ANNEXE QC-246

Mémo technique – Évaluation des bermes de stabilité

MÉMORANDUM TECHNIQUE

Date: 10 septembre 2015 **N° de référence:** QC-246-1527286-20100-MTF-Rev0

À: Christine Baribeau
Mine Canadian Malartic

c.c: Christian Roy, Pascal Lavoie, Carl Pedneault

De: Nicolas Pepin, Karine Doucet et Mayana Kissiova **Adresse courriel:** kdoucet@golder.com

ÉVALUATION DES BERMES DE STABILITÉ NÉCESSAIRES DURANT L'OPÉRATION, PARC À RÉSIDUS, MINE CANADIAN MALARTIC, MALARTIC, QC

1.0 INTRODUCTION

Ce mémorandum technique a pour objectif de répondre à la première partie de la question 246 formulée par le ministère du Développement durable, de l'Environnement et de la Lutte contre les changements climatiques (MDDELCC) :

QC-246 *L'initiateur écrit que la condition postliquéfaction est une condition particulière, dont l'occurrence dépend d'une sollicitation et qu'il n'est pas jugé que les bermes calculées soient nécessaires d'une façon immédiate. Elles, ou toute autre mesure de mitigation pourront être implantées tout au long de la vie du parc. Dans la section « Contingences », il est également écrit que si un séisme fort venait à causer une liquéfaction des résidus, on peut s'attendre à des affaissements locaux ou des déplacements et que cette situation pouvait être gérée par l'ajout de matériel au pourtour, si nécessaire, une fois les surpressions interstitielles dissipées. L'initiateur doit expliquer comment il s'assurera de ne pas avoir d'épanchement de résidus. Il indiquera les mesures d'atténuation à mettre en place en plus de s'engager à les mettre immédiatement en place. […]*

2.0 MISE EN CONTEXTE

Comme présenté dans le rapport de conception du prolongement du parc à résidus[1], les mesures de mitigation proposées dans le cas de post-liquéfaction étudié prennent la forme de surépaisseur de certains rehaussments, qui peut être également référer à l'ajout de bermes de stabilité. . Afin de déterminer l'épaisseur de certains rehaussments et l'ampleur de l'ajout de berme au dessus de la berme de départ, nécessaires au maintien de l'intégrité structurelle du parc à résidus en cas d'une liquéfaction des résidus saturés durant l'opération, les modèles numériques développés lors de la mise à jour de la conception du parc à résidus de 2014 ont été utilisés et une évaluation a été effectuée à différents stades de développement du parc à résidus.

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[1] Annexe 8-1 de l'EIE, Golder 2014a. *Conception du prolongement du parc à résidus et de la halde à stériles, Projet d'extension de la Mine Canadian Malartic*, pour Canadian Malartic GP. Novembre 2014. Doc. 008-1403061-4000-Rev. 0.

La surépaisseur nécessaire a été évaluée pour le parc à résidus lorsque celui-ci atteindra le tiers (1/3) de sa hauteur finale de conception et les deux tiers (2/3) de celle-ci; les bermes de stabilité nécessaires à la pleine hauteur étant déjà présentées dans le rapport de conception[1]. Les analyses ont été faites pour deux coupes typiques du parc à résidus actuellement en opération soient à la digue 5 et à l'ancienne digue 4 (berme de départ ouest). Ces deux coupes présentent les deux conditions principales rencontrées au parc à résidus soient; une première coupe où le niveau d'eau anticipé dans le parc serait près de la surface des résidus, dû à la faible conductivité hydraulique de la digue 5 (situé au pied aval de l'empilement) et une deuxième coupe où le niveau d'eau est anticipé être plus bas que la première coupe, dû à la plus grande conductivité hydraulique de la berme de départ (situé au pied aval de l'empilement).

3.0 MÉTHODOLOGIE

La suite de la compagnie Géostudio a été utilisée pour définir les bermes de stabilité nécessaires pour les cas analysés. La méthodologie utilisée est la suivante:

- La stratigraphie, les propriétés des sols et le taux de déposition utilisés dans les modèles ont été conservé tel que défini dans le rapport de conception[1];
- Les surpressions d'eau générées dans les couches d'argile, causées par le poids des résidus miniers, des stériles miniers et des bermes de stabilité ont été déterminées à l'aide du logiciel SIGMA/W;
- La nappe phréatique a été définie à l'aide du logiciel SEEP/W dans le cas de la digue 4 et a été conservée tel quel dans le cas de la digue 5 afin d'être cohérent avec les analyses utilisées pour la configuration ultime;
- La distribution des bermes de stabilité pour les deux coupes étudiées a été déterminée à l'aide du logiciel SLOPE/W. Ces bermes ont été ajoutées de façon itérative jusqu'à l'obtention de facteurs de sécurité égal ou plus grands que ceux établis pour des conditions d'opération;
- Le facteur de sécurité minimum à rencontrer durant l'opération, en condition post-liquéfaction, doit être supérieur à 1,0;
- La stabilité selon les coupes étudiées a été évaluée pour différents types de rupture.

4.0 RÉSULTATS

4.1 Coupe typique - secteur de la digue 4

Selon les résultats obtenus pour la coupe étudiée à la digue 4 et en fonction des paramètres de résistance utilisés, aucun ajout de bermes de stabilité n'est nécessaire dans ce secteur durant l'opération. Cette conclusion est valable pour la configuration étudiée pour un parc rehaussée au 1/3 et aux 2/3 de sa hauteur finale de conception. Le facteur de sécurité obtenu pour une stabilité en condition statique, pseudo-statique et postliquéfaction, respecte les facteurs de sécurité minimaux requis. La figure 1 illustre la coupe à la digue 4 au 1/3 et aux 2/3 de sa hauteur finale de conception avec en condition post-liquéfaction.

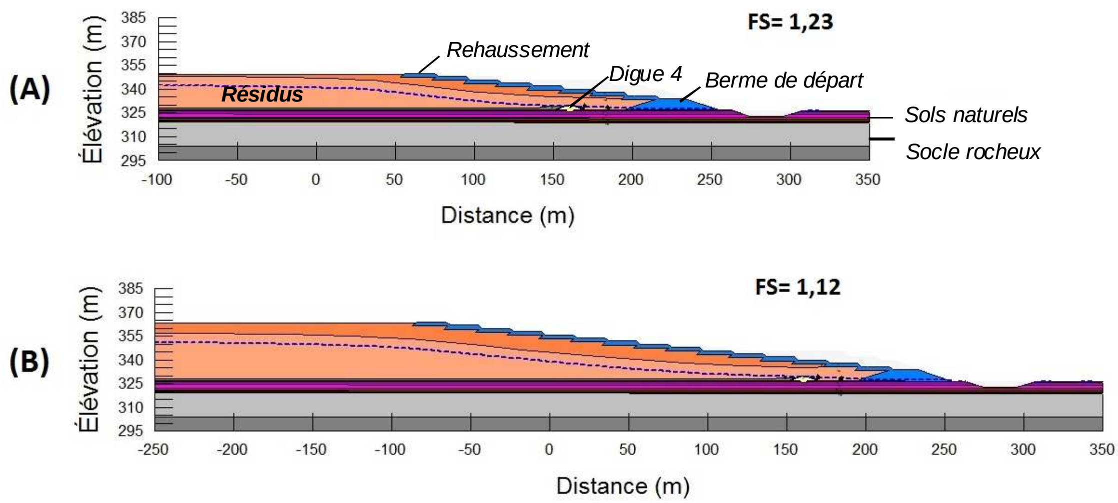

Figure 1: Secteur Digue 4, analyses de stabilité en condition post-liquéfaction pour: A) un rehaussement au 1/3 de sa hauteur maximale de conception et B) un rehaussement au 2/3 de sa hauteur maximale de conception.

Il est à noter que le comportement du parc doit faire l'objet d'un suivi et d'une vérification de la performance tout au long de sa vie. L'évaluation actuelle est faite en se basant sur les résultats d'essais en laboratoire sur les résidus et les matériaux naturels et des données *in-situ* obtenues lors de la caractérisation des résidus effectuées en septembre 2013. Une vérification de l'état *in-situ* des résidus doit être faite à chaque 5 ans environ afin de pourvoir mettre à jour leur paramètres de résistance. Nous croyons que l'évaluation faite en 2013 est suffisamment récente afin de permettre une évaluation prudente du comportement du parc pour l'empilement au tiers. Cependant, une mise à jour de la caractérisation des résidus en place pour l'empilement au deux tiers devrait être effectuée lorsque des résultats *in situ* seront disponibles.

4.2 Coupe typique – secteur de la digue 5

Selon les résultats de notre étude, dans le cas de la coupe à la digue 5, des bermes de stabilité devront être mises en place en aval des différents rehaussements et en aval de la berme de départ en cours d'opération. La figure 2 montre la configuration des bermes nécessaires lorsque le parc est au 1/3 de sa hauteur finale de conception. Il est à noter que cette configuration reflète les besoins en conditions de post-liquéfaction puisqu'elles se sont avérées, comme par le passé, le cas le plus critique. La hauteur des bermes de stabilité nécessaires varie entre 2 m et 6 m. Les structures n'ont pas besoin de bermes de stabilisation pour les conditions statiques ou pseudo-statiques.

Il est cependant important de considérer qu'une fraction non-négligeable des stériles mis en place lors des rehaussements pénètre les résidus et/ou les déplace. La fondation des rehaussements est donc constituée d'une matrice rocheuse et de résidus, plutôt que de résidus seulement, tel que modélisé. La Mine est en train d'établir l'épaisseur réelle de pénétration des stériles, ainsi que la hauteur résultantes des rehaussements actuelles, d'une façon systématique. Les modèles seront mis à jour par la suite afin de refléter encore mieux la situation réelle. Les hauteurs des bermes établies par les modélisations déjà réalisées, tel qu'illustré à la figure 2, fournissent un guide en terme d'envergure nécessaire pour la stabilisation des ouvrages en post-liquéfaction et représentent une évaluation prudente qui peut être d'avantage rafinée.

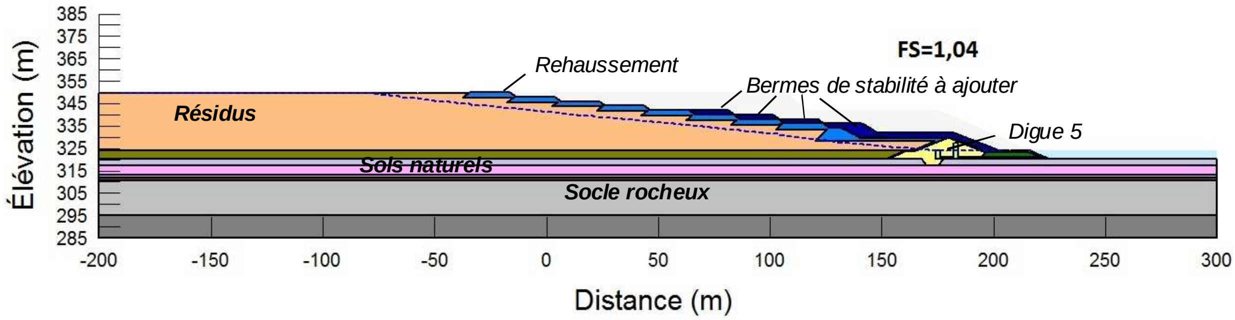

Figure 2: Digue 5, analyses de stabilité en condition post-liquéfaction pour un rehaussement au 1/3 de sa hauteur maximale de conception.

Pour ce qui est des résultats des analyses de la coupe à la digue 5 aux 2/3 de la hauteur finale de conception du parc, des bermes de stabilité doivent aussi être ajoutées pour respecter les facteurs de sécurité ciblés. Toutefois, tel que mentionné plus haut, l'acquisition de nouvelles données de terrain doit être faite dans le futur, dans le cadre de la mise à jour de la caractérisation des résidus prévue à chaque 5 ans ainsi qu'établir, de façon systématique l'épaisseur de pénétration des stériles dans les résidus. Ces données devraient permettre de mettre à jour les paramètres de résistances utilisés dans nos analyses et d'évaluer si des ajustements sont nécessaires lorsque le parc atteindra cette hauteur. La figure 3 montre, en termes de guide, l'ampleur des bermes nécessaires pour la coupe à la digue 5 aux 2/3 de la hauteur finale du parc selon les paramètres utilisés actuellement et basé sur l'évaluation *in-situ* récente. La hauteur des bermes de stabilité varie entre 2 m et 10 m.

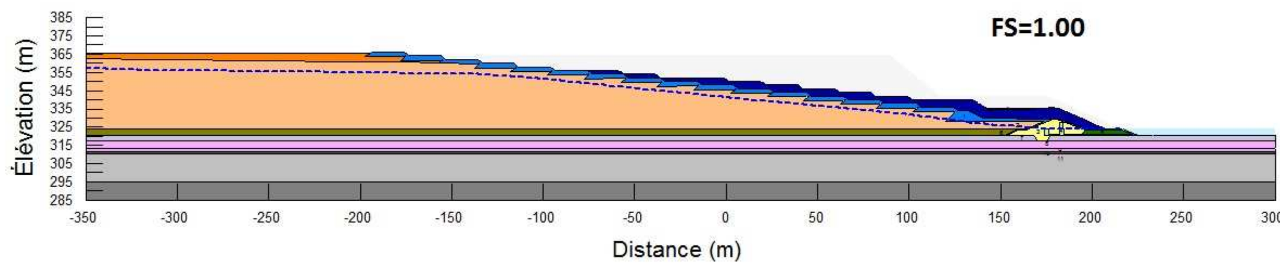

Figure 3: Digue 5, analyses de stabilité en condition post-liquéfaction pour un rehaussement au 2/3 de sa hauteur maximale de conception.

5.0 INTERVENTION RECOMMANDÉE DURANT L'OPÉRATION

Les résultats des analyses de stabilité, selon les données de caractérisation des résidus actuelles, montrent que certains rehaussements, constitués de stériles, doivent avoir une plus grande épaisseur que le 2 m modélisés, pour la configuration étudiée dans le secteur de la digue 5 pour un empilement atteignant environ 1/3 de son hauteur finale (figure 2). Selon la planification actuelle, le parc atteindra l'élévation correspondant au tiers de l'empilement au courant des prochaines années (2016-2017).

Selon l'information disponible, il est évalué que la configuration des bermes de stabilité pour la coupe modélisée de la digue 5 doit être étendue sur un certain périmètre pour refléter les différentes conditions possibles dans le secteur. Le périmètre en jaune illustré sur la figure 4 représente l'étendue minimale recommandée de mise en place des bermes.

Il est cependant important de rappeler que, selon nos modélisations, les bermes ne sont nécessaires que dans un cas de liquéfaction complète de la masse de résidus suite à un séisme de conception (probabilité d'occurrence de 1 dans 2475 ans). L'analyse a été faite aussi en supposant un cas très prudent, où la pénétration de stériles dans la fondation ou la consolidation progressive des résidus, n'est pas prise en compte. Ces facteurs sont susceptibles de changer dans le temps. Les investigations prévues au parc à résidus sur une base régulière au courant des prochaines années vont apporter de nouvelles informations. Il sera donc essentiel de mettre à jour les modèles régulièrement, afin de définir si une stabilisation supplémentaire est nécessaire, sans imposer des contraintes complémentaires à l'opération. Les bermes proposées ont déjà une configuration minimale pour tenter de limiter le déplacement de matériaux et peuvent être mises en place selon au courant des prochaines années compte tenu de la faible occurrence de l'évènement pris en compte.

Le périmètre en orange sur la figure 4 représente, quant à lui, le secteur ne nécessitant pas d'interventions à court terme, selon les résultats d'analyses pour la coupe dans le secteur de la digue 4. Aucune berme n'est nécessaire dans ce cas pour un empilement au tiers de son élévation ultime.

Figure 4: Vue en plan du parc à résidus: périmètre en jaune indiquant l'étendu de la mise en place de bermes de stabilité à court terme, tel que recommandée pour le secteur de la digue 5. Périmètre en pointillée orange indique l'étendu de secteurs ne nécessitant pas l'ajout de berme de stabilité durant l'opération, en fonction des résultats des modélisations effectuées pour le secteur de la digue 4.

Le secteur de la berme de départ est deviendra dans la prochaine année une structure interne au parc. Une nouvelle cellule sera construite à l'est, dans le bassin de polissage actuel en 2015. Ce secteur n'a donc pas fait l'objet d'évaluation de la nécessité de mise en place de mesures de mitigation.

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Annexe A : Conditions générales et limitations (Rapport géotechnique)

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ANNEXE A
Limitations de l'étude

CONDITIONS GÉNÉRALES ET LIMITATIONS
RAPPORT GÉOTECHNIQUE

UTILISATION DU RAPPORT ET DE SON CONTENU

Ce rapport a été préparé pour l'usage exclusif du Client ou de ses agents. Les données factuelles, les interprétations, les commentaires ainsi que les recommandations qu'il contient sont spécifiques au projet tel que décrit dans ce rapport et ne s'appliquent à aucun autre projet ou autre site. Ce rapport doit être lu dans son ensemble, puisque des sections pourraient être faussement interprétées lorsque prises individuellement ou hors contexte. Par ailleurs, le texte de la version finale de ce rapport prévaut sur tout autre texte, opinion ou version préliminaire émis par Golder. Si la conception, l'emplacement ou l'élévation du projet doivent être modifiés et/ou si le projet n'est pas amorcé à l'intérieur d'une période de 18 mois suivant la remise de ce rapport, Golder devrait être consultée pour confirmer que ses recommandations sont encore valides.

Les commentaires, interprétations et recommandations présentés dans ce rapport sont basés sur une évaluation limitée des conditions souterraines tel que décrit ailleurs dans ce texte et sont formulés dans le seul et unique but d'orienter la conception du projet. À moins d'avis contraire, les interprétations, commentaires et les recommandations présentés dans ce rapport ont été formulés à la lumière de nos connaissances concernant les conditions du site, l'utilisation courante et/ou prévue du site, les règlements, normes et critères en vigueur de même que les règles et pratiques professionnelles reconnues et acceptées au moment de l'étude, tenant compte dans tous les cas de l'emplacement du site. Les références aux lois et règlements contenues dans ce rapport sont fournies à titre indicatif, sur une base technique. Comme les lois et règlements sont sujets à interprétation, Golder recommande au Client de consulter ses conseillers juridiques afin d'obtenir les avis appropriés.

Comme certains détails du projet envisagé peuvent ne pas être connus de Golder au moment de la remise de ce rapport, il est recommandé que Golder soit consultée lors de l'élaboration des plans et devis reliés aux considérations géotechniques afin de s'assurer qu'ils demeurent conformes à l'intention et aux recommandations de ce rapport.

Il est aussi recommandé que les services de Golder soient retenus durant la phase de construction afin de confirmer que les conditions souterraines sur l'ensemble du site ne diffèrent pas de façon significative de celles évoquées dans ce rapport et que les activités de construction n'ont aucun impact négatif sur les considérations géotechniques liées à la conception. À cet égard, il importe de souligner que le contrôle des eaux superficielles et/ou souterraines est fréquemment requis comme mesure temporaire ou permanente lors de la construction. Une mauvaise conception du drainage et/ou de l'assèchement peut avoir des conséquences néfastes. De même, les conditions souterraines peuvent être substantiellement modifiées par les activités de construction (circulation de machinerie, excavation, enfoncement de pieux, dynamitage, etc.) ayant cours sur le site ou sur les terrains adjacents ainsi que par l'exposition des sols aux intempéries (gel, sécheresse, pluie, etc.).

Golder ne pourra être tenue responsable de conditions souterraines imprévisibles ni de leurs impacts sur les coûts de construction et l'échéancier de réalisation des travaux. Golder ne pourra être tenue responsable de dommages résultant de conditions qui lui seraient inconnues, de l'inexactitude de données provenant d'autres sources que Golder et de changements ultérieurs aux conditions du site. Golder n'acceptera aucune responsabilité pour les effets de mesures de drainage et/ou d'assèchement à moins d'avoir été spécifiquement consultée et impliquée dans la conception et le suivi du système de drainage et/ou d'assèchement. Golder ne pourra être tenue responsable de dommages résultant de toutes modifications futures aux règlements, normes ou critères applicables de même que de toute utilisation faite du présent rapport par un tiers et/ou à des fins autres que celles pour lesquelles il a été rédigé, de perte de valeur réelle ou perçue du site ni de l'échec d'une quelconque transaction en raison des informations factuelles contenues dans ce rapport.

Le Client de même que tout entrepreneur réalisant des travaux qui s'inspirent de ou qui sont susceptibles d'avoir une incidence sur les considérations géotechniques évoquées dans ce rapport doivent informer

Golder ainsi que l'ingénieur concepteur de tout événement, activité, information, découverte passé, présent ou future susceptible de modifier les conditions souterraines décrites dans ce rapport et leur offrir la possibilité de réviser leurs recommandations ainsi que les plans de construction. Cette obligation couvre aussi le cas où les conditions rencontrées sur le site différeraient de façon significative de celles anticipées dans ce rapport, soit en raison de la variabilité naturelle des conditions souterraines ou en raison d'activités de construction. Il est entendu que la reconnaissance d'un changement des conditions du sol et du roc nécessite qu'un examen soit effectué sur le site par un professionnel qualifié et expérimenté dans la pratique de la géotechnique.

ÉVALUATION DES CONDITIONS SOUTERRAINES

Les travaux d'investigation souterraine effectués par Golder et décrits dans ce rapport furent réalisés conformément aux règles et pratiques professionnelles reconnues et acceptées au moment de leur réalisation. À moins d'avis contraire, les résultats de travaux antérieurs ou simultanés, provenant d'autres sources que Golder, cités et/ou utilisés dans ce rapport furent considérés comme ayant été obtenus en respectant les règles et pratiques professionnelles reconnues et acceptées et comme étant valides.

Les horizons de sols et de roc étant souvent de composition et de géométrie très variables, les descriptions de sondage ne permettent donc que d'estimer approximativement leurs caractéristiques et profils réels. Les contacts entre les différents horizons de sols et/ou de roc sont souvent graduels et, conséquemment, leurs emplacements sur les descriptions de sondage relèvent d'une certaine interprétation. De même, la classification et l'identification des sols et du roc implique une certaine part de jugement. Les descriptions de sol et de roc apparaissant dans ce rapport s'appuient sur des méthodes de classification et d'identification communément acceptées et rejoignent les exigences normales de la pratique professionnelle usuelle de la géotechnique. Par ailleurs, il importe de souligner que la précision des données recueillies et leur interprétation sont tributaires de différents facteurs dont la méthode de sondage, l'espacement entre les sondages, la profondeur d'investigation, la méthode d'échantillonnage, la fréquence d'échantillonnage de même que l'uniformité des conditions souterraines. Certains de ces facteurs, comme la méthode de sondage, l'espacement entre les sondages, la profondeur d'investigation, la méthode d'échantillonnage et la fréquence d'échantillonnage peuvent eux-mêmes être tributaires de contraintes physiques, budgétaires ou d'échéancier convenues avec le Client.

Dans tous les cas, on doit considérer que les résultats obtenus et présentés dans ce rapport ne s'appliquent qu'aux endroits où ont été réalisés les sondages, qu'aux profondeurs d'échantillonnage indiquées et qu'au moment de l'étude. Les conditions souterraines interprétées, tant physiques que quantitatives ou qualitatives, peuvent varier sensiblement entre et au-delà des sondages réalisés et des profondeurs d'échantillonnage indiquées.

Les mesures et caractéristiques de l'eau souterraine présentées dans ce rapport ne sont valables que pour les endroits et les dates spécifiées. Ces conditions peuvent en effet varier selon les saisons, les années ou en raison d'activités ou d'événements sur le site à l'étude ou sur des terrains adjacents.

May 2009

FINAL REPORT

POLISHING POND DYKE BREACH FLOOD INUNDATION STUDY FOR THE MALARTIC PROJECT

Submitted to:
Carl Pednault and Michel Julien
Golder Associates Ltd.
Montreal, Quebec

FINAL REPORT

Report Number: 07-1221-0028.4000.4100

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May 12, 2009

Golder Associates Ltd.
9200, boul. de l'Acadie, bureau 10
Montréal, Québec
Canada H4N 2T2

Attention: Mr. Michel Julien and Mr. Carl Pednault

Re: Final Report – Polishing Pond Dyke Breach Flood Inundation Study for the Malartic Project

Dear Mr. Julien and Mr. Pednault:

We are pleased to submit five hard copies of our final report on the Polishing Pond Dyke Breach Flood Inundation Study for the Malartic project. The final report includes one CD containing one digital copy of the report and the relevant digital inundation map at the back of each report. Please contact the undersigned should you require any clarification regarding this report submission.

Yours truly,

GOLDER ASSOCIATES LTD.

Moniz Mukto, M.Sc.
Water Resources Specialist

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Senior Water Resources Engineer

ACKNOWLEDGEMENTS

The contributions of the following staff of Golder Associates Ltd. to this Polishing Pond Dyke Breach Flood Inundation Study for the Malartic Project are acknowledged:

- Mr. Michel Julien, Project Director for this study, provided overall direction, senior advice and review comments on the study report;
- Mr. Pednault, Project Manager for this study, provided the available data and relevant information from Osisko Mining Corporation (Osisko), coordinated the inputs and participation from Osisko, participated in the field reconnaissance, and provided comments on the study report;
- Mr. Moniz Mukto was responsible for conducting the detailed hydrodynamic modeling analysis including model setup, testing, and preparation of the flood inundation map and study report;
- Dr. Oscar Kalinga was a task manager for conducting the hydrological analysis (i.e. estimation of the Probable Maximum Flood [PMF]). Mr. Murray Fitch and Dr. Getu Biftu also provided senior advice to Dr. Kalinga for the estimation of PMF;
- Mr. Jie Chen assisted Mr. Mukto in conducting the level-pool routing analysis;
- Dr. Hua Zhang was responsible for overseeing the dyke breach modeling analysis, and reviewing the dyke breach flood inundation map and the study report; and,
- Dr. Dejiang Long, a senior reviewer, provided technical guidance and senior input to the study, provided quality control and assurance for the modeling analysis and map preparation, and reviewed the study report.

Executive Summary

Golder Associates Ltd. was commissioned by Osisko Mining Corporation (Osisko) to conduct a dyke breach flood inundation study of a proposed Polishing Pond for the Malartic Project in southwestern Quebec. The main purpose of the study was to prepare a dyke breach flood inundation map for supporting the project hearing.

The proposed Polishing Pond has a water volume of about 6 million m^3 and a surface area of 1.38 km^2 at the operating level of 325.0 m. The Polishing Pond is contained by five dykes. The dyke breach analysis was conducted for two dykes; EM-A and EM-B. The heights of Dykes EM-A and EM-B above the valley bottom are approximately 13.5 m and 9.5 m, respectively.

The FLDWAV model (Version 2-0-0 dated June 1, 2000) developed by the U.S. National Weather Service was used to simulate the outflows from the proposed Polishing Pond and through the downstream valley resulting from potential breaches in Dykes EM-A and EM-B. The study reach length for Dyke EM-A breach flood routing is about 16 km, from Dyke EM-A to Thompson River at Dubuisson. The main structures along the potential dyke breach floodway include a road crossing on the Ruisseau Raymond reach and three road crossings on the Piche River reach. The populated areas along the dyke breach floodway include the Northern Star Mine along route 117 and Dubuisson.

The floodway for a potential Dyke EM-B failure flood is along an Unnamed Creek between Dyke EM-B and Lake Fournière, with a study reach length of about 1.8 km. The main structures along the potential dyke breach floodway are one snowmobile trail and a local road crossing on the Unnamed Creek. The only populated area along the floodway is adjacent to the Rang 7 Road.

Two dyke breach scenarios were analyzed. One scenario was a dyke overtopping failure under an extreme flood event (i.e., design PMF). The other scenario was a fair weather piping failure at the operating water level of 325.0 m in the proposed Polishing Pond. The scenarios were analyzed for Dykes EM-A and EM-B.

A sensitivity analysis was conducted to assess the model's sensitivity to three key model parameters. The analysis shows that the predicted maximum flood levels are more sensitive to the assumed average breach width and less sensitive to the assumed time to failure and estimated Manning's n value.

The modeling results were used to prepare a dyke breach flood inundation map at a scale of 1:50,000. The map shows the flood inundation extent delineation on a base map with 10-m contours, roads and other mapping features. The flood inundation extent mapping shows the areal extents of the potential dyke breach floods.

The modeling results and the flood inundation map show the following:

- In the event of both piping and overtopping failure, all the local road crossings along the floodways of Dykes EM-A and EM-B would likely be overtopped and damaged;
- For Dyke EM-A, the flood peak levels associated with the piping failure flood are approximately 0.6 m lower, on average, than those associated with the overtopping failure flood. The expected flood peak discharges associated with the piping failure flood are 29% smaller, on average, than those associated with the overtopping failure flood. The flood peak levels associated with the piping failure flood would generally arrive at a downstream location later than the overtopping failure flood by up to 1.3 hours;
- For Dyke EM-B, the flood peak levels associated with the piping failure flood are approximately 1.1 m lower, on average, than those associated with the overtopping failure flood. The expected flood peak discharges associated with the piping failure flood are 56% smaller, on average, than those associated with the overtopping failure flood. The flood peak levels associated with the piping failure flood would generally arrive at a downstream location later than the overtopping failure flood by up to 0.6 hours; and,
- In the event of Dyke EM-A failure, some of the residential houses or buildings near Northern Star Mine along route 117 would be flooded as shown on the flood inundation map. In the event of failure of Dyke EM-B, only one residential house or building which is located close to Lake Fournière would be flooded as shown on the flood inundation map. All other houses within Dykes EM-A and EM-B floodways would not be flooded due to their higher ground elevations and based on the predicted flood peak levels. However, it should be noted that the survey was conducted at select and representative house locations.

This dyke breach flood inundation study was conducted based on the available topographic data for the study reaches of the floodways. Efforts were spent in collecting spot elevation data in select locations. However, most of the topographic information used in setting up the model and preparing the flood inundation map was obtained from the 10-m contour map. This introduces some level of uncertainty in the modeling results and flood inundation mapping.

It is recommended that the client develop an improved topographic base map with one-meter contours, and with a preferred elevation accuracy of ±0.5 m. The updated base map should be used for updating the dyke breach inundation modeling analysis and mapping to provide a better reliable basis for preparing the EPP (Emergency Preparedness Plan) and ERP (Emergency Response Plan) in accordance with the 2007 CDA Dam Safety Guidelines.

Table of Contents

1.0 INTRODUCTION

1.1 Background

Golder Associates Ltd. was commissioned by Osisko Mining Corporation (Osisko) to conduct a dyke breach flood inundation study of a proposed New Polishing Pond for the Malartic Project in southwestern Quebec. The main purpose of the study was to prepare a dyke breach flood inundation map. The study was conducted based on available data, and in accordance with industry-accepted standards and the 2007 Canadian Dam Association's (CDA) Dam Safety Guidelines for inundation studies.

1.2 Study Area

Figure 1 shows the study area, including the location of the dykes, and the floodway study reaches for analyzing potential breaches of the dykes.

The proposed New Polishing Pond has a water volume of about 6 million m^3 and a surface area of 1.38 km^2 at the operating level of 325.0 m. The Polishing Pond is contained by five dykes. The dyke breach analysis was conducted for two of these dykes (EM-A and EM-B). The heights of Dykes EM-A and EM-B above the valley bottom are approximately 13.5 m and 9.5 m, respectively.

The study reach length for Dyke EM-A beach flood routing is about 16 km, from the Dyke EM-A to Thompson River at Dubuisson. The main structures along this potential dyke breach floodway include a snowmobile trail on the Ruisseau Raymond reach and three local road crossings on the Piché River reach. The populated areas along the dyke breach floodway include the Northern Star Mine along route 117 and Dubuisson.

The floodway for a potential Dyke EM-B failure flood is along an Unnamed Creek between Dyke EM-B and Lake Fournière, with a study reach length of about 1.8 km. The main structures along this potential dyke breach floodway are one snowmobile trail and a local road crossing on the Unnamed Creek. The only populated area along the floodway is adjacent to the Rang 7 Road.

1.3 Methodology

The FLDWAV model (Version 2-0-0 dated June 1, 2000) developed by the U.S. National Weather Service (NWS) was used to simulate the outflows from the proposed New Polishing Pond and through the downstream valley resulting from potential breaches of Dykes EM-A and EM-B.

In accordance to the 2007 CDA Dam Safety Guidelines, the modeling analysis included simulation of potential piping and overtopping failure scenarios to predict the dyke breach flood peak water levels and flood peak discharges along the study reaches of the Ruisseau Raymond Creek and Piché River downstream of the proposed Dyke EM-A and along the study reach of the Unnamed Creek downstream of the proposed Dyke EM-B. The simulated flood peak levels associated with piping and overtopping failure scenarios were used for mapping the flood inundation extents.

The study does not imply that the selected dyke breach scenarios have a high probability of occurrence but the study was conducted for estimating the flood extents resulting from a potential breach, in the extreme event that it would occur. Two main scenarios of potential dyke breach are described below.

Overtopping Failure Mode

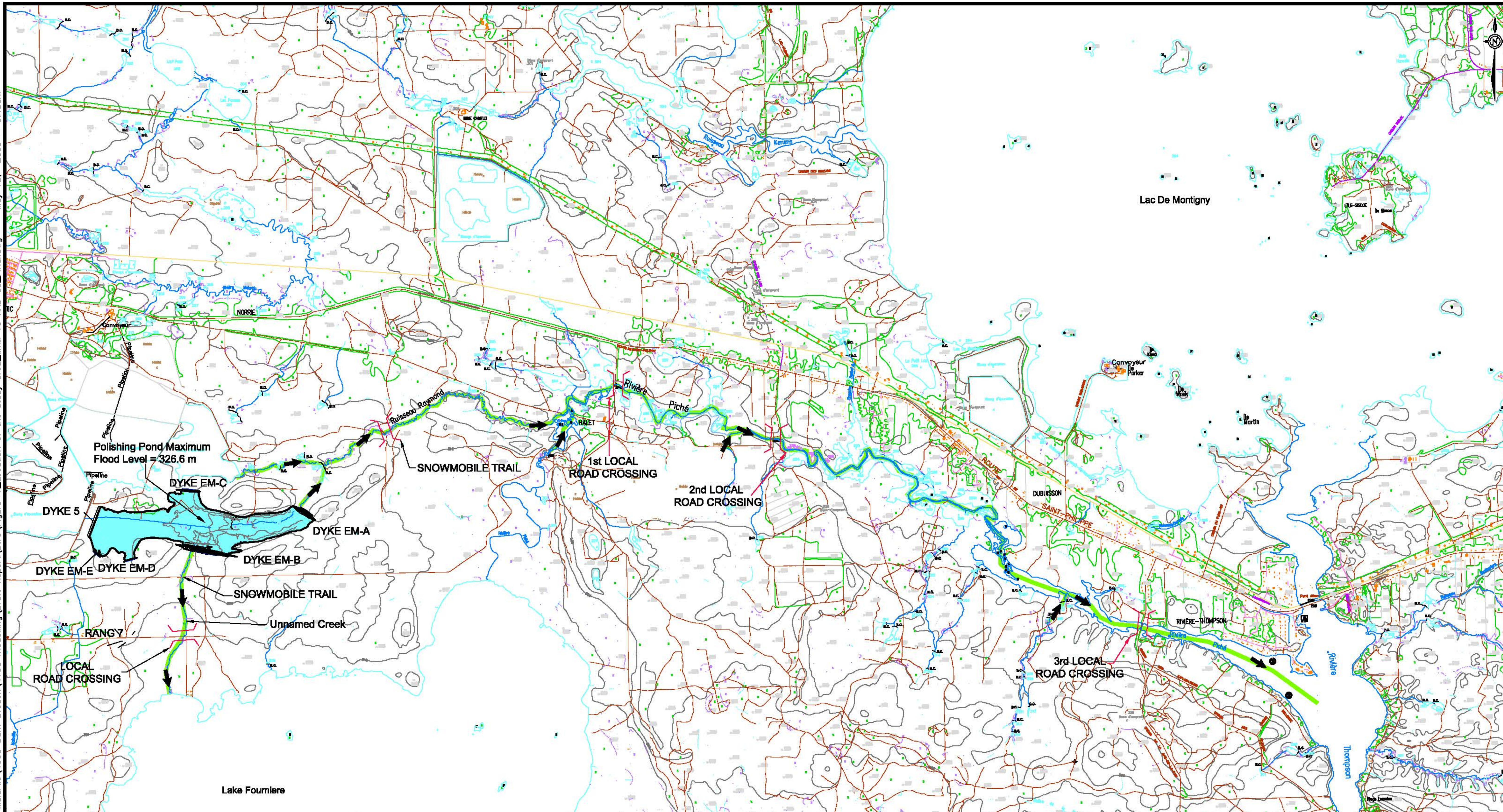
The overtopping failure mode was analyzed for both Dykes EM-A and EM-B under the most extreme flood event (i.e. Probable Maximum Flood) with the assumption of having a free overflow spillway. The free overflow spillway was assumed as it is self operating (i.e. does not require operator assistance) and does not have equipment that could potentially malfunction.

Piping Failure Mode

The piping failure mode was analyzed for both Dykes EM-A and EM-B under fair weather conditions. This type of failure is associated with seepage induced dyke failure.

1.4 Organization

A brief description of the U.S. NWS's FLDWAV model is presented in Section 2. Detailed descriptions about available information, dyke breach modeling methodology and results including the sensitivity analysis for the proposed Dykes EM-A and EM-B are presented in Section 3. Description on the preparation of dyke breach flood inundation mapping and affected areas are presented in Section 4. Section 5 presents the conclusions and recommendations resulted from the dyke breach flood inundation study.



LEGEND

- FLOW DIRECTION
- STUDY REACH

1 0 1

SCALE KILOMETRES

REFERENCE

ORIGINAL DRAWINGS OBTAINED FROM CLIENT. 32c04201h.dwg, 32c04201t.dwg, 32cd101t.dwg, 32da202t.dwg 1:20,000 TOPO MAPS CONVERTED TO MTM ZONE 10

PROJECT

POLISHING POND DYKE BREACH FLOOD INUNDATION STUDY FOR THE MALARTIC PROJECT

TITLE

LOCATION OF THE STUDY AREA

Golder Associates
Calgary, Alberta

PROJECT	07.1221.0028.4000		FILE No.	Study Location
DESIGN	MM	14/10/08	SCALE AS SHOWN	REV. 1
CADD	TY	07/05/09		
CHECK	MM	07/05/09	FIGURE: 1	
REVIEW	HZ	07/05/09		

R:\Active_2007\1221\07-1221-0028 Malartic\4000 Dam Breach\4100 modelling\Draft Report\DWG\Figure-1_Location of the study area_MTM 10 NAD 83_Final.dwg May 12, 2009 - 8:48am

2.0 DYKE BREACH MODEL

The FLDWAV model was selected for this dyke breach inundation study for the Malartic project. The FLDWAV model combines the capabilities of DWOPER and DYKEBRK models developed by the U.S. NWS and supplies additional modeling features.

The hydraulic modeling analysis was conducted using the latest official version of the FLDWAV model (version 2-0-0 dated June 1, 2000) to simulate potential dyke breaches and downstream routing of the resulting dyke breach floods. The model is the most commonly used tool in North America and used as an industry-standard approach for the prediction of the dyke breach flood inundation studies.

FLDWAV is a generalized flood routing model for unsteady or dynamic flow simulation. The governing equations contained in the model are the complete one-dimensional Saint-Venant equations of unsteady flows. Internal boundary equations included in the model, representing the rapidly varied (broad-crested weir) flows through structures (e.g., dykes, bridges and embankments), enable users to specify time-dependent breach. In addition, the model allows the appropriate external boundary equations at the upstream and downstream ends of the routing reach to be specified.

The system of equations in the FLDWAV model is solved by an iterative, nonlinear, weighted four-point implicit finite-difference method. The flows that can be simulated may be either subcritical or supercritical, or a combination of each varying in space and time from one to the other. Fluid properties that can be simulated may obey either the principles of Newtonian or non-Newtonian flows. The hydrograph to be routed may be user-specified as input time series, or it can be developed by the model via user-specified breach parameters, including size, shape and time of development. More detail information about the FLDWAV model can be found the NWS documentation (NWS, 1998).

The FLDWAV model is designed to account for the following effects on the downstream propagation of a flood:

- Bridges and embankments that restrict the flood flows locally;
- Tributary inflows;
- River sinuosity;
- Downstream dykes that have significant flood storage and may be breached by the flood;
- Levees located along the flood route; and,
- Tidal effects.

The standard output from the FLDWAV model includes flood peak levels along the river/valley, time to peak flood levels, flood peak discharges, and discharge and stage (i.e. water surface elevation) hydrographs at the user-selected locations.

3.0 MODELING ANALYSIS

3.1 Modeling Approach

The modeling analyses conducted in this study involved the following approach:

- Estimation of flood spillage capacity at the New Polishing Pond based on Probable Maximum Flood (PMF) estimate, New Polishing Pond volume, configurations of New Polishing Pond and Dykes, and spillway capacity;
- Estimation of FLDWAV model input parameters such as dyke breach parameters, creek/river channel and flood plain dimensions, creek/river channel and floodplain roughness's, and hydraulic boundary conditions;
- A modeling analysis to predict the dyke breach flood hydrographs at the dyke sites and downstream routing of the dyke breach floods including flood peak levels, flood peak discharges, flood water depths and times to flood peak levels; and,
- A sensitivity analysis of the key model parameter values of the dyke breach model to define the degrees of uncertainty associated with the modeling results and to evaluate the effects of uncertainty associated with the accuracy of the dyke breach flood inundation mapping.

3.2 Flood Spillage Capacity at the New Polishing Pond

3.2.1 PMP and PMF Estimates

A flood hydrology study for the New Polishing Pond was carried out in this study. This section summarizes the methods and result of the estimation of the Probable Maximum Flood (PMF) inflow into the Polishing Pond.

HEC-HMS (Hydrologic Engineering Center – Hydrologic Modeling System), an industry-standard modeling package, was used for estimating the PMF. It was assumed that the PMF will flow into the New Polishing Pond from the surrounding sub-watershed drainage areas during mining operation.

Probable Maximum Precipitation (PMP), land types and drainage areas were used as the input data in HEC-HMS for the estimation of PMF. The 24-hour PMP event with a total rainfall depth of 375 mm was derived based on Golder's analysis of Val d'Or data and input from Bill Hogg, formerly of Environment Canada. The PMP was assumed to have a distribution presented in Figure A-1 in Appendix A, with a peak intensity of approximately 59 mm/hr occurring 8 hours after the onset of the storm. The distribution was derived from Chow et al. (1988).

For modeling purposes, two land types were assumed within the New Polishing Pond watershed:

- Tailings, which was assumed to be comparable to a Newly Graded Area; and,
- Natural Land (Muskeg), which was assumed to be comparable to a Forestland-Brush, with poor hydrologic condition.

Both land types were estimated to be comparable to “SCS Soil Group type C”, and therefore having runoff characteristics similar to those of a clay loam soil.

Other key assumptions of the model were as follows:

- The SCS Curve Number loss method and SCS Unit Hydrograph transform method were selected for estimating the direct runoff from the rainstorm;
- No base flow simulation was included because the contribution of groundwater to the total PMF was considered to be negligible;
- The flow from sub-watershed 10F to the Polishing Pond through the South Ditch was limited to 6 m^3/s with the rest of the flow assumed to be diverted south toward Lake Fournière by a high flow diversion (“kick out”) structure; and,
- The HEC-HMS model was run at 5 minutes time step.

A sensitivity analysis was performed to determine if the predicted runoff was strongly influenced by the assumptions of land type and hydrologic condition (or SCS curve number [CN]). The predicted peak PMF flow and volume were found to be relatively consistent over the practical range of potential CN values.

The hourly total PMF inflow hydrograph to the New Polishing Pond is presented in Figure A-2 in Appendix A. The PMF peak flow of 5 minute duration was estimated to be 143 m^3/s, and the PMF peak flow of one hour duration was estimated to be 117 m^3/s.

3.2.2 Level – Pool Routing Analysis

A level-pool routing analysis was carried out for the New Polishing Pond to determine if the pond outflow facility would be capable of passing the PMF without overtopping both Dykes EM-A and EM-B. The analysis provided a basis to determine which hydrologic condition and dyke failure mode (i.e. piping failure or overtopping failure) would represent the worst case scenario for the dyke breach modeling.

The New Polishing Pond flood routing analysis was conducted by assuming that the flood outflows would be conveyed by the spillway.

The level-pool routing analysis was conducted using an in-house model developed by Golder Associates Ltd.

The results of the pond flood routing analysis indicated the following:

- The PMF would overtop the Dykes EM-A and EM-B; and,
- Under a worst-case operational scenario, the pond peak water level during the PMF event was predicted to be 326.6 m, which was 0.1 m above the top of the dykes assuming that the dykes would not be breached during the PMF event.

Figure A-3 in Appendix A presents the results of the routing analysis for the PMF event.

3.3 Model Input Parameters

3.3.1 Dyke Breach Parameters

Dyke breach parameters are defined by the shape and size of a breach and the time required for its development (also called time to failure).

There is no rigorous, theoretical basis available for determining the size, shape or rate of formation of a breach in a water retaining dyke. The U.S. Federal Energy Regulatory Commission (FERC, 1994) provides a recommended range of potential dam breach parameters. The dam breach parameter values were estimated based on the recommendations made by FERC and the empirical formulations provided by Fread (2001).

Table 1 below lists the estimated dyke breach parameter values for the Dykes EM-A and EM-B. The rationale for selecting these dyke breach parameter values is provided below.

Table 1: Selected Dyke Breach Parameter Values

Dyke Breach Parameters	Dyke EM-A		Dyke EM-B	
	Overtopping Failure	Piping Failure	Overtopping Failure	Piping Failure
Average Width of Breach (BR) (m)	67.5	40.5	47.5	28.5
Time to Failure (TFH) (hour)	0.5	1.0	0.5	1.0
Side Slope of Breach (S:1)	1:1	1:1	1:1	1:1
Elevation of Water when Dyke Failure Commences (HFDD) (m)	326.6	325.0	326.6	325.0

Average Width of Breach (BR)

The average width of breach (BR) is the most important dyke breach model parameter. Fread (2001) suggested the following empirical formula for estimating BR parameter:

$$BR = 9.5\, k_0\, (V_r\, H)^{0.25} \qquad (1)$$

where; BR – average width of breach (ft)

k_0 – a coefficient (=1.0 for overtopping failure, and =0.7 for piping failure)

V_r – water volume retained within the reservoir/pond (acre-ft)

H – height of water over the breach bottom (ft)

For the overtopping failure mode, the formula suggests that BR ≅ 5.0*HD for the Dyke EM-A and BR ≅ 6.5*HD for the Dyke EM-B. For the piping failure mode, the formula suggests that BR ≅ 3.2*HD for the Dyke EM-A and BR ≅ 4.2*HD for the Dyke EM-B.

According to the FERC (1994) guidelines, FERC recommends that the lower and upper bound for the BR value should be within one to five times the maximum height of the dykes (HD) (i.e. 1*HD ≤ BR ≤ 5*HD) for earth fill dams.

Following the FERC (1994) guidelines, a BR value equal to be five times the HD value was selected for the Dykes EM-A and EM-B for the overtopping failure mode. A BR value equal to three times the HD value was considered sufficiently conservative for the Dykes EM-A and EM-B for the piping failure mode.

Time to Failure (TFH)

Fread (2001) suggested the following empirical formula for estimating TFH parameter:

$$TFH = 0.3\ V_r^{0.53}/H^{0.9} \qquad (2)$$

Fread's formula gave TFH = 1.0 hour for overtopping failure and TFH = 0.9 hour for piping failure for the Dyke EM-A. For the Dyke EM-B, Fread's formula gave TFH = 1.4 hour for overtopping failure and TFH = 1.2 hour for piping failure.

Time to failure parameter was conservatively selected to be 1.0 hour for piping failure and 0.5 hour for overtopping failure for both Dykes EM-A and EM-B, respectively. These TFH values fall in the range recommended for earth fill dams by FERC (i.e. 0.1 hour ≤ TFH ≤ 1.0 hour).

The predicted downstream flood peak levels are expected to be relatively insensitive to the TFH parameter.

Side Slope of Breach (S)

The horizontal component of the side slope of breach (S) was selected to be one for both failure modes, which was equal to the upper bound of the range recommended for earth fill dams by FERC (i.e. 1/4 ≤ S ≤ 1). The predicted downstream flood peak levels are not expected to be sensitive to this parameter. Therefore, a conservative estimate for S was used in the modeling analysis.

The breach was assumed to commence 0.1 m above the top of the dykes (326.6 m) for the overtopping failure mode. For a typical fair weather piping failure mode, it was assumed that the failure would commence at the operating level of 325.0 m.

3.3.2 Representation of the Creeks, Rivers and Floodplains

The floodways of Dykes EM-A and EM-B were divided into their respective reaches for modeling the dyke breach flood routing in the creek/river channels and flood plains. The study reach length for Dyke EM-A beach flood routing is about 16 km, from the Dyke EM-A to Thompson River at Dubuisson. The main structures along the potential dyke breach floodway include a local road crossing on the Ruisseau Raymond reach and three local road crossings on the Piché River reach. The populated areas along the dyke breach floodway include the Northern Star Mine along route 117 and Dubuisson.

The floodway for a potential Dyke EM-B failure flood is along an Unnamed Creek between Dyke EM-B and Lake Fourniere, with a study reach length of about 1.8 km. The main structures along the potential dyke breach floodway are one snowmobile trail and a local road crossing along the Unnamed Creek. The populated area along the floodway is adjacent to the Rang 7 Road.

The flow conveyance in both the creek/river channels and floodplains was modeled. Osisko supplied the base map that included the 10-m contour elevation, and a number of spot elevation survey points, including elevation surveys for individual houses, within the study area. The available topographic information was limited and did not allow for a very accurate representation of the channel cross-sections and floodplains. Therefore, simplified and uniform cross-sections for the creek or river channels were estimated. Moreover, approximate station and elevation data for the channels and floodplains were obtained from 1-m contour that was generated using the available topographic information. However, it should be noted that since 1m contours are an interpolation obtained from the 10m contours and some spot checks were made on the ground, this available topographical information should be treated with some caution. Further mapping would be required to improve the accuracy of the model.

The channel dimensions (i.e. the depth, bottom width and top width) for the Unnamed Creek along the Dyke EM-B floodway was roughly estimated to be 1 m channel depth, 6 m channel bottom width and 8 m channel top width. For the Dyke EM-A floodway along the Ruisseau Raymond Creek and Piché River, the channel dimensions were roughly estimated to be 1.5 m channel depth, 8 m channel bottom width and 10 m channel top width. The channel sinuosity was roughly estimated to be 1.1 for all the channels.

The channels bed and bank roughness (i.e. Manning's roughness coefficients) were estimated to be 0.04 and 0.10 for the main channels and floodplains, respectively. These coefficients were estimated in consideration of the channel bed/bank materials and meandering conditions, and the floodplain vegetative cover based on the available survey information and aerial photographs of the study reaches.

3.3.3 Initial Hydraulic Conditions

Initial hydraulic conditions for routing the dyke breach floods are not expected to have significant effects on the resulting floods along the floodways of the Dykes EM-A and EM-B, particularly for the resulting flood peak levels. However, a reasonable approximation of the initial hydraulic conditions was required to initiate the numerical computation and to provide a reasonable representation of the likely hydraulic conditions to be expected during the potential dyke breaches. Presented below are the considerations, assumptions and approximations made in the study for specifying the initial hydraulic conditions.

3.3.3.1 Initial Conditions for Modeling the Overtopping Failure Flood

The PMF is the worst-case hydrologic event that could trigger an overtopping dyke breach flood from either Dyke EM-A or Dyke EM-B. It would be overly conservative to assume that a similar PMF event would occur in the entire watershed within the study area during an overtopping failure event at the Dykes EM-A and EM-B. It was considered reasonable to assume that a very wet hydrologic condition would occur in the Unnamed Creek, Ruisseau Raymond Creek and Piché River watersheds. For modeling the dyke overtopping failure flood, it was assumed that the wet hydrologic condition would create a bankfull or slightly over bankfull flow condition in the study reaches.

3.3.3.2 Initial Conditions for Modeling the Piping Failure Flood

For piping failure modeling of the Dykes EM-A and EM-B, it was reasonable to assume that a typical fair weather hydrologic condition would occur along the study reach of the Unnamed Creek, Ruisseau Raymond Creek and Piché River. It was assumed that this fair weather hydrologic condition would create initial flows approximately two-thirds (2/3) of channel bankfull depths in the study reach.

3.3.4 Boundary Conditions for the Model

Accurate survey data such as measured discharges and water surface elevations are required to setup the model boundary conditions. However, as detail survey data were not available, a looped rating curve generated by the model was conservatively assumed as the downstream boundary condition. The looped rating curve for the most downstream section of Dyke EM-A (at Thompson River) and Dyke EM-B (at Lake Fournière) are generated by the model based on Manning's equation where the friction slopes are computed based on the momentum equation. A discharge hydrograph was generated using the spillway characteristics and the hydrograph was used as the upstream boundary condition at the centerline of Dykes EM-A and EM-B.

3.4 Modeling Results

A total of four model runs were made to generate the modeling results for the dyke breach floods for Dykes EM-A and EM-B. Figures A-4 to A-11 in Appendix A graphically present the following main modeling results for the overtopping and piping failure floods.

- Predicted flood peak discharges;
- Predicted flood peak levels;
- Predicted flood peak flow depths; and,
- Predicted time to flood peak levels.

The modeling results are discussed in Sections 4.2 and 4.3 in conjunction with the flood inundation map to describe the predicted impacts and the areas affected by the dyke breach floods.

Tables A-1 and A-2 in Appendix A present the comparisons of the predicted flood arrival time, predicted time to flood peak levels, predicted flood peak levels, and predicted flood peak discharges for the piping and overtopping failure floods. These comparisons show the following:

Dyke EM-A

- The flood peak levels associated with the piping failure flood are, on average, approximately 0.6 m lower than those associated with the overtopping failure flood along the study reach;

- The expected flood peak discharges associated with the piping failure flood are, on average, 29% smaller than those associated with the overtopping failure flood; and,
- The flood peak levels associated with the piping failure flood would generally arrive at a downstream location later than the overtopping failure flood by up to 1.3 hours.

Dyke EM-B

- The flood peak levels associated with the piping failure flood are, on average, approximately 1.1 m lower than those associated with the overtopping failure flood;
- The expected flood peak discharges associated with the piping failure flood are, on average, 56% smaller than those associated with the overtopping failure flood; and,
- The flood peak levels associated with the piping failure flood would generally arrive at a downstream location later than the overtopping failure flood by up to 0.6 hours.

3.5 Sensitivity Analysis

3.5.1 Purpose and Scope

A number of factors affect the accuracy and uncertainty associated with modeling the dyke breach floods in this study. The main factors include the following:

- Uncertainty associated with the dyke breach modeling processes due to the empirical nature of dyke breach modeling formulation in the FLDWAV model and the uncertainty associated with selection of the breach model parameter values;
- Unavailability of any historical flood that had similar magnitude of the flood peak discharges as obtained in the dyke breach floods, and it is not feasible to conduct calibration of the hydraulic model; and,
- Unavailability of the detailed creek/river channel and floodplain survey information to allow a detailed representation of the channel and floodplain cross sections along the study reaches of the Unnamed Creek, Ruisseau Raymond Creek and Piché River.

A conservative approach has been adopted in the study to address the modeling uncertainty and to generate conservative modeling results. This conservative approach is characterized by the following:

- Use of conservative estimates of the dyke breach model parameter values, which fall within the conservative sides of the recommended range available in the literatures and guidelines; and,
- Use of conservative estimates of Manning's n values for the creek/river channels and floodplains for predicting the flood peak levels.

The sensitivity analysis evaluates the effects of the conservative approach used on the modeling results. The sensitivity analysis mainly evaluates the degree of uncertainty associated with the modeling results and flood inundation mapping.

The modeling sensitivity analysis conducted in the study involved three key model parameters, including the average width of breach, time to failure, and the Manning's n. The first two parameters are related to modeling dyke breaching, and the last parameter is for the flood flow routing.

The reservoir routing analysis presented in Section 3.2.2 indicated that the Dykes EM-A and EM-B in the New Polishing Pond would be overtopped during the PMF event. In this scenario, the PMF event was simulated as a worst-case inflow flood that would trigger the overtopping dyke breach flood. Therefore, only the overtopping failure mode was considered in the sensitivity analysis, since an overtopping failure flood would result in more severe downstream flooding than a piping failure flood.

The results of the sensitivity analysis are presented in the following sections.

3.5.2 Sensitivity to Average Breach Width

The predicted dyke breach floods are most sensitive to this model parameter, because it largely controls the resulting outflow hydrograph through a dyke breach. The model sensitivity analysis involved a number of model runs based on three assumed dyke breach widths (BR = 2*HD, BR = 3*HD and BR = 4*HD), in addition to the dyke breach width (BR = 5*HD) used to predict the flood peak levels. The results of these model runs, based on the best estimates of the Manning's n values, were compared to quantify the model sensitivity to the assumed average breach width.

Figures A-12 and A-13 in Appendix A graphically present the differences between the predicted flood peak levels for modeling the Dyke EM-A and EM-B breach flood due to overtopping.

Dyke EM-A

The results of the sensitivity analysis shown on Figure A-12 indicate the following:

- The likely levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming BR = 5*HD range from 0.0 to –0.7 m with an average value of –0.04 m, based on the differences in the predicted flood peak levels between BR = 4*HD and BR = 5*HD;
- The average levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming BR = 5*HD range from 0.0 to –1.4 m with an average value of –0.1 m, based on the differences in the predicted flood peak levels between BR = 3*HD and BR = 5*HD; and,
- The maximum levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming BR = 5*HD range from 0.0 to –2.4 m with an average value of –0.2 m, based on the differences in the predicted flood peak levels between BR = 2*HD and BR = 5*HD.

Dyke EM-B

The results of the sensitivity analysis shown on Figure A-13 indicate the following:

- The likely levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming BR = 5*HD range from 0.0 to –0.4 m with an average value of –0.3 m, based on the differences in the predicted flood peak levels between BR = 4*HD and BR = 5*HD;
- The average levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming BR = 5*HD range from 0.0 to –0.9 m with an average value of –0.6 m, based on the differences in the predicted flood peak levels between BR = 3*HD and BR = 5*HD; and,
- The maximum levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming BR = 5*HD range from 0.0 to –1.6 m with an average value of –1.0 m, based on the differences in the predicted flood peak levels between BR = 2*HD and BR = 5*HD.

3.5.3 Sensitivity to Time to Failure

This sensitivity analysis involved a number of model runs based on the assumed time to failure (TFH = 0.2 hour, TFH = 1.0 hour and TFH = 2.0 hour), in addition to the assumed time to failure (TFH = 0.5 hour) used to predict the flood peak levels. The results of these model runs, based on the best estimates of the Manning's n values, were compared to quantify the model sensitivity.

Figures A-14 and A-15 in Appendix A graphically present the differences between the predicted flood peak levels based on various time-to-failure assumptions for Dykes EM-A and EM-B.

Dyke EM-A

The results of the sensitivity analysis shown on Figure A-14 indicate the following:

- The likely levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming TFH = 0.5 hour range from 0.0 to 0.3 m with an average value of 0.01 m, based on the differences in the predicted flood peak levels between TFH = 0.2 hour and TFH=0.5 hour;
- The average levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming TFH = 0.5 hour range from 0.0 to -0.6 m with an average value of -0.03 m, based on the differences in the predicted flood peak levels between TFH = 1.0 hour and TFH = 0.5 hour; and,
- The average levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming TFH = 0.5 hour range from 0.0 to -1.8 m with an average value of -0.09 m, based on the differences in the predicted flood peak levels between TFH = 2.0 hour and TFH = 0.5 hour.

Dyke EM-B

The results of the sensitivity analysis shown on Figure A-15 indicate the following:

- The likely levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming TFH = 0.5 hour range from 0.0 to 0.1 m with an average value of 0.05 m, based on the differences in the predicted flood peak levels between TFH = 0.2 hour and TFH=0.5 hour;

- The average levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming TFH = 0.5 hour range from 0.0 to -0.2 m with an average value of -0.1 m, based on the differences in the predicted flood peak levels between TFH = 1.0 hour and TFH = 0.5 hour; and,

- The average levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming TFH = 0.5 hour range from 0.0 to -0.6 m with an average value of -0.4 m, based on the differences in the predicted flood peak levels between TFH = 2.0 hour and TFH = 0.5 hour.

3.5.4 Sensitivity to Manning's n

This sensitivity analysis involved a number of model runs based on two assumed sets of Manning's n values for the creek/river channels and floodplains (one set corresponding to the maximum estimates of Manning's n and the other set the minimum estimates of Manning's n), in addition to a set of best estimates of the Manning's n for the model run. The results of these model runs, based on the estimates of assumed breach parameters, were compared to quantify the model sensitivity.

Figures A-16 and A-17 in Appendix A graphically present the differences between the predicted flood peak levels for Dykes EM-A and EM-B.

Dyke EM-A

The results of the sensitivity analysis shown on Figure A-16 indicate the following:

- The likely levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming an expected set of Manning's n for the model run, range from 0.0 to 0.5 m with an average value of 0.04 m, based on the differences in the predicted flood peak levels between the maximum and expected estimated sets of Manning's n; and,

- The maximum levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming an expected set of Manning's n for the model run, range from 0.0 to -0.6 m with an average value of –0.05 m, based on the differences in the predicted flood peak levels between the minimum and expected estimated sets of Manning's n.

Dyke EM-B

The results of the sensitivity analysis shown on Figure A-17 indicate the following:

- The likely levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming an expected set of Manning's n for the model run, range from 0.0 to 0.3 m with an average value of 0.2 m, based on the differences in the predicted flood peak levels between the maximum and expected estimated sets of Manning's n; and,

- The maximum levels of conservatism or levels of uncertainty in the predicted flood peak levels by assuming an expected set of Manning's n for the model run, range from 0.0 to -0.4 m with an average value of –0.3 m, based on the differences in the predicted flood peak levels between the minimum and expected estimated sets of Manning's n.

4.0 FLOOD INUNDATION MAPPING AND AFFECTED AREAS

4.1 Preparation of Dyke Breach Flood Inundation Map

The one-meter contour map used for defining the model cross-sections and mapping the flood inundation extents was created in this study based on the following data:

- 10-m contour map; and,
- A number of spot elevation survey points, including the surveyed elevations of the houses and buildings in the study area.

Drawing 1 shows the flood risk extents on a base map with 10-m contours, roads and other mapping features. The flood inundation map was prepared at a scale of 1:50,000. The flood inundation map along the floodways of Dykes EM-A and EM-B includes the following information:

- Flood inundation extents were delineated based on the 1-m contour elevations and predicted dyke breach flood peak levels at different cross-sections. Mapping of the flood peak levels at locations between the cross sections was determined based on a linear interpolation of the computed flood peak levels at the adjacent cross sections;
- Locations and labels of the Polishing Pond, Dykes, cross sections and road crossings used in the FLDWAV model;
- Locations and labels of houses and buildings along Rang 7, near Lake Fournière, Northern Star Mine, Dubuisson, Thompson River; and,
- Four tables showing the key flooding information at the cross-section and house locations along the floodways of Dykes EM-A and EM-B. The flooding information includes the predicted flood arrival time, time to flood peak level, flood peak level, flood peak discharge, and maximum flood depth for potential piping and overtopping failure floods.

4.2 Main Structures and Areas Affected by Dyke EM-A Breach Flood

4.2.1 Dyke EM-A Site

In the event of an overtopping failure of Dyke EM-A, the flood peak level immediately downstream of the dyke (i.e. at the toe) was predicted to be 321.1 m or approximately 10.2 m above the valley bottom. The time to flood peak level was estimated to be 0.5 hours after commencement of the dyke breach. The predicted flood peak discharge was approximately 4,500 m^3/s. In the event of a piping failure, the flood peak level immediately downstream of the dyke was predicted to be 319.0 m, which was 2.1 m lower than the flood peak level associated with the overtopping failure.

4.2.2 Snowmobile Trail – 1.8 km Downstream of Dyke EM-A

This snowmobile trail crossing at the Ruisseau Raymond Creek, located approximately 1.8 km downstream of Dyke EM-A, would be overtopped and damaged by the dyke breach flood. In the event of overtopping failure, the flood peak level at the road crossing was predicted to be 311.9 m and the corresponding peak flow depth was predicted to be 9.9 m. In this case, the flood would arrive approximately 0.3 hours after the commencement of the dyke breach and the time to flood peak level was estimated to be 0.8 hours. The flood peak discharge was predicted to be approximately 3,250 m^3/s. In the event of a piping failure, the flood peak level at the road crossing was predicted to be 309.9 m, which was 2.0 m lower than the flood peak level associated with the overtopping failure.

4.2.3 First Local Road Crossing – 5.6 km Downstream of Dyke EM-A

This local road crossing at the Piché River, located approximately 5.6 km downstream of Dyke EM-A, would be overtopped and damaged by the dyke breach flood. In the event of overtopping failure, the flood peak level at the road crossing was predicted to be 302.1 m and the corresponding peak flow depth was predicted to be 5.6 m. In this case, the flood would arrive approximately 1.3 hours after the commencement of the dyke breach and the time to flood peak level was estimated to be 2.1 hours. The flood peak discharge was estimated to be approximately 1,030 m^3/s. In the event of a piping failure, the flood peak level at the road crossing was predicted to be 301.6 m, which was 0.5 m lower than the flood peak level associated with the overtopping failure.

In the event of failure of Dyke EM-A, some of the residential houses or buildings near the Northern Star Mine along route 117 between cross-section A7 and A8 would likely be flooded as shown in Drawing 1 and it should be noted that Drawing 1 shows only the select house locations that were recently surveyed by Osisko.

4.2.4 Second Local Road Crossing – 7.8 km Downstream of the Dyke EM-A

This road crossing (i.e. Clairvue street) at the Piché River, located approximately 7.8 km downstream of Dyke EM-A, might be overtopped by the dyke breach flood. In the event of overtopping failure, the flood peak level at the road crossing was predicted to be 299.8 m and the corresponding peak flow depth was predicted to be

4.8 m. In this case, the flood would arrive approximately 1.8 hours after the commencement of the dyke breach and the time to flood peak level was estimated to be 3.6 hours. The flood peak discharge was estimated to be approximately 350 m^3/s. In the event of a piping failure, the flood peak level at the road crossing was predicted to be 299.4 m, which was 0.4 m lower than the flood peak level associated with the overtopping failure.

4.2.5 Third Local Road Crossing – 14.1 km Downstream of the Dyke EM-A

This road crossing (i.e. Des Explorateurs road) at the Piché River, located approximately 14.1 km downstream of Dyke EM-A, might be overtopped by the dyke breach flood. In the event of overtopping failure, the flood peak level at the road crossing was predicted to be 297.9 m and the corresponding peak flow depth was predicted to be 4.9 m. In this case, the flood would arrive approximately 5.2 hours after the commencement of the dyke breach and the time to flood peak level was estimated to be 14 hours. The reason for this approximate 9 hours difference between the flood arrival and flood peak times is that the flood level would rise very slowly at this location. The flood peak discharge was estimated to be approximately 100 m^3/s. In the event of a piping failure, the flood peak level at the road crossing was predicted to be 297.8 m, which was 0.1 m lower than the flood peak level associated with the overtopping failure.

4.2.6 Flooding Condition near the Thompson River

The most downstream boundary is located approximately 16 km downstream of Dyke EM-A, just before the confluence of the Piché River with the Thompson River. In the event of overtopping failure, the flood peak level was predicted to be 297.5 m and the corresponding peak flow depth was predicted to be 5.1 m. The time to flood peak level for overtopping failure was 13.6 hours after commencement of the dyke breach. The flood peak discharge was estimated to be approximately 95 m^3/s. In the event of a piping failure, the flood peak level at this location was predicted to be 297.5 m, which is the same as the overtopping failure.

4.3 Main Structures and Areas Affected by Dyke EM-B Breach Flood

4.3.1 Dyke EM-B Site

In the event of an overtopping failure of Dyke EM-B, the flood peak level immediately downstream of the dyke was predicted to be 320.7 m or approximately 5.3 m above the valley bottom. In the event of overtopping failure, the time to flood peak level was estimated to be approximately 0.5 hour after commencement of the dyke breach. The flood peak discharge was estimated to be approximately 2,040 m^3/s. In the event of a piping failure, the flood peak level immediately downstream of the dyke was predicted to be 319.4 m, which was 1.4 m lower than the flood peak level associated with the overtopping failure.

4.3.2 Snowmobile Trail – 0.5 km Downstream of Dyke EM-B

The snowmobile trail at the Unnamed Creek between Dyke EM-B and Lake Fournière, located approximately 0.5 km downstream of Dyke EM-B would be damaged by the dyke breach flood. In the event of overtopping failure, the flood peak level at the trail was predicted to be 316.6 m and the corresponding peak flow depth was predicted to be 5.1 m. In this case, the flood would arrive approximately 0.1 hours after the commencement of the dyke breach and the time to flood peak level was estimated to be 0.6 hours. The flood peak discharge was estimated to be approximately 1,940 m^3/s. In the event of a piping failure, the flood peak level at the trail crossing was predicted to be 315.4 m, which was 1.2 m lower than the flood peak level associated with the overtopping failure.

4.3.3 Local Road Crossing – 1.2 km Downstream of Dyke EM-B

The local road crossing (i.e. Rang 7 Road) at the Unnamed Creek between Dyke EM-B and Lake Fournière, located approximately 1.2 km downstream of Dyke EM-B would be overtopped and damaged by the dyke breach flood. The flood peak level at the road crossing was predicted to be 311.5 m and the corresponding peak flow depth was predicted to be 4.0 m. In this case, the flood would arrive approximately 0.3 hours after the commencement of the dyke breach and the time to flood peak level was estimated to be 0.8 hours. The flood peak discharge was estimated to be approximately 1,790 m^3/s. In the event of a piping failure, the flood peak level at the road crossing was predicted to be 310.7 m, which is 0.8 m lower than the flood peak level associated with the overtopping failure.

4.3.4 Flooding Condition near Lake Fournière

The most downstream boundary is located approximately 1.8 km downstream of Dyke EM-B, just upstream of the confluence with the Unnamed Creek with the Lake Fournière. At this location, the flood peak level was predicted to be 310.2 m and the corresponding peak flow depth was predicted to be 3.8 m. In this case, the flood would arrive approximately 0.5 hours after the commencement of the dyke breach and the time to flood peak level was estimated to be 1.0 hour. The flood peak discharge was estimated to be approximately 1,590 m^3/s. In the event of a piping failure, the flood peak level at the road crossing was predicted to be 309.8 m, which was 0.4 m lower than the flood peak level associated with the overtopping failure.

In the event of failure of Dyke EM-B, only one residential house or building which was surveyed and is located close to Lake Fournière, would be flooded as shown on Drawing 1.

5.0 CONCLUSIONS AND RECOMMENDATION

The results of the dyke breach flood inundation study of Dykes EM-A and EM-B of the New Polishing Pond support the following conclusions:

- The PMF event would overtop Dykes EM-A and EM-B of the Polishing Pond. Under the worst-case operational scenario, the peak reservoir water level during the PMF event was predicted to be 326.6 m, which is 0.1 m above the design top of dyke elevation of 326.5 m;

- A conservative approach has been adopted in this study to generate conservative modeling results for the dyke breach flood inundation mapping, in consideration of the uncertainty caused by the modeling of the dyke breach process, unavailability of any known historical floods for a reliable calibration of the dyke breach floods, and no detailed creek channel and floodplain survey information. This conservative approach is characterized by the selections of conservative breach model parameter values and conservative Manning's n values for the Unnamed Creek, Ruisseau Raymond Creek and Piché River channels and floodplains. This approach generates the modeling results on a conservative basis for preparing the dyke breach flood inundation map and for evaluating the potential downstream impacts of the dyke breach floods;

- The predicted flood peak levels associated with the overtopping dyke failure flood are more sensitive to the assumed average breach width, and less sensitive to the assumed time to failure and the estimated Manning's n values;

- In the event of piping and overtopping failure, all local road crossings within the Dykes EM-A and EM-B study reaches would likely be overtopped and damaged;

- For Dyke EM-A, the flood peak levels associated with the piping failure flood are approximately 0.6 m lower, on average, than those associated with the overtopping failure flood. The expected flood peak discharges associated with the piping failure flood are 29% smaller, on average, than those associated with the overtopping failure flood. The flood peak levels associated with the piping failure flood would generally arrive at a downstream location later than the overtopping failure flood by up to 1.3 hours;

- For Dyke EM-B, the flood peak levels associated with the piping failure flood are approximately 1.1 m lower, on average, than those associated with the overtopping failure flood. The expected flood peak discharges associated with the piping failure flood are 56% smaller, on average, than those associated with the overtopping failure flood. The flood peak levels associated with the piping failure flood would generally arrive at a downstream location later than the overtopping failure flood by up to 0.6 hours; and,

- In the event of Dyke EM-A failure, some of the residential houses or buildings near Northern Star Mine along route 117 would be flooded as shown on Drawing 1. In the event of failure of Dyke EM-B, only one residential house or building which was surveyed and is located close to Lake Fournière, would be flooded as shown on Drawing 1. All other houses within Dykes EM-A and EM-B floodways would not be flooded due to their higher ground elevations and based on the predicted flood peak levels. However, it should be noted that the survey was only conducted at select and representative house locations.

This dyke breach flood inundation study was conducted based on the available topographic data for the study reaches of the floodways. Efforts were spent in collecting spot elevation data in select locations. However, most of the topographic information used in setting up the model and preparing the flood inundation map was obtained from the 10-m contour map. This introduces some level of uncertainty in the modeling results and flood inundation mapping.

It is recommended that the client develop an improved topographic base map with one-meter contours, or a minimum elevation accuracy of ±0.5 m. The updated base map should be used for updating the dyke breach inundation modeling analysis and mapping to provide a reliable basis for preparing the EPP (Emergency Preparedness Plan) and ERP (Emergency Response Plan) in accordance with the 2007 CDA Dam Safety Guidelines.

Report Signature Page

This report is prepared and reviewed by the following study team members.

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APEGGA PERMIT TO PRACTICE 05122

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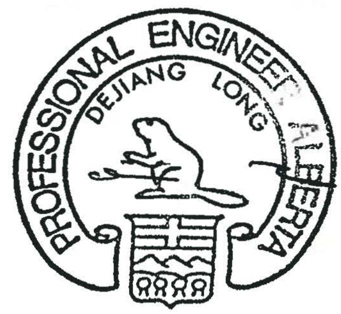

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6.0 THIRD PARTY DISCLAIMER

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7.0 REFERENCES

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APPENDIX A

Modeling Results

Figure A - 1: PMP Design Storm for Val d'Or – Quebec (Total 24 hour rainfall = 375 mm)

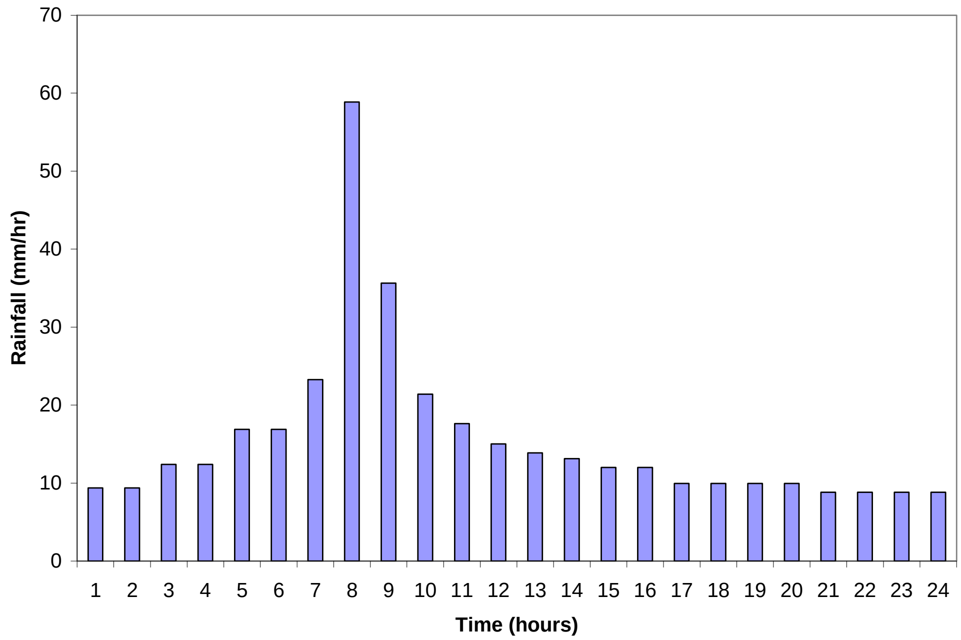


Figure A - 2: Hourly PMF Inflow Hydrograph into the Polishing Pond

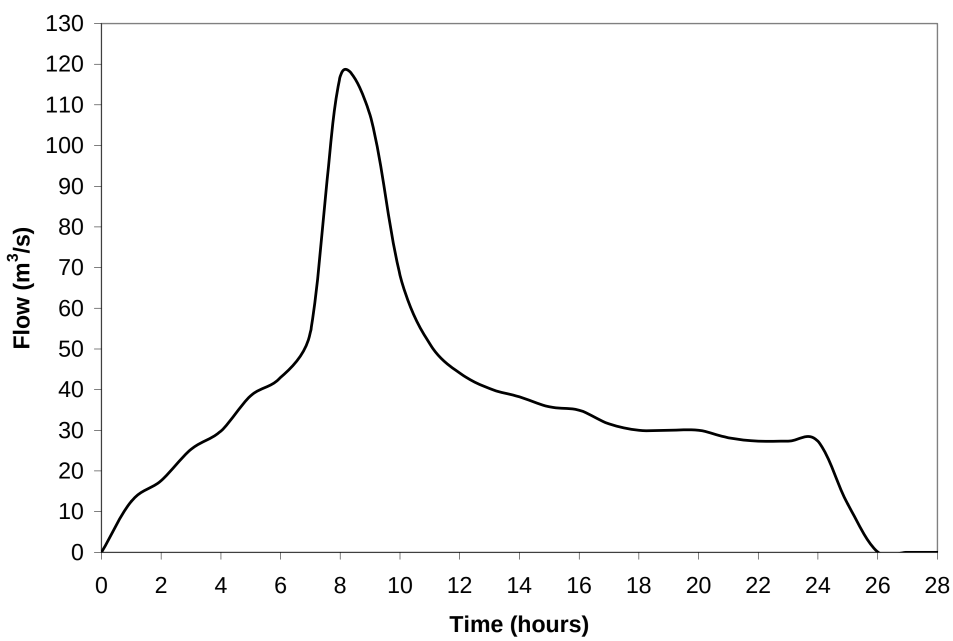


Figure A - 3: Results of Level-Pool Routing Analysis for the Polishing Pond

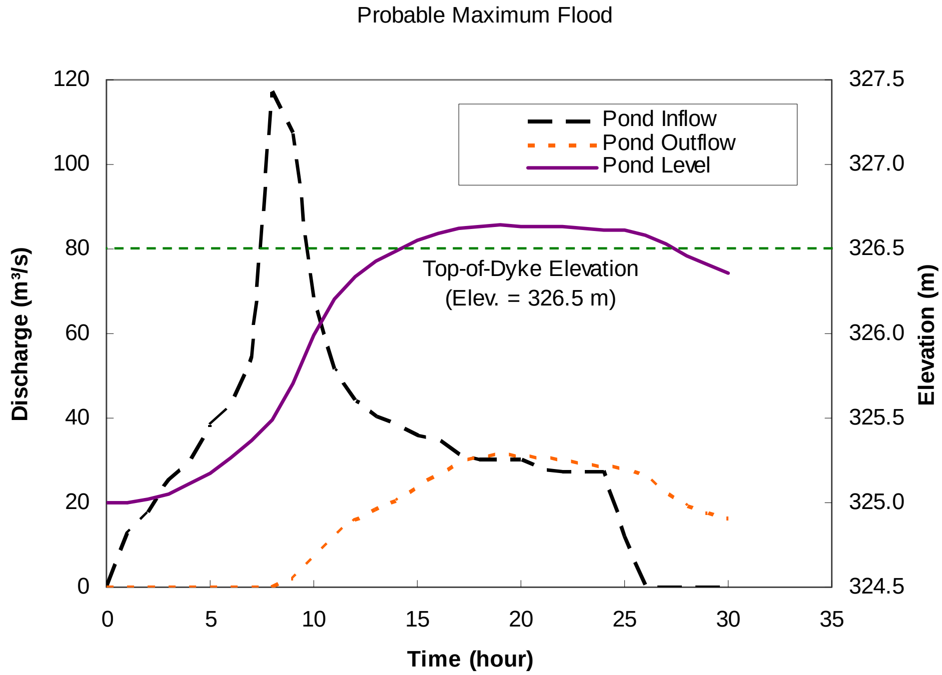


Figure A - 4: Predicted Flood Peak Discharges Associated with Dyke EM-A Breach Floods

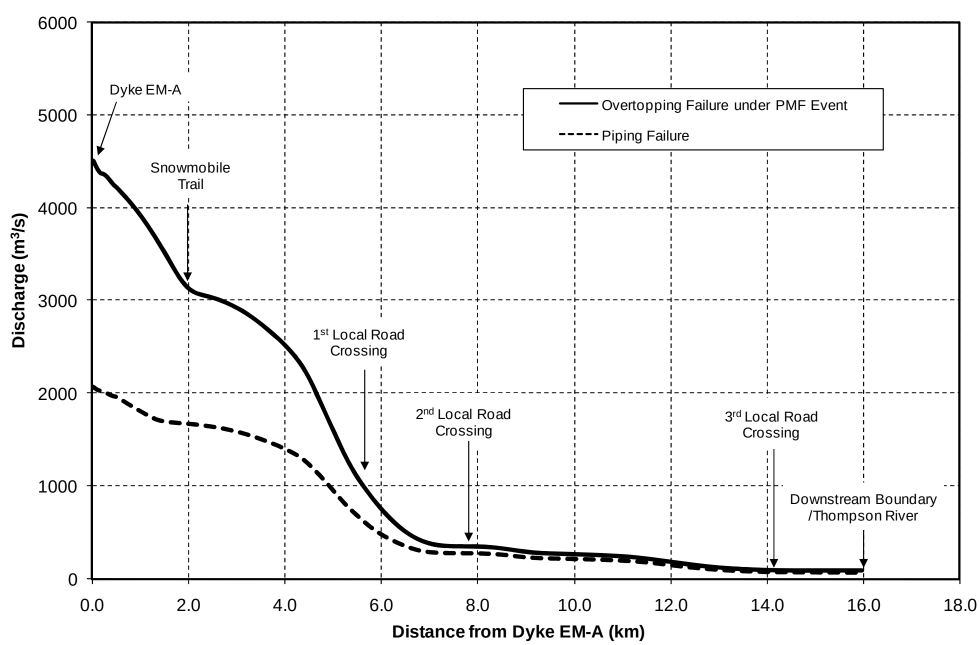


Figure A - 5: Predicted Flood Peak Levels Associated with Dyke EM-A Breach Floods

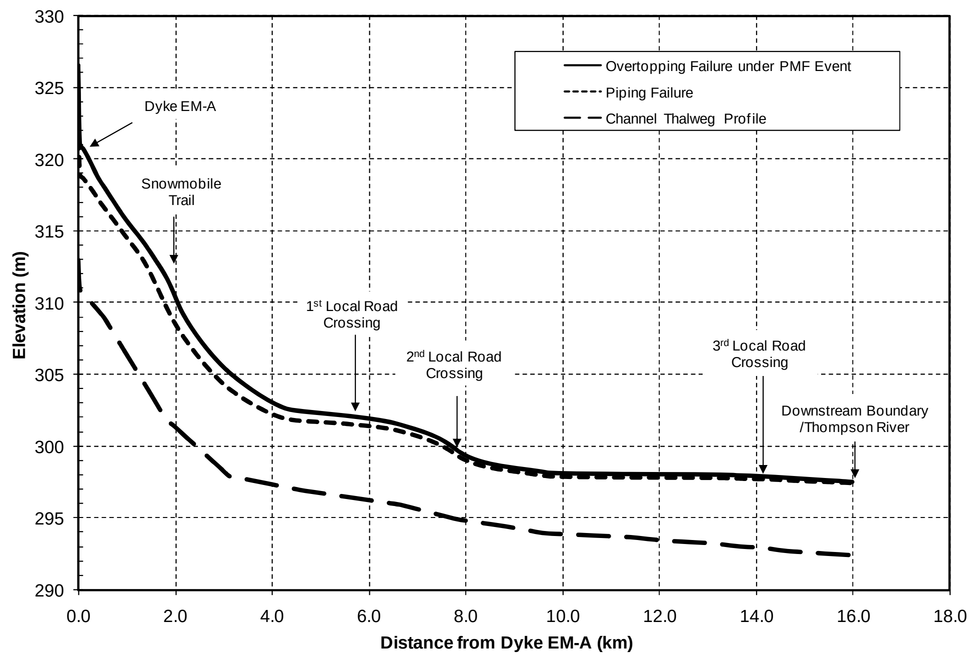


Figure A - 6: Predicted Flood Peak Flow Depths Associated with Dyke EM-A Breach Floods

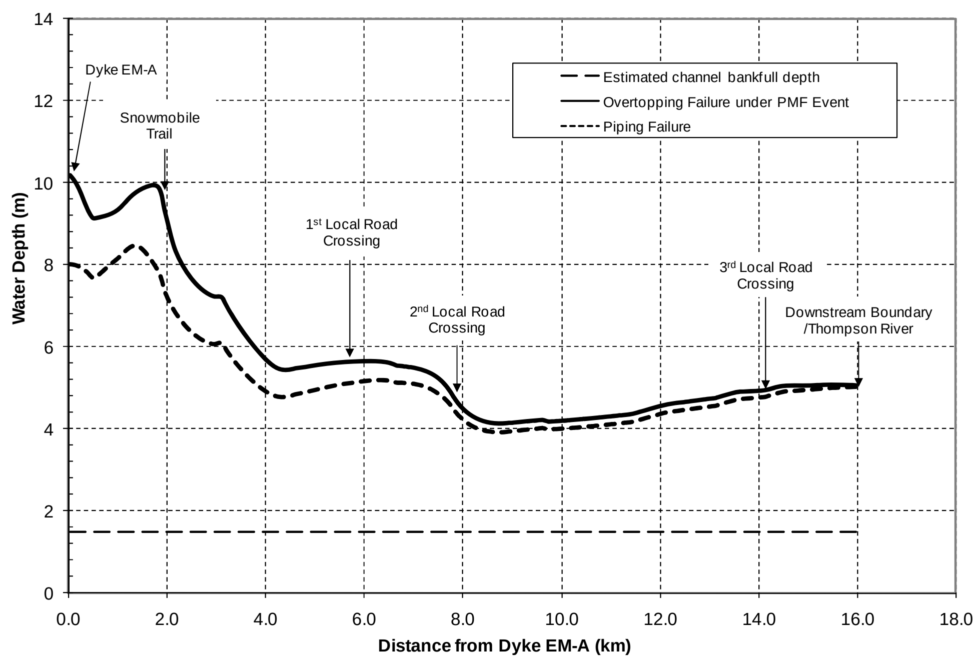


Figure A - 7: Predicted Time to Flood Peak Levels Associated with Dyke EM-A Breach Floods

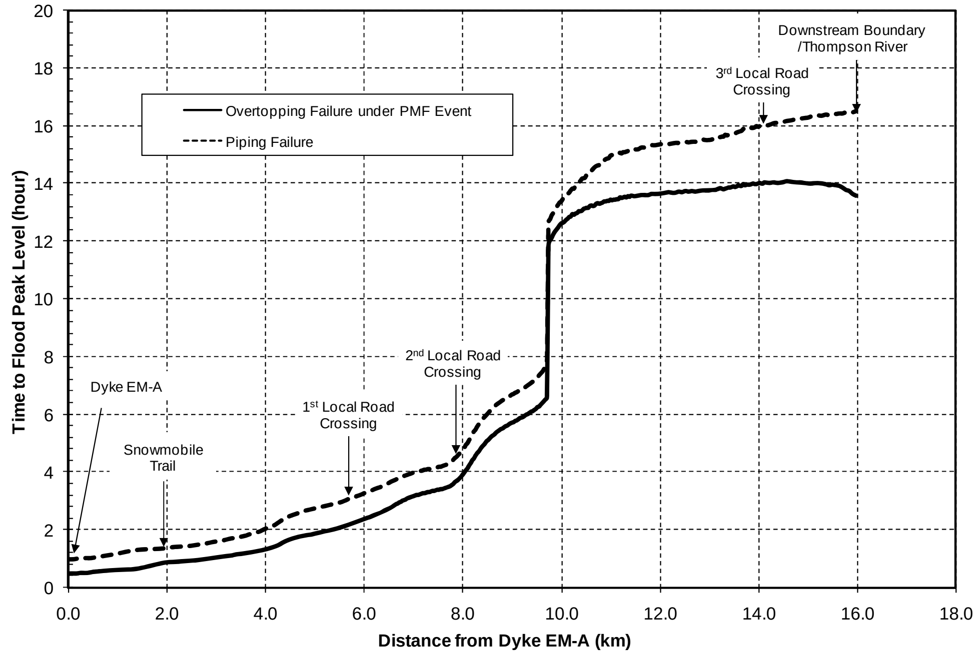


Figure A - 8: Predicted Flood Peak Discharges Associated with Dyke EM-B Breach Floods

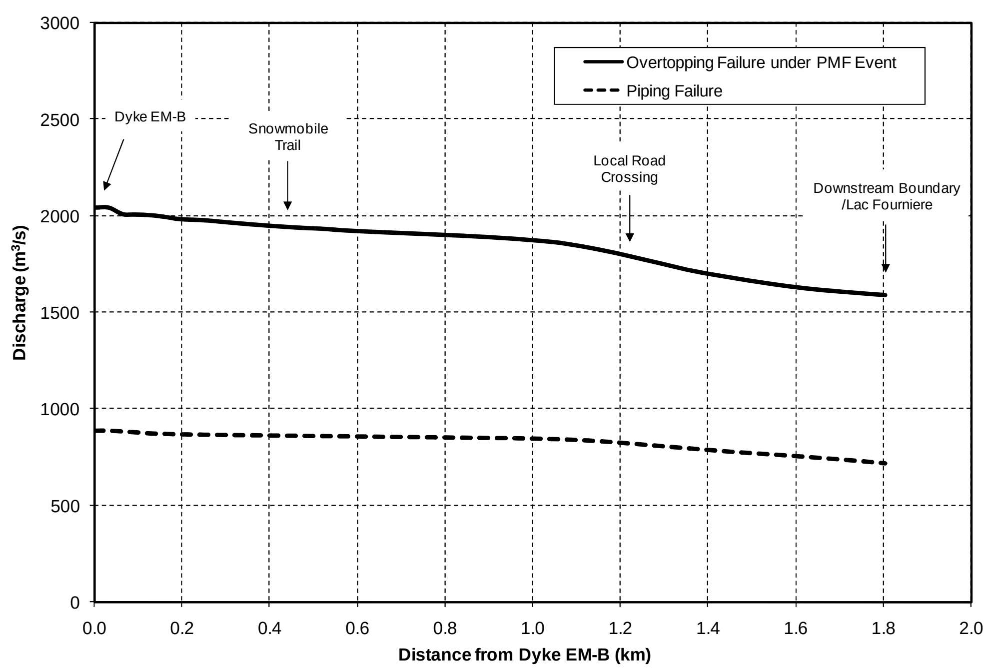


Figure A - 9: Predicted Flood Peak Levels Associated with Dyke EM-B Breach Floods

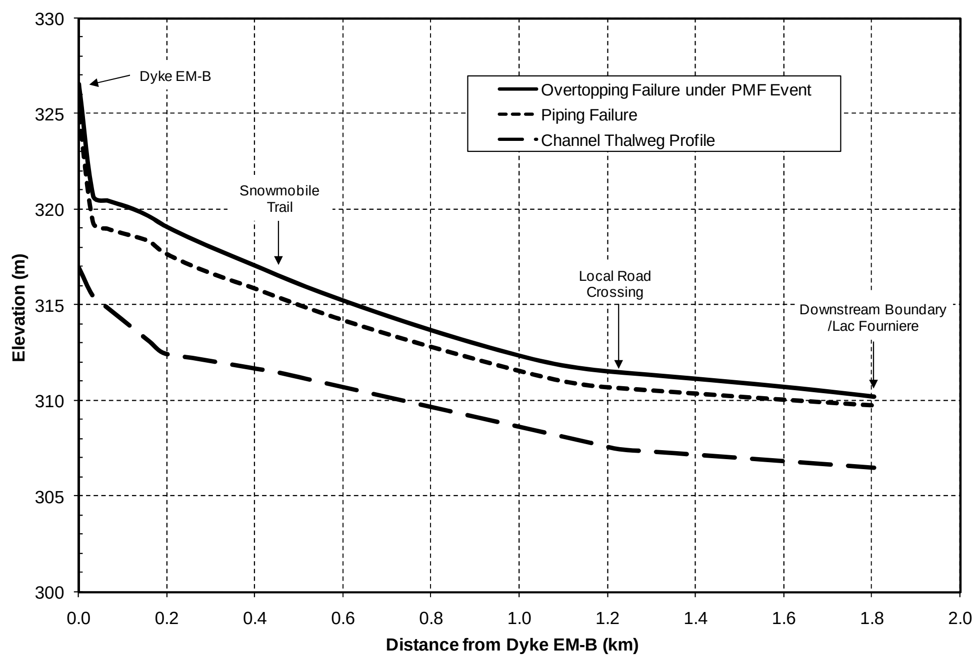


Figure A - 10: Predicted Flood Peak Flow Depths Associated with Dyke EM-B Breach Floods

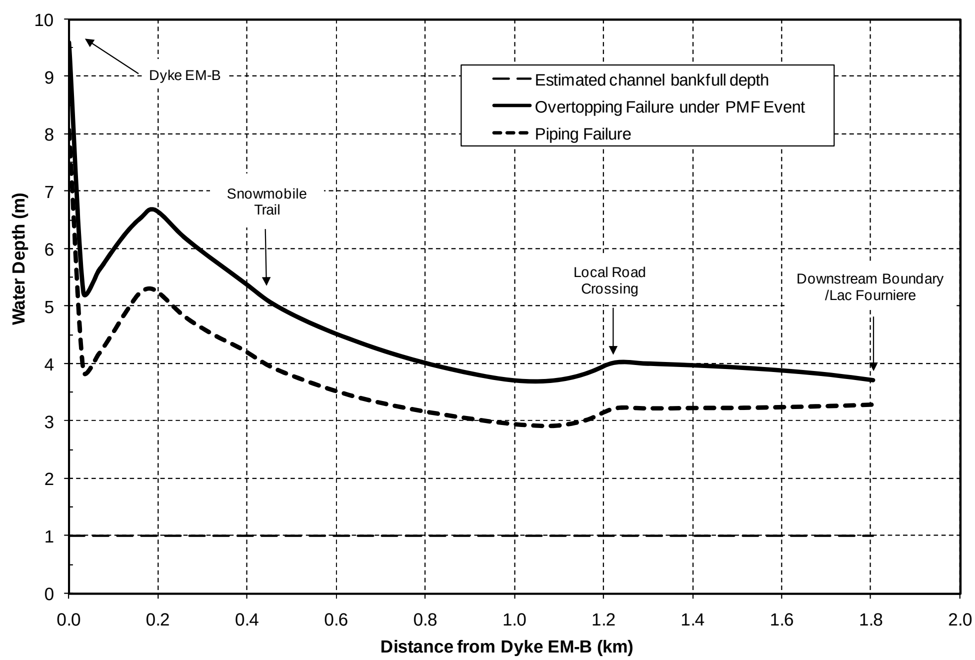


Figure A - 11: Predicted Time to Flood Peak Levels Associated with Dyke EM-B Breach Floods

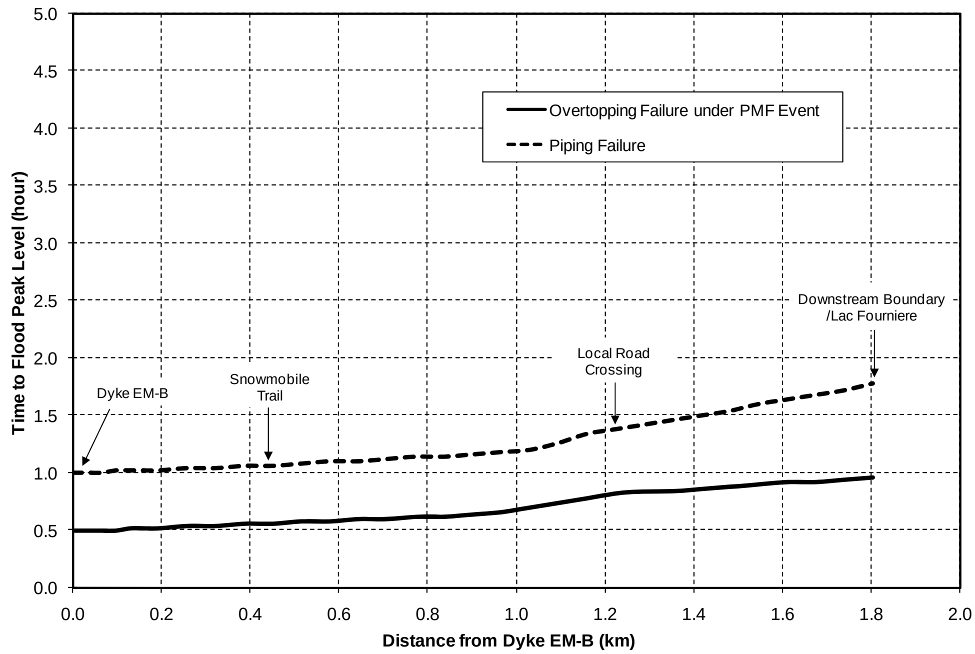


Figure A - 12: Sensitivity of Predicted Flood Peak Levels to the Assumed Dyke Breach Widths for Dyke EM-A

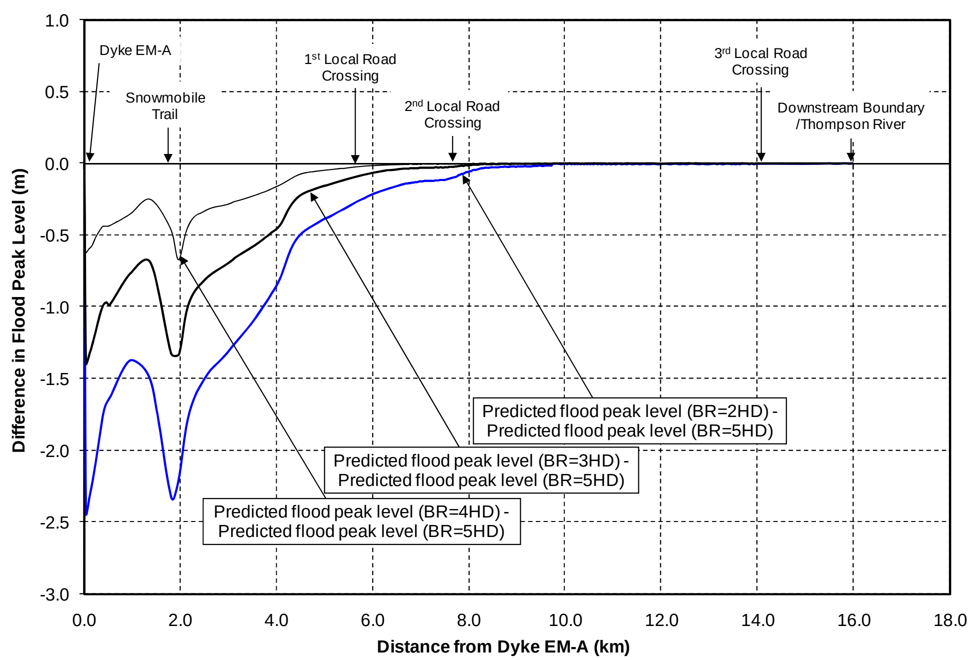


Figure A - 13: Sensitivity of Predicted Flood Peak Levels to the Assumed Dyke Breach Widths for Dyke EM-B

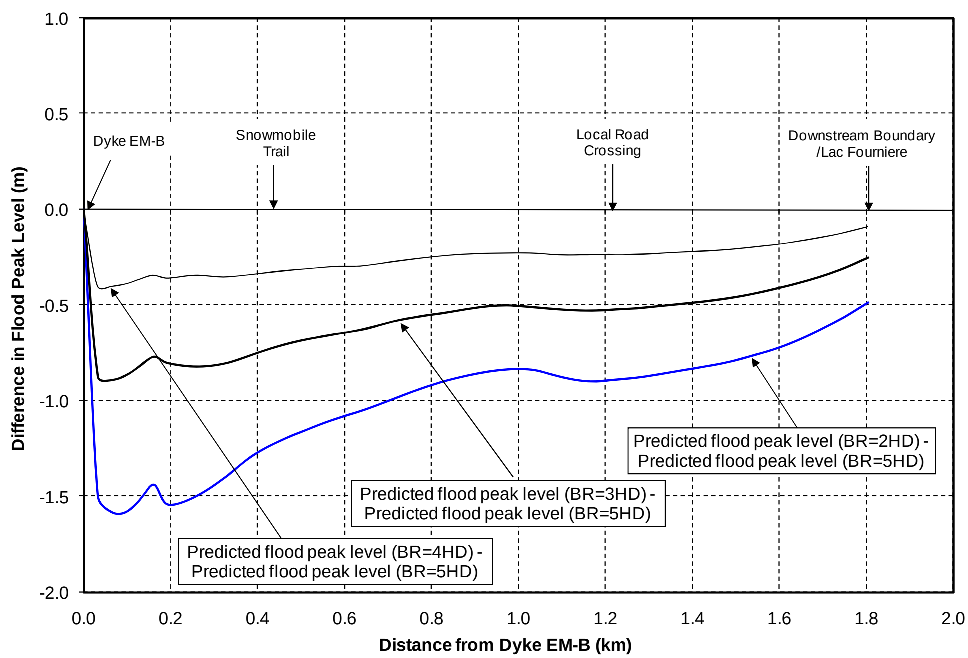


Figure A - 14: Sensitivity of Predicted Flood Peak Levels to the Assumed Times to Failure for Dyke EM-A

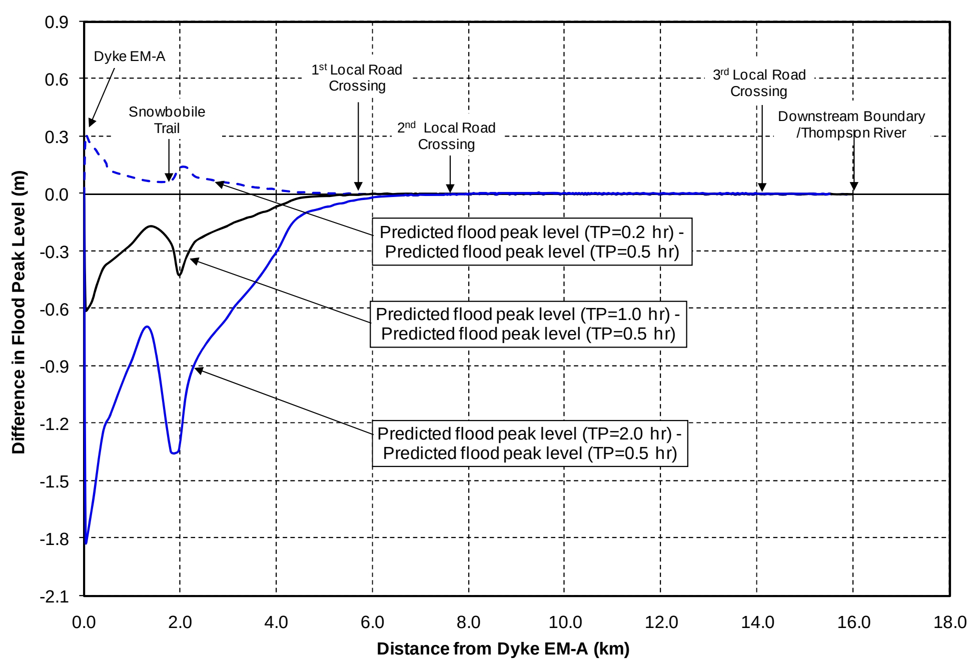


Figure A - 15: Sensitivity of Predicted Flood Peak Levels to the Assumed Times to Failure for Dyke EM-B

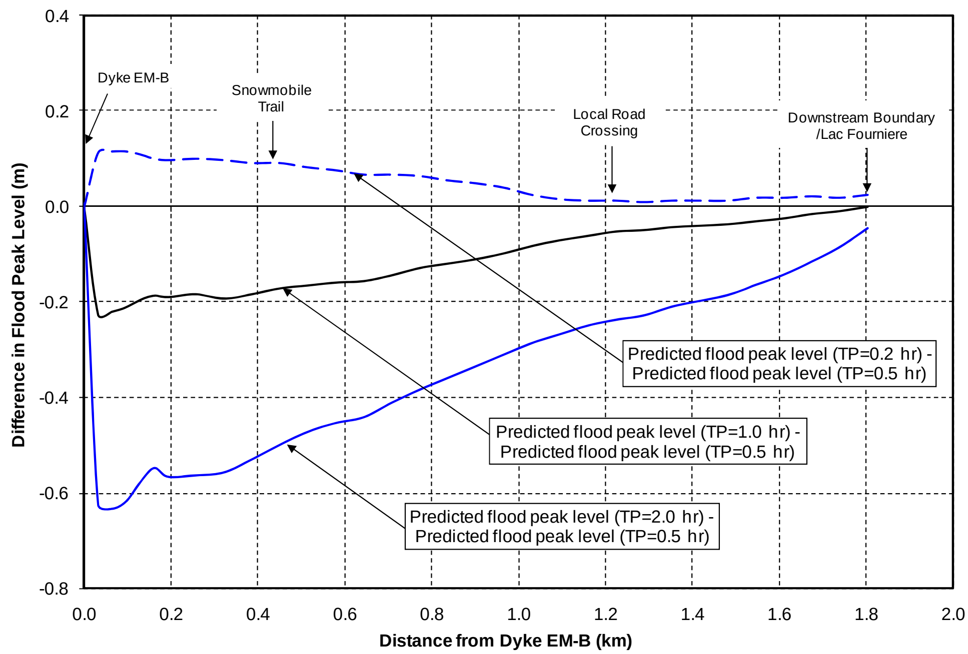


Figure A - 16: Sensitivity of Predicted Flood Peak Levels to Assumed Manning's n for Routing Dyke EM-A Breach Flood

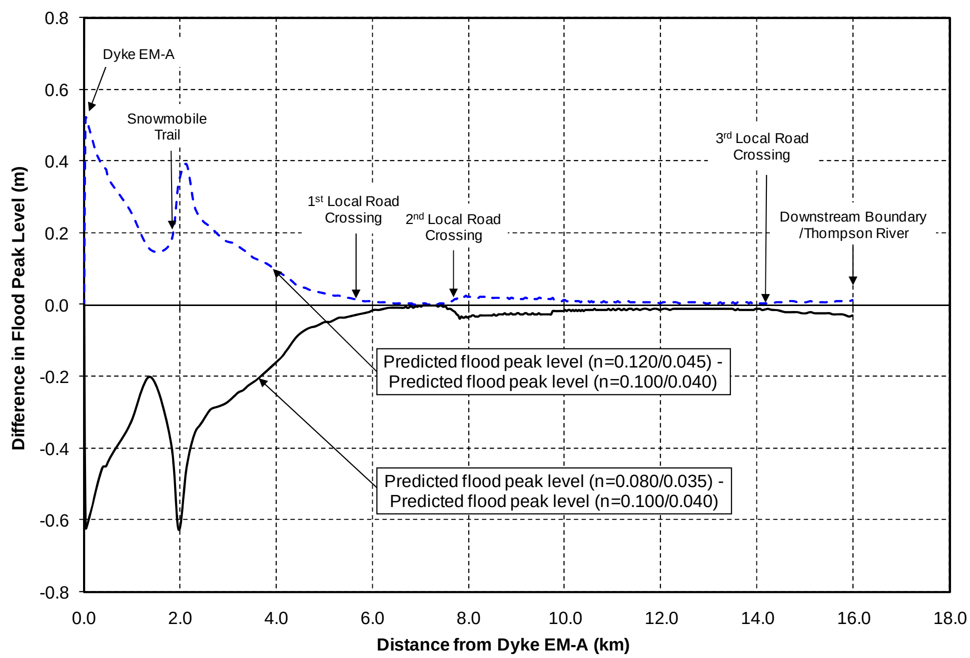


Figure A - 17: Sensitivity of Predicted Flood Peak Levels to Assumed Manning's n for Routing Dyke EM-B Breach Flood

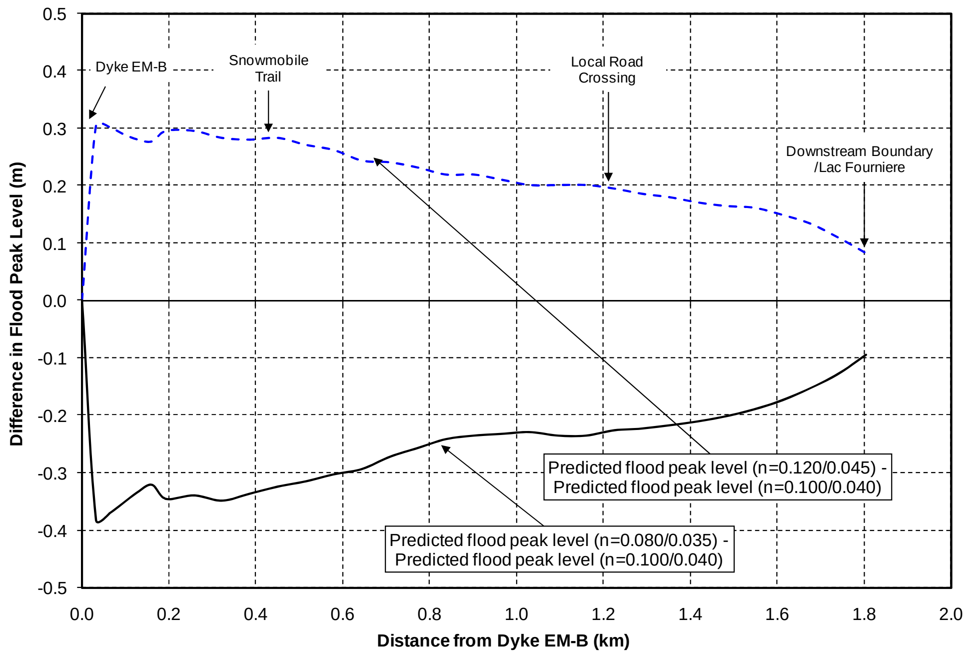


Table A - 1: Dyke EM-A Breach Modeling Results

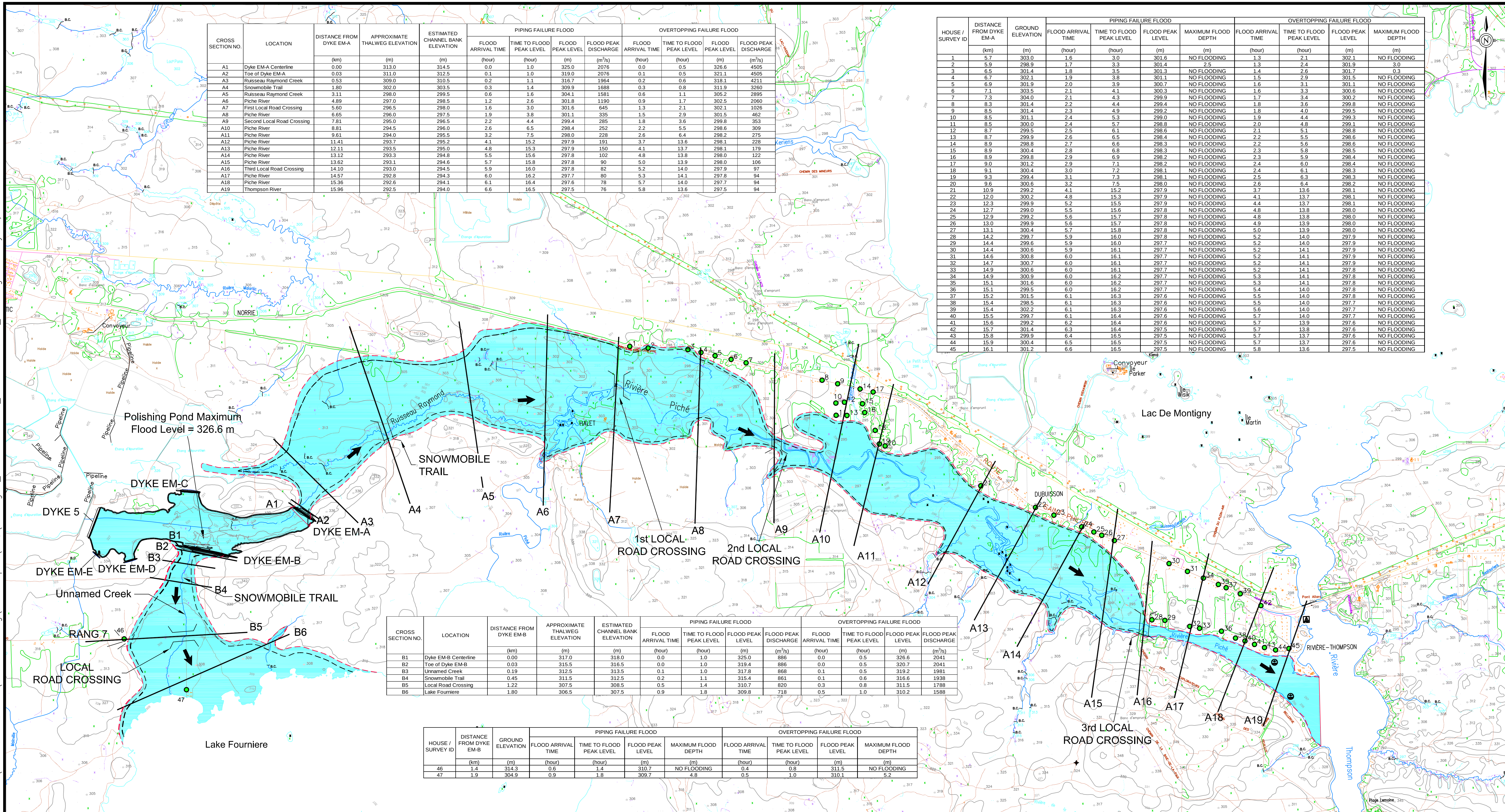
CROSS SECTION NO.	LOCATION	DISTANCE FROM DYKE EM-A	APPROXIMATE THALWEG ELEVATION	ESTIMATED CHNNEL BANK ELEVATION	PIPING FAILURE FLOOD				OVERTOPPING FAILURE FLOOD			
					FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	FLOOD PEAK DISCHARGE	FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	FLOOD PEAK DISCHARGE
		(km)	(m)	(m)	(hour)	(hour)	(m)	(m^3/s)	(hour)	(hour)	(m)	(m^3/s)
A1	Dyke EM-A Centreline	0.00	313.0	314.5	0.0	1.0	325.0	2076	0.0	0.5	326.6	4505
A2	Toe of Dyke EM-A	0.03	311.0	312.5	0.1	1.0	319.0	2076	0.1	0.5	321.1	4505
A3	Raymond Creek	0.53	309.0	310.5	0.2	1.1	316.7	1964	0.2	0.6	318.1	4211
A4	Snowmobile Trail	1.80	302.0	303.5	0.3	1.4	309.9	1688	0.3	0.8	311.9	3260
A5	Raymond Creek	3.11	298.0	299.5	0.6	1.6	304.1	1581	0.6	1.1	305.2	2895
A6	Piché River	4.89	297.0	298.5	1.2	2.6	301.8	1190	0.9	1.7	302.5	2060
A7	First Local Road Crossing	5.60	296.5	298.0	1.6	3.0	301.6	645	1.3	2.1	302.1	1026
A8	Piché River	6.65	296.0	297.5	1.9	3.8	301.1	335	1.5	2.9	301.5	462
A9	Second Local Road Crossing	7.81	295.0	296.5	2.2	4.4	299.4	285	1.8	3.6	299.8	353
A10	Piché River	8.81	294.5	296.0	2.6	6.5	298.4	252	2.2	5.5	298.6	309
A11	Piché River	9.61	294.0	295.5	3.2	7.5	298.0	228	2.6	6.4	298.2	275
A12	Piché River	11.41	293.7	295.2	4.1	15.2	297.9	191	3.7	13.6	298.1	228
A13	Piché River	12.11	293.5	295.0	4.8	15.3	297.9	150	4.1	13.7	298.1	179
A14	Piché River	13.12	293.3	294.8	5.5	15.6	297.8	102	4.8	13.8	298.0	122
A15	Piché River	13.62	293.1	294.6	5.7	15.8	297.8	90	5.0	13.9	298.0	106
A16	Third Local Road Crossing	14.10	293.0	294.5	5.9	16.0	297.8	82	5.2	14.0	297.9	97
A17	Piché River	14.57	292.8	294.3	6.0	16.2	297.7	80	5.3	14.1	297.8	94
A18	Piché River	15.36	292.6	294.1	6.1	16.4	297.6	78	5.7	14.0	297.7	94
A19	Thompson River	15.96	292.5	294.0	6.6	16.5	297.5	76	5.8	13.6	297.5	94

Table A - 2 Dyke EM-B Breach Modeling Results

CROSS SECTION NO.	LOCATION	DISTANCE FROM DYKE EM-B	APPROXIMATE THALWEG ELEVATION	ESTIMATED CHANNEL BANK ELEVATION	PIPING FAILURE FLOOD				OVERTOPPING FAILURE FLOOD			
					FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	FLOOD PEAK DISCHARGE	FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	FLOOD PEAK DISCHARGE
		(km)	(m)	(m)	(hour)	(hour)	(m)	(m^3/s)	(hour)	(hour)	(m)	(m^3/s)
B1	Dyke EM-B Centerline	0.00	317.0	318.0	0.0	1.0	325.0	886	0.0	0.5	326.6	2041
B2	Toe of Dyke EM-B	0.03	315.5	316.5	0.0	1.0	319.4	886	0.0	0.5	320.7	2041
B3	Unnamed Creek	0.19	312.5	313.5	0.1	1.0	317.8	868	0.1	0.5	319.2	1981
B4	Snowmobile Trail	0.45	311.5	312.5	0.2	1.1	315.4	861	0.1	0.6	316.6	1938
B5	Local Road Crossing	1.22	307.5	308.5	0.5	1.4	310.7	820	0.3	0.8	311.5	1788
B6	Lake Fournière	1.80	306.5	307.5	0.9	1.8	309.8	718	0.5	1.0	310.2	1588

APPENDIX B

Dyke Breach Flood Inundation Map



CROSS SECTION NO.	LOCATION	DISTANCE FROM DYKE EM-A	APPROXIMATE THALWEG ELEVATION	ESTIMATED CHANNEL BANK ELEVATION	PIPING FAILURE FLOOD				OVERTOPPING FAILURE FLOOD			
					FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	FLOOD PEAK DISCHARGE	FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	FLOOD PEAK DISCHARGE
		(km)	(m)	(m)	(hour)	(hour)	(m)	(m^3/s)	(hour)	(hour)	(m)	(m^3/s)
A1	Dyke EM-A Centerline	0.00	313.0	314.5	0.0	1.0	325.0	2076	0.0	0.5	326.6	4505
A2	Toe of Dyke EM-A	0.03	311.0	312.5	0.1	1.0	319.0	2076	0.1	0.5	321.1	4505
A3	Ruisseau Raymond Creek	0.53	309.0	310.5	0.2	1.1	316.7	1964	0.2	0.6	318.1	4211
A4	Snowmobile Trail	1.80	302.0	303.5	0.3	1.4	309.9	1688	0.3	0.8	311.9	3260
A5	Ruisseau Raymond Creek	3.11	298.0	299.5	0.6	1.6	304.1	1581	0.6	1.1	305.2	2895
A6	Piche River	4.89	297.0	298.5	1.2	2.6	301.8	1190	0.9	1.7	302.5	2060
A7	First Local Road Crossing	5.60	296.5	298.0	1.6	3.0	301.6	645	1.3	2.1	302.1	1026
A8	Piche River	6.65	296.0	297.5	1.9	3.8	301.1	335	1.5	2.9	301.5	462
A9	Second Local Road Crossing	7.81	295.0	296.5	2.2	4.4	299.4	285	1.8	3.6	299.8	353
A10	Piche River	8.81	294.5	296.0	2.6	6.5	298.4	252	2.2	5.5	298.6	309
A11	Piche River	9.61	294.0	295.5	3.2	7.5	298.0	228	2.6	6.4	298.2	275
A12	Piche River	11.41	293.7	295.2	4.1	15.2	297.9	191	3.7	13.6	298.1	228
A13	Piche River	12.11	293.5	295.0	4.8	15.3	297.9	150	4.1	13.7	298.1	179
A14	Piche River	13.12	293.3	294.8	5.5	15.6	297.8	102	4.8	13.8	298.0	122
A15	Piche River	13.62	293.1	294.6	5.7	15.8	297.8	90	5.0	13.9	298.0	106
A16	Third Local Road Crossing	14.10	293.0	294.5	5.9	16.0	297.8	82	5.2	14.0	297.9	97
A17	Piche River	14.57	292.8	294.3	6.0	16.2	297.7	80	5.3	14.1	297.8	94
A18	Piche River	15.36	292.6	294.1	6.1	16.4	297.6	78	5.7	14.0	297.7	94
A19	Thompson River	15.96	292.5	294.0	6.6	16.5	297.5	76	5.8	13.6	297.5	94

HOUSE / SURVEY ID	DISTANCE FROM DYKE EM-A	GROUND ELEVATION	PIPING FAILURE FLOOD				OVERTOPPING FAILURE FLOOD			
			FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	MAXIMUM FLOOD DEPTH	FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	MAXIMUM FLOOD DEPTH
	(km)	(m)	(hour)	(hour)	(m)	(m)	(hour)	(hour)	(m)	(m)
1	5.7	303.0	1.6	3.0	301.6	NO FLOODING	1.3	2.1	302.1	NO FLOODING
2	5.9	298.9	1.7	3.3	301.4	2.5	1.3	2.4	301.9	3.0
3	6.5	301.4	1.8	3.5	301.3	NO FLOODING	1.4	2.6	301.7	0.3
4	6.7	302.1	1.9	3.8	301.1	NO FLOODING	1.5	2.9	301.5	NO FLOODING
5	6.9	301.9	2.0	3.9	300.7	NO FLOODING	1.6	3.1	301.1	NO FLOODING
6	7.1	303.5	2.1	4.1	300.3	NO FLOODING	1.6	3.3	300.6	NO FLOODING
7	7.3	304.0	2.1	4.3	299.9	NO FLOODING	1.7	3.4	300.2	NO FLOODING
8	8.3	301.4	2.2	4.4	299.4	NO FLOODING	1.8	3.6	299.8	NO FLOODING
9	8.5	301.4	2.3	4.9	299.2	NO FLOODING	1.8	4.0	299.5	NO FLOODING
10	8.5	301.1	2.4	5.3	299.0	NO FLOODING	1.9	4.4	299.3	NO FLOODING
11	8.5	300.0	2.4	5.7	298.8	NO FLOODING	2.0	4.8	299.1	NO FLOODING
12	8.7	299.5	2.5	6.1	298.6	NO FLOODING	2.1	5.1	298.8	NO FLOODING
13	8.7	299.9	2.6	6.5	298.4	NO FLOODING	2.2	5.5	298.6	NO FLOODING
14	8.9	298.8	2.7	6.6	298.3	NO FLOODING	2.2	5.6	298.6	NO FLOODING
15	8.9	300.4	2.8	6.8	298.3	NO FLOODING	2.3	5.8	298.5	NO FLOODING
16	8.9	299.8	2.9	6.9	298.2	NO FLOODING	2.3	5.9	298.4	NO FLOODING
17	9.0	301.2	2.9	7.1	298.2	NO FLOODING	2.4	6.0	298.4	NO FLOODING
18	9.1	300.4	3.0	7.2	298.1	NO FLOODING	2.4	6.1	298.3	NO FLOODING
19	9.3	299.4	3.1	7.3	298.1	NO FLOODING	2.5	6.3	298.3	NO FLOODING
20	9.6	300.6	3.2	7.5	298.0	NO FLOODING	2.6	6.4	298.2	NO FLOODING
21	10.9	299.2	4.1	15.2	297.9	NO FLOODING	3.7	13.6	298.1	NO FLOODING
22	12.0	300.2	4.8	15.3	297.9	NO FLOODING	4.1	13.7	298.1	NO FLOODING
23	12.3	299.9	5.2	15.5	297.9	NO FLOODING	4.4	13.7	298.1	NO FLOODING
24	12.7	299.0	5.5	15.6	297.8	NO FLOODING	4.8	13.8	298.0	NO FLOODING
25	12.9	299.2	5.6	15.7	297.8	NO FLOODING	4.8	13.8	298.0	NO FLOODING
26	13.0	299.9	5.6	15.7	297.8	NO FLOODING	4.9	13.9	298.0	NO FLOODING
27	13.1	300.4	5.7	15.8	297.8	NO FLOODING	5.0	13.9	298.0	NO FLOODING
28	14.2	299.7	5.9	16.0	297.8	NO FLOODING	5.2	14.0	297.9	NO FLOODING
29	14.4	299.6	5.9	16.0	297.7	NO FLOODING	5.2	14.0	297.9	NO FLOODING
30	14.4	300.6	5.9	16.1	297.7	NO FLOODING	5.2	14.1	297.9	NO FLOODING
31	14.6	300.8	6.0	16.1	297.7	NO FLOODING	5.2	14.1	297.9	NO FLOODING
32	14.7	300.7	6.0	16.1	297.7	NO FLOODING	5.2	14.1	297.9	NO FLOODING
33	14.9	300.6	6.0	16.1	297.7	NO FLOODING	5.2	14.1	297.8	NO FLOODING
34	14.9	300.9	6.0	16.2	297.7	NO FLOODING	5.3	14.1	297.8	NO FLOODING
35	15.1	301.6	6.0	16.2	297.7	NO FLOODING	5.3	14.1	297.8	NO FLOODING
36	15.1	299.5	6.0	16.2	297.7	NO FLOODING	5.4	14.0	297.8	NO FLOODING
37	15.2	301.5	6.1	16.3	297.6	NO FLOODING	5.5	14.0	297.8	NO FLOODING
38	15.4	298.5	6.1	16.3	297.6	NO FLOODING	5.5	14.0	297.7	NO FLOODING
39	15.4	302.2	6.1	16.3	297.6	NO FLOODING	5.6	14.0	297.7	NO FLOODING
40	15.5	299.7	6.1	16.4	297.6	NO FLOODING	5.7	14.0	297.7	NO FLOODING
41	15.6	299.2	6.2	16.4	297.6	NO FLOODING	5.7	13.9	297.6	NO FLOODING
42	15.7	301.4	6.3	16.4	297.5	NO FLOODING	5.7	13.8	297.6	NO FLOODING
43	15.8	299.9	6.4	16.5	297.5	NO FLOODING	5.7	13.7	297.6	NO FLOODING
44	15.9	300.4	6.5	16.5	297.5	NO FLOODING	5.7	13.7	297.6	NO FLOODING
45	16.1	301.2	6.6	16.5	297.5	NO FLOODING	5.8	13.6	297.5	NO FLOODING

CROSS SECTION NO.	LOCATION	DISTANCE FROM DYKE EM-B	APPROXIMATE THALWEG ELEVATION	ESTIMATED CHANNEL BANK ELEVATION	PIPING FAILURE FLOOD				OVERTOPPING FAILURE FLOOD			
					FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	FLOOD PEAK DISCHARGE	FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	FLOOD PEAK DISCHARGE
		(km)	(m)	(m)	(hour)	(hour)	(m)	(m^3/s)	(hour)	(hour)	(m)	(m^3/s)
B1	Dyke EM-B Centerline	0.00	317.0	318.0	0.0	1.0	325.0	886	0.0	0.5	326.6	2041
B2	Toe of Dyke EM-B	0.03	315.5	316.5	0.0	1.0	319.4	886	0.0	0.5	320.7	2041
B3	Unnamed Creek	0.19	312.5	313.5	0.1	1.0	317.8	868	0.1	0.5	319.2	1981
B4	Snowmobile Trail	0.45	311.5	312.5	0.2	1.1	315.4	861	0.1	0.6	316.6	1938
B5	Local Road Crossing	1.22	307.5	308.5	0.5	1.4	310.7	820	0.3	0.8	311.5	1788
B6	Lake Fourniere	1.80	306.5	307.5	0.9	1.8	309.8	718	0.5	1.0	310.2	1588

HOUSE / SURVEY ID	DISTANCE FROM DYKE EM-B	GROUND ELEVATION	PIPING FAILURE FLOOD				OVERTOPPING FAILURE FLOOD			
			FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	MAXIMUM FLOOD DEPTH	FLOOD ARRIVAL TIME	TIME TO FLOOD PEAK LEVEL	FLOOD PEAK LEVEL	MAXIMUM FLOOD DEPTH
	(km)	(m)	(hour)	(hour)	(m)	(m)	(hour)	(hour)	(m)	(m)
46	1.4	314.3	0.6	1.4	310.7	NO FLOODING	0.4	0.8	311.5	NO FLOODING
47	1.9	304.9	0.9	1.8	309.7	4.8	0.5	1.0	310.1	5.2

LEGEND

- CONTOUR LINE (10 m)
- DYKE BREACH FLOOD INUNDATION EXTENT (OVERTOPPING)
- DYKE BREACH FLOOD INUNDATION EXTENT (PIPING)
- POLISHING POND
- B1-B6 / A1-A13 CROSS-SECTION NUMBER AND LOCATION
- ROAD CROSSING
- FLOW DIRECTION
- ROAD
- WATERCOURSE, CREEK, RIVER OR LAKE
- HOUSE / SURVEY LOCATION

1 0 1
SCALE KILOMETRES

REFERENCE

ORIGINAL DRAWINGS OBTAINED FROM CLIENT. 32c04201h.dwg, 32c04201t.dwg, 32cd101t.dwg, 32da202t.dwg 1:20,000 TOPO MAPS CONVERTED TO MTM ZONE 10

PROJECT
POLISHING POND DYKE BREACH FLOOD INUNDATION STUDY FOR THE MALARTIC PROJECT

TITLE
DYKE BREACH FLOOD INUNDATION MAP

Golder Associates
Calgary, Alberta

PROJECT	07.1221.0028.4000		FILE No.	Flood Inundation
DESIGN	MM	12/03/09	SCALE AS SHOWN	REV. 2
CADD	TY	07/05/09		
CHECK	MM	07/05/09	DRAWING: 1	
REVIEW	HZ	07/05/09		

APPENDIX C

A CD Containing the Report and the Dyke Breach Flood Inundation Map

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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