07



Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 1207 hrs	Total Exceedance Time = 1382 hrs
% Exceedance Time = 2.8	% Exceedance Time = 3.2
Number of Exceedance Events = 188	Number of Exceedance Events = 179
Cumul. Effective Discharge(m^3) = 5.310E+05	Cumul. Effective Discharge(m^3) = 4.661E+05
Cumul. Effective Work Index(J/m^2) = 8.325E+07	Cumul. Effective Work Index(J/m^2) = 7.239E+07

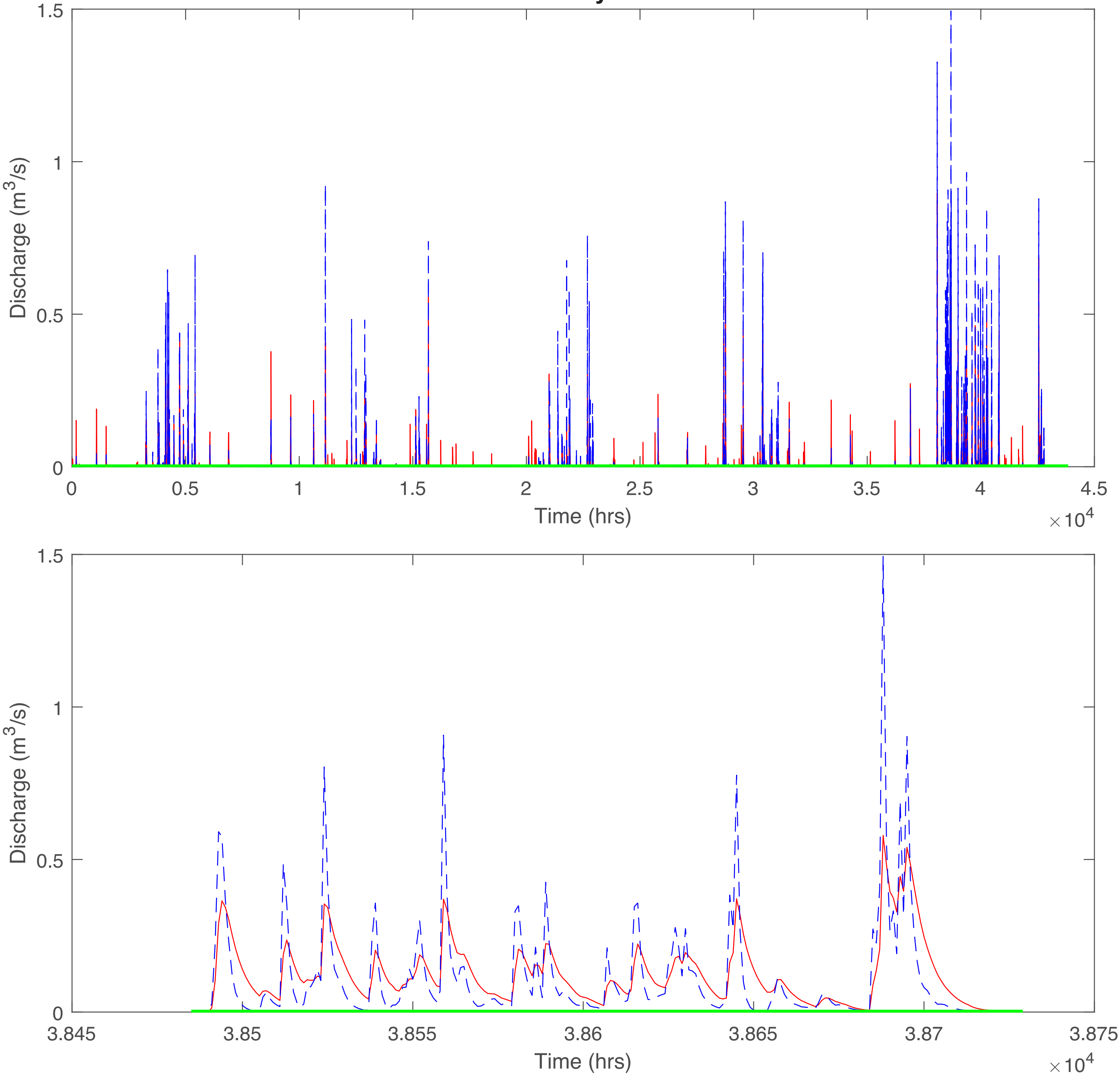
Moonshine Valley Reserve Stream - 0.003cms Threshold



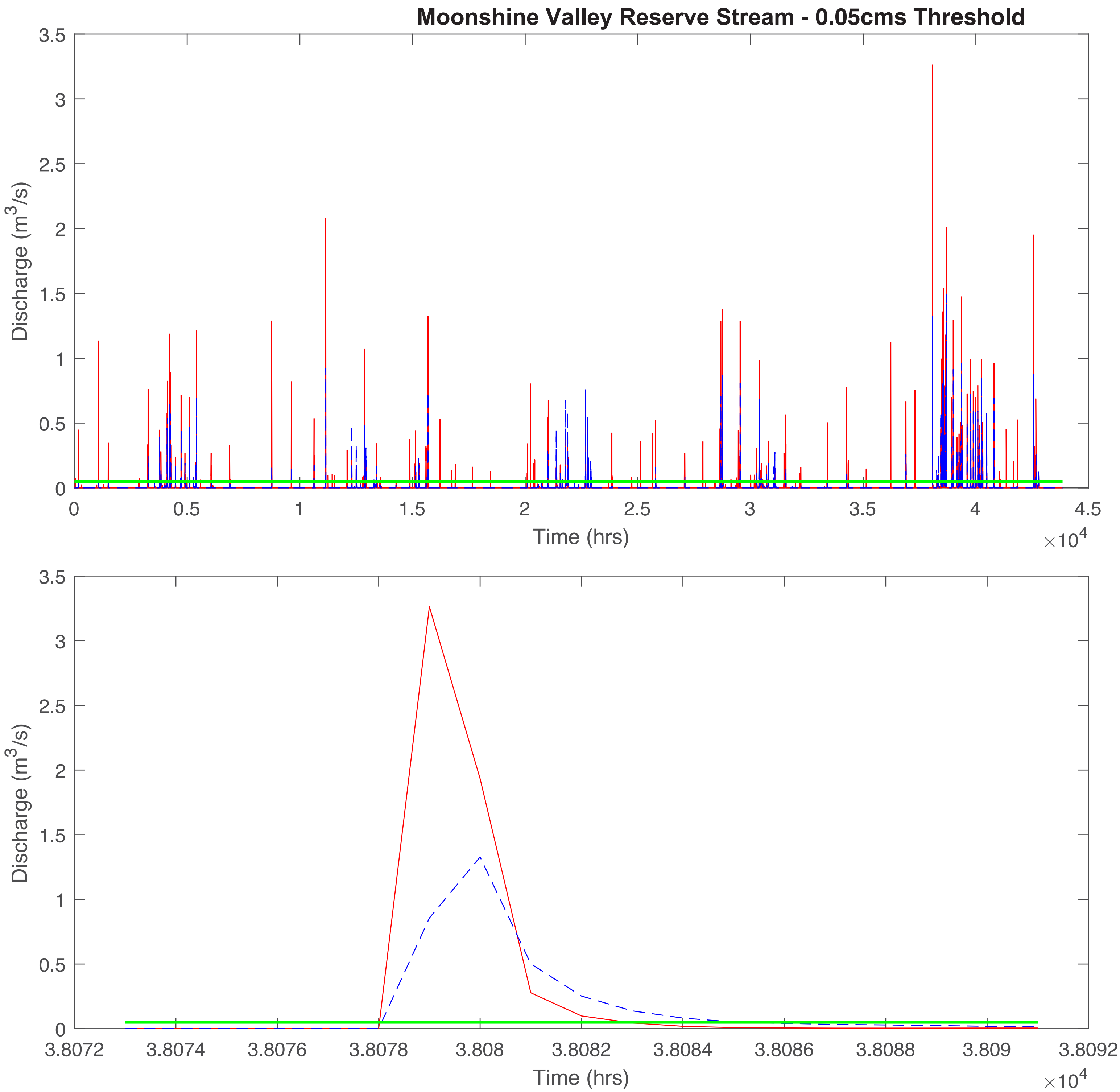


Pre-Development (Blue)	Post-Development (Red)
Total Exceedance Time = 3336 hrs	Total Exceedance Time = 3140 hrs
% Exceedance Time = 7.6	% Exceedance Time = 7.2
Number of Exceedance Events = 184	Number of Exceedance Events = 287
Cumul. Effective Discharge(m^3) = 7.387E+05	Cumul. Effective Discharge(m^3) = 9.824E+05
Cumul. Effective Work Index(J/m^2) = 8.436E+07	Cumul. Effective Work Index(J/m^2) = 1.099E+08

Moonshine Valley Reserve Stream - 0.003cms Threshold



Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 3336 hrs	Total Exceedance Time = 5136 hrs
% Exceedance Time = 7.6	% Exceedance Time = 11.7
Number of Exceedance Events = 184	Number of Exceedance Events = 181
Cumul. Effective Discharge(m^3) = 7.387E+05	Cumul. Effective Discharge(m^3) = 9.277E+05
Cumul. Effective Work Index(J/m^2) = 8.436E+07	Cumul. Effective Work Index(J/m^2) = 1.076E+08



Pre-Development (Blue)

Total Exceedance Time = 997 hrs

% Exceedance Time = 2.3

Number of Exceedance Events = 177

Cumul. Effective Discharge(m^3) = 4.683E+05

Cumul. Effective Work Index(J/m^2) = 4.440E+07

Post-Development (Red)

Total Exceedance Time = 1131 hrs

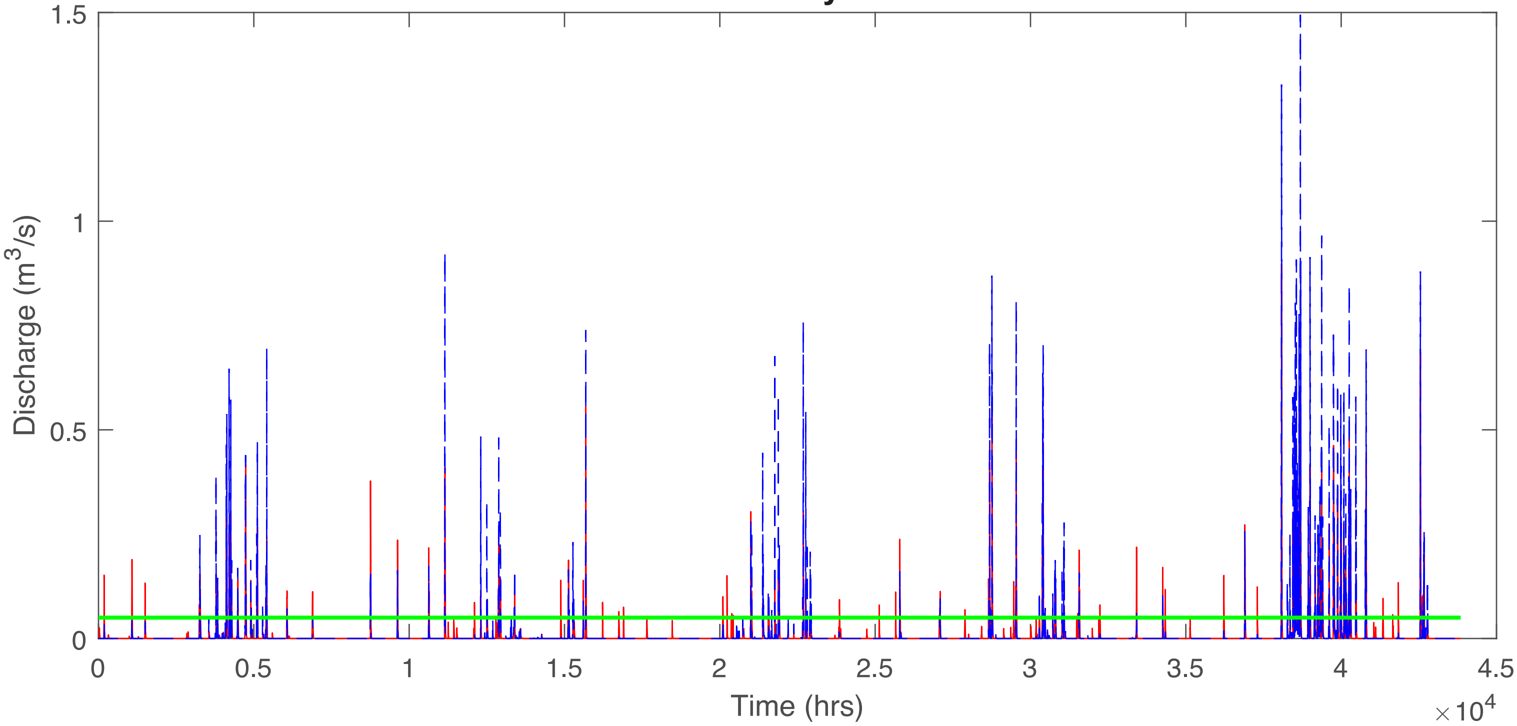
% Exceedance Time = 2.6

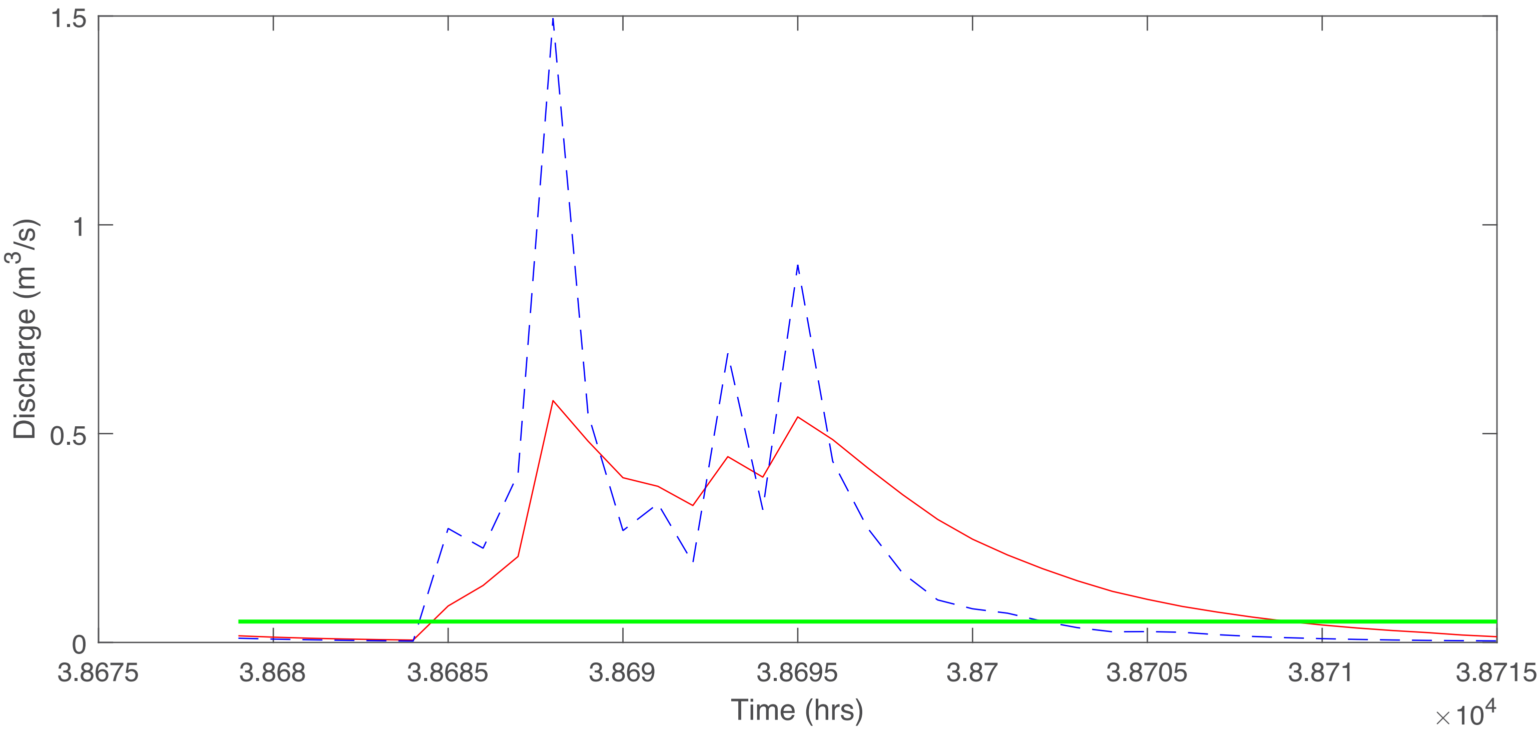
Number of Exceedance Events = 286

Cumul. Effective Discharge(m^3) = 7.109E+05

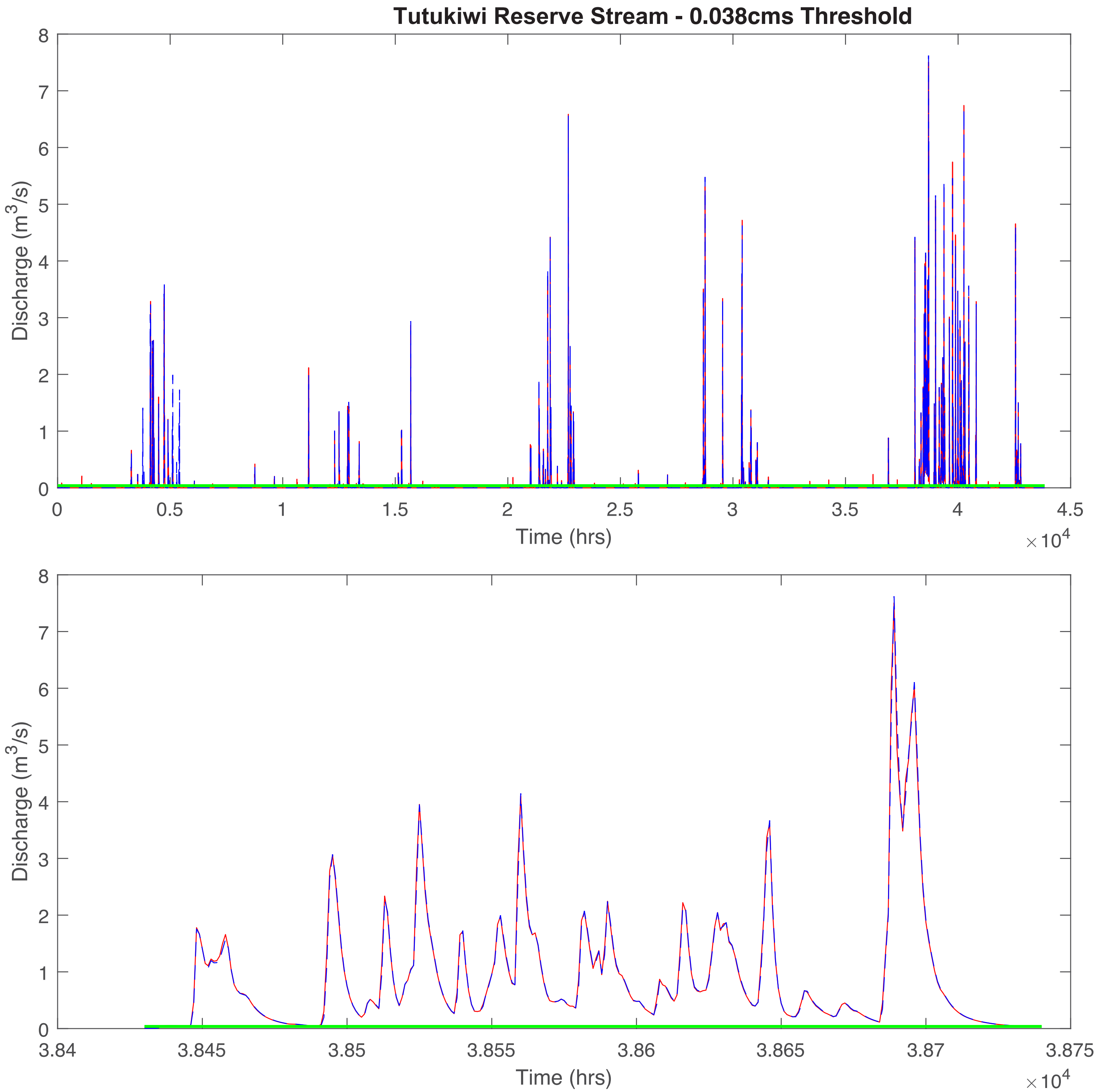
Cumul. Effective Work Index(J/m^2) = 6.694E+07

Moonshine Valley Reserve Stream - 0.05cms Threshold





Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 997 hrs	Total Exceedance Time = 1605 hrs
% Exceedance Time = 2.3	% Exceedance Time = 3.7
Number of Exceedance Events = 177	Number of Exceedance Events = 147
Cumul. Effective Discharge(m^3) = 4.683E+05	Cumul. Effective Discharge(m^3) = 4.897E+05
Cumul. Effective Work Index(J/m^2) = 4.440E+07	Cumul. Effective Work Index(J/m^2) = 4.593E+07



Pre-Development (Blue)

Total Exceedance Time = 3466 hrs

% Exceedance Time = 7.9

Number of Exceedance Events = 100

Cumul. Effective Discharge(m^3) = 6.067E+06

Cumul. Effective Work Index(J/m^2) = 1.494E+08

Post-Development (Red)

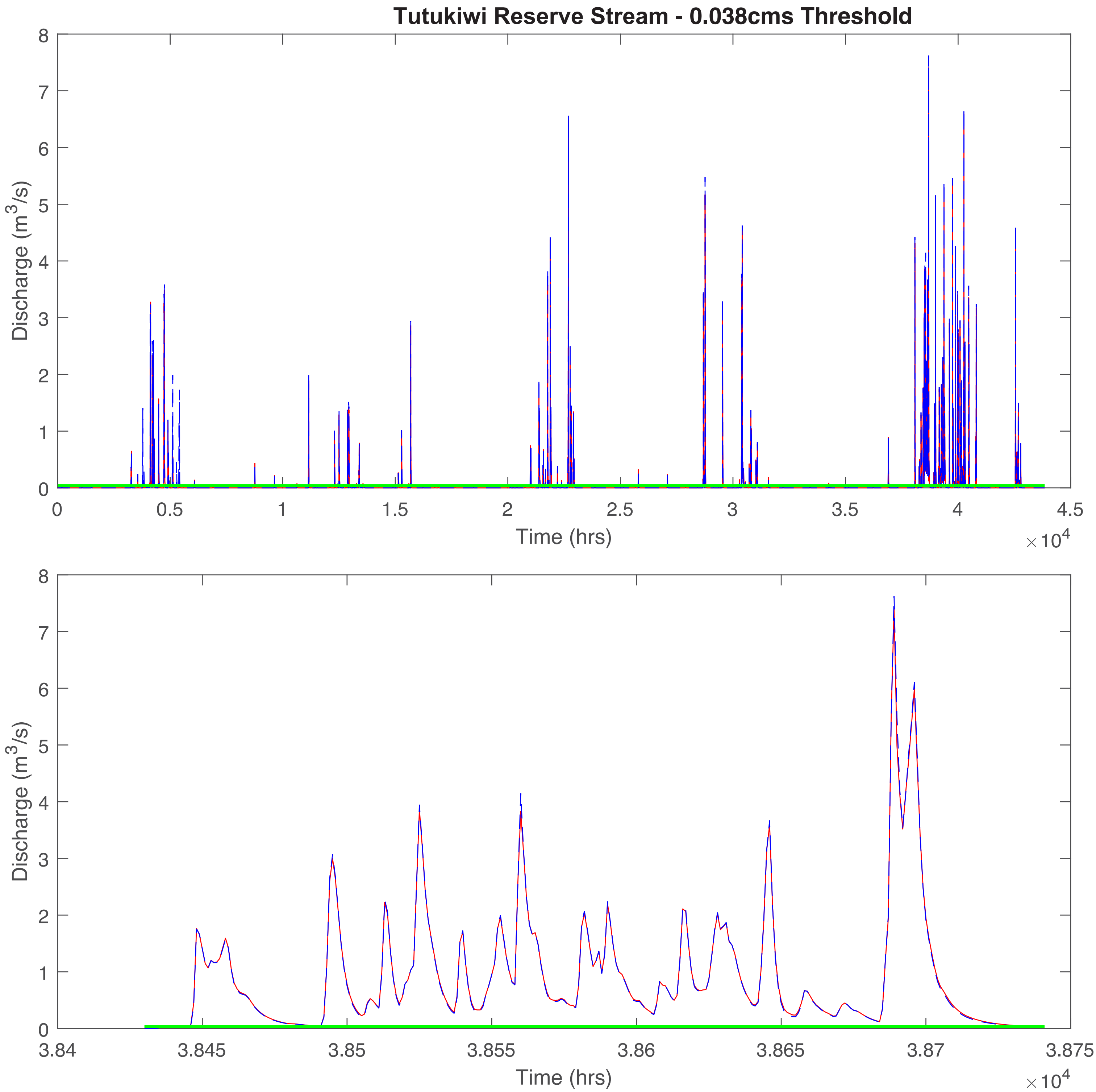
Total Exceedance Time = 3432 hrs

% Exceedance Time = 7.8

Number of Exceedance Events = 149

Cumul. Effective Discharge(m^3) = 5.850E+06

Cumul. Effective Work Index(J/m^2) = 1.439E+08



Pre-Development (Blue)

Total Exceedance Time = 3466 hrs

% Exceedance Time = 7.9

Number of Exceedance Events = 100

Cumul. Effective Discharge(m^3) = 6.067E+06

Cumul. Effective Work Index(J/m^2) = 1.494E+08

Post-Development - with SWM Control (Red)

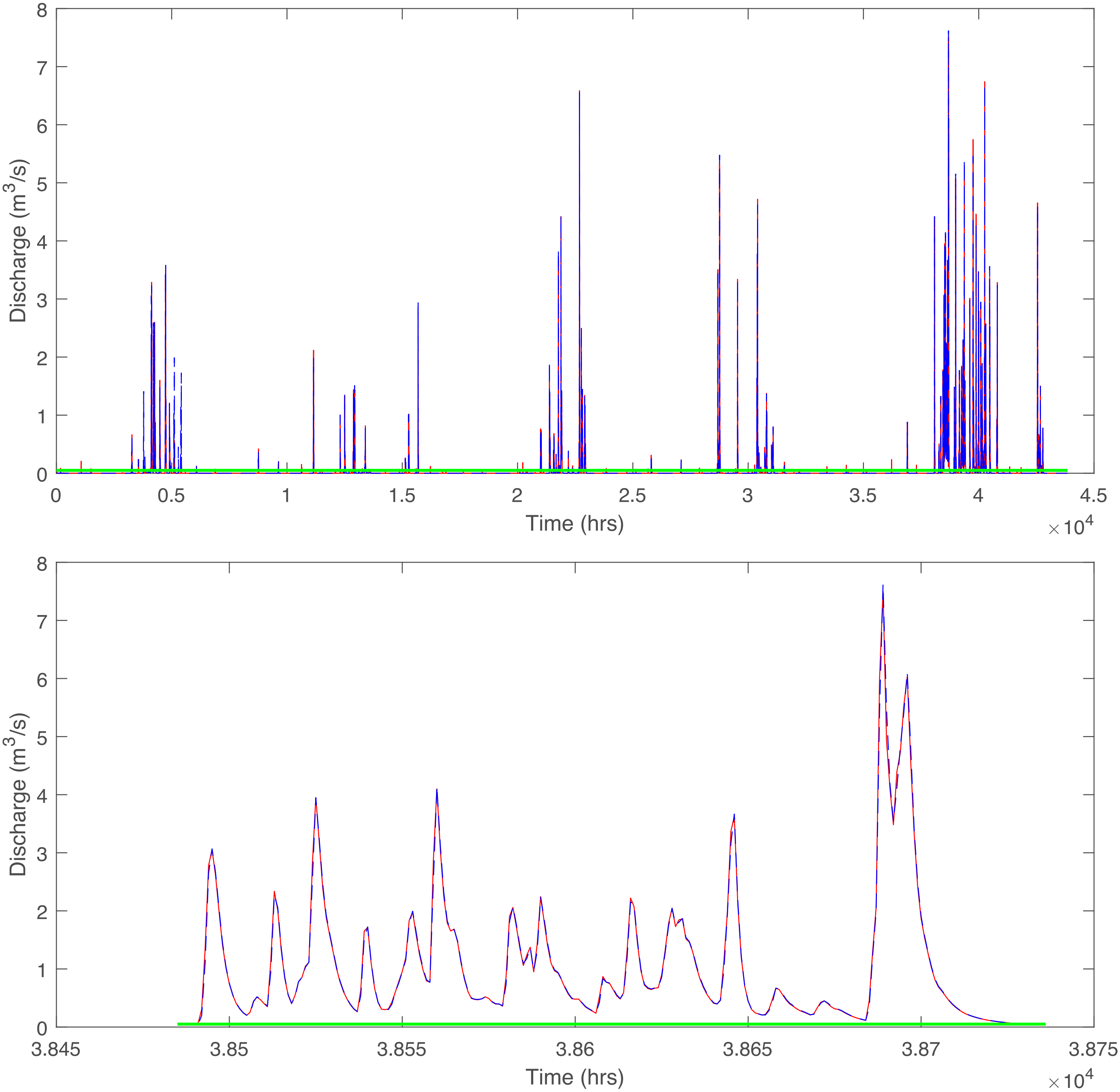
Total Exceedance Time = 3373 hrs

% Exceedance Time = 7.7

Number of Exceedance Events = 98

Cumul. Effective Discharge(m^3) = 5.819E+06 Cumul. Effective Work Index(J/m^2) = 1.434E+08

Tutukiwi Reserve Stream - 0.05cms Threshold



Pre-Development (Blue)

Total Exceedance Time = 3171 hrs

% Exceedance Time = 7.2

Number of Exceedance Events = 102

Cumul. Effective Discharge(m^3) = 5.924E+06

Cumul. Effective Work Index(J/m^2) = 1.436E+08

Post-Development (Red)

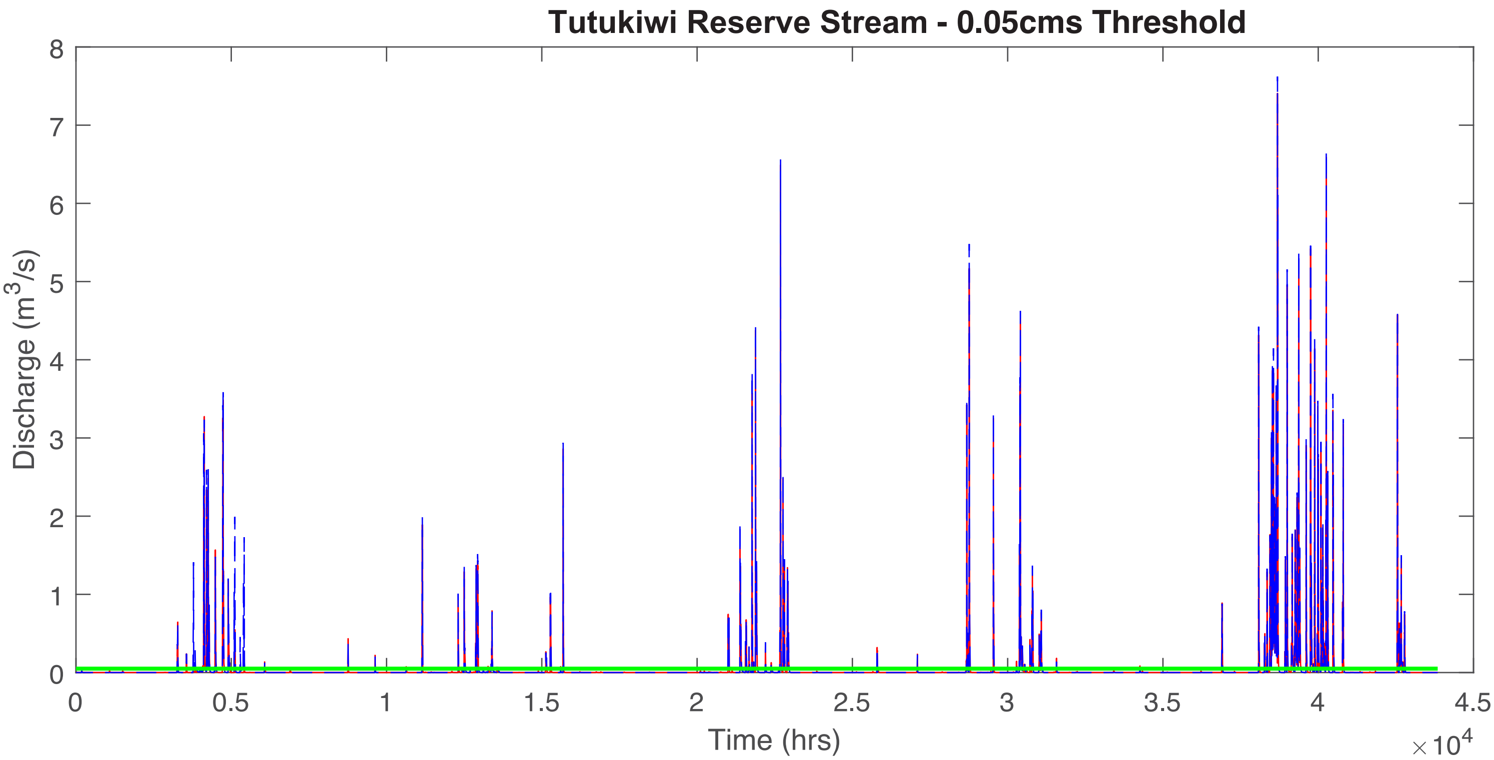
Total Exceedance Time = 3110 hrs

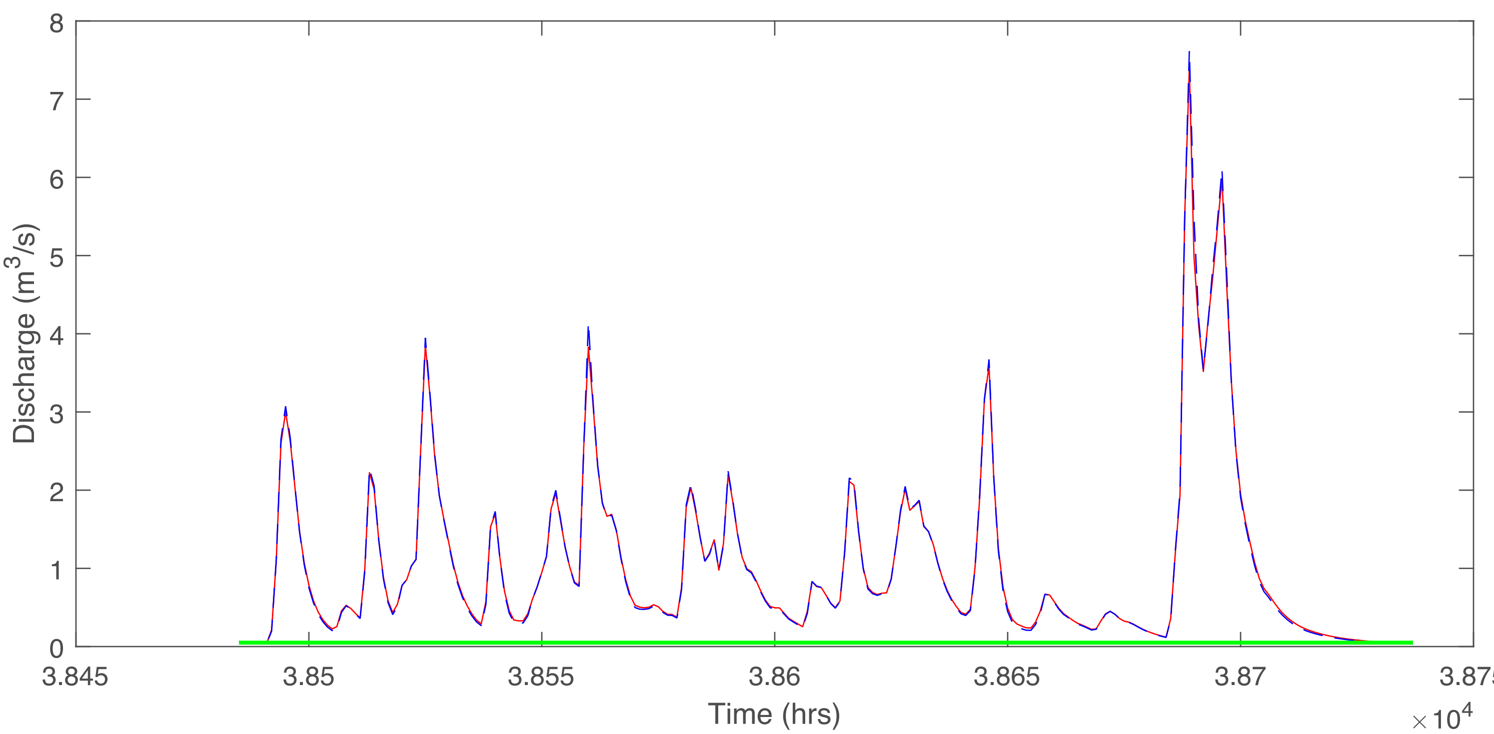
% Exceedance Time = 7.1

Number of Exceedance Events = 144

Cumul. Effective Discharge(m^3) = 5.709E+06

Cumul. Effective Work Index(J/m^2) = 1.383E+08





Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 3171 hrs	Total Exceedance Time = 3065 hrs
% Exceedance Time = 7.2	% Exceedance Time = 7.0
Number of Exceedance Events = 102	Number of Exceedance Events = 103
Cumul. Effective Discharge(m^3) = 5.924E+06	Cumul. Effective Discharge(m^3) = 5.681E+06
Cumul. Effective Work Index(J/m^2) = 1.436E+08	Cumul. Effective Work Index(J/m^2) = 1.378E+08

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The Power of Commitment

Technical Memorandum

22 June 2023

To	Michael Duindam	**Contact No.**	+64 27 4721393
Copy to	Plan Change G team	**Email**	tony.miller@ghd.com
From	Tony Miller	**Project No.**	12588291
Project Name	Proposed Plan Change G: Aokautere Urban Growth		
Subject	Stormwater Expert Evidence - Stream Erosion Assessment Summary (Rev 4)		

1. Introduction

Proposed Plan Change (PC) G was notified on 8 August 2022, following which a series of submissions were received relating to the potential effects of erosion in the gullies downstream of the development. As a result of the submissions, further analysis was carried out to better understand the receiving environment both in its' present day and post development.

1.1 Purpose of this Memorandum

This Technical Memorandum summarises the risks to the receiving gullies and potential mitigation that may be required to address those risks in order to enable development.

It should be noted that the mitigation measures discussed below are preliminary only to inform discussions with other PC G experts. The purpose of this memorandum is to emphasize the erosion risks and extent of mitigation that is required.

2. Gully Erosion Summary

Erosion and downcutting risks have been estimated assuming a 100-year design life and are based on site observations and our understanding of the extent of future development. The gullies have been labelled to align with the *Aokautere Structure Plan – Ecological Features, Constraints and Restoration* report prepared by Forbes Ecology (June 2020). The chainages referred to below are taken from the figures appended to this memorandum.

Table 1 provides a detailed summary of the risks and mitigation measures proposed; however, this can be further simplified as follows:

– Gullies 12 and 13, which are outside of the PC area, are at risk of significant erosion due to their existing landuse, which may propagate upstream into the PC area. In order to minimise the effects of development and not exacerbate or accelerate the erosion in the adjacent land, the following is recommended:
  - The **Gully 4** and **Gully 8** pond outlets be extended to the PC boundary.
  - The ponds in **Gully 6** and **Gully 11** be designed such that it allows for future downcutting at the outlet.
  - Monitoring and future stream bed restoration be provided in **Gully 12**.

- The proximity of **Gully 13** may impact parts of the structure plan as downcutting continues. The extent of the impact is to be confirmed by a geotechnical engineer.
- Additional ponding areas / below-ground dams be constructed (with appropriate fish passage) along **Gully 1** and **Gully 3**.
- The proposed roads crossing the gullies be designed to hold back / control water, simulating a dam affect, in **Gully 1** and **Gully 3**.
- Remediation of the existing culvert / stream crossing is required in **Gully 3**.
- Monitoring is recommended for the existing culvert and dam outlet in **Gully 11**, as well as the bridge crossing the Moonshine Valley Creek in **Gully 12**.
- Significant downcutting is predicted in **Gully 3** (identified as an intermittent stream), with the following potential mitigation measures:
  - Pipe the medium flow (20 l/s up to 450 l/s) with base flow remaining in the stream. Approximate pipe diameter to be 450 mm PE to bottom of gully.
  - Pipe the medium to high flows (20 l/s to 1000 l/s) with base flow of up to 20 L/s remaining in the stream. Approximate pipe diameter to be 600 mm PE to bottom of gully.
  - Install stream bed armouring along the entire length.
  - Install check dams every 25 m.

  If this cannot be achieved, then 2 to 6m+ of downcutting can be expected in Gully 3, regardless of development as it is currently showing active downcutting.
- Due to the significant downcutting predicted in Gully 3, active monitoring is recommended in **Gully 3b**. Should downcutting be observed, options to address that downcutting include:
  - Pipe the medium or high flows, with base flows remaining in the stream.
  - Stream bed armouring.
- The ponds in **Gully 3** must be designed in unison and installed prior to construction of the downstream road crossing the gully.

Technical Memorandum

Table 1 Stream erosion risks and mitigation measures for PC G

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
Gully 1 (Church Stream)			
0	200	– Existing stream is relatively stable and showing signs of accretion.	N/R
200	350	– Existing downcutting of the stream - 0.5 to 1 m. – Further 0.5 to 1 m downcutting estimated to year 2120.	Outside of PC area
350	480	– Existing stream currently incised by 0.5 to 1.2. m up to existing 3 m high waterfall at ~CH 370. – Potential for up to 4 m downcutting below valley floor above existing waterfall.	Outside of PC area No mitigation considered in private property. Alternatively, the dam proposed at CH 490 could be moved to CH 380 (see below) to reduce downcutting in this reach.
480	900	– Potential for up to 4 m downcutting below valley floor to 2120.	Below-ground dam with embedded fish passage at CH 490 and CH 750 within PC area. Upstream of CH 490 dam future downcutting up to 0.8 m predicted with proposed mitigation. From future dam at CH 750, downcutting up to CH 900 will again be limited up to 0.8 m.
900	1020	– Existing stream is actively downcutting of 0.5 to 0.8 m above existing dam at CH 900.	Culvert under future road at CH 900 to be designed to hold back water with discharge designed to resist future downcutting downstream. No further downcutting predicted in this reach with the proposed mitigation of a pond and roadway.
1020	1400	– Potential for up to 0.5 m downcutting immediately upstream of the dam at CH 900, increasing up to 4 m downcutting below valley floor at CH 1400. – The gully gradient over this reach is slightly less than Gully 3 and hence the reduced potential for excessive downcutting.	On going monitoring of this reach. This will determine the timing of installation. Active intervention may include similar options recommended for Gully 3 (see below), or below-ground dams at CH 1100 and CH 1300. Future downcutting with the proposed mitigation predicted to be up to 0.8 m over this reach.

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
Gully 2			
0	250	– Gully/ slope downstream of PC area drops rapidly from the PC area down to the MSV creek. The vegetative cover is sparsely vegetated bush. There is a smaller catchment area and only one contributing pond. – Downstream of the PC boundary, downcutting of the riverlets over time expected to be 0.8 m (or more).	Outside of PC area
250	546	– Gully within PC area shown as vegetated bush with small catchment area and no contributing ponds. – Limited erosion anticipated within the gully itself, estimated 0 to 0.8 m.	Vegetation as per proposed PC
Gully 3			
0	25	– Significant downcutting observed with 1.5 -2 m drop at existing culvert under Moonshine Valley Road (MSV). – Potential future undercutting of existing culvert outlet leading to partial/failure of MSV road embankment.	Remediation of culvert outlet with fish friendly let down structure – this is required today/**now** due to existing downcutting. Further remediation and monitoring of the MSV stream downstream of the culvert.
25	325	– Upstream of the culvert at CH 25 existing downcutting of 0.8 to 1.5 m. – Further downcutting expected to year 2120. Assuming the downstream existing culvert remains, further downcutting of stream up to the PC boundary could be limited to another 1 m below the existing incisement.	N/R 1 to 2.5 m future downcutting expected over this reach.
325	400	– Current stream downcutting of ~1 m. – Active downcutting < 1.5 m at CH 325. – Downcutting predicted to increase up to 2 m to CH 400 post-development without mitigation.	Below-ground dam with embedded fish passage at CH 325 at the PC boundary with potential to move this downstream into the reserve land with the required permissions. Future downcutting upstream of the proposed mitigation predicted to be up to 0.8 m over this reach.

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
400	1150	– Active erosion with existing 2 m scour hole observed at existing gully crossing, downstream of existing culvert at CH 1100. – Downstream of the culvert and existing scour hole the gully invert shows some distress with < 1 m incisement. – Without mitigation to prevent future downcutting to year 2120, the downcutting potential is estimated between 2 and 6 m+ regardless of development.	Significant intervention required to moderate downcutting. This may include: 1. Piping medium flows (e.g., 20 l/s up to 450 l/s) from CH 300 to CH 1150 with DN450 PE pipe. Base flow of up to 20 L/s will remain in stream, noting this is classified as an intermittent stream. Higher flows will continue down valley floor as duration would be short and infrequent. Downcutting with mitigation predicted to be 1 to 2 m with ongoing monitoring and moderate intervention required to limit further downcutting and erosion following significant storm events (flows over 450 l/s). 2. Piping medium to high flows (20 l/s up to 1000 l/s) from CH 300 to CH 1150 with DN600 PE pipe. Base flow will remain in stream, noting this is classified as an intermittent stream. Spillway overflows (estimated to be 1 to 2 m^3/s) are likely to occur in overdesign events for short periods. Downcutting with mitigation predicted to be up to 2 m, but with monitoring and future intervention only after over-design events. 3. Continuous rock channel / stream bed armouring from CH 300 to CH 1150. Downcutting with mitigation predicted to be managed to less than 2 m with active management and riprap top up every year and following major storm events. If no active management, the scour holes could reach up to 5 m deep. 4. Check dams located every 25 m between CH 300 to CH 1150. Downcutting with mitigation predicted to be 0.8 m, however maintenance of the check dams will need to be undertaken following each major rain event. If no active management, scour holes up to 2 m deep or more could form and progress upstream.
1150	1250	– Potential downcutting due to outlet flows from existing and future ponds.	Culvert under future road at CH 1150 to be designed to hold back water in new dry attenuation pond, with pond outfall at CH 1250 engineered to resist erosion. No downcutting is predicted as the pipes, culverts and dams are a fully engineered solution.
1250	1650	– Potential downcutting due to outlet flows from existing and future ponds. – Without mitigation potential for 1-2 m deep scour holes with waterfalls that will progress upstream.	All ponds upstream of the new road crossing to be designed as a system to collect and discharge attenuated volume that limits the potential for stream bed degradation. Downcutting with mitigation predicted to be limited to less than 0.8 m.

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
Gully 3a			
0	326	– Small catchment area with no contributing ponds for development. Limited to no downcutting predicted.	N/R
Gully 3b			
0	300	– Existing development upstream. – Significant size of upstream catchment has potential for downcutting downstream of the lower dam, potentially up to 1.8 m deep.	Initial mitigation required is ongoing monitoring. Should downcutting be observed, options include: 1. Piping of gully flow (medium or high) with base flows remaining. 2. Stream bed armouring. Downcutting with either mitigation predicted to be 0.8 m or less.
300	500	– Pond at CH 300 being designed. Ponds at CH 400 and CH 500 already constructed.	Review of design to limit potential for downstream erosion
500	969	– Existing development and a small catchment of future development. The piped flow is all expected to be contained within the piped catchment. Downstream ponds will attenuate flows plus one future attenuation area within the future development area.	N/R
Gully 4			
0	200	– Gully/ slope downstream of PC area drops rapidly from the PC area down to the existing receiving pond as part of the MSV creek system. The vegetative cover is sparsely vegetated bush. There is a smaller catchment area and only one contributing pond. – Downstream of the PC boundary, downcutting of the riverlets over time expected to be 0.8 m (or more).	Outside of PC area. Shift the proposed outlet of the Gully 4 pond to the PC boundary with appropriate erosion protection to limit erosion to 0.8 m. Alternatively, pipe the medium flow from the pond outfall at CH 300 to the base of the slope (CH 100) to eliminate erosion over the steeper portion in private land. Monitoring of slope erosion on private land would initiate intervention commencing. This is only possible with private landowner consent.
200	341	– Gully within PC area shown as vegetated bush with small catchment area and only one contributing pond. – Limited erosion anticipated within the gully itself, estimated up to 1.2 m at chainage 200.	Reduce downcutting almost completely within PC area with a medium flow pond outlet bypass to discharge the PC boundary. This still has potential impact downstream (refer above). Alternatively pipe pond outfall to CH 100 within private property (refer above).
Gully 5			
0	323	– Gully shown as vegetated bush with small catchment area and no proposed ponds. Limited to no downcutting predicted.	N/R

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
Gully 6			
0	280	– Relatively small catchment with only one contributing pond, however this section is pastural farming and prone to erosion. – Potential 0.5 to 1.2 m downcutting predicted.	Outside of PC area. Gully erosion can be slowed with a change of landuse, however this is beyond this scope.
280	380	– Downstream downcutting predicted to propagate to the PC boundary and to the proposed pond. – Potential downcutting of 1.2 at CH 280 to 0.5 m at CH 380 predicted.	The low flow outlet and downstream dam buttress design at CH 380 to allow for future downcutting of the gully downstream of the dam outlet. 1.2 m at CH 280 to 0.5 m at CH 380 still expected with proposed mitigation. Alternatively, increase pond volume by 30-50%, extend time and reduce outlet flows from the lower half of the pond empty cycle to reduce future erosion to no more than 0.5 m.
380	522	– Stable gully floor predicted.	N/R
Gully 7			
0	110	– Significant downcutting predicted in receiving system (Gully 13 at CH 950), estimated between 1.5 to 2.5 m. This is unlikely to propagate to the PC boundary.	N/R
110	185	– Gully shown as vegetated bush with small catchment area and no proposed ponds. Limited to no downcutting predicted.	N/R
Gully 8			
0	302	– Significant downcutting predicted in receiving system (Gully 13 at CH 1100), estimated between 1.5 to 2.5 m. – Downcutting predicted to be 0.5 to 1.5 m at PC boundary (CH 110) if landuse remains as pastoral farming. Downcutting at CH 250 predicted to be less than 0.5 m.	Shift the proposed outlet of the Gully 8 pond to the Plan Change boundary with appropriate erosion protection to ensure the outfall erosion from the pond is limited. Downcutting at PC boundary (CH 110) limited to 0.5-1.5 m. Downcutting at CH 250 limited to <0.5 m.
Gully 9			
0	130	– Significant downcutting predicted in receiving system (Gully 13 at CH 1150), estimated between 1.5 to 2.0 m. This is likely to propagate to the PC boundary by 2120. – Downcutting from Gully 13 will propagate upstream of the confluence with Gully 9. Over this reach (CH 0 – 130) and by year 2120, downcutting predicted to be between 0.5 to 1.5 m.	Outside of PC area. Gully erosion can be slowed with a change of landuse, however this is beyond this scope.

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
130	990	– Steep gully sides show signs of slippage and regression. A slip at CH 650 appears to have lost approx. 2.5 m of flat area of the table land over a distance of 10 to 15 m. This is likely representative of future slippage. – Downcutting due to Gully 13 downcutting propagating and discharge from upstream pond expected to range from 1.5 m at CH 250 to less than 0.5 m at the head of the gully by year 2120.	Options to reduce the extent of downcutting within Gully 9 by addressing the pond discharge include: 1. Redirect the pond outfall to Gully 10. Proposed mitigation will reduce downcutting to 1.2 m at CH 130 to ~0 at the head of the gully. 2. Pipe the pond discharge (up to the medium flow) to the PC boundary. Proposed mitigation will reduce downcutting to 1.5 m at CH 130 to 0.5 at CH 150 and ~0 m at the head of the gully. Monitoring and potential restoration required following significant rain events. 3. Extend the piped discharge to Gully 13 (outside PC area). Proposed mitigation will reduce downcutting to 0.8 m at CH 130 to ~0 at the head of the gully. Neither option mitigates the risk of downcutting in Gully 13.
Gully 10			
0	90	– Active downcutting observed in receiving system (Gully 13 at CH 1850), estimated around 0.5 m. Further downcutting predicted to be 1.5 m. – Downcutting of 0.5 to 1 m is likely to propagate to the PC boundary if the landuse remains farmed hill country. If landuse changes, some reduced degree of downcutting is still predicted.	Outside of PC area
90	337	– Relatively small catchment with no contributing ponds. Erosion is expected to be modest. – Towards 2120 the Gully 13 erosion will make its way upstream regardless of any PC development.	N/R
Gully 11			
0	350	– Existing downcutting estimated between 1.5 to 2 m, and is showing active signs of erosion. – By 2120 total downcutting at CH 350 estimated to be between 3 to 4 m.	Proposed culvert at CH 350 to be cognisant of the potential for additional downcutting at the outfall of up to 1.5 m. I.e., pre-empt the downcutting by dropping the culvert below the existing gully floor, thereby allowing the outfall to discharge to a future bed level below the existing. Proposed pond outlet should be shifted upstream of the culvert at CH 350 and incorporated into the culvert design. The notional empty level would be set at the existing stream bed level upstream of the culvert.

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
350	650	– Existing downcutting estimated between 2 to 1.5 m and is showing signs of active erosion and bank instability. – By 2120 total downcutting at CH 650 estimated to be ~5 m.	Proposed culvert at CH 650 to be cognisant of the potential for additional downcutting at the outfall of ~1.5 m. I.e., pre-empt the downcutting by dropping the culvert below the existing gully floor, thereby allowing the outfall to discharge to a future bed level below the existing. Proposed pond at CH 800 could be moved into the main channel and shifted downstream (~CH 700) to be incorporated with the culvert design at CH 650. The proposed pond notional empty level would be set at the existing stream bed level upstream of the culvert.
650	900	– Existing downcutting has incised the channel by 1.5 to 2 m over this reach. – By 2120 downcutting estimated between 3 to 5 m at CH 650, reducing up to 3 m erosion at CH 900. Downcutting expected to further worsen after year 2120.	Upstream of pond at CH 800 [potentially CH 700 (refer above)], only ongoing monitoring of reach. Potential future mitigation may include check dams or below-ground dams. Downcutting still expected at a reduced rate of 2.5 m total (additional 1 m). Intervention required if that is exceeded.
900	1400	– The current stream is showing modest signs of active downcutting and incisement. – Further downcutting of 0.5 to 0.7 m predicted and could undercut the culvert at CH 1400. – The proposed pond at CH 900 will prevent downstream incisement from progressing upstream of the pond.	Monitor downcutting at culvert at CH 1400 with the possibility of remediation being required.
1400	1800	– The culvert at CH 1400 and existing dam at CH 1600 will limit extent of downcutting along this reach. – Potential for downcutting at the outlet of the dam at CH 1600.	Active monitoring of dam outlet at CH 1600 for erosion with the possibility of remediation.
Gully 11a			
0	499	– Relatively small catchment area. The gully floor gradient steepens significantly and may lead to erosion in the future erosion. – The max predicted downcutting of the stream inverts over the design period is less than 1.5 m.	N/R
Gully 11b			
0	361	– Relatively small catchment area. The gully floor gradient steepens significantly and may lead to erosion in the future erosion. – The max predicted downcutting of the stream inverts over the design period is less than 1.5 m.	N/R

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
Gully 12 (Moonshine Valley Creek)			
0	1750	– No noticeable downcutting over this reach. However, on outside of bends there is some observed toe erosion of the outer bends. – No expected change post-development.	N/R
1750	2580	– The existing stream over this reach is surrounded by dwellings and has incised the flood plain between 1.5 to 2.5 m. There is some downcutting and general lack of natural bed armour. – Widening of the stream bed will cause ongoing stream bank failure and increased erosion rates on the outer bends is predicted regardless of development. – Downcutting is predicted to be modest, however continued erosion on the outer bends may lead to sporadic bank collapse and failures. This is regardless of the development, however it will be accelerated by development. – Due to lack of supply of natural be armouring, there is a noticeable degradation of the streambed level. At CH 2850 (i.e., confluence with Gully 3) the streambed appears to have degraded by 0.5 m. – To year 2120 further degradation expected between 0 and a further 1 m.	Outside of PC area. Monitoring and future stream restoration, including bank stabilisation as required. Discussion between PNCC Planners and Geotech over the consequences of this observation is recommended by GHD, as there is no reasonable mitigation and natural erosion will continue to occur regardless of development.
2620	3150	– The stream appears to have down cut 0.5 m below the bed as anticipated when the bridge was constructed. – Further bridge maintenance to the abutments may be required over time. – A further 0.5 m of downcutting is predicted. As above the outer bends will continue to erode leading to bank failures with a probability of more on the outside of bends.	Outside of PC area. Monitoring of bridge crossing Moonshine Valley Creek at CH 3150. There is a potential for bridge abutment works to protect the toe of the abutments from being undercut as the stream bed level continues to drop. Downcutting expected to continue with a further 0.5 m predicted.
3330	3450	– Existing ponds in the reserve area may require scheduled maintenance due to sediment loading from upstream catchment. This is independent of the PC, however increased loading from the PC area will only increase the frequency of the loading and subsequent cleaning required.	Ongoing maintenance of existing ponds expected. This will include regular de-silting on a ~ 5 to 10 year basis.
Gully 13			
0	500	– Gully commences at pond and progresses upstream through a bushed catchment. – This stream was not inspected, however downcutting and bank erosion is likely. This will continue regardless of development, but will be accelerated by development.	Monitoring and future stream restoration, including bank stabilisation.

Chainage 1	Chainage 2	Risks / Issues	Proposed Mitigation
500	1050	– This stream is characterised by over steep banks and gully sides with evidence of multiple slips. The land has been “recently” raised through uplift (in the geological context) and the stream valleys are in active downcutting mode. Meandering of the valley floor is evident. – Significant downcutting of 1.5 to 2.5 m predicted if the landuse remains as pastural farming. – This downcutting may impact some adjacent properties in the PC area.	Outside of PC area. The downcutting is active and will remain.
1600	1850	– As this is further up the gully, the future downcutting is predicted to reduce because the side slips that fall into the valley floor will need to be removed before further downcutting can occur. – Active downcutting observed, estimated around 0.5 to 1.5 m. – Further downcutting of 1.5 to 3 m is predicted if the landuse remains as pastural farming. – This downcutting may impact adjacent properties in the PC area.	Outside PC area

Technical Memorandum

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Investigations undertaken in respect of this memorandum are constrained by the particular site conditions, such as the location of buildings, services and vegetation. As a result, not all relevant site features and conditions may have been identified in this memorandum.

Project name	Proposed Plan Change G: Aokautere Urban Growth
Document title	Stormwater Expert Evidence | Stream Erosion Assessment Summary
Project number	12588291
File name	12588291-MEM_PC G - Stream erosion comments.docx

Status Code	Revision	Author	Reviewer		Approved for issue		
			Name	Signature	Name	Signature	Date
S1	0	R Baugham	T Miller	Tony Miller	A Chisholm	on file	1 Jun 23
S2	1	R Baugham	T Miller	Tony Miller	A Chisholm		

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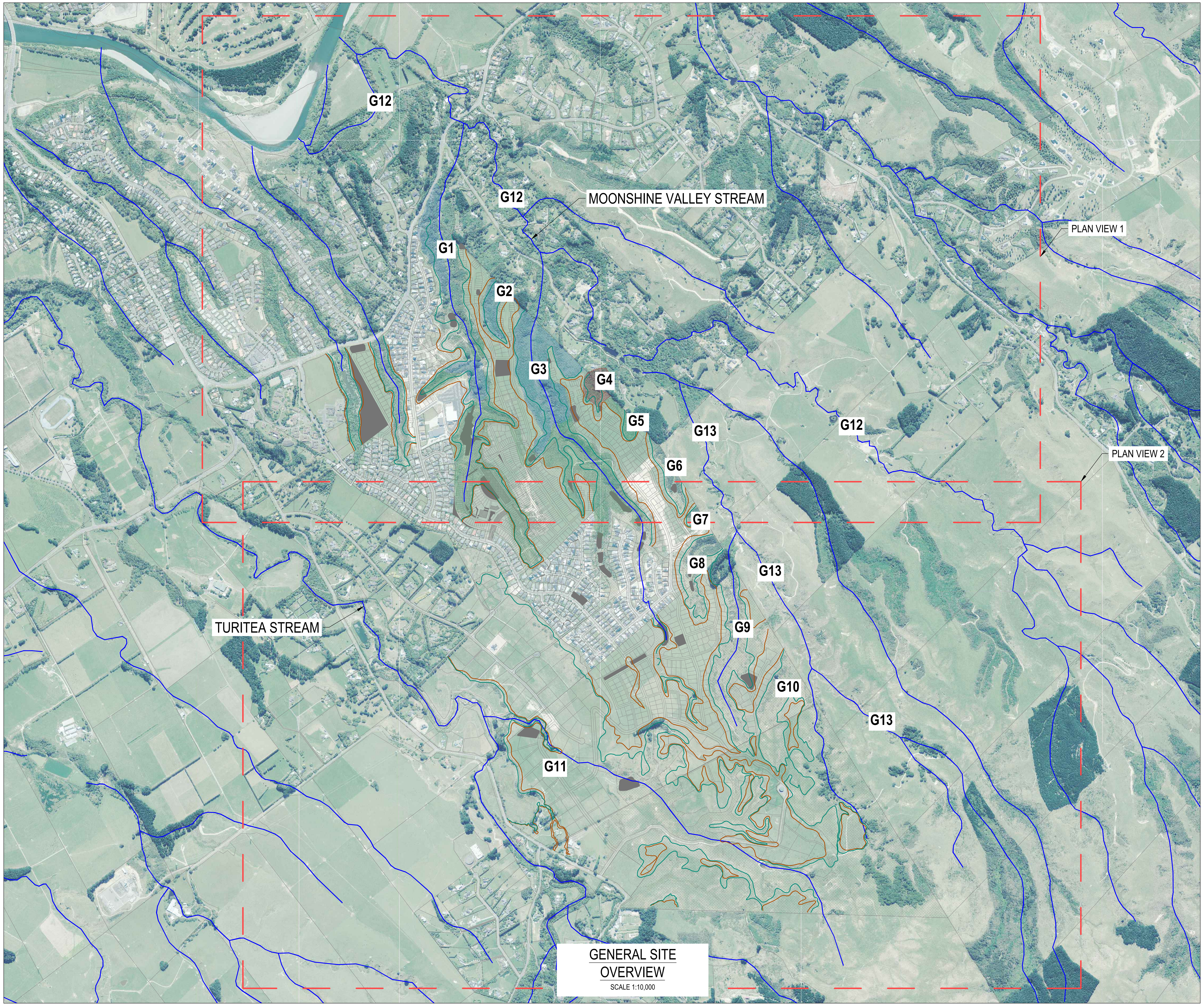

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GHD

PALMERSTON NORTH CITY COUNCIL
AOKAUTERE STORMWATER PLAN

GENERAL OVERVIEW

Project Number	12588291
Revision	A
Date	05.2023

Figure 00

Plot Date: 31 May 2023 - 3:03 pm
Plotted by: Clay O'Donnell
Cad File No: N:\NZ\Palmerston North\Projects\51\12588291\CADD\Drawings\12588291-C100-C106-RA.dwg

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PALMERSTON NORTH CITY COUNCIL
AOKAUTERE STORMWATER PLAN

PLAN VIEW 1
GULLY 1, 1A, 1B, 2, 3, 3A, 4, 5, 6, 12

Project Number	12588291
Revision	A
Date	05.2023

Figure 01

Plot Date: 31 May 2023 - 3:03 pm Plotted by: Clay O'Donnell Cad File No: N:\NZ\Palmerston North\Projects\51\12588291\CADD\Drawings\12588291-C100-C106-RA.dwg

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SCALE 1:5000 AT ORIGINAL SIZE

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AOKAUTERE STORMWATER PLAN

PLAN VIEW 2
GULLY 7, 8, 9, 10, 11, 11A, 11B

Project Number	12588291
Revision	A
Date	05.2023

Figure 02

Plot Date: 31 May 2023 - 3:03 pm Plotted by: Clay O'Donnell Cad File No: N:\NZ\Palmerston North\Projects\51\12588291\CADD\Drawings\12588291-C100-C106-RA.dwg

Technical Memorandum

28 August 2023

To	Anita Copplestone, Shannon Johnston	**Contact No.**	+64 6 355 7181
Copy to	Plan Change G team	**Email**	reiko.baugham@ghd.com
From	Reiko Baugham	**Project No.**	12588291
Project Name	Proposed Plan Change G: Aokautere Urban Growth		
Subject	Proposed Stormwater and Stream Erosion Mitigation		

This memorandum follows on from a number of discussions and further work regarding stormwater management in the Proposed Plan Change G (PCG) area.

In order to manage stormwater from existing and proposed residential development in Aokautere,[1] the following package of measures have been proposed to reduce the volume and velocity of runoff generated by development. The approach focuses on first avoiding, then reducing and minimising the generation of adverse effects, through on-site control of stormwater contaminants and flows, planning controls, restoration of natural systems, and infrastructure works. The proposed controls include:

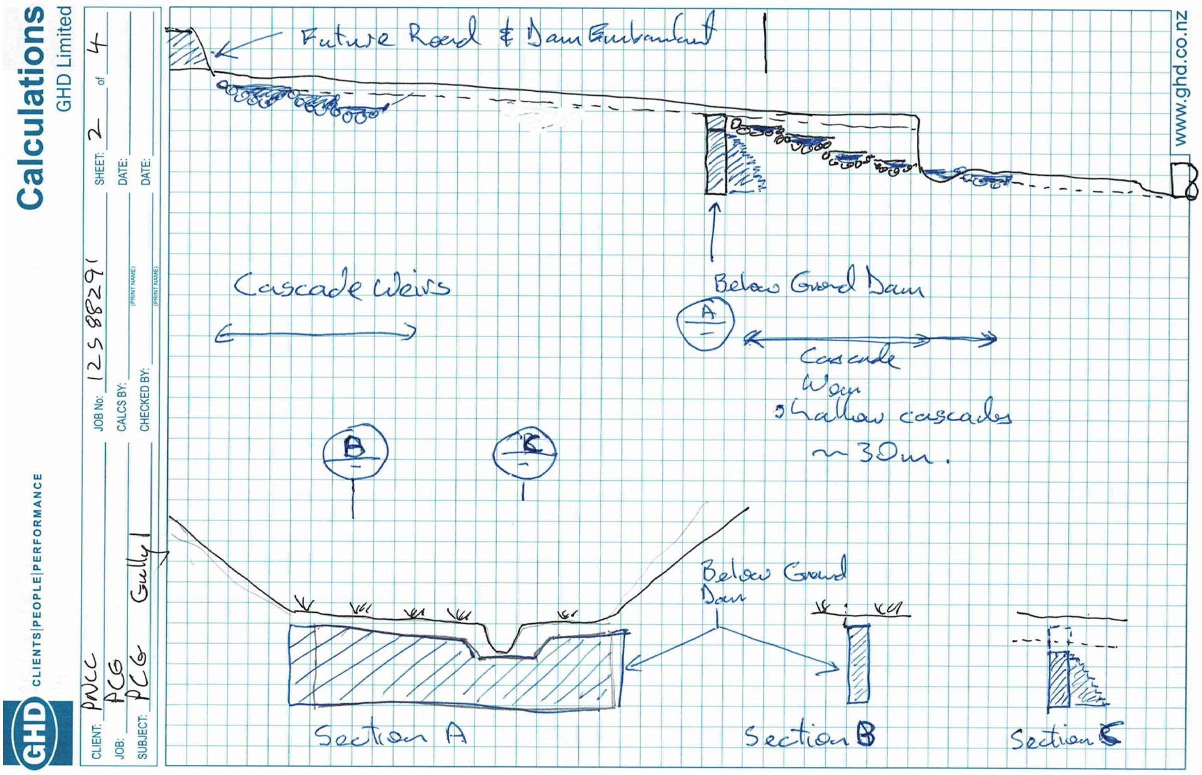
1. Revegetate the gully sides, including through forestation.
2. Avoid direct discharge of stormwater over gully edges.
3. Limit impervious areas within new development to reduce the increase in stormwater runoff. A performance standard of 40% permeable surface per lot is proposed for suburban areas, and 25% for medium density areas. (Compare this with the operative standard in the District Plan, which is 30%).
4. Implement water sensitive design elements to retain stormwater on the plateaus (i.e., incorporating stormwater storage within the raingarden designs which are required as part of the new roads).
5. Avoid engineering works in areas of moderate to very high vegetative constraint.
6. Locate attenuation ponds offline as far as practicable.
7. Enlarge the proposed stormwater ponds to increase holding capacity and slow the rate of discharge to reduce the risk of downstream erosion.
8. Implement in-stream stabilisation measures in limited reaches to reduce steep gradients.

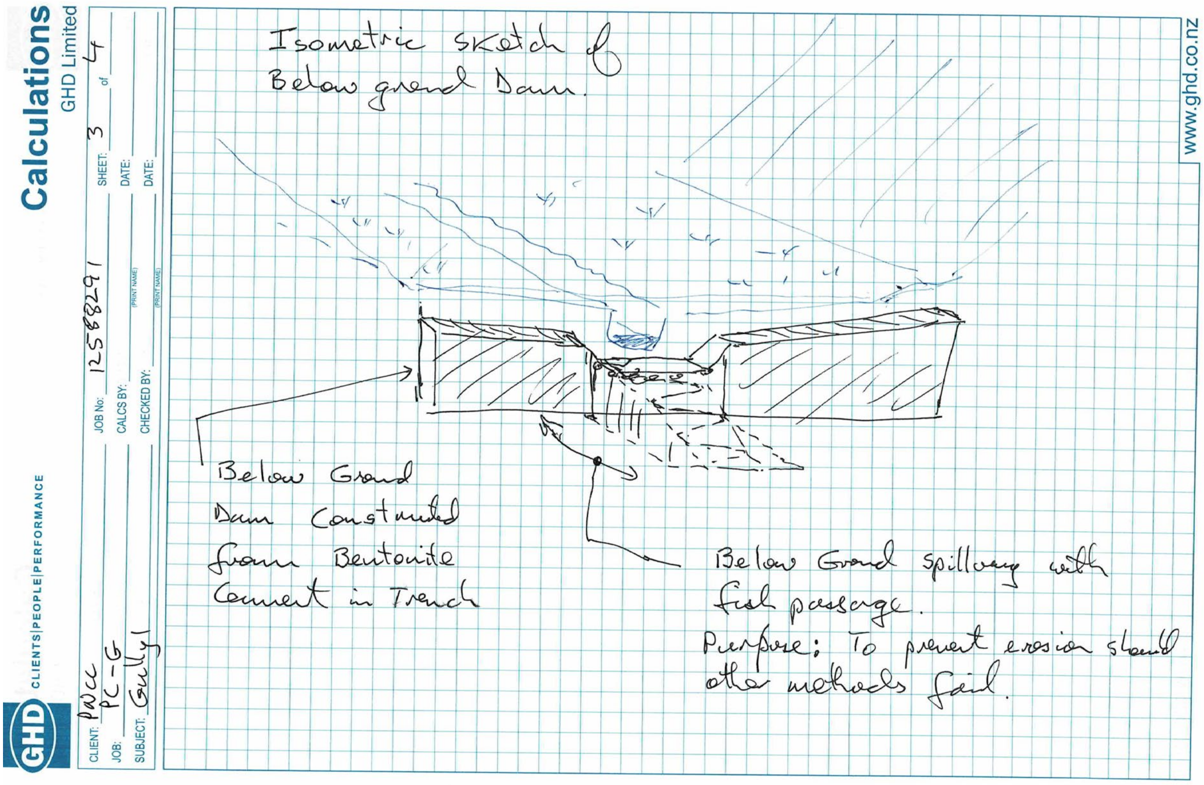
Part of the strategy involves engineering works designed to reduce velocities and sediment transport during frequent rainfall events. Table 1 summarises the proposed stormwater controls within the gullies themselves and indicative impact on the streams. The types of controls proposed include those set out below:

– Below Ground Dams – Dam structure built below ground to prevent future downcutting that will be caused by a downstream waterfall / scour point from regressing further up the stream. This will likely require:
  - A series of shallow weirs downstream (described further below); and
  - A buried spillway in event of future failure of the let-down structure.

[1] The PCG area includes existing development and is already zoned residential (although yet to be developed) in some areas. The objective of the stormwater strategy is to not only manage the effects of future development through PCG, but also remediate the adverse effects of recent development.

- The following shows a typical long section and cross section of what this BGD could look like.

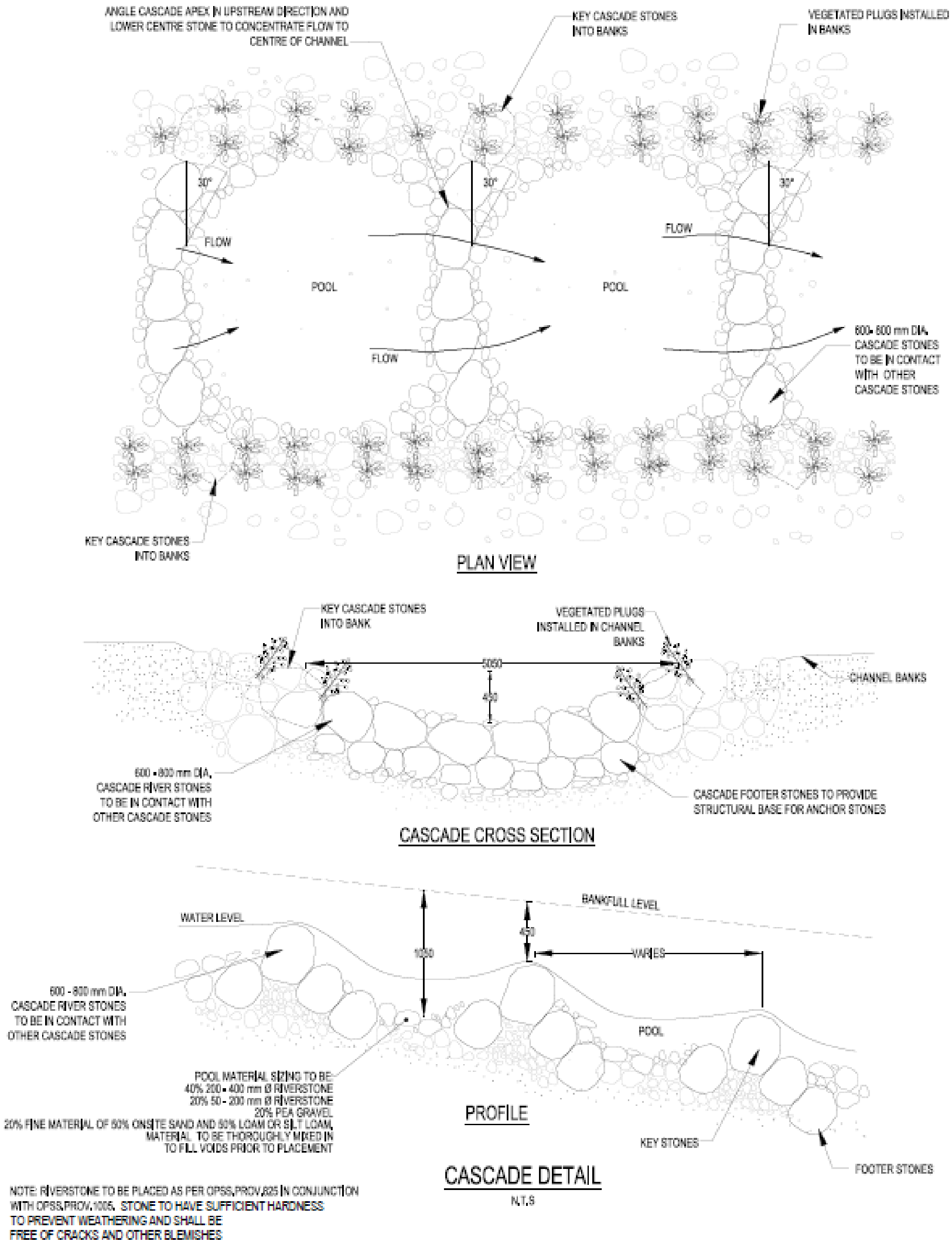




– Offline detention ponds – Dry pond located adjacent to the stream, allowing low flows to continue down the stream and high flows to be diverted into the attenuation area for gradual release back into the stream. The area designated as the dry pond / detention pond will be vegetated and still serve as the wider river corridor and will flood during rain events.

– Online detention ponds – Delineation of the floodplain area with an embankment / above-ground structure to hold back flow during large rain events. Low flows will continue along the stream and fish passage will be incorporated outlet design, pipe beneath the dam and up through a spiral fish ladder in the upstream MH. Ponding will occur behind the embankment during large rainfall events and slowly drawdown.

– Shallow cascading weirs – A series of shallow weirs / cascades along the stream bed to allow a gradual longitudinal grade change. The use of shallow weirs, or rock riffles, along the stream bed can reduce the gradient of the stream, thereby decreasing the velocity and erosive forces. When correctly engineered and constructed, these weirs can support biodiversity by providing areas of low-flow ponds and refuges during times of high flow that can support various fish species. These are required for areas that high a steeper gradient and is at risk of significant downcutting and erosion.



Technical Memorandum

Table 1 Proposed stormwater controls within the gullies

Chainage	Proposed Mitigation	Stream Impacts[2]
Gully 1 (Church Stream)		
400	– Below-ground dam with embedded fish passage. – Shallow cascading weirs to control downstream flow.	– Potential stream loss ≈ 45 m of initial disturbance, reinstated with 40 m of cascading weirs
850	– Below-ground dam with embedded fish passage. – Shallow cascading weirs to control downstream flow.	– Potential stream loss ≈ 40 m of reinstated cascading weirs
950	– Offline attenuation pond on upstream side of future road. – Piping of stream for 60 m to cross beneath the future road and bypass attenuation area, consisting of large diameter culvert to enable fish passage. – At the upstream end, a spiral fish passage would be provided within a large MH to allow fish upstream and yet control high flow from above whilst diverting high flow to the adjacent offline pond	– Total stream loss ≈ 60 m of oversized culvert beneath the dam. The culvert would be oversized for flow and have a gentle gradient to allow for fish passage. At the upstream end of the culvert there would be a required vertical climb. This can be achieved with a large MH and spiral fish passage ladder around the internal circumference of the MH
1000	– Shallow cascading weirs to reduce gradient and slow stream flows.	– Potential stream loss ≈ 105 m of reinstated cascading weirs – I.e., 3 lots of approximately 35 m long cascades
1200	– Attenuation pond at top end of side gully.	– Upstream end of gully with piped infrastructure immediately upstream. No predicted stream loss expected.
1400	– Online attenuation pond. Intermittent stream will remain, and during large events wider ponding will occur within the gully floor along the length of the embankment.	– Upstream end of gully with piped infrastructure immediately upstream. – Potential stream loss ≈ 40 m of oversized culvert with fish passage beneath the dam at ch 1400. – Oversize culvert will terminate in an oversized MH incorporating a spiral fish passage. MH will also control and affect high flow into storage.
Gully 2		
	Ponds to be located on promontory at the top of the escarpment.	

[2] Does not include initial disturbance, only final outcome

Chainage	Proposed Mitigation	Stream Impacts[2]
Gully 3		
25	– Existing culvert (beneath Moonshine Valley Road) to be upgraded to included fish passage.	– Improvement to stream ≈ 15 m
325	– Below-ground dam with embedded fish passage. – Shallow cascading weirs to control downstream flow.	– Potential stream loss ≈ 40 m of reinstated cascading weirs
350	– Shallow cascading weirs to reduce gradient and slow stream flows.	– Potential stream loss ≈ 210 m of reinstated cascading weirs – Possible 6 lengths of cascading weirs approx. each 35 m long
850	– Attenuation pond at top end of side gully.	– Upstream end of gully with piped infrastructure immediately upstream. No predicted stream loss expected.
1150	– Dam embankment at future road crossing. Culvert to be designed for fish passage. – Inline dry pond along intermittent stream. Stream will remain, with wider ponding area within the gully floor during large rainfall events.	– Potential stream loss ≈ 40 m beneath dam and associated road crossing. – Oversized pipe constructed with shallow grade, will terminate at over sized MH incorporating spiral fish passage up to old stream bed. MH will extend upwards to control high flow into the dry storage basin
1250	– Inline dry pond to remedy the effects of the existing development and pond discharge. Stream will remain, with wider ponding area within the gully floor during large rainfall events.	– Potential stream loss ≈ 40 m with an oversized culvert with fish passage will terminate at oversized MH incorporating spiral fish passage up to old stream bed.
2000	– Attenuation pond between development areas.	– None.
Gully 3a		
	Pond to be located on promontory. Let down pipe to the gully 3 floor.	
Gully 3b		
0 - 200	– Two new online ponds. Features to include oversized gentle graded culvert and over sized inlet MH incorporating spiral fish passage	– Potential stream loss ≈ 60 m of oversized shallow graded pipes beneath the pond embankment
250-500	- Existing ponds to be remedied to reduce downstream effects (i.e., enlarged with better outlet controls).	- No new predicted stream loss as ponds are existing
850 - 1000	Ponds to be located on the top plateau. At the top end of the gully system, these ponds would be considered offline.	- No predicted stream loss
Gully 4		
	Ponds to be located on promontory	- No predicted stream loss
Gully 5		
	Pond to be located on promontory	- No predicted stream loss

Chainage	Proposed Mitigation	Stream Impacts[2]
Gully 6		
400	– Offline dry pond along ephemeral stream.	– Upstream end of gully along ephemeral stream. No predicted stream loss expected.
Gully 7		
400	Pond to be located on promontory	- No predicted stream loss
Gully 8		
302	– Inline dry pond above intermittent stream reach at top end of gully. Ponding within the gully floor only expected during larger rainfall events.	– Upstream end of gully with piped infrastructure immediately upstream. No predicted stream loss expected.
Gully 9		
600	– Offline attenuation pond.	– No predicted stream loss
Gully 10		
	None	- No predicted stream loss
Gully 11		
200	Pond to be located on promontory	
350	– Road crossing proposed to be a bridge.	– Potential stream loss ≈ 20 m around bridge construction
650	– Road crossing proposed to be a bridge.	– Potential stream loss ≈ 20 m around bridge construction
800	– Offline attenuation pond(s).	– None.
1400	– Existing culvert to be upgraded to include fish passage.	– Improvement to stream ≈ 20 m
Gully 11a		
	None	- No predicted stream loss
Gully 11b		
	None	- No predicted stream loss
Gully 12 (Moonshine Valley Creek)		
	None	- No predicted stream loss
Gully 13		
	None	- No predicted stream loss

Technical Memorandum

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Technical Memorandum

Project name	Proposed Plan Change G: Aokautere Urban Growth
Document title	Stormwater / Stream Erosion Management Summary
Project number	12588291
File name	12588291-MEM_PC G - Proposed Stormwater and Stream Erosion Mitigation2023-09-12.docx

Status Code	**Revision**	**Author**	**Reviewer**		**Approved for issue**		
			Name	**Signature**	**Name**	**Signature**	**Date**
S1	0	R Baugham	T Miller				
[Status code]							

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Technical Memorandum

August 30, 2023

To	PNCC	**Contact No.**	[Enter text]
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From	Sarah Irwin & Jeff Doucette	**Project No.**	12588291
Project Name	Model Update – Technical Memo: Aokautere Plan Change G		
Subject	Results of additional modelling incorporating the proposed mitigation measures		

1. Introduction

This technical memorandum has been prepared to document the details added to the hydrologic and hydraulic models developed to inform the Aokautere – Plan Change G Stormwater Management Strategy (2022 GHD Report). It should be read in conjunction with the 2022 GHD Report, which provides further information on the project background, modelling objectives, data inputs, methods, and design concepts for the stormwater management controls. Any changes from the 2022 GHD Report are included herein. The models developed for the 2022 GHD Report are referred to as the "original models".

1.1 Purpose of this memorandum

This Technical Memorandum is provided as a final communication to summarise the results of additional modelling carried out following the evolution of the stormwater strategy following Notification on 8 August 2022. (i.e., incorporates the changes made to the mitigation measures as set out in the "Stream Erosion Mitigation Memo (September 2023).

This memorandum documents the updated modelling results and erosion threshold analysis since the submission of the 2022 GHD Report to inform the Expert Evidence being prepared by Tony Miller and Reiko Baugham.

1.2 Scope and limitations

This Technical Memorandum: has been prepared by GHD for Palmerston North City Council and may only be used and relied on by Palmerston North City Council for the purpose agreed between GHD and Palmerston North City Council as set out in section 1.1 of this memorandum.

GHD otherwise disclaims responsibility to any person other than Palmerston North City Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to *update this report to account for events or changes occurring subsequent to the date that the report was prepared.*

GHD has prepared this memorandum on the basis of information provided by the Client and others who provided information to GHD (which may also include Government authorities), which GHD has not independently verified or checked for the purpose of this memorandum. GHD does not accept liability in connection with such unverified information, including errors and omissions in the memorandum which were caused by errors or omissions in that information.

The opinions, conclusions and any recommendations in this memorandum are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

Investigations undertaken in respect of this memorandum are constrained by the particular site conditions, such as the location of buildings, services and vegetation. As a result, not all relevant site features and conditions may have been identified in this memorandum.

2. Data inputs

The model updates were performed using the data inputs applied in the original modelling. A summary of the datasets is provided below.

2.1 Topography

A 2018 digital elevation model (DEM) was obtained from Palmerston North City Council (PNCC) Open Data to represent the topography within the study area. A 2015 DEM was obtained from the Land Information New Zealand (LINZ) Data Service to characterise the topography of the upstream drainage area. The topographic data was used to delineate sub catchment boundaries, identify the natural drainage paths, and calculate slopes of the sub catchments, channels, and reticulation systems.

2.2 Land cover and soil

Land cover data was obtained from the Land Cover Database (LCDB) v40 layer of the Landcare Research Portal, added June 27, 2014. Land cover information was used to determine the imperviousness and hydraulic roughness of the sub catchment areas. Existing residential development within the Plan Change G (PC G) area has been incorporated in the post-development scenario only; the pre-development scenario assumes typical farming activity.

Surficial soil information was obtained from the FSL Particle Size Classification layer from the Land Resource Information System (LRIS) portal, Landcare Research, added on June 7, 2020. Soil information was used to determine infiltration parameters in the model.

2.3 Rainfall

Historical and climate change projected rainfall hyetographs were created using a Normalised 24-hour Design Storm distribution from the *Guidelines for Stormwater Runoff Modelling in Auckland Region* document (TP-108), and rainfall intensities from the National Institute of Water and Atmospheric Research (NIWA) High Intensity Rainfall Desing System (HIRDS). The revised modelling was performed for the 2-year, 10-year, and 100-year return period events only. An hourly rainfall timeseries was also obtained from the NIWA National Climate Database for the Palmerston North EWS gauge for the 2012 to 2016 time-period to perform a continuous model simulation for the erosion assessment.

2.4 Subdivision layout

The preliminary subdivision layout of the PC G provided by McIndoe Urban (received 16 October 2019) and subsequent revisions (final dated 11 May 2022) were used in the stormwater assessment. The structure

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plan layouts were utilised along with the existing development and topography to delineate post-development sub catchment boundaries, layout preliminary reticulation and major drainage networks, and select preliminary site discharge locations.

3. Model development

Hydrologic and hydraulic modelling of the study area was undertaken to characterise stormwater runoff conditions to inform the flood and erosion assessments, including the quantification of pre-development and uncontrolled post-development runoff flows and volumes at the site discharge locations. Major discharge locations (i.e., Outfalls) are located on the receiving watercourses (gullies) where they exit the study area (**Figure 1**), and minor discharge locations are located at the proposed detention pond outlets (**Figure 2**).

The site discharge locations were selected based on a combination of the developed infrastructure, preliminary structure plan layout, and existing topography to carry out the assessments. Actual stormwater discharge locations from development areas into the receiving environment will ultimately depend on the site-specific design to be completed as par of the consenting.

The modelling was performed using the PCSWMM software (Computational Hydraulics International, 2021). PCSWMM is a spatial decision support system for the U.S. Environmental Protection Agency SWMM 5 software. The model requires input of topographical features (catchment area, flow length, slope), ground cover conditions (hydraulic roughness, depression storage), infiltration parameters (curve number or infiltration rate), rainfall (design storm hyetographs, continuous timeseries), and drainage paths (channel lengths, roughness) in order to effectively simulate the stormwater runoff conditions of a subject site. The model output files are provided in **Attachment A**.

3.1 Pre-development model

The objectives of the pre-development model are to establish flood control targets for the predicted post-development flow rates to be controlled to within the study and provide a baseline hydrologic condition for the post-development condition to be compared to in the erosion assessment.

3.1.1 Sub catchment delineation

No changes to the pre-development sub catchment boundaries presented in the 2022 GHD Report were made as part of the model updates. The pre-development sub catchments are shown on **Figure 1**.

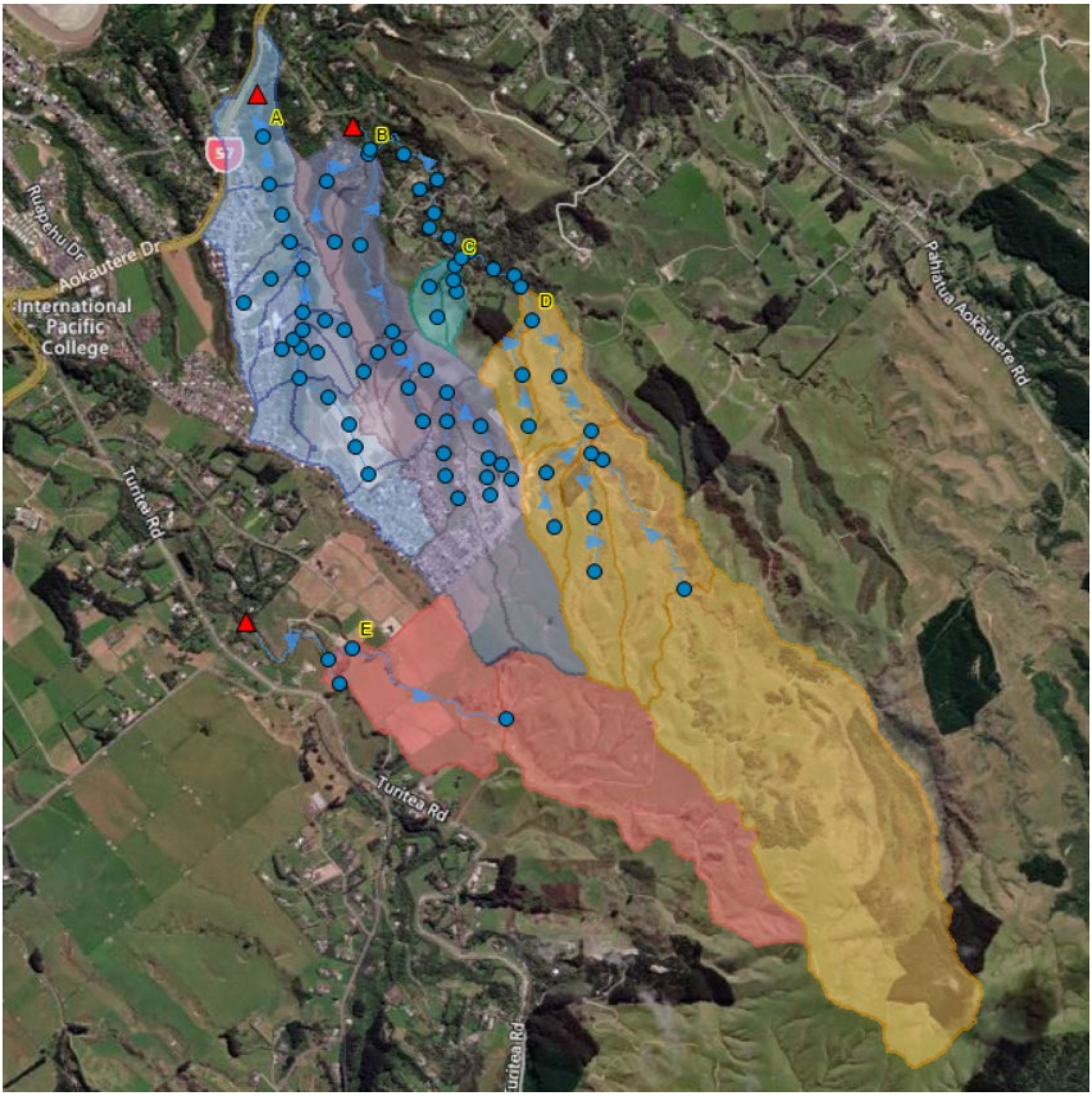

Figure 1 ***Pre-development sub catchment boundaries and major discharge locations***

3.1.2 Sub catchment parameters

The original pre-development sub catchment parameters were reviewed, and some modifications were made to increase the consistency between the approaches used to characterise the pre-and post-development models. The updated pre-development model parameters were characterised as follows:

- Flow lengths were measured as the longest flow path to the main channel plus a fraction (approximately 10%) of the main channel length to account for the portion of the total flow length that travels quickly as channelized flow.
- Sub catchment slopes were calculated along the path of the flow length.
- An imperviousness of 0% was assigned to all sub catchments to represent historical conditions, prior to any subdivision development.

- Manning's 'n' values for overland flow were assigned based on the land cover type and spatially averaged over the sub catchments. The Manning's 'n' values by land cover type are summarised in Table 3.1 of the 2022 GHD Report.
- A typical depression storage value of 5 mm was assigned for all pervious land cover types following the initial abstraction depths summarized in Table 3.1 of TP-108.

The SCS curve number method was used to calculate infiltration as per the 2022 GHD Report. The sub catchments were assigned curve numbers of 61 or 74 for the silt or loam surficial soil type, respectively, with a grassland cover.

It is important to note that the model updates were applied to the sub catchments draining to Outfalls A, B, C, D, and E only.

3.1.3 Rainfall inputs

For the flood assessment, rainfall was modelled using a design storm approach based on the requirements outlined in the PNCC Engineering Standards for Land Development (ESLD) (2019). The updated models were used to calculate the pre-development flow rates for the 2-year, 10-year, and 100-year return period events only.

Historical rainfall data were used to form the hyetographs for the pre-development model, and the climate change projected rainfall was used to form hyetographs for the post-development model using the RCP 6.0 climate change scenario for the 2081 to 2100 time period (as per PNCC ESLD). The RCP 6.0 2081 to 2100 rainfall intensities are generally 11 to 14% larger than historical values.

A continuous modelling approach was used to inform the erosion assessment to calculate the total erosive forces imposed on the receiving watercourses across the representative range of flow events, which utilised 5-years of continuous hourly rainfall data as input.

No changes to the design storm hyetographs or continuous rainfall timeseries applied in the 2022 GHD Report were implemented.

3.1.4 Drainage system

No changes to the pre-development drainage system presented in the 2022 GHD Report were made as part of the model updates.

3.2 Post-development uncontrolled model

The post-development model was developed to include the full extent of residential development within the study area, excluding stormwater management controls. The predicted post-development flow rates and volumes were compared to the target flow rates computed by the pre-development model to determine the storage volume required to attenuate the 2-year, 10-year, and 100-year post-development peak flow rates to the corresponding target peak flow rates.

3.2.1 Sub catchment delineation

The post-development sub catchment areas were modified from the pre-development sub catchments in the study area to represent the preliminary structure plan layout. The lots were assumed to be graded toward the roads, where minor flows are captured and conveyed through a reticulated storm sewer system and major flows are conveyed along the roading corridor.

The post-development sub catchments presented in the 2022 GHD Report were based on the preliminary structure plan layout by McIndoe Urban (received 16 October 2019). The modelling update incorporated minor modifications to the sub catchment boundaries to ensure the full developable area was represented in the model based on the subsequent revisions to the structure plan layout (final dated 11 May 2022). The post-development sub catchments are shown on **Figure 2**.

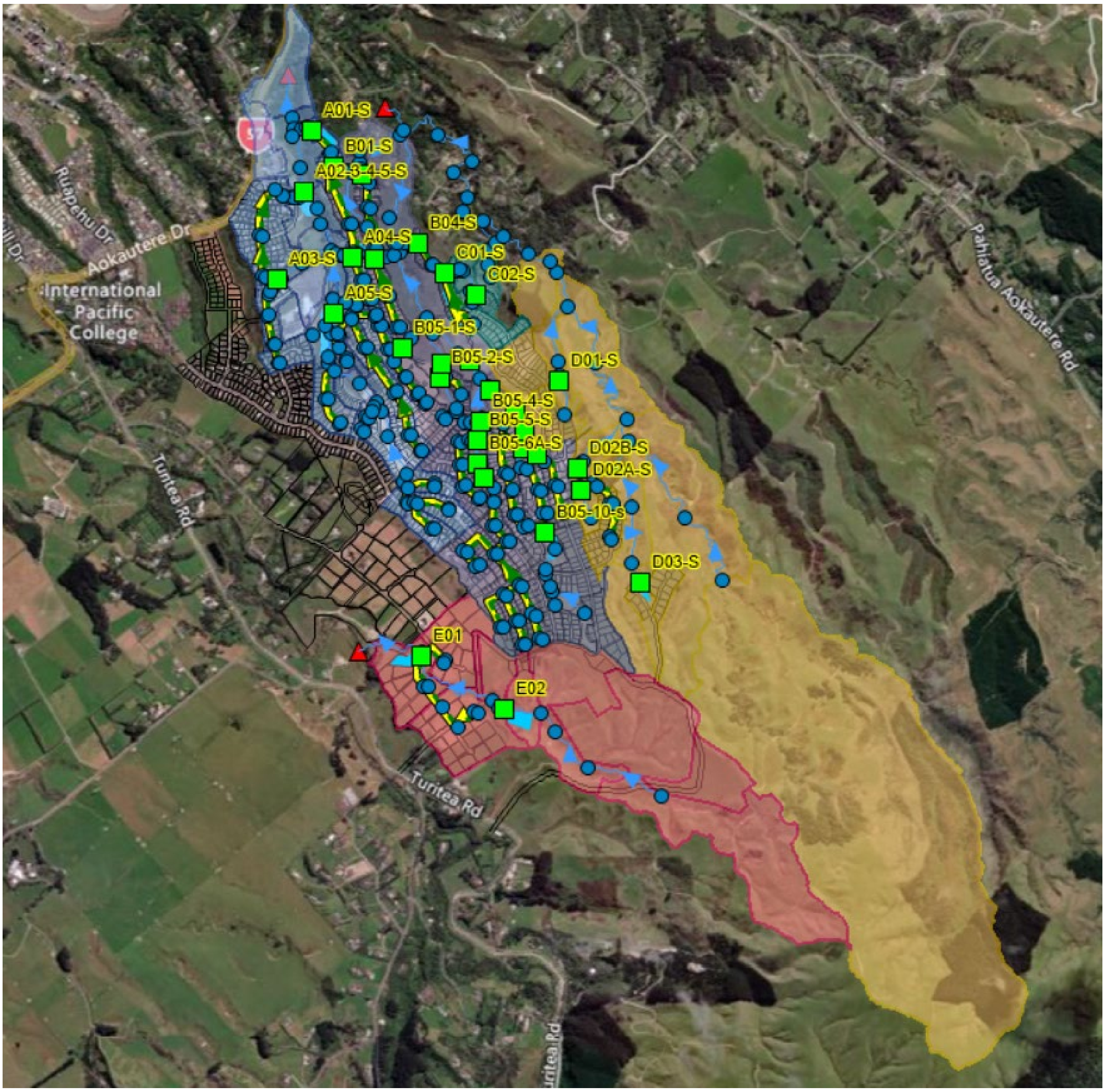

Figure 2A *Overview of post-development sub catchment boundaries and minor discharge locations*

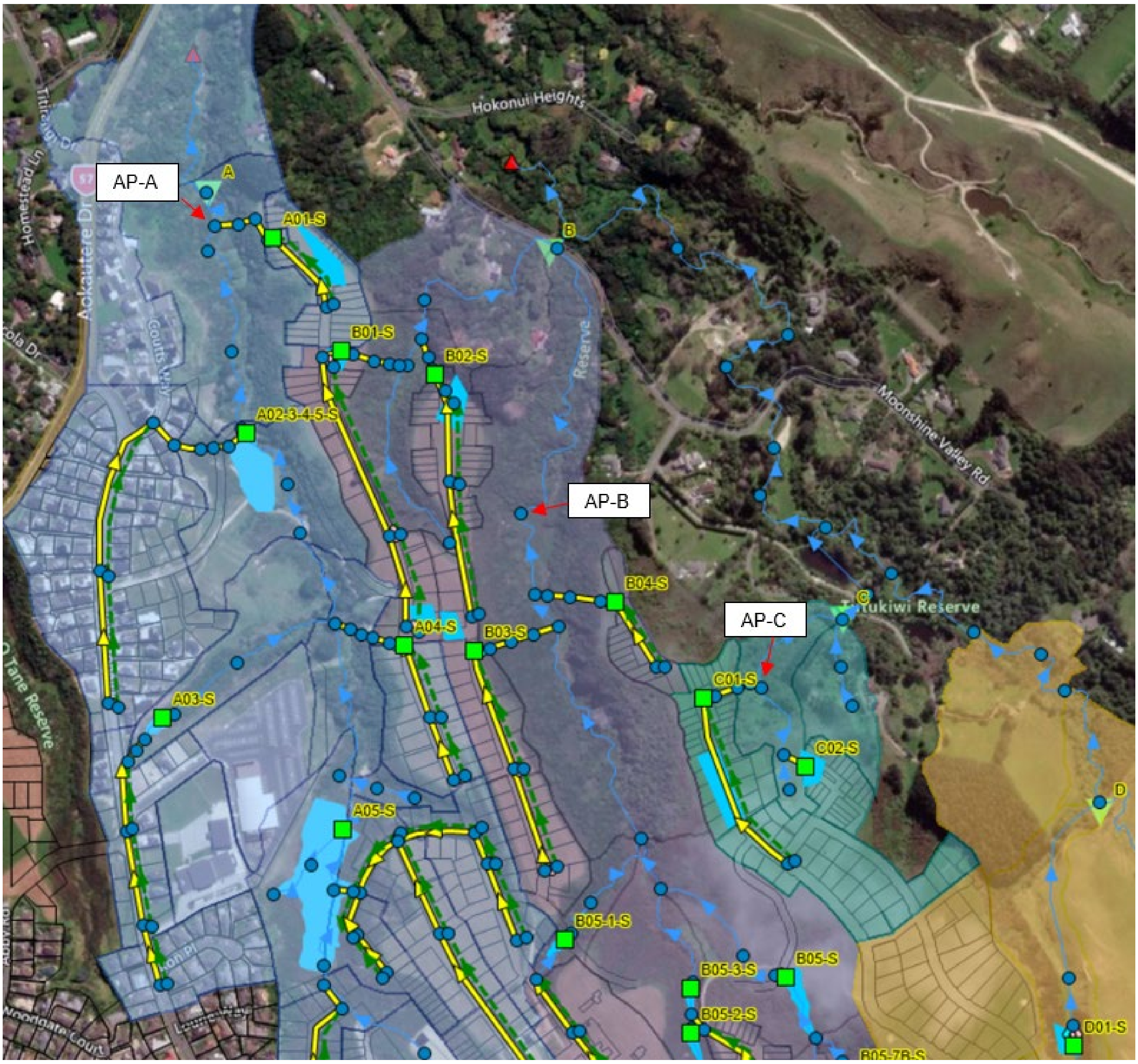

Figure 2B Post-development sub catchment boundaries (north section) and minor discharge locations

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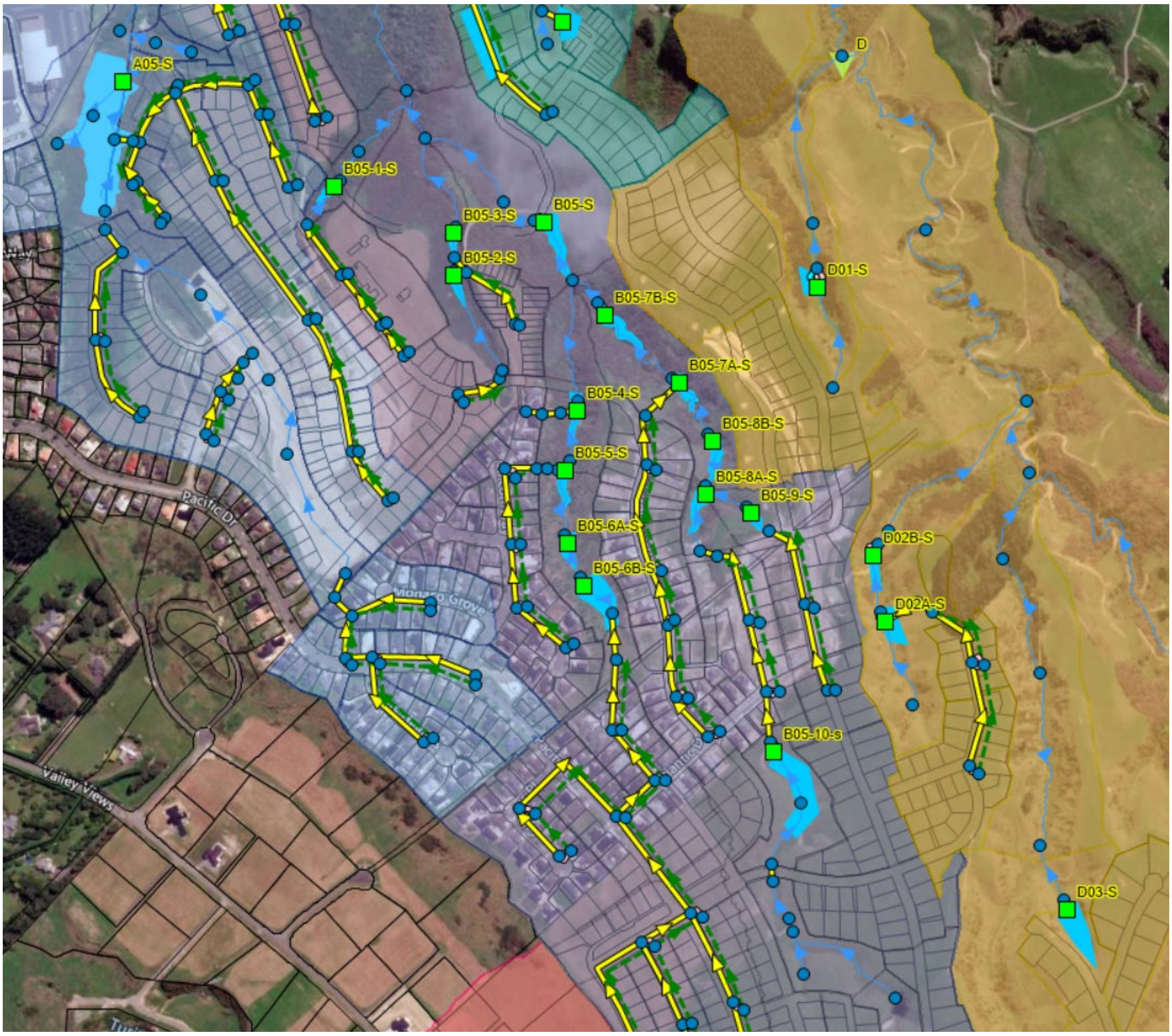

Figure 2C ***Post-development sub catchment boundaries (central section) and minor discharge locations***

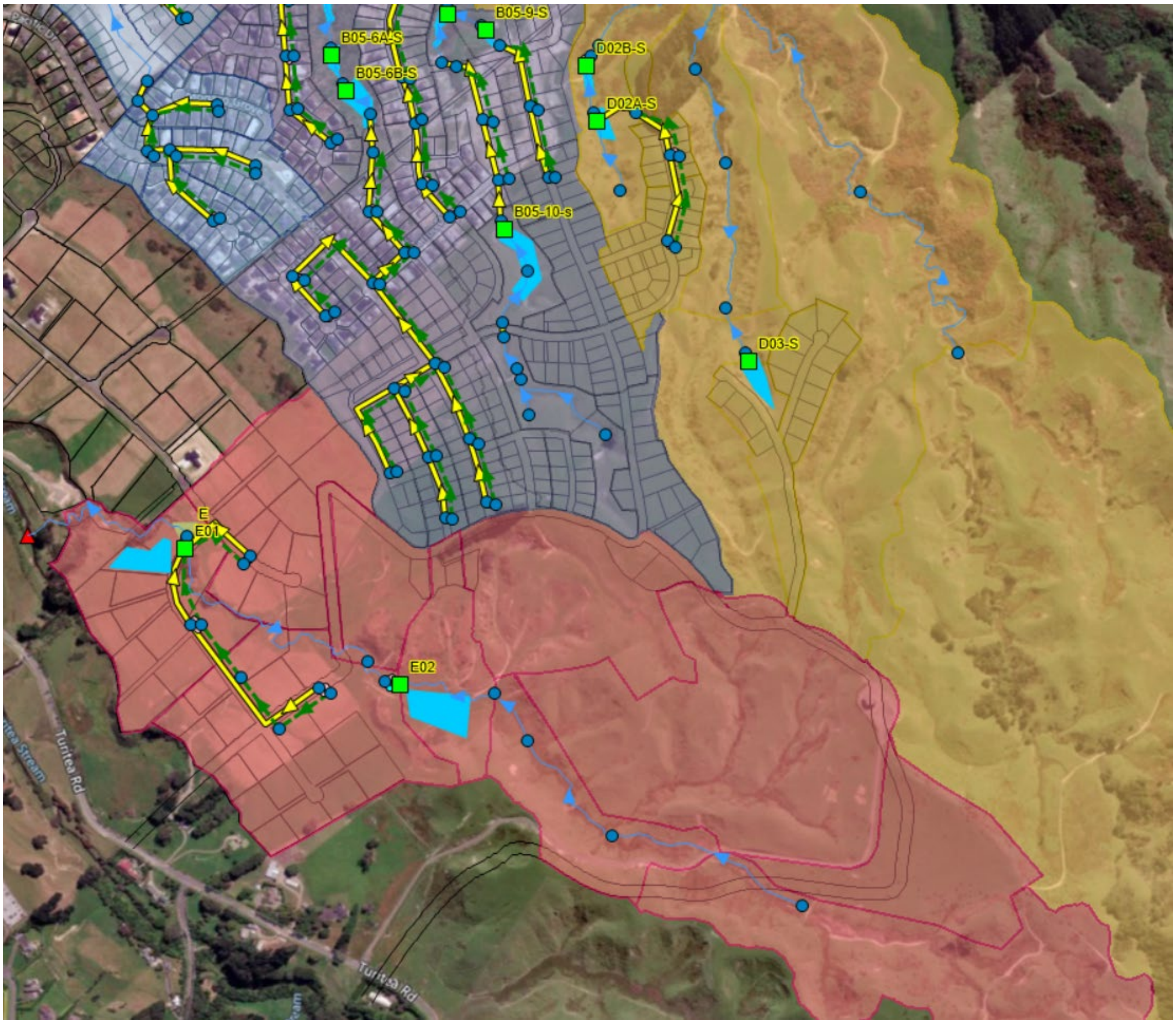

Figure 2D ***Post-development sub catchment boundaries (south section) and minor discharge locations***

3.2.2 Sub catchment parameters

The original post-development sub catchment parameters were reviewed and modified to add conservatism and increase consistency between the approaches used to characterise the pre-and post-development models. The post-development sub catchments located within the developable area were characterised as follows:

- Flow lengths were measured as the longest flow path from the back of the lots to the roading corridor plus a fraction (approximately 10%) of the road length to account for the portion of the total flow length that travels quickly as channelized flow.
- Sub catchment slopes were calculated along the path of the flow length based on the existing topography. A minimum slope of 2% was assigned, which considers future grading works.
- Most developable areas were assigned an imperviousness of 70% (i.e., 30% permeable) (based on typical PNCC practice for recent structure plans.
- Manning's 'n' values of 0.013 and 0.15 were assigned to the impervious (asphalt, concrete) and pervious (maintained grass) land cover types, respectively (USEPA, 2015).

- Typical depression storage values of 2 mm and 5 mm were assigned to the impervious and pervious land cover types, respectively (USEPA, 2015; TP-108).

In the original post-development model, the sub catchments were characterised using one set of "lumped" parameters based on aggregated land cover and soil types, which is simpler, but still an appropriate approach for a feasibility-level study. The updated models utilise the imperviousness parameter, which allows the sub catchments to be characterised using two sets of Manning's 'n' and depression storage values: one set for impervious and the other for pervious land cover types. In addition, in PCSWMM, infiltration models are applied to the pervious land cover types only.

The runoff volumes generated by both impervious and pervious areas are routed directly to the sub catchment outlets, which connect to the roading corridors (then the reticulated storm sewer system), or the natural drainage channels. The model does not account for internal routing that may occur if, for example, runoff from the impervious areas (roofs, driveways, roads) is directed to lawns or rain gardens prior to discharging to the reticulated storm sewer system or the gullies. As such, the model assumes all runoff generated within the catchment will discharge to the designated outfall and will therefore be controlled.

The natural sub catchments within the study area were characterised following the approach used for the pre-development model.

As per the pre-development model, the post-development model updates were applied to the sub catchments draining to Outfalls A, B, C, and D only.

3.2.3 Rainfall inputs

No changes to the design storm hyetographs or continuous rainfall timeseries utilised in the 2022 GHD Report were implemented.

3.2.4 Drainage system

No changes to the approach used to size the reticulated storm sewer and major drainage systems presented in the 2022 GHD Report were implemented. Very minor modifications to the reticulated storm sewer system were made to accommodate the site discharge locations.

4. Stormwater runoff assessment

A flood assessment, erosion assessment, and water quality assessment were undertaken to define the targets for design of the stormwater management controls in the development area for runoff generated by the 2-year to 100-year return period rainfall events.

4.1 Flood assessment

The proposed stormwater management design criterion for flood management is to provide hydraulic neutrality by controlling the post-development peak flow rates to pre-development (target) levels through the provision of storage.

Pre-development peak flow rates were calculated at the major discharge locations using the hydrological and hydraulic models described in section 3. Then pre-development unit peak flow rates were calculated by dividing the peak flow rates by the corresponding contributing drainage area. The resultant pre-development unit peak flow rates were multiplied by the contributing drainage areas of the major and minor discharge locations and additional assessment points located within the same system of the post-development model to establish the target peak flow rates that the post-development peak flow rates are to be controlled to. For example, the pre-development unit peak flow rate at Outfall A was used to establish the target peak flow rates at all discharge locations and assessment points within the Outfall A contributing drainage area of the post-development model.

The target peak flow rates were calculated for a range of return periods. This updated flood assessment demonstrates that flood control can be achieved for the runoff generated by the 2-year, 10-year, and 100-year return period rainfall events.

4.2 Erosion assessment

The purpose of the erosion assessment is to understand the impact of the proposed development on the potential for erosion in the receiving watercourses, which is determined both by consideration of the erosivity of the watercourse bed and bank soils, as well as the magnitude and duration of flows within the watercourse.

The first factor, the erosivity of the watercourse bed and bank soils, is represented by an erosion threshold, representing the flow level at which the bed or bank material will experience entrainment. Theoretical erosion threshold discharges determined for the Stormwater Management Strategy (GHD, 2022) were 0.001 m^3/s for Aokautere Church Stream (Outlet A and Assessment Point A), 0.003 m^3/s for Moonshine Valley Reserve Stream (Outlet B and Assessment Point B) and 0.038 m^3/s for Tutukiwi Reserve Stream (Outlet C, D and Assessment Point C). Assessment Point E is located on Turitea Stream which was not previously assessed during the erosion threshold analysis. The theoretical erosion threshold discharges were determined for fine grained non-cohesive material.

The theoretical erosion threshold discharges were very conservative and are very low flows which may not be representative of actual threshold conditions given that the flow depths were estimated to be 0.024 m, 0.029 m and 0.083 m for typical cross sections in Aokautere Church Stream, Moonshine Valley Reserve Stream and Tutukiwi Reserve Stream respectively. These low flows would not reach the channel banks and only cover a very small portion of the channel beds. A higher threshold of 0.050 m^3/s was also assessed in the study (GHD, 2022) to represent additional shear resistance provided by underlying cohesive fine-grained materials, which may be more realistic in the absence of further field testing.

It is important to remember that the representative erosion threshold discharge is the discharge at which the median grain size (d_{50}) material starts to move. It is not a measure of erosion. It is only an estimate of the entrainment threshold. Erosion will only occur if underlying cohesive material is eroded and transported material is not replaced by transport of material from upstream. Erosion exceedance analysis provides a method to compare the amount of potential within the system to transport material and potentially cause erosion between different flow scenarios.

Exceedance criteria were determined through the use of a continuous hydrologic model to compare time series of discharge over several years. The frequency or cumulative time of exceedance provides a simple comparison of the amount of time the discharge is above the erosion threshold. It does not however account for the excess work above the erosion threshold. The excess work is dependent on the magnitude and duration of the exceedance. It can be represented by the Cumulative Effective Work Index or the total amount of stream power above the erosion threshold as defined by the threshold shear stress. It is calculated following the method described in Rowney and MacRae (1992):

$$PWR = \sum (\tau_o - \tau_{thr}) V \Delta t$$

Where PWR is the cumulative stream energy expended above a threshold value.

τ_o is the instantaneous shear stress at any boundary station.

τ_{thr} is the threshold shear at this boundary station.

Δt time step

V velocity

The Cumulative Effective Work Index was determined for the post-development scenarios and compared to the pre-development conditions to determine potential impacts on the local watercourse geomorphology. Given the uncertainly of the actual erosion threshold discharge, a sensitivity analysis was also performed to assess each major discharge location performance respective to a range of erosion thresholds. Results from the erosion assessment are presented in **Section 5.3**.

4.3 Water quality assessment

The 2022 GHD Report presents a comprehensive list of stormwater management options for water quality control, highlighting wetlands, wet ponds, and bioretention devices.

Wetlands and wet ponds are designed to provide water quality treatment through the provision of a permanent pool volume, and slow release of the water quality volume (WQV) over a 24-hour detention period. The WQV is equivalent to the runoff volume generated by the 90th percentile storm event over all impervious areas within the catchment. The 90th percentile rainfall depth over a 24-hour period is approximately 15 mm based on the Palmerston North AWS record for the 1991 to 2019 period. The WQV also defines the minimum permanent pool volume of the stormwater management devices. Wetlands and wet ponds provide the additional benefit of effective water quantity control through the active storage volume.

Bioretention devices temporarily store, treat, and infiltrate runoff at the source, and are designed to accommodate the water quality flow rate (WQF) calculated using a rainfall intensity of 10 mm/hour (an approximate 90th percentile annual rainfall intensity for Palmerston North determined using the Normalised 24-hour Design Storm distribution from TP-108) for all impervious areas. The minimum area for a bioretention bed is calculated as the WQF divided by the infiltration rate of the engineered filter media, which is typically 1000 mm/hour or less for water quality control only. A safety factor of 0.5 is applied to the infiltration rate to account for clogging of the filter material as per the Stormwater Management Devices in the Auckland Region guideline document (GD 001).

It is important to add that the detention and release of the water quality volume over a 24-hour period is also used to provide stream protection for damage by erosion as outlined in GD 001. The incorporation of bioretention devices into the stormwater management plan can also provide erosion protection by capturing and infiltrating the first 15 mm of a rainfall event over a 24-hour period. For the bioretention areas to provide detention or retention of the WQV the filter media must have an infiltration rate in the range of 50 to 300 mm/hour, the underlying soils must have a minimum infiltration rate of 2 mm/hour, and the device must drain out within 72 hours. The bioretention devices would need to be resized based on the total detention volume equal to the sum of the ponding, media, and drainage storage volumes, or the total retention volume equal to the sum of the infiltration and evapotransporation volumes. The sizing procedure is outlined in GD01.

5. Stormwater concept design

5.1 Overview

For the Aokautere Plan Change the following design criteria are recommended to be adopted:

- Maintain hydraulic neutrality by controlling runoff peak flows to pre-development levels for the 2-year to 100-year return period events to manage flood risk.
- Further control of peak flows needed to match the pre-development erosion threshold exceedance cumulative effective work index in the Aokautere Church Stream (Outfall A), Moonshine Valley Reserve Stream (Outfall B), and Tutukiwi Reserve Stream (Outfall C)
- Treatment of the 90th percentile rainfall volume (i.e., 15 mm) from the development areas through a stormwater treatment device or multi-device system (i.e., wetlands, wet ponds, bioretention)

The location and design of stormwater management controls needs to be considered in the context of a number of factors, including (but not limited to):

- The proposed development layout,
- Sensitive environmental areas (i.e., areas of high vegetation or ecological value),
- Topography of the site,
- Constructability, and
- Ongoing maintenance requirements.

As per the stormwater concept design presented in the 2022 GHD Report, where possible (i.e., when not constrained by other factors), stormwater controls were located in the gully system to minimise the impact on developable area.

An environmental constraint study for the Plan Change was completed by Forbes Ecology in parallel with the original stormwater assessment, which identified areas of terrestrial/vegetation and aquatic ecological value. Note that the aquatic potential of the Aokautere Church Stream and Moonshine Valley Reserve Stream were assessed to be higher downstream of the “low” value vegetation areas, where the streams transition from intermittent to permanent. As well, several wetland and forest areas have been identified by Forbes Ecology in the Waters Land property of the Structure Plan area (catchment E02); these areas were avoided in the placement of conceptual stormwater facilities.

The suitability of gullies to contain stormwater management features was assessed based on the vegetation and aquatic ecology constraints (e.g., no stormwater controls were recommended for areas of “medium” or greater vegetation value), constructability and access, as well as topography of the gullies themselves. A conceptual grading assessment was undertaken to determine the available storage volumes within the gullies where stormwater controls are proposed. Several stormwater controls are in narrow, steep portions of the gully with limited opportunities for stormwater detention. In these cases, the required storage volume is distributed between multiple stormwater controls that would be built in series within the gully system.

Where storage within the gullies could not be achieved, the ponds on the plateaus were located outside of the revised 20 and 30-degree setback lines, as provided by Tonkin and Taylor.

5.2 Controlled post-development model (conceptual detention pond design)

The stormwater design concept consists of detention ponds in the gullies and on the plateaus, and bioretention devices in the developable area. **Figure 2** shows the locations and indicative footprints of the detention ponds based on the updated modelling as blue polygons.

In the 2022 GHD Report the detention ponds were incorporated into the post-development hydrologic and hydraulic model to represent the controlled post-development condition. Then 5-year continuous simulations were run using both the pre-development and controlled post-development models to produce flow timeseries for the erosion threshold analysis. The detention pond storage areas were characterised by simplified area-depth relationships, where area remained constant with changing depth, and the outlets allowed the target peak flow rate to be released from the storage for the corresponding return period rainfall event. For concept design purposes, pond discharge rates corresponding to inflow rates between the design peak flow rates were interpolated, and pond discharge for inflow rates less than the 2-year peak flow rate were interpolated between 0 and the 2-year target peak flow rate.

To add conservatism into the stormwater concept design the following details were incorporated into the updated controlled post-development model:

- Detention pond storage areas were characterised by an area-depth relationship assuming area increases linearly from 0 m^2 to the maximum surface area over the depth.

- Outlets were replaced by combinations of orifice and weir controls, which were included to demonstrate that multi-level control of the 2-year to 100-year peak flow rates can be achieved. A minimum orifice size of 70 mm was used for the detention ponds on the plateaus and in the upstream gully reaches as recommended in GD 001. The minimum orifice size was increased to 100-150 mm for centralised detention ponds on the downstream gully reaches, which have a much larger contributing drainage areas, to balance erosion protection with maintenance considerations.

Table 1 compares the target and controlled peak flow rates at the major discharge locations, minor discharge locations, and additional assessment points for the 2-year, 10-year, and 100-year return period rainfall events. The detention ponds are located immediately upstream of the minor discharge locations.

Table 2 summarises the available storage depth, volume, and surface area of the detention ponds plotted on **Figure 2** and incorporated into the hydrologic and hydraulic model. It also summarises the maximum design depth, volume, surface area, and available freeboard corresponding to the runoff generated from the 100-year rainfall event adjusted for climate change. It is recommended to reconfigure the detention ponds through grading or provision of storage to have a minimum freeboard of 0.3 m.

Table 1 *Detention pond flood assessment performance summary*

Discharge location/ assessment point	Drainage area (ha)	2-year peak flow rate (m^3/s)		10-year peak flow rate (m^3/s)		100-year peak flow rate (m^3/s)	
		Target[1]	Controlled[2]	Target	Controlled	Target	Controlled
Major discharge locations and additional assessment points							
A	71.1	0.476 (0.701)	0.572	1.687 (2.314)	1.400	4.193 (5.449)	4.561
B	97.8	0.828 (1.241)	0.500	3.002 (4.051)	1.824	7.172 (8.908)	5.421
C[3]	8.9	0.068 (0.105)	0.092	0.258 (0.349)	0.277	0.617 (0.782)	0.708
D	228.9	1.293 (1.896)	1.718	4.486 (6.091)	5.926	11.453 (14.675)	13.970
E	97.8	0.273 (0.411)	0.209	1.122 (1.640)	0.825	3.649 (5.002)	3.575
AP A	68.1	0.456 (0.672)	0.439	1.616 (2.216)	0.825	4.015 (5.218)	3.775
AP B	86.7	0.734 (1.100)	0.376	2.661 (3.591)	1.381	6.358 (7.897)	3.870
AP C	7.4	0.076 (0.118)	0.063	0.290 (0.392)	0.203	0.693 (0.878)	0.512
Minor discharge locations at detention pond outlets[4]							
A01	1.571	0.011 (0.015)	0.014	0.037 (0.051)	0.074	0.093 (0.120)	0.376
A02-3-4-5	**60.9**	**0.408 (0.600)**	**0.374**	**1.444 (1.980)**	**0.947**	**3.589 (4.664)**	**3.231**
A03	4.6	0.031 (0.046)	0.051	0.110 (0.150)	0.539	0.272 (0.354)	1.030
A04	2.0	0.014 (0.020)	0.015	0.048 (0.066)	0.151	0.119 (0.155)	0.466
A05	**32.3**	**0.216 (0.318)**	**0.149**	**0.765 (1.050)**	**0.346**	**1.902 (2.472)**	**0.512**
B01	2.4	0.020 (0.031)	0.015	0.074 (0.100)	0.164	0.177 (0.220)	0.489

B02	2.2	0.018 (0.027)	0.015	0.066 (0.089)	0.166	0.158 (0.196)	0.501
B03	2.8	0.024 (0.036)	0.051	0.086 (0.116)	0.408	0.205 (0.255)	0.655
B04	0.8	0.007 (0.011)	0.012	0.026 (0.035)	0.055	0.062 (0.077)	0.200
B05	**56.0**	**0.474 (0.711)**	**0.087**	**1.720 (2.321)**	**0.446**	**4.109 (5.104)**	**2.288**
B05-1	4.1	0.034 (0.052)	0.017	0.125 (0.168)	0.191	0.298 (0.371)	0.922
B05-2	1.1	0.010 (0.014)	0.018	0.035 (0.047)	0.021	0.084 (0.104)	0.024
B05-3	2.3	0.019 (0.029)	0.026	0.071 (0.095)	0.031	0.169 (0.209)	0.038
B05-4	24.9	0.210 (0.315)	0.181	0.761 (1.027)	0.962	1.819 (2.259)	3.606
B05-5	22.3	0.189 (0.284)	0.343	0.686 (0.926)	1.072	1.639 (2.036)	3.431
B05-6A	17.5	0.148 (0.221)	0.417	0.536 (0.723)	0.936	1.280 (1.590)	2.970
B05-6B	15.7	0.133 (0.199)	0.481	0.481 (0.648)	0.871	1.148 (1.426)	2.841
B05-7A	31.2	0.264 (0.396)	0.119	0.959 (1.294)	0.554	2.291 (2.845)	1.946
B05-7B	31.2	0.264 (0.396)	0.033	0.959 (1.294)	0.409	2.291 (2.845)	1.753
B05-8A	22.8	0.193 (0.289)	0.136	0.699 (0.944)	0.627	1.671 (2.075)	1.628
B05-8B	22.8	0.193 (0.289)	0.101	0.699 (0.944)	0.424	1.671 (2.075)	1.356
B05-9	4.7	0.040 (0.060)	0.059	0.146 (0.196)	0.597	0.348 (0.432)	1.064
B05-10	14.7	0.125 (0.187)	0.110	0.452 (0.610)	0.299	1.080 (1.341)	0.635
C01	4.7	0.048 (0.075)	0.028	0.183 (0.248)	0.099	0.439 (0.556)	0.171
C02	0.8	0.008 (0.012)	0.012	0.030 (0.041)	0.041	0.072 (0.091)	0.117
D01	**9.1**	**0.052 (0.076)**	**0.051**	**0.179 (0.244)**	**0.458**	**0.458 (0.587)**	**2.007**
D02A	4.2	0.023 (0.034)	0.013	0.081 (0.111)	0.030	0.208 (0.266)	0.110
D02B	**4.2**	**0.023 (0.034)**	**0.011**	**0.081 (0.111)**	**0.019**	**0.208 (0.266)**	**0.094**
D03	**4.7**	**0.027 (0.039)**	**0.015**	**0.093 (0.127)**	**0.054**	**0.239 (0.306)**	**0.226**
E01	**97.8**	**0.273 (0.411)**	**0.209**	**1.122 (1.640)**	**0.825**	**3.649 (5.002)**	**3.575**
E02	68.7	0.192 (0.288)	0.154	0.787 (1.152)	0.535	2.562 (3.512)	2.562

Notes:

1. Target is equal to the pre-development flow rate using historical rainfall. Pre-development flow rate using climate change adjusted rainfall is provided in brackets.
2. Green – attenuates impacts of development + climate change; Orange – attenuates impacts of development only; Red – does not attenuate development impacts.
3. Drainage area is significantly increased from pre- to post-development conditions; therefore, the target flow rate is equal to the pre-development flow rate at Outfall C.
4. Bolded numbers represent centralised detention ponds with larger contributing drainage areas.

Table 2 *Detention pond geometry summary*

Minor discharge location	Available depth (m)	Available volume (m^3)	Available surface area (m^2)	Maximum depth (m)	Maximum volume (m^3)	Available freeboard (m)
A01	2.5	513	410	2.25	415	0.25
A02-3-4-5	3.0	5,936	3,957	2.94	5,695	0.06
A03	3.0	726	484	2.80	631	0.20
A04	2.5	613	490	2.26	499	0.24
A05	3.0	18,462	12,308	2.71	15,028	0.29
B01	2.5	813	650	2.26	663	0.24
B02	2.5	638	510	2.26	521	0.24
B03	2.5	700	560	2.27	578	0.23
B04	2.0	220	220	1.73	165	0.27
B05	3.0	2,121	1,414	2.83	1,886	0.17
B05-1	3.0	516	344	2.79	446	0.21
B05-2	3.0	636	424	1.34	126	1.66
B05-3	3.0	417	278	2.7	338	0.30
B05-4	3.0	1,142	761	2.93	1,086	0.07
B05-5	3.0	1,010	673	2.92	955	0.08
B05-6A	3.0	1,134	756	2.90	1,058	0.10
B05-6B	3.0	3,366	2,244	2.85	3,044	0.15
B05-7A	3.0	1,140	760	2.85	1,028	0.15
B05-7B	3.0	1,560	1,040	2.82	1,380	0.18
B05-8A	3.0	1,140	760	2.79	984	0.21
B05-8B	3.0	1,275	850	2.82	1,124	0.18
B05-9	3.0	449	299	2.80	390	0.20
B05-10	3.0	5,415	3,610	2.49	3,724	0.51
C01	2.3	2,760	2,400	2.29	2,744	0.01
C02	2.0	210	210	1.93	196	0.07
D01	3.0	1,305	870	2.85	1,176	0.15
D02A	3.0	1,889	1,259	2.38	1,191	0.62
D02B	3.0	861	574	1.82	317	1.18
D03	3.0	2,844	1,896	2.95	2,754	0.05
E01	2.5	8,530	6,824	2.41	7,925	0.09

Minor discharge location	Available depth (m)	Available volume (m^3)	Available surface area (m^2)	Maximum depth (m)	Maximum volume (m^3)	Available freeboard (m)
E02	3.0	16,809	11,206	2.95	16,257	0.05

Table 1 includes the target peak flow rates calculated using the historical climate, with the pre-development peak flow rates calculated using the climate change adjusted rainfall in brackets. The performance results show that the combined effect of the detention ponds can attenuate the increased runoff due to both development and climate change at Outfalls B and E, and Assessment Points A, B, and C for the 2-year, 10-year, and 100-year return period events. Hydraulic neutrality is also achieved for the 10-year peak flow rate at Outfall A. The detention ponds located in the contributing drainage areas of Outfalls C and D are able to attenuate the increased runoff due to development only for the 2-year, 10-year, and 100-year events, which can be explained by the increase in contributing drainage area to Outfall C from pre-development conditions (i.e., 6.6 ha in pre-development conditions to 8.9 ha in post-development conditions, which was further reduced from the 2022 GHD Report), and the large undeveloped area that drains to Outfall D. Outfall D was selected as the most upstream point that captures impacts from development within the D01, D02A, D02B, and D03 sub catchments; however, it has a small developable area compared to its undeveloped area. The developable area is approximately 18.1 ha, compared to the total contributing drainage area of 228.9 ha. The detention ponds located within the Outfall D catchment are too small to accommodate the increased runoff due to climate change over 228.9 ha.

Due to the spatial constraints discussed above (i.e., valuable vegetation, existing development, steep topography) it was not possible to achieve hydraulic neutrality at each of the minor discharge locations. However, the combined performance of the detention ponds resulted in hydraulic neutrality at centralised points on the gully systems. This is achieved by overcontrolling runoff in the detention ponds where space is available. The centralised detention ponds are bolded in **Table 1**. These ponds have relatively larger contributing drainage areas and provide larger storage volumes.

It is important to note that there is a 3.0 ha area at the north end of the Outfall A catchment that is currently developed. Due to vegetative constraints, development, and topographic constraints, the area discharges directly to Outfall A uncontrolled. Assessment Point A was included to assess the performance of the detention ponds immediately upstream of where the uncontrolled runoff enters the system. Similarly, Assessment Point B was located downstream on the gully system, but upstream B01 and B02, which are undersized due to limited space within the developable area. Assessment Point C is located at the most upstream point that captures the discharge from both C01 and C02. It excludes runoff from the undeveloped area that is impacted by climate change.

The detention ponds that do not meet the performance criteria are typically located on the plateaus or in the steep upper reaches of the gully systems. Most of their available storage is utilised to attenuate the 90th percentile rainfall depth over a 24-hour period for erosion protection. Pond outflow hydrographs for the runoff corresponding to the 90th percentile rainfall event is included in **Attachment B**. Due to the long drawdown time, the conceptual detention ponds are at risk of being not empty in back-to-back rainfall events. However, based on the size of the ponds the probability of back-to-back large rainfall events overtopping the ponds is considered to be low. This should be confirmed as part of detailed design by running a long-term time series to test the performance of the pond over multiple years using actual rainfall data.

5.3 Erosion assessment results

An erosion sensitivity analysis was completed for both existing and proposed flow scenarios. The erosion exceedance was compared between pre-development and post-development-controlled scenarios for a range of erosion thresholds. **Figure 3** shows the plot of percent differences of the cumulative effective work index between pre-development and post-development controlled scenarios for each discharge point. The point when the percent difference in cumulative effective work index falls below 0% is the point when the proposed mitigation measures decrease erosion potential within the receiving watercourse. The results show that the overall cumulative effective work decreases if the erosion threshold is greater than 0.01 m^3/s for outlet C, 0.02 m^3/s for outlets B, AP_B and AP_C, 0.03 m^3/s for outlet A and 0.04 m^3/s for outlet AP_A. Outlet D experiences a decrease in cumulative effective work at the smallest threshold of 0.003 m^3/s.

An erosion threshold discharge between 0.010 m^3/s and 0.040 m^3/s seems reasonable given the size of the watercourses and the presence of underlying cohesive material. This low discharge would be a very shallow flow and would have minimal impacts on the channel banks. These thresholds could be checked in the field by measuring flows and observing potential sediment entrainment.

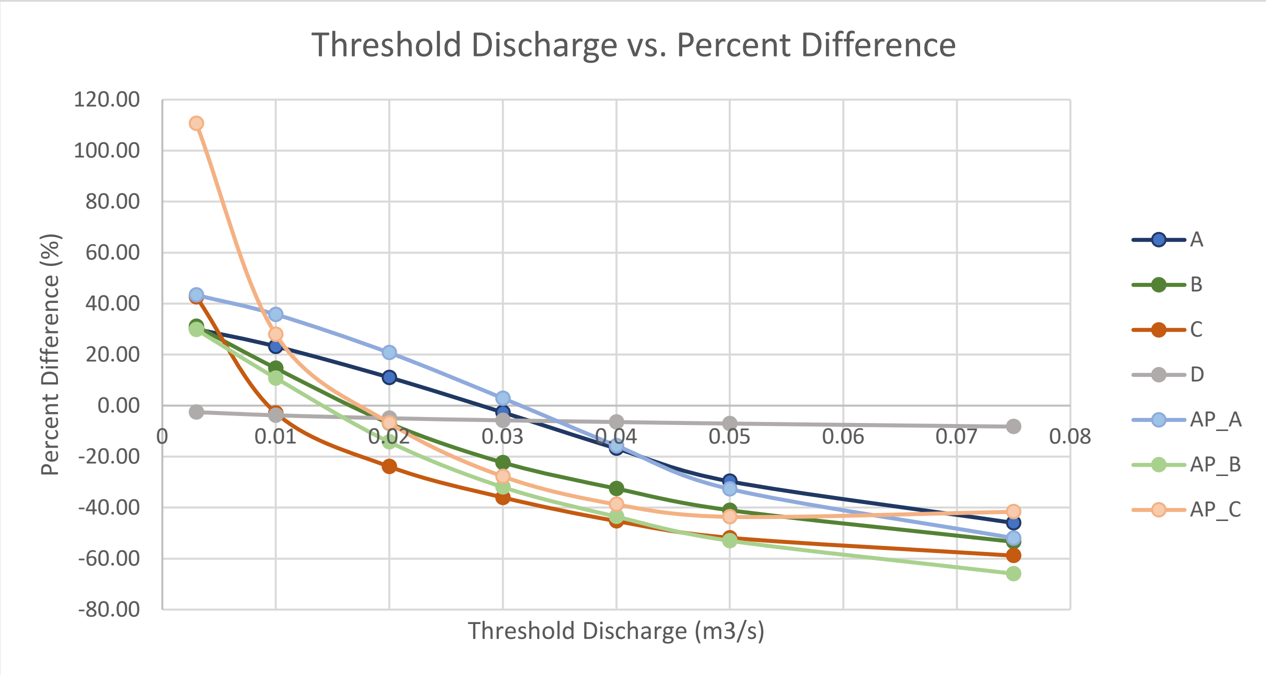

Figure 3 ***Threshold Discharge vs. Cumulative effective work index for pre- and post-development controlled conditions.***

The erosion exceedance analysis of the preliminary controlled post-development conditions indicate that the proposed stormwater detention volumes required for flood risk mitigation would be adequate to mitigate erosion risk, assuming the channel bed can withstand flows up to 0.010 m^3/s – 0.020 m^3/s in Moonshine Valley Reserve Stream (Points B and AP_B) and 0.03 m^3/s to 0.04 m^3/s in Aokautere Church Stream (Points A and AP_A) before the median sized material begins to move. However, this is in comparison only to the pre-development conditions and rates of erosion. The Aokautere Church Stream and Moonshine Valley Reserve Stream will remain highly sensitive to erosion in the future regardless of upstream development and will continue to erode and degrade in a manner that may create slope stability risk or water quality impacts.

5.4 Peak flow impact assessment

The proposed stormwater management is designed to reduce peak flows and reduce erosion in the downstream environment. Although peaks were reduced from pre-development to post-development conditions, this does not necessarily make the receiving watercourse stable. Erosion within gullies is a natural process and is anticipated to continue, despite a reduction in peak flow rates. To prevent erosion from occurring within the natural gullies over a 100-year planning horizon, additional measures would be needed to prevent erosion, or alternatively an adequate setback on the river terrace plateaus would need to be established to account for future erosion and downcutting within the gullies.

5.5 Water quality control

No changes were made to the method used to calculate the WQV, which is equivalent to permanent pool storage in the detention ponds, or to calculate the WQF to size the bioretention devices. However, these calculations were updated to reflect the minor changes to the minor discharge locations from the 2022 GHD Report. **Table 3** summarises the updated WQV and bioretention footprints for water quality control only required within the developable areas.

Table 3 Updated bioretention footprints and permanent pool volumes.

Minor discharge location	Area (ha)	Impervious area (ha)	Percent impervious area (%)	Runoff coefficient (-)	WQF (m^3/hour)	Bioretention footprint (m^2)	Permanent pool volume (m^3)
A01	1.57	1.1	70	0.68	106.9	220	170
A02(1)	8.5	5.95	70	0.68	578.0	1,160	900
A03	4.62	3.24	70	0.68	314.7	630	490
A04	2.03	1.42	70	0.68	139.0	280	220
A05	32.3	20.6	64	0.62	2,015.5	4,040	3,090
B01	2.42	1.69	70	0.68	164.2	330	260
B02	2.2	1.5	70	0.68	146.7	300	230
B03	2.8	2.0	70	0.68	190.4	390	300
B04	0.8	0.6	70	0.68	57.3	120	90
B05-1	4.1	2.9	70	0.68	276.9	560	430
B05-2-3	2.3	1.6	70	0.68	156.4	320	250
B05-4-5-6	24.8	14.7	59	0.58	1,448.8	2,900	2,210
B05-7-8-9-10	31.2	16.9	54	0.54	1,673.6	3,350	2,530
C01	4.7	3.3	70	0.68	317.7	640	500
C02	0.8	0.5	70	0.68	52.5	110	90
D01	9.2	6.4	70	0.68	622.7	1,250	970
D02	4.2	2.9	70	0.68	282.7	570	440
D03	4.8	3.3	70	0.68	324.5	650	510
E01	68.7	13.0	19	0.22	1,509.8	3,020	1,950
E02	97.8	27.5	28	0.30	2,966.9	5,940	4,130

It is important to reiterate that the bioretention devices were conservatively not included in the controlled post-development model used to size the detention ponds for flood and erosion control. It is recommended to use bioretention devices for water quality control as well as erosion protection, as they can also provide runoff control at the source for more frequent events, prior to discharging to the gully system.

6. Conclusions and Recommendations

This technical memorandum summarises the updates to the hydrologic and hydraulic models used to inform the Aokautere Plan Change G Stormwater Management Strategy. Updated sizing and locations of the detention ponds and sizing of bioretention devices are provided to meet the design criteria. The intent of this work is to provide indicative footprints for the required stormwater management controls for the structure plan area as a feasibility test only; these locations and footprints are to be confirmed through subdivision design stage to assess site-specific requirements and constructability. Further, the detention pond designs should include a geotechnical analysis to demonstrate that the proposed detention ponds will not cause slope stability issues due to increased wetting of the soils, or removal of vegetation to construct the outlet berms.

The detention pond outlet structures must be carefully designed, and the system must be regularly inspected and maintained to prevent clogging of the low-capacity orifices included for erosion control. Alternatively, it may be determined through detailed design that erosion protection can be achieved within the developable area through different combinations of bioretention devices, detention pond storage, or super pipe storage within the reticulation system. Prior to the approval of any engineering plans for the proposed developments, the developer should be required to provide a stormwater management report and design demonstrating compliance with these design criteria and concepts which includes the method for capturing all runoff up to the 100-year event on the plateaus. The designed stormwater controls may need to be assessed holistically to demonstrate that hydraulic neutrality is met within the gully system.

7. References

Auckland Council, Stormwater Management Devices in the Auckland Region, Guideline Document 2017/001 Version 1, 2017.

Auckland Regional Council, Guidelines for stormwater runoff modelling in the Auckland Region, Technical Publication No. 108, April 1999.

US Environmental Protection Agency, Storm Water Management Model User's Manual Version 5.1, 2015.

Regards

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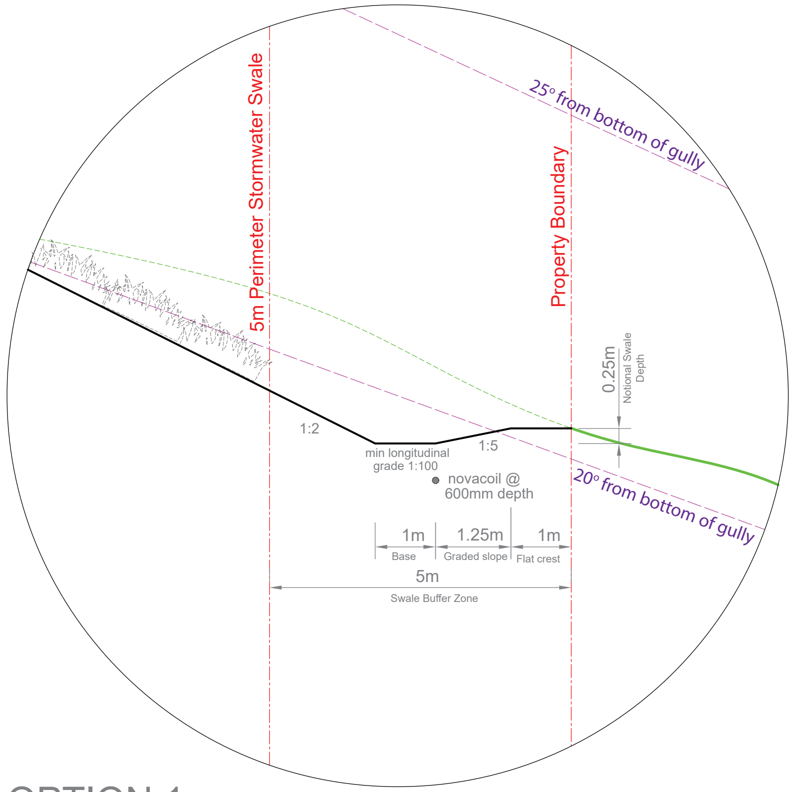
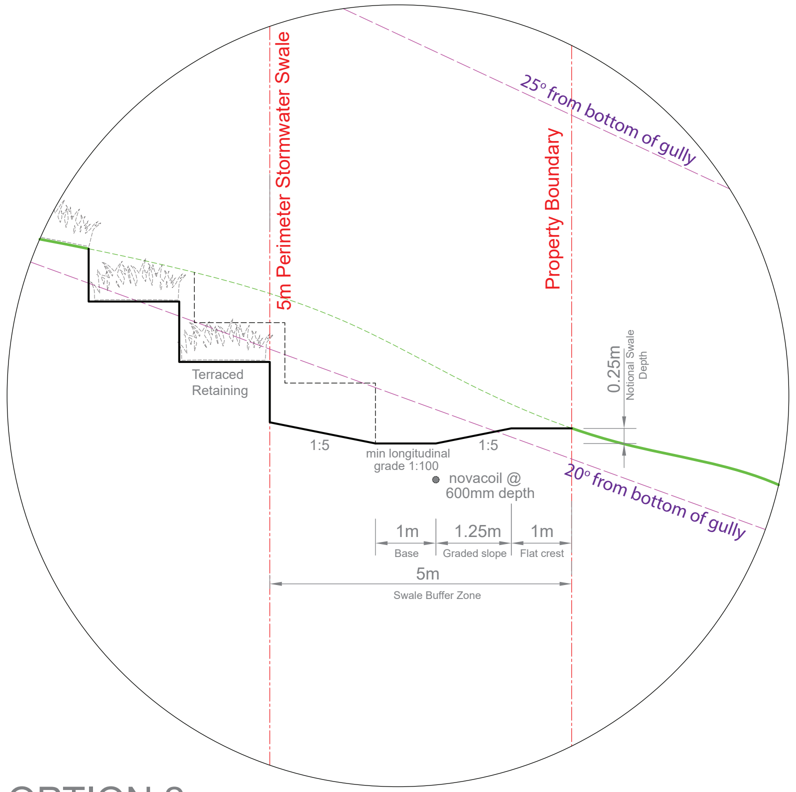
CROSS-SECTION OPTIONS - STORM WATER SWALE

Appendix E



OPTION 1



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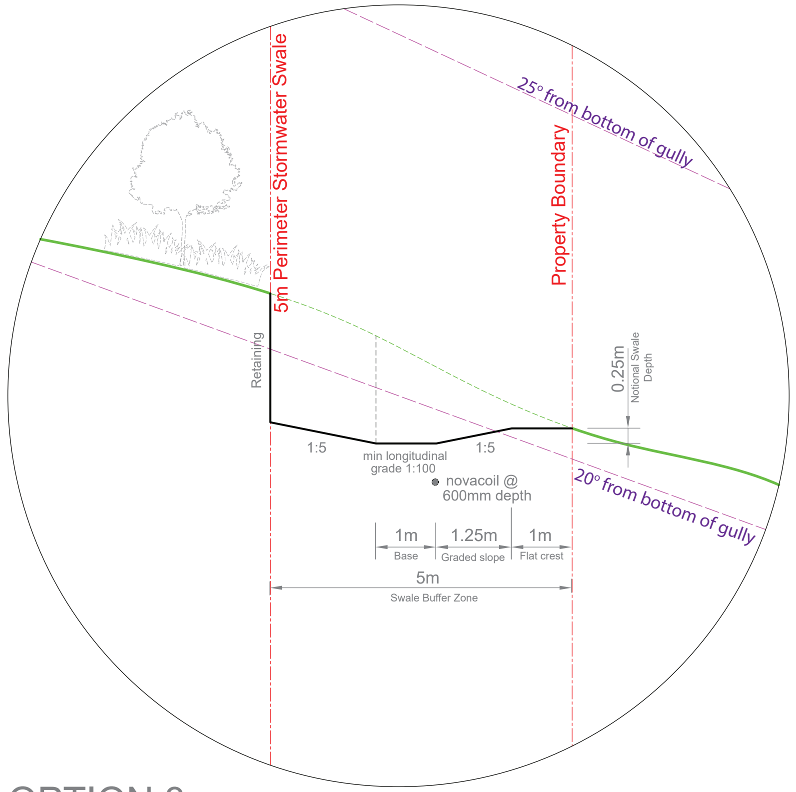

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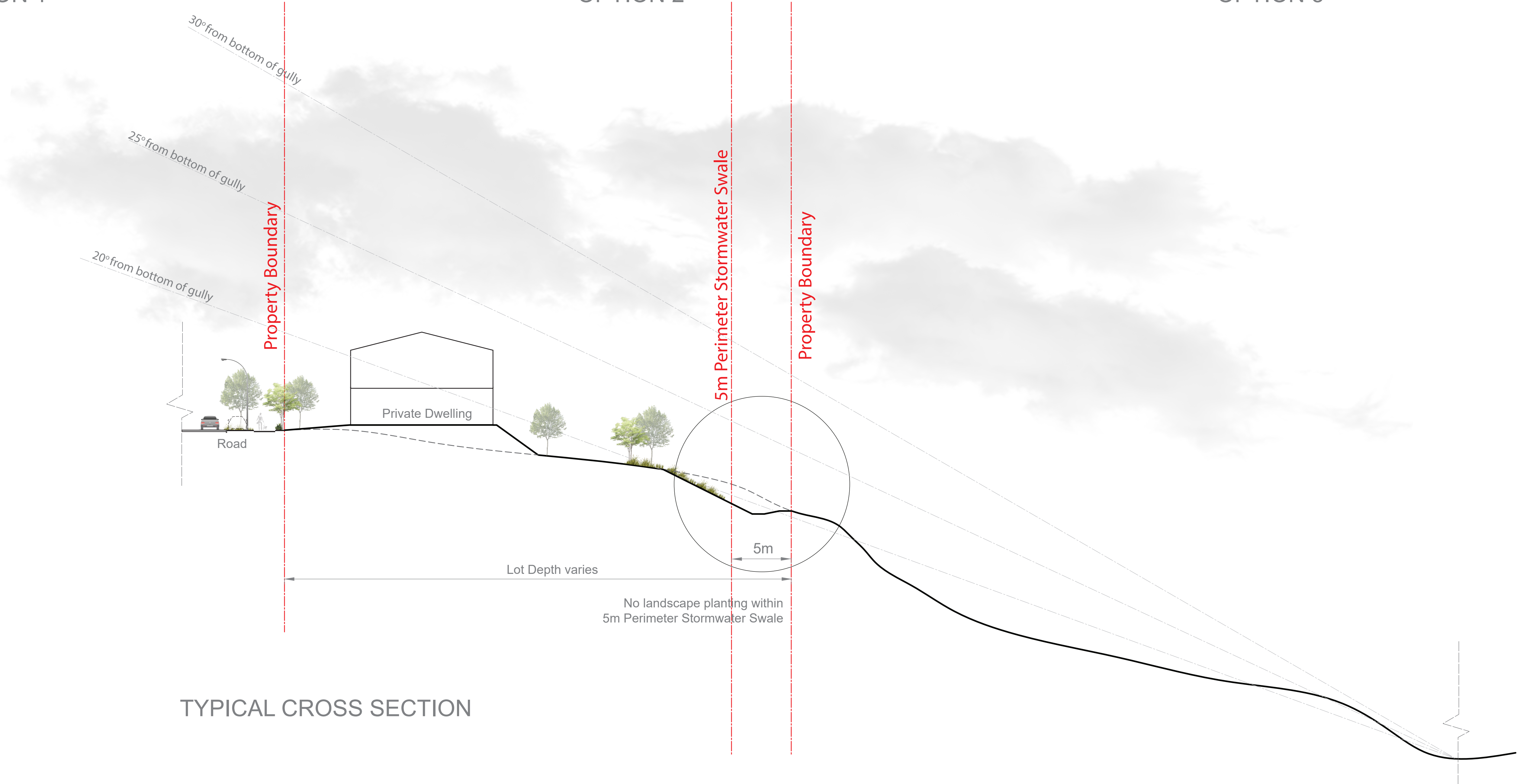

TYPICAL CROSS SECTION