Ruisseau Park Ravine (Blue Willow) Rehabilitation Phase 1: Class Environmental Assessment (EA) and Functional Design

Final

City of Ottawa
Project # TPB208018

Prepared for:

City of Ottawa
655 Shefford Road, Ottawa, ON, K1J 1A6

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Executive Summary

Introduction

Wood Environment & Infrastructure Solutions, a Division of Wood Canada Limited (Wood) has been retained to conduct the Phase 1: Municipal Class Environmental Assessment (EA) and Functional Design for the Ruisseau Park Ravine (Blue Willow) Rehabilitation. The Class EA is being conducted by the City of Ottawa to address issues associated with channel widening and creek bank erosion and valley slope stability concerns along the Ruisseau Park Ravine, a tributary to Mud Creek (ref. Figure 1.1 Study Area Location Plan). Uncontrolled drainage from the urbanized area upstream of the SWM facility is conveyed to the Blue Willow SWM facility, through the Ruisseau Park Ravine, which has led to potential ravine slope stability issues on both public and private lands. The slope stability issues, and erosion concerns have arisen following the urbanization of the contributing drainage area during the 1990's.

Study Area

The study area is located within an urbanized area of the Chapel Hill South neighbourhood, which is a suburb of the City of Ottawa, located 13 km (+/-) east from the downtown core. The study area is bound by Innes Road to the north, Mud Creek to the south, Pagé Road to the east, and the Federal Greenbelt to the west, with total drainage area of 106.30 ha (+/-) to the Mud Creek tributary (ref. Figure 1.1).

Class Environmental Process

The City of Ottawa Ruisseau Park Ravine (Blue Willow) Rehabilitation Class EA and Functional Design has been prepared in accordance with the Municipal Engineers Association (MEA) Class Environmental (Class EA) procedures. The Class EA is following the Municipal Class Environmental Assessment process (Phases 1-4 and Schedule B) which is outlined in the Municipal Engineers Association document.

Existing Conditions Assessment

The key findings of the Ruisseau Park Ravine baseline conditions have been summarized as follows:

Hydrology and Hydraulics

1. A PCSWMM model has been developed for the assessment with a contributing drainage area to the Ruisseau Park Ravine 106.30 ha (+/-) and peak flow rates ranging from 6.70 m^3/s (+/-) to 17.64 m^3/s (+/-) for the 2-100 year design storm events.
2. The contributing drainage area for this study is 33.12 ha (+/-) greater than the drainage area identified in the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) with a 16.17 % increase in imperviousness compared to the previous study.
3. The water surface elevation in the SWMF for the 25 year, 50 year, and the 100 year design storm events have been simulated to overtop the embankment at the downstream end of the facility while the Chapel Hill South SWM study (J.L. Richards and

Associates Limited Consulting Engineers and Planners, July 1989) indicated that the facility would not be overtopped during a 100 year design storm event.

4. A 40 year semi-continuous simulation of the PCSWMM model has been executed to assess the shear stress of the channel banks at multiple channel sections based on the flow rates, depth of flow, and the velocity.

Stream Morphology

5. The stream morphology team from Coldwater Consulting conducted field reconnaissance and a geomorphic assessment. The increased imperviousness of the contributing drainage area, in addition to the implementation of the drainage network, has led to an increase in discharge to the channel during precipitation events; this has led to downcutting and planform migration of the stream and slope instability issues at several locations.
6. The Rapid Geomorphic Assessment (RGA) undertaken for this project identified that more than 75% of the channel, or the upper three (3) reaches, are classified as In Adjustment, while the lower portion of the channel is In Transition.
7. The Rapid Stream Assessment Technique (RSAT) indicates that the upper half of the stream is in Poor condition while the lower half of the stream is in Fair condition which generally corresponds to the RGA results.
8. Based on the geomorphic assessment, multiple locations within Ruisseau Park Ravine have been identified as requiring bank stabilization to mitigate further erosion.

Geotechnical

9. The seven (7) boreholes and two (2) Piezocone Penetration Testing (CPT) holes advanced by Wood's Geotechnical team identified the soils within the ravine as primarily being clay and silty sand; bedrock was not encountered within the advanced boreholes.
10. Slope stability analyses have been performed for eight (8) slopes in which three (3) slopes, SLP2, SLP4, and SLP6 have been identified as requiring a bank stabilization treatment to prevent progressive erosion, while there is concern that erosion at the toe of a fourth slope, SLP7, may contribute to slope instability.
11. The sensitive clay soils at the site are consistent with the clay soils which were identified at locations of previous retrogressive slope failure sites near Lemieux, Ontario; the soils at the site are vulnerable to an earthflow risk. However, in the absence of an adverse triggering conditions the earthflow slides may never occur at this site.

Natural Systems

12. The ELC delineation completed by Wood's Natural Heritage Team during the field investigation identified four (4) communities within the study area; manicured landscapes, Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite, Reed Canary Grass Graminoid Meadow, and Fresh-Moist Willow Lowland Deciduous Forest.

13. A preliminary desktop Species at Risk (SAR) screening identified 15 SAR, which have the potential to occur with the Project Location, while eight (8) SAR of those species have been identified with a moderate to high potential of occurrence in the study area.
14. Potential snake hibernaculum was found within the Reed Canary Grass Graminoid Meadow during the field investigation along the west bank of the SWMF. Other potential Significant Wildlife Habitat (SWH) within the Project Location include bat maternity colonies, habitat of special concern or rare wildlife species, and potential turtle nesting areas.

Alternative Assessment

Following the screening of a long list of creek improvement alternatives to address the existing creek bank erosion conditions that are to be integrated with slope stability alternatives, the seven (7) short-listed alternatives have been evaluated based on the functionality, impacts to the natural environment, impacts to the social environment, and the capital and maintenance costs associated with the alterative. The assessment of the seven (7) short listed alternatives has been summarized as follows:

Alternative 1 Do Nothing has been screened from further consideration as it would not address the existing creek bank erosion conditions and would not help facilitate slope mitigation works.

Alternative 2 Stormwater Management has been screened out from further consideration as it would not address existing creek bank erosion conditions, that said, LID BMPs or source controls could be implemented with a combined alternative. The LID BMPs are viewed as a complimentary long-term alternative.

Alternative 3 Creek Realignment would not fully address the creek bank erosion conditions, as such it has been considered as part of a combined alternative.

Alternative 4 Creek Partial Enclosure has been screened out for further consideration, as it would not address the existing creek bank erosion conditions and would result in long term maintenance requirements and eventual replacement.

Alternative 5 Creek Control Structures would reduce creek erosion risk, by reducing flow regime energy, but would not fully address the existing creek bank erosion conditions, as such it has been considered as part of a combined alternative.

Alternative 6 Channel Hardening would stabilize the existing channel in key locations, but may result in erosion in new locations, as such the alternative has been considered as part of a combined alternative.

Alternative 7 Combination of Alternatives, considers various aspects of the alternatives indicated to be assessed in a combined alternative. The combined alternatives have been advanced for further consideration and are assessed in detail within the following sections.

The individual Alternatives 1-6 would not address the existing creek bank erosion conditions, however, the combination of Alternatives 3, 5, and 6 would address existing erosion conditions. A summary of the combination of alternatives which have been assessed and evaluated is provided as follows:

Alternative 7a Realignment, Reshaping and Hardening (Hardening Focus) combines realignment of the creek to address existing slope stability issues and reshaping and hardening of the creek cross-section to address erosion issues. The channel would be realigned away from the slopes of concern to provide space to implement slope stabilization works at the slopes of concern. The channel realignment would not reduce the creek energy as the shear stress would exceed critical values for the native material. As such, channel hardening would be required to address the creek energy.

Alternative 7b Realignment, Structures and Hardening (Structures Focus) involves a combination of the control structures of Alternative 5 with the realignment and naturalized armouring of Alternative 7a. This alternative incorporates weir-pool features (control structures) to dissipate the creek energy. Weir-pool or step-pool features are well suited for channel with relatively steep gradients. The pool below each of the six (6) weir structures would be armoured to prevent scour. The weir structures would reduce the energy gradeline (surface slope) and consequently the velocity and erosion potential. The realignment of the channel would be consistent with Alternative 7a to move the channel away from the slopes of concern to provide space to implement slope stabilization works. However, channel hardening and bank hardening with void-filled rip rap would still be required at select locations to address the creek energy (shear stress).

Alternative 7c Realignment, Structures and Naturalization (Adaptation Focus) involves a combination of the reshaping and realignment of Alternative 7a/b with the use of habitat features and natural construction materials to restrain the stream's morphological evolution. Unlike Alternatives 7a and 7b, this design would allow for some stream evolution to continue, albeit in a controlled manner. The identified eroded locations would be protected while limiting the disturbance of the existing adjacent areas. The channel realignment would be consistent with the realignment proposed for alternatives 7a/b; the realigned channel would be widened and hardened with void will riprap. Cross vanes and log weirs would be constructed within the channel to reduce the erosion potential, similar to Alternative 7b. Monitoring would be required for this alternative as the channel naturally evolves.

The Preferred Alternative selected from the Short-Listed Alternatives is **Alternative 7b: Realignment, Structures and Hardening (Structures Focused)**.

The creek realignment facilitates, opportunity to provide slope stabilization works, while the weir-pool structures provide a long-term solution to the ongoing creek bank erosion by reducing the creek energy and minimize the construction footprint. The weir-structures are intended to reduce maintenance by the city, compared to using cross vanes and log weirs, which will require repair and replacement, as material degrade and move.

Analysis of the proposed slope stabilization work, proposed creek realignment, and toe protection work has been undertaken to confirm the acceptability of the stabilization works while providing preliminary recommendations for the slopes of concern. The void-filled riprap banks and the vegetated riprap upper banks would provide adequate long-term erosion protection along the realigned slope toes. The combination of the proposed creek alignment, proposed bank repairs, and channel treatments, would satisfy the required factor of safety (FoS) for the slopes of concern.

Recommendations

The following recommendations have been prepared based on the findings of this study:

1. The preferred alternative identified from this assessment, Alternative 7b Realignment, Structures and Hardening (Structures Focused), costed at $2.6M has been recommended to be advanced to the next phase of planning and design. The six (6) control structures will dissipate the channel energy while the realignment of the channel will move the channel away from the slopes of concern to facilitate the slope stabilization measures.
2. The City should continue monitoring slopes 6 and 7 until construction has been completed to document the condition at those slopes. Monitoring of the ravine to ensure progressive creek bank erosion does not destabilize slopes prior to construction of the recommended creek improvement sand slope protection works is recommended. Based on Stantec’s March 2022 Slope Stability Observation Program memorandum and the observation of soil being placed at 13 Sprucewood, a temporary 0.5 m high stone buttress at the toe of slope behind 13 Sprucewood, would prevent slope instability.
3. Slope stabilization works recommended for the four (4) slopes of concern include placing clay fill material within the former channel, in addition to riprap. The estimated cost for the fill material is $361,000.
4. The slopes and channel banks should be revegetated following the construction works with native tree species, shrubs, and ground cover and the City’s forestry department should be consulted when identifying the species for the site. Cedars are recommended to be planted to address residential privacy concerns as the cedar bushes will provide year-round visual barrier.
5. Three (3) maintenance access paths should be constructed, not only to provide access during construction, rather to also provide access following construction should maintenance be required at or near the channel and the control structures; the estimated cost of the maintenance assess paths is $26,000.
6. It is recommended that the number of trees to be removed is kept to a minimum to reduce the amount of disturbance to potential bat species at risk habitat; 77 trees have been identified for removal to facilitate the construction of the preferred alternative while 40 additional trees can likely be preserved if there is no interference during construction.

7. The use of LID BMPs should be promoted in conjunction with other alternatives as a long-term objective to reduce the runoff conveyed to the Blue Willow SWM facility. Implementation of the LID BMPs within the public realm could be undertaken during the reconstruction of the roadways. Similarly, LID measures could be implemented within the private properties, for example by redirecting downspouts to pervious areas, or by installing permeable pavers instead of asphalt driveways at the time of reconstruction.
8. Consultation should be undertaken during the detailed design with agencies, First Nations groups, and neighbourhood associations and residents to ensure their understanding of the proposed mitigation works and to address any concerns for the project.

Note de synthèse

Introduction

La Ville d’Ottawa a retenu Wood – Solutions en environnement et infrastructure, division de Wood Canada limitée (Wood), pour réaliser la phase 1 du projet de réfection du ravin du parc Ruisseau (Blue Willow), soit l’évaluation environnementale (EE) municipale de portée générale et la conception fonctionnelle. Cette EE de portée générale concerne le ruisseau Blue Willow, un affluent du ruisseau Mud. Elle vise à régler les problèmes associés à l’élargissement du chenal, à l’érosion des berges du ruisseau et à l’instabilité du talus le long du ravin du parc Ruisseau (voir le plan du secteur à l’étude à la figure 1.1). Les problèmes de stabilité et d’érosion sont apparus à la suite de l’urbanisation de la zone de drainage pendant les années 1990.

Secteur à l’étude

Le secteur à l’étude fait partie d’une zone urbanisée du quartier Chapel Hill-Sud, qui est une banlieue de la Ville d’Ottawa, située à 13 km (+/-) à l’est du centre-ville. Il est cerné par le chemin Innes au nord, le ruisseau Mud au sud, le chemin Pagé à l’est et la Ceinture de verdure fédérale à l’ouest, ce qui représente une zone de drainage totale de 106,30 ha (+/-) jusqu’à l’affluent du ruisseau Mud (voir la figure 1.1).

Processus d’évaluation environnementale de portée générale

L’EE de portée générale et la conception fonctionnelle du projet de réfection du ravin du parc Ruisseau (Blue Willow) de la Ville d’Ottawa ont été préparées conformément aux procédures d’évaluation environnementale de portée générale de la Municipal Engineers Association. L’EE de portée générale suit le processus (phases 1 à 4 et annexe B) décrit dans le document de cette entité.

Évaluation de la situation actuelle

Voici les principaux constats sur les conditions actuelles du ravin du parc Ruisseau :

Hydrologie et hydraulique

1. Aux fins de l’évaluation, un modèle PCSWMM a été élaboré avec une zone de drainage pour le ravin du parc Ruisseau de 106,30 ha (+/-) et des débits de pointe allant de 6,70 m^3/s (+/-) à 17,64 m^3/s (+/-) pour les averses types survenant tous les 2 à 100 ans.
2. La zone de drainage définie pour cette étude couvre 33,12 ha (+/-) de plus que celle utilisée dans l’étude sur la gestion des eaux pluviales (GEP) de Chapel Hill-Sud (J.L. Richards and Associates Limited Consulting Engineers and Planners, juillet 1989), l’imperméabilité ayant augmenté de 16,17 % par rapport à l’étude précédente.
3. Selon les résultats des simulations, le niveau de l’eau pour les averses types survenant tous les 25, 50 et 100 ans dépasse le remblai à l’extrémité aval du ravins, tandis que l’étude sur la gestion des eaux pluviales de Chapel Hill-Sud (J.L. Richards and Associates Limited Consulting Engineers and Planners, juillet 1989) indiquait qu’il n’y aurait pas de débordement en cas d’averse type survenant tous les 100 ans.

4. Une simulation semi-continue sur 40 ans du modèle PCSWMM a été réalisée pour évaluer la force tractrice sur les berges du chenal dans plusieurs sections du cours d'eau en fonction du débit, de la profondeur de l'écoulement et de la vitesse.

Morphologie du cours d'eau

5. L'équipe de morphologie des cours d'eau de Coldwater Consulting a effectué une reconnaissance sur le terrain et une évaluation géomorphologique. L'imperméabilité accrue de la zone de drainage contributive, combinée à la mise en place du réseau de drainage, a entraîné une augmentation du ruissellement dans le chenal pendant les précipitations, ce qui cause des problèmes d'encaissement, de modification en plan du cours d'eau et d'instabilité du talus à plusieurs endroits.
6. L'évaluation géomorphologique rapide réalisée pour ce projet a permis de déterminer que plus de 75 % du chenal, soit les trois (3) tronçons supérieurs, sont classés comme étant en cours d'ajustement, tandis que la partie inférieure du chenal est en transition.
7. La technique d'évaluation rapide des cours d'eau indique que la moitié supérieure du cours d'eau est en mauvais état, tandis que la moitié inférieure est dans un état passable, ce qui correspond en gros aux résultats de l'évaluation géomorphologique rapide.
8. D'après l'évaluation géomorphologique, pour freiner l'érosion, il faudrait stabiliser les berges de plusieurs tronçons du ravin du parc Ruisseau.

Géotechnique

9. Les sept (7) trous de forage et les deux (2) trous d'essais de pénétration au piézocône réalisés par l'équipe géotechnique de Wood ont permis de déterminer que le sol dans le ravin se compose principalement d'argile et de sable limoneux; dans les trous de forage qui ont été creusés, aucun substrat rocheux n'a été rencontré.
10. Des analyses de stabilité du talus ont été effectuées pour huit (8) talus; trois (3) d'entre eux, SLP2, SLP4 et SLP6, nécessiteraient un traitement de stabilisation des berges si l'on veut prévenir l'érosion progressive. L'érosion au pied d'un quatrième talus, SLP7, pourrait contribuer à l'instabilité du talus.
11. Les sols argileux sensibles trouvés sont semblables aux sols argileux qui ont été identifiés là où un glissement rétrogressif est survenu près de Lemieux, en Ontario; ces sols sont en effet exposés à un risque de glissement de terrain. Cependant, si les conditions de déclenchement ne sont pas réunies, il se pourrait qu'il n'y ait jamais de glissement de terrain à cet endroit.

Systèmes naturels

12. La classification écologique des terres effectuée par l'équipe du patrimoine naturel de Wood au cours de l'enquête sur le terrain a permis d'identifier quatre (4) aires dans le secteur à l'étude : paysages soigneusement entretenus, écosite de forêt mixte d'érables à sucre et de pruches sur sol sec à frais, prairie graminoïde à alpiste roseau, et forêt feuillue de saules en basses terres (milieu humide soumis à la crue).

13. Un examen administratif préliminaire des espèces en péril a permis d'identifier quinze (15) espèces en péril qui sont susceptibles de se trouver dans la zone du projet, huit (8) d'entre elles ayant un potentiel d'occurrence allant de modéré à élevé dans le secteur à l'étude.
14. Un possible hibernacle de couleuvre a été découvert dans la prairie graminoïde à alpiste roseau au cours de l'étude sur le terrain le long de la rive ouest des installations de GEP. Parmi les autres habitats fauniques d'importance potentiels, on compte des pouponnières de chauves-souris, des habitats d'espèce sauvages vulnérables ou rares, et de possibles aires de nidification de tortues.

Analyse des options

À la suite de l'examen préalable d'une longue liste d'options pour résoudre le problème d'érosion des berges du ruisseau, auxquelles doivent être intégrées des solutions pour stabiliser le talus, sept (7) options ont été évaluées à l'égard de leur fonctionnalité, de leurs répercussions sur l'environnement naturel et social et des coûts d'immobilisations et d'entretien à prévoir. Voici en bref l'analyse qui en a été faite :

L'option 1 (statu quo) a été exclue, car elle ne permettrait pas de régler les problèmes actuels d'érosion des berges du ruisseau et ne faciliterait pas les travaux de stabilisation des talus.

L'option 2 (gestion des eaux pluviales) a également été écartée, car elle ne permettrait pas non plus de régler les problèmes d'érosion. Cela dit, des pratiques exemplaires d'aménagement à faible impact ou des contrôles à la source pourraient être combinés à une autre option. Il s'agirait d'une option complémentaire à long terme.

L'option 3 (détournement du ruisseau) ne permettrait pas de régler complètement les problèmes d'érosion des berges; elle est donc considérée comme pouvant faire partie d'une solution à plusieurs volets.

L'option 4 (enfouissement partiel du ruisseau) a été écartée, car elle ne permettrait pas de régler les problèmes actuels d'érosion des berges, et l'ouvrage nécessiterait des frais d'entretien considérables.

L'option 5 (ouvrages de régulation du ruisseau) atténuerait le risque d'érosion en réduisant l'énergie du régime d'écoulement, mais elle ne permettrait pas de régler complètement les problèmes d'érosion; par conséquent, elle est considérée comme pouvant faire partie d'une solution à plusieurs volets.

L'option 6 (fortification du ruisseau) stabiliserait le chenal existant à des endroits stratégiques, mais pourrait entraîner de l'érosion à d'autres endroits, de sorte qu'elle est considérée comme pouvant faire partie d'une solution à plusieurs volets.

L'option 7 (combinaison d'options) intègre divers aspects des options qui sont considérées comme pouvant faire partie d'une solution à plusieurs volets. Ces solutions ont été soumises à un examen plus approfondi et sont présentées en détail ci-dessous.

Les options individuelles 1 à 6 ne permettraient pas de régler les problèmes actuels d'érosion des berges du ruisseau, raison pour laquelle une combinaison des options 3, 5 et 6 a été considérée. Voici un résumé de l'analyse des différentes combinaisons d'options qui ont été évaluées :

L'option 7a (détournement, remodelage et fortification [accent sur la fortification]) combine le détournement du ruisseau (pour régler le problème de stabilité du talus) avec le remodelage et la fortification en section transversale du ruisseau (pour régler les problèmes d'érosion). Le chenal serait détourné des talus qui posent problème, ce qui ferait de la place pour les travaux de stabilisation. Le détournement du chenal ne réduirait pas l'énergie du ruisseau, car la force tractrice dépasserait les valeurs critiques pour le matériau d'origine. Par conséquent, la fortification du chenal serait nécessaire pour gérer l'énergie du ruisseau.

L'option 7b (détournement, ouvrages et fortification [accent sur les ouvrages]) combine les ouvrages de régulation de l'option 5 avec le détournement et le blindage naturalisé de l'option 7a. Cette option comprend des bassins de déversement (ouvrages de régulation) pour dissiper l'énergie du ruisseau. Ces bassins (aussi appelés bassins en gradins) sont particulièrement indiqués pour les chenaux ayant une pente relativement raide. Le bassin sous chacun des six (6) déversoirs serait blindé pour en prévenir l'affouillement. Les déversoirs réduiraient la ligne de charge (la pente de la surface) et, par conséquent, la vitesse et le potentiel d'érosion. Comme dans l'option 7a, le chenal serait détourné des talus qui posent problème, ce qui ferait de la place pour les travaux de stabilisation de talus. Toutefois, il faudrait quand même fortifier le chenal et les berges à certains endroits au moyen d'un enrochement pour remplir les vides si l'on veut réguler l'énergie du ruisseau (la force tractrice).

L'option 7c (détournement, ouvrages et naturalisation [accent sur l'adaptation]) combine le remodelage et le détournement des options 7a et 7b avec l'utilisation de caractéristiques de l'habitat et de matériaux de construction naturels pour limiter l'évolution morphologique du ruisseau. Contrairement aux options 7a et 7b, cette solution permettrait au cours d'eau de poursuivre son évolution, quoique de façon contrôlée. Les endroits déjà érodés seraient protégés et on limiterait la perturbation des zones adjacentes. Le chenal serait détourné de la même façon que celle qui est proposée pour les options 7a et 7b, et serait élargi et fortifié au moyen d'un enrochement pour remplir les vides. Des structures à ailettes croisées et des déversoirs en rondins seraient construits dans le chenal afin de réduire le potentiel d'érosion, un peu comme pour l'option 7b. Par ailleurs, une surveillance serait nécessaire, puisque le chenal continuerait à évoluer de façon naturelle.

L'option privilégiée parmi celles qui ont été retenues est l'**option 7b (détournement, ouvrages et fortification [accent sur les ouvrages])**.

Le détournement du ruisseau faciliterait les travaux de stabilisation des talus, et les bassins de déversement constitueraient une solution à long terme pour contrer l'érosion des berges en diminuant l'énergie du ruisseau et réduiraient au minimum l'empreinte du chantier. Avec les bassins de déversement, la Ville aura moins d'entretien à faire que si elle utilisait des structures à ailettes croisées et des déversoirs en rondins, qu'il faudrait réparer et remplacer à mesure que leurs matériaux se dégraderaient et se déplaceraient.

Une analyse des travaux proposés de stabilisation des talus, de détournement du ruisseau et de protection du pied du talus a été réalisée pour vérifier l'acceptabilité des travaux de stabilisation et formuler des recommandations préliminaires pour les tronçons du talus qui posent problème. L'enrochement (pour remplir les vides, végétalisé sur la partie supérieure) offrirait une protection adéquate à long terme contre l'érosion au pied des talus le long des tronçons détournés. La combinaison de ce qui est envisagé relativement au tracé, à la réparation des berges et au traitement du chenal satisferait au facteur de sécurité requis pour les talus qui posent problème.

Recommandations

Les recommandations suivantes ont été élaborées en fonction des résultats de la présente étude :

1. Il est recommandé de faire passer à l'étape de planification et de conception l'option privilégiée parmi celles qui ont été retenues dans la présente évaluation, soit l'option 7b (détournement, ouvrages et fortification [accent sur les ouvrages]), dont le coût est estimé à 2,6 M $. Les six (6) ouvrages de régulation dissiperont l'énergie du chenal, tandis que le détournement du chenal éloignera celui-ci des talus qui posent problème et facilitera les mesures de stabilisation des talus.
2. La Ville devrait continuer de surveiller les talus 6 et 7 jusqu'à ce que les travaux de construction soient terminés pour réunir des informations sur l'état de ces talus. Il est également recommandé de surveiller le ravin pour vérifier que l'érosion progressive des berges ne déstabilise pas les talus avant la réalisation des travaux recommandés de protection des talus sablonneux. Selon la note de Stantec sur le programme d'observation de la stabilité des talus de mars 2022 et l'observation du sol placé au 13, place Sprucewood, un contrefort en pierre temporaire de 0,5 m de hauteur au pied du talus situé derrière cette propriété assurerait la stabilité du talus.
3. Les travaux de stabilisation recommandés pour les quatre (4) talus qui posent problème comprennent le placement de matériaux de remblai argileux dans l'ancien chenal, en plus de l'enrochement. Le coût estimatif des matériaux de remblai est de 361 000 $.
4. Les talus et les berges du chenal devraient être revégétalisés après les travaux de construction avec des essences d'arbre indigènes, des arbustes et des plantes tapissantes. Les Services forestiers de la Ville devraient être consultés au moment de choisir les essences. Il est recommandé de planter des thuyas pour assurer la protection de la vie privée, car ils constitueront une barrière visuelle toute l'année.
5. Trois (3) voies d'accès d'entretien devraient être aménagées, pour assurer l'accès au site non seulement pendant la construction, mais aussi après les travaux si un entretien est requis à proximité du chenal et des ouvrages de régulation. Le coût estimatif de ces voies d'accès est de 26 000 $.
6. Il est recommandé de réduire au minimum le nombre d'arbres à enlever afin de perturber le moins possible l'habitat des espèces de chauves-souris qui pourraient être en péril. L'option privilégiée prévoit l'enlèvement de 77 arbres, et 40 autres arbres pourront vraisemblablement être préservés s'ils ne nuisent pas aux travaux.

7. L’utilisation des pratiques exemplaires d’aménagement à faible impact devrait être encouragée conjointement avec d’autres options si l’on veut réduire à long terme le ruissellement vers l’installation de gestion des eaux pluviales Blue Willow. Ces pratiques dans le domaine public pourraient être mises en œuvre au moment de reconstruire la chaussée. De même, des mesures d’aménagement à faible impact pourraient être introduites sur les propriétés privées : rediriger les tuyaux de descente pluviale vers les zones perméables, ou installer des pavés perméables plutôt que de l’asphalte dans les entrées de cour au moment de la reconstruction.
8. Au cours de l’étape de la conception détaillée, il y aurait lieu d’organiser des consultations avec les organismes, les groupes autochtones, les associations de quartier et les riverains afin de s’assurer qu’ils comprennent bien les travaux proposés et de répondre à leurs préoccupations.

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1.0 Introduction

1.1 Purpose/Overview

Wood Environment & Infrastructure Solutions, a Division of Wood Canada Limited (Wood) has been retained to conduct the Phase 1: Municipal Class Environmental Assessment (EA) and Functional Design for the Ruisseau Park Ravine (Blue Willow) Rehabilitation. The Class EA is being conducted by the City of Ottawa to address issues associated with channel widening and creek bank erosion and valley slope stability concerns along the Ruisseau Park Ravine, a tributary to Mud Creek (ref. Figure 1.1 Study Area Location Plan). Uncontrolled drainage from the urbanized area upstream of the SWM facility is conveyed to the Blue Willow SWM facility, through the Ruisseau Park Ravine, which has led to potential ravine slope stability issues on both public and private lands. The slope stability issues, and erosion concerns have arisen following the urbanization of the contributing drainage area during the 1990's.

The slope within Ruisseau Ravine, behind 9 Sprucewood Place, failed in 2017. Emergency rehabilitation works were undertaken to address the slope failure which included placing rock fill material on the slope to mitigate the potential for continued slope failure. The City of Ottawa initiated the Class EA to develop a comprehensive solution to protect private and public property from slope instability concerns, minimize long-term maintenance requirements, while minimizing impacts to the natural features within Ruisseau Ravine.

The Final Ruisseau Park Ravine (Blue Willow) Rehabilitation Class EA Report documents the background information reviewed for the study, establishes a baseline (existing condition) characterization of the study area pertaining to the hydrology (soils, land use, and topography), hydraulics and drainage features, stream morphology, geotechnical conditions, and the natural systems. Alternatives have been identified to mitigate erosion of the channel banks while addressing the slope stability concerns. The alternatives have been evaluated and a preferred alternative has been selected while an implementation plan has been provided to mitigate the areas of concern.

1.2 Study Area

The study area is located within an urbanized area of the Chapel Hill South neighbourhood, which is a suburb of the City of Ottawa, located 13 km (+/-) east from the downtown core. The study area is bound by Innes Road to the north, Mud Creek to the south, Pagé Road to the east, and the Federal Greenbelt to the west, with total drainage area of 106.30 ha (+/-) to the Mud Creek tributary (ref. Figure 1.1).

The north or upstream end of Ruisseau Park Ravine is located near the intersection of Orléans Boulevard at Creek Crossing Street/Longleaf Drive and the channel within the ravine is then aligned 450 m (+/-) southward toward the outlet structure and embankment. The ravine is bound on the west by residential properties at Blue Willow Crescent and Sprucewood Place and bound on the east by residential properties on Auburn Ridge Drive. Stormwater is conveyed to the ravine via four (4) storm sewer pipe outfalls; a 1200 mm and a 1800 mm storm sewer pipe outfall are located at the north end of the ravine, a 975 mm storm sewer pipe outfall is located on the west side near the downstream end, and a 900 mm storm sewer pipe outfall is located on the east side near the downstream end.

Figure 1.1. Chapel Hill Neighbourhood and Mud Creek Tributary Study Area Location Plan

1.3 Class Environmental Assessment Process

The Ontario Environmental Assessment Act provides for *"…the betterment of the people of the whole or any part of Ontario by providing for the protection, conservation and wise management in Ontario of the environment."* An approved Class Environmental Assessment (Class EA) document describes the process that a proponent must follow for a class or group of undertakings in order to satisfy the requirements of the Environmental Assessment Act (ref. Municipal Engineers Association (MEA) Class EA process (ref. Municipal Class Environmental Assessment, MEA 2015), and represents a method of obtaining an approval under the Environmental Assessment Act and provides an alternative to carrying out individual environmental assessments for each separate undertaking or project within the class.

The City of Ottawa Ruisseau Park Ravine (Blue Willow) Rehabilitation Class EA and Functional Design has been prepared in accordance with the Municipal Engineers Association (MEA) Class Environmental (Class EA) procedures. The Class EA is following the Municipal Class Environmental Assessment process (Phases 1-4 and Schedule B) which is outlined in the Municipal Engineers Association document (ref. Figure 1.2).

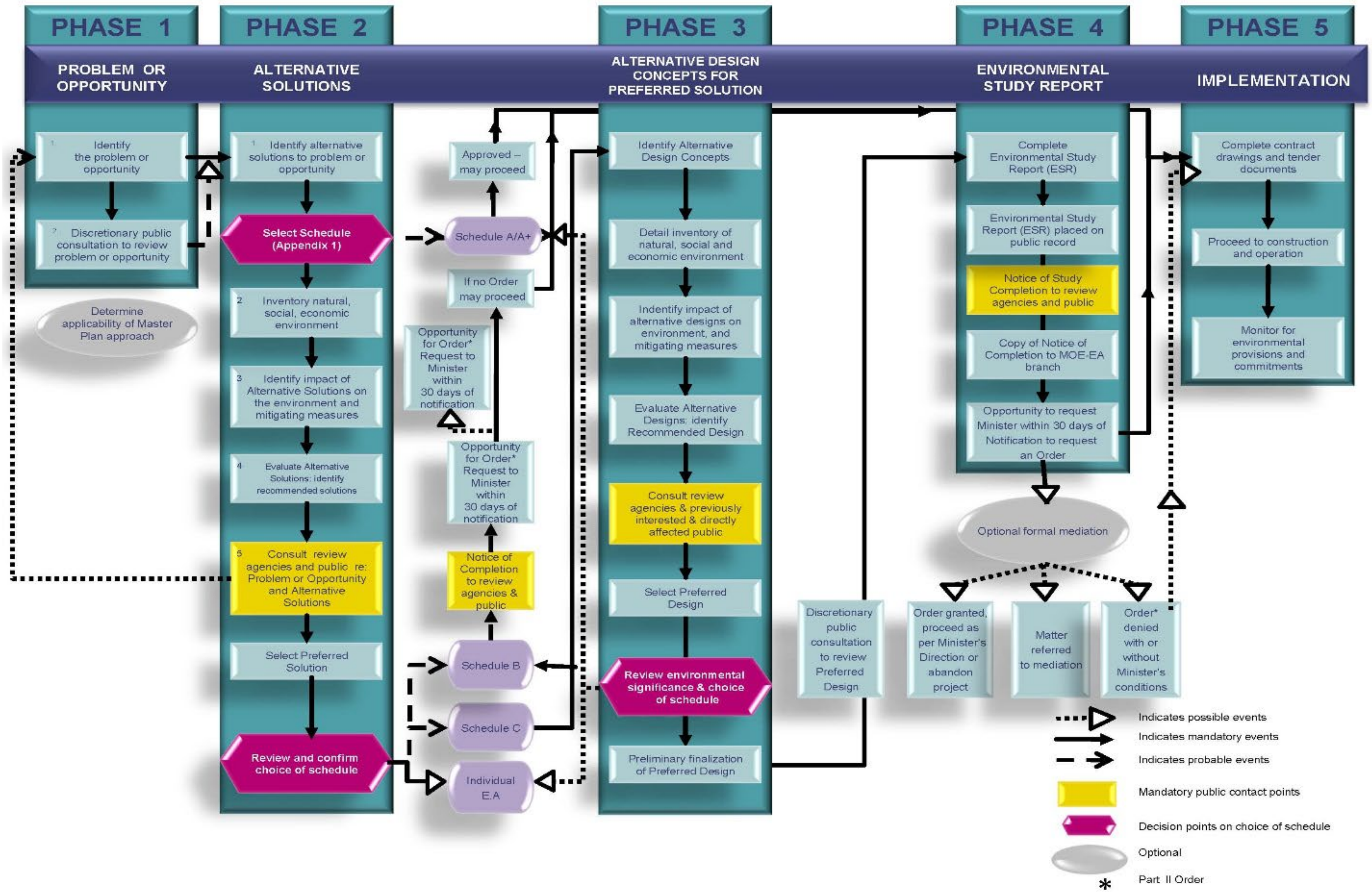

Figure 1.2. Municipal Class Environmental Assessment Process (ref. Municipal Class Environmental Assessment, MEA 2015)

1.4 Public/Agency Consultation

As noted, the Ruisseau Park Ravine (Blue Willow) Rehabilitation project is subject to the Class EA process; as such it has been conducted according to the requirements outlined in the governing MEA, Municipal Class EA process. The study approach has been established to meet the following objectives:

i. Protection of the environment, including natural, social and economic components of the environment.

ii. Participation of a broad range of stakeholders in the study process to allow for sharing of ideas, education, testing of creative solutions and developing alternatives.

iii. Documentation of the study process in compliance with all phases of the Municipal Class EA process.

The Municipal Class EA requires notification of, and consultation with, relevant stakeholders (ref. Appendix A for Public Consultation). The Project Team has ensured that stakeholders were notified early in the planning process, and throughout the study as per the following:

1.4.1 Notice of Study Commencement

The Notice of Study Commencement was mailed out to stakeholders, review agencies, First Nations, as well as local residents. The notice was published in the local newspaper on June 11 and June 25, 2020. A copy of the notice is included within Appendix A.

1.4.2 Public Meetings

The Project Team presented the project to the public and stakeholder during two (2) Public Information Centres:

- Public Information Centre No. 1, March 31, 2021
- Public Information Centre No. 2, March 30, 2022

Both the Public Information Centres were held virtually due to COVID restrictions of meeting in person. City staff met with Councillor Dudas on February 23, 2021 and March 11, 2022 to review the material to be presented at each Public Information Centre.

The first Public Information Centre (PIC#1) was presented virtually via Zoom by the Project Team and posted on the City's website for the review and commenting period until April 14, 2021; the content on the slides was provided in French and English. Copies of the notice were mailed out to stakeholders, review agencies, First Nations, as well as local residents. Twenty-one (21) comments were received prior to or during the virtual presentation with responses provided by the Project Team (ref. Appendix A) Questions from the public attending Public Information Centre No. 1, focused on the reasons for the Class EA, integration with other studies, examples of similar studies and creek works, and potential actions required by the public.

The second Public Information Centre (PIC#2) was presented virtually via Zoom by the Project Team and posted on the City's website for the review and commenting period until April 14, 2022; the content on the slides was provided in French and English. Copies of the notice were mailed out to stakeholders, review agencies, First Nations, as well as local residents. Thirty-one (31) comments were received prior to or during the virtual presentation with responses provided by the Project Team (ref. Appendix A). Questions from the public attending Public Information Centre No. 2, focused on potential construction impacts and mitigation measures, construction schedule, potential for the formation of a neighbourhood committee for input on proposed revegetation, communication during detailed design to residents and use of the pathway in the ravine.

2.0 Background Review

2.1 General

Background information has been provided by the City of Ottawa for this study. The information has consisted of reports, drawings, guidelines, Geographic Information Systems (GIS) mapping, and hydrologic and hydraulic modelling data. In addition to the provided information, Annis O'Sullivan Vollebekk Ltd. (AOV) as a subconsultant to Wood, has conducted topographic survey of the Ruisseau Park Ravine, completed as of October 21, 2020. A summary listing of the information used for this study has been provided in the following subsections.

2.1.1 Reports and Studies

- Mud Creek Cumulative Impact Study, City of Ottawa (Stantec Consulting Ltd. And JTB Environmental Systems, May 2020)
- Chapel Hill Park and Ride – Alternate Site Existing Conditions Report -Draft, Appendix A – Natural Environment Report, (Stantec, March 2016)
- Sewer Design Guidelines, Second Edition (City of Ottawa, October 2012)
- Slope Stability Guidelines for Development Applications in the City of Ottawa (Golder Associates, September 2004)
- Stormwater Management Facility Certificate of Approval, Minto Developments Inc. (Ministry of the Environment, March 2001).
- Storm Detention Pond, Certificate of Approval (Sewage), Minto Developments Inc. (Ministry of the Environment, June 1990).
- Stormwater Management Study, Chapel Hill South, Minto Construction Limited (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989).
- Report of Subsurface Investigation Proposed Residential Development South of Innes Road at Orleans Boulevard Gloucester, Ontario, for Minto Construction Limited (John D. Paterson & Associates Ltd., July 28, 1987).

2.1.2 Correspondence

- Temporary Slope Repair – 9 Sprucewood Pl. – Ruisseau Park, Meeting Notes (Stantec Consulting Ltd., May 12, 2017).
- Geotechnical Assessment Proposed Provisional Slope Stabilization #9 Spruce Wood Place, Ottawa, Ontario, Memorandum (Stantec Consulting Ltd., May 3, 2017).
- Chapel Hill Park and Rise Schedule B Environmental Assessment Study – Existing Conditions and Environmental Constraints Memorandum (Stantec Consulting Ltd., November 5, 2015).
- Slope Stability Analysis, Blue Willow SWM Facility, Blue Willow Crescent – Ottawa, letter addressed to Ms. Eva Spal, City of Ottawa (Paterson Group Inc., October 6, 2011).
- Comments Regarding Erosion Control, Chapel Hill South Stage IV – Phase 2, Gloucester, Ontario, letter addressed to Mr. Marcel Denomme, Minto Developments Incorporated (John D. Paterson & Associates Ltd., June 26, 1997).
- Review of Site Grading – Phase 2 and Swimming Pool Considerations, Chapel Hill South, Stage IV, Gloucester, Ontario, letter addressed to Mr. Marcel Denomme, Minto Developments Incorporated (John D. Paterson & Associates Ltd., June 24, 1997).

- Grade Raise Review, Chapel Hill South, Stage 4, letter addressed to Mr. Gord Elliot, City of Gloucester (John D. Paterson & Associates Ltd., May 9, 1994).
- Chapel Hill South Phase 4, Review of Engineering Drawings, letter addressed to Mr. Rodney Pitchers, The Corporation of the City of Gloucester (Minto Developments Inc., October 27, 1993).

2.1.3 Drawings

- Grading Plan, General Plan of Services, As-Built Drawings, Chapel Hill Stage IX, Minto Developments Inc., City of Gloucester (Atrel Engineering Ltd., August 2000).
- As Built Plan, Du Ruisseau Park (Recreation and Culture, March 1996).
- Storm Outlet, Chapel Hill South, City of Gloucester (Cumming Cockburn Limited Consulting Engineers and Planners, September 1989).

2.1.4 GIS and Mapping Data

The City of Ottawa has provided 105 GIS data sets for use in this study area comprised of shapefiles, two (2) digital elevation models (DEM), and twelve (12) aerial imagery tiles ranging from 1976-2019. A comprehensive list of the available GIS data is provided in Appendix B. Some of the more relevant shapefiles pertaining to this study are provided below:

- Storm sewer
- Manholes/Catch Basins
- Catch basin leads
- Imperviousness
- OMAFRA soils data
- Land Use Mapping
- Contour Data
- Property Parcels

2.1.5 Topographic Survey Data

AOV has conducted two (2) rounds of topographic survey within the Ruisseau Park Ravine, in early May 2020 and late October 2020. The topographic survey has included the low flow channel (thalweg and banks), slope stability areas of concern, toe of slope, top of slope, pathways, fence lines, stormwater management facility, geotechnical boreholes (in public property) and other items in the ravine relevant to the Municipal Class EA and required discipline assessments.

2.1.6 Modelling

A PCSWMM hydrologic model that was prepared to assess the Mud Creek existing conditions, has been providing a base model for the hydrologic assessment of the Ruisseau Park Ravine.

- PCSWMM Model - Mud Creek Existing Conditions Revised 09 100 Year 12 Hour SCS (Stantec, April 2018)

3.0 Existing Conditions Assessment

3.1 Baseline Hydrology

3.1.1 Overall Drainage Areas

A review of the contributing drainage area to the Ruisseau Park Ravine has been undertaken to develop an overall drainage area boundary plan (ref. Figure 3.1). The drainage area boundaries have been developed through a review of the topographic data provided by the City of Ottawa, in addition to the Stormwater Management Study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989), aerial imagery provided by the City, and Google™ Maps (Street View). Five (5) contributing drainage areas have been identified within the study area which are based on the contributing areas to the four (4) storm sewer outfalls to the Blue Willow SWM facility, in addition to the Ruisseau Park Ravine area, which contributes runoff directly to the Blue Willow SWM facility. The size of the drainage areas, as well as a short description of each area's outlet, is provided in **Table 3.1**.

Table 3.1. Drainage Area Summary

Drainage Area ID	Drainage Area (ha)	Description
100	4.16	Runoff from the Ruisseau Park Ravine and the rear yards of the adjacent private residential properties directly contributes to the Blue Willow SWM facility.
200	9.75	The drainage area west of the SWM facility discharges via a 975 mm diameter storm sewer through a pedestrian path at Blue Willow Crescent and Des Pruches Avenue toward the downstream end of the SWM facility. Major system flow would also be conveyed through the pedestrian path.
300	15.70	The drainage area east of the SWM facility discharges via a 900 mm diameter storm sewer through a pedestrian path at Auburn Ridge Drive toward the downstream end of the SWM facility. Major system flow would also be conveyed through the pedestrian path. There is a major-minor system split at the south end of Pagé Road, where major system flow is conveyed to Mud Creek.
400	8.17	This drainage area is north of drainage area 200 and discharges to the north end of Ruisseau Park Ravine via a 1200 mm diameter storm sewer, 25 m (+/-) south of the Orleans Boulevard and Creek Crossing Street/Longleaf Drive Intersection. Major system flow would be conveyed to Ruisseau Park Ravine via a sag at Orleans Boulevard in the east boulevard.

Drainage Area ID	Drainage Area (ha)	Description
500	68.51	This drainage area is north of Creek Crossing Street/Longleaf Drive and discharges to the north of Ruisseau Park Ravine via an 1800 mm diameter storm sewer on the south east corner of the Orleans Boulevard and Creek Crossing Street/Longleaf Drive Intersection. The major system is conveyed to the sag in the road and boulevard on Orleans Boulevard.
Total Area	**106.30**	

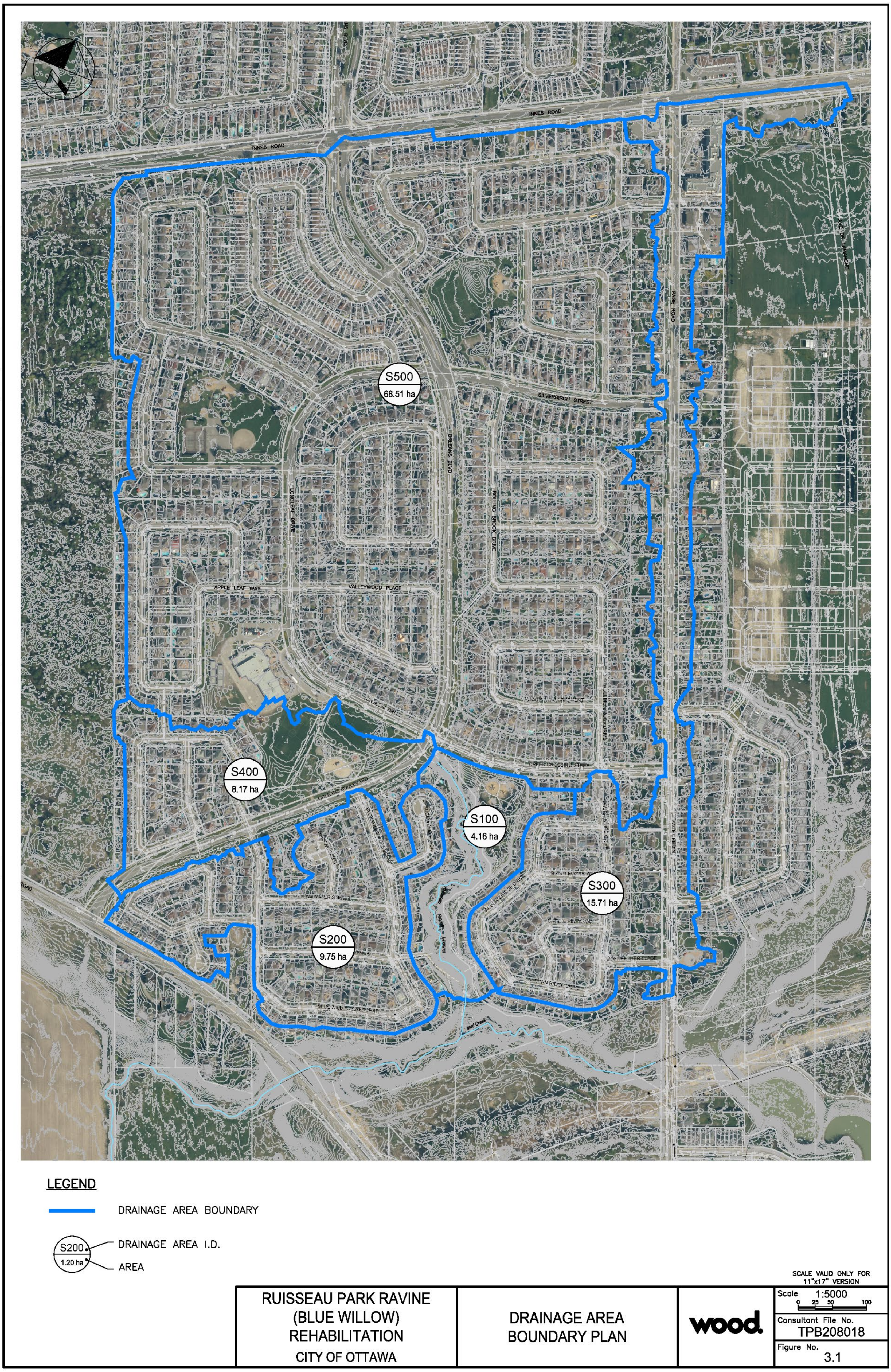

Figure 3.1. Drainage Area Boundary Plan

As shown in Table 3.1, the total contributing drainage area to the Blue Willow SWM facility is 106.30 ha with four (4) of the drainage areas contributing via storm sewers and major system conveyance, while the fifth drainage area directly contributes runoff to the Blue Willow SWM facility. The outfall locations are shown in Figure 3.2

Figure 3.2. Outfall Location Plan

3.2 Hydrologic Modelling

As per the approved Work Plan, hydrologic analyses for the Ruisseau Park Ravine have been completed using the PCSWMM integrated hydraulic/hydrologic modelling software. The PCSWMM model uses the EPA-approved SWM engine as the central analytical platform for hydrologic and hydraulic analyses and includes a GIS-based pre-processor and post-processor to facilitate hydrologic model development and analysis of results. The EPA SWMM analytical methodology is fully supported and maintained by the US EPA. PCSWMM can be used to effectively consider aspects such as infiltration, impervious coverage, and storm sewer and roadway conveyance/storage.

3.2.1 Previous Modelling

An existing conditions PCSWMM hydrologic/hydraulic model was prepared for the 367 ha (+/-) contributing area to the 2.7 m (+/-) span Mud Creek tributary culvert at Navan Road (ref. Figure 1.1). The PCSWMM model was developed by Stantec (April 2018) in support of the City of Ottawa's Municipal Class Environmental Assessment (Class EA) for the Mud Creek Cumulative Impact Study (CIS); based on that study the contributing drainage area to the Ruisseau Park Ravine was estimated as 111 ha (+/-). The subject PCSWMM model is reflective of a high-level lumped hydrologic model with combined drainage areas conveyed to defined drainage outlets (flow nodes) and does not contain storm sewer infrastructure, major system (road) conveyance routes; only one (1) catchment was defined for the contributing drainage area to the Ruisseau Park Ravine while other lumped catchments were defined for the drainage areas contributing to the Mud Creek tributary A storage node with a stage-storage rating curve was incorporated to represent the Ruisseau Park Ravine, while an outlet with a stage-outflow rating curve and a weir were used to represent the Blue Willow online stormwater management (SWM) facility outlet to Mud Creek. These elements do not reflect the Ruisseau Park Ravine channel hydraulics.

As part of the review of the PCSWMM model, it was observed that the coordinate system was set to the NAD83 UTM 9N coordinate system, while the City of Ottawa GIS data were saved to the NAD83 MTM Zone 9 coordinate system. The NAD83 UTM 9N coordinate system is 400 km west of Washington State, as such the supporting PCSWMM model shapefiles have been converted to the NAD83 MTM Zone 9 coordinate system.

The Horton infiltration methodology was applied to the subcatchments in the PCSWMM model with a minimum infiltration rate of 13.2 mm/hr., which is reflective of a loam soil based on the saturated hydraulic conductivity. The imperviousness of the contributing drainage area to the Ruisseau Park Ravine was established as 42.1 % (+/-) which is reflective of a low density residential urbanized area.

The PCSWMM model received from the City has been used to develop the existing conditions PCSWMM model in support of this study.

3.2.2 Subcatchment Delineation

PCSWMM's automated watershed delineation tool has been used to establish initial subcatchment boundaries based on the DEM received from the City. The boundaries have been reviewed and refined from aerial imagery and Google Street View™ to ensure the boundaries are reasonable. Where necessary, the boundaries have been revised to reflect anticipated drainage patterns for residential lots with split drainage; the refined subcatchment boundaries are provided in Figure 3.3. The limits of the external drainage boundaries have been revised based on the adjustments to the study area subcatchments; these adjustments are primarily reflective of residential split drainage patterns and the removal of the area on Pagé Road north of Innes Road. The total drainage area of the 120 discretized subcatchments is 106.30 ha (+/-) with an average area of 0.89 ha (+/-).

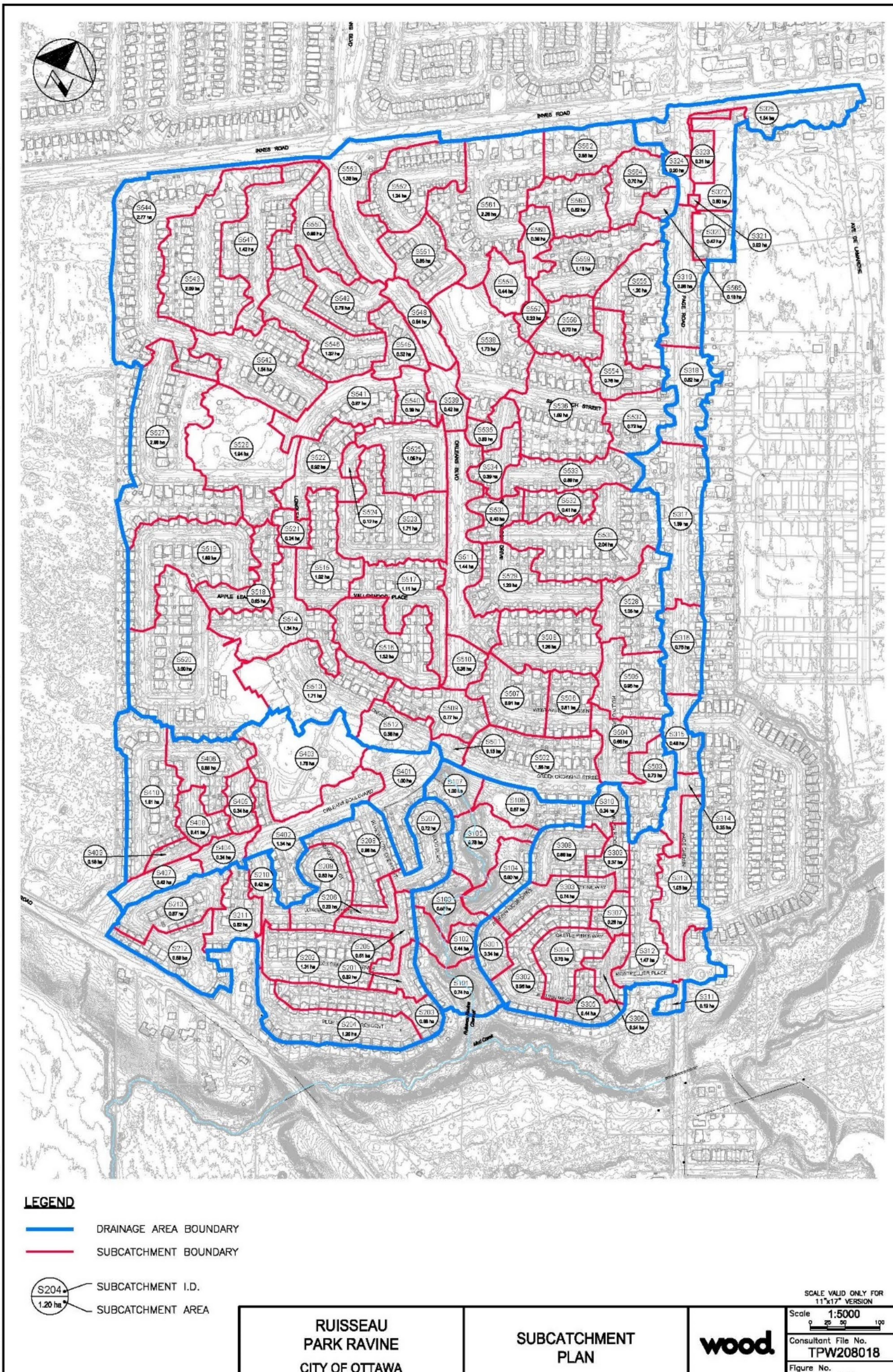

Figure 3.3. Subcatchment Plan

3.2.3 Rainfall Abstractions

A review of the Minto Construction Limited soils report (John D. Paterson & Associates Ltd., July 28, 1987) for the study area has indicated that the surficial soils consist of topsoil overlaying fine silty sand and a deep deposit of marine clay; the silty sand layer was noted as being *thicker in the southern portion of the study area*.

A review of the Ontario Ministry of Agriculture, Food and Rural Affairs (OMAFRA) surficial soils GIS mapping data provided by the City of Ottawa, has indicated four (4) known soil types within the study area and two (2) unassigned or undocumented soil types (ref. Figure 3.4); Bearbrook to the northwest of the study area, Farmington to the northeast, Allendale to the central, and Carlsbad to the south of the study area. The first undocumented soil is located on Pagé Road from Innes Road to Mud Creek, while the second is identified as Eroded Channel, located within Ruisseau Park Ravine.

The composition of Bearbrook soils is primarily clay and poorly drained, Farmington soils is medium to moderately fine loam and well drained, Allendale soils is sand and clay and poorly drained, and Carlsbad soils is primarily sand and well drained. These soils are located on both sides of the Pagé Road undocumented soils and have been applied to the Page Road undocumented soils where applicable in lieu of more recent surficial soil data. Bore holes have been advanced within Ruisseau Park Ravine in support of this study; the bore hole logs indicate that the soils within the channel area are primarily clay and soil parameters have been applied accordingly. The distribution of each soil type within the study area is provided in **Table 3.2**.

Table 3.2. Study Area Soil Composition

Soil Type	OMAFRA Soil Assigned/Not Assigned	Area (ha)	Total Area (ha)	Total Area (%)
Allendale	Assigned	31.37	35.04	33.0
Allendale	Not Assigned	3.67	35.04	33.0
Bearbrook	Assigned	30.81	40.37	38.0
Bearbrook	Not Assigned	9.55	40.37	38.0
Carlsbad	Assigned	26.34	26.34	24.8
Farmington	Assigned	0.38	2.16	2.2
Farmington	Not Assigned	1.78	2.16	2.2
Eroded Channel	Not Assigned	2.39	2.36	2.0
Total			**106.30**	**100.0**

As shown in Table 3.2, the soil distribution in the study area is primarily comprised of the Allendale, Bearbrook, and Carlsbad soils. The Farmington and Eroded Channel soils represent a minor portion of the soils within the overall study area. An area weighting approach has been used to determine the proportion of each soil type within the study area.

Consistent with the City's Mud Creek Cumulative Impact Study (Stantec Consulting Ltd. And JTB Environmental Systems, May 2020), the Horton infiltration methodology has been applied for the PCSWMM model infiltration parameters within the study area. The Horton infiltration methodology employs five (5) user input parameters to simulate the infiltrative capacity of the surficial soil conditions, as identified in **Table 3.3**.

Table 3.3. Horton Infiltration Parameter Summary

Parameter	Description	Units
Maximum Infiltration Rate	Maximum infiltration rate of a dry soil on the Horton infiltration rate curve	mm/hr
Minimum Infiltration Rate	The minimum infiltration rate on the Horton infiltration rate curve and is equivalent to the soils saturated hydraulic conductivity	mm/hr
Decay Constant	Infiltration rate decay constant for the Horton infiltration rate curve	Unitless
Drying Time	The time required for a fully saturated soil to completely dry	Days
Max Infiltration Volume	Maximum infiltration volume possible, this parameter is optional for the PCSWMM model simulation and is not applicable	mm

The values applied for these parameters have been selected based on the soil classification as identified from available OMAFRA surficial soils mapping (ref. Figure 3.4). The corresponding Horton soil parameters sourced from the User's Guide to SWMM5, 13th Edition (W. James, L.E. Rossman, and W.R.C. James, November 2010) provided in Table 3.4, have been applied to the soils within the study area. Area weighting has been used for each parameter where multiple soil classification types were located within one subcatchment.

Table 3.4. Horton Infiltration Parameters

Name	Soil Type (OMAFRA Soil Maps)	Max Infiltration Rate (mm/hr)	Min Infiltration Rate (mm/hr)	Drying Time (Days)
Allendale	Sandy Loam	76.2	10.92	4
Bearbrook	Clay	25.4	0.25	5
Carlsbad	Loamy Sand	127	29.97	3
Farmington	Loam	76.2	3.30	5
Eroded Channel	Clay	25.4	0.25	5

It is indicated in the User's Guide to SWMM5, 13th Edition (W. James, L.E. Rossman, and W.R.C. James, November 2010) that the decay constant should be confirmed with field data; however, if field data are not available to confirm this parameter, then a value of 4.14 can be applied. This suggests that the infiltration rate of the soil will decrease by 98 % during the first hour of precipitation.

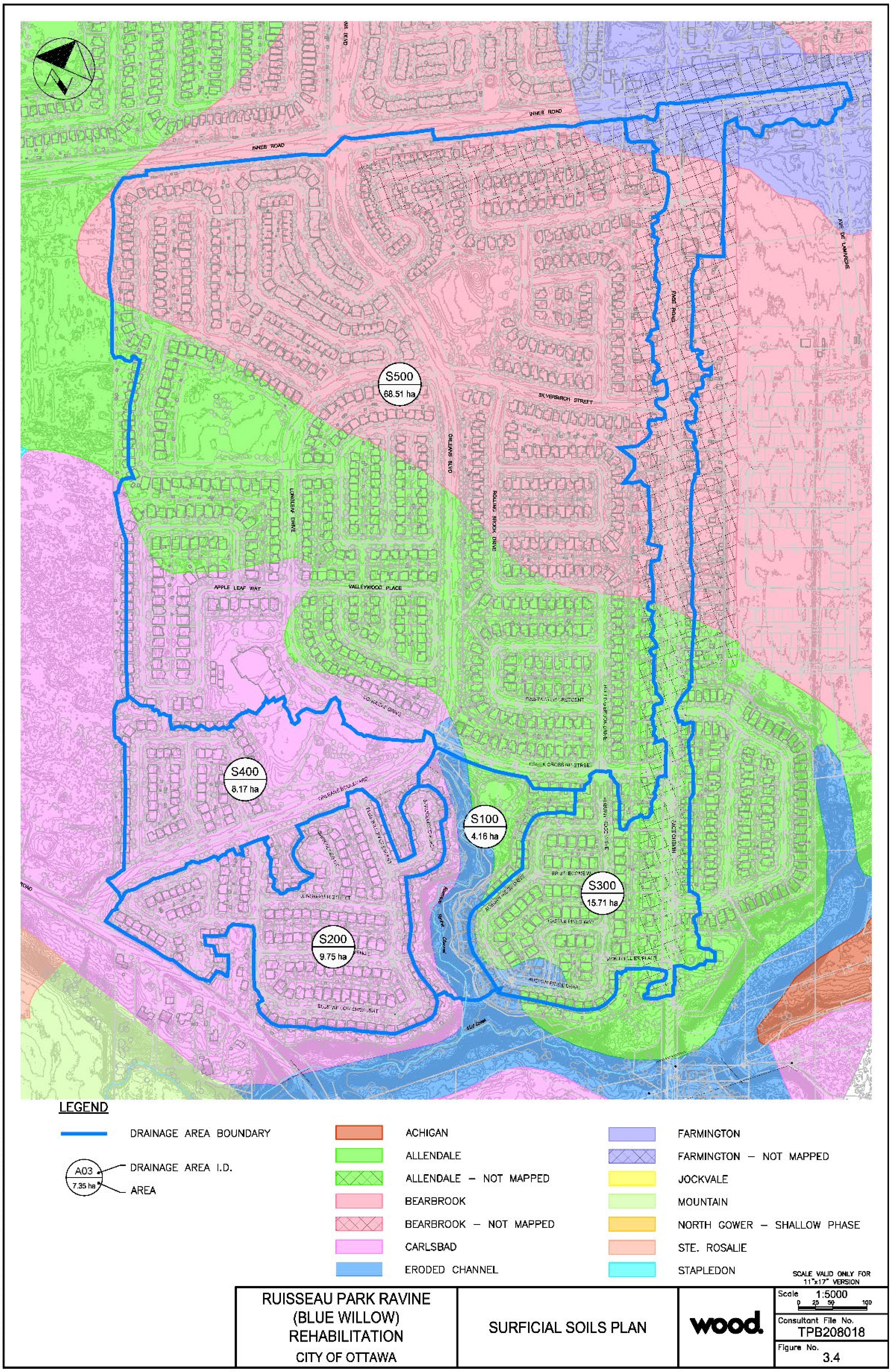

Figure 3.4. Surficial Soils Plan

3.2.4 Land Use Cover

As noted earlier, the land use within the study area is primarily low density residential (Zones R1 and R3) with parks and open space, minor institutional (school), and local commercial. Wood's typical approach for determining the impervious coverage of a large study area is to assign imperviousness rates to specific land use zones based on sample measurements. However, the City has provided a GIS polygon shapefile that identifies the impervious surfaces within the study area which includes buildings, driveways, sidewalks, roads, and parking lots. This shapefile has been used to areally weight the imperviousness for the subcatchments within the study area. Upon review of the shapefile, it has been observed that the shapefile did not identify amenity areas such as patios, walkways, and pools. From Wood's experience, amenity areas should be considered in established neighbourhoods as they can significantly increase the imperviousness, which is often not accounted for in the original stormwater management design of the contributing drainage area. Pools have been included in the residential amenity area features as hard covers are common on the tops of pools and would contribute to the runoff generated within the amenity areas.

Impervious samples have been obtained for five (5) representative subcatchments within the study area. One (1) subcatchment has been selected for each of the drainage areas except for drainage area 500, where two (2) subcatchments have been selected as there are two (2) residential zones, R1 and R3, within this drainage area. The measurements of the sampled amenity areas and the comparison between the initial and the revised areally weighted imperviousness is provided in **Table 3.5**.

Table 3.5. Subcatchment Amenity Area Samples and Comparison.

Subcatchment (Residential Zone)	Area (ha)	Initial Imperv. Area (ha)	Initial Subcatch. Imperv. (%)	Amenity Imperv. Area (ha)	Amenity Area Imperv (%)	Revised Subcatch. Imperv. (%)	Differ. (%)
S202 (R1)	1.311	0.669	51.04	0.124	19.32	60.49	9.46
S312 (R1)	1.474	0.491	33.28	0.107	10.87	40.53	7.25
S406 (R1)	0.862	0.471	54.67	0.055	14.08	61.05	6.38
S563 (R1)	0.823	0.340	41.27	0.069	14.20	49.61	8.34
S550 (R3)	0.990	0.505	51.00	0.012	2.53	52.24	1.24

The addition of the measured amenity areas to the impervious area demonstrates an increase in the subcatchment imperviousness ranging from 1.24 % (+/-) to 9.46 % (+/-) as shown in Table 3.5. The amenity area imperviousness has been applied for the residential areas within the designated drainage areas in the PCSWMM model. The imperviousness has been areally weighted to include the amenity area imperviousness and the impervious areas identified in the City's shapefile.

The total impervious area of the study area is 55.09 ha (+/-) resulting in an imperviousness of 51.83 % (+/-). The total drainage area and the imperviousness from the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) have been compared to the existing conditions study (ref. Table 3.6).

Table 3.6. Drainage Area and Imperviousness Comparison between the Existing Condition and the Chapel Hill SWM Study (1989)

Scenario	Drainage Area (ha)	Impervious Area (ha)	Imperviousness (%)
SWM Study (1989)	73.18	26.09	35.66
Existing Conditions	106.30	55.09	51.83
Difference	33.12	28.99	16.17

As noted within Table 3.6, there is an increase in both the drainage area and the imperviousness of the study area contributing to Ruisseau Park Ravine and the Blue Willow SWMF, since the initial drainage report was submitted in 1989. The Chapel Hill Stormwater Management Report (SWM) (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) indicated the proposed conditions contributing drainage area to the Ruisseau Ravine (Blue Willow SWM Facility) as 73.18 ha (ref. Table 1 within the Chapel Hill SWM Report within Appendix B). The Chapel Hill South SWM Report also indicated the pre-development conditions contributing drainage area to the Ruisseau Ravine as 86.1 ha (ref. Figure 5 and Table 2 within the Chapel Hill SWM Report within Appendix B).

While the pre-development drainage area plan excerpt shown within the Chapel Hill South SWM Report does not depict the proposed buildout, it is noted that the pre-development drainage areas do not extend to the site boundary to the west of the site, does not include the west side of Pagé Road, and does not include the contributing area from Innes Road. The existing conditions drainage area delineated for the current study has included the noted areas. The excluded areas at the west of the site, Pagé Road, and Innes Road largely represent the 33 ha (+/-) difference between the drainage areas delineated for the Chapel Hill South SWM Report and the Ruisseau Park Ravine (Blue Willow) Class EA. The pre-development drainage areas cannot be accurately measured from the scanned imaged within the Chapel Hill South SWM Report to verify the sub-catchment areas in comparison to the existing conditions drainage area. Similarly, the proposed condition drainage areas were not provided in a drainage area plan within the Chapel Hill South SWM Report.

3.2.5 Other Hydrologic Parameters

Other parameters relevant to the integrated hydrologic modelling include overland flow length, watershed slope, Manning's Roughness Coefficients for overland flow, and depression storage.

In the PCSWMM (and EPA-SWMM) methodology, overland flow length is applied to represent internal routing within the subcatchment which affects the time of concentration. Based on Wood's previous experience, for resolute subcatchment sizes (average drainage area of 0.89 ha +/- for the current study), simulated peak flow is much less sensitive to variations in this parameter as compared to other model parameters. Given the small subcatchment areas, the overland flow length has been directly measured as the sheet flow length (i.e. back of the property line to the roadway). The overland flow length has been rounded to the nearest 5 m interval. In addition, subcatchments of a similar size and shape have applied the same flow length.

A typically constructed lot slope for residential subcatchments of 2% has been applied for subcatchments within the study area as a default value. Slopes have been revised however in identified steep drainage areas, such as within Ruisseau Park Ravine.

From Wood's experience, simulated peak flow and runoff are generally insensitive to changes in the other noted hydrologic parameters (Manning's Roughness Coefficients and Depression Storage). For the purposes of base model development typical parameters have been applied. The initial parameter values are shown in **Table 3.7**.

Table 3.7. PCSWMM Subcatchment Hydrologic Parameters

Subcatchment Parameter	Value
Flow Length (m)	As Measured
Slope (%)	2% or as Measured (steep areas)
Manning's Roughness - Impervious	0.013
Manning's Roughness - Pervious	0.25
Depression Storage - Impervious (mm)	1
Depression Storage - Pervious (mm)	5
Subarea Routing (%)	35

Sub-area routing is defined as the percentage of the modelled impervious land segment which is routed across the pervious land segment, as noted in Section 3.2.3. An estimated value of 35 % has been assumed in this case based on Wood's experience with other modelling studies. This approach has been taken as the runoff from residential structures within the study area can be conveyed across a front or rear yard lawn prior to conveyance to the storm sewer system through a catch basin.

3.3 PCSWMM Hydraulic Modelling

3.3.1 Storm Sewers

The storm sewers and manholes have been parameterized as per the GIS shapefiles received from the City for the study area to convey runoff from the subcatchments to the Blue Willow SWMF. The pipe geometry, pipe diameter, and invert elevations have been provided in the GIS shapefiles; however, these data have not been validated. As-built drawings have not been provided for the storm sewer system.

The PCSWMM model has been developed as a dual drainage model; runoff generated from each subcatchment is initially conveyed to the major system and routed to the minor system through catch basins represented as orifices. The orifices' dimensions within the PCSWMM model have been determined to be equivalent to the sum of the open area of the inlet elements; the number of catch basins between manholes have been counted and equivalent size opening has been determined based upon assuming a uniform size for each catch basin. A value of 0.013 has been selected for Manning's coefficient for all the sewer pipes within the focus area which is commonly used for concrete pipe.

3.3.2 Road-way Elements and Spill Conditions

The major system or road-way components within the PCSWMM model have been established based on a typical two-lane road right-of-way (ROW) cross-section within the study area. The ROW cross-section is representative of an urban curb and gutter cross-section measured from aerial imagery. The major systems and minor systems generally have coincident locations and have largely been modelled as such. However, select locations have been identified with major-minor splits where a major system is conveyed in a direction that is not coincident with the minor system and has been modelled accordingly where appropriate. Furthermore, sags or *sawtoothing* in the ROW have been input into the model as per the provided manhole rim elevation data. This is consistent with the design of the study area as noted in the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989). This type of a major system typically provides attenuation of major system runoff at the *sawtoothing* locations and would result in peak flow rates with greater accuracy than those generated from the Mud Creek Cumulative Impact Stud (Stantec Consulting Ltd. And JTB Environmental Systems, May 2020).

Spill conditions have also been incorporated into the model to simulate low points in the ROW where water would be conveyed off the ROW and have been represented by either a rectangular open conduit or a transect obtained from the City's DEM. The rectangular open conduits typically represent a location where runoff is conveyed across a flat surface between identified manhole locations where a defined road is not shown. Transects have been obtained from the City's DEM for spill elements from the ROW into the Ruisseau Park Ravine at defined low points in the ROW with a curb depression.

3.3.3 Stormwater Management Facility

The Blue Willow SWMF is a dry quantity control facility that incorporates the entirety of Ruisseau Park Ravine and is a tributary to Mud Creek. The SWMF discharge is controlled by an outlet structure at the south, downstream, end of the facility; the outlet structure consists of an 18.5 m (+/-) long, 375 mm pipe, with an inlet grate at the base of the SWMF, connected by a manhole to a 44.0 m (+/-) long 975 mm pipe which discharges via an outfall structure to Mud Creek. Two (2) relief inlets are located at the top of the manhole near the top of the embankment, one (1) 450 mm side inlet and one (1) 1 m x 1 m bottom draw inlet which provide emergency discharge to the 975 mm pipe. The maximum depth within the SWMF prior to spilling over the embankment is 6.26 m (+/-).

A stage-storage rating curve and a stage discharge rating curve represent the SWMF in the City's PCSWMM model for the City's Mud Creek Cumulative Impact Study (Stantec Consulting Ltd. And JTB Environmental Systems, May 2020). The City's DEM has been used to create a stage storage rating curve to validate the received stage storage rating curve; both rating curves display a similar shape and result in approximately the same maximum storage area as simulated for the 2-100 year design storm events (ref. Appendix C); the differences in storage volume ranges from 0.86 % to 6.8 % for the 2-100 year design storm events. The similarities would suggest that the received rating curve is acceptable for use within the PCSWMM model.

The outlet rating curve from the City's PCSWMM model has been applied as per the received data. However, an outlet weir that was also input into the received model to simulate spill over the top of the embankment has been revised; the spill elevation has been updated to reflect the surveyed top of the embankment of 82.86 m (+/-), a decrease from the received weir elevation of 83 m. The Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) indicates that the spill elevation for the SWMF is 83.0 m which is the likely source of the received weir spill elevation.

Validation of the received outlet rating curve has been undertaken by inputting the available survey data for the outlet infrastructure into the PCSWMM model in place of the outlet rating curve. Not all the required outlet infrastructure components have been surveyed as some were inaccessible, such as the inverts of the storm sewer pipes within the outlet structure manhole due to the depth of the manhole. Notwithstanding, assumptions have been made regarding selected inverts and the available opening area on the three (3) inlets; the inlets are covered with grates that restrict the available opening area. Additionally, the received outlet rating curve has 36 ordinates at 0.2 m stage intervals, making it a highly discretized rating curve. Furthermore, the method in which the received outlet rating curve was generated for the PCSWMM model was not provided.

Despite these issues, the peak outflow results for the 2-100 year storm events using the available surveyed infrastructure are reasonably comparable to the peak outflow results using the received rating curve. The greatest discrepancies are for the 2-10 year storm event peak flow rates with differences ranging from -22.36 % to +27.51 % (-0.24 m^3/s to +0.39 m^3/s) while the 25-100 year peak flow rates offer a more suitable comparison differing from -1.55 % to -0.71 % (-0.10 m^3/s to -0.05 m^3/s). The simulated difference in the frequent storm events is due to the uncertainty regarding the inverts and opening area of the relief inlets as they become activated during the 5 and 10 year design storm events. The similarities of the outlet flow rates for the less frequent storm events is due to the consistent spill elevation from the SWMF. It is worth noting that the 2 and 5 year discharge rates of 0.62 m^3/s and 0.77 m^3/s from the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) are both exceeded when compared to the received rating curve and the available infrastructure peak flow rates.

Regardless of the differences for the frequent storm event outlet peak flow rates, the less frequent storm event outlet peak flow rates are comparable suggesting that the received rating curve is acceptable for the use in the PCSWMM. The comparison of the peak outflow rates is provided in Appendix C.

3.3.4 Precipitation

Consistent with the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989), the PCSWMM model has been simulated with the City's 3-hour Chicago design storm events. However, Chapel Hill South SWM study only simulated the 2 year, 5 year, and 100 year design storm events, while this study has simulated the 2 year, 5 year, 10 year, 25 year, 50 year, and the 100 year design storm events. The City's 3-hourChicago storm events have been used to assess the peak flow rates contributing to Ruisseau Ravine and the Blue Willow SWM facility as a comparison to the peak flow rates reported within previously completed studies for the site.

In addition to the City's 3 hour Chicago storm events, the PCSWMM has also been simulated with the City's 2 to 100 year 12 hour SCS storm events for the generation of the simulated water surface elevations for floodline mapping assessment. While the City of Ottawa Sewer Design Guidelines specify that the Chicago storm is generally applied for urbanized areas, such as the contributing area to Ruisseau Ravine, the City has requested that the 12 hour SCS storm events be used for the floodline mapping exercise.

As per the approved work plan for the study, continuous simulation modelling has been executed in addition to the event-based (Design Storm) modelling. This approach typically yields greater accuracy while allowing for the assessment of potential erosion impacts, based on the erosion threshold targets. The City of Ottawa has provided an hourly semi-continuous precipitation dataset from 1967 to 2007 obtained from the MacDonald-Cartier International Airport; the dataset is not a true continuous dataset as it has excluded the years of 2001 and 2005 and does not contain winter precipitation data from December to April of each year. This dataset has been used for this assessment and is consistent with the semi-continuous precipitation dataset applied for the City's Mud Creek Cumulative Impact Study (Stantec Consulting Ltd. And JTB Environmental Systems, May 2020). The precipitation data have not been supplemented with additional data nor have climatology factors such as evaporation or temperature been applied for the data as the semi-continuous simulation model has been used to assess potential erosion impacts and not provide a water budget.

3.4 PCSWMM Hydrologic/Hydraulic Model Baseline Results

3.4.1 Peak Flow Rates

The PCSWMM hydrologic/hydraulic model has simulated the City's 2-100 year 3 hour Chicago design storm events in addition to the 15 mm and 25 mm 4 hour Chicago storm events to determine the peak flow rates discharging to the Ruisseau Park Ravine from the contributing drainage area. The results are presented in **Table 3.8**.

Table 3.8. Design Storm Peak Inflow Rates to the Blue Willow SWMF

Drainage Area I.D.	Model Element	Flow Location	Peak Flow Rate (m^3/s) 15 mm	Peak Flow Rate (m^3/s) 25 mm	Peak Flow Rate (m^3/s) 2 Year	Peak Flow Rate (m^3/s) 5 Year	Peak Flow Rate (m^3/s) 10 Year	Peak Flow Rate (m^3/s) 25 Year	Peak Flow Rate (m^3/s) 50 Year	Peak Flow Rate (m^3/s) 100 Year
500	Storm Sewer	1800 mm	1.63	3.12	4.53	7.34	8.30	8.88	9.28	9.65
400	Storm Sewer	1200 mm	0.16	0.28	0.41	0.59	0.73	1.03	1.20	1.38
400/500	Major System	1200 mm ROW Spill	0.00	0.01	0.02	0.06	0.19	0.39	0.68	1.12
300	Storm Sewer	900 mm	0.36	0.78	1.16	1.90	2.18	2.32	2.54	2.57
300	Major System	900 mm ROW Spill	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
200	Storm Sewer	975 mm	0.32	0.56	0.74	1.10	1.38	1.76	2.09	2.36
200	Major System	975 mm ROW Spill	0.00	0.00	0.00	0.00	0.01	0.02	0.03	0.04
100	Subcat.	S101 - S107	0.00	0.02	0.12	0.36	0.56	0.84	1.04	1.26
Total			**2.47**	**4.77**	**6.70**	**11.31**	**13.19**	**14.97**	**16.32**	**17.64**

As shown in Table 3.8, the 2–100 year total peak flow rates conveyed to the SWMF from the contributing drainage area range from 6.70 m^3/s to 17.64 m^3/s. The individual peak flow rates do not sum to the total provided in the table due to the timing of the individual inflow hydrographs. These peak flow rates have been used for the validation of the model.

3.4.2 SWMF Inflow Validation

A unitary peak flow analysis has been undertaken to validate the peak inflow results contributing to the SWMF obtained from the PCSWMM hydrologic/hydraulic model (ref. **Table 3.9**).

Table 3.9. Unitary Peak Flow Rate for the 100 Year Design Storm Event

Peak Flow Rate (m^3/s)	Contributing Drainage Area (ha)	Unitary Flow Rate ($m^3/s/ha$)
17.64	106.30	0.166

The 100 year storm event peak flow rate contributing to the Ruisseau Park Ravine has been used for a unitary peak flow analysis in comparison to unitary flow results obtained from calibrated models for other hydrologic assessments within southern Ontario conducted by Wood (ref. Figure 3.5). The results indicate that the Ruisseau Park Ravine unitary flows tend to be higher in comparison to the previously calibrated data from the alternate projects. It has been noted that the unitary flows obtained for the calibrated comparison models are primarily from areas with lower imperviousness than the contributing drainage area to the Ruisseau Park Ravine (55.09 % imperviousness).

Additionally, the comparable models have been simulated with alternative infiltration methodologies such as Green and Ampt or SCS Curve Number, with variable soil conditions ranging from highly infiltrative to poorly infiltrative. Notwithstanding, the unitary flow rate contributing to Ruisseau Park Ravine is within the range of the comparable unitary flow rates and would be considered appropriate for the assessment.

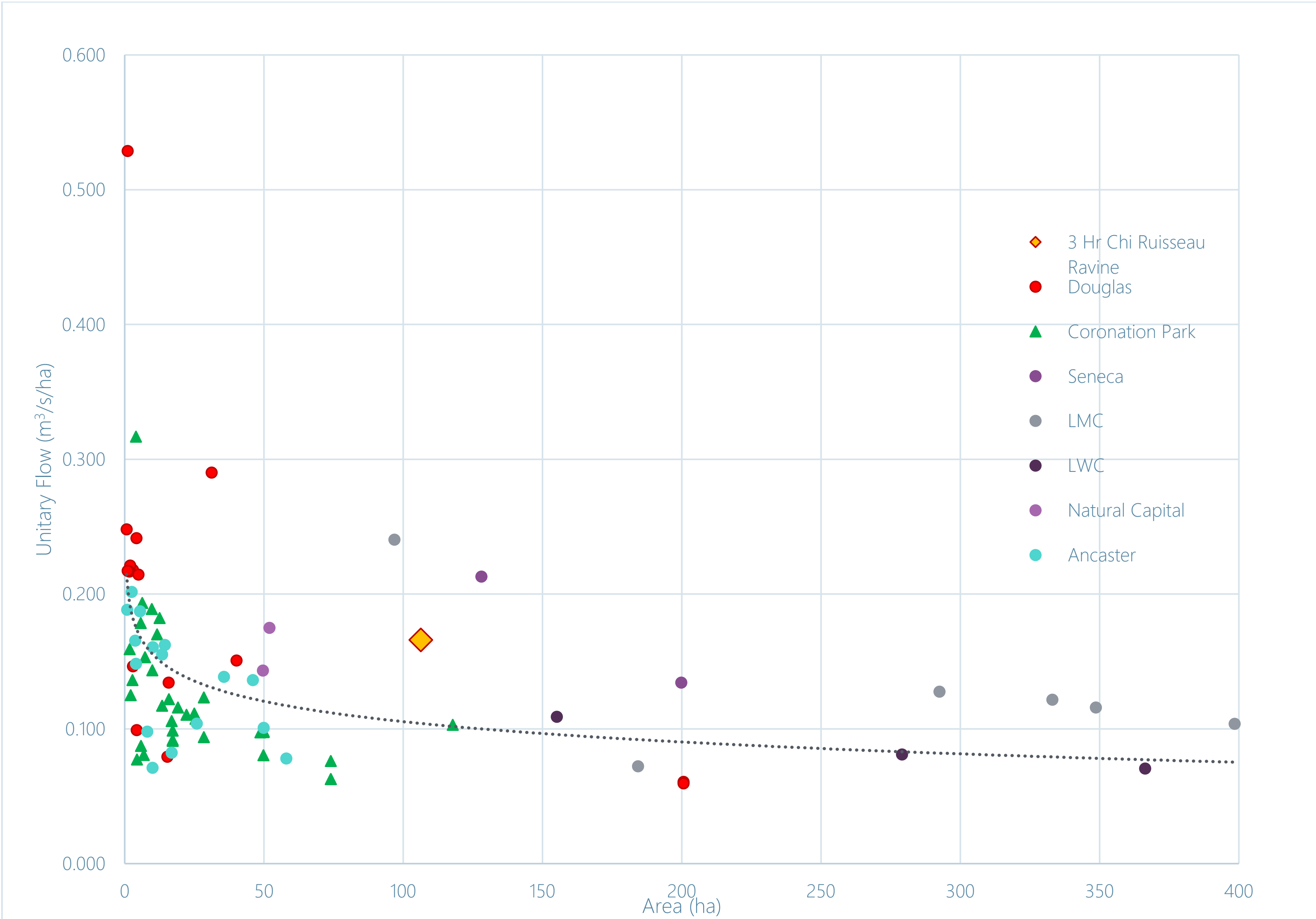
0.600
0.500
0.400
0.300
0.200
0.100
0.000
Unitary Flow (m³/s/ha)
0
50
100
150
200
250
300
350
400
Area (ha)
3 Hr Chi Ruisseau Ravine
Douglas
Coronation Park
Seneca
LMC
LWC
Natural Capital
Ancaster

Figure 3.5. Urban 100 Year Unitary Peak Flow Rate Comparison

To further validate the model results, a comparison of the peak flow rates from the received PCSWMM model and the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) has been conducted (ref. **Table 3.10**)., each of which have been assessed with the 100 year 3 hour Chicago design storm event.

Table 3.10. 100 Year Design Storm Event Peak Flow Rate Comparison to Other Studies

Study	Peak Flow Rate (m^3/s)	Difference (m^3/s)	Difference (%)
Chapel Hill South SWM Study (1989)	14.20	-3.44	-19.5
Mud Creek Cumulative Impact Study (May 2020)	22.92	+5.28	+29.9
Ruisseau Park Ravine (Blue Willow) Rehabilitation (Ongoing)	17.64	-	-

As shown in Table 3.10, the peak flow rate contributing to the SWMF in the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) was 14.2 m^3/s which is 3.44 m^3/s (+/-) less than the peak flow rate from the current study. The difference can likely be attributed to an increase in imperviousness from the previous study, an increase in the contributing drainage area, and a difference in modelling software; PCSWMM has been used for this assessment while the OTTSWMM modelling software was used for the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989).

The peak flow rate contributing to the SWMF in the City's Mud Creek Cumulative Impact Study (Stantec Consulting Ltd. And JTB Environmental Systems, May 2020) is 22.92 m^3/s which is 5.28 m^3/s greater than the peak flow rate from the current study. The difference in the peak flow rate can likely be attributed to the discretization of the subcatchments for the contributing drainage area, the addition of a major system simulated with *sawtoothing* as per the design, and the addition of the storm sewer routing elements.

3.4.3 HEC-RAS Model and Subsequent PCSWMM Model Revisions

As per the approved work plan for the study, a one-dimensional (1D) HEC-RAS model has been developed to assess the water surface elevations and velocities within Ruisseau Park Ravine. Previous HEC-RAS modelling was not available for this assessment, as such, a new HEC -RAS model has been developed to provide the necessary results.

The City's DEM has been merged with the topographic survey data to create a combined DEM to be used for development of the hydraulic cross sections within the HEC-RAS model. Cross sections have been established at 25 m (+/-) spacing along the 450 m (+/-) channel. When possible, the cross sections have been created at locations where survey data have been obtained for the low flow channel.

Field reconnaissance indicates that the low flow channel is winding with shoals, and some natural debris; a Manning's roughness coefficient of 0.045 has been applied for the channel. Several mature trees are located in the floodplain with some debris and undergrowth; a Manning's roughness coefficient of 0.100 has been applied for the overbank areas.

Upon review of the HEC-RAS model existing condition results, it was determined that the downstream boundary conditions used for the HEC-RAS model would not adequately represent the conditions at the site, due to the presence of the outlet structure. The outlet structure is represented by a rating curve within the PCSWMM model and has been incorporated into the HEC-RAS model. The inflow rates to the SWMF are greater than the outflow rates of the boundary condition and the HEC-RAS model will extrapolate the outlet rating curve stage discharge relationship to match the outflow rate to correspond with the inflow rate. The extrapolated outflow rate is assessed on the outlet rating curve within HEC-RAS to generate a corresponding water surface elevation that exceeds the range of acceptable values for the 2-100 year design storm events.

In addition to the HEC-RAS outlet rate issues, the inlet flow rates obtained from the PCSWMM model cannot be directly applied to the HEC-RAS model. The high-water surface elevations associated with the stormwater detention facility generate a backwater condition within the channel, resulting in low flow velocities that are likely less erosive to the ravine slopes. The HEC-RAS model does not account for the backwater condition associated with the stormwater detention facility. For these reasons, the HEC-RAS model has not been used to generate the water surface elevations for the floodline mapping or to generate the velocities within the channel.

The HEC-RAS model sections and channel have been incorporated into the PCSWMM hydrologic/ hydraulic model in place of the storage node which represents the storage upstream of the stormwater management facility control structure. Using this approach, the flood elevations, flow velocities, and peak flow rates within the channel can then be extracted directly from the PCSWMM model. This will also eliminate the need for the HEC-RAS model going forward and resolve the issue with the downstream boundary condition resulting from the outlet structure within the HEC-RAS hydraulic model.

There are inherent differences in the outflow, storage volume, and max water surface elevation between the PCSWMM modelling which has simulated the stormwater management system with the channel versus just a storage node. Additionally, the outlet weir as per the received model to simulate spill over the top of the embankment, has been revised at the request of the City. A transect of the top of the embankment between the Blue Willow SWM facility and Mud Creek has been developed using survey data and supplemented with DEM data and input into the PCSWMM model.

The PCSWMM model has been re-simulated for the City's 12 hour SCS design storm events as requested in addition to the 3 hour Chi storm events for the updated PCSWMM model (the spill weir was replaced with the transect generated with the survey data and the DEM data). The spill elevation has been set at 82.85 m (+/-) as per the received survey data. A comparison of the water surface elevations generated for those two (2) models has been provided in **Table 3.11** in addition to the water surface elevations generated for the 3 hour Chicago storm events using the storage rating curve.

Table 3.11. Water Surface Elevation Comparison (m)

Duration and Distribution	Storage	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
12 Hr SCS	Channel/ Transects	81.42	82.18	82.59	82.96	83.07	83.13
3 Hr Chi	Channel/ Transects	81.30	82.08	82.50	82.91	83.05	83.12
3 Hr Chi	Storage Rating Curve	81.35	82.19	82.62	83.00	82.12	82.21

As shown within Table 3.11, the PCSWMM model water surface elevations for the 2 – 100 year 12 hour SCS storm events with the channel and transects from the HEC-RAS model are generally within the 8 cm (+/-) of the water surface elevation results generated from the 3 hour Chicago storm events with a storage rating curve. The 2-10 year storm events for each scenario would be contained to the SWM facility, while the 25-100 year storm events for each scenario would overtop the embankment between the SWM facility and Mud Creek; the over topping depth for the 25-100 year 12 hour SCS storm events would be 11 cm, 22 cm, and 28 cm respectively at the lowest embankment elevation of 82.85 m (+/-). The 2 – 100 year 12 hour SCS storm event water surface elevations have been graphically represented on Figure 3.6.

The Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) indicates that the SWMF would provide sufficient storage volume up to and including the 100 year design storm event with a 100 year water surface elevation of 82.75 m. The noted differences in the study area imperviousness and the total drainage area have likely contributed to the difference in the 100 year water surface elevation between that study and this study.

The water surface elevations, generated for the 3 hour Chicago storm events with the storage rating curve, identified in Ruisseau Ravine Class EA Existing Conditions Report (Wood, December 19, 2020) have been used for the slope stability analyses (ref. Section 3.8) and are not reflective of the updated water surface elevations used for the flowline mapping. The difference between the two (2) sets of results is considered inconsequential.

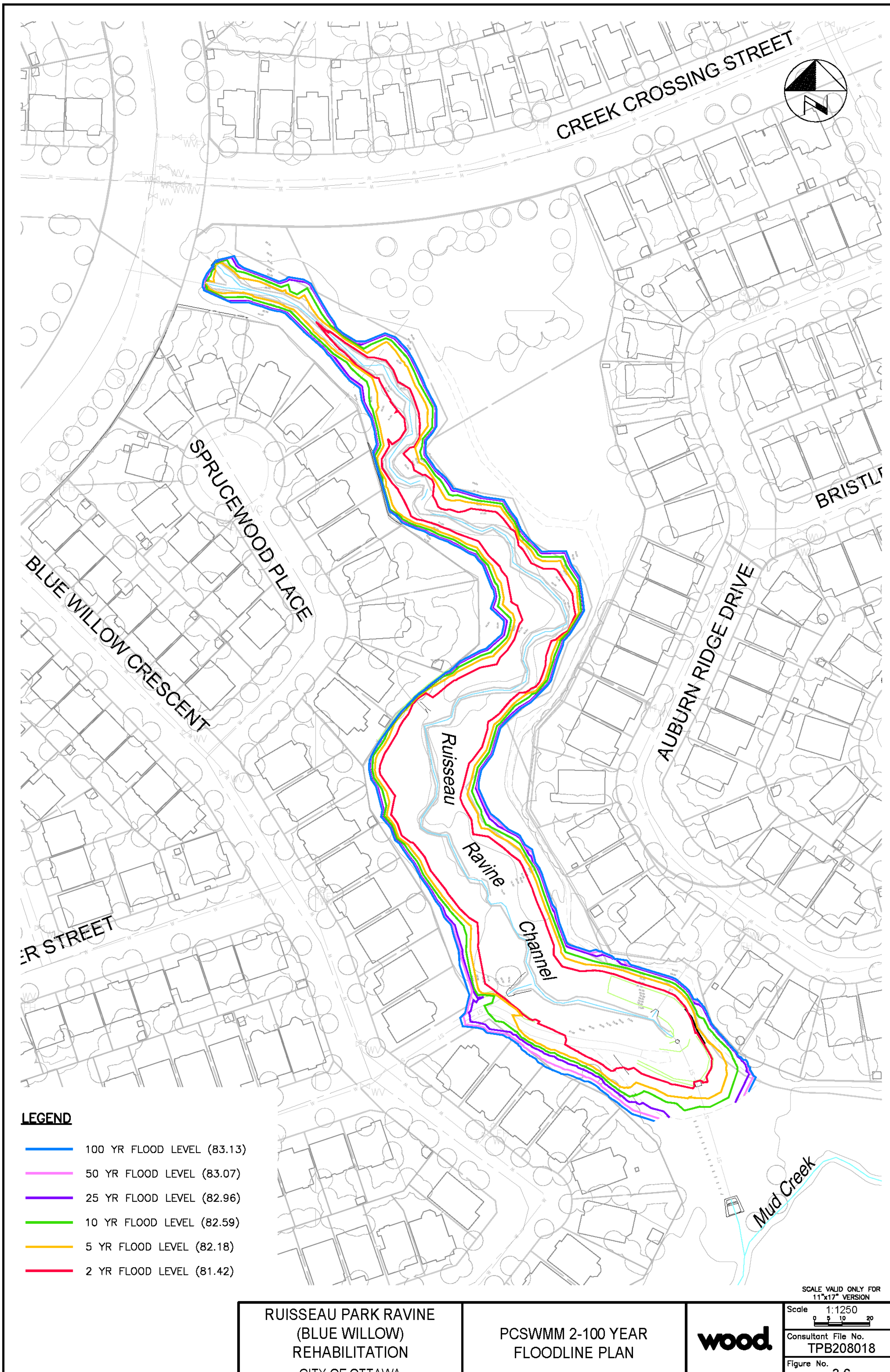

Figure 3.6. PCSWMM 2-100 Year Floodline Plan

3.4.4 Existing Conditions Semi-Continuous Simulation

The PCSWMM model has been executed for a semi-continuous simulation using the 40 year precipitation dataset provided by the City. The depth, flow rates, and velocities have been extracted from the channel conduits represented within Ruisseau Ravine. The semi-continuous simulation results have been used to evaluate the shear stress, or erosion potential within the channel. The shear stress for seven (7) channel conduits of interest have been documented at key locations along the channel alignment and have been used for the stream morphology assessment. A summary of the existing and proposed condition semi-continuous simulations results are presented in Section 6.1 while the full assessment is provided in Appendix J.

3.5 Stream Morphology

A stream morphology assessment has been undertaken by Coldwater Consulting on May 21, 2020, to assess the geomorphology and erosion conditions from the upstream end of the Ruisseau Park Ravine toward the stormwater management facility. The stream conveyed through Ruisseau Park Ravine is presently in a degraded condition and undergoing a rapid morphological evolution which is leading to the geotechnical instability of the steep banks of the ravine within which it runs. It is considered that the rate with which this stream is evolving has likely been accelerated by the impacts of urbanization of the local area in the early 1990’s. The increased imperviousness of the contributing drainage area resulting from urbanization, in addition to the implementation of the major and minor drainage network, has led to the stream experiencing an increase in discharge from rainfall events. The resulting downcutting and planform migration of the stream has led to slope instability issues at several locations which have the potential to endanger adjacent properties. Examples of the erosion are provided in Figures 3.7 to 3.9.

Figure 3.7. The left bank is eroded, located in the upper reach of the channel

Figure 3.8. Failure of the left bank opposite a rip rap protected slope

Figure 3.9. Eroded banks and the roots of an established tree have been exposed

A Rapid Geomorphic Assessment (RGA) has been undertaken for this project which uses observations of channel instability indicators to quantify channel sensitivity based on aggradation, degradation, channel widening and planimetric adjustment. The results of these four (4) categories are combined to provide a channel Stability Index (SI) classified as *In Regime*, *In Transition*, and *In Adjustment*. More than 75% of the channel, or the upper three (3) reaches, are classified as *In Adjustment*, while the lower portion of the channel is *In Transition*.

A Rapid Stream Assessment Technique (RSAT) has also been undertaken which combines ecological observations with fluvial geomorphic indicators. A numerical rating is developed for this assessment to produce a score that corresponds to a rating of *Excellent*, *Good*, *Fair*, or *Poor*. The RSAT scores indicate that the upper half of the stream is in *Poor* condition while the lower half of the stream if in *Fair* condition which generally corresponds to the RGA results.

The stream is undergoing active erosion along most of its length; areas of special concern where erosion is cutting into a high bank or where continued erosion could lead to property damage are identified in the Stream Morphology Existing Conditions Report (Appendix E) and in Figure 3.10. It is recommended that these areas at a minimum be addressed as part of the rehabilitation; however, isolated interventions may only protect those areas addressed and new erosion hotspots may develop. It is therefore recommended that a mitigation approach be adopted that offers protection to the entire stream.

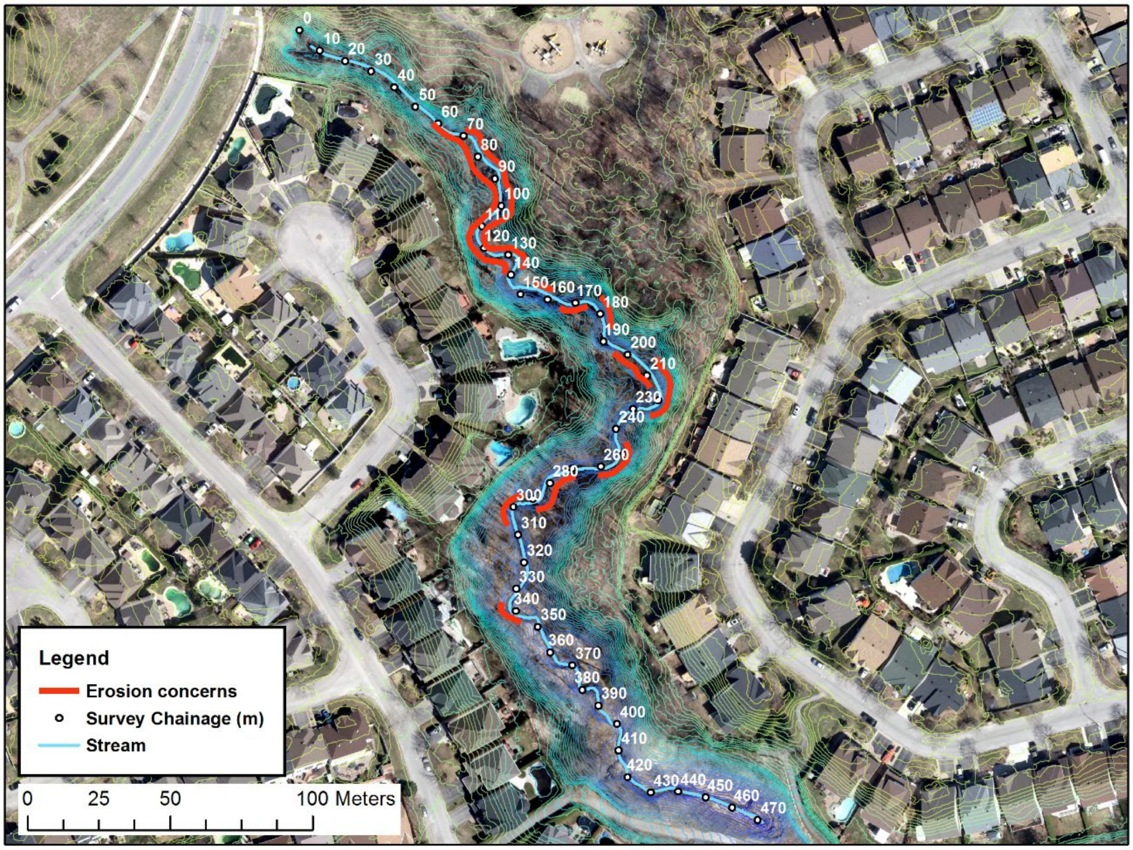

Figure 3.10. Locations of Erosion Concern

3.6 Geotechnical Assessment

3.6.1 Slope Stability

Slope stability concerns have arisen within the Ruisseau Ravine study area due to the failure of the slope behind the residential property at 9 Spruce Wood Place, during May, 2017. The slope failed as a result of bank erosion at the toe of the slope. Emergency rehabilitation works were undertaken to address the slope failure which included placing rock fill material on the slope to mitigate the potential for continued slope failure. The rock fill material was not intended to be a permanent solution and would require a long-term solution to mitigate the potential future slope failure. The City has continued to monitor the slope until a permanent solution can be implemented.

Site reconnaissance was performed by a qualified Wood Geotechnical Engineer in May 2020. The site conditions of the ravine slopes were inspected at various location and the observations were documented with photos. At the time of site reconnaissance, the water levels within the creek had a maximum depth of 0.3 m (+/-) above the creek bed. Most of the ground surface in vicinity of the channel was covered with leaves and the clayey soil was exposed near the base of creek banks and at the toes of several slopes. Erosion at the creek banks and the slope toes was observed at heights approximately ranging from 0.3 m to 2 m above the creek levels.

The geotechnical investigation consisted of advancing seven (7) boreholes and two (2) Piezocone Penetration Testing (CPT) holes between 12 August 2020 to 4 September 2020. The borehole depths range from 2.0 m (+/-) to 12.8 m (+/-) while the CPT hole depths were 12.5 m (+/-) in depth. Soil samples from the boreholes indicated that clay was encountered in all the advanced boreholes while silty sand was encountered in three (3) boreholes. The depth of the clay ranged from exposure at the surface to a depth of 12.8 m (+/-) which was the termination depth of Borehole BH1, while the silty sand extended to a depth of 0.7 m (+/-). Bedrock was not encountered within the advanced borehole depths. The borehole plan is shown in Figure 3.11.

Groundwater levels have been measured in the installed monitoring wells in Boreholes BH3 and BH5 at minimum depths of 2.1 m and 3.1 m (+/-) below grade respectively on September 4, 2020. The water level can fluctuate and be greater during the spring months and in response to formative precipitation events (> 15mm)..

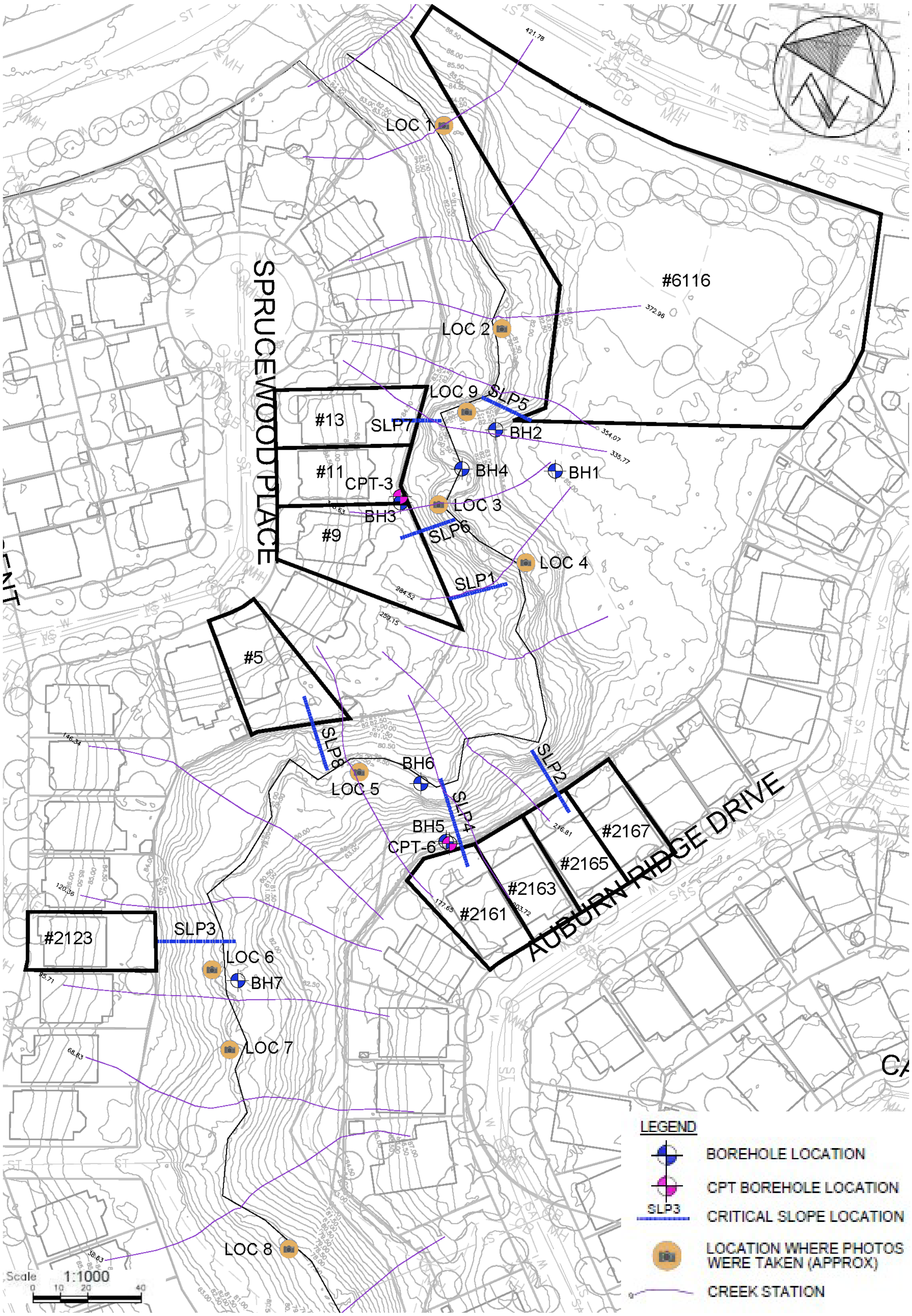

Figure 3.11. Borehole and Existing Critical Slope Location Plan

A total of six (6) slopes have been initially identified to be of concern during the site reconnaissance. After discussions with the City, two (2) additional slopes at the existing rock retaining wall areas located at the west of the ravine were added to the study. Accordingly, a total of eight (8) slopes have been analyzed for stability. The critical slope locations that have been analyzed for this assessment are shown on Figure 3.11. The factual information and the results of the slope assessment are provided in the preliminary Geotechnical Report in Appendix F.

Slope stability analyses have been performed using Slope/W (version 8.16.2.14053, GeoStudio, 2016) to determine the existing slope conditions. The global slope stability analyses considered various situations including the long-term static condition, the seismic condition, and the rapid drawdown (25-year and 100-year flood) during the flooding periods on each slope. For preliminary analyses, the acceptable Factors of Safety (FOS) against the potential slope instability in conjunction with the peak soil shear strength were as recommended by Ontario Ministry of Natural Resources (2011) (OMNR 2011): 1.5, greater than 1.0, and 1.2 for the long-term static condition, seismic condition, and the rapid drawdown condition during the flooding periods, respectively.

Following the assessment, three (3) slopes, SLP2, SLP4, and SLP6 have been identified as likely requiring a bank stabilization treatment to prevent progressive erosion. Slopes SLP2 and SLP4 are 6.7 m (+/-) and 8.2 m (+/-) in height respectively with slope inclinations of 30^0 (+/-) and 25^0 (+/-) respectively. Near vertical drops of greater than 2.7 m (+/-) were observed at the toe of both slopes. Slope SLP6 is covered with a layer of rock fill of unknown thickness to temporarily stabilize the slope due to previously failure. The height of the slope is 4.0 m (+/-) with a slope inclination of 30^0 (+/-). The slope stability analysis results for these slopes are provided in **Table 3.12**.

Table 3.12. Summary of the Global Stability Analysis

Slope No.	Referenced Borehole No.	Nearby Property	Type of Analysis	Calculated Factor of Safety
SLP 2	BH5, BH6, and CPT-5	2165 Auburn Ridge Drive	Long Term (Drained)	1.4
SLP 2	BH5, BH6, and CPT-5	2165 Auburn Ridge Drive	Seismic (Undrained)	1.4
SLP 2	BH5, BH6, and CPT-5	2165 Auburn Ridge Drive	Rapid Drawdown (25-Year Flood)	1.2
SLP 2	BH5, BH6, and CPT-5	2165 Auburn Ridge Drive	Rapid Drawdown (100-Year Flood)	1.2
SLP 4	BH5, BH6, and CPT-5	2161 Auburn Ridge Drive	Long Term (Drained)	1.4
SLP 4	BH5, BH6, and CPT-5	2161 Auburn Ridge Drive	Seismic (Undrained)	1.2

Slope No.	Referenced Borehole No.	Nearby Property	Type of Analysis	Calculated Factor of Safety
SLP 4	BH5, BH6, and CPT-5	2161 Auburn Ridge Drive	Rapid Drawdown (25-Year Flood)	1.1
SLP 4	BH5, BH6, and CPT-5	2161 Auburn Ridge Drive	Rapid Drawdown (100-Year Flood)	1.1
SLP 6	BH3, BH4, and CPT-3	Existing temporary rockfill at 9 Sprucewood Place	Long Term (Drained)	1.1
SLP 6	BH3, BH4, and CPT-3	Existing temporary rockfill at 9 Sprucewood Place	Seismic (Undrained)	0.7
SLP 6	BH3, BH4, and CPT-3	Existing temporary rockfill at 9 Sprucewood Place	Rapid Drawdown (25-Year Flood)	0.7
SLP 6	BH3, BH4, and CPT-3	Existing temporary rockfill at 9 Sprucewood Place	Rapid Drawdown (100-Year Flood)	0.7
SLP 7	BH3, BH4, and CPT-3	13 Sprucewood Place	Long Term (Drained)	1.5
SLP 7	BH3, BH4, and CPT-3	13 Sprucewood Place	Seismic (Undrained)	2.8
SLP 7	BH3, BH4, and CPT-3	13 Sprucewood Place	Rapid Drawdown (25-Year Flood)	1.4
SLP 7	BH3, BH4, and CPT-3	13 Sprucewood Place	Rapid Drawdown (100-Year Flood)	1.4

The calculated factor of safety for the slopes shown in Table 3.12 does not meet the FOS criteria against potential slope instability recommended by the OMNR. These slopes correspond to three (3) of the locations identified with erosion concerns in the Geomorphology Existing Conditions Report.

Based on the results of analyses, the Slope SLP 7 may marginally meet the criteria of OMNR 2011 (Minimum FOS of 1.5) for the long-term normal condition. That said, subsequent to the analyses, it was indicated to the City that settlement was observed within the rear yard of 13 Sprucewood Place. The main concern is the possible continuous erosion at the slope toe resulting in the undermining of the adjacent existing retaining wall and cause slope failure in the future.

Following the discussions with the City of the findings in the above noted preliminary geotechnical report it was decided to conduct additional soil testing and analyses to examine the risks associated with the potential for retrogressive slope failures known to occur in sensitive clay in the Ottawa area. The technical memoranda presenting the additional soil testing and the retrogressive failure analyses are also included in Appendix F.

3.6.2 Retrogressive Failure

An assessment has been undertaken for the slopes of concern, SLP 2, 4, 6, and 7, to determine the potential of retrogressive failure at the slopes (ref. Appendix F). Retrogressive failure is the failure of a slope commencing at the toe of the slope with successive segments of slope failure; the failure is typically initiated by a localized rotational slip and can extend upland, several hundred metres or more. A literature review of previous retrogressive failures, including earthflow slides in sensitive Leda Clay in 1971 and 1993 on the east bank of the South Nation River near Lemieux, Ontario, identified a sensitive marine clay below stiffer soils consisting of marine-estuarine sands, detaic silts, and sands. It is believed the earthflow was triggered by pre-slide remolding of the sensitive clay due to historic landsides in 1895 and 1910, the wettest winter on record, and a pre-slide river peak flow of 22.4 m^3/s compared to the average flow rate of 12.7 m^3/s.

Laboratory tests conducted on the soils obtained from the Ruisseau Ravine site, including direct shear tests, triaxial consolidated isotropic undrained compression tests, and the piezocone penetration tests, suggest the clay soil found at the site would meet the criteria noted within the literature review, and the soils would be vulnerable to an earthflow risk. However, in the absence of an adverse triggering conditions the earthflow slides may never occur at this site.

The most probable triggering factor for earthflow/ retrogressive is the loss of toe of bank from creek erosion which may result in an initial slope rotational failure. Another major factor contributing to the remolding of the sensitive clay is the presence of high groundwater in the tableland in conjunction with a fissured / cracked clay cap exercising a persistent horizontal driving force on the sliding soil mass of existing slopes. Due to the significant slope movement, the residual soil strength characteristics become more relevant in the determination of the resisting forces. The most effective mitigation is prevention of the toe loss through the implementation of bank stabilization with buttressing, such as armour stone, and / or slope flattening combined with protection against erosion and scour, etc. to meet the requirements against slope instability as per OMNR 2011 and to avoid potential progressive failure.

It is understood that the City has been conducting site observations at SLP 6 and SLP7 and will continue to conduct site observations. As part of the site observations at SLP 6 and SLP7 it was learnt that settlement was observed within the rear yard of 13 Sprucewood Place, as such, continuous monitoring of the existing slopes at SLP 6, SLP 7 and along the remaining creek is required.

As indicated by the site visit reports prepared by Stantec, significant signs of erosion have been observed since the inception of the visual slope observation program in 2021. A 0.6 m deep void directly below the limestone stacked wall at 13 Sprucewood Place and the close distance between the wall and the crest of the steeper slope/bank of less than 1 m were reported. As such, temporary remedial creek bank erosion and slope stabilization measures (e.g. gabions, stone, sand bags at the toe of slope) may need to be considered by the City prior to implementing long-term erosion mitigation works within the creek, depending on results of future observations.

3.7 Natural Systems

Any efforts to mitigate the erosion and slope stability concerns within the Ruisseau Park Ravine may require vegetation removal; as such, a natural systems assessment has been undertaken to document the pre-mitigation works presence and distribution of vegetation communities, potential Species at Risk (SAR) and SAR habitat, and Significant Wildlife Habitat (SWH). The following summarizes the findings of the SAR investigation for the natural systems assessment to inform the mitigation planning; the SAR Technical Memorandum is provided in Appendix G.

The City of Ottawa Official Plan (2003) provides a vision for the future growth of the city and a policy framework to guide the city's physical development. A review of the Official Plan Schedules indicate that a Natural Heritage System (NHS) and unstable slopes (environmental constraint states) are shown to coincide with the Study Area. The NHS is protected by policies that state that the development should not negatively impact the feature or its function. The areas with unstable slopes are not permitted to be used for public utilities and municipal services and facilities unless such use is necessary in the area.

A field investigation was conducted on May 11, 2020, to collect information by a qualified professional from Wood Staff. Ecological Land Classification (ELC) was used to characterize the ecosites within the Project Location and identify the presence of rare or sensitive plants and vegetation communities. Wildlife documentation was based on incidental observations, including evidence of tracks and/or scat, as well as available habitat. Significant Wildlife Habitat and Species at Risk were based on field observations and background information. No specific inventory was completed for roosting trees (for SAR bat species), however, the potential for select trees was accessed.

The ELC delineation completed by Wood during the field investigation identified the following communities within the Project Location:

- Manicured landscapes
- Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite (FOM3-2)
- Reed Canary Grass Graminoid Meadow (MEGM3-8)
- Fresh-Moist Willow Lowland Deciduous Forest (FOD7-3)

A preliminary desktop SAR screening of available reports, databases, and mapping was completed in May 2020 for the Project Location. The desktop screening identified 15 SAR, which have the potential to occur with the Project Location. The eight (8) SAR with a moderate to high potential of occurrence in the study area are summarized in Table 3.13.

Table 3.13. Species at Risk with Moderate to High Potential for Occurrence Summary

Species Name, Status (SARA, ESA, S-Rank)[1,2,3,4], and Data Source	Preferred Habitat	Potential for SAR Habitat/ Occurrence within the Project Location
Birds		
Bank Swallow (*Riparia riparia*) SARA: Threatened ESA: Threatened S-Rank: S4B Source: OBBA	Bank Swallows nest in burrows in natural and human-made settings where there are vertical faces in silt and sand deposits. Many nests are on banks of rivers and lakes, but they are also found in active sand and gravel pits or former ones where the banks remain suitable (COSEWIC, 2013).	**Moderate** – There are some steep banks along the tributary as a result of the problematic erosion in the Project Location but due to the smaller size of the creek and the larger size of the trees in the Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite, and Fresh-Moist Willow Lowland Deciduous Forest Type, the trees act as a barrier to access the steep banks. Habitat within the immediate Project Location is considered limited. This species may use the Project Location for foraging.
Barn Swallow (*Hirundo rustica*) SARA: Threatened ESA: Threatened S-Rank: S4B Source: OBBA	Barn Swallows have shifted largely to nesting in and on artificial structures, including buildings, bridges and road culverts, and prefer various open habitats for foraging including grassy fields, pastures, agricultural crops and over open water (Heagy, et al., 2014).	**Moderate** – The culverts related to the SWM facility and creek were checked for nests, and none were found. This species may use the Project Location for foraging.
Chimney Swift (*Chaetura pelagica*) SARA: Threatened ESA: Threatened S-Rank: S4B, S4N Source: OBBA	Chimney swifts forage aerially over virtually any habitat. Nesting and roosting take place in a dark sheltered spot with vertical surfaces to cling to. This may include large hollow trees, chimneys, and other structures (COSEWIC, 2007).	**Moderate** – No chimney structures found within the Project Location. This species may use the Project Location for foraging.

Species Name, Status (SARA, ESA, S-Rank)[1,2,3,4], and Data Source	Preferred Habitat	Potential for SAR Habitat/ Occurrence within the Project Location
Eastern Wood-pewee (*Contopus virens*) SARA: Special Concern ESA: Special Concern S-Rank: S4B Source: OBBA, NHIC	Eastern Wood-pewee breeding in mature to intermediate-aged forests with an open understory, often being associated with clearings and edges. Migrants may occur in a wide variety of habitats (COSEWIC, 2012).	**Moderate** – Deciduous forest communities occur along the creek valley within the Project Location. Suitable habitat exists outside the Project Location.
Reptiles		
Snapping Turtle (*Chelydra serpentina*) SARA: Special Concern ESA: Special Concern S-Rank: S4 Source: ORAA, Stantec	Snapping Turtles prefer slow-moving waters with a soft mud bottom and dense aquatic vegetation. Established populations are most often located in ponds, sloughs, shallow bays or river edges and slow streams and wetlands. Individuals can also exist in developed areas (e.g., golf course ponds, irrigation canals); however, it is unlikely that populations persist in such habitats. Snapping Turtles can occur in highly polluted waterways, but environmental contamination is known to limit reproductive success (COSEWIC, 2008).	**Moderate** – Suitable habitat can be found within the Project Location. Snapping turtle could use the creek and surrounding habitat, such as the Reed Canary Grass Graminoid Meadow Type, for nesting. However, this record likely comes from an off-site location as the area is severely fragmented from suitable habitat.
Mammals		
Tri-colored Bat (*Perimyotis subflavus*) SARA: Endangered	Within treed habitats, Tri-colored Bat primarily roosts in tree foliage (mainly within oak leaves). Leaf roosts are shaped like umbrellas with a “roof” and a hollow	**Moderate** – The Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite, and Fresh-Moist Willow Lowland Deciduous Forest Type contained vegetation species preferred by Tri-colored Bat.

Species Name, Status (SARA, ESA, S-Rank)[1,2,3,4], and Data Source	Preferred Habitat	Potential for SAR Habitat/ Occurrence within the Project Location
ESA: Endangered S-Rank: S3 Source: OMA	core where bats rest. Studies have shown that oak leaves are a preferred roost site. Maple leaves are also selected, although less commonly. It is thought that Tri-colored Bat may prefer roost trees in more open woodlands, as opposed to deep woods. Roosts in tree cavity are used less frequently than Myotis species (Ontario Ministry of Natural Resources, 2011).	
Little Brown Myotis (*Myotis lucifugus*) SARA: Endangered ESA: Endangered S-Rank: S3 Source: OMA	The Little Brown Myotis occurs across the southern half of Canada and is especially associated with humans, often forming nursery colonies in buildings, attics, and other man-made structures. Little Brown Myotis forage over water where their diet consists of aquatic insects, mainly midges, mosquitoes, mayflies, and caddisflies. They also feed over forest trails, cliff faces, meadows, and farmland where they consume a wide variety of insects, from moths and beetles to crane flies (Ontario Ministry of Natural Resources, 2011).	**Moderate** – Possible bat maternity roosts were found within the in the Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite, and Fresh-Moist Willow Lowland Deciduous Forest Type.
Invertebrates		
Monarch (*Danaus plexippus*) SARA: Special Concern	Monarch is very widely distributed across North America and found in a wide variety of habitats. Populations fluctuate dramatically but have been generally	**Moderate** – No milkweed plants were found within the Reed Canary Grass Graminoid Meadow Type. The Reed Canary Grass Graminoid Meadow Type could be used for foraging.

Species Name, Status (SARA, ESA, S-Rank)[1,2,3,4], and Data Source	Preferred Habitat	Potential for SAR Habitat/ Occurrence within the Project Location
ESA: Special Concern S-Rank: S2N, S4B Source: TEA, Stantec	declining likely due to habitat destruction on the hibernation grounds in Mexico, as well as pesticide use and other factors on the vast breeding grounds. Monarchs require Milkweeds (*Asclepias*) to lay their eggs and will use a variety of other flowers for adult food (COSEWIC, 2016).	

Determination and screening of Significant Wildlife Habitat (SWH) was not included in the project scope. However, potential and confirmed SWH was observed by Wood during the field investigation and has been documented. Potential snake hibernaculum was found within the Reed Canary Grass Graminoid Meadow Type along the west bank of the SWM facility. Other potential SWH within the Project Location include bat maternity colonies, habitat of special concern or rare wildlife species, and potential turtle nesting areas.

A summary of environmental findings and recommendations are as follows:

- Consultation with the City of Ottawa will be required to determine if an EIS is required. Work within or adjacent to the NHS is not permitted unless it is demonstrated that there will be no negative impact on the feature or its ecological function.
- The field investigation did not include targeted surveys for wildlife (i.e., breeding bird surveys), and was completed at a time when definitive identification could not be completed for some vegetative species due to dormancy. As such, the confirmation of presence or absence of species cannot be made with certainty. However, the likelihood of SAR presence in the Project Location is none to moderate.
- Potential bat SAR habitat was documented during the field investigation. Potential bat maternity roost trees were found throughout the Project Location within the Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite, and Fresh-Moist Willow Lowland Deciduous Forest Type. Some potential bat maternity roost trees may require removal for Project work. It is recommended that the number of trees to be removed is kept to a minimum to reduce the amount of disturbance to potential bat SAR habitat;
- The functional design should minimize work around and within the potential SWH features such as the snake hibernaculum, turtle nesting areas and should avoid the Graminoid Mineral Meadow Marsh Ecoelement as it is confirmed SWH;
- Development within this SWH (the Graminoid Mineral Meadow Marsh ecoelement) is not permitted unless the development has no negative impacts on the feature or its ecological function. The best mitigation measure is to avoid this habitat. If development is to occur adjacent to seeps and springs a water balance study to demonstrate that the mitigation measures will be effective in managing groundwater flows could be completed. It should also demonstrate that water quality in the seep or spring will not be adversely affected.
- To reduce vegetation removal, access routes and work areas should utilize existing official and ad hoc trails through the Project Location;
- Should any SAR be encountered on-site during project implementation, work in the area should be stopped, and the MECP should be contacted;
- Given the presence of potential bat maternity roost trees and the breeding bird window, vegetation removal should not take place between May 1 and September 31 in any given year;

- Removal of all vegetation shall occur outside of the general migratory birds nesting period (April 1 to August 31), and activities shall occur in accordance with the MBCA and Migratory Bird Regulations. These timing constraints should not be perceived as absolutes. This period represents the core breeding period, although some species may nest in March and September. Ultimately, the objective from a compliance perspective is not to circumvent the MBCA. As such, due diligence measures should be implemented and documented for any nest searching efforts, including record control, to ensure compliance with the MBCA;
- If bird nests protected under the MBCA, *Fish and Wildlife Conservation Act, 1997* or ESA are encountered at any stage of the project these species and their nests must not be disturbed, tormented, injured in any way, destroyed, and/or separated from young. During construction, work must stop in the vicinity of the sighting until further direction is provided. A protective buffer area should be established around the nest and should be determined in consultation with a qualified Avian Biologist, as well as the Ministry of Natural Resources and Forestry, MECP and/or Canadian Wildlife Service, as necessary;
- All work zones shall be clearly marked on detailed design drawings in relation to natural heritage features identified in the Technical Memorandum provided in Appendix G;
- An Erosion and Sediment Control (ESC) plan shall be prepared prior to construction and conform to industry best management practices and recognized standard specifications. The ESC Plan shall follow the Erosion and Sediment Control Guidelines for Urban Construction (Greater Golden Horseshoe Conservation Authorities, 2006); and
- All disturbed areas within the construction site will be stabilized and revegetated as soon as conditions allow.

A Tree Inventory has been undertaken for the existing public park for the areas considered to be disturbed. The results of the inventory are provided in Appendix G for the preferred design alternative. The Tree Inventory will be used to inform the functional design and the potential impacts on tree cover in the Project Location.

4.0 Baseline Condition Key Findings

The key findings of the Ruisseau Park Ravine baseline conditions have been summarized as follows:

Hydrology and Hydraulics

1. A PCSWMM model has been developed for the assessment with a contributing drainage area to the Ruisseau Park Ravine 106.30 ha (+/-) and peak flow rates ranging from 6.70 m^3/s (+/-) to 17.64 m^3/s (+/-) for the 2-100 year design storm events.
2. The contributing drainage area for this study is 33.12 ha (+/-) greater than the drainage area identified in the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) with a 16.17 % increase in imperviousness compared to the previous study.
3. The water surface elevation in the SWMF for the 25 year, 50 year, and the 100 year design storm events have been simulated to overtop the embankment at the downstream end of the facility while the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) indicated that the facility would not be overtopped during a 100 year design storm event.
4. A 40 year semi-continuous simulation of the PCSWMM model has been executed to assess the shear stress of the channel banks at multiple channel sections based on the flow rates, depth of flow, and the velocity.

Stream Morphology

5. The stream morphology team from Coldwater Consulting conducted field reconnaissance and a geomorphic assessment. The increased imperviousness of the contributing drainage area, in addition to the implementation of the drainage network, has led to an increase in discharge to the channel during precipitation events; this has led to downcutting and planform migration of the stream and slope instability issues at several locations.
6. The Rapid Geomorphic Assessment (RGA) undertaken for this project identified that more than 75% of the channel, or the upper three (3) reaches, are classified as In Adjustment, while the lower portion of the channel is In Transition.
7. The Rapid Stream Assessment Technique (RSAT) indicates that the upper half of the stream is in Poor condition while the lower half of the stream is in Fair condition which generally corresponds to the RGA results.
8. Based on the geomorphic assessment, multiple locations within Ruisseau Park Ravine have been identified as requiring bank stabilization to mitigate further erosion.

Geotechnical

9. The seven (7) boreholes and two (2) Piezocone Penetration Testing (CPT) holes advanced by Wood's Geotechnical team identified the soils within the ravine as primarily being clay and silty sand; bedrock was not encountered within the advanced boreholes.

10. Slope stability analyses have been performed for eight (8) slopes in which three (3) slopes, SLP2, SLP4, and SLP6 have been identified as requiring a bank stabilization treatment to prevent progressive erosion, while there is a concern that active erosion observed at the toe of a fourth slope, SLP7, raises a concern of potential near term slope instability.
11. The sensitive clay soils at the site are consistent with the clay soils which were identified at locations of previous retrogressive slope failure sites near Lemieux, Ontario; the soils at the site are vulnerable to an earthflow risk/ retrogressive slope failure. However, in the absence of an adverse triggering conditions the earthflow slides may never occur at this site.

Natural Systems

12. The ELC delineation completed by Wood's Natural Heritage Team during the field investigation identified four (4) communities within the study area; manicured landscapes, Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite, Reed Canary Grass Graminoid Meadow, and Fresh-Moist Willow Lowland Deciduous Forest.
13. A preliminary desktop Species at Risk (SAR) screening identified 15 SAR, which have the potential to occur with the Project Location, while eight (8) SAR of those species have been identified with a moderate to high potential of occurrence in the study area.
14. Potential snake hibernaculum was found within the Reed Canary Grass Graminoid Meadow during the field investigation along the west bank of the SWMF. Other potential Significant Wildlife Habitat (SWH) within the Project Location include bat maternity colonies, habitat of special concern or rare wildlife species, and potential turtle nesting areas.

5.0 Identification and Evaluation of Alternative Solutions

A long list of creek improvement alternatives has been developed to address the existing creek bank erosion conditions and to be integrated with slope stability alternatives. For the current study the evaluation of alternatives has been divided into a Long-list screening exercise and Short-list detailed alternative assessment. The following is a preliminary list of potential alternatives for the study area:

5.1 Identification and Evaluation of Alternative Solutions

The following provides an outline of potential alternatives to address the creek erosion concerns. Seven (7) alternatives have been considered for mitigating the identified problem:

1. Do Nothing
2. Stormwater Management (SWM) and Low Impact Development (LID) Best Management Practices (BMPs)
3. Creek Realignment
4. Creek Partial Enclosure
5. Creek Control Structures
6. Channel Hardening
7. Combinations

Slope stabilization alternatives to address the existing slope stability concerns are assessed along with the short-listed creek improvement alternatives. The following slope stabilization alternatives are being considered:

1. Bioengineering
2. Localized slope regrading
3. Mechanical stabilization solutions (e.g., TerraFirm™)

5.2 Descriptions of the Long List of Alternatives

The following provides a description of the creek improvement long-list of alternatives.

i. **Do Nothing**
No measures are proposed to mitigate the existing erosion and bank stabilization concerns. This alternative represents baseline conditions, and its evaluation is required by the Municipal Class EA process, however it is not considered as preferred since the erosion problems would not be addressed.

ii. **Stormwater Management and Low Impact Development Best Management Practices**
Various stormwater management alternatives are available to address the existing erosion and bank stabilization concerns (ref. Figure 5.1). Stormwater management would be designed to reduce the volume and/or rate of runoff conveyed to the ravine from the upstream developed area. The goal would be to reduce the runoff rate and volume such

that the frequency and duration of exceedance of the erosion thresholds of the Study Area reaches are reduced, thereby resulting in less erosion. The implementation of additional stormwater management facilities options may be limited to sub surface detention facilities (ref. Figure 5.2) due to the existing developed area. These could be implemented where available public lands such as in parks or the municipal rights-of-way (ROW).

Source controls in the form of Low Impact Development Best Management Practices (LID BMPs) have become an important component of mitigating runoff at source. Where implemented, LID BMPs (e.g., bioswales, infiltration trenches, vegetated buffer strips, rain barrels, etc.) are distributed throughout the development area and can be incorporated into the landscaping of individual lots and public ROW (ref. Figure 5.3) and can provide water balance, stormwater quality and erosion control functions. LID BMPs, like traditional stormwater management, are typically applied to new greenfield development where they can be more readily incorporated into the urban planning fabric. Retrofitting LID practices in existing urban areas and neighbourhoods is possible, however, often challenging. Also, due to the relatively "young" age of existing infrastructure within the upstream Chapel Hill development, significant rehabilitation will not take place for several decades and therefore would not address the existing concerns in a timely manner.

Figure 5.1. Krizsanderson Stormwater Management Facility, Kitchener, Designed by Wood

Figure 5.2. Underground Storage System, Courtesy Stormtech

Figure 5.3. Bioretention System, Courtesy Sustainable Technologies

iii. Creek Realignment

The creek realignment alternative employs a holistic approach to resolving channel issues and re-establishing a self-sustaining channel. Generally, this is employed when there are several issues of varying severity over a substantial length of channel. Realignment is also particularly beneficial when the issues are a result of the current planform alignment and there is sufficient area in the valley for relocation. Channel realignment is the development of a new planform alignment that is determined based on the surrounding conditions, existing channel conditions, and general principles of natural channel design. The channel is realigned away from at-risk features, such as infrastructure or valley walls, the channel construction requires fewer hard structures and allows for the use of more natural techniques. Vegetation can be employed to establish the banks and new bed structures can be created to sustain channel morphology. Channel reshaping can also be undertaken at the time of realignment to develop a suitable low flow channel with proper bank treatments and flood plain.

The primary limitations of this alternative are the higher relative capital costs and the amount of disturbance to the surrounding terrestrial environment. However, it is often considered to provide the highest potential for long-term stability and, therefore, would require minimal long-term maintenance and associated costs.

The implementation of channel realignment could be undertaken with three (3) levels of magnitude; localized, reach-based, and fulsome. Localized channel realignment could be employed to modify individual bends in the channel away from localized areas of erosion concern such as the toe of the valley wall. Reach-based realignment is similar to the localized channel realignment however would be employed over a greater length of the channel where more persistent stability concerns are present. The fulsome realignment of the channel is the most expansive and disruptive of the three (3) as the entire channel is realigned to a preferred alignment.

Examples of creek realignment have been provided in Figures 5.4 and 5.5, which demonstrate the post construction realigned creek and the realignment 1 year following construction respectively. As shown within the figures, there is an opportunity for the planform of the realigned creek to be designed to avoid mature and established vegetation within the grading limits. Mature vegetation adjacent to the former channel that may have been threatened at the top of an eroding bank could be stabilized and potentially thrive once the former channel has been filled in.

Figure 5.4. Creek Realignment, Post Construction, West Morrison Creek

Figure 5.5. Creek Realignment, 1 Year Following Construction, West Morrison Creek

iv. Creek Partial Enclosure

This alternative involves the enclosure or burial of the creek (ref. Figure 5.6). Inflow at the top of the channel would enter and be carried in a culvert through Reaches 1 and 2, daylighting in Reach 3. As a rule, the design of any channel restoration project should attempt to minimize the length of any channel enclosure. Stream enclosure eliminates the connection between the riparian zone and the aquatic habitat. The elimination of the input of organic matter, solar input and its effects on stream temperature, natural channel morphology, and vegetation can have a strong impact on fish and invertebrate communities. To address these concerns, an inflow structure could be constructed to bypass a controlled discharge to supply a low flow channel through the existing creek. Rather than relocating the channel away from at-risk features, such as infrastructure or valley walls, this technique removes the discharge of the larger events which is the cause of the stream erosion and slope instability issues. The existing creek corridor would then need to be reconstructed with a channel morphology consistent with the local environment and reduced in-channel hydrology.

The primary limitations of this alternative are the very high capital costs and the amount of disturbance to the surrounding terrestrial environment. The costs to re-establish a natural channel through the park, if required, would similarly be very high. However, it would provide a high potential for long-term stability and therefore, the creek would require minimal long-term maintenance and associated costs, that said the enclosure would require maintenance and would eventually require replacement.

The creek enclosure would be considered a last resort measure that would only be considered at locations where there is no other possibility to protect properties and infrastructure from slope stability issues.

Figure 5.6. Creek Enclosure, Courtesy State of Washington

v. Creek Control Structures

This alternative uses structures to dissipate the creek's energy at specific locations which have been designed to withstand the erosive forces of flow conveyed through the channel. This leads to a reduction in velocity elsewhere along the creek and a corresponding reduction in erosional forces. Each structure only impacts its local area but can be used in groups to address issues over a larger length of channel. Control structures are a form of grade control and, as such, constrain vertical movement of a creek. They are well suited to cases where a channel is downcutting, degrading, or head-cutting, and so can work well for locations where the channel is constrained within a valley and the horizontal planform migration is restricted. Creek control structures must be applied with care in cases where the channel is entrenched or heavily incised, such as in the present case. Incised channels are cut off from their floodplains and, consequently, larger hydrologic events lead to deep flows within the channel which can reduce the effectiveness of the control structures.

Creek control structures can take a range of forms, including rock weirs, step-pools, log weirs, cross vanes, and rock riffles (ref. Figure 5.7). The choice of structures type depends on several features, such as the creek width, cross-section shape, channel slope, meander pattern, substrate, construction access, and available materials. Weir-pool or step-pool features are the most appropriate for steep channels.

Protection of long eroding sections with high flows requires construction of several rock weir-pool structures. The number and separation of the weirs depend on the individual size of each weir. The rock armour would be sized to withstand the stream's hydrology without reshaping. Rock weirs in wooded areas can be constructed as sloping rock armour ramps above a rock pool. The sloping design reduces the likelihood that debris will collect behind the weir.

Control structures, based on input from the Rideau Valley Conservation Authority (RVCA), are not required to consider fish passage from Mud Creek, based on the downstream stormwater management facility control structure and associated flow velocities. The control structures will require pools downstream to dissipate energy. Control structures would typically consist of stone positioned to focus low flow through the middle of the structure to prevent creek bank erosion.

Figure 5.7. Creek Control Structures, Courtesy Coldwater Consulting

vi. Channel Hardening

This alternative uses local or spot treatments to address channel issues which are small-scale or isolated. Generally, it is used to mitigate bank erosion related to infrastructure within the channel (sewer outfalls and bridges) or protect structures and property beyond the banks. Protect-in-place is preferable for locations where the channel is highly constrained by the surrounding area and therefore the channel footprint cannot be altered. Surrounding constraints could consist of development, infrastructure, topography, or sensitive habitat (trees, wetlands, etc.). In these cases, protect-in-place is a preferred solution as it limits disturbance to the surrounding area and can address immediate concerns at the selected locations. The primary limitation of this alternative is that it may not fully address long-term stability issues and future repairs, or maintenance may be required if issues persist.

The implementation of bank hardening treatment, such as armour stone or rip rap (ref. Figure 5.8), would need to be sized appropriately based on the channel velocity identified for energy dissipation. This alternative would provide bank protection for channel velocities that are greater than the velocities that could be mitigated with bioengineering or planting treatments. Appropriate bedding and backfill material would need to be selected as per the location. If sufficient space is available, channel reshaping can also be undertaken at the time of hardening to develop a suitable low flow channel with proper bank treatments and a connection to an over bank area.

Sections that need less protection could still be 'hardened' using vegetative plantings at the creek banks to provide stabilization of the bank material. Bioengineering bank stabilization is the implementation of various bank treatments to mitigate the potential for surface erosion. This treatment provides improved slope stability over other planting solutions by increasing the soil shear strength. Bioengineering bank treatments include but are not limited to:

- Brush mattress/ layers,
- Fascines
- Live staking
- Vegetated crib walls
- Vegetated angular stone

Bioengineering treatments are typically not intended to be used as a standalone treatment for most banks if toe erosion is the primary mechanism of bank erosion. When toe erosion is present, rock toe protection or coir logs may be required.

A vegetated rock buttress is a combination of strategically placed rocks and vegetation implemented on a slope to provide bank protection. The vegetation can enhance the structure of the bank treatment and assist to reduce surface erosion while providing long-term stability. The materials can be implemented on steep slopes as excavation is generally not required.

Figure 5.8. Channel Hardening using Armour Stone and Riprap

vii. Combinations

Combinations of alternatives have been considered when standalone alternatives do not provide adequate solutions to the existing creek erosion condition, and/or when there are increased benefits from using multiple approaches to addressing the erosion conditions. A combined alternatives would consider creek realignment, reshaping, hardening and structures.

Slope Stability Mitigation Alternatives

The following provides a description of the slope stability mitigation long-list of alternatives.

Slope stabilization techniques are intended to mitigate slope instability by introducing stabilizing forces or limiting the driving forces of instability and thereby increase the factor of safety for the slope to mitigate the potential for slope failure. These techniques could be applied to slope SLP 6 in place of the existing standalone stone treatment. Several stabilization techniques are available, however three (3) techniques that are preliminarily considered for this site include bioengineering, which is the application of a vegetative surface cover, re-grading the slope, and applying a mechanical stabilization solution such as a TerraFirm™ slope stabilization system.

The bioengineering slope stabilization approach is similar to the bioengineering approach for bank stabilization; however, the vegetative surface cover and plant roots are applied to reinforce the soils on the slope surface to anchor the soil. This approach can lead to an increase in the shear resistance of the soil to mitigate the potential for slope failure. The treatment techniques include but are not limited to:

- Fascines
- Live staking
- Brush layering/matting

The bioengineering treatments are typically not intended to be applied as a standalone treatment, particularly if bank erosion at the toe of the slope is a concern; toe erosion can contribute to the destabilization of the slope. Other slope stabilization techniques such as the application of a toe protection may be necessary.

Localized regrading of the slope is the process of geometrically altering and flattening the slope to improve the slope stability. This technique can be constrained by the available area at the base and top of the slope for regrading; the Ruisseau Park Ravine channel is generally at the base of the unstable slopes and private property is generally at the top of the slope, limiting the regrading application. Nevertheless, this technique should be considered to mitigate the potential for slope failure where sufficient area be available.

Mechanical slope stabilization is the process of drilling and grouting an anchoring system to the slope face which are mechanically fixed to a grid matting on the slope surface. Multiple slope stabilization vendors area available including TerraFirm™, developed by Terrafix. The design of the system would need to be confirmed with the vendor to satisfy the slope stability requirements. These systems typically allow for mature trees to remain on the face of the slope, while brush and ground cover would need to be removed prior to the installation of the system.

5.3 Evaluation of the Long List of Alternatives

5.3.1 Evaluation Criteria

As required by the Municipal Environmental Assessment process, the evaluation must consider relevant criteria beyond the functional efficacy of each alternative (i.e., ability to address the existing creek erosion and slope stability problem). The various alternatives described in the previous sections have been assessed to determine their impact and opportunity related to four environments:

(i)	**Functional Environment**	-	This environment considers the ability of the alternative to address the problem and how it may impact existing physical systems
(ii)	**Natural Environment**	-	Impacts or opportunities that an alternative may have related to the natural environment (i.e., terrestrial and aquatic habitat, water quality, etc.)

(iii)	**Social Environment**	**-**	Impacts or opportunities relating to the interaction of the community/neighbourhood with the implementation of the proposed alternative
(iv)	**Economic Environment**	-	Capital and maintenance costs associated with an alternative, both in the short term and long term

Within each environment, relevant and representative criteria must be selected for the evaluation. Each evaluation criterion needs to be assigned a significance weighting and, based on the assessment, each alternative will then be screened or short-listed. Evaluation criteria are summarized by environment as follows:

Functional Environment

Erosion Mitigation: The ability of each alternative to address the existing creek erosion condition within the Study Area. Mitigation of erosion is considered positive. Erosion is the main problem in the Study Area.

Slope Stability: The stability of some valley walls within the Study Area is considered to be at risk due to the ongoing creek bank erosion. Reducing risk to slope stability is considered positive. Slope erosion or a slope failure could result in loss of table land, both private and municipal, however no structures are considered to be in immediate danger. While additional slope stabilization efforts may be necessary pending further review, the alternatives have been assessed as to the degree of mitigating existing creek bank erosion, and the associated change in risk to slope stability.

Mitigate Impact to Existing Infrastructure: The ability of each alternative to be implemented and simultaneously mitigate any impact to existing infrastructure in the creek reaches, specifically the existing storm sewer outlets, pathways and the stormwater management facility.

Natural Environment

Terrestrial Environment: The terrestrial environment (wildlife, habitat, and vegetation) would be affected in the short-term through construction and in the long-term associated with the proposed restoration plan. Unmitigable habitat degradation is considered negative, whereas an increase in habitat quality is considered positive. The study area is heavily wooded, with the ELC delineation identifying four (4) communities within the study area; manicured landscapes, Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite, Reed Canary Grass Graminoid Meadow, and Fresh-Moist Willow Lowland Deciduous Forest. There is a potential for 15 Species at Risk (SAR) in the study area, while eight (8) SAR of those species have been identified with a moderate to high potential of occurrence in the study area.

Aquatic Environment: Aquatic habitat would be affected in the short-term through construction and in the long-term associated with any measures taken to stabilize the creek and riparian areas. Un-mitigatable habitat degradation is considered negative, whereas an increase in habitat quality is considered positive. The creek has reaches with good quality direct fish habitat, that said, fish species would have to traverse the stormwater management facility structure from Mud Creek to enter the valley, and the creek does not have upstream connections to other watercourses.

Social Environment

Construction Impacts to Private Property: The creek is adjacent to private residential properties, and as such construction may result in construction vehicles behind private properties and associated temporary nuisances such as dust, noise, etc. Any impact to private property is considered negative. In addition to construction noise and dust, the temporary loss of privacy would be considered negative.

Public Safety: The ongoing erosion can compromise the stability of creek banks and valley walls which are proximate to rear yards and pedestrian trails through the Study Area. Improved public safety would be considered positive. Public safety is considered paramount on any municipal lands and as such this criterion has thus been assigned a high significance.

Economic Environment

Capital Cost: Capital costs include all costs associated with the design, permitting and construction of the alternative, including any land costs. High costs are considered negative, low costs are considered positive.

Maintenance Cost: Maintenance costs are associated with work required to maintain the design condition or function of the alternative. High costs are considered negative, low costs are considered positive. This criterion has been assigned a moderate significance.

5.4 Evaluation of the Long List of Alternatives

In order to evaluate the long-list of alternatives, the 'impact' or 'benefit' of each alternative on a given criterion has been determined. Impact has been determined to be either Positive, Positive/Neutral, Neutral, Negative/Neutral or Negative relative to the existing (baseline) condition. For example, **Alternative 3: Creek Realignment** would have a positive impact on the Erosion Mitigation criterion, while it would have a negative impact on the Terrestrial Environment criterion. The impact is deemed to be neutral where the alternative does not change the existing condition.

Table 5.3 summarizes the evaluation of each alternative relative to the criteria discussed above and provides the impact score and associated justification. The Summary row indicates if the alternative has been Screened Out or Short-listed. Generally, if an alternative has been deemed to be either infeasible, ineffective with respect to the Problem Statement, or to have significant and unjustifiable impacts, it has been screened out. The Long-List Screening has screened out six (6)) alternatives and short-listed three (3) combination alternatives. Justification for the screened-out alternatives is provided in the following paragraphs. Short-listed alternatives are discussed further in Section 5.5.

Alternative 1: Do Nothing has been screened out as it does not address the problem statement.

Alternative 2: Stormwater Management and Low Impact Development Measures has been screened out as a standalone alternative due to the limited number of suitable sites for new stormwater management facilities proximate to major existing storm sewer outfalls. New facilities at these locations would require underground storage within City parks, which would be costly and highly disruptive. The one (1) City Park, Blue Willow Park would have space to implement an underground storage tank, however at a high cost to the City and it would only reduce peak flows to one (1) of the three (3) storm sewer outlets.

An underground storage tank has been input into the PCSWMM model to simulate the proposed storage facility upstream of the north outfalls. The peak flow rate results for the City's 2-100 year SCS storm events are provided within **Table 5.1** with a comparison to the existing condition peak flow rates.

Table 5.1. SWM Control Peak Flow Rate Comparison to Existing Conditions

Scenario	SWM Facility Peak Flow Rates (m^3/s) 2 Year	SWM Facility Peak Flow Rates (m^3/s) 5 Year	SWM Facility Peak Flow Rates (m^3/s) 10 Year	SWM Facility Peak Flow Rates (m^3/s) 25 Year	SWM Facility Peak Flow Rates (m^3/s) 50 Year	SWM Facility Peak Flow Rates (m^3/s) 100 Year
Existing Conditions	4.46	6.93	8.27	9.35	9.94	10.55
Proposed SWM Facility	3.37	5.23	5.93	7.00	7.66	8.18
Difference (m^3/s)	-1.09	-1.70	-2.35	-2.35	-2.28	-2.37
Difference (%)	-24.5	-24.5	-28.4	-25.1	-22.9	-22.5

As shown within Table 5.1, the peak flow rates would be reduced for all storm events with peak flow rate decreases ranging from 1.09 m^3/s (+/-) to 2.37 m^3/s (+/-), or 22.5 % (+/-) to 28.4 % (+/-). The storage tank volume for the 100 year storm event has been simulated as 7800 m^3 (+/-), with a tank height of 2.5 m (+/-). The peak flow rate decreases provided with this alternative would benefit Blue Willow Creek due to the decreased potential for erosive peak flow rates. The cost of implementing an underground storage tank has been estimated at $3,121,000 based on a unitary rate of $400/$m^3$ for a plastic storage tank.

Given the obvious constraints and cost of implementation, an underground storage facility is not considered feasible to implement as part of the current study.

Low Impact Development Best Management Practices (LID BMPs) could potentially be implemented throughout the contributing drainage area as means of capturing runoff for infiltration with the use of source controls. As common practice, LID BMPs are typically designed to capture and infiltrate runoff from the more frequent storm events, such as a 25 mm storm event up to the 5 year storm event. The PCSWMM model has been revised to simulate the potential implementation of LID BMP source controls with a 30 % uptake of lot level (private property) source controls and 50 % uptake of right-of-way (public property) source controls. These implementation rates have been based on the capture of the 25 mm and simulated for the 2-100 year storm events to compare the peak flow rates at the north outfall to the existing conditions peak flow rates; this comparison is provided in **Table 5.2**.

Table 5.2. LID BMP Peak Flow Rate Comparison to Existing Conditions

Scenario	SWM Facility Peak Flow Rates (m^3/s) 2 Year	SWM Facility Peak Flow Rates (m^3/s) 5 Year	SWM Facility Peak Flow Rates (m^3/s) 10 Year	SWM Facility Peak Flow Rates (m^3/s) 25 Year	SWM Facility Peak Flow Rates (m^3/s) 50 Year	SWM Facility Peak Flow Rates (m^3/s) 100 Year
Existing Conditions	4.46	6.93	8.27	9.35	9.94	10.55
Proposed SWM Facility	4.26	6.82	8.19	9.30	9.90	10.51
Difference (m3/s)	-0.20	-0.10	-0.08	-0.05	-0.03	-0.04
Difference (%)	-4.5	-1.5	-1.0	-0.5	-0.3	-0.4

As shown within Table 5.2, the implementation of the source controls would have minimal impact on the peak flow rates of the 2-100 year storm event at the north outfall to Blue Willow Creek; the simulated source controls would provide 4.5 % to 0.3 % (+/-) reduction in peak flow rates. In addition to not providing a significant reduction in peak flow rates, this alternative would take a number of years to implement and would not provide short-term benefit to mitigate the erosion concerns within Blue Willow Creek.

Low Impact Development Best Management Practices have been screened out as a standalone alternative. The city does not have the ability to implement source controls over the contributing area within a reasonable timeline to reduce the existing creek erosion problem. The city will still promote the use of LID BMPs in conjunction with other alternatives. Further assessment of LID BMPs has been provided in the short-listed assessment.

Alternative 3: Creek Realignment has been screened out as a standalone alternative, as it would not address the existing slope stability concerns and would not address the vertical slope that is required by the creek within the confines of the valley system. Creek realignment would also be limited by the steep valley sides and potential impacts to existing stable creek banks and valley slopes. The alternative will be considered within a Combined Alternative.

Alternative 4: Creek Partial Enclosure has been screened out as an alternative in constructing a creek enclosure as the creek banks and valley slopes would have to be significantly disturbed by the excavation to incorporate a flow conduit. Based on the flow regime within the valley, the flow conduit or enclosure would not convey all flows and would still result in erosive conditions for infrequent storm events. In addition, the existing creek erosion and slope stability sites would still require remediation, along with the significant construction impacts.

A creek enclosure would also impact aquatic habitat conditions, with aquatic species being potentially being pulled into the enclosure.

Based on the significant construction disturbance anticipated and that a flow enclosure or conduit would not address the existing creek erosion conditions and valley slope stability concerns, this alternative has been screened from further consideration.

Alternative 5: Creek Control Structures has been screened out as a standalone alternative, based on it not addressing the existing creek bank erosion and slope stability concerns. The creek control structures would dissipate creek energy to prevent future erosion of the creek banks but would not mitigate the existing creek erosion conditions. The alternative will be considered within a Combined Alternative.

Alternative 6: Channel Hardening has been screened out as a standalone alternative, as it would not dissipate the creek energy, would require significant lengths of channel hardening at the upstream end of the creek. That said channel hardening would protect the creek banks from further erosion and would allow toe protection to the valley slopes. The alternative will be considered within a Combined Alternative.

Table 5.3. High Level Screening of Long List of Alternatives

Evaluation Environment	Evaluation Criteria	Alternative 1: Do Nothing		Alternative 2: SWM and LID		Alternative 3: Creek Realignment		Alternative 4: Creek Partial Enclosure		Alternative 5: Creek Control Structures		Alternative 6: Channel Hardening		Combination of Alternatives 7	
Functional	Erosion Mitigation	Existing erosion unmitigated	-2	Limited short-term benefits. Medium degree of mitigation if implemented for all existing development without SWM	2	Realigned creek sections would be designed to be stable within the existing flow regime but would be limited by existing steep valley slopes.	2	Creek enclosure would reduce creek erosion risk, by reducing flows across the full flow regime.	2	Creek control structures would reduce creek erosion risk, by reducing flow regime energy.	2	Isolated hardening works would stabilize the existing channel in key locations but may result in erosion in new locations.	-1	Creek control structures coupled with localized creek rehabilitation, hardening and realignment would reduce creek erosion risk, by reducing flow regime energy.	2
Functional	Slope Stability[1]	Existing risk to slope stability unmitigated by not addressing creek bank erosion	-2	Limited short-term benefits. Reduced creek bank erosion risk would result in reduced risk to further slope instability	1	Creek realignment would reduce the risk to slope stability by facilitating creek bank stabilization and slope stability works and moving the creek away from slopes of concern	2	Future slope stability risk would be reduced, but the existing slope stability risk would not be mitigated due to the eroded creek banks.	1	Future slope stability risk would be reduced, but the existing slope stability risk would not be mitigated, due to the existing eroded creek banks	2	Toe armouring would mitigate the risk to existing slope stability but may result in other slopes being undermined.	-1	Future slope stability risk would be reduced and would require localized bank measures and localized creek realignment to protect toe of slope.	2
Functional	Impact to Infrastructure	Existing risk to infrastructure unmitigated	-2	Limited short-term benefits. Reduced erosion resulting from would reduce risk to infrastructure	2	Creek realignment to be established to protect existing trail system	2	The creek enclosure system itself, would become infrastructure that could be impacted. Impacts to existing infrastructure should be limited	1	Reduced erosion would reduce risk to infrastructure	1	Protect in place would mitigate the risk of impact to existing infrastructure	2	Reduced erosion risk and with bank protection measures and localized creek bank measures and creek realignment, existing infrastructure would be protected	2
Natural	Terrestrial Environment	No direct impacts. Potential for continued loss of riparian and tableland habitat and species	0	Limited short-term benefits. Reduced erosion and would reduce risk of further loss of riparian and tableland habitat and species.		Loss of mature trees. Short term impact to riparian species. Long term stabilization of riparian habitat.	1	Terrestrial impacts would be significant from implementing the enclosure both in the short and long-term.	-2	Reduced erosion and would reduce risk of further loss of riparian and tableland habitat and species	1	Minor Loss of mature trees. Short term impact to riparian species. Long term stabilization of riparian habitat.	1	Loss of mature trees. Short term impact to riparian species. Long term stabilization of riparian habitat	1

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Evaluation Environment	Evaluation Criteria	Alternative 1: Do Nothing		Alternative 2: SWM and LID		Alternative 3: Creek Realignment		Alternative 4: Creek Partial Enclosure		Alternative 5: Creek Control Structures		Alternative 6: Channel Hardening		Combination of Alternatives 7	
Natural	Aquatic Environment	Continued degradation of habitat	-2	Limited short-term benefits. Reduced erosion and improved water quality would benefit aquatic habitat.	2	Opportunity for improved habitat in upper reaches. No water quality benefits.	2	The creek enclosure would have a negative impact on aquatic habitat based on flow above a threshold being conveyed through a pipe	-2	Depending on structure form and function, the structures may or may not impact aquatic habitat. Pool creation would be a benefit.	1	Some opportunity for improved habitat in upper reaches. No water quality benefits	1	Depending on structure form and function of structures and armouring, aquatic habitat may be impacted.	1
Social	Construction	No impact	2	High social impacts. Construction on both public and private properties	-2	Large construction area/duration proximate to private property	-2	Significant construction impacts throughout the ravine, proximate to private property	-2	Localized construction impacts where structures are to be installed.	-1	Localized construction impacts where channel hardening to be installed	-1	Localized construction impacts where structures, armouring are to be installed. Additional impacts from localized creek bank measures and realignment.	-2
Social	Public Safety	Existing risk associated with erosion adjacent to slopes and trails unmitigated	-2	Limited short-term benefits. Mitigated creek erosion reduces risk to slopes and public	1	Creek moved away from slopes and trails, risk reduced to public	2	Future creek erosion risk reduced. Existing erosion still unmitigated. Risk of high flows to inlet and public safety	-2	Mitigated creek erosion reduces risk to slopes and public	1	Mitigated creek erosion reduces risk to slopes and public	1	Mitigated creek erosion reduces risk to slopes and public.	2
Economic	Capital Cost	None	2	High	-2	High	-2	High	-2	Medium	1	Medium	1	High	-2
Economic	Maintenance Cost	Potential for maintenance related to ongoing erosion (e.g., fallen tree removal, sediment accumulation, etc.) and slope failure mitigation.	-1	Maintenance of source controls challenging, reconstruction would be required at end of design life	-2	Natural channel design requires minimal maintenance	2	Would require continual maintenance and eventual replacement at end-of-life cycle.	-2	Control structures should require moderate maintenance	2	Channel hardening should require minimal maintenance	2	Control structures, creek bank measures and realignment should require minimal maintenance	2

Evaluation Environment	Evaluation Criteria	Alternative 1: Do Nothing		Alternative 2: SWM and LID		Alternative 3: Creek Realignment		Alternative 4: Creek Partial Enclosure		Alternative 5: Creek Control Structures		Alternative 6: Channel Hardening		Combination of Alternatives 7	
Summary		Screened Out. Not effective.		SWM Storage Screened Out. Source Controls (LID BMPs) short-listed to compliment a Preferred Combination Alternative		Short-Listed in combination with other alternatives (Alternatives 7a-7c)		Screened Out		Short-Listed in combination with other alternatives (Alternatives 7a-7c		Short-Listed in combination with other alternatives (Alternatives 7a-7c		**Short-listed for further assessment**	

Note 1: The evaluation provided within this table is strictly based on creek improvement measures, therefore, the slope stability evaluation criteria are pertinent to only those alternatives which may reduce the risk of slope instability. Additional slope stabilization efforts may be required, pending further review.

Positive	**2**	**Neutral/Positive**	**1**	**Neutral**	**0**	**Neutral/Negative**	**-1**	**Negative**	**-2**

Table 5.3 provides a high-level screening assessment of seven (7) alternatives. As a result of the screening process, the following has been determined:

- Alternative 1 Do Nothing has been screened from further consideration as it would not address the existing creek bank erosion conditions and would not help facilitate slope mitigation works.
- Alternative 2 Stormwater Management has been screened out from further consideration as it would not address existing creek bank erosion conditions, that said, LID BMPs or source controls could be implemented with a combined alternative. The LID BMPs are viewed as a complimentary long-term alternative.
- Alternative 3 Creek Realignment would not fully address the creek bank erosion conditions, as such it has been considered as part of a combined alternative.
- Alternative 4 Creek Partial Enclosure has been screened out for further consideration, based not addressing the existing creek bank erosion conditions and would result in long term maintenance requirements and eventual replacement.
- Alternative 5 Creek Control Structures would reduce creek erosion risk, by reducing flow regime energy, but would not fully address the existing creek bank erosion conditions, as such it has been considered as part of a combined alternative.
- Alternative 6 Channel Hardening would stabilize the existing channel in key locations, but may result in erosion in new locations, as such the alternative has been considered as part of a combined alternative.
- Alternative 7 Combination of Alternatives, will consider various aspects of the alternatives indicated to be assessed in a combined alternative. The combined alternatives have been advanced for further consideration and are assessed in detail within the following sections.

5.5 Assessment of the Short List of Combination Alternatives

A combination of Alternatives (3, 5, and 6) has been short listed. The following provides a description of three (3) combination alternatives and information on opportunities and constraints related to each alternative the assessment of each alternative leading to selection of the Preferred Alternative.

5.5.1 Alternative 7a: Realignment, Reshaping and Hardening (Hardening Focus)

Alternative 7a combines realignment of the creek to address existing slope stability issues and reshaping and hardening of the creek cross-section to address erosion issues.

The creek would be realigned at two points to allow slope stabilization works to be constructed (see **Figure 5.9.**). The two realignments each provide space to permit construction of a 2:1 slope: SLP2 and SLP4 are addressed by a 56 m long realignment; SLP6 and SLP7 are addressed by a 41 m long realignment. The creek cross-section within each realigned section would be constructed as a trapezoidal section with a 4 m wide base and the banks reduced to a 2:1 (H:V) side slope. The lower side slopes will lead to an increased flow cross-section during large rainfall events and the associated decrease in flow velocity. A 1 m wide low flow channel would be constructed in the channel base to ensure sufficient flow depth during low flow periods. Isometric plots of the channel corridor as it presently exists and of Alternative 7a with the proposed realignments are shown in **Figure 5.11**.

Figure 5.10. Proposed Works for Alternative 7a

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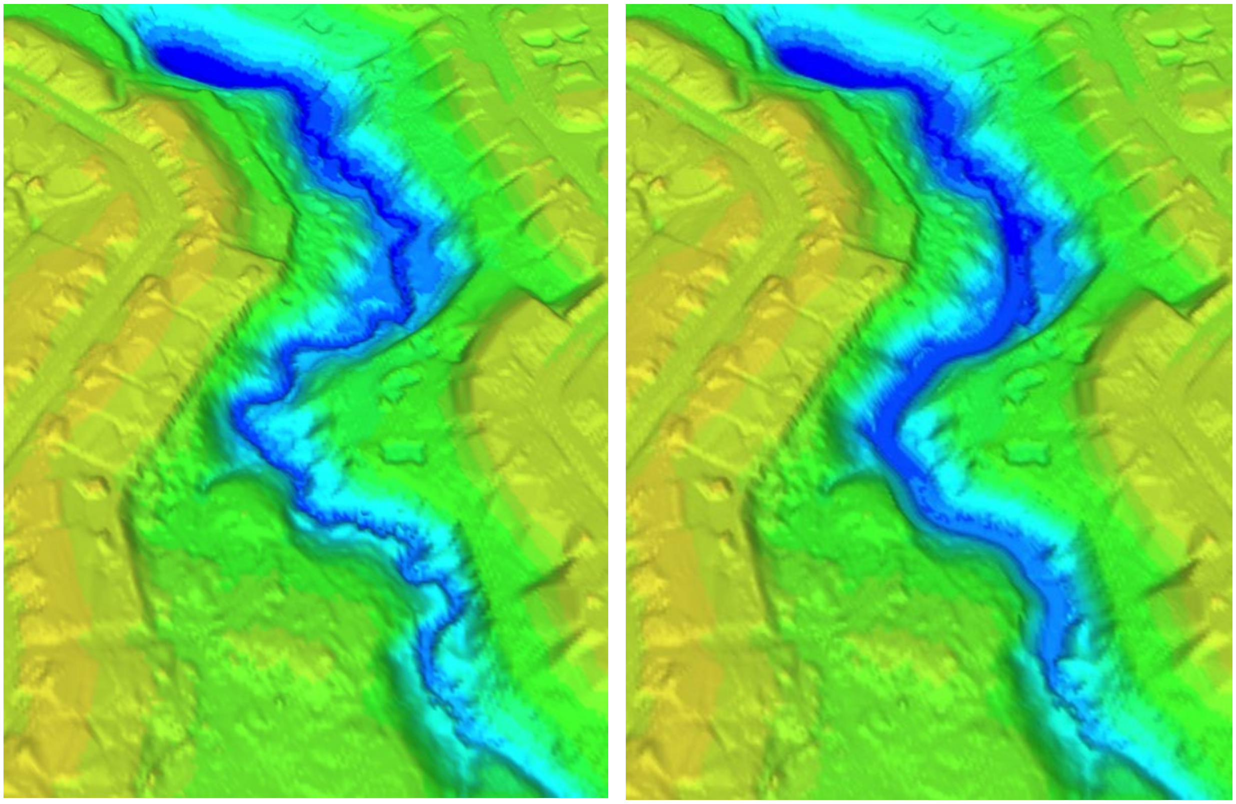

Figure 5.11. Topography, Looking Southeast: Existing Channel (left), Alternative 7a (right)

Hydraulic modelling of the widened and reshaped channel was conducted using HEC-RAS. This is a somewhat complicated process because the controlled outflow from the creek results in a backwater effect for many large rainfall events; therefore, boundary conditions for the model (water levels at the Blue Willow outflow and discharge at the inflow culverts) were extracted from the 40-year PCSWMM simulation. Only events during which the discharge exceed 3 m^3/s were included in the analysis, which resulted in approximately individual 30 storms. Analysis of the simulations showed that even with the increased cross-section area, the shears in the channel were found to exceed the critical value for the native substrate an unacceptable amount; consequently, channel realignment by itself is inadequate and additional measures are required to limit erosion.

The erosion limiting approach adopted in this alternative is channel hardening. The creek bed and banks would be lined with void-filled riprap (VFR) (ref. Appendix H) and the upper banks with vegetated riprap. VFR is a densely packed mix of rock, cobbles, gravels, sands, and soil that is designed to emulate natural rock riffle material. The finer fractions of the well-graded mix fill all voids and act as an internal filter eliminating the requirement of a separate bedding layer between subgrade and applied rock. This also eliminates the need for geotextiles. Also, the material can support riparian vegetation. The gradation of the VFR would be varied along the channel based on the local shear stress. The widened channel allows a finer, cobble-sized gradation to be used in the realigned sections than in the adjacent sections. Vegetated riprap is a technique widely applied in stream restoration projects wherein live-stake plantings are incorporated into the stone armouring. Stakes can be placed during stone placement or can be driven into the bank in gaps in the riprap after stone placement.

The construction process would start with the clean-up of the existing stream corridor and removal of deadfall and endangered trees. Trees may also need to be removed where the creek is to be widened or realigned. The realigned sections would be constructed with a widened and reshaped cross-section and would be armoured with VFR. The remaining sections of Reaches 1 to 3 would require bank reshaping and armouring with VFR, although Reach 3 works would be on a smaller scale. No works are proposed for Reach 4. The total estimated project cost is $3.1M which is based on a Class D cost estimate provided within Appendix I.

This approach, especially the use of VFR and vegetated riprap, mimics what is commonly found in steep gravel or cobble streams which, in general, have slopes near to that of the site. This Alternative would support a healthy aquatic habitat and provide a reliable approach for the long-term protection of the stream, park, and infrastructure.

5.5.2 Alternative 7b: Realignment, Structures and Hardening (Structures Focus)

Alternative 7b involves a combination of the control structures of Alternative 5 with the realignment and naturalized armouring of Alternative 7a.

Control structures, in the form of weir-pool features, would be used to dissipate the stream's energy at specific sites rather than along the entire stream length. The aim of this is to reduce the requirements for channel hardening. Weir-pool, or step-pool, features are well suited to channels with relatively steep gradients. Because of the wooded location, each weir would be constructed as a sloping rock armour ramp above a rock pool. The construction of the ramps would not only reduce the slope across the section between each weir, thus reducing erosive forces over this area, but would reduce debris collection. The rock armour for the ramps and banks would be sized to withstand the stream's extreme hydrology without reshaping.

As was described in Alternative 7a, this alternative calls for the creek to be realigned at two points to allow slope protection works to be constructed (ref. **Figure 5.12**). The two realignments each provide space to permit construction of a 2:1 slope: SLP2 and SLP4 are addressed by the 56 m long Realignment 1; SLP6 and SLP7 by the 41 m long Realignment 2. The creek cross-section within each realigned section would be constructed as a trapezoidal section with a 4 m wide base and the banks reduced to a 2:1 (H:V) side slope. The lower side slopes will lead to an increased cross-section during large rainfall events and the associated decrease in flow velocity. A 1 m wide low flow channel would be constructed in the channel base to ensure sufficient flow depth during low flow periods. Isometric plots of the channel corridor as it presently exists and with the proposed realignments are shown in **Figure 5.14**.

wood.

Figure 5.13, Proposed Works for Alternative 7b

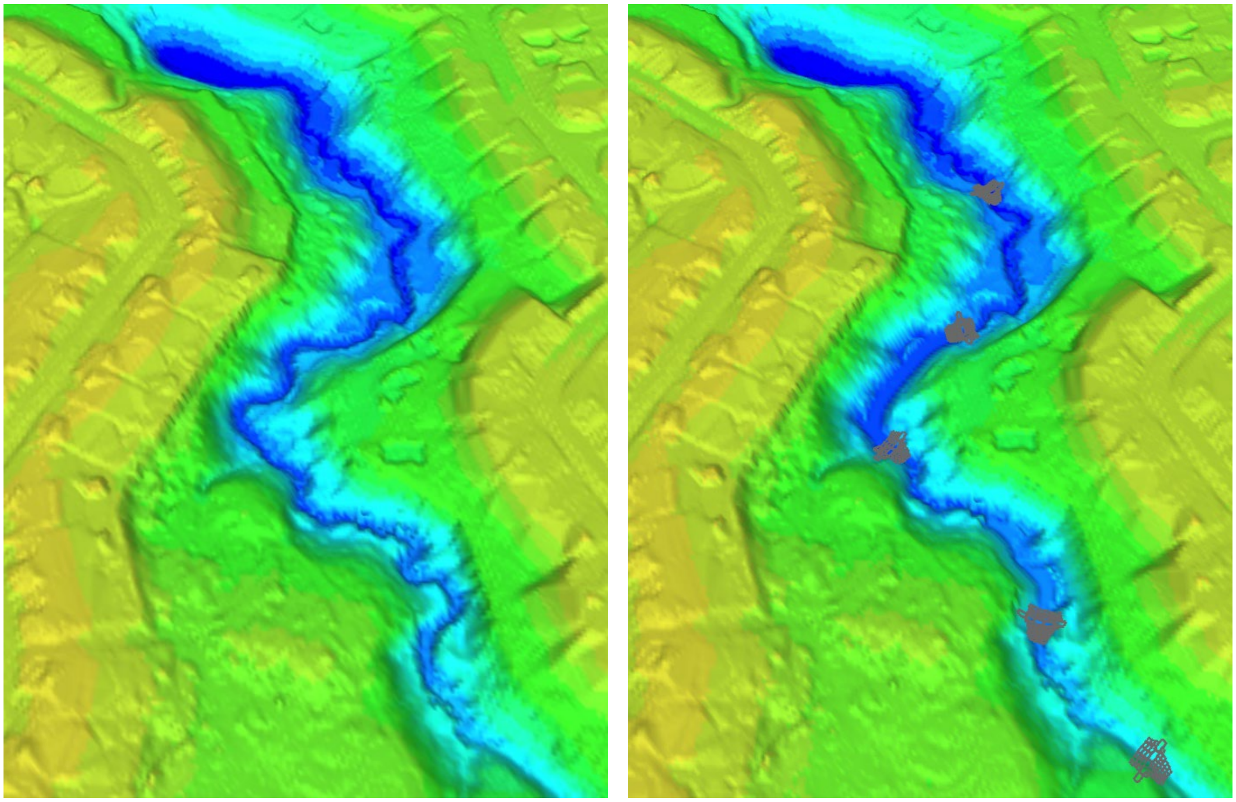

Figure 5.14. Topography, Looking Southeast: Existing Channel (left); Alternative 7b (right)

Hydraulic modelling of Alternative 7b was conducted using HEC-RAS and the same 30 rainfall event sequence approach described in the Alternative 7a section. Analysis of the simulations showed that because of the high discharge the reshaping and the weirs would not sufficiently control shears and some hardening would be required in certain sections to limit erosion, although the required protection would not need to be as robust or as extensive as that required with Alternative 7a (gravel-sized as opposed to cobble-sized required for Alternative 7a). The stream banks in the areas adjacent to the realigned sections would be reshaped to a 2:1 (H:V) side slope and lined with VFR. Vegetated riprap would be used where appropriate.

The construction process would start with the clean-up of the existing stream corridor and removal of dead fall and endangered trees. Trees may also need to be removed where the creek is to be realigned. Reach 1 would see some spot repairs to address localized bank stability. Much of Reach 2 would require reshaping and armouring. During this process, the creek would be realigned to move it away from slopes SLP2-SLP4 and slopes SLP6-SLP7. This would be followed by the construction of five weir-pool structures. The weirs would be approximately 1.5 m in height and would be positioned as shown in Figure 5.13.

The structures (ref. Figure 5.13) would be designed with their crests higher at sides, lower in middle. (The details of the design would be developed during the design phase of the project.) The upstream slope would be mild (5H:1V) to minimize debris entrapment and all open joints would be filled with river gravel (chinked) to minimize debris entrapment. The pool below each weir would be armoured to prevent scour. The design would resist flanking protection by embedding armour stone 1m horizontally into each bank. The stone size would be approximately 100kg-500kg, which would exceed the hydraulic stone stability criteria. The weir-pool structures would be required in Reaches 1, 2 and 3. No works are proposed for Reach 4.

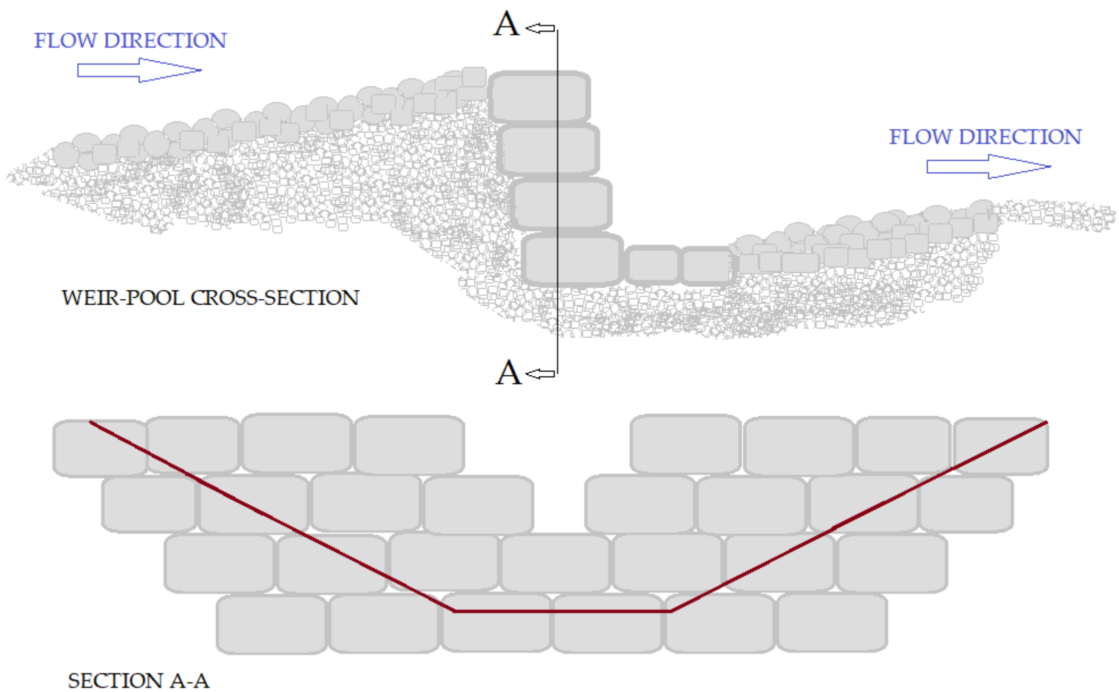

Figure 5.15. Weir pool cross-section (top and section view (bottom)

This approach mimics what is commonly found in steep gravel or cobble streams which, in general, have slopes near to that of Blue Willow Creek. An example of the impact of the weir-pools on the flows in the creek is illustrated in Figure 5.14, which shows the water level in the creek during the early part of a rainfall event at the present time and with the Alt. 7b design (the weir-pool structures locations are shown by the vertical lines). The weirs reduce the surface slope and, consequently, the flow speed and erosion potential of the creek. A design with a greater number of smaller weirs was also examined but it too was found to require channel hardening. In both cases, slope reductions produced by the weirs were too small to overcome the high discharge confined in the incised channel. The use of weir-pool structures would, however, result in less extensive and less robust hardening than would be required under Alternative 7a (gravel-sized as opposed to cobble-sized). The alternative's use of VFR and vegetated riprap would be a suitable alternative for the long-term protection of the stream and could support a healthy aquatic habitat.

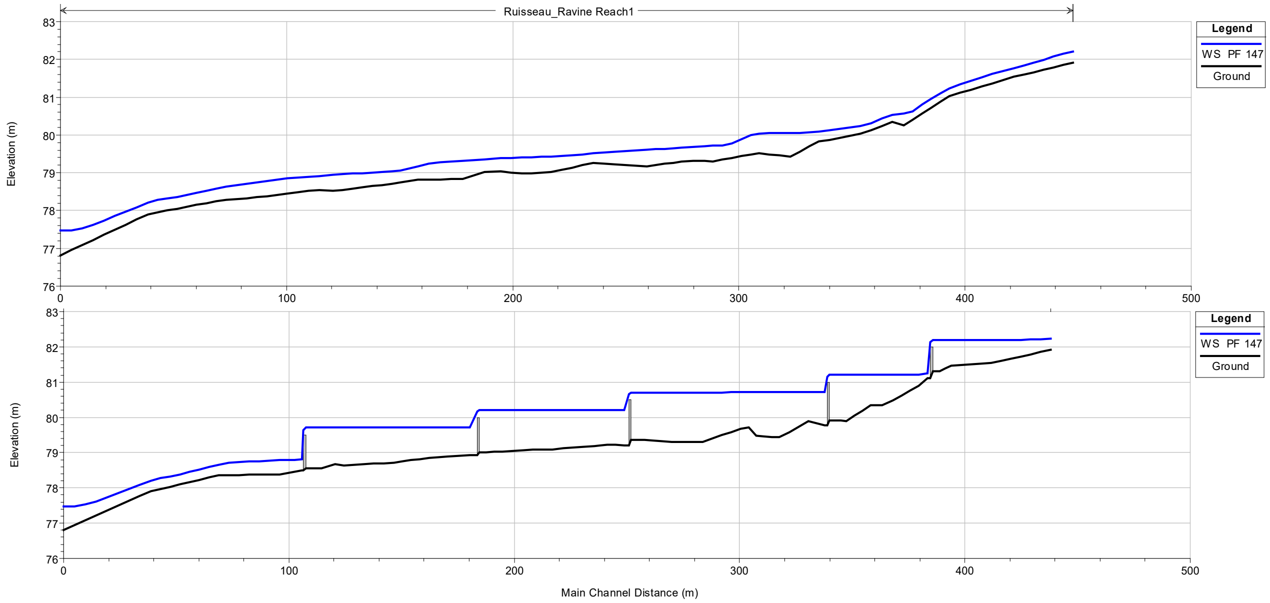

Figure 5.16. Typical Event Flow Pattern: Existing Conditions (top); Alternative 7b (bottom)

5.5.3 Alternative 7c: Realignment, Structures and Naturalization (Adaptation Focus)

Alternative 7c involves a combination of the reshaping and realignment of Alternative 7 with the use of habitat features and natural construction materials to restrain the stream's morphological evolution. Unlike Alternatives 7a and 7b, this design would allow for some stream evolution to continue, albeit in a controlled manner.

The existing creek channel is incised and thus is not connected to a floodplain which would lead to a reduction in shear stresses during extreme events. This results in a concentration of flow within the channel and excessive erosion of the bed and banks. This erosion will be addressed in the two realigned sections (realigned to allow slope protection works to be constructed) where the stream would be widened, reshaped, and hardened. Other eroding areas would be fixed in place using log weirs and cross-vanes (ref. Figures 5.15 and 5.16 respectively). The approach in Alternative 7c is to protect those locations which are presently eroding while limiting disturbance of adjacent areas. This approach allows for continued evolution; therefore, an adaptive management approach will be required going forward since it is likely that some additional works would need to be undertaken over time.

Figure 5.17. Log Weir, Courtesy Cambria County Conservation

Figure 5.18. Cross Vane, Courtesy Coldwater Consulting

The proposed works are shown in **Figure 5.19.** The construction process would start with the clean-up of the existing stream corridor and removal of dead fall and endangered trees. Trees may also need to be removed where the creek is to be realigned. Reach 2 would see the construction of the two realigned sections to move the creek away from slopes SLP2-SLP4 and slopes SLP6-SLP7 so that slope stabilization works could be constructed. The realigned sections would be widened, banks flattened to 2H:1V and hardened with VFR. Cross-vanes (see Appendix H) and log weirs would be constructed of natural materials to provide flow resistance during high flow events would be constructed at the upstream ends of the two realigned sections and at several other locations. These features, like to more robust weir-pools of Alternative 7b, lower the water slope over the reach upstream and reduce erosion potential.

The bed in Reach 1 has previously been armoured with 150 mm riprap which has been washed out and is scattered downstream. A set of log weirs is proposed here to reduce localized bank erosion issues. Bends in Reach 3 would be stabilized using log walls. No works are proposed for Reach 4. The total estimated project cost is $2.4M which is based on a Class D cost estimate provided within Appendix I.

Alternative 7c aims to protect only those locations which are presently eroding while limiting negative impacts on adjacent areas. Because there is less intervention and modification to the creek than the other combination Alternatives, the creek will continue to naturally evolve. The creek will need to be monitored to track any changes, verify that the protective measures remain effective, and to enable proactive measures to be taken, if required. It should be expected that some additional works outside of the realigned sections will need to be undertaken in the future. Isometric plots of the channel corridor as it presently exists and with the proposed realignments are shown in **Figure 5.20**.

Figure 5.19. Proposed Works for Alternative 7c

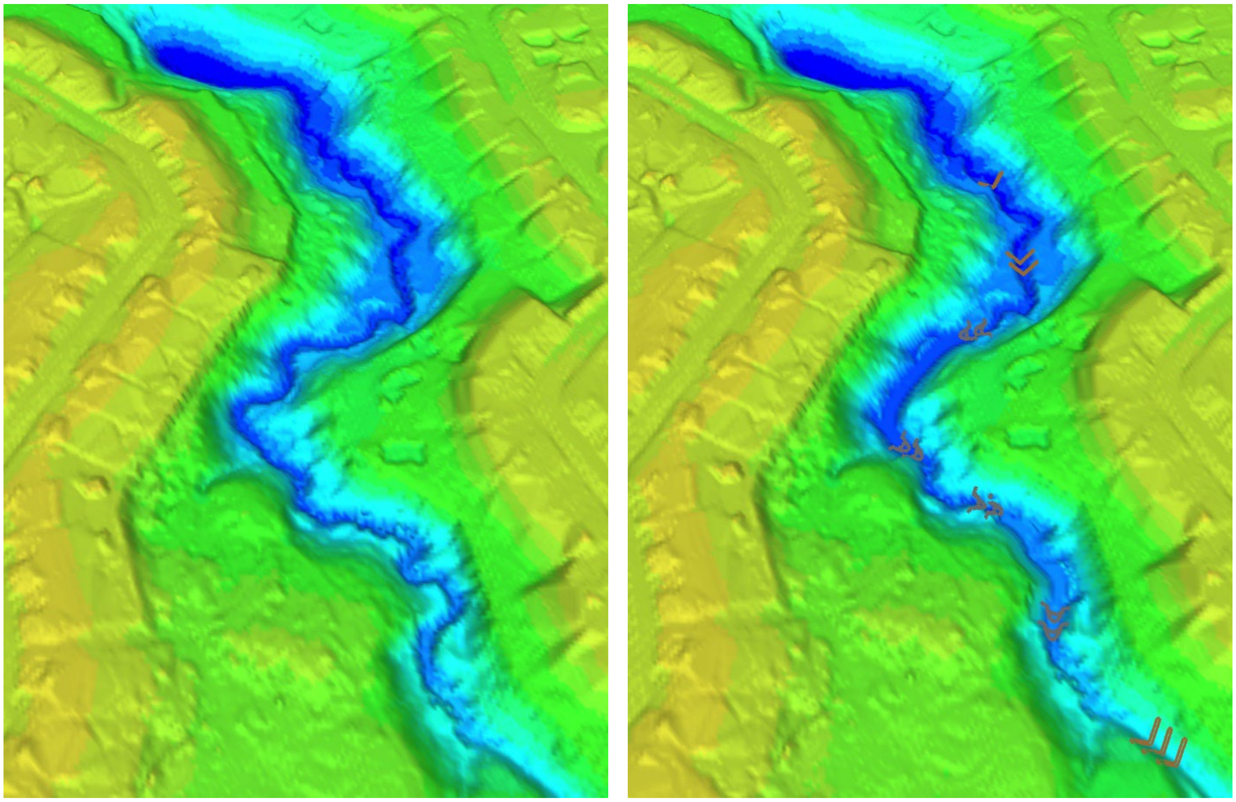

Figure 5.20. Topography, Looking Southeast: Existing Channel (left); Alternative 7c (right)

5.6 Comparison of Short List of Combination Alternatives

Alternatives 7a-7c have been compared in Tables 5.4 and Table 5.5. Table 5.4 indicates the locations of the proposed creek bank erosion alternative measures, while Table 5.5 provides the positives and negatives of each alternative. The stations identified within Table 5.4 are shown on Figure 5.19.

Table 5.4. Comparison of Short-List Alternatives

Section	Distance	Alt. 7a (Hardening Focus)	Alt. 7b (Structures Focus)	Alt. 7c (Adaptation Focus)
A	0 m – 100 m	• Reshaping cross-section to 2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap[1].	• Construct pool-weir structure at 54 m. • Spot repairs at selected locations.	• Construct multiple log weir structures near 55 m.
B	100 m – 160 m	• Realign channel away from slopes SLP2-SLP4. • Construct new cross-section with 2:1 bank	• Realign channel away from slopes SLP2-SLP4. • Construct new cross-section with 2:1 bank	• Realign channel away from slopes SLP2-SLP4. • Construct new cross-section with

Section	Distance	Alt. 7a (Hardening Focus)	Alt. 7b (Structures Focus)	Alt. 7c (Adaptation Focus)
		slope and low-flow channel. • Harden profile using void-filled riprap.	slope and low-flow channel. • Harden profile using void-filled riprap[1]. • Construct pool-weir structure at 100 m.	2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap[1]. • Construct multiple cross-vane structures near 100 m and 160 m.
C	160 m – 190 m	• Reshaping cross-section to 2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap.	• Reshaping cross-section to 2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap.	• Reshaping cross-section to 2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap.
D	190 m – 270 m	• Realign channel away from slopes SLP6-SLP7. • Construct new cross-section with 2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap.	• Construct pool-weir structures at 190 m and 270 m. • Realign channel away from slopes SLP6-SLP7. • Construct new cross-section with 2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap.	• Construct multiple cross-vane structures near 190 m and 270 m. • Realign channel away from slopes SLP6-SLP7. • Construct new cross-section with 2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap.
E	270 m – 350 m	• Reshaping cross-section to 2:1 bank slope and low-flow channel. • Harden profile using void-filled riprap.	• Construct pool-weir structure at 350 m. • Construct bend protection at 295 m and 335 m. • Spot repairs at selected locations.	• Construct multiple log weir structures near 310 m and 350 m. • Construct bend protection at 295 m and 335 m.
F	350 m – 470 m	• No works proposed	• No works proposed	• No works proposed

[1] Void-filled riprap composition varies according to design conditions (e.g., void-filled riprap used for Alt. 7a would be composed of larger material than that used in Alt. 7b).

Figure 5.21. Creek Sections used in Table 5.4

Table 5.5. Comparison of Positives and Negatives for Short-Listed Alternatives

Alternatives	Positives	Negatives	Costs ($)1
Alternative 7A (Hardening Focus)	• Creates the space necessary for the slope stabilization works to be constructed at SLP2, SLP4, SLP6 and SLP7 • The approach would eliminate stream evolution and meandering, thus fixing the stream in its present location. • The approach would eliminate erosion of the toe of the slopes and prevent over-steepening of the valley slopes.	• The approach would require significant disturbance to the valley, vegetation, and aquatic habitat during construction • Many mature trees would need to be removed. • The capital costs would be high • Construction access would be difficult.	$3.1M

wood.

Alternatives	Positives	Negatives	Costs ($)1
	• The approach would not require significant monitoring or maintenance.		
Alternative 7B	• Creates the space necessary for the slope stabilization works to be constructed at SLP2, SLP4, SLP6 and SLP7 • The approach would eliminate stream reshaping and meandering, thus fixing the stream in its present location. • The approach would eliminate erosion of the toe of the slopes and prevent over-steepening of the valley slopes. • Some natural features would be incorporated into the design and would likely be favourably received by the local community. • The approach would not require significant monitoring or maintenance.	• The capital costs would be high • The approach would require significant disturbance to the valley, vegetation, and aquatic habitat during construction. • Some mature trees would need to be removed. • Construction access would be difficult.	$2.6M
Alternative 7C	• Creates the space necessary for the slope stabilization works to be constructed at SLP2, SLP4, SLP6 and SLP7 • The approach would halt stream reshaping and meandering at the presently endangered locations. • The approach would eliminate erosion of the toe of the presently endangered slopes. • The adaptive approach would require less disturbance to the valley, vegetation, and aquatic habitat during construction than other Alternatives.	• Fewer mature trees would need to be removed. • Natural erosion processes will continue in non-critical areas. • Areas not presently eroding may develop problems. • Ongoing monitoring and adaptation would be required. • The monitoring and maintenance costs would be high.	$2.4M

Alternatives	Positives	Negatives	Costs ($)1
	• Natural channel features would be incorporated into the design and would likely be favourably received by the local community. • The capital costs would be lower than the other two combination Alternatives.	• Construction access may be difficult at certain locations.	

1. Costs based on Class D are provided in Appendix I.

5.7 Slope Stability Measures for Short Listed Combination Alternatives

Based on Section 5.1, three (3) techniques have been preliminarily considered for this site, including toe-protection, localized slope regrading to 2H:1V, and mechanical stabilization solutions (e.g. TerraFirm™) to strengthen the surficial erosion control and vegetation protection on Slope No. SL6, if required. The proposed slope mitigation works would be consistent with all three (3) short listed alternatives described above.

Based on the conceptual plans described for each of the short-listed alternatives, the existing creek would be realigned at two areas for the slope stabilization work; this includes construction of 2H:1V slopes adjacent to SLP2 and SLP4 for approximate 56 m in length (Realignment 1) and 2H:1V slopes adjacent to SLP6 and SLP7 for approximate 41 m in length (Realignment 2). Accordingly, two sections of the existing creek would be backfilled along the existing slope toes to the new grade elevations after completion of the planned realignment work. Issues of the potential slope instability induced by the erosion of existing slope toes would be resolved in these sections because the void space between the existing side slopes along the existing creek would be completely backfilled to the planned elevations and strengthen the global stability of slopes.

The conceptual new creek cross-section within each realigned section would consist of a trapezoidal section of 4 m in width at the base with two 2H:1V side slopes along the realigned creek bank. A low flow channel of 1 m in width would be constructed along the new channel base. Depending on the topographic conditions along the realigned sections, the creek bed and banks would be lined with void-filled riprap (VFR) as described above and the upper banks with vegetated riprap to strengthen the global stability of slopes. The vegetated riprap would be used where appropriate. For the temporary excavation of the new channel, an open cut on a temporary slope 3H:1V could be considered during construction; however temporary shoring or trench box would be required for further excavation of the low flow channel to the planned founding levels, depending on the topographic and subsurface conditions.

The five (5) weir-pool structures proposed for Alternative 7b, at approximately 1.5 m in height, would be constructed at a distance of approximately 50 m in the upper stream and a distance of 100 m in the lower stream. Detailed dimensions of the weir-pool structures have not been established; however temporary shoring or trench box would be required for further excavation of the weir-pool structures to the planned founding levels, depending on the topographic and subsurface conditions.

The boulder sized cascades and VFR proposed for Alternative 7c would be placed at the upstream ends of the two realigned sections and at one lower location to provide flow resistance during high flow events. The spot treatment would be considered in other presently eroding areas using logs and boulder clusters to limit further disturbance of adjacent areas.

Figure 5.20 indicates the preliminary locations of the proposed realigned creek areas with additional bank stabilization. Additional bank stabilization treatment may be necessary pending review of the functional design for the sections adjacent to the slopes of concern. The Alternative 7b is the preferable alternative as discussed above; however, the Alternative 7c may be considered and/or combined with the Alternative 7b at some locations, based on further analysis during functional design.

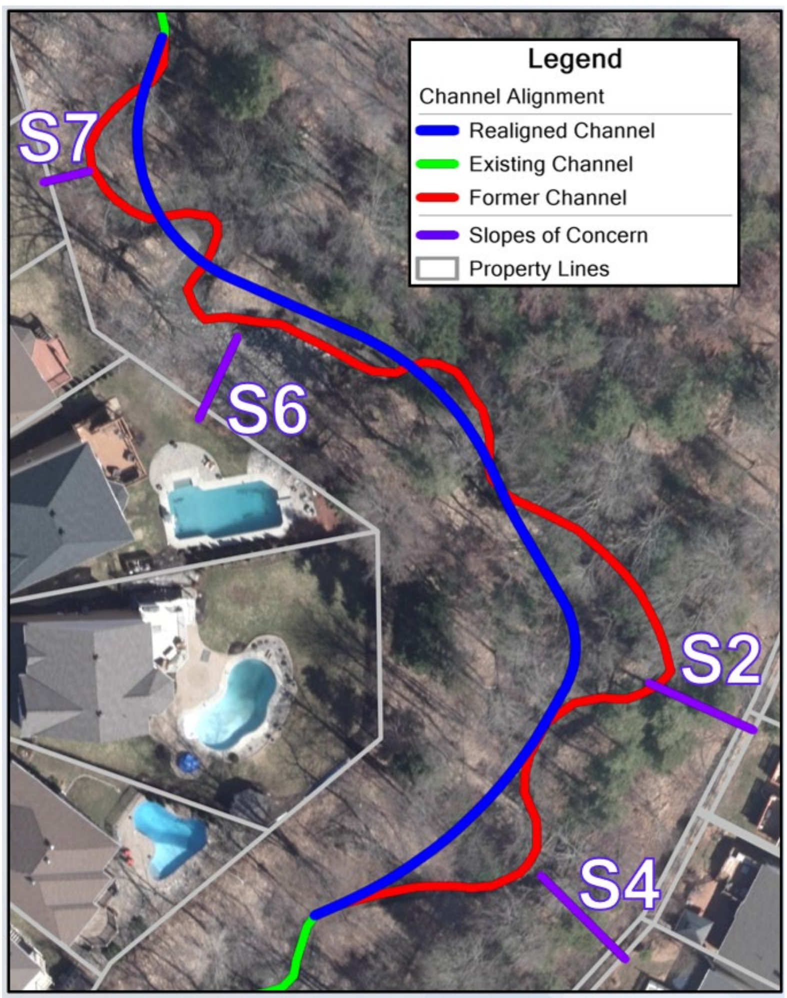

Figure 5.22. Preliminary Locations of Proposed Realigned Creek Areas with Additional Bank Stabilization

From the viewpoint of geotechnical aspects, the following objectives can be achieved after the implementation of Alternative 7b:

- The realigned creek would provide sufficient space to implement rehabilitation/stabilization works along the proposed new 2H:1V slope with vegetation and bank protection, pending confirmation;
- The existing creek would be backfilled to the new grade elevations after completion of the planned realignment work;
- Issues of potential slope instability induced by the erosion of the existing slope toes would be resolved in these sections;
- The void filled riprap banks and the vegetated riprap upper banks would provide adequate long term erosion protection along the realigned slope toes; and
- Additional bank stabilization treatment may need to be considered at slope toes of SPL2, SPL4, SPL6 and SPL7, and other similar banks, if needed, to strengthen global slope stability prior to creek realignment. The type of the additional slope stabilization measures remains to be confirmed.

The total estimated project cost is $2.9M which includes costs for construction, landscaping, fees, HST and contingency. The cost estimate does not include costs associated with excess soil management as per O.Reg. 388/22; these costs will need to be reviewed at the detailed design stage

5.8 Evaluation of LID BMPs

While the implementation of LID BMPs as a standalone alternative would be beneficial for mitigating the erosive flow rates contributing to Blue Willow Creek, significant rehabilitation of existing infrastructure will not take place for several decades and therefore would not address the existing erosion concerns in a timely manner. The use of LID BMPs should be promoted in conjunction with other alternatives as a long-term objective. Implementation of the LID BMPs within the public realm could be undertaken during the reconstruction of the roadways. Similarly, LID measures could be implemented within the private properties, for example by redirecting downspouts to pervious areas, or by installing permeable pavers instead of asphalt driveways at the time of reconstruction

6.0 Preferred Alternative

The Preferred Alternative selected from the Short-Listed Alternatives is **Alternative 7b: Realignment, Structures and Hardening (Structures Focused)**. The alternative has been selected from the three (3) combined alternatives for the following reasons as summarized below:

i. Creek control structures coupled with localized creek bank rehabilitation and realignment(s) would reduce creek erosion risk, by reducing flow regime energy, which in turn will provide opportunities to implement integrated slope protection works (ref. Drawing 1).

ii. Future slope stability risk would be reduced by creek control structures and associated localized creek bank rehabilitation and realignment(s). Slope protection works will include toe of slope protection facilitated by the creek realignments, slope regrading, revegetation, and potential engineering measures. The same approach would apply to each of the three (3) combined alternatives.

iii. Existing infrastructure would be protected from the proposed creek improvements and integrated slope stability works.

iv. Creek bank erosion would be mitigated, and the creek improvements would reduce risk of further loss of riparian and tableland habitat and species.

v. Mitigated creek erosion and integrated slope stability works would reduce the risk to public, by reducing the potential for slope failure and impacts to private property (ref. Drawing 2)

vi. Construction impacts would be limited to where structures are to be installed, along with the localized creek bank measures and creek realignment(s), which will be concentrated in the upper end of the ravine. Construction access has also been considered, as to how to minimize disturbances to the valley and public (ref. Drawings 3 and 5).

vii. Control structures, creek bank measures and realignment should require minimal maintenance (ref. Drawing 4).

6.1 Preferred Alternative 7b PCSWMM Model Continuous Simulation

The HEC-RAS model developed for the assessment of Alternative 7b and functional design has been incorporated into the PCSWMM model to be executed for the 40 year semi-continuous simulation. The notable revisions from the short-listed assessment to functional design to the hydraulic elements and in comparison, to the existing conditions model include the following:

- Six (6) weir structure transects (versus five (5) for short-listed assessment) have been added to the channel as per Alternative 7b; the structures within the transects are 1.25 m to 1.5 m in height with varying top widths, and a 0.3 m slot (low flow) width has been included in each structure.
- A 1 m deep plunge pool has been encoded beyond each weir structure conduit.

- The conduit lengths have been revised to coincide with the realigned portion of the channel now a single longer reach being realigned versus two shorter lengths, while the channel slope for the realigned channel has been revised to 0.9 % as per the functional design.
- The low flow channel has been revised as per the proposed cross sections.

The effectiveness of the proposed design has been assessed by comparing the existing and proposed (Alternative 7b) condition semi-continuous simulation erosion modelling results (ref. Appendix J). Since the alignment of the creek has been altered and new structures have been added to the PCSWMM model, the results can only be compared at certain sections, or conduits, that are common to both scenarios. Eight of these conduits have been selected; a description for each conduit is provided in Table 6.1.

Table 6.1. Conduits Used for Erosion Assessment

Conduit	Proposed Conditions Scenario Description
421.78	Upstream of weir-pool P1
393.28	Between weir-pools P1 and P2
372.96	Between weir-pools P2 and P3
354.07	Upstream of weir-pool P3
308.63	Between weir-pools P3 and P4, near Slope 6
216.81	Between weir-pools P4 and P5, near Slope 4
146.34	Upstream of weir-pool P6
68.83	Lower creek, downstream of all works

A comparison of the average annual effective work at each conduit for the existing conditions and proposed (Alternative 7b) conditions are shown in Figure 6.1 and are summarized in Table 6.2. Except for conduit 308.63, the erosion potential for the creek is greatly reduced with the Alt. 7B works in place. As noted above, conduit 308.63 lies within the realigned section which will be hardened with riprap; hence, this simulation, which assumes clay at each conduit, is a conservative estimate for Alt. 7B conduit 308.63.

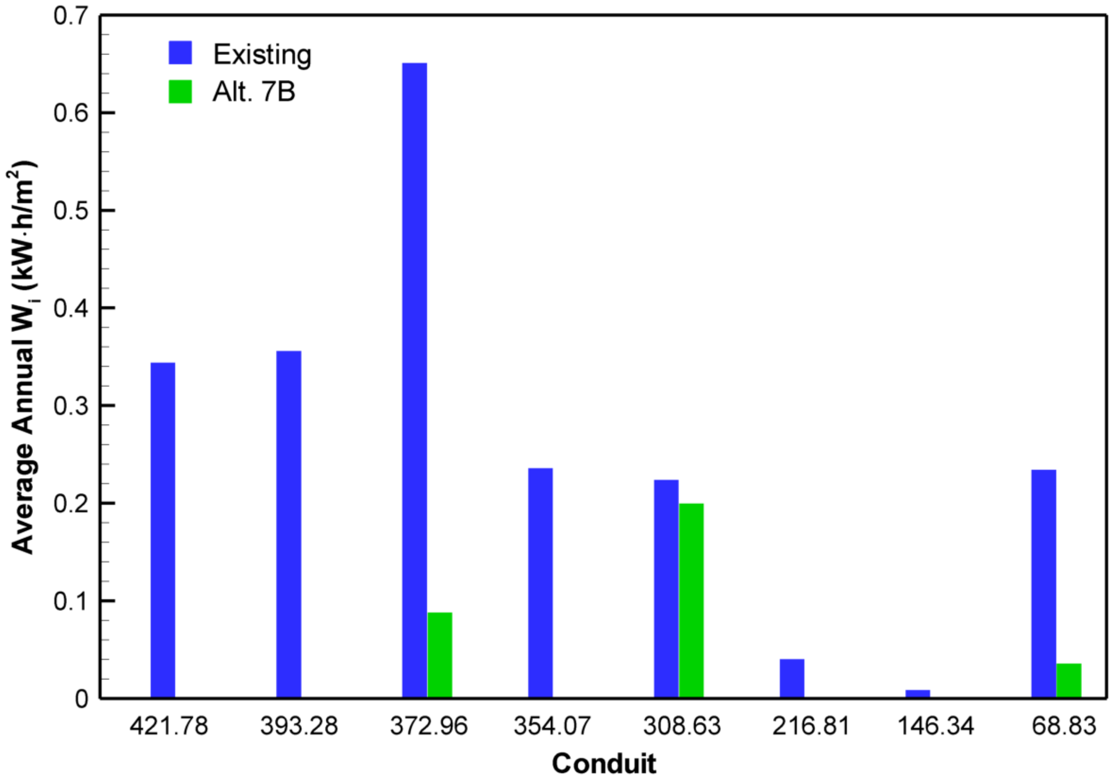

Figure 6.1. Average Annual Effective Work at Each Conduit

Table 6.2. Value and Change in Average Annual Effective Work Index, W_i (kW h/m^2)

Conduit	W_i (kW·h/m^2) Existing	W_i (kW·h/m^2) Alt. 7B	Change (%)
421.78	3.44E-01	7.55E-06	-100%
393.28	3.56E-01	0.00E+00	-100%
372.96	6.51E-01	8.81E-02	-86%
354.07	2.36E-01	0.00E+00	-100%
308.63	2.24E-01	2.00E-01	-11%
216.81	4.03E-02	1.64E-03	-96%
146.34	8.55E-03	2.79E-05	-100%
68.83	2.34E-01	3.57E-02	-85%

The effective work results for the existing conditions exhibited erosion across all eight conduits, consistent the field observations. Consequently, the results for the proposed (Alternative 7b) conditions work are shown, in all but one conduit (which in practice would be armoured with riprap), to be at least 85% lower. This confirms the suitability of the design for protection of Blue Willow Creek from erosion.

6.2 Preferred Alternative 7b Slope Stabilization

Analysis of the proposed slope stabilization work, proposed creek realignment, and toe protection work has been undertaken to confirm the acceptability of the stabilization works while providing preliminary recommendations for the slopes of concern; the analysis and reporting are provided within Appendix K.

As per the preferred alternative, vegetated rip rap has been recommended for the proposed realigned creek channel. The combination of the proposed creek alignment, proposed bank repairs, and channel treatments, would satisfy the required factor of safety (FoS)., The minimum target FoS discussed in Section 3.6.1 have been updated to 1.3, greater than 1.1, and 1.0 for the long-term static condition, seismic condition, and the rapid drawdown condition during the flooding periods, respectively. The target FoS values have considered the retrogressive assessment and the residual strength of the soils. The associated limits for the surcharges are also indicated within Table 6.3. The approximate extent of areas to be repaired adjacent to the slopes of concern are shown on Drawing 1.

6.2.1 Results of the Analyses

The results of the global slope stability analyses are summarized within Table 6.3 and the figures of analyses are provided within Appendix K.

Table 6.3. Results of Global Slope Stability Analyses

Slope No.	Referenced Borehole No.	Adjacent Property	Scenario	Maximum Allowable Surcharge Load (kPa)	Type of Analysis	Calculated Factor of Safety	Reference Figure No. (Appendix K)
SLP2	BH5, BH6, and CPT-5	Existing slope behind 2165 Auburn Ridge Drive (left bank)	Existing	N/A	Long Term (Drained)	1.0	C2.1.1
SLP2	BH5, BH6, and CPT-5	Existing slope behind 2165 Auburn Ridge Drive (left bank)	Proposed Condition	4	Long Term (Drained)	1.3	C2.2.1
SLP2	BH5, BH6, and CPT-5	Existing slope behind 2165 Auburn Ridge Drive (left bank)	Proposed Condition	4	Seismic (Undrained)	1.3	C2.2.2
SLP2	BH5, BH6, and CPT-5	Existing slope behind 2165 Auburn Ridge Drive (left bank)	Proposed Condition	4	Rapid Drawdown (25-Year Flood)	1.1	C2.2.3
SLP2	BH5, BH6, and CPT-5	Existing slope behind 2165 Auburn Ridge Drive (left bank)	Proposed Condition	4	Rapid Drawdown (100-Year Flood)	1.1	C2.2.4
SLP2	BH5, BH6, and CPT-5	Opposite side of existing slope behind 2165 Auburn Ridge Drive (right bank)	Proposed Condition	18	Long Term (Drained)	1.3	C2.3.1

Slope No.	Referenced Borehole No.	Adjacent Property	Scenario	Maximum Allowable Surcharge Load (kPa)	Type of Analysis	Calculated Factor of Safety	Reference Figure No. (Appendix K)
SLP2	BH5, BH6, and CPT-5	Opposite side of existing slope behind 2165 Auburn Ridge Drive (right bank)	Proposed Condition	18	Seismic (Undrained)	1.5	C2.3.2
SLP2	BH5, BH6, and CPT-5	Opposite side of existing slope behind 2165 Auburn Ridge Drive (right bank)	Proposed Condition	18	Rapid Drawdown (25-Year Flood)	1.1	C2.3.3
SLP2	BH5, BH6, and CPT-5	Opposite side of existing slope behind 2165 Auburn Ridge Drive (right bank)	Proposed Condition	18	Rapid Drawdown (100-Year Flood)	1.1	C2.3.4
SLP4	BH5, BH6, and CPT-5	Existing slope behind 2161 Auburn Ridge Drive (left bank)	Existing	N/A	Long Term (Drained)	1.0	C4.1.1
SLP4	BH5, BH6, and CPT-5	Existing slope behind 2161 Auburn Ridge Drive (left bank)	Proposed Condition	12	Long Term (Drained)	1.3	C4.2.1
SLP4	BH5, BH6, and CPT-5	Existing slope behind 2161	Proposed Condition	12	Seismic (Undrained)	1.2	C4.2.2

Slope No.	Referenced Borehole No.	Adjacent Property	Scenario	Maximum Allowable Surcharge Load (kPa)	Type of Analysis	Calculated Factor of Safety	Reference Figure No. (Appendix K)
		Auburn Ridge Drive (left bank)					
SLP4	BH5, BH6, and CPT-5	Existing slope behind 2161 Auburn Ridge Drive (left bank)	Proposed Condition	12	Rapid Drawdown (25-Year Flood)	1.2	C4.2.3
SLP4	BH5, BH6, and CPT-5	Existing slope behind 2161 Auburn Ridge Drive (left bank)	Proposed Condition	12	Rapid Drawdown (100-Year Flood)	1.2	C4.2.4
SLP4	BH5, BH6, and CPT-5	Opposite side of existing slope behind 2161 Auburn Ridge Drive (right bank)	Proposed Condition	6	Long Term (Drained)	1.3	C4.3.1
SLP4	BH5, BH6, and CPT-5	Opposite side of existing slope behind 2161 Auburn Ridge Drive (right bank)	Proposed Condition	6	Seismic (Undrained)	1.6	C4.3.2
SLP4	BH5, BH6, and CPT-5	Opposite side of existing slope behind 2161 Auburn Ridge Drive (right bank)	Proposed Condition	6	Rapid Drawdown (25-Year Flood)	1.1	C4.3.3

Slope No.	Referenced Borehole No.	Adjacent Property	Scenario	Maximum Allowable Surcharge Load (kPa)	Type of Analysis	Calculated Factor of Safety	Reference Figure No. (Appendix K)
SLP4	BH5, BH6, and CPT-5	Opposite side of existing slope behind 2161 Auburn Ridge Drive (right bank)	Proposed Condition	6	Rapid Drawdown (100-Year Flood)	1.1	C4.3.4
SLP6	BH1, BH3, BH4, and CPT-3	Existing slope behind 9 Sprucewood Place (right bank)	Existing	N/A	Long Term (Drained)	1.3	C6.1.1
SLP6	BH1, BH3, BH4, and CPT-3	Existing slope behind 9 Sprucewood Place (right bank)	Proposed Condition	52	Long Term (Drained)	1.3	C6.2.1
SLP6	BH1, BH3, BH4, and CPT-3	Existing slope behind 9 Sprucewood Place (right bank)	Proposed Condition	52	Seismic (Undrained)	1.2	C6.2.2
SLP6	BH1, BH3, BH4, and CPT-3	Existing slope behind 9 Sprucewood Place (right bank)	Proposed Condition	52	Rapid Drawdown (25-Year Flood)	1.3	C6.2.3
SLP6	BH1, BH3, BH4, and CPT-3	Existing slope behind 9 Sprucewood Place (right bank)	Proposed Condition	52	Rapid Drawdown (100-Year Flood)	1.3	C6.2.4

Slope No.	Referenced Borehole No.	Adjacent Property	Scenario	Maximum Allowable Surcharge Load (kPa)	Type of Analysis	Calculated Factor of Safety	Reference Figure No. (Appendix K)
SLP6	BH1, BH3, BH4, and CPT-3	Opposite slope behind existing slope behind 9 Sprucewood Place (left bank)	Proposed Condition	34	Long Term (Drained)	1.3	C6.3.1
SLP6	BH1, BH3, BH4, and CPT-3	Opposite slope behind existing slope behind 9 Sprucewood Place (left bank)	Proposed Condition	34	Seismic (Undrained)	1.4	C6.3.2
SLP6	BH1, BH3, BH4, and CPT-3	Opposite slope behind existing slope behind 9 Sprucewood Place (left bank)	Proposed Condition	34	Rapid Drawdown (25-Year Flood)	1.2	C6.3.3
SLP6	BH1, BH3, BH4, and CPT-3	Opposite slope behind existing slope behind 9 Sprucewood Place (left bank)	Proposed Condition	34	Rapid Drawdown (100-Year Flood)	1.2	C6.3.4
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Behind 13 Sprucewood Place (right bank)	Existing	N/A	Long Term (Drained)	1.0	C7.1.1

Slope No.	Referenced Borehole No.	Adjacent Property	Scenario	Maximum Allowable Surcharge Load (kPa)	Type of Analysis	Calculated Factor of Safety	Reference Figure No. (Appendix K)
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Behind 13 Sprucewood Place (right bank)	Proposed Condition	16	Long Term (Drained)	1.3	C7.2.1
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Behind 13 Sprucewood Place (right bank)	Proposed Condition	16	Seismic (Undrained)	1.6	C7.2.2
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Behind 13 Sprucewood Place (right bank)	Proposed Condition	16	Rapid Drawdown (25-Year Flood)	1.3	C7.2.3
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Behind 13 Sprucewood Place (right bank)	Proposed Condition	16	Rapid Drawdown (100-Year Flood)	1.3	C7.2.4
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Opposite side of existing slope behind 13 Sprucewood Place (left bank)	Proposed Condition	38	Long Term (Drained)	1.3	C7.3.1
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Opposite side of existing slope behind 13 Sprucewood Place (left bank)	Proposed Condition	38	Seismic (Undrained)	1.4	C7.3.2

Slope No.	Referenced Borehole No.	Adjacent Property	Scenario	Maximum Allowable Surcharge Load (kPa)	Type of Analysis	Calculated Factor of Safety	Reference Figure No. (Appendix K)
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Opposite side of existing slope behind 13 Sprucewood Place (left bank)	Proposed Condition	38	Rapid Drawdown (25-Year Flood)	1.3	C7.3.3
SLP7	BH1, BH2, BH3, BH4, and CPT-3	Opposite side of existing slope behind 13 Sprucewood Place (left bank)	Proposed Condition	38	Rapid Drawdown (100-Year Flood)	1.3	C7.3.4

N/A Not applicable

Well-graded sand and gravel (e.g., OPSS 1010 Granular B Type I or Select Subgrade Material) placed in maximum 300 mm thick lifts and compacted to at least 95 % Standard Proctor maximum dry density (SPMDD) is the preferred option to bulk backfill the abandoned portion of the stream channel. However, compactable clay, either native or imported, may also be used for the bulk backfilling. Based on visual and tactile examination of the soil samples retrieved from the boreholes, the upper native stiff clay may generally be used, provided that it has been reconditioned to ensure the water content is at/or near optimum and the maximum clod size is less than 100 mm. The clay should be placed in approximately 200 mm loose lifts and compacted to at least 92 % SPMDD. Unsuitable materials such as organic soils, boulders, cobbles, frozen soils, etc., should not be used. Conservatively, the analyses have only modelled clay being used as the bulk backfill material.

The void-filled riprap (R-50 as per OPSS 1004) for the realigned channel protection should be 400 mm thick. The bottom portion of the channel has been proposed to be sloped at 5H:1V while the upper banks of channel has been proposed to be sloped at 2H:1V. The void-filled rip rap should be extended 2 m above the invert of the channel, as shown in Drawing 2. Extending the riprap will further reduce the risks of erosion that could lead to potential slope undermining and failures. The fabric should also be placed between the clay, sand and gravel (if used), and R-50 riprap.

Where the existing slopes are stepper than 2H:1V, the existing slope should be graded (cut and/or fill) to a maximum slope steepness of 2H:1V.

6.2.2 Surcharge Load

A distributed surcharge load has been applied along the modelled slopes to determine the maximum distributed surcharge load that the slope could support. This has been undertaken to estimate the quantity of mature vegetation and other incidental loads that the slopes could safely support. A distributed load has been applied from approximately 2 m above the new channel invert to the top of slope. It is assumed that the area between the invert and 2 m above the invert will experience frequent waterflow and will not support heavy vegetation growth.

The results of the maximum distributed surcharge load for each slope are shown in Table 6.3 and the distributed loads ranged from 4 kPa to 52 kPa.

6.2.3 Limits of the Slope Repairs and Future Work Considerations

Based on a review of the proposed cross sections for the areas near the slope of concerns (ref. Appendix K), the repair limits of the slopes of concern were identified and are shown in Drawing No. A, within Appendix K. These extents are approximate and intended only for preliminary estimates of the works and materials involved with the repair and stabilization of the four (4) slopes of concern. If more accurate limits of the repair extents are required, additional and more in-depth slope stability analyses should be performed along other sections of the proposed realigned creek.

Ongoing slope stability issues at Slope SLP 7 may require additional mitigation works. Site Review 8 (Stantec, March 30, 2022), documents fallen trees and signs of active erosion observed since the start of the slope visual inspection program at the areas of concern within Ruisseau Ravine. City Staff noted that the rear yard of the residential property at 13 Sprucewood Place has settled since the Slope SLP6 failure in 2017. As such, the resident at 13 Sprucewood Place has been placing approximately 20 bags of topsoil in the rear yard to address the settling (Jolliet – Chipps, April 5, 2022). City Staff have advised the resident to stop placing topsoil to avoid adding additional weight to the slope, which may further increase the slope instability.

Due to the issues noted in the Site Review 8 summary (Stantec, March 30, 2022) and correspondence with City Staff (Jolliet – Chipps, April 5, 2022) it is recommended that a rockfill buttress (approximately 0.5 m in height) be constructed prior to construction of the recommended creek and slope stability works to prevent further bank erosion and undermining of the slope. The rockfill buttress will prevent further bank erosion and undermining of the slope. This rockfill buttress could be used as backfill material when backfilling the existing creek. Continuation of the slope monitoring by the City is also recommended, with potential additional quantification of slope characteristics by survey.

It is understood that pending the detailed design of the recommended creek and slope stability works, that the construction could commence late 2023. Depending on the timeline until the construction works are initiated, further bank instabilities may occur, and hence other sections of the current creek alignment may require repairs. Prior to construction, the current slopes should be analysed to determine if the existing slopes will support loads generated from the construction equipment and related activities.

7.0 Preferred Alternative Functional Design

The preferred alternative, **Alternative 7b: Realignment, Structures and Hardening (Structures Focused)**, has been further developed through functional design (ref. Drawings 1-5), to consider construction access, overall disturbance area and tree preservation, removal and restoration areas.

7.1 Construction Access and Staging Area

This will be accomplished with three (3) dedicated construction access paths to the ravine, two (2) of which are through Ruisseau Park at Creek Crossing Street, while the third construction access is from Auburn Ridge Drive (ref. Drawings 1, 3 and 5).

The middle construction access will follow the path that was used to conduct the emergency mitigation works at Slope SLP 6 in 2017. The access paths will remain following the construction of the preferred alternative should there be a need to conduct maintenance within the ravine in the future.

The staging area will be located adjacent to Creek Crossing Street (ref. Drawings 1, 3 and 5) within the Ruisseau Park. The staging area location has been selected to allow contractor access to the road and the north end of the construction area. The staging area and construction accesses and areas will be off limits to the public for safety reasons. As the northern construction accesses will be fenced off to the public, the playground area should be available to the public.

7.2 Disturbance and Restoration Areas

In review of the proposed construction works within the ravine, an area of disturbance has been identified for the anticipated limits of construction (ref. Drawing 3). The existing trees and vegetation will be disturbed within these limits due to the construction and re-grading at the 4 slopes of concern. However, not all trees within the area of disturbance will need to be removed (ref. Drawing 5). The intention is to preserve areas where no construction or limited construction will be taking place. Construction limits will be refined during detailed design, to further reduce the area of disturbance. Approximately 72 trees have been determined to be within disturbance area, which would need to be removed to facilitate construction. Restoration areas would be replanted with native grass seed mixes, shrubs and trees with the objectives of minimizing the duration for plants to stabilize disturbed areas, long-term enhancement of the habitat within the ravine and to provide privacy to the properties backing on to the ravine.

7.3 Preliminary Cost Estimate

The Class ‘D’ Cost Estimate has been updated to a Class ‘C’ Cost Estimate to include the additional details determined when preparing the functional design, including construction access, staging areas and restoration area. The preliminary cost estimate is $2.6M (ref. Appendix I).

8.0 Implementation

8.1 Phase 2: Detailed Design and Construction

The recommended works for the site, creek realignment, control structures, naturalized armouring, and slope stabilization, should be advanced to the next stages of planning and design. The next phase of the Class EA process includes the detailed design of the site, followed by the construction of the recommended works. The City has indicated that the detailed design would be completed by early 2023 and the construction of the channel and slope works would be undertaken during 2023-2024. Prior to the construction of the proposed mitigation works, additional tasks are required to be completed, including consultation with agencies, First Nations, neighbourhood associations and residents, obtaining agency approvals, and identifying trees for removal and preservation.

8.1.1 Consultation – Agencies, First Nations, Neighbourhood Associations

Consultation should be undertaken during detailed design with the agencies including the Rideau Valley Conservation Authority, Ministry of Environmental, Conservation and Parks, and the Ministry of Northern Development, Mines, Natural Resources, and Forestry. The purpose of the consultation is to identify agency concerns for the proposed mitigation alternative and address those concerns prior to requesting the agency approvals. This ensures that the City has completed the necessary due diligence for the project.

Consultation with First Nations provides the respective groups the opportunity to exercise their social and economic interest in lands and natural resources. Furthermore, this provides an increased role for First Nations groups to participate in the decision-making process.

The local residents and the respective neighbourhood associations should be consulted prior to construction to ensure their understanding of the proposed mitigation works and the potential impacts during the construction period. The proposed works will require the use of heavy machinery within the ravine, the removal of trees which can impact the privacy of the properties adjacent to the ravine, and closure of the trail system through the park.

8.1.2 Agency Approvals

Agency approvals are required given the location of the Ruisseau Park Ravine within the regulated limit of the Rideau Valley Conservation Authority. Additional agency approvals may be required due to the mitigation works within a functioning stormwater management facility including approval from the Ministry of Environment Conservation and Parks (MECP), and also from the MECP approval to any SAR determined during detailed design.

8.1.3 Tree Removals

As noted within Section 6.2.4, trees will need to be removed within the construction area to provide access for construction vehicles, facilitate the realignment of the channel, and implement the slope stabilization measures. A tree survey has been conducted to identify trees within the anticipated area of disturbance to document the tree structural condition, health, diameter at breast height, species, and location. A total of 216 trees have been identified within the anticipated area of disturbance.

Trees have been identified for removal based on the location relative to the proposed maintenance access locations and proposed channel and slope works. Trees within the proposed realigned channel will have to be removed, while trees on the slopes of concern can potentially be preserved dependent on the depth of fill place at the base of the tree. A preliminary review of the required fill depth for the slopes of concern has noted that the health of the existing trees could be negatively impacted if the depth of fill around the existing trees exceeds 7 cm. As such, trees located in areas where the depth of fill exceeds 7 cm will likely have to be removed. Lastly, the maintenance accesses can be moderately realigned to avoid mature, large calibre (greater than 15 cm diameter at breast height) as necessary.

As a preliminary assessment of the trees identified for removal for the preferred alternative, based on the trees which have been surveyed, 77 trees would need to be removed from within the area of disturbance. (ref. Drawing 5). These trees are primarily in locations that will undergo significant alteration from the present state, such as the maintenance path, weir structure locations, and the realigned channel. Of the 77 trees, 66 have a diameter at breast height of 0.15 m or greater, and of those 66 trees, 15 have a less than fair condition (i.e. poor or dead condition).

In addition to the 77 trees identified for removal within area of disturbance, there are an additional 40 trees that can likely be preserved within the area of disturbance based on grading and location; this will need to be confirmed at the detailed design stage. Some of those 40 trees may need to be removed if construction cannot be completed should the trees be preserved.

8.2 Restoration

The project team has been and will continue to consult with the City's Forestry Department to ensure that the tree canopy will be replaced with a diverse selection of native tree species following the construction works. The tree species selected for replacement will be undertaken during the detailed design phase of the project. However, native trees, shrubs, and ground cover vegetation have been identified that could potentially be used at the site and are summarized as follows:

- Trees: Black Walnut, Sugar Maple, Basswood, and White Birch
- Shrubs: Dogwood and Bebb's Willow
- Vegetation: Red Clover, Black Eyed Susan, New England Aster, Canada Bluegrass, Fox Sedge

If the residents adjacent to the ravine are concerned about their privacy following the removal of some trees during the construction process, cedars are recommended to be planted near the top of the slopes as they will provide a year-round visual barrier.

A preliminary cost for trees, shrubs and the restoration of ground cover on the slopes and the lower creek bank has been estimated at $100,016 which includes costs for fees, HST and contingency. The replanting of trees and shrubs have been incorporated into the Alternative 7b cost estimate (ref. Appendix I), with approximately 80 trees and 240 shrubs to be planted.

8.3 Post Construction Monitoring

To determine the success of the proposed creek realignment and slope mitigation works, post construction monitoring for a minimum of two (2) years will be required, with the duration to be determined in consultation with the City during detailed design. The following provides a high-level list of some of monitoring requirements, which would require further development during detailed design:

- Channel Geomorphic Conditions: As built channel surveys (plan and profile) and channel cross-section surveys to be conducted at key channel locations after construction and prior to completing the monitoring term. Erosion pins in creek banks to be located at strategic locations, with regular measurements taken. Sediment depth within pools and at weir structures to be measured. Observations of stream planform and adjustments to the completed works.
- Slope Stability: Topographic survey of each slope mitigation area to be conducted after construction and at least once prior to the end of the monitoring term. Slope pins to be placed at key locations and measured regularly during the monitoring period.

9.0 Conclusions and Recommendations

9.1 Conclusions

The following conclusions have been reached for the Ruisseau Park Ravine (Blue Willow) Rehabilitation Class EA:

Hydrology and Hydraulics

15. A PCSWMM model has been developed for the assessment with a contributing drainage area to the Ruisseau Park Ravine 106.30 ha (+/-) and peak flow rates ranging from 6.70 m^3/s (+/-) to 17.64 m^3/s (+/-) for the 2-100 year design storm events.
16. The contributing drainage area for this study is 33.12 ha (+/-) greater than the drainage area identified in the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) with a 16.17 % increase in imperviousness compared to the previous study.
17. The water surface elevation in the SWMF for the 25 year, 50 year, and the 100 year design storm events have been simulated to overtop the embankment at the downstream end of the facility while the Chapel Hill South SWM study (J.L. Richards and Associates Limited Consulting Engineers and Planners, July 1989) indicated that the facility would not be overtopped during a 100 year design storm event.
18. A 40 year semi-continuous simulation of the PCSWMM model has been executed to assess the shear stress of the channel banks at multiple channel sections based on the flow rates, depth of flow, and the velocity.

Stream Morphology

19. The stream morphology team from Coldwater Consulting conducted field reconnaissance and a geomorphic assessment. The increased imperviousness of the contributing drainage area, in addition to the implementation of the drainage network, has led to an increase in discharge to the channel during precipitation events; this has led to downcutting and planform migration of the stream and slope instability issues at several locations.
20. The Rapid Geomorphic Assessment (RGA) undertaken for this project identified that more than 75% of the channel, or the upper three (3) reaches, are classified as In Adjustment, while the lower portion of the channel is In Transition.
21. The Rapid Stream Assessment Technique (RSAT) indicates that the upper half of the stream is in Poor condition while the lower half of the stream is in Fair condition which generally corresponds to the RGA results.
22. Based on the geomorphic assessment, multiple locations within Ruisseau Park Ravine have been identified as requiring bank stabilization to mitigate further erosion.

Geotechnical

23. The seven (7) boreholes and two (2) Piezocone Penetration Testing (CPT) holes advanced by Wood's Geotechnical team identified the soils within the ravine as primarily being clay and silty sand; bedrock was not encountered within the advanced boreholes.
24. Slope stability analyses have been performed for eight (8) slopes in which three (3) slopes, SLP2, SLP4, and SLP6 have been identified as requiring a bank stabilization treatment to prevent progressive erosion, while there is concern that erosion at the toe of a fourth slope, SLP7, may contribute to slope instability.
25. The sensitive clay soils at the site are consistent with the clay soils which were identified at locations of previous retrogressive slope failure sites near Lemieux, Ontario; the soils at the site are vulnerable to an earthflow risk. However, in the absence of an adverse triggering conditions the earthflow slides may never occur at this site.

Natural Systems

26. The ELC delineation completed by Wood's Natural Heritage Team during the field investigation identified four (4) communities within the study area; manicured landscapes, Dry-Fresh Sugar Maple- Hemlock Mixed Forest Ecosite, Reed Canary Grass Graminoid Meadow, and Fresh-Moist Willow Lowland Deciduous Forest.
27. A preliminary desktop Species at Risk (SAR) screening identified 15 SAR, which have the potential to occur with the Project Location, while eight (8) SAR of those species have been identified with a moderate to high potential of occurrence in the study area.
28. Potential snake hibernaculum was found within the Reed Canary Grass Graminoid Meadow during the field investigation along the west bank of the SWMF. Other potential Significant Wildlife Habitat (SWH) within the Project Location include bat maternity colonies, habitat of special concern or rare wildlife species, and potential turtle nesting areas.

Following the screening of a long list of creek improvement alternatives to address the existing creek bank erosion conditions that are to be integrated with slope stability alternatives, the seven (7) short-listed alternatives have been evaluated based on the functionality, impacts to the natural environment, impacts to the social environment, and the capital and maintenance costs associated with the alterative. The assessment of the seven (7) short listed alternatives has been summarized as follows:

Alternative 1 Do Nothing has been screened from further consideration as it would not address the existing creek bank erosion conditions and would not help facilitate slope mitigation works.

Alternative 2 Stormwater Management has been screened out from further consideration as it would not address existing creek bank erosion conditions, that said, LID BMPs or source controls could be implemented with a combined alternative. The LID BMPs are viewed as a complimentary long-term alternative.

Alternative 3 Creek Realignment would not fully address the creek bank erosion conditions, as such it has been considered as part of a combined alternative.

Alternative 4 Creek Partial Enclosure has been screened out for further consideration, based not addressing the existing creek bank erosion conditions and would result in long term maintenance requirements and eventual replacement.

Alternative 5 Creek Control Structures would reduce creek erosion risk, by reducing flow regime energy, but would not fully address the existing creek bank erosion conditions, as such it has been considered as part of a combined alternative.

Alternative 6 Channel Hardening would stabilize the existing channel in key locations, but may result in erosion in new locations, as such the alternative has been considered as part of a combined alternative.

Alternative 7 Combination of Alternatives is required as the individual Alternatives 1-6 would not address the existing creek bank erosion conditions, however, the combination of Alternatives 3, 5, and 6 would address existing erosion conditions. A summary of the combination of alternatives which have been assessed and evaluated is provided as follows:

Alternative 7a Realignment, Reshaping and Hardening (Hardening Focus) combines realignment of the creek to address existing slope stability issues and reshaping and hardening of the creek cross-section to address erosion issues. The channel would be realigned to away from the slopes of concern to provide space to implement slope stabilization works at the slopes of concern. The channel realignment would not reduce the creek energy as the shear stress would exceed critical values for the native material. As such, channel hardening would be required to address the creek energy. The estimated channel mitigation cost is $3.1M based on a Class D cost estimate (ref. Appendix I).

Alternative 7b Realignment, Structures and Hardening (Structures Focus) involves a combination of the control structures of Alternative 5 with the realignment and naturalized armouring of Alternative 7a. This alternative incorporates weir-pool features (control structures) to dissipate the creek energy. Weir-pool or step-pool features are well suited for channel with relatively steep gradients. The pool below each of the six (6) weir structures would be armoured to prevent scour. The weir structures would reduce the energy gradeline (surface slope) and consequently the velocity and erosion potential. The realignment of the channel would be consistent with Alternative 7a to move the channel away from the slopes of concern to provide space to implement slope stabilization works. However, channel hardening and bank hardening with void-filled rip rap would still be required at select locations to address the creek energy (shear stress). The estimated channel mitigation cost is $2.6M based on a Class D cost estimate (ref. Appendix I).

Alternative 7c Realignment, Structures and Naturalization (Adaptation Focus) involves a combination of the reshaping and realignment of Alternative 7a/b with the use of habitat features and natural construction materials to restrain the stream's morphological evolution. Unlike Alternatives 7a and 7b, this design would allow for some stream evolution to continue, albeit in a controlled manner. The identified eroded locations would be protected while limiting the disturbance of the existing adjacent areas. The channel realignment would be consistent with the realignment proposed for alternatives 7a/b; the realigned channel would be widened and hardened with void will riprap. Cross vanes and log weirs would be constructed within the channel to reduce the erosion potential, similar to Alternative 7b. Monitoring would be required for this alternative as the channel naturally evolves. The estimated channel mitigation cost is $2.4M based on a Class D cost estimate (ref. Appendix I).

The Preferred Alternative selected from the Short-Listed Alternatives is **Alternative 7b: Realignment, Structures and Hardening (Structures Focused)** at a cost of $2.6M using a Class C Cost Estimate determined for the function design (ref. Appendix I).

Analysis of the proposed slope stabilization work, proposed creek realignment, and toe protection work has been undertaken to confirm the acceptability of the stabilization works while providing preliminary recommendations for the slopes of concern. The void-filled riprap banks and the vegetated riprap upper banks would provide adequate long-term erosion protection along the realigned slope toes. The combination of the proposed creek alignment, proposed bank repairs, and channel treatments, would satisfy the required factor of safety (FoS) for the slopes of concern.

9.2 Recommendations

The following recommendations have been prepared based on the findings of this study:

9. The preferred alternative identified from this assessment, Alternative 7b Realignment, Structures and Hardening (Structures Focused), has been recommended to be advanced to the next phase of planning and design, with a estimated cost of $2.6M based on a Class cost estimate (ref. Appendix I). The six (6) control structures will dissipate the channel energy while the realignment of the channel will move the channel away from the slopes of concern to facilitate the slope stabilization measures.
10. The City should continue monitoring slopes 6 and 7 until construction has been completed to document the condition at those slopes. Monitoring of the ravine to ensure progressive creek bank erosion does not destabilize slopes prior to construction of the recommended creek improvement sand slope protection works is recommended. Based on Stantec's March 2022 Slope Stability Observation Program memorandum and the observation of soil being placed at 13 Sprucewood, a temporary 0.5 m high stone buttress at the toe of slope behind 13 Sprucewood, could be implemented to prevent slope instability
11. Slope stabilization works recommended for the four (4) slopes of concern include placing clay fill material within the former channel, in addition to riprap. The estimated cost for the fill material is $361,000.
12. The slopes and channel banks should be revegetated following the construction works with native tree species, shrubs, and ground cover and the City's forestry department should be consulted when identifying the species for the site. Cedars are recommended to be planted to address residential privacy concerns as the cedar bushes will provide year-round visual barrier.

13. Three (3) maintenance access paths should be constructed, not only to provide access during construction, rather to also provide access following construction should maintenance be required at or near the channel and the control structures; the estimated cost of the maintenance assess paths is $26,000.
14. It is recommended that the number of trees to be removed is kept to a minimum to reduce the amount of disturbance to potential bat species at risk habitat; 77 trees have been identified for removal to facilitate the construction of the preferred alternative while 40 additional trees can likely be preserved if there is no interference during construction.
15. The use of LID BMPs should be promoted in conjunction with other alternatives as a long-term objective to reduce the runoff conveyed to the Blue Willow SWM facility. Implementation of the LID BMPs within the public realm could be undertaken during the reconstruction of the roadways. Similarly, LID measures could be implemented within the private properties, for example by redirecting downspouts to pervious areas, or by installing permeable pavers instead of asphalt driveways at the time of reconstruction.
16. During detailed design, consultation should be undertaken with agencies, First Nations groups, and neighbourhood associations and residents to ensure their understanding of the proposed mitigation works and to address any concerns for the project.

Report prepared by,

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Appendix A:
Public Consultation

Appendix B:
Background Data

Appendix C:
PCSWMM Rating Curve Validation

Appendix D:
Inflow Rate Summation

Appendix E:

Stream Morphology Existing Conditions Report

Appendix F:

Geotechnical Existing Conditions Report and Retrogressive Failure Technical Memorandum

Appendix G:

Species at Risk Technical Memorandum and Tree Inventory

Appendix H:

Stream Morphology Mitigation Alternatives

Appendix I:
Preliminary Cost Estimate

Appendix J:

Existing and Proposed Conditions Continuous Simulation Erosion Assessment

Appendix K:

Geotechnical Alternative Assessment