Implantation d'un terminal méthanier à Lévis
Étude d'impact sur l'environnement

Complément à l'étude d'impact sur l'environnement

Réponses aux questions et commentaires des agences réglementaires

Addenda I – Sismicité dans la zone d'implantation du terminal (en réponse aux questions CA-014 à CA-023)

Rabaska

SNC•LAVALIN Environnement

Octobre 2006

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1 INTRODUCTION

Trois rapports sont fournis en Annexe:

- Le rapport préliminaire d'étude sismique locale est fourni en Annexe A (Atkinson 2006) ;
- L'addendum au rapport géotechnique Terratech 2006 (Terratech Addendum 2006) est fourni en Annexe B ; ce rapport inclut les informations tectoniques et sismiques du rapport géotechnique antérieur (Terratech 2005), ainsi que l'évaluation du potentiel de liquéfaction des sols basée sur les résultats du rapport d'étude sismique locale (Atkinson 2006) ;
- Le rapport préliminaire de paléosismologie (Tuttle 2006) est fourni en Annexe C.

Les valeurs d'accélération données pour le site de Rabaska en 2005 par Séismes Canada (sur la base des modèles nationaux utilisés pour le CNB 2005), étaient reproduites dans le rapport Terratech (2005) et sont maintenant inclues au rapport Terratech (Addendum 2006). Ces valeurs ne pourront être utilisées que pour les éléments non critiques du terminal. Conformément aux normes CSA-Z276, NFPA-59A et EN-1473, les éléments critiques tels que les réservoirs de GNL, seront calculés suivant les valeurs d'accélérations fournies par l'étude sismique locale (dans sa version finale), plus fiables que les valeurs du CNB 2005 pour le site considéré.

Le calcul du risque sismique local est basé sur des hypothèses prudentes, en particulier la possibilité que la zone sismique du Charlevoix, située à environ 70 km, puisse s'étendre jusqu'au secteur d'implantation du terminal est prise en compte. L'étude de paléosismologie est actuellement en cours (mentionnée en réponse aux questions CA-014s2 à CA-024s2). Elle a pour objectif de déterminer la fréquence de grands tremblements de terre préhistoriques sur le site considéré et dans le Charlevoix, afin de mieux cerner les facteurs affectant le risque sismique local. Tel que le montre le rapport préliminaire de paléosismologie, les investigations menées à la fin de l'été 2006 dans la région de La Malbaie et dans la région de Lévis/Québec, ont permis de localiser et de dater quelques affleurements de couches sédimentaires propices à cette analyse. Les résultats obtenus sont prometteurs mais encore insuffisants pour conclure avec certitude. Des investigations complémentaires seront donc menées à des périodes plus favorables pour l'observation sur le terrain (lorsque la végétation aura disparu), soit à la fin de l'automne 2006 ou au début de l'été 2007, selon les conditions climatiques.

Si les résultats de ces compléments d'investigation sont concluants en regard des contraintes appliquées au secteur étudié sur les fréquences d'occurrence de grands séismes au cours des derniers 10 000 ans, l'étude sismique locale (Atkinson 2006), ainsi que l'évaluation du potentiel de liquéfaction (Terratech Addendum 2006) seront révisés en conséquence.

La vitesse de l'onde transversale du rocher sur lequel reposera la base des réservoirs, a été estimée à près de 800 m/s, ce qui correspond à la limite B/C des conditions de sols (« rocher » / « terre ferme »). L'analyse de risque sismique local est préparée pour ces conditions. La réponse du sol situé au-dessus de ce rocher et l'amplification résultante des mouvements, seront considérées pour toute structure dont les fondations ne sont pas situées sur le rocher.

L'étude sismique locale préliminaire, fournit des valeurs d'accélération supérieures à celles préconisées par le CNB 2005. Par exemple, pour une période de retour de 2 500 ans, le PGA est égal à 0,45 g, soit 30% de plus que le PGA du CNB 2005 qui est égal à 0,34 g.

Bien que la valeur du PGA soit plus élevée que dans l'étude de comparaison de site (Roche 2004a), le site retenu pour l'implantation du terminal reste acceptable du point de vue sismique et présente le risque sismique le plus faible par rapport aux deux autres sites qui avaient été considérés, Gros Cacouna et Pointe Saint-Denis.

Rabaska a fait le choix de concevoir les réservoirs de GNL suivant un seuil de SSE (« Safe Shutdown Earthquake ») correspondant à la période de retour préconisée par la norme EN 1473, plus contraignante que celle des normes CSA-Z276 et NFPA-59A actuelles. La révision de la norme EN 1473 devant être publiée en 2007, c'est cette version qui sera applicable pour l'ingénierie détaillée des réservoirs de GNL, avec une période de retour de 5 000 ans.

Les valeurs d'accélération obtenues dans l'étude sismique locale préliminaire pour cette période de retour (PGA égal à 0,64 g), ne nécessitent pas d'utiliser des isolateurs sismiques pour les réservoirs de GNL.

Les objectifs de performance des réservoirs vis-à-vis du séisme, bien qu'exprimés différemment, sont comparables entre les normes CSA-Z276, NFPA-59A ou EN 1473 ; seule la période de retour du SSE induit une différence significative. C'est donc ce choix d'une période de retour et la définition consécutive des accélérations spectrales de conception dans les périodes de vibration critiques pour les réservoirs, qui auront le plus d'influence sur leur performance. Dans le cas des réservoirs de Rabaska, les accélérations spectrales de conception dans les périodes de vibration critiques, ne pourront pas être inférieures aux accélérations spectrales fournies dans la version finale de l'étude sismique locale.

En conclusion, les réservoirs de GNL seront conçus avec la période de retour recommandée par l'EN 1473 (5 000 ans dans la version 2007), plus prudente que les codes CSA-Z276 (2007), NFPA-59A (2006) et CNB (2005) recommandant 2 500 ans ; les spectres sismiques utilisés seront le résultat d'une étude sismique locale (complétée d'une étude de paléosismologie), plus fiable que les modèles sismiques nationaux utilisés par le CNB. La philosophie de l'EN 1473 consiste à prendre en compte une conception anti-sismique très prudente par le choix d'une période de retour élevée, et donc des marges de conception importantes. Le niveau de sécurité du terminal est donc assuré grâce à cette approche prudente. Les émissions accidentelles provoquées par des séismes sont ainsi considérées comme un risque négligeable. Le séisme est cependant considéré dans le plan d'urgence du terminal.

1.1 Rappel des questions relatives à la séismicité / installations GNL

COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006	ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006
CA-014	**CA-014s2 à CA-024s2**
Référence : Tome 3, volume 1, section 2.2.5 et 4.4.6 *Il n'existe aucune preuve qu'on a tenu compte des commentaires génériques de Ressources naturelles Canada sur les risques sismiques associés aux installations de GNL (voir l'annexe 1) établis pour d'autres projets de GNL au Canada. L'étude aborde toutefois certains des points soulevés par les commentaires génériques, principalement en recommandant le recours au code européen EN1473, étant donné que les initiateurs le jugent plus strict que les codes américain ou canadien. Il semble qu'il n'y ait pas eu d'évaluation du risque sismique propre au site, et l'évaluation de la sismicité et du risque sismique présentée dans l'EIE ressemble davantage à de la documentation de type examen préalable qu'à l'évaluation en profondeur à laquelle on serait en droit de s'attendre.* ***Question/Commentaire** :* *Si un plan d'intervention d'urgence en cas de secousse sismique a été établi, veuillez en dévoiler les détails.* **REPONSE** Le risque séismique a été considéré pendant l'atelier d'identification des risques (HAZID) dans la rubrique des risques externes. Ce sujet est documenté au tome 3, volume 2, annexe F-1, section 5.1.3 et annexe 1. Les séismes sont aussi considérés dans la liste des scénarios d'urgence (voir tableau 36 de l'annexe F-1). Le plan des mesures d'urgence du terminal méthanier Rabaska sera basé sur une évaluation systématique des besoins et des exigences pour la préparation aux situations d'urgence. Une telle évaluation est généralement désignée sous le nom d'analyse de la préparation aux situations d'urgence (Emergency Preparedness analysis). Les résultats de l'analyse quantitative	***Généralités :*** *Bien que nous attendions le rapport sur les risques sismiques propre au site, la plupart des points ont été traités de manière satisfaisante.* ***Commentaires particuliers :*** *Les conséquences de modifications futures à la EN1473 en ce qui a trait à la période de récurrence du séisme majoré de sécurité (SSE) pourraient être abordées dans le rapport sur les risques sismiques propre au site. Ressources naturelles Canada envisagerait sérieusement toute proposition de prendre pour SSE la mesure de 1:5000 ans (comme il est proposé pour la version de la norme EN1473 prévue aux alentours de 2007 au lieu du SSE actuel de 1:10 000 ans pourvu que cela n'affecte pas la sécurité. Pour une autre installation canadienne de GNL au stade de la planification, il a été proposé d'utiliser des normes comme les normes CSA Z276 et NFPA 59A mais avec une mesure de 1:5000 ans pour le calcul du SSE. Si le niveau de performance tiré de la norme EN1473 avec une période de récurrence de 1:5000 ans se compare à ces mesures de calcul, il pourrait être considéré comme acceptable.* **REPONSE** L'étude sismique locale préliminaire a été réalisée. Actuellement, nous procédons à une étude détaillée des paramètres d'accélération des sols à retenir dans la conception anti-sismique, incluant les niveaux de probabilité associés aux conditions SSE et une analyse spécifique visant à déterminer les accélérations propres au site de Rabaska. Afin d'obtenir la meilleure définition possible pour les prévisions d'accélération des sols, nous menons une étude de paléosismologie avec le concours d'une experte de niveau

COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006	ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006
des risques sont pris en compte dans l'analyse de la préparation aux situations d'urgence. Le plan d'urgence sera développé en se basant sur l'analyse de la préparation aux situations d'urgence et devra être achevé 6 mois avant la mise en service du terminal. Un plan préliminaire des mesures d'urgence est donné au tome 3, volume 2, annexe F-1, section 10.	international (Dr M. Tuttle) dans le but de cerner avec plus de précision les facteurs affectant à long terme les niveaux de séisme locaux, par comparaison avec les niveaux de la zone sismique de Charlevoix. L'étude paléosismologie s'appliquera à détecter et à comparer, dans les régions de Lévis et Charlevoix, des indices ou preuves de niveaux de séisme pour une période remontant à 10 000 ans. Ces investigations sont en cours. Le rapport et les réponses aux questions posées seront déposés ultérieurement.
CA-015	**CA-015s2**
Référence : Tome 2, tableau 4.2 ***Question/Commentaire :*** *Ce tableau cerne bien les niveaux relatifs de risque sismique des trois sites choisis, mais l'appréciation du site de Lévis/Beaumont comme « acceptable » est subjective.* **REPONSE** Une étude de comparaison des sites (Roche 2004a) avait identifié les valeurs suivantes d'accélérations horizontales maximales au sol (PGA) avec une probabilité de 2 % sur 50 ans (issues du projet en 2004 de révision du code national du bâtiment) : • Ville-Guay (Lévis-Beaumont) : 0,36g • Gros Cacouna : 0,56g • Pointe Saint Denis : 1,1g Ainsi le risque sismique avait été qualifié de modéré à Ville-Guay, élevé à Gros Cacouna et extrême à Pointe Saint Denis. Le risque sismique beaucoup plus élevé du site de Gros-Cacouna et surtout de Pointe Saint Denis, ne rendait pas, a priori, ces sites inacceptables d'un point de vue sismique (voir ci-dessous), mais aurait impliqué une conception anti-sismique des installations plus complexe. Par comparaison, le niveau de risque du site	*Réponse acceptable mais nous attendons l'étude des risques sismiques.* **REPONSE** Voir réponse à la question CA-14s2.

COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006	ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006
de Ville-Guay laissait présager une conception plus conventionnelle, de ce fait ce site a été jugé préférable par le promoteur. La prise en compte du risque sismique (sur des bases uniquement bibliographiques) lors des étapes de sélection des sites, est rendue possible par l'expérience acquise en matière de conception anti-sismique des installations de GNL. En effet, il existe dans le monde de nombreux exemples d'installations GNL construites dans des zones hautement sismiques, et à ce jour, il n'y a aucun cas connu de défaillance de réservoir de GNL dû à un tremblement de terre. Sans être exhaustif, voici quelques exemples de localisations reconnues pour leur haute séismicité : • le Japon (25 terminaux dont les mises en service s'échelonnent de 1969 à 2003, totalisant 166 réservoirs de GNL) avec en particulier la baie de Tokyo et la baie d'Osaka; • la Turquie (terminal de Marmara mis en service en 1994, 3 réservoirs); • la Grèce (terminal de Révithoussa mis en service en 2000, 2 réservoirs). Certains de ces terminaux ont été touchés par des séismes majeurs comme par exemple les séismes d'Osaka-Kobé en 1995 et d'Izmit en 1999; dans les deux cas aucun dommage significatif aux installations de GNL n'a été rapporté. Ces exemples illustrent le fait que le risque sismique a depuis longtemps été intégré aux différents codes utilisés pour la conception des installations de GNL de par le monde (japonais, américains ou européens) et que les techniques de construction anti-sismique sont maîtrisées. Cela repose notamment sur la réalisation systématique d'analyse de risque sismique propre au site choisi. Plus un site présente un risque sismique a priori élevé, plus tôt cette analyse est menée pour confirmer le choix d'un site et fournir les données de calcul des structures. Dans le cas du site de Lévis-Beaumont, la séismicité modérée n'est pas de nature à remettre en cause le choix de ce site. L'analyse de risque sismique est actuellement en cours afin	

COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006	ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006
de confirmer les choix techniques de l'ingénierie préliminaire et d'utiliser les résultats pour la phase d'ingénierie détaillée. Le rapport d'étude sismologique sera disponible sous peu.	
CA-018	**CA-018s2**
Référence : Tome 3, section 4.4.6.6 *Il semble que la base des fondations des réservoirs de GNL reposera sur un substrat rocheux fracturé, c.-à-d. que tous les sédiments seront excavés et qu'aucune partie des fondations des réservoirs de GNL ne reposera sur du sable, de la boue ou de l'argile. Il semble que le substrat rocheux ne soit guère compétent (force portante de 250 kPa) si on compare cette force à celle du substrat rocheux sous-jacent, mentionnée dans le paragraphe suivant.* ***Question/Commentaire** :* *Il faut évaluer la vitesse de l'onde transversale de ce matériau pour pouvoir convertir le risque sismique standard sur « terre ferme » auquel on peut s'attendre à la base des fondations des réservoirs de GNL. Si la vitesse de l'onde transversale n'est pas connue, il peut être acceptable de la considérer comme « terre ferme » mais il ne faut invoquer aucune désamplification des mouvements du sol (comme ce serait le cas sur le roc).* **REPONSE** La capacité portante du socle rocheux a été réévaluée à 500 kPa dans le rapport géotechnique final (Terratech 2006). Cette pression admissible sur le rocher est suffisante pour supporter les réservoirs de GNL (en règle générale, 250 kPa est le minimum requis pour ce genre de structure). L'analyse du risque sismique local, incluant ce substrat, est en cours et le rapport d'étude sismologique sera disponible sous peu.	*Réponse acceptable, mais sera examiné plus en profondeur dans l'étude des risques sismiques.* **REPONSE** Voir réponse à la question CA-14s2.

<table>
<tr><th>COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006</th><th>ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006</th></tr>
<tr><td>CA-019</td><td></td></tr>
<tr><td>Référence : Tome 3, section 4.4.6.8

Question/Commentaire :

Les valeurs données par le CNB 2005 pour le site (coordonnées supposées 46.820N 71.062O) sont légèrement supérieures à celles que l'on retrouve dans l'EIE (tirées des valeurs correspondant à Lévis) étant donné la présence d'un gradient provenant de la zone sismique de Charlevoix toute proche. Les valeurs données par le CNB 2005 pour le site sont Sa(0,2) = 0,58, Sa(0,5) = 0,32, Sa(1,0) = 0,15 et Sa(2,0) = 0,052 g et PGA = 0,35 g. Bien qu'elle ne se retrouve pas dans le CNB 2005, la valeur PGV calculée à l'aide du même modèle et de la même méthode est 0,15 m/s. Les valeurs PGA et PGV sont à peu près deux fois plus élevées que celles du CNB 1985/1995, ce qui est caractéristique de nombreux sites, étant donné la baisse de niveau de probabilité entre 1985/95 et 2005.

REPONSE

Nous prenons note de l'erreur qui s'était glissée dans l'EIE. Il s'agit d'une erreur de transcription qui est sans conséquence pour l'ingénierie préliminaire car ce sont bien les valeurs rappelées ci-dessus qui ont été utilisées (voir rapport Terratech 2005 transmis à l'ACÉE - CA-026), et qui seront également incluses au rapport d'étude sismique qui sera disponible sous peu.</td><td></td></tr>
<tr><td>CA-020</td><td>CA-020s2</td></tr>
<tr><td>Référence : Tome 3, section 4.4.6.8

Il ne faut pas utiliser les valeurs du CNB pour la conception critique de l'usine étant donné que les valeurs de probabilités supérieures à 2 %/50 ans ne sont considérées comme fiables que pour la construction de bâtiments standard ou de structures présentant une fiabilité semblable. Les valeurs applicables à des probabilités inférieures à 2 %/50 ans (p. ex. 1 %/50 ans) risquent de ne</td><td>Nous attendons l'étude des risques sismiques.

REPONSE

Voir réponse à la question CA-14s2.</td></tr>
</table>

COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006	ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006
pas être des indicateurs fiables du risque sismique véritable puisqu'elles proviennent d'un modèle à l'échelle du pays qui ne peut, pour des raisons pratiques, qu'être très général. Par exemple, la position des limites de la zone sismique utilisées pour délimiter les positions des secousses sismiques à l'origine du risque peuvent être évaluées à la lumière de connaissances locales détaillées, ce qui donne une évaluation supérieure ou inférieure du risque. ***Question/Commentaire** :* *Par conséquent, pour cette usine de GNL, il faut effectuer une évaluation du risque sismique propre au site. Un commentaire précis est que si le modèle « H » est le modèle de contrôle pour la plupart des périodes à 2 %/50 ans, les valeurs du modèle « R » sont assez proches et la limite du modèle « R » à Québec constitue une valeur très brute et est probablement située trop à l'ouest (c.-à-d., éloignée du site de Rabaska), de sorte que l'évaluation du risque sismique est sans doute trop basse.* **REPONSE** L'analyse du risque sismique local est en cours et le rapport d'étude sismologique sera disponible sous peu.	
CA-021	**CA-021s2**
Référence : Tome 3, section 4.4.6.8 *« une période de retour de 10 000 ans » : Ressources naturelles Canada voudra vérifier la norme EN1473, mais est d'accord avec l'idée d'utiliser la norme la plus stricte.* ***Question/Commentaire :*** *Il faudra effectuer une évaluation propre au site pour évaluer les mouvements du sol.*	*Accepté, mais se reporter au commentaire ci-dessus concernant les normes.* **REPONSE** Voir réponse à la question CA-14s2.

COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006	ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006
REPONSE L'analyse du risque sismique local est en cours et le rapport d'étude sismologique sera disponible sous peu. Pour information des extraits pertinents de la norme EN 1473 (1997) sont reproduits ci-dessous. Les seuils OBE et SSE sont définis comme suit aux paragraphes 3.10 et 3.11 : • **« OBE (Operating Basis Earthquake = séisme de maintien en exploitation) :** Un OBE, défini pour toute installation, est le séisme maximal n'entraînant aucun dommage et pour lequel un redémarrage et un fonctionnement peuvent être effectués en toute sécurité. Pour cet événement de probabilité plus élevée, la sécurité du public est assurée sans provoquer la perte commerciale de l'installation. Un OBE doit nécessiter une analyse de structures pour les conditions d'état limite de service ». • **« SSE (Safe Shut Down Earthquake = séisme d'arrêt de sécurité) :** Un SSE, défini pour toute installation, est le séisme maximal pour lequel les fonctions et les mécanismes essentiels de mise en sécurité sont conçus pour être préservés. Un dommage permanent sans perte de l'intégrité globale des installations est possible suite à ce phénomène de faible probabilité. L'installation ne doit pas être maintenue en service sans un examen détaillé et une analyse de structures pour les conditions d'état limite ultime ». La période de retour pour les seuils OBE et SSE est précisée au paragraphe 4.2.4 de la norme EN1473 : « Les études géologiques, tectoniques et sismologiques permettent de déterminer : • le séisme d'arrêt de sécurité (SSE); • le séisme de maintien en exploitation (OBE). Elles doivent être définies : • soit d'une manière probabiliste, comme étant les tremblements de terre de probabilité d'occurrence égale à un séisme pour 10 000 ans pour le SSE	

COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006	ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006
et d'un séisme pour 475 ans pour l'OBE; et/ou • soit d'une manière déterministe. Dans ce cas, le SSE correspond au séisme maximum historiquement vraisemblable susceptible de se produire; son épicentre étant positionné de la façon la plus pénalisante par rapport à ses effets en termes d'intensité sur le site, tout en restant compatible avec les données géologiques et sismiques. Les accélérations pour un OBE doivent être la moitié de celles définies pour un SSE ». L'intégralité de cette norme est disponible en version française et anglaise sur le site de l'Association Française de Normalisation : http://www.boutique.afnor.fr À noter que la norme EN 1473 fait actuellement l'objet d'un projet de révision qui devrait être proposé d'ici quelques mois (édition finale prévue en 2007). Ce projet envisage de réduire la période de retour pour le SSE à 5 000 ans.	
CA-022	
Référence : Tome 3, section 4.4.6.8 ***Question/Commentaire :*** *La conception de la salle de commande devrait tenir compte du fait que les installations reposent sur des sédiments (et non sur le roc, voir p. 4.15) : son exploitation en permanence peut exiger une conception selon un niveau supérieur à celui du CNB.* **REPONSE** Voir la réponse à la question CA-020.	
CA-023	
Référence : Tome 3, section 4.17.2 ***Question/Commentaire :*** *Le promoteur estime que, selon l'évaluation actuelle du risque sismique, il ne semble pas nécessaire de prévoir une isolation sismique. Il convient de noter*	

COMPLÉMENT A L'ÉTUDE D'IMPACT – MAI 2006	ADDENDA B À L'ÉTUDE D'IMPACT – AOÛT 2006
que les mouvements du sol 1/10 000 ans pourraient être deux fois (ou 1,5 fois, ou 4 fois – à déterminer) plus intenses que les valeurs du CNB 2005, de sorte que les solutions techniques précises ne sont peut-être pas encore évidentes. **REPONSE** L'analyse du risque sismique local qui est en cours précisera les valeurs d'accélération à retenir. Le rapport d'étude sismologique sera disponible sous peu. Les conclusions actuelles relatives à la conception des réservoirs de GNL sont basées sur l'expérience. La conception finale des réservoirs sera précisée à l'ingénierie de détail.	

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ANNEXE A

Rapport préliminaire d’étude sismique locale

EARTHQUAKE HAZARD ANALYSIS:

RABASKA LNG FACILITIES, QUEBEC

Preliminary Report

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For:
SNC-Lavalin Inc.
Revision 4 : Sept. 26, 2006

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SEISMIC HAZARD ANALYSIS: RABASKA LNG FACILITIES, QUEBEC

Executive Summary

A site-specific seismic hazard assessment was performed for the proposed LNG terminal site at Rabaska, Quebec. The analysis determines the expected earthquake ground motions over a range of probability levels, including 1/500, 1/1000, 1/2500 and 1/5000. The 1/500 per annum (p.a.) ground motion corresponds to the "Operating Basis Earthquake" (OBE) level in LNG facility codes such as CSA Z276 (Canadian, upcoming 2007 edition), NFPA59A (U.S., 2006 edition) and EN1473 (Europe, upcoming 2007 edition), while the 1/2500 p.a. motions correspond to the "Safe Shutdown Earthquake" (SSE) level in CSA Z276 (2007) and NFPA59A (2006). The SSE level in the EN1473 (2007) is 1/5000 p.a. The choice of probability level for the OBE and SSE is made by the Owner and will meet the requirements of the Canadian code. Additionally, it is my understanding that the Owner intends to apply for the design of LNG tanks the more stringent SSE return period recommended by EN1473. The ground motions are calculated for the site ground-motion conditions of "soft rock" (NEHRP B/C boundary, with shear-wave velocity in the top 30 m of approximately 800 m/s). The emphasis in this study is on deriving the range of estimates, including the impact of the chief sources of uncertainty. Weighted mean-hazard results are also provided (Table 2) for each probability level.

The results can be summarized in simplified terms as follows. At the probability level of 1/2500, the expected peak ground acceleration (PGA) for the soft-rock (B/C) site conditions at Rabaska is approximately 0.45g. For comparison, the acceleration at Rabaska from the national seismic hazard maps produced by the Geological Survey of Canada (2003), for the 1/2500 p.a. probability, is 0.34g (for the GSC reference condition of NERHP C). The ground motions at this probability level (1/2500) correspond approximately to a magnitude 7 earthquake occurring in the Charlevoix seismic zone, at a distance of about 70 km from the site, or a magnitude 6 local earthquake, about 20 km from the site. At the probability level of 1/5000, the expected PGA at Rabaska is approximately 0.64g, corresponding approximately to a magnitude 7 earthquake at about 50 km, or a magnitude 6 earthquake at about 15 km from the site.

1 - INTRODUCTION

This report presents a seismic hazard assessment for the site of the proposed LNG terminal facilities at Rabaska, Quebec (46.82N, 71.06W) for annual exceedence probabilities in the range from 1/500 to 1/5000. By comparison, the CSA Z276 guidelines for LNG facilities (upcoming 2007 edition) are expected to refer to ground motions for an Operating Basis Earthquake (OBE) with an annual probability of 1/500 and a Safe Shutdown Earthquake (SSE) with an annual probability of 1/2500; these probability levels (1/500 and 1/2500) match those in the U.S. Standard NFPA59A (2006 edition). The

European code, EN1473 (Europe, upcoming 2007 edition) refers to an OBE probability of 1/500 and an SSE probability of 1/5000. The analysis determines the likelihood of ground motion at the site by considering the magnitudes, rates of occurrence, and locations of earthquakes, using the probabilisitic Cornell-McGuire method. The method is widely used throughout North America and forms the basis for seismic zoning maps in building codes in Canada (Adams and Halchuck, 2003). This assessment represents an update and site-specific refinement of the type of estimate provided in the National Seismic Hazard maps by the Geological Survey of Canada (GSC, Adams and Halchuck, 2003); the results of this study address more specifically the tectonic setting of the Rabaska site, and incorporate new information on seismicity and ground motion relations from the last 10 years of data. To include new and more complete information, a range of possible models to describe the seismic setting is defined.

In analyzing the engineering effects of ground motion, both the amplitude and frequency content of the vibrations are important. Therefore the seismic ground motions are expressed using the response spectrum (PSA(f)), which shows the maximum acceleration that a simple structure would experience as a function of its natural frequency. The response spectrum result is a Uniform Hazard Spectrum (UHS), in which the amplitude for each frequency corresponding to a specified exceedence probability is provided. The peak ground acceleration (PGA) for this probability is also estimated, as is the peak ground velocity (PGV). The frequency associated with the PGA varies, but in general the PGA is associated with high-frequency motions (near 10 Hz); the PGV is associated with motions near 2 Hz. The UHS results of this study are presented in the figures and tables provided in Section 3.

Time histories of ground motion that match the UHS for specified probability levels will be developed in a later phase of the project. The time histories may be derived by modifying real earthquake records that are appropriate for eastern Canadian rock sites, for magnitude-distance ranges that dominate the hazard at Rabaska. The modifications are done to spectrally match the original record to the target UHS through an iterative process of amplitude adjustment in the frequency domain.

2 - SEISMIC HAZARD ANALYSIS METHOD

2.1 Overview

Seismic hazard analyses in eastern Canada are based on probabilistic concepts which allow incorporation of both geologic interpretations of seismic potential and statistical data regarding the locations and sizes of past earthquakes. The Cornell-McGuire method (Cornell, 1968; McGuire, 1976, 1977, 2004) has proven particularly well-suited to calculate expected ground motions for a wide range of seismic hazard environments, offering flexibility in the consideration of spatial and temporal characteristics of regional earthquake occurrence, and the basic physics of the earthquake process.

In general, it is difficult to correlate seismicity with specific faults. Earthquakes typically occur at depths of 5 to 20 km, on faults that have no surface expression. Furthermore, faults mapped on the surface in eastern Canada were formed hundreds of millions of years ago, and may bear little relation to current seismic activity. Thus there is no clear-cut relationship between observed faults and seismicity. (Note: This is apparent in Figure 2, showing Charlevoix seismicity in comparison to mapped faults.) Geotechnical reports for Rabaska (Terratech, 2006) are consistent with this view. The site geology consists of folded and faulted Paleozoic strata formed approximately 500 million years ago as part of the Appalachian province. During the Taconian Orogeny, the sediments were pushed over the underlying Precambian basement rocks, which lie approximately 4 km below the Paleozoic sediments at the site. Major tectonic activity in the region ceased about 400 million years ago. Investigations by Terratech, using seismic refraction geophysical survey and diamond core drilling, along with two trial excavations, have provided information on the quality of the rock and its overall structure. They conclude that there is no evidence of recent faulting identified in the exposed strata at the site area or in boreholes. For example, the rock core recovered in some boreholes present evidence of faulting, but the rock appears to be healed as indicated by cementing of the fault with secondary minerals such as calcite (Terratech, 2006). It is important to recognize that in this region, the Appalachian rocks are underlain at depth by older Precambrian sequences, in which the seismicity occurs; most seismicity in the Charlevoix seismic source zone occurs between 7 and 15 km below the surface, with earthquakes occurring to depths of up to 30 km (Lamontagne et al., 2000). Thus we would expect modern earthquake-related faulting to occur below the Appalachian rock sequence, rather than within it. Any such faulting would leave little surficial evidence, other than perhaps the disturbance of post-glacial sediments if shaking was sufficiently strong. The examination of post-glacial soils near the site during the trial excavation (which was conducted over a known fault or fold) led to the conclusion that there is no clear evidence that the soil materials were tectonically disturbed (Terratech, 2006).

The spatial distribution of earthquakes is described by defining seismic source zones (faults or areas, which may contain groups of faults) on the basis of seismotectonic interpretations; the earthquake potential of these zones is generally assumed to be uniform. The frequency of earthquake occurrence within each source zone is described by a magnitude recurrence relationship, truncated at an upper magnitude bound, Mx. Earthquake ground motion relations provide the link between the occurrence of earthquakes of various magnitudes and the resulting ground motion levels at any site of interest. The probability of exceeding a specified level of ground motion at a site can then be calculated by integrating hazard contributions over all magnitudes and distances, including all source zones. To obtain ground motion levels or earthquake response spectra for a specified probability, calculations are repeated for a number of ground motion values, for all desired ground motion parameters, and interpolation is used to determine the relationship between ground-motion amplitude and annual probability.

The Cornell-McGuire framework has been well-accepted in all parts of North America. In Canada, it forms the basis for the seismic hazard maps in the National Building Code of Canada (NBCC 1985 and beyond), and is the usual basis for seismic hazard evaluations of all important engineered structures. The results are generally expressed as a Uniform Hazard Spectrum (UHS), in which the amplitude for each frequency corresponding to a specified target probability is provided. The peak ground acceleration (PGA) and velocity (PGV) for the target probability may also be estimated. When time histories of ground-motion are required for use in engineering analyses, these may be derived to be consistent with the expected ground motion characteristics of the UHS for the target probability. The analysis methods used to generate UHS results and time histories are described in more detail by McGuire (2004).

2.2 Treatment of Uncertainty

It has long been recognized that seismic hazard analyses are subject to greater uncertainties than those associated with most environmental phenomena. Two types of uncertainty exist:

- random uncertainty due to the physical variability of earthquake processes
- model uncertainty due to incomplete knowledge concerning the processes governing earthquake occurrence and ground motion generation (eg. uncertainties in input parameters to hazard analysis).

The first type of uncertainty is incorporated directly into the Cornell-McGuire analysis framework, and is included in a standard 'best-estimate' seismic hazard result. The second type of uncertainty implies a spread of possible results about those that might be considered a best estimate. This type of uncertainty can cause differences in results, among alternative hypotheses, of factors of more than two. It also implies that, as new information on seismic hazard becomes available (through seismic monitoring and research) hazard estimates may change significantly from those developed at an earlier time.

Seismic hazard analysis procedures have been developed in recent years to formally evaluate the level of model uncertainty (sometimes referred to as epistemic uncertainty) in hazard analyses. A logic tree approach is often used to represent each input parameter by a simple probability distribution, thereby producing a family of possible output hazard curves, with associated weights (McGuire, 2004). Such an approach has been used in hazard analyses for critical engineered structures such as nuclear power plants (eg. Atkinson, 1990), and has also been used in the latest national seismic hazard maps (Adams and Halchuck, 2003). The logic tree approach is simply a way of formalizing consideration of the implications of alternative assumptions. It is most useful in cases where there is a range of competing alternative hypotheses that significantly impact the seismic hazard results. A full logic tree can be used to define the mean hazard and fractiles (eg. median, 84th percentile) expressing confidence in the estimated UHS. Alternatively, a "logic shrub", including the most significant branches of the logic tree, can be used to determine the mean-hazard UHS by weighting the alternatives for each of the key uncertainties (while leaving fixed the parameters that exert only a minor influence on the results). In this preliminary evaluation of hazard, we focus on a sensitivity approach, which displays the alternative results that are obtained under various alternative input assumptions. This approach is most useful to identify the key uncertainties, and determine the appropriate scope for further refinements to the analyses. We also use a trimmed logic "shrub" to provide weighted mean-hazard UHS results for a range of probabilities.

2.2.1 Seismic Source Zones

A relevant aspect of the treatment of uncertainty in the new national seismic hazard maps, produced by the Geological Survey of Canada (GSC), concerns the issue of alternative seismotectonic hypotheses. Two alternative approaches to defining seismic source zones were defined. In one model (the Historical model), it was assumed that future large earthquakes in eastern Canada will be concentrated in zones of very limited spatial extent, in which they have occurred in the recent past (about 200 years of historical earthquake data on the location of large eastern earthquakes). This model implies high hazard in a few local zones, and low hazard elsewhere.

In the second GSC model (the Iapetan Rift model), it was assumed that future large earthquakes in eastern Canada will occur at random in broad source zones of major crustal weakness, as developed during tectonic rifting episodes associated with the Iapetus Ocean. These zones of weakness include the many ancient rift fault structures, formed about 500 to 700 million years ago, that follow the St. Lawrence and Ottawa River valleys. It is believed that future large events in eastern Canada are most likely to occur within these rifted zones (Adams and Basham, 1989). In the 'rift' hazard model, earthquake activity is smoothed over the entire extent of the rifted regions. This results in enhanced ground motion estimates in parts of the zone that have had low seismicity rates within the period of historical record, and reduced ground motion estimates in areas that have had high seismicity. Figure 1 shows the GSC zones for the two models they consider, in relation to regional seismicity (Note: all events were converted to moment magnitude, as discussed later in the report).

In the GSC hazard analysis approach, which they term the robust approach, the higher of the ground motion estimates from these two alternative zonation models is adopted as the mapped ground-motion parameter (Adams and Halchuck, 2003). This captures a significant geologic uncertainty in most of the populated regions of the St. Lawrence Valley and is appropriate for the purposes of the national hazard maps. However, it is not necessarily a "worst case" for all sites, and is actually unconservative for areas of the St. Lawrence that have higher-than-average levels of seismicity, but lie outside the concentrated zone of activity defined for Charlevoix. Thus it is warranted to examine carefully alternative models for Rabaska, in order to accurately assess and understand the seismic hazard setting and its implications. To do this requires defining additional seismic source models that could be applicable to the Rabaska site, expressing a fuller, more site-specific range of interpretations that the limited regional set considered by the GSC. The definition of these alternative source interpretations is standard practice in state-of-the-art seismic hazard analyses.

As shown in Figure 1, the Rabaska site lies about 60 km southwest of the active Charlevoix seismic zone. The Charlevoix zone is anomalously active for an intraplate environment, with 5 earthquakes of M>6 since the mid-1600s, and hundreds of micro-earthquakes recorded there every year (Lamontagne et al., 2000). The earthquakes occur in Precambrian basement, on reactivated Iapetan rift faults that are hidden in the St. Lawrence and its south shore by several kilometers of Appalachian nappes and hundreds of meters of Quaternary sediments. Although the major faults are defined geophysically (from remote sensing techniques), as shown in Figure 2, the seismicity is seen to be diffuse within the crustal volume and not specifically confined to the interpreted major fault structures (Lamontagne et al., 2000). Consequently, there is uncertainty in the geographic extent of the structures that may participate in this active zone. Furthermore, the relevance of the mapped faults and their specific locations (as per Figure 2) to the seismic hazard at the site is questionable. This is an uncertainty that was not evaluated in the GSC model, but is important for site-specific hazard to Rabaska; specifically, we need to address the possibility that the Charlevoix zone may extend to the Rabaska site. This scenario is not likely, but possible. Another uncertainty not evaluated in the GSC models concerns the actual levels of seismicity at Rabaska. It can be seen on Figure 1, from the density of plotted epicenters, that the activity levels in the site area are lower than those near Charlevoix, but higher than those in other areas of the St. Lawrence or Ottawa Valley. Thus the GSC rift model (IRM zone) may tend to underestimate the seismic hazard at Rabaska.

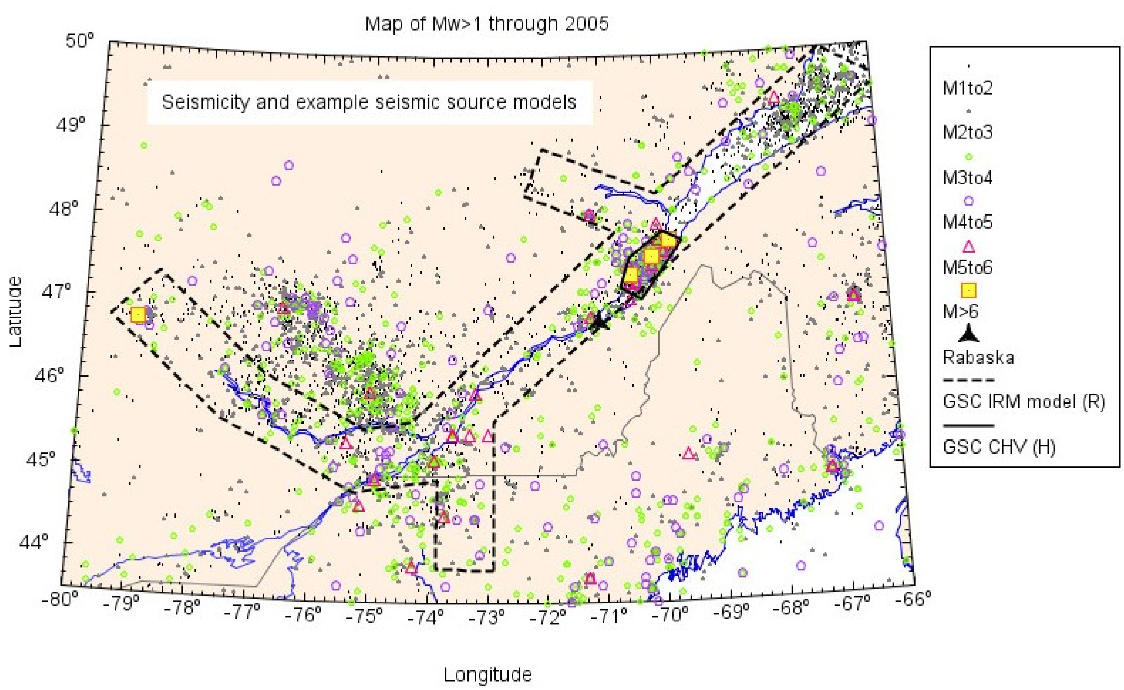

*Figure 1 – Recorded seismicity (**M**>1) through 2005 along the GSC source zone models used in the national seismic hazard maps (dashed black line is IRM rift model, solid black line is Charlevoix zone).*

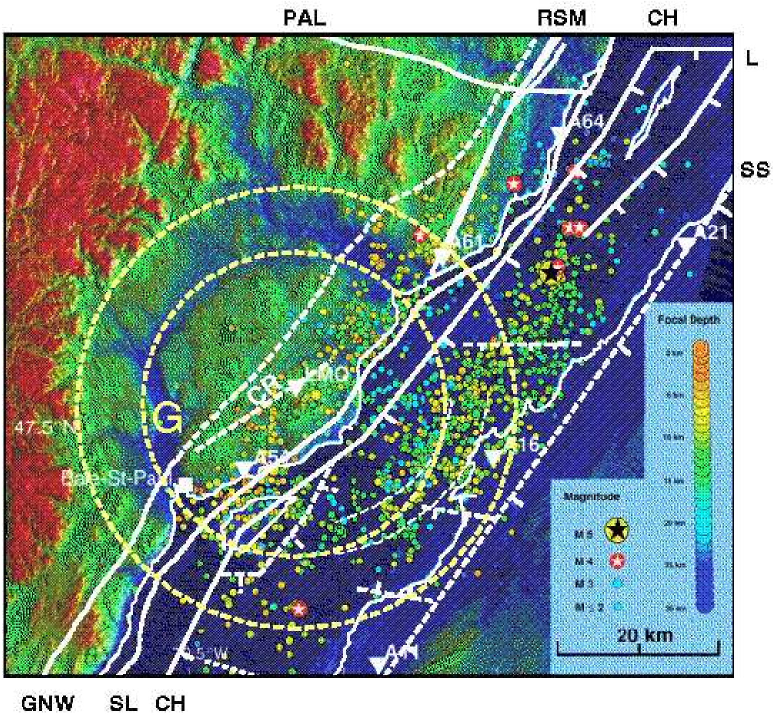

Figure 2 – Structural model of Charlevoix seismic zone, including faults, seismicity, seismograph stations, and Charlevoix impact crater. PAL=Palissades fault; RSM= Rang St.-Mathilde fault; SL=St.-Laurent fault; CH=Charlevoix fault; L+lievres fault; SS=South shore fault; G=peripheral graben of the impact structure; CR=Crater fault; GNS=Fouffre NW fault. (after Lamontagne et al., 2000).

To adequately consider the implications of the local seismicity rates, and the uncertainty in the extent of the Charlevoix activity, we define two alternative representations of the seismic zonation for Rabaska. These are shown on Figure 3. The “confined” Rabaska model (Model A), denoted “RAB_c” on Figure 3, is our best estimate of the source zone boundaries based on historical seismicity. It is based on enclosing regions that are spatially homogeneous in their seismicity levels, along the St. Lawrence system of faults. In this model, the local seismicity at Rabaska is not connected to the Charlevoix activity. To test the importance of the actual boundaries used to define the local zone around Rabaska, an alternative version of this basic “confined seismicity model” is drawn, in which a somewhat larger local zone for Rabaska, denoted “RAB_alt” on Figure 3, is considered (Model C). In addition, we consider a less-likely scenario that acknowledges the uncertainty in the actual areal extent of the Charlevoix activity, by defining a broader Charlevoix zone that extends to the site area, denoted “CHV_b” on Figure 3 (Model B). We also consider the extended IRM model of the GSC (Figure 1) as an alternative source model (Model D). For this project, it is not necessary to consider sources of seismicity at greater distances than those covered by these source zones, as they have insignificant impact on hazard at Rabaska. The set of alternative source models that have been defined here provide a more site-specific description of the seismic setting at Rabaska than do the two source zones used in the GSC regional model developed for the national hazard maps, and more fully cover the range of uncertainty in interpretation of hazard for the Rabaska site.

A recommended approach in the use of these alternative models is to weight the probabilities of ground-motion exceedence from the alternative models according to their likelihood of being correct, based on current knowledge. For example, the CHV_b model and the IRM model (models B and D) both have a low likelihood (say 10%) based on historical seismicity patterns. This differs from the GSC "robust" approach, which takes the worst case of a more limited set of models; for a site-specific assessment, the weighted approach is generally accepted as preferred practice.

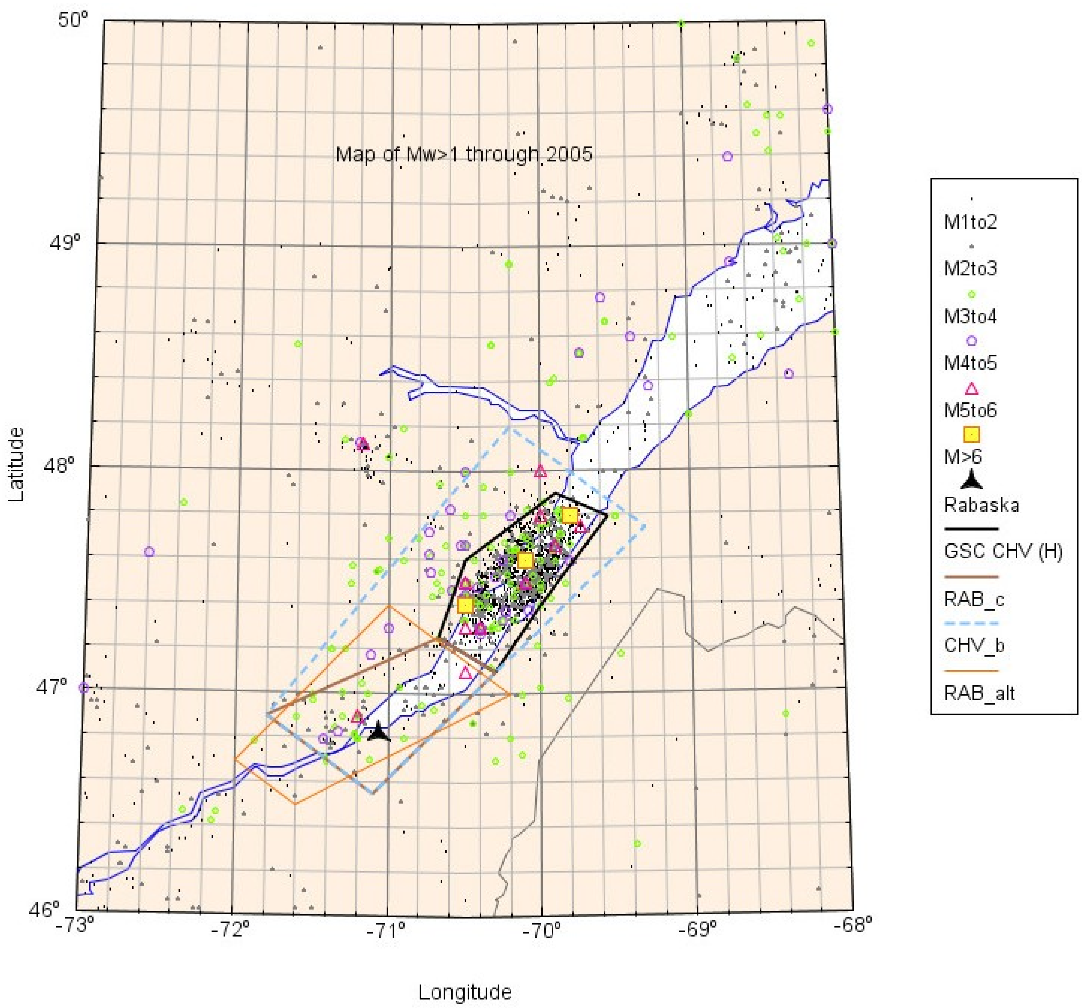

*Figure 3 – Recorded seismicity (**M**>1) through 2005 along with the alternative source zone models defined to represent uncertainty in seismic source zonation. Three combinations are considered: (Model A) (CHV-H + RAB_c); (Model B)) (CHV_b); (Model C)) (CHV-H + RAB_alt).*

2.2.2 Ground-Motion Relations

Uncertainties in the ground motion relations are often the most important uncertainty in a seismic hazard analysis. They are assessed by considering three alternative sets of ground-motion relations. The first is the Atkinson and Boore (1995) relations used in the 2005 national seismic hazard maps. These relations were based on a stochastic point-source model of ground motion, with the parameters calibrated using regional seismographic data. More recent relations are also included. The Hybrid-Empirical relation of Campbell (2003) is used to consider the implications of this ground-motion model, which is based on making suitable modifications to strong-motion relations from other data-rich regions such as California. An updated relation by Atkinson and Boore (2005) is also included; this relation uses a stochastic finite-fault model of ground motions, incorporating new data on attenuation and source parameters that has been gathered in the last 10 years. Figure 4 shows these alternative relations. All relations shown are defined for the horizontal component for hard-rock site conditions (near-surface shear-wave velocity ≥ 2000 m/s), which is the standard reference condition for ground-motion relations in eastern North America (ENA). However, it is known that the average near-surface velocity at Rabaska is about 800 m/s (Terratech, 2006); this corresponds to the boundary between NEHRP B and NEHRP C conditions. Therefore, following the hazard computations for hard rock, all results will be converted to B/C boundary conditions, using a procedure discussed Section 3.2. In the hazard calculations, all relations are converted to use the hypocentral distance measure for consistency with the seismic hazard software. The implications of the alternative relations are displayed to show sensitivity, and they are weighted (assuming equal weights for each) to produce mean-hazard results; this is typical practice to handle uncertainty in the ground-motion relations.

Other sources of uncertainty include those in the maximum magnitude and in the recurrence parameters. The sensitivity to these parameters is less important, as will be shown later in the report.

In summary, the analysis in this report fully incorporates *random variability* in earthquake locations and ground motions. *Model uncertainty* is incorporated by examining the sensitivity of results in order to define the key uncertainties: these are the uncertainty in seismotectonic model for the site source region and the uncertainty in ground-motion relations. For these key parameters, several alternative models are defined and the implications for the UHS at specified probability levels are determined.

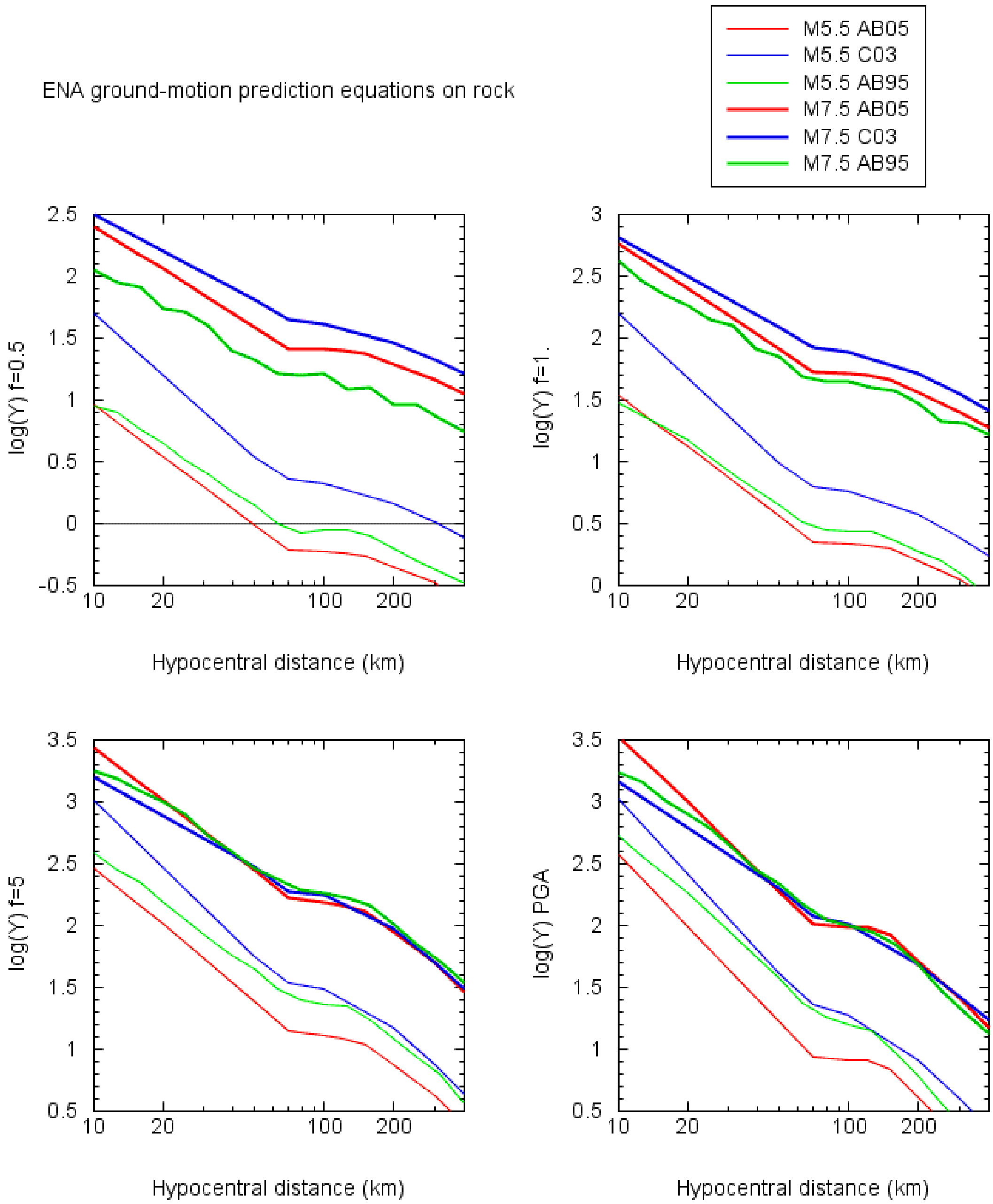

Figure 4 – Comparison of alternative ground-motion models used in seismic hazard analysis for PSA at f=0.5, 1, 5 Hz, and PGA (AB95=Atkinson and Boore, 1995; C03=Campbell, 2003; AB05=Atkinson and Boore, 2005). All relations converted to hypocentral distance. All for NEHRP A.

2.3 Input Parameters for Seismic Hazard Analysis

The input parameters for the seismic hazard analysis include the seismic source zonation, the magnitude recurrence parameters and maximum earthquake magnitude for each source zone, and the ground motion relations for response spectra at several vibration frequencies and PGA.

2.3.1 Seismic source models

Figure 3 shows the alternative seismic source models, based on clusters of historical seismicity and their uncertainty, along with the regional seismicity data as obtained from the Geological Survey of Canada through 2005 (www.seismo.nrcan.gc.ca). Three combinations are defined in this study: (Model A- confined seismicity) = GSC_H + RAB_c; (Model B – broad seismicity) = CHV_b; and (Model C- alternative confined model) = GSC_H + RAB_alt. We also consider a fourth model (Model D), based on the IRM zone defined by the GSC (Figure 1). The first of these models is preferred based on the historical seismicity and location of rift faults along the St. Lawrence.

The magnitude scale currently used in the GSC catalogue is the Nuttli magnitude scale (MN). The moment magnitude scale, **M**, was used in this study, because the ground motion relations are given in terms of moment magnitude. (Note: moment magnitude is similar to the more familiar "Richter magnitude" that is often used to describe the size of events in California.) For events with no moment magnitude determination, a conversion was made from Nuttli magnitude using the relation of Atkinson and Boore (1995) for ENA, or from local magnitude (for older events for which no MN is available) via an empirical relationship derived from data for southeastern Canada. These relations are:

$$\mathbf{M} = -0.39 + 0.98\ \mathrm{MN}$$
$$\mathbf{M} = 0.800 + 0.838\ \mathrm{ML}$$

For small to moderate events, the moment magnitude tends to be about 0.5 units less than the Nuttli magnitude for the same event. For example, events with MN of 3.5 have a moment magnitude of 3.0. The 2005 Riviere du Loup, Quebec earthquake had an MN of 5.4, and a moment magnitude of **M**5.0. The events of Figures 1 and 3 are plotted in terms of their moment magnitudes. All known events of **M**>1 are plotted, although the catalogue is not complete for the smaller events.

2.3.2 Magnitude Recurrence Relations

Recurrence data, expressing the relative frequency of occurrence of earthquakes within a zone as a function of magnitude, can generally be fit to the Gutenberg-Richter relation:

$$\mathrm{Log}\ N(M) = a - b\ \mathbf{M}$$

where N(M) is the number of events per annum of magnitude ≥**M**, **M** is moment magnitude, and *a* and *b* are the rate and slope of the relation. In most parts of the world, *b* values are in the range from 0.8 to 1., while *a* values vary widely depending on the activity level of the region.

The magnitude recurrence relations obtained for the source zones of Figure 3 are shown in Figures 5 through 7 (Models A to C, respectively). The recurrence relation for the IRM model of the GSC (Model D, Figure 1) is also shown on Figure 6. In developing these relations, uneven completeness of the catalogue was accounted for. This was accomplished by estimating the annual rate for events of different magnitudes separately, using, for each magnitude, seismicity data for the time period for which reporting of those data is complete. These completeness intervals are as follows:

Region	Year to begin statistics for:					
	M2	**M**3	**M**4	**M**5	**M**6	**M**7
St.Lawrence	1982	1920	1860	1810	1810	1810

Thus the annual rate of **M**3 events is based on just the last few decades, while the annual rate of **M**5 events considers all events from the early 1800's.

The minimum magnitude for the hazard calculations is **M**5.0, as smaller events do not cause damage to well-engineered structures. The maximum magnitude (Mx) is generally assumed to be in the range from **M** 7.0 to 7.5, based on global studies of maximum magnitudes for similar tectonic regions (Johnston, 1996). Johnston noted that 7.0 is the largest magnitude observed globally for unrifted stable continential interior shield regions such as those outside the St. Lawrence Valley. For rifted areas, maximum magnitudes are higher. Results are not very sensitive to this choice, as shown below. A value of Mx=7.5 is chosen for all zones, as they all include Iapetan rift faults. The largest events in eastern Canada have had **M** of about 7.2 (eg. 1929 Grand Banks earthquake); those in the St. Lawrence Valley have not exceeded **M** 7 within the period of historical record (for example, the 1925 Charlevoix earthquake had **M**=6.4; Bent, 1992).

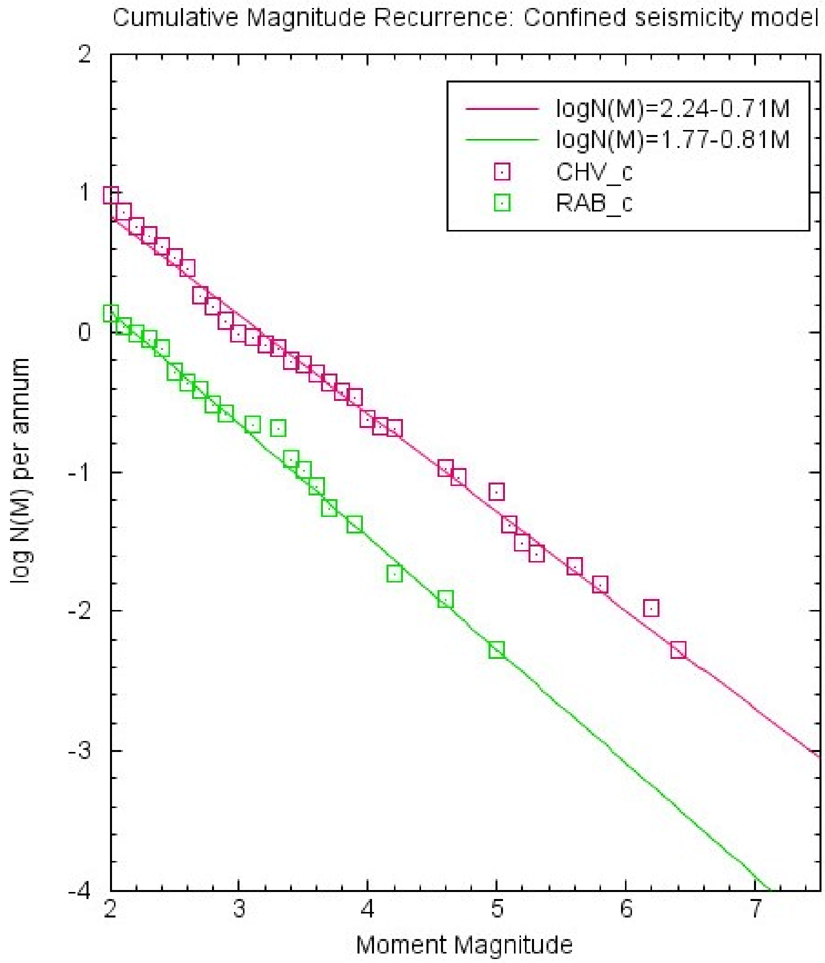

Figure 5 – Recurrence Relations for Confined Source Zone Model A (CHV_c=Charlevoix, confined; RAB_c=Rabaska, confined).

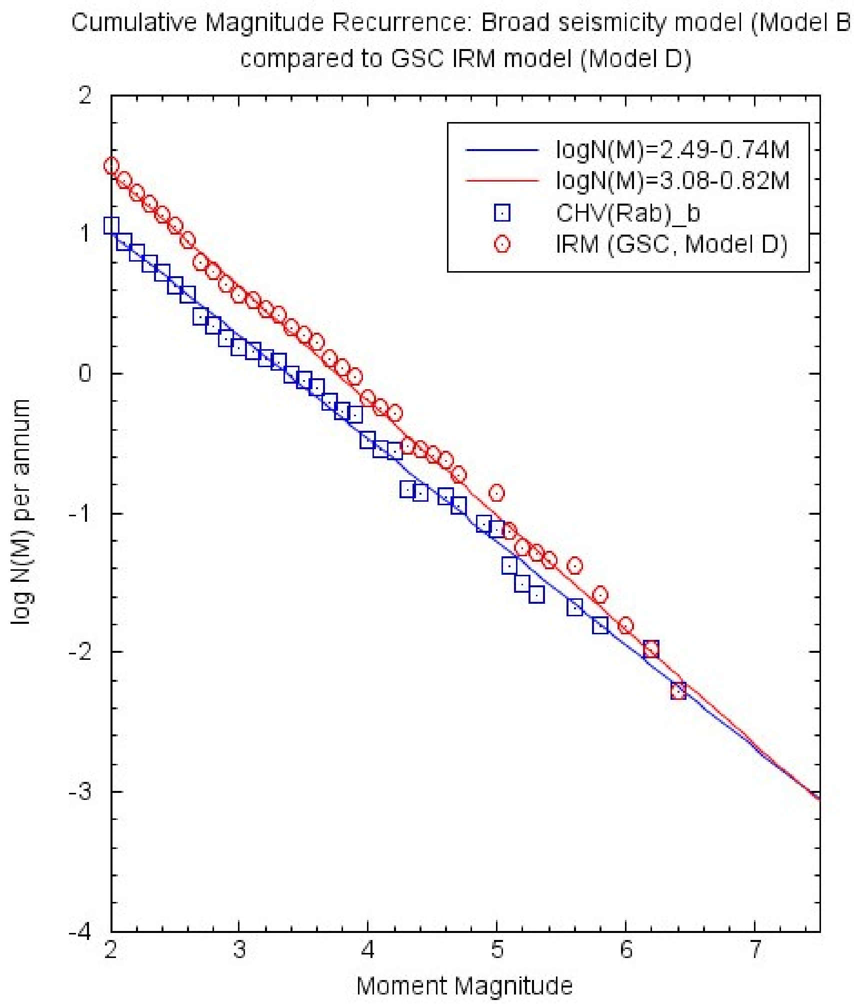

Figure 6 – Recurrence Relations for Broad Source Zone Model B and the GSC IRM source Model D. (CHV(Rab)_b=Charlevoix, broad)

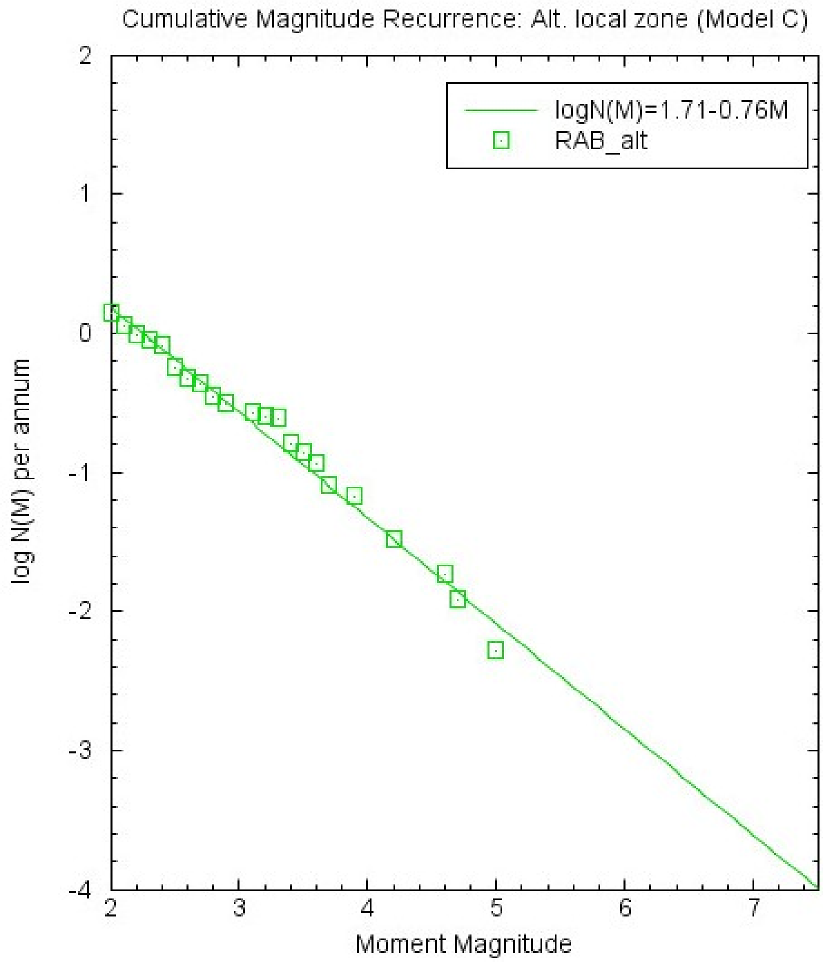

Figure 7 – Recurrence Relation for alternative local zone RAB_alt used in Confined Model C.

For each model, the appropriate source geometry as shown in Figure 3 (or 1) is applied, with the associated recurrence relations for each zone of the model, as shown in Figures 5 to 7; contributions to hazard are integrated from **M**=5.0 to **M**=7.5.

2.3.3 Ground motion relations

Three alternative sets of ground motion relations are adopted as described in Section 2.2. These include the Atkinson and Boore (1995) relations, the Campbell (2003) Hybrid Empirical relations, and the Atkinson and Boore (2005) relations; the relations are equally weighted. All relations are for hard-rock sites in eastern North America. All have been converted to equivalent relations for hypocentral distance for consistency with their application in the seismic hazard computations (see EPRI, 2004). They provide PGA, PGV and response spectra (5% damped pseudo-acceleration) for the random horizontal component of motion, on bedrock, as a function of moment magnitude and distance from the earthquake source. These relations have been validated against the eastern ground motion database (Atkinson and Boore, 1995; 2005). The Atkinson and Boore (1995) relations are those adopted in the GSC calculations for the national seismic hazard maps (Adams and Halchuck, 2003), whereas the Campbell (2003) and Atkinson and Boore (2005) relations include more recent information. Random uncertainty in the relations was modeled by a lognormal distribution of ground motion amplitudes about these median relations, with a standard deviation of 0.25 log (base 10) units for high frequencies, increasing to 0.30 units at low frequencies. This random uncertainty is consistent with recent studies (eg. Atkinson and Boore, 1995; EPRI, 2004).

It should be noted that the ground motion relations apply to hard rock sites (eg. shear-wave velocity>2000 m/s). Shear-wave velocity studies at Rabaska suggest an average shear-wave velocity of about 800 m/s in the near-surface (Terratech, 2006). Thus the resulting motions need to be modified from NEHRP A to NEHRP B/C boundary conditions. This modification will be performed on the hard-rock results as described in Section 3.2.

3 - RESULTS OF SEISMIC HAZARD ANALYSIS

3.1 Sensitivity to Input Assumptions

Using the input parameters given in the previous section, the PGA, PGV and response spectra were computed for a range of probabilities using the Cornell-McGuire method. The values of PGA and PSA (5% damped), for the horizontal component of motion on hard rock for these probabilities are displayed in a number of plots. The UHS is for hard-rock site conditions (shear-wave velocity near surface > 2000 m/s), and will subsequently be modified for the local NEHRP B/C conditions.

The peak ground acceleration (PGA) is plotted for reference at a frequency of 100 Hz, but the shape of the curve between 40 Hz and 100 Hz is arbitrary (no spectral values were calculated for frequencies above 40 Hz). The PGA refers to the maximum acceleration of the ground shaking during the seismic event (ie. the peak amplitude on a free-field record of ground acceleration versus time) – it does not have an actual associated frequency, as the frequency at which the PGA occurs will depend on the earthquake magnitude and distance. The response spectrum shows the maximum acceleration of a damped single-degree-of-freedom oscillator, when subjected to the input record of ground acceleration versus time. Oscillators with a high natural frequency will respond to input ground motions that are rich in high frequency content, while oscillators with low natural frequency will respond more strongly to input ground motions that are rich in low frequency content.

The sensitivity of results to alternative sets of input parameters is shown in Figures 8 to 10, for a probability level of 2% in 50 years (0.0004 per annum); this is the probability level used in the 2005 national seismic hazard maps and specified for the SSE in recent LNG codes (CSA and NFPA). Figure 8 shows the Uniform Hazard Spectrum (UHS) at this probability level for the three alternative ground-motion models, using the Confined seismicity model in each case (Model A); this illustrates sensitivity to the ground-motion relations. Figure 9 shows the UHS for the alternative source zone models, using the AB95 ground-motion relations in all cases (to show sensitivity to source zone model). Figure 10 illustrates the sensitivity of the results to the parameters of the recurrence relations (slope b and rate a of the Gutenberg Richter relation, and maximum magnitude), for the Confined seismicity Model A (AB95 relations). This was evaluated by considering the implications of the following cases, which are considered reasonable given the uncertainty in actual activity rates and in the magnitude conversions used to obtain them: (i) double the calculated rate of $\mathbf{M}\geq 5$ in the local source zone, using a fixed regional b-value (based on GSC IRM model) of 0.8 (Mx=7.5); (ii) use the observed rate of $\mathbf{M}\geq 5$, but with a shallower slope, of b=0.7 (Mx=7.5); (iii) use the best-estimate recurrence parameters, but with Mx=7.0. The results shown on Figures 8 to 10 confirm that the most important parameters are the seismic source zone model and the ground-motion relations. Note that the uncertainty in source model, as indicated in Figure 9, effectively includes uncertainty in the recurrence relations, as the different source models imply different seismicity rates.

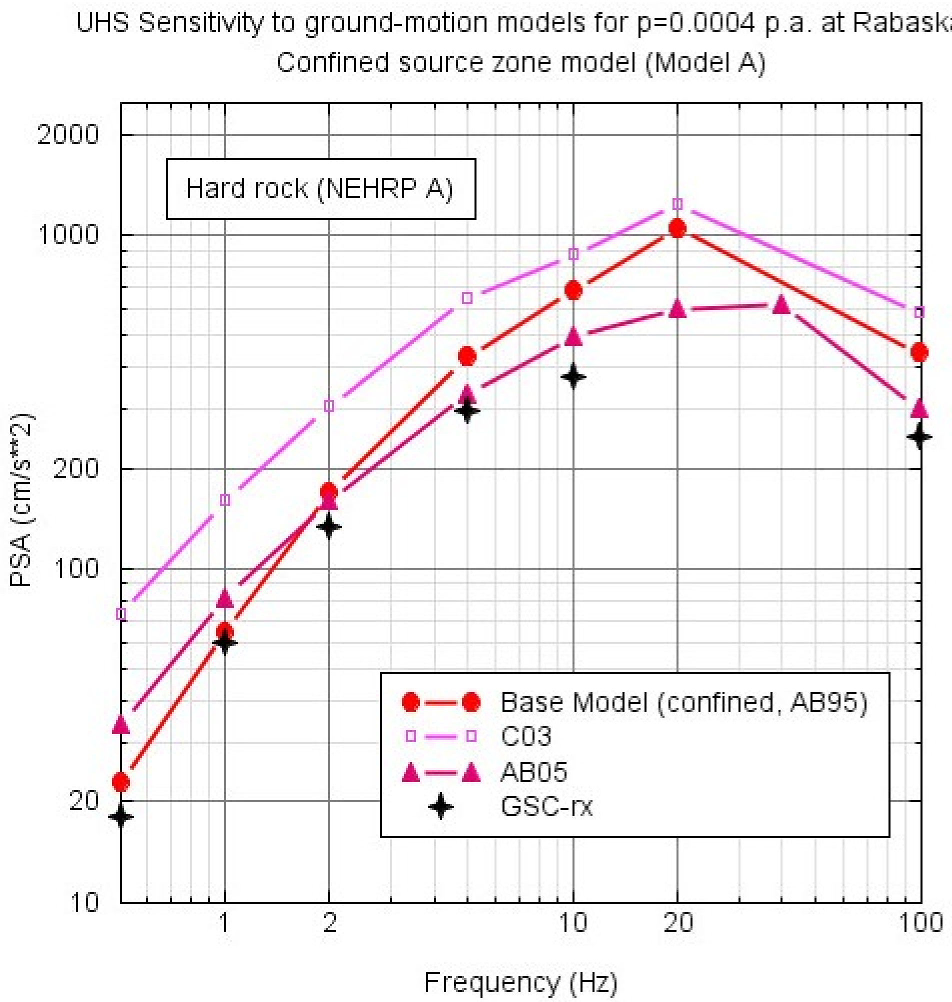

Figure 8 – Sensitivity of UHS for 2% in 50 year probability to alternative ground-motion relations, assuming the Confined seismicity model (Base Model=Confined AB95, EPRI Hybrid Empirical, Atkinson and Boore, 2005). GSC "robust model" results are also shown. All for NEHRP A.

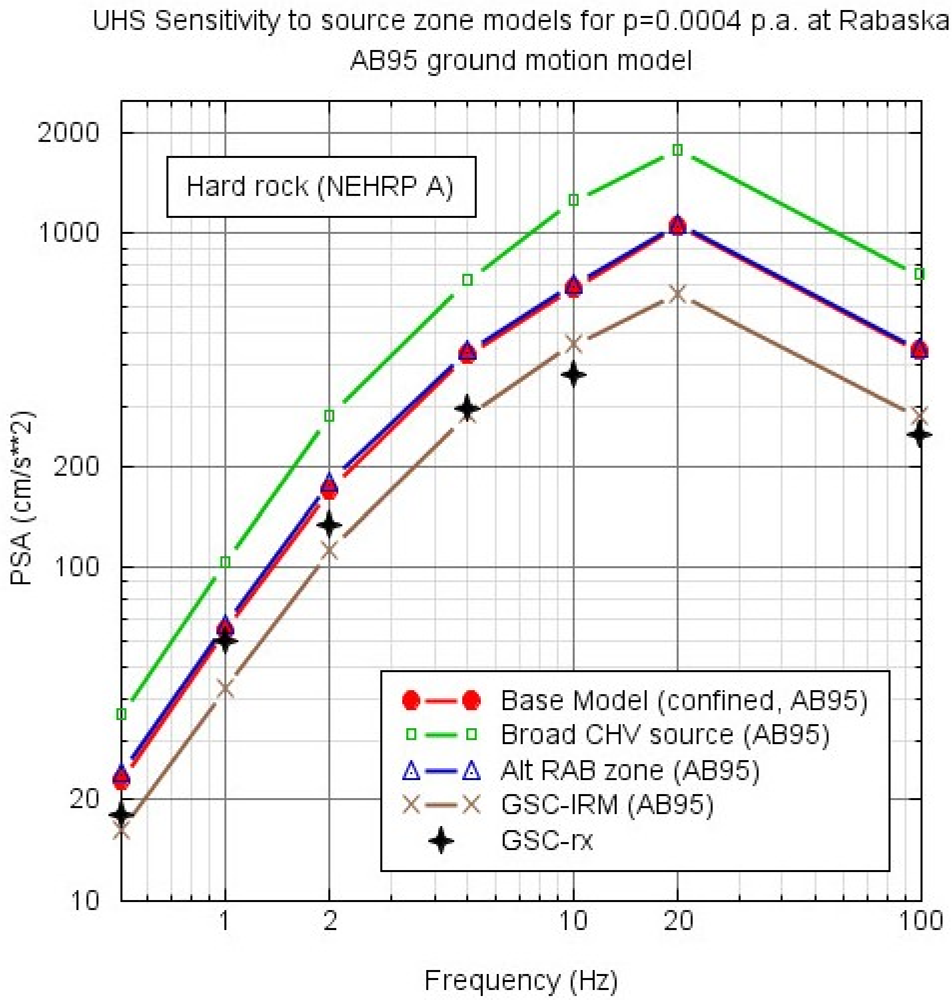

Figure 9 – Sensitivity of UHS for 2% in 50 year probability to source model (Base Model=Confined Model A with AB95; Broad CHV=Model B; Alt RAB=Model C; GSC-IRM=Model D). GSC "robust model" results from national seismic hazard maps are also shown. All for NEHRP A.

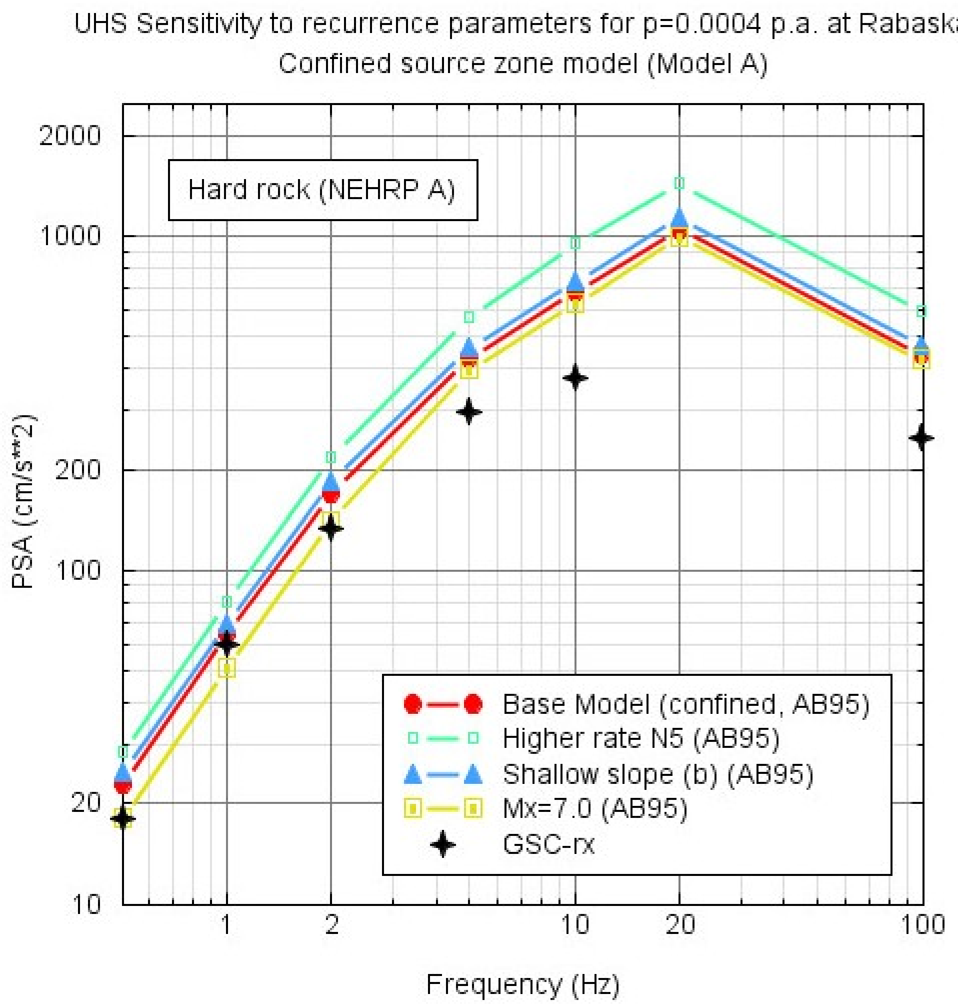

Figure 10 – Sensitivity of UHS for 2% in 50 year probability to recurrence parameters including maximum magnitude (for Base model A) and seismicity recurrence parameters. GSC "robust model" results are also shown. All for NEHRP A.

To provide insight on what types of events correspond to the UHS at low probabilities, Figure 11 compares the Model A UHS (for AB95 ground-motion relations) to median+σ response spectra and PGA predicted by the Atkinson and Boore (2005) ground-motion relations. The median+σ is used for the comparison as hazard contributions tend to be dominated by events with amplitudes about one standard deviation above the median. The UHS for an annual probability of 0.0004 (1/2500) is approximately matched at low frequencies by an event of **M**7 at 70 km, corresponding to a large event within the Charlevoix seismic zone. At high frequencies, the UHS is approximately matched by an event of **M**6 at 20 km, corresponding to a moderate local earthquake. This local event could occur on any of the many buried rift faults in the area, most likely at depths of 10 km or greater.

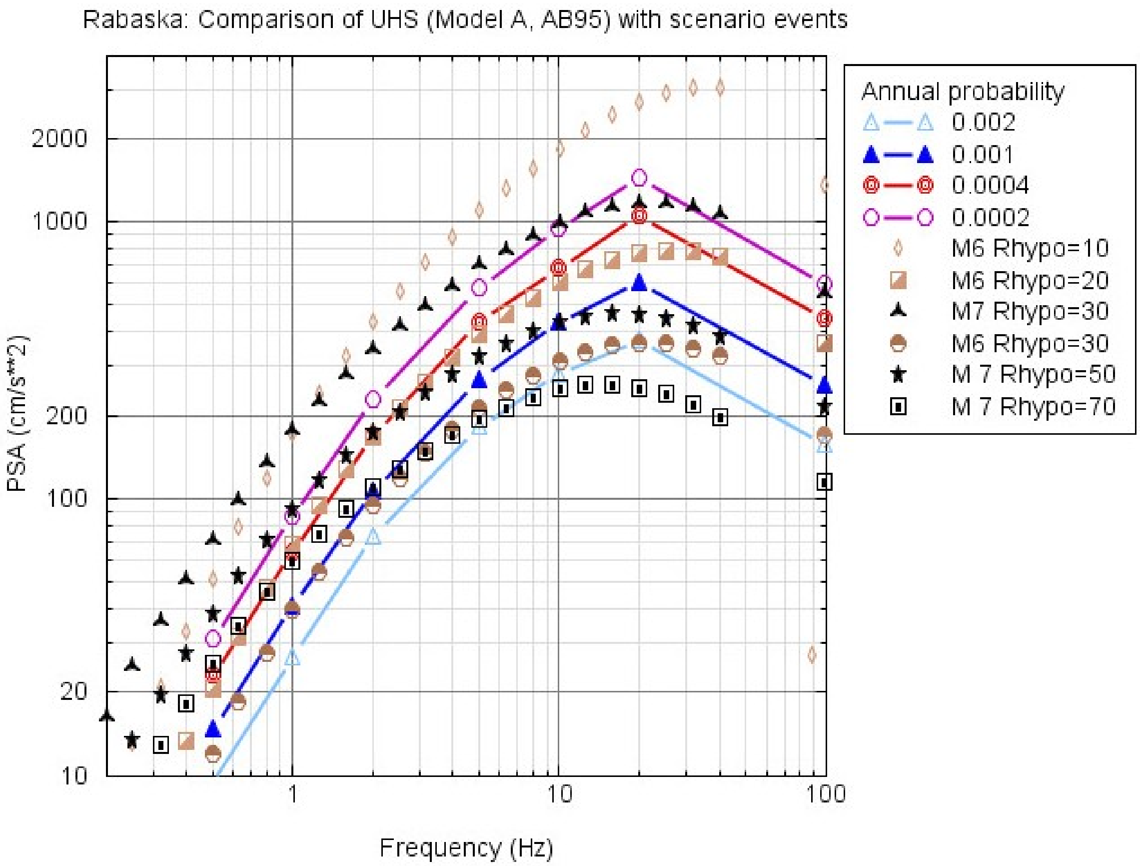

*Figure 11 – Comparison of Rabaska UHS for Model A (AB95) to median plus sigma predicted ground motions for **M**6 to 7 events according to Atkinson and Boore (2005).*

3.2 Conversion of Results from NEHRP A to NEHRP B/C

The seismic hazard computations were performed for hard-rock site conditions (NEHRP A, with near-surface shear-wave velocities > 1500 m/s), as most of the ENA ground-motion relations are only available for this site condition. The recent relations of Atkinson and Boore (2005) are provided as separate equations for two site conditions: hard-rock (NEHRP A) and the NEHRP B/C boundary (shear-wave velocity 760 m/s). By taking the ratio of the response spectra for NEHRP B/C to that for NEHRP A, the dependence of the site amplification on magnitude and distance may be evaluated. This is shown in Figure 12. The site amplification has a weak dependence on distance and magnitude, except for PGA, for which the distance dependence is strong. (This is a consequence of the changing frequency content of PGA with distance.)

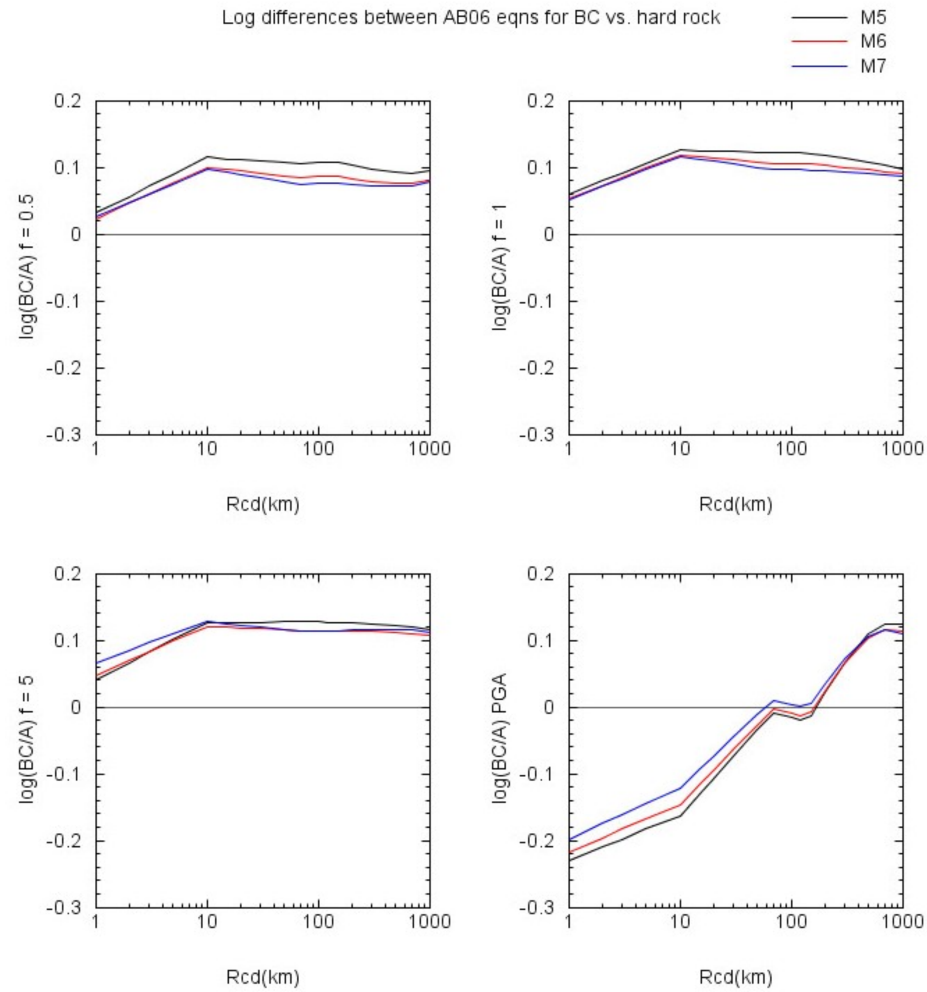

Figure 12 – Log$_{10}$ of the ratio of predicted ground motions for NEHRP BC boundary (760 m/s) to that for NEHRP A (>1500 m/s), based on Atkinson and Boore (2005). Ratio is shown for frequencies of 0.5, 1 and 5 Hz, and for PGA, for magnitudes 5, 6 and 7, as a function of closest distance to the fault.

To accurately model the implications of the site amplification, it is best to perform the seismic hazard analysis directly for the site conditions of interest. Since this can only be done for the AB05 relations (as the others are not available for B/C boundary), the following approach is adopted. The hazard is calculated at Rabaska, using Model A and the AB05 ground-motion relations, for both NEHRP A and B/C boundary. We then take the ratio of the calculated UHS ground motion, at several probability levels covering the complete range of interest, to determine the net effect of the site amplification at Rabaska on the UHS. As shown on Figure 13, the amplification factor depends only weakly on the probability of the ground motion. A smoothed curve that is a good representation of the amplification for all probabilities of interest is therefore adopted as the B/C amplification factor (black line on Figure 13). This function results in amplification, by as much as a factor of 1.4, over most frequencies. At very high frequencies (>10 Hz), and for PGA, there is actually a de-amplification (factor<1), due to the high-frequency energy absorption of the softer rock materials in the near surface.

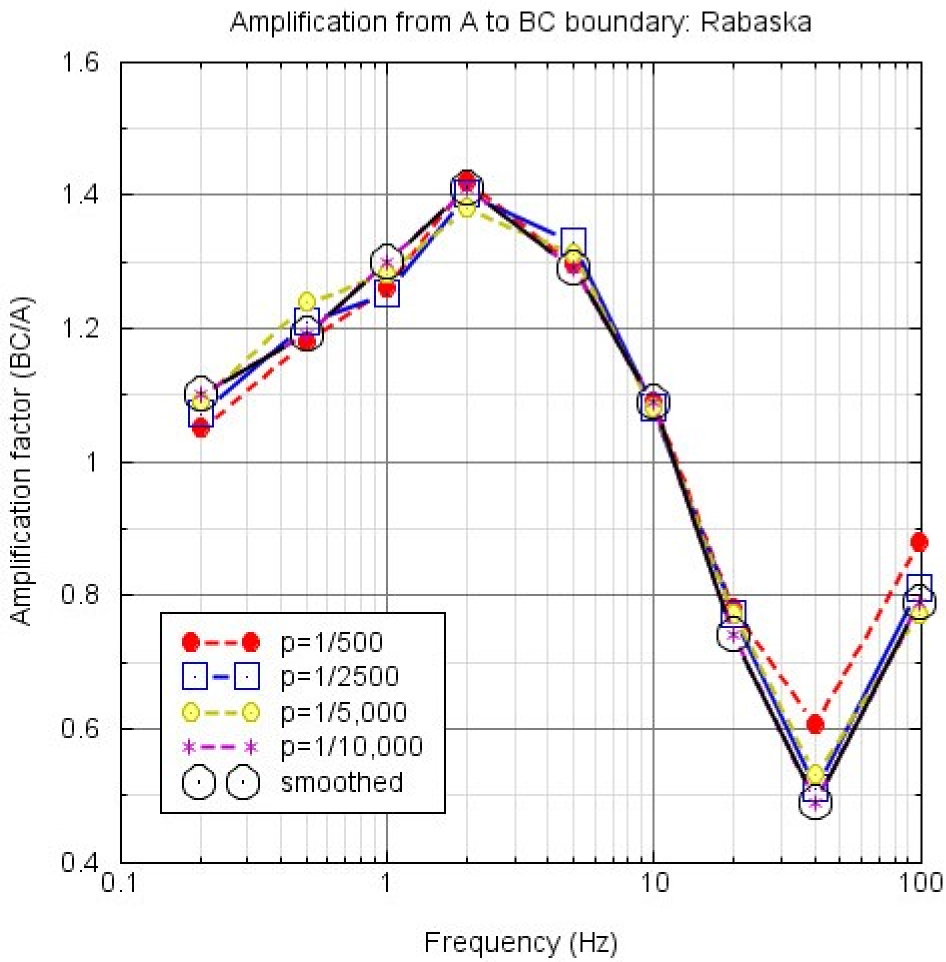

Figure 13 – Factors to convert hazard results for hard rock (NEHRP A) to results for B/C boundary, as calculated from the ratio of results for the AB05 relations for hard rock to the results for the AB05 relations for B/C boundary (under Model A). The smoothed valued (black dots) are adopted to convert the results for all models, over a wide range of probabilities.

To provide UHS results for the B/C boundary conditions on which the tanks will be situated, all UHS results computed for NEHRP A are multiplied by the smoothed factors shown in Figure 13, and listed in Table 1. For facilities to be located on soil, the B/C motions need to be further amplified for the overlying soils, based on a site-specific soil response analysis.

Table 1 – Amplification Factors at Rabaska for UHS ground-motions for B/C boundary, relative to computed results for NEHRP A.

Frequency (Hz)	Amplification Factor (BC/A)
0.2	1.1
0.5	1.2
1.0	1.3
2.0	1.4
5.0	1.35
10.	1.1
20.	0.8
40.	0.6
PGA	0.9

Figure 14 shows the B/C boundary (soft rock) 1/2500 UHS at Rabaska for the base case estimate (Model A, AB95 ground-motion model), along with the range of estimates that is obtained by taking the minimum and maximum of the UHS values for the 4 source models, and 3 ground-motion models (eg. minimum and maximum values for the 12 cases). The high variability in the range of estimates is similar for other probability levels (eg. 1/1000, 1/5000).

A weighted-mean-hazard result is also shown on Figure 14. This is derived by weighting the probabilities of exceedence of each analyzed ground-motion amplitude across the hazard cases considered, to obtain a weighted-mean-hazard UHS. The weights considered are an interpretation of the relative likelihood of each alternative, as follows:

Source Models:

Model A (Rabaska confined)	0.5
Model B (Broad Charlevoix)	0.1
Model C (Rabaska alternative)	0.3
Model D (GSC IRM)	0.1

Ground Motion Models:

Atkinson and Boore (1995)	0.33
Campbell (2002)	0.33
Atkinson and Boore (2005)	0.34

Figure 15 shows the corresponding plot for the B/C boundary 1/500 UHS. All of the estimates have been converted to B/C boundary using the factors of Figure 13. Also shown in the GSC estimate for “firm ground” conditions (NEHRP C). Note that while the GSC estimate for Rabaska appears to be very unconservative for hard-rock conditions in relation to the results of this study (Figure 9), when we consider the results for soft rock the discrepancy between this study and the GSC estimate is reduced. This is because the GSC results included a very conservative conversion from NEHRP A to NEHRP C, which counteracts the unconservatism in their source model.

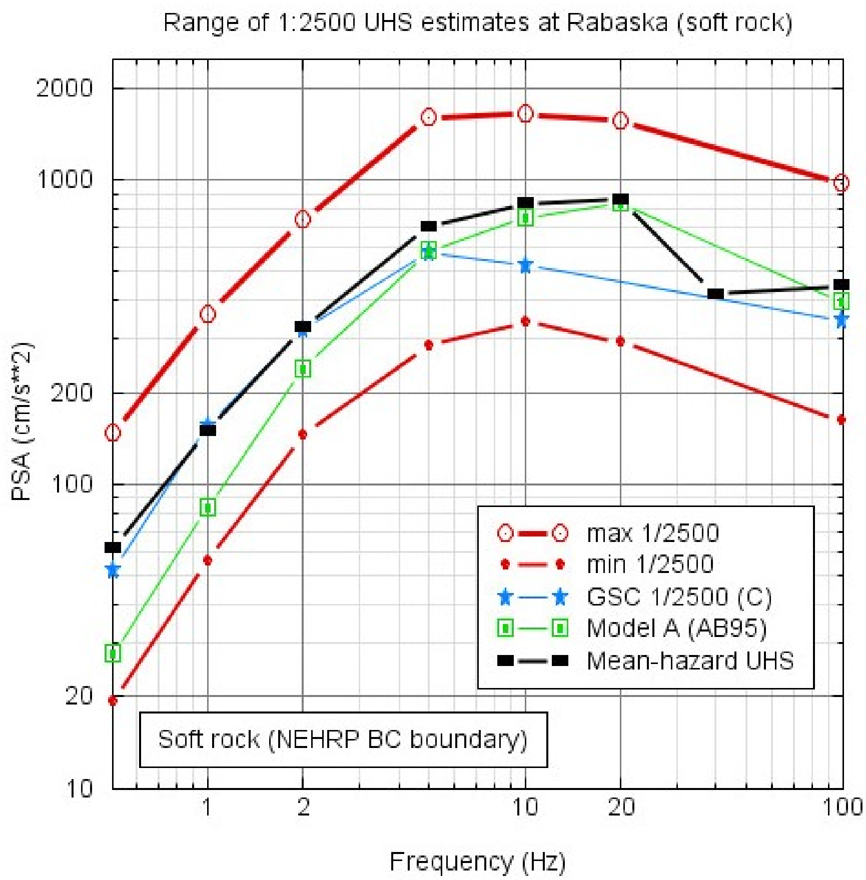

Figure 14 – UHS at Rabaska converted to B/C boundary conditions, for 1/2500 p.a. Red lines show minimum and maximum estimates from the 3 source models, considering the 3 ground-motion models. Green line shows base-case estimate for Model A, AB95 ground-motion relations. Blue line shows GSC value for NEHRP C. Black line is weighted mean-hazard UHS.

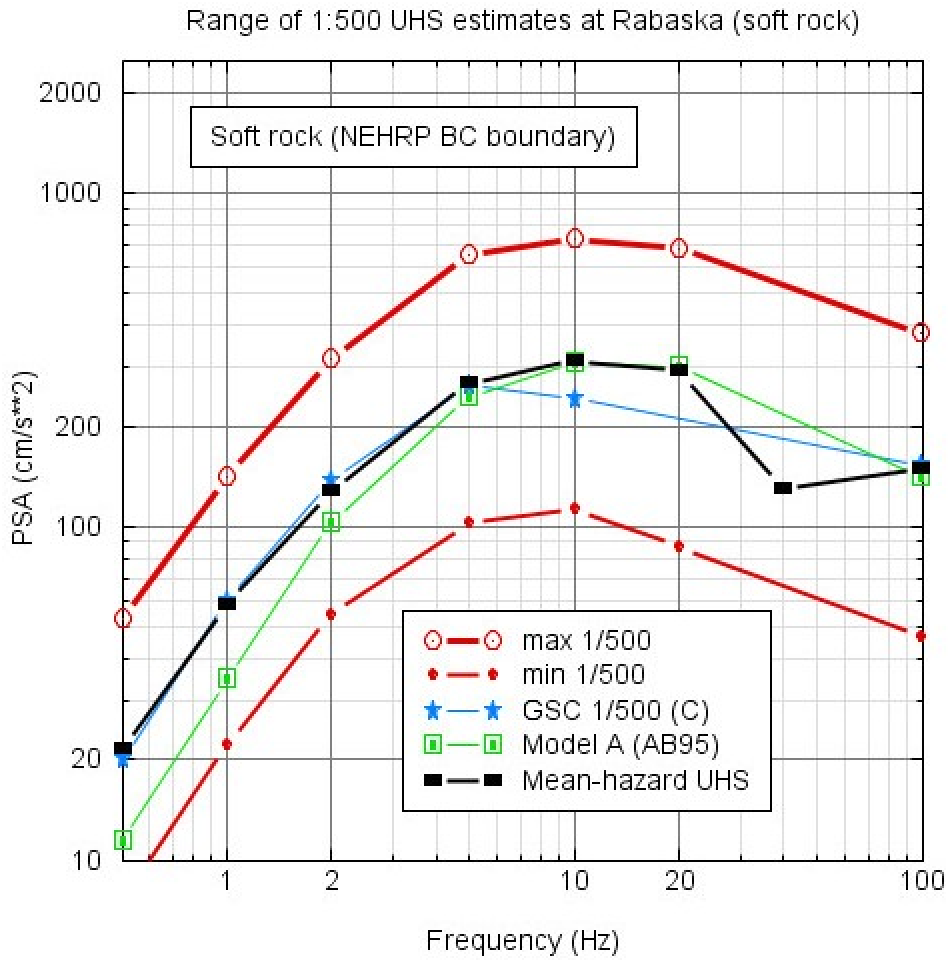

Figure 15 – UHS at Rabaska converted to B/C boundary conditions, for 1/500 p.a. Red lines show minimum and maximum estimates from the 3 source models, considering the 3 ground-motion models. Green line shows base-case estimate for Model A, AB95 ground-motion relations. Blue line shows GSC value for NEHRP C. Black line is weighted mean-hazard UHS.

On Figures 14 and 15 it is noted that the range of plausible estimates, considering the full range from minimum to maximum, is very large. It represents more than a factor of two about the mean estimate. As shown earlier, the primary contributors are uncertainty in the seismic source models and in the ground-motion relations. Both the upper and lower end of the range represents combinations that are relatively unlikely, due to the use of a low-likelihood source model (the broadened Charlevoix zone or the IRM model). A better estimate of the most likely motions can be obtained by weighting the various inputs to the models and obtaining weighted-mean-hazard UHS ground motions, as illustrated by the black lines in Figures 14 and 15.

The figures above are provided to illustrate the UHS for two example probability levels: 1/500 and 1/2500. Computations have been performed for a wider range of annual probabilities, including 1/500, 1/1000, 1/2500 and 1/5000. Table 2 provides weighted-mean-hazard UHS ground motions, using the weights provided above, for Rabaska for B/C boundary site conditions, for each of these probability levels.

Table 2 – Weighted-Mean-Hazard Ground Motions for Rabaska, for 5% damped horizontal-component PSA, PGA (cm/s^2) and PGV (cm/s), for B/C boundary site conditions, for a range of annual probabilities

Frequency(Hz)	1/500	1/1000	1/2500	1/5000
0.1	0.85	1.5	2.6	3.9
0.2	3.1	5.5	9.4	14
0.5	22	34	61	86
1	59	88	149	215
2	127	200	327	472
5	270	422	702	986
10	315	499	831	1222
20	294	471	858	1188
40	130	227	421	665
PGA	149	250	446	630
PGV	4.7	7.3	11.7	16.9

3.3 Vertical Component Motions

The UHS were derived for the horizontal component of motion. For some analyses, the vertical component of motion is also required. The vertical UHS may be obtained by applying the factors (V/H) as listed in Table 3 to the corresponding horizontal-component UHS. These are empirically-derived factors for rock sites in eastern Canada, based on analysis of seismographic data (Siddiqqi and Atkinson, 2002). The V/H factors should be applied to the spectra for B/C boundary conditions.

Table 3 – Vertical-to-Horizontal component spectral ratio, for ENA rock sites

Frequency(Hz)	V/H ratio
≤0.5	1.
1.0	0.88
2.0	0.82
5.0	0.74
≥10.	0.71
PGA	0.71

3.4 Long-Period Motions

The expected motions for periods as long as 10 seconds (frequency 0.1 Hz) are required for some analyses for the LNG tanks (eg. sloshing is long-period behaviour).

ENA ground-motion relations do not provide predictive equations for periods longer than 5 sec. However, the simulations performed by Atkinson and Boore (2005) for their most recent ENA ground-motion relations considered periods as long as 10 seconds. On Figure 16, the ratio of predicted response spectra for 0.2 Hz (5 sec) to that for 0.1 Hz (10 sec) is plotted for large events, in the magnitude-distance range that dominates the hazard for long-period motions. The ratio is approximately independent of distance, and has only a weak dependence on magnitude in the relevant magnitude range (6.5 to 7.5). The mean $\log_{10}$ ratio is 0.65±0.11 for **M**6.5, decreasing to 0.50±0.19 for **M**7.5. For this study, we take the mean ratio calculated for **M**7 to 7.5 (at ≤100 km), which is 0.56±0.18 in log units, as being the best estimate. Estimated long-period motions can thus be obtained from computed UHS motions at 0.2 Hz by multiplying the 0.2 Hz PSA by the factor 0.275 ($=1/10^{0.56}$). This ratio was used to compute the 0.1 Hz motions provided in Table 2, based on the results for 0.2 Hz.

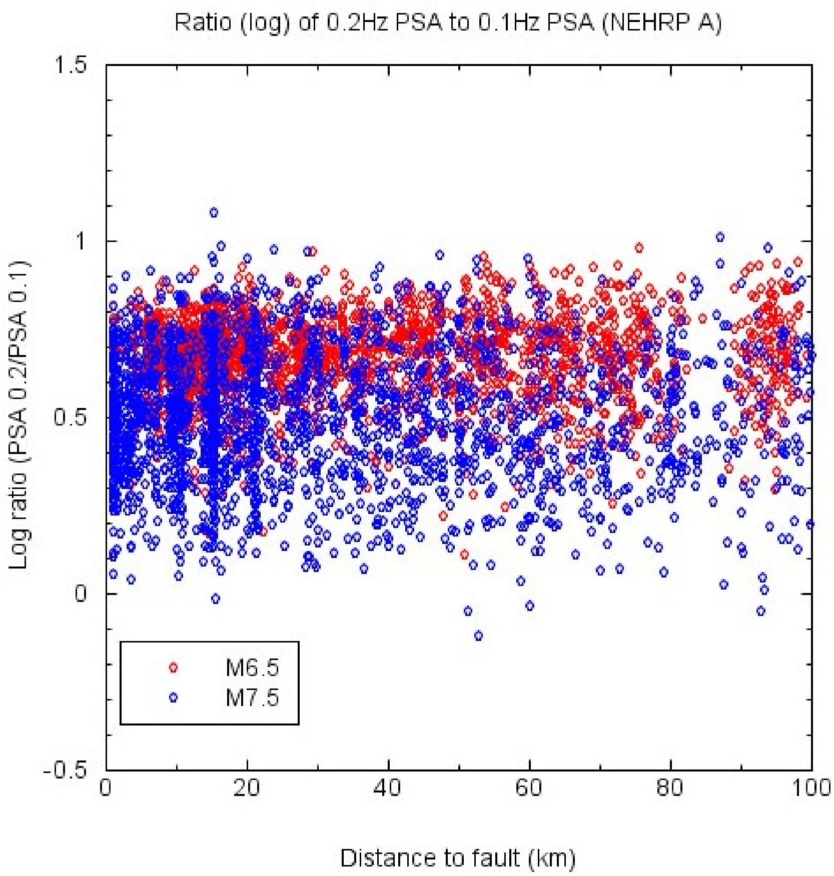

*Figure 16 – Log ratio of PSA for 0.2Hz to PSA for 0.1 Hz, for **M**6.5 to 7.5, based on Atkinson and Boore (2005) simulations for hard rock.*

3.5 Results for other damping levels

The UHS results presented in the preceding have been for a damping level of 5% of critical. The corresponding results for other levels of damping may also be required. These may be obtained by multiplying the ground-motion values for 5% damping by the following factors (Table 4); these factors were obtained for rock sites in eastern Canada by Atkinson and Pierre (2004), for frequencies in the range from 0.5 to 20 Hz. For frequencies <0.5 Hz, the 0.5 Hz values are adopted, while for frequencies >20 Hz, the 20 Hz values are adopted.

Table 4 – Multiplicative factors to convert UHS results for 5% damping to other damping levels (Atkinson and Pierre, 2004).

Frequency(Hz)	**0.5%**	**1%**	**2%**	**3%**	**7%**	**10%**	**15%**
≤0.5	1.350	1.266	1.174	1.103	0.922	0.835	0.736
0.8	1.450	1.361	1.231	1.134	0.908	0.810	0.701
1	1.550	1.414	1.260	1.148	0.901	0.793	0.679
1.3	1.675	1.469	1.289	1.163	0.894	0.777	0.657
2	1.800	1.576	1.332	1.187	0.880	0.758	0.627
3.2	1.950	1.685	1.385	1.208	0.872	0.743	0.614
5	2.100	1.765	1.420	1.226	0.862	0.734	0.599
7.9	2.200	1.841	1.442	1.234	0.860	0.729	0.597
10	2.300	1.871	1.456	1.240	0.860	0.729	0.598
13	2.350	1.902	1.471	1.246	0.860	0.729	0.600
≥20	2.400	1.950	1.485	1.249	0.858	0.730	0.606

3.6 Recommendations and Conclusions

This study has provided a range of estimates for expected ground motions at Rabaska, for a range of probability levels. There is large uncertainty in the estimates due to their sensitivity to the seismic source model and the ground-motion relations. There are two approaches that can be taken to deal with this uncertainty:

1. A logic tree approach can be used to weight the alternative models and obtain a mean-hazard UHS. As there are a limited number of uncertainties that are significant, a simple logic "shrub" based on the 12 alternative cases presented here captures most of the uncertainty. An illustration of this approach is provided in Figures 14 and 15, and the results given in Table 2 reflect this approach. The uncertainties could be defined in more detail to produce a full logic tree, that could be used to provide not just a mean-hazard UHS, but to more fully describe the distribution of results in terms of fractiles: median, 84th percentile, and so on. This would lead to an

improved description of the uncertainty in the results, but would not reduce the overall amount of uncertainty, nor greatly impact the estimate of the mean-hazard UHS.

2. Further work can be undertaken to reduce the uncertainty. The most promising approach for uncertainty reduction is to use paleoseismic investigations to try to determine whether the recurrence rates of large events in the site area (Rabaska zone of Figure 3) are significantly lower than those in the Charlevoix zone (also on Figure 3). Referring to Figure 5, the historical seismicity data implies that there is an order of magnitude difference in the recurrence rates of large events in these two regions. However, due to our uncertainty in the actual areal extent of the Charlevoix seismicity, we have effectively assigned a high uncertainty to this inference. If paleoseismic investigations of post-glacial (last 10,000 years) soils can establish that they have been repeatedly disturbed in the Charlevoix region, but not in the Rabaska region, then we could discount the hypothesis that the Charlevoix zone may extend to the Rabaska area. This would greatly reduce the uncertainty in seismic source modeling.

In an effort to reduce the actual uncertainty in the seismic hazard estimates, by refining our knowledge of the extent of the Charlevoix region of seismicity, an experienced paleoseismologist, Dr. M. Tuttle, has been retained by Rabaska to perform a comparative paleoseismic investigation of the Charlevoix and Rabaska regions. The results of this study are not yet available, but could result in a significant reduction in the UHS motions. Until this work is completed, the actual earthquake hazard analysis is considered preliminary. It will be repeated, and the ground motion values and conclusions revised accordingly, when the paleoseismic investigation results are available.

The final UHS defined for the OBE and SSE (B/C site conditions) may be used as input for modal analyses of structures on soft rock, as the input spectrum for soil response analyses for structures founded on soil, and as the target spectrum for the development of site time histories. Time histories appropriate to ENA soft-rock conditions should be developed considering both short-period and long-period hazard sources.

Table 5 provides a summary of the weighted-mean-hazard results (horizontal component), based on this study, in comparison to the GSC results used in the NBCC national seismic hazard maps (provided for 1/500 and 1/2500 p.a. probabilities). The complete study results for all probabilities are provided in Table 2.

Table 5 – Rabaska weighted-mean-hazard 5% damped PSA results (cm/s^2) for B/C boundary site conditions. NBCC UHS values (NEHRP Class C) at Rabaska are also shown.

Frequency(Hz)	**NBCC 1/500**	**This study 1/500**	**NBCC 1/2500**	**This study 1/2500**
0.1		0.85		2.6
0.2		3.1		9.4
0.5	20	22	51	61
1.	59	59	151	149
2	133	127	314	327
5	261	270	565	702
10	236	315	512	831
20		294		858
PGA	149	149	337	446
PGV		4.7	15	12

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ANNEXE B

Addendum au rapport géotechnique Terratech 2006

ADDENDUM TO GEOTECHNICAL SITE STUDY REPORT PHASE 3

SEISMICITY AND SOIL LIQUEFACTION POTENTIAL

RABASKA LIMITED PARTNERSHIP

Rabaska – LNG Receiving Terminal
Levis, Quebec

Our File :
T-1050-C (604238)

September 2006

Terratech

Rabaska – LNG Receiving Terminal
Levis, Quebec

Addendum to Geotechnical Site Study Report (Phase 3)
Seismicity and Soil Liquefaction Potential

Rabaska Limited Partnership

Our File T-1050-C (604238)
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29 September 2006

Mr. Roger Wild, Eng.
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SUBJECT: Addendum to Geotechnical Site Study Report (Phase 3)
Rabaska – LNG Receiving Terminal
Levis, Quebec
Seismicity and Soil Liquefaction Potential
Our File: T-1050-C (604238)

Dear Sir:

We are pleased to submit the enclosed Addendum Report that comprises information and comments on seismicity and soil liquefaction potential at the site of the Rabaska - LNG Receiving Terminal, in Levis, Quebec. This addendum report refers to the geotechnical data obtained at the project site and gathered in Terratech Report T-1050-C (604238) dated May 2006 "Geotechnical Site Study Report (Phase 3)".

This document is based on several seismic inputs gathered by Terratech from literature and subsurface investigation. The report also uses the seismic data supplemented in the Earthquake Hazard Analysis Preliminary Report issued in September 2006 by Mrs. Gail M. Atkinson, Ph.D., Engineering Seismologist.

We trust that the information included in this Addendum will be to your satisfaction. Please do not hesitate to contact us should you require additional information or should you have any questions.

TERRATECH
Division of SNC-Lavalin Environment Inc.

Raymond Bousquet, Eng., M.A.Sc.
Senior Geotechnical Engineer and Project Director

RB/ds

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1. INTRODUCTION

The services of Terratech**,** a Division of SNC-Lavalin Environment Inc., were retained by Rabaska Limited Partnership to carry out a Phase 3 geotechnical site study at the proposed Rabaska - LNG Receiving Terminal, located in Levis, Quebec. All of the geotechnical data obtained at the project site through on-site subsurface investigation works and laboratory testing are presented in Terratech Report T-1050-C (604238) dated May 2006 “Geotechnical Site Study Report (Phase 3)”.

Section 3.3 of the above report was specifically designated to cover and discuss the seismicity and the soil liquefaction issues of the Rabaska Site. However at the time of issue of the Geotechnical Site Study Report (Phase 3) on 23 May 2006, a specific site earthquake hazard assessment study was underway. This hazard study was since initiated and the following preliminary report was issued:

- Gail M. Atkinson 2006: “Earthquake Hazard Analysis: Rabaska LNG Facilities, Quebec, / Preliminary Report”, September 2006.

This Addendum Report addresses the issues of seismicity (earthquake history, local faults and fault activity and seismic hazard) and soil liquefaction potential.

2. SEISMICITY

2.1 Earthquake History

In the past, earthquakes strongly felt in the area of study had their epicenters located in the Charlevoix / Kamouraska and the Saguenay regions. Damaging earthquakes were recorded in 1663, 1791, 1860, 1870, and 1925. More recently, the Saguenay earthquake in 1988 was the most important. But from time to time, milder tremors with epicenters close to Quebec City have also shaken the region without causing any appreciable damage (Geoscape Quebec, 2006). In 1997, an earthquake occurred at Cap-Rouge, some 25 km southwest of the project site. Studies have demonstrated that this event was related to a reactivation of a deep fracture in the Canadian Shield.

Typically, the majority of earthquakes in eastern Canada are caused by sudden movements along ancient faults and are associated with the St. Lawrence River rift.

Additional information and comments related to earthquake historic data are provided in Atkinson, G.M. 2006 (specifically in Sections 2.2.1 and 2.3, pp. 7-10, and 13-14).

2.2 Local Faults and Fault Activity

The three following specialists were contacted by Terratech on the issue of fault activity:

- ***Dr Maurice Lamontagne***, Canadian Geological Commission, Ottawa, Ontario;
- ***Prof. Jacques Locat***, Département de géologie et de génie géologique, Université Laval, Quebec;
- ***Prof. Pierre-André Bourque***, Département de géologie et de génie géologique, Université Laval, Quebec.

From discussing with the above specialists, and in reference to Bourque web site documentation (see Section 6 - References), the following statements are provided:

- As stated in Section 3.2 of Terratech Report T-1050-C (604238) dated May 2006, the major faults of the region are overthrust type faults. They are shallow and do not penetrate the whole thickness of the lithosphere. They were active during the Ordovician Time until the end of the raise of Appalachian Front (- 450 My to - 400 My approx.)

- The Logan Fault, which runs through the St. Lawrence River and is routed almost horizontally below the south shore area and below the Appalachian Mountains was very active during the Ordovician Era (-450 My). It was totally stabilized since at least - 400 My. It is unlikely, and almost impossible, that this major fault would be reactivated, since the fault is shallow and does not penetrate all the lithosphere. Furthermore, because it belongs to a thrust type fault, we believe that if the said fault was the locus of seismic activity, earthquakes would occur all along the St. Lawrence River, and also on the south shore regions and even in New England (eastern U.S.A.), which is not the case.

- Past records have shown that most of the local earthquakes originate from the Charlevoix region, and not from Quebec City area. Furthermore, it is impossible to correlate this seismic activity that occurs at great depth, with any of the inferred faulting that is present at rather shallow depths at the project site. This is consistent with the above stated belief that the local major faults are not active.

Findings from the boreholes support the above statements concerning the probable inactivity of the (slip) faults presumably associated with ductile phase folding. For example: the rock core recovered in Borehole BH-108-05 (see Terratech Report T-1050-C (604238) dated May 2006) presents evidence of faulting but the rock appears to be healed. This may reasonably lead us to conclude that this fault or inferred fault was cemented over time with secondary minerals such as calcite.

Nevertheless, in order to conclude formally on the faulting activity at least after the latest glacial age (- 12 ky), Trial Excavation TE-B-05 (see Terratech Report T-1050-C (604238) dated May 2006) was carried out in the glacial deposit over a known seismic refraction anomaly (possible fault or fold) to observe any evidence of movement or disturbance in the soils that could be related to a recent fault (or fold) activity. In this respect, while this excavation was being carried out, attention paid to the overburden provided no clear evidence that the soil materials were disturbed to the bedrock other than by human activity.

For additional comments on the issue of fault activity, the reader should refer to Appendix 1 of this document, specifically to Section 2.1 (pp. 4 and 5) of Earthquake Hazard Analysis Report, written by Mrs. Gail M. Atkinson, Ph.D.

2.3 <u>Seismic Hazard</u>

2.3.1 Site Specific Seismic Hazard Assessment

As required by Codes CSA-Z276, NFPA-59A and EN-1473, a site-specific seismic hazard assessment study was recently carried out by Mrs. Gail M. Atkinson, Ph.D., Engineering Seismologist. The earthquake hazard analysis determined the expected earthquake ground motion over a range of probability levels, for the following return periods:

- 500 years;
- 1000 years;
- 2500 years;
- 5000 years.

For the detailed results of ground motion, the reader should refer to Section 3.2 (Table 2, p. 28) of Report "Earthquake Hazard Analysis", written by Mrs. Gail M. Atkinson, Ph.D.

All critical equipment of the Rabaska LNG Terminal will be designed using the ground motion calculated by the specific seismic hazard assessment.

2.3.2 Seismic Design based on NBCC

The National Building Code of Canada (NBCC) is the reference for seismic design of structures in Canada. A new version of the NBCC came out in September 2005.

As for seismic provisions, the new 2005 version of the NBCC presents two main differences compared to the former 1995 version:

- The Geological Survey of Canada has updated its seismic hazard models (detailed description in Open File # 4459, Adams and Halchuck, 2003);
- A new design approach has been adopted. In particular, it now considers a 2500 year return period (probability of exceedance of 2% in 50 years) and has moved from using peak ground motion to spectral accelerations. Calculation methods have been adjusted consequently.

For the site for the Rabaska Site, which is located at 46,82° North and –71,06° East, peak and spectral accelerations have been evaluated by GSC seismologist Stephen Halchuck (2005). These are summarized in Table 2-1.

Table 2-1
Seismic Hazard Calculation
by Earthquake Canada (Halchuck, 2005)

	1 : 100 years		**1 : 476 years**		**1 : 1000 years**		**1 : 2500 years**		**RGC**
Period	**SA (g)**		**SA (g)**		**SA (g)**		**SA (g)**		**factors**
(s)	**FG**	**HR**	**FG**	**HR**	**FG**	**HR**	**FG**	**HR**	
0.1	0.100	0.072	0.241	0.173	0.338	0.243	0.522	0.376	1.39
0.2	0.107	0.055	0.266	0.137	0.381	0.196	0.577	0.297	1.94
0.5	0.049	0.021	0.136	0.057	0.206	0.087	0.320	0.134	2.38
1	0.018	0.007	0.060	0.023	0.096	0.037	0.154	0.060	2.58
2	0.006	0.002	0.020	0.007	0.031	0.011	0.052	0.018	2.86
Peak acceleration	0.067	0.048	0.152	0.109	0.226	0.163	0.344	0.247	1.39

Note:
FG: Firm Ground soil condition (Reference Class C soil condition for 2005 NBCC)
HR: Hard Rock soil condition (Class A)

As indicated, the spectral acceleration data are available for Class C (FG / Firm Ground or Soft Rock) and for Class A (HR / Hard Rock) conditions. All non-critical equipment of the Rabaska LNG Terminal will be designed using the ground motion calculated by NBCC 2005.

3. SOIL LIQUEFACTION POTENTIAL – LAND SITE

3.1 General

The resistance of foundation materials to seismic loading, mainly the liquefaction potential, was determined by Terratech for the proposed LNG Process Area, where soil deposits are generally deeper than 5 or 6 m (Boreholes BH-401-05, BH-501-05 to BH-504-05 and BH-506-05), and locally extend to depths greater than 16 m (Boreholes BH-401-05 and BH-503-05). Borehole BH-505-05 was excluded from the analyses as bedrock was encountered at a depth of 0.8 m below existing ground surface.

Based on the above borehole results, the granular soil deposits are generally compact to dense as the measured (uncorrected) N_{SPT} index mostly range from 20 to 50. Such materials are generally not expected to be potentially liquefiable. Still, the potential for liquefaction has been checked using the Seed, H.B. and Idriss, I.M. 1971 "simplified procedure" as reviewed and updated by Youd, T.L. et al. 2001. This method compares the "cyclic stress ratio" (CSR) and the "cyclic resistance ratio" (CRR) to estimate the factor of safety against liquefaction.

The assessment of soil liquefaction potential is essentially based on the deterministic approach proposed by Youd, T.L. et al. 2001. Input in terms of peak ground acceleration (PGA) and earthquake magnitude (M_w) was provided by the Report "Earthquake Hazard Analysis" (see Appendix I).

3.2 Estimation of Cyclic Stress Ratio, CSR

The CSR, the imposed cyclic stress ratio or "the seismic demand on a soil layer", is estimated based on the peak ground acceleration at the site.

$$CSR = 0.65\frac{a_{\max}}{g}\cdot\frac{\sigma_0}{\sigma'_0}\cdot r_d$$

The following peak ground accelerations (PGA) were provided in Table 2 (p. 28) of Atkinson, G.M. 2006.

- ***Return period of 500 years:***
  PGA: a_{max} = 149 cm/s^2 (or 0.15 g)

- ***Return period of 1 000 years:***
  PGA: a_{max} = 250 cm/s^2 (or 0.25 g)

- ***Return period of 2 500 years:***
  PGA: a_{max} = 446 cm/s^2 (or 0.45 g)

- ***Return period of 5 000 years:***
  PGA: a_{max} = 630 cm/s^2 (or 0.64 g)

In compliance with the method proposed by Youd, T.L. et al. 2001, computed CSR values were by necessity adjusted and converted into CSR / $K\sigma$ values whenever the acting effective overburden vertical stress exceeds 100 kPa. This procedure was required to allow a direct comparison with the CRR reference values or curves, and the determination of factors of safety with respect to soil liquefaction.

The CSR / $K\sigma$ values are shown on Figures 1A to 4A and 1B to 4B in Appendix II. They were determined only for computed $(N_1)_{60}$ data that are less than 30, as the method precludes the use of higher corrected N values by stating that such denser soils will not be prone to liquefaction. In this perspective and in assessing the depths where liquefaction could occur, consideration was given to both the graphical and tabulated data that are presented in Appendix I.

3.3 Estimation of Cyclic Resistance Ratio, CRR

The liquefaction resistance or the “capacity of the soil to resist liquefaction”, expressed in the term CRR, is estimated based on field density measurements and the so-called Seed chart as modified by NCEER and NCEER/NSF Workshops (Youd, T.L. et al. 2001). In Figures 1A to 4A (Appendix II), reference CRR_{corr} curves are shown for soils with fine contents (particles smaller than 0.080 mm) of 5 %, 15 % and 35 %. The

CRR_{corr} values given on Figures 1B to 4B are in reference to soils with a fines content of 5 %, which constitutes a conservative CRR profile.

The CRR values were estimated for the following earthquake magnitudes, on the basis of information provided in Atkinson, G.M. 2006:

- ***Return period of 500 years*** (Ref.: Appendix II, Figures 1A and 1B):
  Magnitude, M_w : 6.00 (Richter)

- ***Return period of 1000 years*** (Ref.: Appendix II, Figures 2A and 2B):
  Magnitude, M_w : 6.25 (Richter)

- ***Return period of 2500 years*** (Ref.: Appendix II, Figures 3A and 3B):
  Magnitude, M_w : 6.50 (Richter)

- ***Return period of 5000 years*** (Ref.: Appendix II, Figures 4A and 4B):
  Magnitude, M_w : 6.75 (Richter)

3.4 Factor of Safety with respect to Soil Liquefaction

The CRR_{corr} values (curves) and the CSR / $K\sigma$ profiles (points) are compared and the factor of safety is estimated as $FS = CRR_{corr} / [CSR / K\sigma]$ (Appendix II / Figures 1B to 4B). If the factor of safety is lower than 1.0, the soil should be considered as potentially liquefiable. However soils above the water table or with a noticeable clay content (i.e. soils with some clay or clayey soils) are not prone to liquefaction.

In reference to Figures 1A to 4A and with respect only to the CRR curve (FC = 5%) applicable to soils with no more than 5 % of fine particles smaller than 0.080 mm, the following <u>"very safe soil liquefaction predictions"</u> are given (but will be discussed further in this report):

- ***Return period of 500 years*** (Ref.: Appendix II, Figure 1A):
  - Borehole BH-401-05: *no liquefaction*.
  - Boreholes BH-501-05 to BH-504-05 and BH-506-05: *no liquefaction.*

- ***Return period of 1000 years*** (Ref.: Appendix II, Figure 2A):
  - Borehole BH-401-05: <u>potential liquefaction</u> from 13.7 to 17.5 m.
  - Boreholes BH-501-05 and BH-502-05: *no liquefaction.*
  - Borehole BH-503-05: <u>potential liquefaction</u> from 10.5 to 11.5 m.
  - Boreholes BH-504-05 and BH-506-05: *no liquefaction* from 0 to 6.5 or 6.6 m depth.

- ***Return period of 2500 years*** (Ref.: Appendix II, Figure 3A):
  - Borehole BH-401-05: <u>potential liquefaction</u> from 12.2 to 17.5 m.
  - Borehole BH-501-05: <u>potential liquefaction</u> from 3.8 to 5.8 m.
  - Borehole BH-502-05: *no liquefaction*.
  - Borehole BH-503-05: <u>potential liquefaction</u> from 10.5 to 11.5, 16.6 to 17.0, and 19.7 to 20.6 m.
  - Borehole BH-504-05: *no liquefaction* from 0 to 6.5 m depth.
  - Borehole BH-506-05: <u>potential liquefaction</u> from 4.6 to 5.1 m, and *no liquefaction* from 0 to 4.6 m and from 5.1 to 6.6 m depth.

- ***Return period of 5000 years*** (Ref.: Appendix II, Figure 4A):
  - Borehole BH-401-05: <u>potential liquefaction</u> from 7.6 to 8.3 m and from 9.4 to 17.5 m.
  - Borehole BH-501-05: <u>potential liquefaction</u> from 3.1 to 5.8 m.
  - Borehole BH-502-05: *no liquefaction*.
  - Borehole BH-503-05: <u>potential liquefaction</u> from 2.3 to 2.8, 10.5 to 12.7, 16.6 to 17.0, and 19.7 to 20.6 m.
  - Borehole BH-504-05: <u>potential liquefaction</u> from 0.8 to 1.3 m, and *no liquefaction* from 0 to 0.8 m and from 1.3 to 6.5 m depth.
  - Borehole BH-506-05: <u>potential liquefaction</u> from 4.6 to 5.1 m and *no liquefaction* from 0 to 4.6 m and from 5.1 to 6.6 m depth.

In reference to Tables 4-1 and 4-3 (pages 24 and 26) of the "Geotechnical Site Study Report (Phase 3)" of 23 May 2006, granular materials encountered at the project site (mainly sand and silt or gravely sand with some silt) were found to contain variable and often large quantities of fines, ranging from 8 to 66 % and averaging 32 %, and also clay size particles (smaller than 0.002 mm) ranging from 3 to 26 % and averaging 11 %. With the above gradation results, particles smaller than 0.005 mm within the predominantly granular materials would likely range from 4 to 35 % and average 16 %, thus suggesting that a large portion of the granular soils, irrelevant of the in-situ compactness, will not be prone to liquefaction, as the "Chinese Criteria" (Youd, T.L. et al. 2001) stipulates that soils with more than 15 % of particles smaller than 0.005 mm will not liquefy. In consideration of known fine contents (based on results of grain size analyses) or inferred values thereof (from visual description of recovered soils samples), remarks were added in Figures 1B to 4B, leading to the following "realistic soil liquefaction predictions":

- ***Return period of 500 years*** (Ref.: Appendix II, Figures 1A and 1B):
  - Borehole BH-401-05: *no liquefaction*.
  - Boreholes BH-501-05 to BH-504-05 and BH-506-05: *no liquefaction*.

- ***Return period of 1000 years*** (Ref.: Appendix II, Figures 2A and 2B):
  - Borehole BH-401-05: *no liquefaction* from 13.7 to 17.5 m as the visually examined silty sand likely contains at least 20 % of fines.
  - Boreholes BH-501-05 and BH-502-05: *no liquefaction.*
  - Borehole BH-503-05: <u>potential liquefaction</u> from 10.5 to 11.5 m, as the sand with some silt to silty could contain a little as 10 % of fines.
  - Boreholes BH-504-05 and BH-506-05: *no liquefaction* from 0 to 6.5 or 6.6 m depth.

- ***Return period of 2500 years*** (Ref.: Appendix II, Figures 3A and 3B):
  - Borehole BH-401-05: <u>potential liquefaction</u> from 12.2 to 17.5 m, as the visually examined silty sand likely contains at least 20 % of fines.

- Borehole BH-501-05: *no liquefaction* as the silt and sand from 3.8 to 5.8 m was found, from grain size analyses, to contain more than 35 % of fines
- Borehole BH-502-05: *no liquefaction*.
- Borehole BH-503-05: <u>potential liquefaction</u> from 10.5 to 11.5, 16.6 to 17.0, and 19.7 to 20.6 m, as the visually examined sand with some silt to silty sand could contain as little as 10 % of fines.
- Borehole BH-504-05: *no liquefaction* from 0 to 6.5 m depth.
- Borehole BH-506-05: *no liquefaction* from 4.6 to 5.1 m as the silty sand probably contains more than 35 % of fines based on grain size results on adjacent soil samples, and *no liquefaction* from 0 to 4.6, and from 5.1 to 6.6 m depth.

- ***Return period of 5000 years*** (Ref.: Appendix II, Figures 4A and 4B):
  - Borehole BH-401-05: <u>potential liquefaction</u> from 7.6 to 8.3 m, and 9.4 to 17.5 m, as the visually examined sand with some silt or silty sand likely contain some 10 or 20 % of fines.
  - Borehole BH-501-05: *no liquefaction* as the silt and sand from 3.1 to 5.8 m was found, from grain size analyses, to contain more than 35 % of fines
  - Borehole BH-502-05: *no liquefaction*.
  - Borehole BH-503-05: *no liquefaction* from 2.3 to 2.8 m, as the gravelly silt contains more than 35 % of fines. <u>Potential liquefaction</u> from 10.5 to 12.7 m as the sand with some silt can contain as little as 10 % of fines. <u>Potential liquefaction</u> from 16.6 to 17.0, and 19.7 to 20.6 m as the visually examined sand with some silt or silty sand could contain as little as 10 % of fines.
  - Borehole BH-504-05: *no liquefaction* from 0.8 to 1.3 m, as the silty sand contains more than 20 % of fines, and *no liquefaction* from 0 to 0.8 and from 1.3 to 6.5 m depth.

- Borehole BH-506-05: *no liquefaction* from 4.6 to 5.1 m, as the silty sand probably contains more than 35 % of fines based on grain size results on adjacent soil samples, and *no liquefaction* from 0 to 4.6 and from 5.1 to 6.6 m depth.

3.5 Comments about Soil Liquefaction

From the above "realistic soil liquefaction predictions", the following comments are provided in view of facilities founded on shallow footings located in the overburden:

(1) With a return period of 500 years, no soil liquefaction will occur.

(2) With a return period of 1000 years, soil liquefaction is possible very locally, in particular between depth 10.5 to 11.5 m at the site of one borehole (BH-503-05).

(3) With a longer return period of 2500 years, soil liquefaction is possible locally at the site of two boreholes (BH-401-05 and BH-503-05) out of a total of seven boreholes put down in the proposed LNG Process Area. Furthermore, at these locations the liquefaction of soil would potentially be generated at depths greater than 10 and 12 m, and also would be confined to a sand stratum some 5 m in thickness (BH-401-05) and to three deep and compact sand layers that are no thicker than 1 m (BH-503-05). At the site of Boreholes BH-504-05 and BH-506-05, which were terminated at an approximate depth of 6.5 m below grade in dense to very dense soils and without encountering bedrock, the presence below a depth of 6.5 m of compact soils with a low content of fines, and thus prone to liquefaction, is possible as such liquefiable soils were encountered in Borehole BH-401-05. This situation needs to be further investigated during the detailed engineering phases of the project

(4) With the more stringent return period of 5000 years, and based on the available subsurface information, soil liquefaction is possible at least at the site of two boreholes (BH-401-05 and BH-503-05) out of a total of seven boreholes carried out in Process Area. The vertical extent of the probable soil liquefaction would involve a 0.7 m thick soil layer at 7.5 m depth and a 6 m thick stratum beyond 9.4 m depth (BH-401-05), and three soils strata ranging in thickness from 0.4 to

2.2 m below a depth of 10.5 m (BH-503-05). As stated in (3) and in view of a possible situation that should be further investigated during the detailed engineering phases of the project, soil liquefaction remains a possibility in the vicinity of Boreholes BH-504-05 and BH-506-05 at depths greater than 6.5 and 6.6 m.

(5) From the above consideration and statements, the risk of soil liquefaction is seen as non-existing or negligible for return periods of 1 000 years or less, as soil liquefaction is foreseen as potentially occurring only very locally and at a depth greater than 10 m.

(6) With return periods of 2500 and 5000 years, soil liquefaction may potentially occur more frequently but only at depths greater than 10 to 12 m or 7 to 10 m respectively. This assessment is specific for the sites of Boreholes BH-401-05 and BH-503-05, and also possibly (and subjected to further subsurface investigations) at a depth greater than 6.5 m in the vicinity of Boreholes BH-504-05 and BH-506-05. At these locations, soil liquefaction is seen as constituting a limited or low hazard with respect to future performance of lightly loaded isolated shallow foundations, as the rather thick generally dense soils extending to 7 or 10 m depth that are not prone to liquefaction would provide a reliable bearing backup in case of local softening of deeper soils under severe earthquake conditions. With due consideration to the residual and reduced strength of deep soils during the liquefaction process, isolated lightly loaded footings seated at shallow depths (1.8 to 2.0 m depth range) are possible, as the upper layers of dense soil will readily dissipate and eliminate most of the stress induced by the footings into the deep softened "liquefied" soils. To achieve this goal (of limiting the induced stress into the deep strata prone to liquefaction), the footings should generally not exceed 2 to 3 m in width, exert vertical pressures no larger than 230 and 150 kPa respectively, and be at least laterally spaced at 6 m. With closer footings spaced at about 4 to 5 m, the above acting pressures (at foundation level) should be limited to 130 and 100 kPa respectively. At this preliminary phase of the project, it is inferred that all contemplated facilities (except the LNG storage tanks already seated on rock) would comply in terms of surface footing loading and layout to the above limitations. During the detail engineering studies, soil liquefaction issues for the

return period of 5000 years need to be addressed more closely in terms of soils surface loading, plant layout and soil gradation.

4. SOIL LIQUEFACTION POTENTIAL – MARINE SITE

Through a direct comparison of subsurface data presently available at the marine site or Jetty (from past investigations by others) with respect to the information obtained by Terratech at the land site of the contemplated LNG facilities, and with due consideration to the gradation and the relative density of similar type of soils, it is possible to extrapolate data for the marine site from some of the soil liquefaction predictions generated from the detailed analyses at the land site.

From this preliminary exercise, which in future times should by necessity be followed by detailed investigations, testing and analyses, the potential liquefaction of marine sediments under severe earthquake conditions, was assessed along the proposed pile supported jetty and the ship docking facilities. Soil liquefaction is thus foreseen to locally involve only the first 5 to 6 m of sediments, typically sand with traces of silt or sand with gravel, constituting the bottom of the St. Lawrence River, specifically at the site of Boreholes F-1, F-2A et F-3 (carried out by Laboratoires d'Expertises de Québec Ltée). Soil liquefaction would also occur within a thin layer of the bottom sand, less than 0.9 m in thickness, at the site of Boreholes F-8 et F-9, whereas no soil liquefaction was predicted at the locus of Borehole F-7, except very locally at an approximate depth of 10 m in bottom sediments.

The above preliminary predictions support the possibility of rather local soil liquefaction phenomena and somewhat preclude a general slip failure of the riverbed sediments. In this perspective and within this preliminary phase of the project, the piles of the jetty and docking facilities should be designed with due consideration for the temporary loss of lateral support within the predicted soil liquefaction zone.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General

In addition to the recommendations already provided in Section 6 of Terratech Report T-1050-C (604238) “Geotechnical Site Study Report (Phase 3)” issued on 23 May 2006, the following is proposed regarding soil liquefaction potential at the project site.

5.2 LNG Storage Tanks

As stated in Section 6.2 of Terratech Report T-1050-C, the LNG storage tanks will be founded on bedrock. Soil liquefaction is therefore not an issue for LNG storage tanks.

5.3 LNG Process Area

Soil liquefaction under earthquake conditions within the LNG Process Area was discussed in Section 3. Based on “realistic” predictions for 500 to 1000 year recurrence periods, the phenomenon would have no effect on shallow foundations. With the longer 2500 and 5000 year recurrence periods, negligible or no consequences are foreseen with respect to future behaviour and integrity of lightly loaded shallow foundations (see definition in Section 3.5, Item 6), as a result of or due to soil softening that could occur locally at deeply below the footings. This assessment should be addressed more closely during the detailed engineering studies

5.4 Unloading Lines

Soil liquefaction under earthquake conditions may occur locally in the shallow layers of loose sand that were encountered at less than 2 or 3 m depth in Boreholes BH-302-05 to BH-304-05. To avoid damages to the unloading lines and facilities, the shallow loose soils should be excavated and replaced by well compacted granular fill materials, or the foundations should be seated on deeper and more competent soils, or on bedrock.

5.5 Marine structures

Based on the preliminary assessments, soil liquefaction at the marine site under the most severe earthquake conditions is only possible locally within the first 5 to 6 m of sediments in the St. Lawrence riverbed. General slip failures of bottom sediments are presently not considered an issue. Additional investigations and testing together with formal soil liquefaction analyses shall be performed at the marine site as part of the future detailed engineering studies.

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7. PERSONNEL

Sections 1, and 3 to 7 of this Addendum Report were prepared by Mr. Raymond Bousquet, Eng., M.A.Sc. Mrs. Catherine Bédard, Eng., M.A.Sc., carried out the detailed soil liquefaction software calculations whose results are presented in Appendix I, and provided assistance to the preparation of Section 3 of this report. Sections 2.1 and 2.2 of the report were written by Mr. Jean-Jacques Hébert, Geologist, and Mr. Yves Boulianne, Eng. Section 2.3.1 of the present document refers to a specialty report on Earthquake Hazard analysis prepared by Mrs. Gail M. Atkinson, Ph.D., Engineering Seismologist. Section 2.3.2 was prepared by Dr. Denise Leahy, Eng., Dr. Eng.

This document was reviewed for ISO Conformity by Mr. Henri Madjar, Eng., M.A.Sc.

TERRATECH
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With ISO 9001 by:

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Appendix I
Soil Liquefaction Data and Figures

Terratech

Figure 1A
Soil liquefaction - 1:500 years
Site Rabaska
a_{max} = 0.15g and magnitude 6.0 on Richter scale

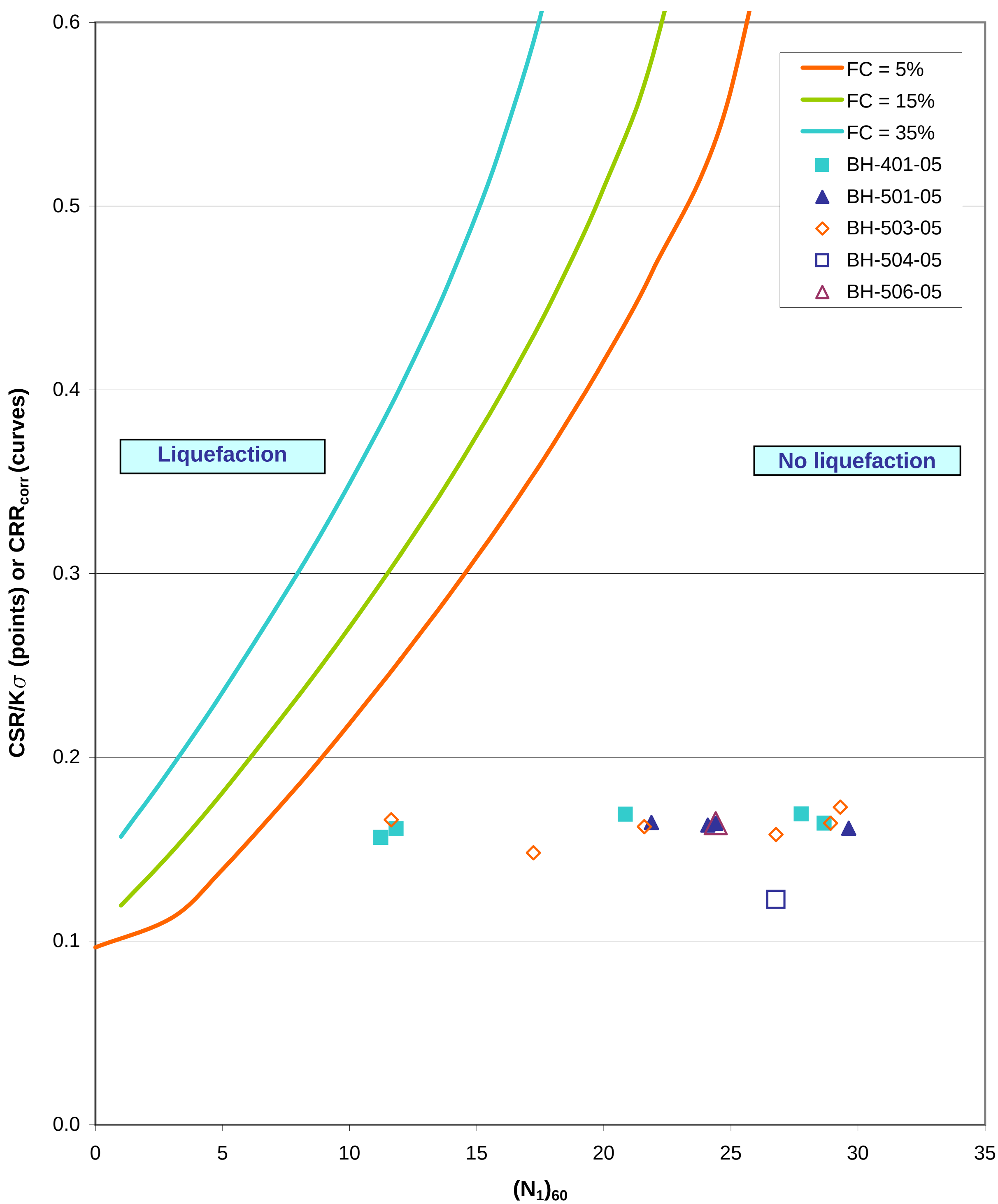

Figure 1B
Soil liquefaction for 1:500 years
Site Rabaska
a_{max} = 0.15g and magnitude of 6.0 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	r_d	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-401-05 (SS-1)	0.00	0.30	3	6	6	1.00	1.70	0.75	0.29	1.00	3.8	0.10	0.10	0.06	0.12	1.26	Sand	Above water level : no liquefaction
BH-401-05 (SS-2)	0.70	1.00	45	21	15	0.99	1.70	0.75	1.12	1.00	57.4	---	---	---	---	---	Sand	
BH-401-05 (SS-3)	1.50	1.80	37	38	24	0.99	1.70	0.75	1.01	1.00	47.2	---	---	---	---	---	Sand	
BH-401-05 (SS-4)	2.20	2.50	100	53	32	0.98	1.70	0.75	1.66	1.00	127.5	---	---	---	---	---	Sand	
BH-401-05 (SS-6)	3.00	3.30	100	69	41	0.97	1.56	0.80	1.65	1.00	125.2	---	---	---	---	---	Sand	
BH-401-05 (SS-8)	3.80	4.10	100	86	50	0.97	1.42	0.85	1.62	1.00	120.4	---	---	---	---	---	Sand	
BH-401-05 (SS-10)	4.60	4.90	54	103	59	0.96	1.30	0.85	1.14	1.00	59.9	---	---	---	---	---	Sand	
BH-401-05 (SS-11)	5.30	5.60	38	118	67	0.96	1.23	0.85	0.93	1.00	39.6	---	---	---	---	---	Sand	
BH-401-05 (SS-12)	6.10	6.40	34	134	76	0.95	1.15	0.95	0.90	1.00	37.2	---	---	---	---	---	Sand	
BH-401-05 (SS-13)	7.60	7.90	29	166	92	0.94	1.04	0.95	0.79	1.00	28.7	0.16	0.16	0.40	0.76	4.65	Sand	
BH-401-05 (SS-15)	9.00	9.30	100	195	108	0.93	0.96	0.95	1.41	0.95	91.4	---	---	---	---	---	Sand	
BH-401-05 (SS-17)	9.40	9.70	31	204	112	0.92	0.94	0.95	0.78	0.96	27.8	0.16	0.17	0.36	0.70	4.13	Sand	
BH-401-05 (SS-20)	12.20	12.50	25	263	144	0.84	0.83	1.00	0.67	0.88	20.8	0.15	0.17	0.23	0.44	2.58	Sand	
BH-401-05 (SS-21)	13.70	14.00	15	294	161	0.80	0.79	1.00	0.51	0.89	11.8	0.14	0.16	0.13	0.25	1.55	Sand	
BH-401-05 (SS-22)	15.30	15.60	15	328	178	0.76	0.75	1.00	0.49	0.87	11.2	0.14	0.16	0.12	0.24	1.53	Sand	
BH-501-05 (SS-1)	0.00	0.30	6	6	6	1.00	1.70	0.75	0.41	1.00	7.7	0.10	0.10	0.09	0.18	1.85	Sand	Above water level : no liquefaction
BH-501-05 (SS-2)	0.70	1.00	29	21	15	0.99	1.70	0.75	0.90	1.00	37.0	---	---	---	---	---	Sand	
BH-501-05 (SS-3)	1.50	1.80	17	38	24	0.99	1.70	0.75	0.69	1.00	21.7	0.15	0.15	0.24	0.46	3.04	Clayey soil	Clayey soil: no liquefaction
BH-501-05 (SS-4)	2.30	2.60	24	55	33	0.98	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-501-05 (SS-5)	3.10	3.40	24	71	42	0.97	1.54	0.80	0.80	1.00	29.6	0.16	0.16	0.44	0.86	5.31	Sand	
BH-501-05 (SS-6)	3.80	4.10	20	86	50	0.97	1.42	0.85	0.72	1.00	24.1	0.16	0.16	0.27	0.53	3.25	Sand	
BH-501-05 (SS-7)	4.60	4.90	22	103	59	0.96	1.30	0.85	0.73	1.00	24.4	0.16	0.16	0.28	0.54	3.30	Sand	
BH-501-05 (SS-8)	5.30	5.60	21	118	67	0.96	1.23	0.85	0.69	1.00	21.9	0.16	0.16	0.24	0.46	2.82	Sand	
BH-501-05 (SS-9)	6.10	6.40	47	134	76	0.95	1.15	0.95	1.06	1.00	51.4	---	---	---	---	---	Sand	
BH-501-05 (SS-10)	6.80	7.10	46	149	83	0.94	1.10	0.95	1.02	1.00	47.9	---	---	---	---	---	Sand	
BH-501-05 (SS-11)	7.60	7.90	136	166	92	0.94	1.04	0.95	1.71	1.00	134.5	---	---	---	---	---	Sand	
BH-501-05 (SS-12)	9.10	9.40	100	197	109	0.92	0.96	0.95	1.41	0.94	90.9	---	---	---	---	---	Sand	
BH-502-05 (SS-1)	0.00	0.30	7	6	6	1.00	1.70	0.75	0.44	1.00	8.9	0.10	0.10	0.10	0.20	2.06	Sand	Above water level : no liquefaction
BH-502-05 (SS-2)	0.70	1.00	64	21	19	0.99	1.70	0.75	1.33	1.00	81.6	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-3)	1.50	1.80	100	38	28	0.99	1.70	0.75	1.66	1.00	127.5	---	---	---	---	---	Sand	
BH-502-05 (SS-5)	2.20	2.50	47	53	36	0.98	1.67	0.75	1.13	1.00	58.9	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-6)	3.00	3.30	36	69	45	0.97	1.49	0.80	0.97	1.00	43.0	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-7)	3.80	4.10	25	86	54	0.97	1.36	0.85	0.79	1.00	29.0	0.15	0.15	0.41	0.79	5.23	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-8)	4.50	4.80	28	101	62	0.96	1.27	0.85	0.81	1.00	30.3	---	---	---	---	---	Sand	

Figure 1B
Soil liquefaction for 1:500 years
Site Rabaska
a_{max} = 0.15g and magnitude of 6.0 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	r_d	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-503-05 (SS-1)	0.00	0.30	13	6	6	1.00	1.70	0.75	0.60	1.00	16.6	0.10	0.10	0.18	0.34	3.50	Sand	Above water level : no liquefaction
BH-503-05 (SS-2)	0.70	1.00	30	21	15	0.99	1.70	0.75	0.91	1.00	38.3	---	---	---	---	---	Sand	
BH-503-05 (SS-3)	1.50	1.80	40	38	24	0.99	1.70	0.75	1.05	1.00	51.0	---	---	---	---	---	Sand	
BH-503-05 (SS-4)	2.30	2.60	21	55	33	0.98	1.70	0.75	0.76	1.00	26.8	0.16	0.16	0.33	0.64	4.06	Sand	
BH-503-05 (SS-5)	3.10	3.40	74	71	42	0.97	1.54	0.80	1.41	1.00	91.4	---	---	---	---	---	Sand	
BH-503-05 (SS-7)	3.80	4.10	60	86	50	0.97	1.42	0.85	1.25	1.00	72.3	---	---	---	---	---	Sand	
BH-503-05 (SS-8)	4.50	4.80	52	101	58	0.96	1.32	0.85	1.13	1.00	58.2	---	---	---	---	---	Sand	
BH-503-05 (SS-10)	5.30	5.60	54	118	67	0.96	1.23	0.85	1.11	1.00	56.2	---	---	---	---	---	Sand	
BH-503-05 (SS-11)	6.10	6.40	34	134	76	0.95	1.15	0.95	0.90	1.00	37.2	---	---	---	---	---	Sand	
BH-503-05 (SS-12)	7.60	7.90	31	166	92	0.94	1.04	0.95	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-503-05 (SS-14)	9.20	9.50	100	200	110	0.92	0.95	0.95	1.40	0.93	90.5	---	---	---	---	---	Sand	
BH-503-05 (SS-17)	9.70	10.00	67	210	116	0.91	0.93	1.00	1.16	0.92	62.3	---	---	---	---	---	Sand	
BH-503-05 (SS-19)	10.50	10.80	13	227	125	0.89	0.90	1.00	0.50	0.95	11.6	0.16	0.17	0.13	0.25	1.49	Sand	
BH-503-05 (SS-20)	12.10	12.40	35	260	143	0.84	0.84	1.00	0.80	0.87	29.3	0.15	0.17	0.43	0.82	4.75	Sand	
BH-503-05 (SS-22)	13.60	13.90	47	292	159	0.80	0.79	1.00	0.90	0.81	37.2	---	---	---	---	---	Sand	
BH-503-05 (SS-23)	15.10	15.40	73	323	176	0.76	0.75	1.00	1.09	0.73	55.0	---	---	---	---	---	Sand	
BH-503-05 (SS-24)	16.60	16.90	30	355	193	0.72	0.72	1.00	0.69	0.80	21.6	0.13	0.16	0.24	0.46	2.81	Sand	
BH-503-05 (SS-26)	18.20	18.50	42	389	211	0.68	0.69	1.00	0.79	0.74	28.9	0.12	0.16	0.41	0.78	4.78	Sand	
BH-503-05 (SS-28)	19.70	20.00	26	420	228	0.64	0.66	1.00	0.61	0.78	17.2	0.12	0.15	0.18	0.35	2.39	Sand	
BH-503-05 (SS-29)	21.20	21.50	82	452	245	0.60	0.64	1.00	1.07	0.62	52.4	---	---	---	---	---	Sand	
BH-503-05 (SS-30)	22.80	23.10	100	485	262	0.56	0.62	1.00	1.16	0.57	61.7	---	---	---	---	---	Sand	
BH-503-05 (SS-33)	24.00	24.30	100	510	276	0.53	0.60	1.00	1.14	0.56	60.2	---	---	---	---	---	Sand	
BH-504-05 (SS-1)	0.00	0.30	5	6	6	1.00	1.70	0.75	0.37	1.00	6.4	0.10	0.10	0.08	0.16	1.64	Clayey soil	Above water level and clayey soil : no liquefaction
BH-504-05 (SS-2)	0.80	1.10	21	23	18	0.99	1.70	0.75	0.76	1.00	26.8	0.12	0.12	0.33	0.64	5.23	Sand	
BH-504-05 (SS-3)	1.50	1.80	24	38	26	0.99	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-504-05 (SS-4)	2.30	2.60	34	55	35	0.98	1.69	0.75	0.97	1.00	43.1	---	---	---	---	---	Sand	
BH-504-05 (SS-5)	3.00	3.30	86	69	43	0.97	1.53	0.80	1.51	1.00	105.1	---	---	---	---	---	Sand	
BH-504-05 (SS-7)	3.80	4.10	100	86	52	0.97	1.39	0.85	1.60	1.00	118.1	---	---	---	---	---	Sand	
BH-504-05 (SS-10)	4.90	5.20	100	109	64	0.96	1.25	0.85	1.52	1.00	106.2	---	---	---	---	---	Sand	
BH-504-05 (SS-12)	5.70	6.00	68	126	73	0.95	1.17	0.95	1.28	1.00	75.6	---	---	---	---	---	Sand	
BH-504-05 (SS-13)	6.10	6.40	100	134	78	0.95	1.14	0.95	1.53	1.00	107.9	---	---	---	---	---	Sand	

Figure 1B
Soil liquefaction for 1:500 years
Site Rabaska
a_{max} = 0.15g and magnitude of 6.0 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	r_d	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-506-05 (SS-1)	0.00	0.30	6	6	6	1.00	1.70	0.75	0.41	1.00	7.7	0.10	0.10	0.09	0.18	1.85	Sand	Above water level : no liquefaction
BH-506-05 (SS-2)	0.80	1.10	24	23	16	0.99	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-506-05 (SS-3)	1.50	1.80	36	38	24	0.99	1.70	0.75	1.00	1.00	45.9	---	---	---	---	---	Sand	
BH-506-05 (SS-4)	2.30	2.60	44	55	33	0.98	1.70	0.75	1.10	1.00	56.1	---	---	---	---	---	Sand	
BH-506-05 (SS-6)	3.00	3.30	100	69	41	0.97	1.56	0.80	1.65	1.00	125.2	---	---	---	---	---	Sand	
BH-506-05 (SS-8)	3.80	4.10	32	86	50	0.97	1.42	0.85	0.92	1.00	38.5	---	---	---	---	---	Sand	
BH-506-05 (SS-9)	4.60	4.90	22	103	59	0.96	1.30	0.85	0.73	1.00	24.4	0.16	0.16	0.28	0.54	3.30	Sand	
BH-506-05 (SS-10)	5.30	5.60	29	118	67	0.96	1.23	0.85	0.81	1.00	30.2	---	---	---	---	---	Sand	
BH-506-05 (SS-11)	6.10	6.40	75	134	76	0.95	1.15	0.95	1.33	1.00	82.0	---	---	---	---	---	Sand	

N: blow count/0.3 m
σ_{vo} : total overburden stress
σ_{vo} : effective overburden stress
r_d: stress reduction coefficient
C_N: correction factor foroverburden pressure
C_R: correction factor for rod lenght
Dr : relative density
$K\sigma$: correction for high overburden stresses
$(N_1)_{60}$: correction of N
CSR: cyclic stress ratio
$CRR_{7.5}$: cyclic resistance ratio for magnitude 7.5
CRR_{corr}: cyclic resistance ratio for a different magnitude (MSF)
MSF: magnitude scaling factor

LEGEND	
Above ground water level	Depth < 9.15 m
Below ground water level	Depth > 9.15 m

Granular soil
Clayey soil

--- **$(N1)_{60}$ >= 30 : No liquefaction**

*** Factor of Safety: F.S.= ($CRR_{7.5}$/CSR)*MSF*$K\sigma$**

* Note: factor ofs safety calculated for 5% fine content

Figure 2A
Soil liquefaction - 1:1000 years
Site Rabaska
a_{max} = 0.25g and magnitude 6.25 on Richter scale

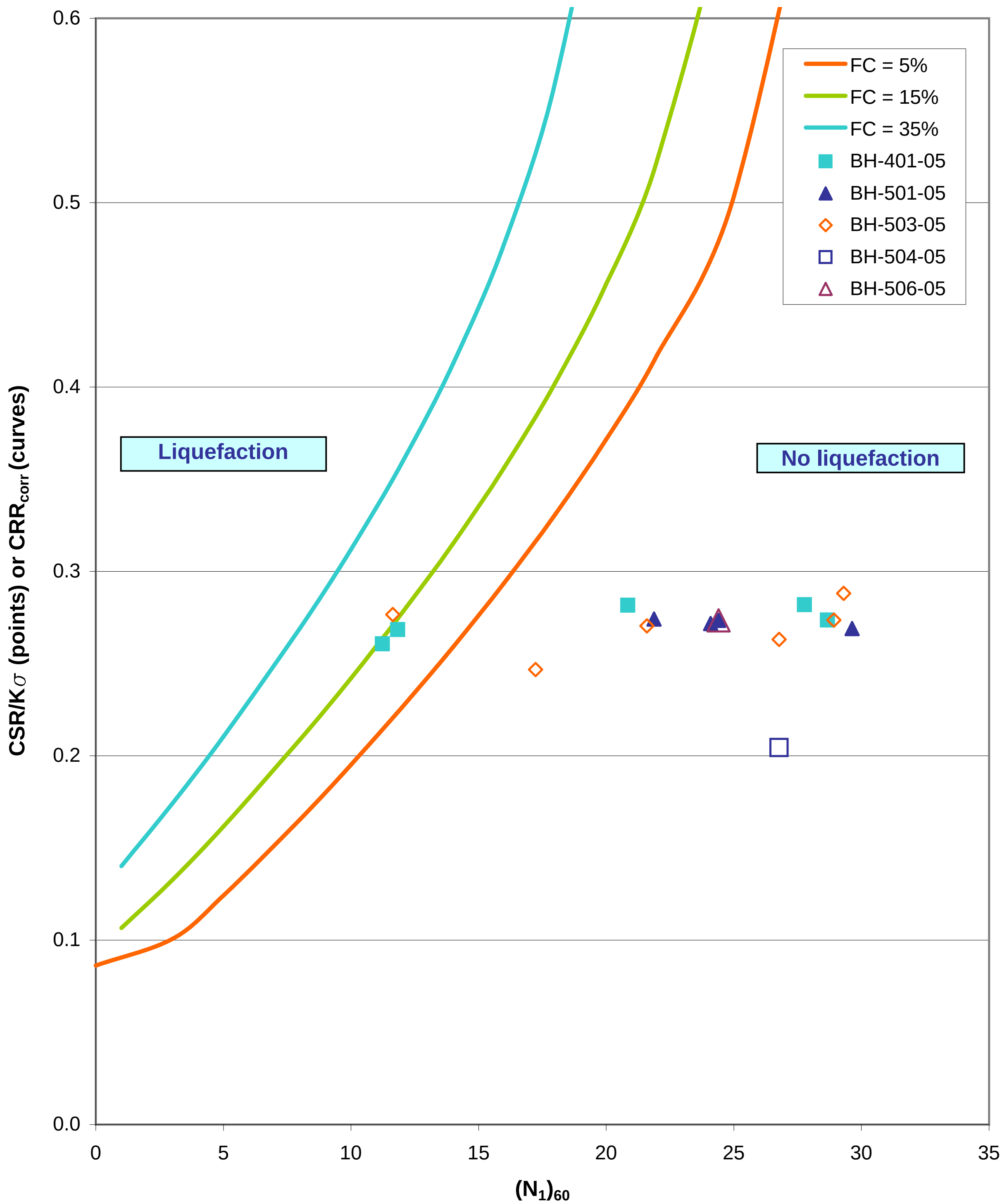

Figure 2B
Soil liquefaction for 1:1000 years
Site Rabaska
a_{max} = 0.25g and magnitude of 6.25 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	r_d	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-401-05 (SS-1)	0.00	0.30	3	6	6	1.00	1.70	0.75	0.29	1.00	3.8	0.16	0.16	0.06	0.11	0.68	Sand	Above water level : no liquefaction
BH-401-05 (SS-2)	0.70	1.00	45	21	15	0.99	1.70	0.75	1.12	1.00	57.4	---	---	---	---	---	Sand	
BH-401-05 (SS-3)	1.50	1.80	37	38	24	0.99	1.70	0.75	1.01	1.00	47.2	---	---	---	---	---	Sand	
BH-401-05 (SS-4)	2.20	2.50	100	53	32	0.98	1.70	0.75	1.66	1.00	127.5	---	---	---	---	---	Sand	
BH-401-05 (SS-6)	3.00	3.30	100	69	41	0.97	1.56	0.80	1.65	1.00	125.2	---	---	---	---	---	Sand	
BH-401-05 (SS-8)	3.80	4.10	100	86	50	0.97	1.42	0.85	1.62	1.00	120.4	---	---	---	---	---	Sand	
BH-401-05 (SS-10)	4.60	4.90	54	103	59	0.96	1.30	0.85	1.14	1.00	59.9	---	---	---	---	---	Sand	
BH-401-05 (SS-11)	5.30	5.60	38	118	67	0.96	1.23	0.85	0.93	1.00	39.6	---	---	---	---	---	Sand	
BH-401-05 (SS-12)	6.10	6.40	34	134	76	0.95	1.15	0.95	0.90	1.00	37.2	---	---	---	---	---	Sand	
BH-401-05 (SS-13)	7.60	7.90	29	166	92	0.94	1.04	0.95	0.79	1.00	28.7	0.27	0.27	0.40	0.68	2.49	Sand	
BH-401-05 (SS-15)	9.00	9.30	100	195	108	0.93	0.96	0.95	1.41	0.95	91.4	---	---	---	---	---	Sand	
BH-401-05 (SS-17)	9.40	9.70	31	204	112	0.92	0.94	0.95	0.78	0.96	27.8	0.27	0.28	0.36	0.62	2.21	Sand	
BH-401-05 (SS-20)	12.20	12.50	25	263	144	0.84	0.83	1.00	0.67	0.88	20.8	0.25	0.28	0.23	0.39	1.39	Sand	
BH-401-05 (SS-21)	13.70	14.00	15	294	161	0.80	0.79	1.00	0.51	0.89	11.8	0.24	0.27	0.13	0.22	0.83	Silty sand	No liquefaction with 20 % fines
BH-401-05 (SS-22)	15.30	15.60	15	328	178	0.76	0.75	1.00	0.49	0.87	11.2	0.23	0.26	0.12	0.21	0.82	Silty sand	No liquefaction with 20 % fines
BH-501-05 (SS-1)	0.00	0.30	6	6	6	1.00	1.70	0.75	0.41	1.00	7.7	0.16	0.16	0.09	0.16	0.99	Sand	Above water level : no liquefaction
BH-501-05 (SS-2)	0.70	1.00	29	21	15	0.99	1.70	0.75	0.90	1.00	37.0	---	---	---	---	---	Sand	
BH-501-05 (SS-3)	1.50	1.80	17	38	24	0.99	1.70	0.75	0.69	1.00	21.7	0.25	0.25	0.24	0.41	1.63	Clayey soil	Clayey soil: no liquefaction
BH-501-05 (SS-4)	2.30	2.60	24	55	33	0.98	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-501-05 (SS-5)	3.10	3.40	24	71	42	0.97	1.54	0.80	0.80	1.00	29.6	0.27	0.27	0.44	0.77	2.85	Sand	
BH-501-05 (SS-6)	3.80	4.10	20	86	50	0.97	1.42	0.85	0.72	1.00	24.1	0.27	0.27	0.27	0.47	1.75	Sand	
BH-501-05 (SS-7)	4.60	4.90	22	103	59	0.96	1.30	0.85	0.73	1.00	24.4	0.27	0.27	0.28	0.48	1.77	Sand	
BH-501-05 (SS-8)	5.30	5.60	21	118	67	0.96	1.23	0.85	0.69	1.00	21.9	0.27	0.27	0.24	0.41	1.51	Sand	
BH-501-05 (SS-9)	6.10	6.40	47	134	76	0.95	1.15	0.95	1.06	1.00	51.4	---	---	---	---	---	Sand	
BH-501-05 (SS-10)	6.80	7.10	46	149	83	0.94	1.10	0.95	1.02	1.00	47.9	---	---	---	---	---	Sand	
BH-501-05 (SS-11)	7.60	7.90	136	166	92	0.94	1.04	0.95	1.71	1.00	134.5	---	---	---	---	---	Sand	
BH-501-05 (SS-12)	9.10	9.40	100	197	109	0.92	0.96	0.95	1.41	0.94	90.9	---	---	---	---	---	Sand	
BH-502-05 (SS-1)	0.00	0.30	7	6	6	1.00	1.70	0.75	0.44	1.00	8.9	0.16	0.16	0.10	0.18	1.10	Sand	Above water level : no liquefaction
BH-502-05 (SS-2)	0.70	1.00	64	21	19	0.99	1.70	0.75	1.33	1.00	81.6	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-3)	1.50	1.80	100	38	28	0.99	1.70	0.75	1.66	1.00	127.5	---	---	---	---	---	Sand	
BH-502-05 (SS-5)	2.20	2.50	47	53	36	0.98	1.67	0.75	1.13	1.00	58.9	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-6)	3.00	3.30	36	69	45	0.97	1.49	0.80	0.97	1.00	43.0	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-7)	3.80	4.10	25	86	54	0.97	1.36	0.85	0.79	1.00	29.0	0.25	0.25	0.41	0.71	2.81	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-8)	4.50	4.80	28	101	62	0.96	1.27	0.85	0.81	1.00	30.3	---	---	---	---	---	Sand	

Figure 2B
Soil liquefaction for 1:1000 years
Site Rabaska
a_{max} = 0.25g and magnitude of 6.25 on Richter scale

Borehole	Depth (m)	Depth (m)	N (blows / 0.3 m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	r_d (-)	C_N (-)	C_R	Dr	$K\sigma$	$(N_1)_{60}$ (blows / 0.3 m)	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
BH-503-05 (SS-1)	0.00	0.30	13	6	6	1.00	1.70	0.75	0.60	1.00	16.6	0.16	0.16	0.18	0.30	1.88	Sand	Above water level : no liquefaction
BH-503-05 (SS-2)	0.70	1.00	30	21	15	0.99	1.70	0.75	0.91	1.00	38.3	---	---	---	---	---	Sand	
BH-503-05 (SS-3)	1.50	1.80	40	38	24	0.99	1.70	0.75	1.05	1.00	51.0	---	---	---	---	---	Sand	
BH-503-05 (SS-4)	2.30	2.60	21	55	33	0.98	1.70	0.75	0.76	1.00	26.8	0.26	0.26	0.33	0.57	2.18	Sand	
BH-503-05 (SS-5)	3.10	3.40	74	71	42	0.97	1.54	0.80	1.41	1.00	91.4	---	---	---	---	---	Sand	
BH-503-05 (SS-7)	3.80	4.10	60	86	50	0.97	1.42	0.85	1.25	1.00	72.3	---	---	---	---	---	Sand	
BH-503-05 (SS-8)	4.50	4.80	52	101	58	0.96	1.32	0.85	1.13	1.00	58.2	---	---	---	---	---	Sand	
BH-503-05 (SS-10)	5.30	5.60	54	118	67	0.96	1.23	0.85	1.11	1.00	56.2	---	---	---	---	---	Sand	
BH-503-05 (SS-11)	6.10	6.40	34	134	76	0.95	1.15	0.95	0.90	1.00	37.2	---	---	---	---	---	Sand	
BH-503-05 (SS-12)	7.60	7.90	31	166	92	0.94	1.04	0.95	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-503-05 (SS-14)	9.20	9.50	100	200	110	0.92	0.95	0.95	1.40	0.93	90.5	---	---	---	---	---	Sand	
BH-503-05 (SS-17)	9.70	10.00	67	210	116	0.91	0.93	1.00	1.16	0.92	62.3	---	---	---	---	---	Sand	
BH-503-05 (SS-19)	10.50	10.80	13	227	125	0.89	0.90	1.00	0.50	0.95	11.6	0.26	0.28	0.13	0.22	0.80	Sand, some silt	**Liquefaction is possible with 10 % fines**
BH-503-05 (SS-20)	12.10	12.40	35	260	143	0.84	0.84	1.00	0.80	0.87	29.3	0.25	0.29	0.43	0.73	2.55	Sand	
BH-503-05 (SS-22)	13.60	13.90	47	292	159	0.80	0.79	1.00	0.90	0.81	37.2	---	---	---	---	---	Sand	
BH-503-05 (SS-23)	15.10	15.40	73	323	176	0.76	0.75	1.00	1.09	0.73	55.0	---	---	---	---	---	Sand	
BH-503-05 (SS-24)	16.60	16.90	30	355	193	0.72	0.72	1.00	0.69	0.80	21.6	0.22	0.27	0.24	0.41	1.51	Sand	
BH-503-05 (SS-26)	18.20	18.50	42	389	211	0.68	0.69	1.00	0.79	0.74	28.9	0.20	0.27	0.41	0.70	2.56	Sand	
BH-503-05 (SS-28)	19.70	20.00	26	420	228	0.64	0.66	1.00	0.61	0.78	17.2	0.19	0.25	0.18	0.32	1.28	Sand	
BH-503-05 (SS-29)	21.20	21.50	82	452	245	0.60	0.64	1.00	1.07	0.62	52.4	---	---	---	---	---	Sand	
BH-503-05 (SS-30)	22.80	23.10	100	485	262	0.56	0.62	1.00	1.16	0.57	61.7	---	---	---	---	---	Sand	
BH-503-05 (SS-33)	24.00	24.30	100	510	276	0.53	0.60	1.00	1.14	0.56	60.2	---	---	---	---	---	Sand	
BH-504-05 (SS-1)	0.00	0.30	5	6	6	1.00	1.70	0.75	0.37	1.00	6.4	0.16	0.16	0.08	0.14	0.88	Clayey soil	Above water level and clayey soil : no liquefaction
BH-504-05 (SS-2)	0.80	1.10	21	23	18	0.99	1.70	0.75	0.76	1.00	26.8	0.20	0.20	0.33	0.57	2.80	Sand	
BH-504-05 (SS-3)	1.50	1.80	24	38	26	0.99	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-504-05 (SS-4)	2.30	2.60	34	55	35	0.98	1.69	0.75	0.97	1.00	43.1	---	---	---	---	---	Sand	
BH-504-05 (SS-5)	3.00	3.30	86	69	43	0.97	1.53	0.80	1.51	1.00	105.1	---	---	---	---	---	Sand	
BH-504-05 (SS-7)	3.80	4.10	100	86	52	0.97	1.39	0.85	1.60	1.00	118.1	---	---	---	---	---	Sand	
BH-504-05 (SS-10)	4.90	5.20	100	109	64	0.96	1.25	0.85	1.52	1.00	106.2	---	---	---	---	---	Sand	
BH-504-05 (SS-12)	5.70	6.00	68	126	73	0.95	1.17	0.95	1.28	1.00	75.6	---	---	---	---	---	Sand	
BH-504-05 (SS-13)	6.10	6.40	100	134	78	0.95	1.14	0.95	1.53	1.00	107.9	---	---	---	---	---	Sand	

Figure 2B
Soil liquefaction for 1:1000 years
Site Rabaska
a_{max} = 0.25g and magnitude of 6.25 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	r_d	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-506-05 (SS-1)	0.00	0.30	6	6	6	1.00	1.70	0.75	0.41	1.00	7.7	0.16	0.16	0.09	0.16	0.99	Sand	Above water level : no liquefaction
BH-506-05 (SS-2)	0.80	1.10	24	23	16	0.99	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-506-05 (SS-3)	1.50	1.80	36	38	24	0.99	1.70	0.75	1.00	1.00	45.9	---	---	---	---	---	Sand	
BH-506-05 (SS-4)	2.30	2.60	44	55	33	0.98	1.70	0.75	1.10	1.00	56.1	---	---	---	---	---	Sand	
BH-506-05 (SS-6)	3.00	3.30	100	69	41	0.97	1.56	0.80	1.65	1.00	125.2	---	---	---	---	---	Sand	
BH-506-05 (SS-8)	3.80	4.10	32	86	50	0.97	1.42	0.85	0.92	1.00	38.5	---	---	---	---	---	Sand	
BH-506-05 (SS-9)	4.60	4.90	22	103	59	0.96	1.30	0.85	0.73	1.00	24.4	0.27	0.27	0.28	0.48	1.77	Sand	
BH-506-05 (SS-10)	5.30	5.60	29	118	67	0.96	1.23	0.85	0.81	1.00	30.2	---	---	---	---	---	Sand	
BH-506-05 (SS-11)	6.10	6.40	75	134	76	0.95	1.15	0.95	1.33	1.00	82.0	---	---	---	---	---	Sand	

N: blow count/0.3 m
σ_{vo} : total overburden stress
σ_{vo} : effective overburden stress
r_d: stress reduction coefficient
C_N: correction factor foroverburden pressure
C_R: correction factor for rod lenght
Dr : relative density
$K\sigma$: correction for high overburden stresses
$(N_1)_{60}$: correction of N
CSR: cyclic stress ratio
$CRR_{7.5}$: cyclic resistance ratio for magnitude 7.5
CRR_{corr}: cyclic resistance ratio for a different magnitude (MSF)
MSF: magnitude scaling factor

LEGEND	
Above ground water level	Depth < 9.15 m
Below ground water level	Depth > 9.15 m

Granular soil
Clayey soil

--- $(N1)_{60}$ >= 30 : No liquefaction

*** Factor of Safety: F.S.= ($CRR_{7.5}$/CSR)*MSF*$K\sigma$**

* Note: factor ofs safety calculated for 5% fine content

Figure 3A
Soil liquefaction - 1:2500 years
Site Rabaska
a_{max} = 0.45g and magnitude 6.5 on Richter scale

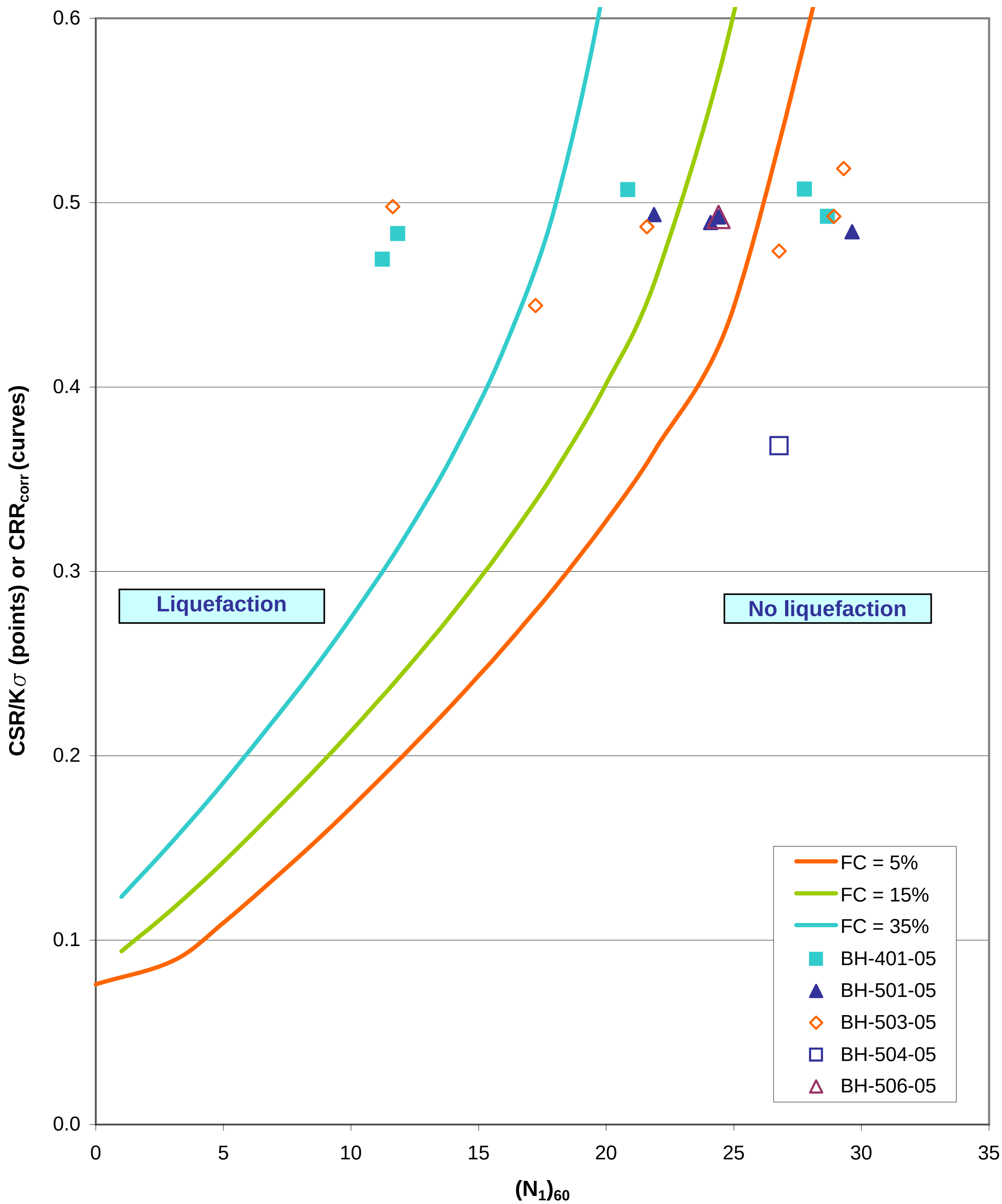

Figure 3B
Soil liquefaction for 1:2500 years
Site Rabaska
a_{max} = 0.45g and magnitude of 6.5 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	rd	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-401-05 (SS-1)	0.00	0.30	3	6	6	1.00	1.70	0.75	0.29	1.00	3.8	0.29	0.29	0.06	0.10	0.33	Sand	Above water level : no liquefaction
BH-401-05 (SS-2)	0.70	1.00	45	21	15	0.99	1.70	0.75	1.12	1.00	57.4	---	---	---	---	---	Sand	
BH-401-05 (SS-3)	1.50	1.80	37	38	24	0.99	1.70	0.75	1.01	1.00	47.2	---	---	---	---	---	Sand	
BH-401-05 (SS-4)	2.20	2.50	100	53	32	0.98	1.70	0.75	1.66	1.00	127.5	---	---	---	---	---	Sand	
BH-401-05 (SS-6)	3.00	3.30	100	69	41	0.97	1.56	0.80	1.65	1.00	125.2	---	---	---	---	---	Sand	
BH-401-05 (SS-8)	3.80	4.10	100	86	50	0.97	1.42	0.85	1.62	1.00	120.4	---	---	---	---	---	Sand	
BH-401-05 (SS-10)	4.60	4.90	54	103	59	0.96	1.30	0.85	1.14	1.00	59.9	---	---	---	---	---	Sand	
BH-401-05 (SS-11)	5.30	5.60	38	118	67	0.96	1.23	0.85	0.93	1.00	39.6	---	---	---	---	---	Sand	
BH-401-05 (SS-12)	6.10	6.40	34	134	76	0.95	1.15	0.95	0.90	1.00	37.2	---	---	---	---	---	Sand	
BH-401-05 (SS-13)	7.60	7.90	29	166	92	0.94	1.04	0.95	0.79	1.00	28.7	0.49	0.49	0.40	0.60	1.22	Sand	
BH-401-05 (SS-15)	9.00	9.30	100	195	108	0.93	0.96	0.95	1.41	0.95	91.4	---	---	---	---	---	Sand	
BH-401-05 (SS-17)	9.40	9.70	31	204	112	0.92	0.94	0.95	0.78	0.96	27.8	0.48	0.51	0.36	0.55	1.08	Sand	
BH-401-05 (SS-20)	12.20	12.50	25	263	144	0.84	0.83	1.00	0.67	0.88	20.8	0.45	0.51	0.23	0.34	0.68	Silty sand	**Liquefaction is possible with 20% fines**
BH-401-05 (SS-21)	13.70	14.00	15	294	161	0.80	0.79	1.00	0.51	0.89	11.8	0.43	0.48	0.13	0.20	0.41	Silty sand	**Liquefaction is possible with 20% fines**
BH-401-05 (SS-22)	15.30	15.60	15	328	178	0.76	0.75	1.00	0.49	0.87	11.2	0.41	0.47	0.12	0.19	0.40	Silty sand	**Liquefaction is possible with 20% fines**
BH-501-05 (SS-1)	0.00	0.30	6	6	6	1.00	1.70	0.75	0.41	1.00	7.7	0.29	0.29	0.09	0.14	0.48	Sand	Above water level : no liquefaction
BH-501-05 (SS-2)	0.70	1.00	29	21	15	0.99	1.70	0.75	0.90	1.00	37.0	---	---	---	---	---	Sand	
BH-501-05 (SS-3)	1.50	1.80	17	38	24	0.99	1.70	0.75	0.69	1.00	21.7	0.45	0.45	0.24	0.36	0.80	Clayey soil	Clayey soil: no liquefaction
BH-501-05 (SS-4)	2.30	2.60	24	55	33	0.98	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-501-05 (SS-5)	3.10	3.40	24	71	42	0.97	1.54	0.80	0.80	1.00	29.6	0.48	0.48	0.44	0.68	1.39	Sand	
BH-501-05 (SS-6)	3.80	4.10	20	86	50	0.97	1.42	0.85	0.72	1.00	24.1	0.49	0.49	0.27	0.42	0.85	Silt and sand	No liquefaction with > 35% fines
BH-501-05 (SS-7)	4.60	4.90	22	103	59	0.96	1.30	0.85	0.73	1.00	24.4	0.49	0.49	0.28	0.43	0.87	Silt and sand	No liquefaction with > 35% fines
BH-501-05 (SS-8)	5.30	5.60	21	118	67	0.96	1.23	0.85	0.69	1.00	21.9	0.49	0.49	0.24	0.37	0.74	Silt and sand	No liquefaction with > 35% fines
BH-501-05 (SS-9)	6.10	6.40	47	134	76	0.95	1.15	0.95	1.06	1.00	51.4	---	---	---	---	---	Sand	
BH-501-05 (SS-10)	6.80	7.10	46	149	83	0.94	1.10	0.95	1.02	1.00	47.9	---	---	---	---	---	Sand	
BH-501-05 (SS-11)	7.60	7.90	136	166	92	0.94	1.04	0.95	1.71	1.00	134.5	---	---	---	---	---	Sand	
BH-501-05 (SS-12)	9.10	9.40	100	197	109	0.92	0.96	0.95	1.41	0.94	90.9	---	---	---	---	---	Sand	
BH-502-05 (SS-1)	0.00	0.30	7	6	6	1.00	1.70	0.75	0.44	1.00	8.9	0.29	0.29	0.10	0.16	0.54	Sand	Above water level : no liquefaction
BH-502-05 (SS-2)	0.70	1.00	64	21	19	0.99	1.70	0.75	1.33	1.00	81.6	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-3)	1.50	1.80	100	38	28	0.99	1.70	0.75	1.66	1.00	127.5	---	---	---	---	---	Sand	
BH-502-05 (SS-5)	2.20	2.50	47	53	36	0.98	1.67	0.75	1.13	1.00	58.9	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-6)	3.00	3.30	36	69	45	0.97	1.49	0.80	0.97	1.00	43.0	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-7)	3.80	4.10	25	86	54	0.97	1.36	0.85	0.79	1.00	29.0	0.45	0.45	0.41	0.62	1.37	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-8)	4.50	4.80	28	101	62	0.96	1.27	0.85	0.81	1.00	30.3	---	---	---	---	---	Sand	

Figure 3B
Soil liquefaction for 1:2500 years
Site Rabaska
a_{max} = 0.45g and magnitude of 6.5 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	rd	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	$CSR/K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-503-05 (SS-1)	0.00	0.30	13	6	6	1.00	1.70	0.75	0.60	1.00	16.6	0.29	0.29	0.18	0.27	0.92	Sand	Above water level : no liquefaction
BH-503-05 (SS-2)	0.70	1.00	30	21	15	0.99	1.70	0.75	0.91	1.00	38.3	---	---	---	---	---	Sand	
BH-503-05 (SS-3)	1.50	1.80	40	38	24	0.99	1.70	0.75	1.05	1.00	51.0	---	---	---	---	---	Sand	
BH-503-05 (SS-4)	2.30	2.60	21	55	33	0.98	1.70	0.75	0.76	1.00	26.8	0.47	0.47	0.33	0.51	1.07	Sand	
BH-503-05 (SS-5)	3.10	3.40	74	71	42	0.97	1.54	0.80	1.41	1.00	91.4	---	---	---	---	---	Sand	
BH-503-05 (SS-7)	3.80	4.10	60	86	50	0.97	1.42	0.85	1.25	1.00	72.3	---	---	---	---	---	Sand	
BH-503-05 (SS-8)	4.50	4.80	52	101	58	0.96	1.32	0.85	1.13	1.00	58.2	---	---	---	---	---	Sand	
BH-503-05 (SS-10)	5.30	5.60	54	118	67	0.96	1.23	0.85	1.11	1.00	56.2	---	---	---	---	---	Sand	
BH-503-05 (SS-11)	6.10	6.40	34	134	76	0.95	1.15	0.95	0.90	1.00	37.2	---	---	---	---	---	Sand	
BH-503-05 (SS-12)	7.60	7.90	31	166	92	0.94	1.04	0.95	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-503-05 (SS-14)	9.20	9.50	100	200	110	0.92	0.95	0.95	1.40	0.93	90.5	---	---	---	---	---	Sand	
BH-503-05 (SS-17)	9.70	10.00	67	210	116	0.91	0.93	1.00	1.16	0.92	62.3	---	---	---	---	---	Sand	
BH-503-05 (SS-19)	10.50	10.80	13	227	125	0.89	0.90	1.00	0.50	0.95	11.6	0.47	0.50	0.13	0.19	0.39	Sand, some silt	**Liquefaction is possible with 10% fines**
BH-503-05 (SS-20)	12.10	12.40	35	260	143	0.84	0.84	1.00	0.80	0.87	29.3	0.45	0.52	0.43	0.65	1.25	Sand	
BH-503-05 (SS-22)	13.60	13.90	47	292	159	0.80	0.79	1.00	0.90	0.81	37.2	---	---	---	---	---	Sand	
BH-503-05 (SS-23)	15.10	15.40	73	323	176	0.76	0.75	1.00	1.09	0.73	55.0	---	---	---	---	---	Sand	
BH-503-05 (SS-24)	16.60	16.90	30	355	193	0.72	0.72	1.00	0.69	0.80	21.6	0.39	0.49	0.24	0.36	0.74	Sand, some silt	**Liquefaction is possible with 10% fines**
BH-503-05 (SS-26)	18.20	18.50	42	389	211	0.68	0.69	1.00	0.79	0.74	28.9	0.37	0.49	0.41	0.62	1.25	Sand	
BH-503-05 (SS-28)	19.70	20.00	26	420	228	0.64	0.66	1.00	0.61	0.78	17.2	0.35	0.44	0.18	0.28	0.63	Sand, some silt	**Liquefaction is possible with 10% fines**
BH-503-05 (SS-29)	21.20	21.50	82	452	245	0.60	0.64	1.00	1.07	0.62	52.4	---	---	---	---	---	Sand	
BH-503-05 (SS-30)	22.80	23.10	100	485	262	0.56	0.62	1.00	1.16	0.57	61.7	---	---	---	---	---	Sand	
BH-503-05 (SS-33)	24.00	24.30	100	510	276	0.53	0.60	1.00	1.14	0.56	60.2	---	---	---	---	---	Sand	
BH-504-05 (SS-1)	0.00	0.30	5	6	6	1.00	1.70	0.75	0.37	1.00	6.4	0.29	0.29	0.08	0.13	0.43	Clayey soil	Above water level and clayey soil : no liquefaction
BH-504-05 (SS-2)	0.80	1.10	21	23	18	0.99	1.70	0.75	0.76	1.00	26.8	0.37	0.37	0.33	0.51	1.37	Sand	
BH-504-05 (SS-3)	1.50	1.80	24	38	26	0.99	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-504-05 (SS-4)	2.30	2.60	34	55	35	0.98	1.69	0.75	0.97	1.00	43.1	---	---	---	---	---	Sand	
BH-504-05 (SS-5)	3.00	3.30	86	69	43	0.97	1.53	0.80	1.51	1.00	105.1	---	---	---	---	---	Sand	
BH-504-05 (SS-7)	3.80	4.10	100	86	52	0.97	1.39	0.85	1.60	1.00	118.1	---	---	---	---	---	Sand	
BH-504-05 (SS-10)	4.90	5.20	100	109	64	0.96	1.25	0.85	1.52	1.00	106.2	---	---	---	---	---	Sand	
BH-504-05 (SS-12)	5.70	6.00	68	126	73	0.95	1.17	0.95	1.28	1.00	75.6	---	---	---	---	---	Sand	
BH-504-05 (SS-13)	6.10	6.40	100	134	78	0.95	1.14	0.95	1.53	1.00	107.9	---	---	---	---	---	Sand	

Figure 3B
Soil liquefaction for 1:2500 years
Site Rabaska
a_{max} = 0.45g and magnitude of 6.5 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	rd	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-506-05 (SS-1)	0.00	0.30	6	6	6	1.00	1.70	0.75	0.41	1.00	7.7	0.29	0.29	0.09	0.14	0.48	Sand	Above water level : no liquefaction
BH-506-05 (SS-2)	0.80	1.10	24	23	16	0.99	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-506-05 (SS-3)	1.50	1.80	36	38	24	0.99	1.70	0.75	1.00	1.00	45.9	---	---	---	---	---	Sand	
BH-506-05 (SS-4)	2.30	2.60	44	55	33	0.98	1.70	0.75	1.10	1.00	56.1	---	---	---	---	---	Sand	
BH-506-05 (SS-6)	3.00	3.30	100	69	41	0.97	1.56	0.80	1.65	1.00	125.2	---	---	---	---	---	Sand	
BH-506-05 (SS-8)	3.80	4.10	32	86	50	0.97	1.42	0.85	0.92	1.00	38.5	---	---	---	---	---	Sand	
BH-506-05 (SS-9)	4.60	4.90	22	103	59	0.96	1.30	0.85	0.73	1.00	24.4	0.49	0.49	0.28	0.43	0.87	Silty Sand	No liquefaction with > 35% fines
BH-506-05 (SS-10)	5.30	5.60	29	118	67	0.96	1.23	0.85	0.81	1.00	30.2	---	---	---	---	---	Sand	
BH-506-05 (SS-11)	6.10	6.40	75	134	76	0.95	1.15	0.95	1.33	1.00	82.0	---	---	---	---	---	Sand	

N: blow count/0.3 m
σ_{vo} : total overburden stress
σ_{vo} : effective overburden stress
r_d: stress reduction coefficient
C_N: correction factor foroverburden pressure
C_R: correction factor for rod lenght
Dr : relative density
$K\sigma$: correction for high overburden stresses
$(N_1)_{60}$: correction of N
CSR: cyclic stress ratio
$CRR_{7.5}$: cyclic resistance ratio for magnitude 7.5
CRR_{corr}: cyclic resistance ratio for a different magnitude (MSF)
MSF: magnitude scaling factor

LEGEND	
Above ground water level	Depth < 9.15 m
Below ground water level	Depth > 9.15 m

Granular soil
Clayey soil

--- $(N_1)_{60}$ >= 30 : No liquefaction

*** Factor of Safety F.S.= ($CRR_{7.5}$/CSR)*MSF*$K\sigma$**

Note: factor of safety calculated for 5% fine content

Figure 4A
Soil liquefaction - 1:5000 years
Site Rabaska
a_{max} = 0.64g and magnitude 6.75 on Richter scale

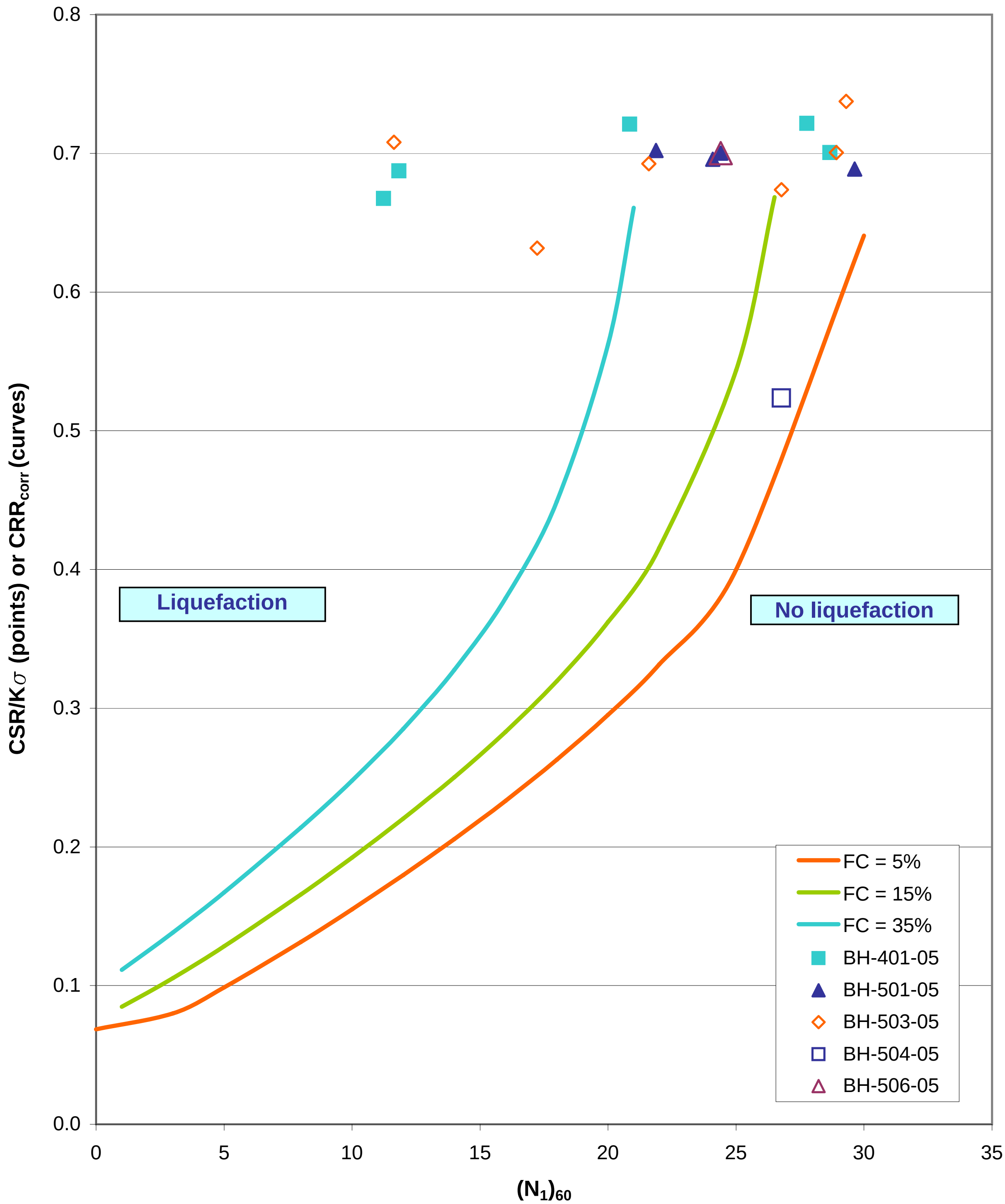

Figure 4B
Soil liquefaction for 1:5000 years
Site Rabaska
a_{max} = 0.64g and magnitude of 6.75 on Richter scale

Borehole	Depth (m)	Depth (m)	N (blows / 0.3 m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	rd (-)	C_N (-)	C_R	Dr	$K\sigma$	$(N_1)_{60}$ (blows / 0.3 m)	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
BH-401-05 (SS-1)	0.00	0.30	3	6	6	1.00	1.70	0.75	0.29	1.00	3.8	0.42	0.42	0.06	0.09	0.21	Sand	Above water level : no liquefaction
BH-401-05 (SS-2)	0.70	1.00	45	21	15	0.99	1.70	0.75	1.12	1.00	57.4	---	---	---	---	---	Sand	
BH-401-05 (SS-3)	1.50	1.80	37	38	24	0.99	1.70	0.75	1.01	1.00	47.2	---	---	---	---	---	Sand	
BH-401-05 (SS-4)	2.20	2.50	100	53	32	0.98	1.70	0.75	1.66	1.00	127.5	---	---	---	---	---	Sand	
BH-401-05 (SS-6)	3.00	3.30	100	69	41	0.97	1.56	0.80	1.65	1.00	125.2	---	---	---	---	---	Sand	
BH-401-05 (SS-8)	3.80	4.10	100	86	50	0.97	1.42	0.85	1.62	1.00	120.4	---	---	---	---	---	Sand	
BH-401-05 (SS-10)	4.60	4.90	54	103	59	0.96	1.30	0.85	1.14	1.00	59.9	---	---	---	---	---	Sand	
BH-401-05 (SS-11)	5.30	5.60	38	118	67	0.96	1.23	0.85	0.93	1.00	39.6	---	---	---	---	---	Sand	
BH-401-05 (SS-12)	6.10	6.40	34	134	76	0.95	1.15	0.95	0.90	1.00	37.2	---	---	---	---	---	Sand	
BH-401-05 (SS-13)	7.60	7.90	29	166	92	0.94	1.04	0.95	0.79	1.00	28.7	0.70	0.70	0.40	0.54	0.77	Sand, some silt	**Liquefaction is possible with 10 % fines**
BH-401-05 (SS-15)	9.00	9.30	100	195	108	0.93	0.96	0.95	1.41	0.95	91.4	---	---	---	---	---	Sand	
BH-401-05 (SS-17)	9.40	9.70	31	204	112	0.92	0.94	0.95	0.78	0.96	27.8	0.69	0.72	0.36	0.50	0.69	Sand, some silt	**Liquefaction is possible with 10 % fines**
BH-401-05 (SS-20)	12.20	12.50	25	263	144	0.84	0.83	1.00	0.67	0.88	20.8	0.64	0.72	0.23	0.31	0.43	Silty sand	**Liquefaction is possible with 20% fines**
BH-401-05 (SS-21)	13.70	14.00	15	294	161	0.80	0.79	1.00	0.51	0.89	11.8	0.61	0.69	0.13	0.18	0.26	Siltty sand	**Liquefaction is possible with 20% fines**
BH-401-05 (SS-22)	15.30	15.60	15	328	178	0.76	0.75	1.00	0.49	0.87	11.2	0.58	0.67	0.12	0.17	0.25	Silty sand	**Liquefaction is possible with 20% fines**
BH-501-05 (SS-1)	0.00	0.30	6	6	6	1.00	1.70	0.75	0.41	1.00	7.7	0.42	0.42	0.09	0.13	0.31	Sand	Above water level : no liquefaction
BH-501-05 (SS-2)	0.70	1.00	29	21	15	0.99	1.70	0.75	0.90	1.00	37.0	---	---	---	---	---	Sand	
BH-501-05 (SS-3)	1.50	1.80	17	38	24	0.99	1.70	0.75	0.69	1.00	21.7	0.64	0.64	0.24	0.33	0.51	Clayey soil	Clayey soil: no liquefaction
BH-501-05 (SS-4)	2.30	2.60	24	55	33	0.98	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-501-05 (SS-5)	3.10	3.40	24	71	42	0.97	1.54	0.80	0.80	1.00	29.6	0.69	0.69	0.44	0.61	0.88	Silt and sand	No liquefaction with > 35% fines
BH-501-05 (SS-6)	3.80	4.10	20	86	50	0.97	1.42	0.85	0.72	1.00	24.1	0.70	0.70	0.27	0.38	0.54	Silt and sand	No liquefaction with > 35% fines
BH-501-05 (SS-7)	4.60	4.90	22	103	59	0.96	1.30	0.85	0.73	1.00	24.4	0.70	0.70	0.28	0.38	0.55	Silt and sand	No liquefaction with > 35% fines
BH-501-05 (SS-8)	5.30	5.60	21	118	67	0.96	1.23	0.85	0.69	1.00	21.9	0.70	0.70	0.24	0.33	0.47	Silt and sand	No liquefaction with > 35% fines
BH-501-05 (SS-9)	6.10	6.40	47	134	76	0.95	1.15	0.95	1.06	1.00	51.4	---	---	---	---	---	Sand	
BH-501-05 (SS-10)	6.80	7.10	46	149	83	0.94	1.10	0.95	1.02	1.00	47.9	---	---	---	---	---	Sand	
BH-501-05 (SS-11)	7.60	7.90	136	166	92	0.94	1.04	0.95	1.71	1.00	134.5	---	---	---	---	---	Sand	
BH-501-05 (SS-12)	9.10	9.40	100	197	109	0.92	0.96	0.95	1.41	0.94	90.9	---	---	---	---	---	Sand	
BH-502-05 (SS-1)	0.00	0.30	7	6	6	1.00	1.70	0.75	0.44	1.00	8.9	0.42	0.42	0.10	0.14	0.34	Sand	Above water level : no liquefaction
BH-502-05 (SS-2)	0.70	1.00	64	21	19	0.99	1.70	0.75	1.33	1.00	81.6	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-3)	1.50	1.80	100	38	28	0.99	1.70	0.75	1.66	1.00	127.5	---	---	---	---	---	Sand	
BH-502-05 (SS-5)	2.20	2.50	47	53	36	0.98	1.67	0.75	1.13	1.00	58.9	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-6)	3.00	3.30	36	69	45	0.97	1.49	0.80	0.97	1.00	43.0	---	---	---	---	---	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-7)	3.80	4.10	25	86	54	0.97	1.36	0.85	0.79	1.00	29.0	0.64	0.64	0.41	0.56	0.87	Clayey soil	Clayey soil: no liquefaction
BH-502-05 (SS-8)	4.50	4.80	28	101	62	0.96	1.27	0.85	0.81	1.00	30.3	---	---	---	---	---	Sand	

Figure 4B
Soil liquefaction for 1:5000 years
Site Rabaska
a_{max} = 0.64g and magnitude of 6.75 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	rd	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-503-05 (SS-1)	0.00	0.30	13	6	6	1.00	1.70	0.75	0.60	1.00	16.6	0.42	0.42	0.18	0.24	0.58	Sand	Above water level : no liquefaction
BH-503-05 (SS-2)	0.70	1.00	30	21	15	0.99	1.70	0.75	0.91	1.00	38.3	---	---	---	---	---	Sand	
BH-503-05 (SS-3)	1.50	1.80	40	38	24	0.99	1.70	0.75	1.05	1.00	51.0	---	---	---	---	---	Sand	
BH-503-05 (SS-4)	2.30	2.60	21	55	33	0.98	1.70	0.75	0.76	1.00	26.8	0.67	0.67	0.33	0.46	0.68	Gravelly silt	No liquefaction with > 35% fines
BH-503-05 (SS-5)	3.10	3.40	74	71	42	0.97	1.54	0.80	1.41	1.00	91.4	---	---	---	---	---	Sand	
BH-503-05 (SS-7)	3.80	4.10	60	86	50	0.97	1.42	0.85	1.25	1.00	72.3	---	---	---	---	---	Sand	
BH-503-05 (SS-8)	4.50	4.80	52	101	58	0.96	1.32	0.85	1.13	1.00	58.2	---	---	---	---	---	Sand	
BH-503-05 (SS-10)	5.30	5.60	54	118	67	0.96	1.23	0.85	1.11	1.00	56.2	---	---	---	---	---	Sand	
BH-503-05 (SS-11)	6.10	6.40	34	134	76	0.95	1.15	0.95	0.90	1.00	37.2	---	---	---	---	---	Sand	
BH-503-05 (SS-12)	7.60	7.90	31	166	92	0.94	1.04	0.95	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-503-05 (SS-14)	9.20	9.50	100	200	110	0.92	0.95	0.95	1.40	0.93	90.5	---	---	---	---	---	Sand	
BH-503-05 (SS-17)	9.70	10.00	67	210	116	0.91	0.93	1.00	1.16	0.92	62.3	---	---	---	---	---	Sand	
BH-503-05 (SS-19)	10.50	10.80	13	227	125	0.89	0.90	1.00	0.50	0.95	11.6	0.67	0.71	0.13	0.18	0.25	Sand, some silt	**Liquefaction is possible with 10% fines**
BH-503-05 (SS-20)	12.10	12.40	35	260	143	0.84	0.84	1.00	0.80	0.87	29.3	0.64	0.74	0.43	0.58	0.79	Sand, some silt	**Liquefaction is possible with 10% fines**
BH-503-05 (SS-22)	13.60	13.90	47	292	159	0.80	0.79	1.00	0.90	0.81	37.2	---	---	---	---	---	Sand	
BH-503-05 (SS-23)	15.10	15.40	73	323	176	0.76	0.75	1.00	1.09	0.73	55.0	---	---	---	---	---	Sand	
BH-503-05 (SS-24)	16.60	16.90	30	355	193	0.72	0.72	1.00	0.69	0.80	21.6	0.55	0.69	0.24	0.32	0.47	Sand, some silt	**Liquefaction is possible with 10% fines**
BH-503-05 (SS-26)	18.20	18.50	42	389	211	0.68	0.69	1.00	0.79	0.74	28.9	0.52	0.70	0.41	0.56	0.79	Sand, some silt	**Liquefaction is possible with 10% fines**
BH-503-05 (SS-28)	19.70	20.00	26	420	228	0.64	0.66	1.00	0.61	0.78	17.2	0.49	0.63	0.18	0.25	0.40	Sand, some silt	**Liquefaction is possible with 10% fines**
BH-503-05 (SS-29)	21.20	21.50	82	452	245	0.60	0.64	1.00	1.07	0.62	52.4	---	---	---	---	---	Sand	
BH-503-05 (SS-30)	22.80	23.10	100	485	262	0.56	0.62	1.00	1.16	0.57	61.7	---	---	---	---	---	Sand	
BH-503-05 (SS-33)	24.00	24.30	100	510	276	0.53	0.60	1.00	1.14	0.56	60.2	---	---	---	---	---	Sand	
BH-504-05 (SS-1)	0.00	0.30	5	6	6	1.00	1.70	0.75	0.37	1.00	6.4	0.42	0.42	0.08	0.11	0.27	Clayey soil	Above water level and clayey soil : no liquefaction
BH-504-05 (SS-2)	0.80	1.10	21	23	18	0.99	1.70	0.75	0.76	1.00	26.8	0.52	0.52	0.33	0.46	0.87	Silty sand	No liquefaction with > 20% fines
BH-504-05 (SS-3)	1.50	1.80	24	38	26	0.99	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-504-05 (SS-4)	2.30	2.60	34	55	35	0.98	1.69	0.75	0.97	1.00	43.1	---	---	---	---	---	Sand	
BH-504-05 (SS-5)	3.00	3.30	86	69	43	0.97	1.53	0.80	1.51	1.00	105.1	---	---	---	---	---	Sand	
BH-504-05 (SS-7)	3.80	4.10	100	86	52	0.97	1.39	0.85	1.60	1.00	118.1	---	---	---	---	---	Sand	
BH-504-05 (SS-10)	4.90	5.20	100	109	64	0.96	1.25	0.85	1.52	1.00	106.2	---	---	---	---	---	Sand	
BH-504-05 (SS-12)	5.70	6.00	68	126	73	0.95	1.17	0.95	1.28	1.00	75.6	---	---	---	---	---	Sand	
BH-504-05 (SS-13)	6.10	6.40	100	134	78	0.95	1.14	0.95	1.53	1.00	107.9	---	---	---	---	---	Sand	

Figure 4B
Soil liquefaction for 1:5000 years
Site Rabaska
a_{max} = 0.64g and magnitude of 6.75 on Richter scale

Borehole	Depth	Depth	N	σ_{vo}	σ_{vo}'	rd	C_N	C_R	Dr	$K\sigma$	$(N_1)_{60}$	CSR	CSR/$K\sigma$	$CRR_{7.5}$	CRR_{corr}	Factor of Safety *	Description	Remarks
	m	m	blows / 0.3 m	kPa	kPa	-	-				blows / 0.3 m							
BH-506-05 (SS-1)	0.00	0.30	6	6	6	1.00	1.70	0.75	0.41	1.00	7.7	0.42	0.42	0.09	0.13	0.31	Sand	Above water level : no liquefaction
BH-506-05 (SS-2)	0.80	1.10	24	23	16	0.99	1.70	0.75	0.82	1.00	30.6	---	---	---	---	---	Sand	
BH-506-05 (SS-3)	1.50	1.80	36	38	24	0.99	1.70	0.75	1.00	1.00	45.9	---	---	---	---	---	Sand	
BH-506-05 (SS-4)	2.30	2.60	44	55	33	0.98	1.70	0.75	1.10	1.00	56.1	---	---	---	---	---	Sand	
BH-506-05 (SS-6)	3.00	3.30	100	69	41	0.97	1.56	0.80	1.65	1.00	125.2	---	---	---	---	---	Sand	
BH-506-05 (SS-8)	3.80	4.10	32	86	50	0.97	1.42	0.85	0.92	1.00	38.5	---	---	---	---	---	Sand	
BH-506-05 (SS-9)	4.60	4.90	22	103	59	0.96	1.30	0.85	0.73	1.00	24.4	0.70	0.70	0.28	0.38	0.55	Silty Sand	No liquefaction with > 35% fines
BH-506-05 (SS-10)	5.30	5.60	29	118	67	0.96	1.23	0.85	0.81	1.00	30.2	---	---	---	---	---	Sand	
BH-506-05 (SS-11)	6.10	6.40	75	134	76	0.95	1.15	0.95	1.33	1.00	82.0	---	---	---	---	---	Sand	

N: blow count/0.3 m
σ_{vo} : total overburden stress
σ_{vo} : effective overburden stress
r_d: stress reduction coefficient
C_N: correction factor foroverburden pressure
C_R: correction factor for rod lenght
Dr : relative density
$K\sigma$: correction for high overburden stresses
$(N_1)_{60}$: correction of N
CSR: cyclic stress ratio
$CRR_{7.5}$: cyclic resistance ratio for magnitude 7.5
CRR_{corr}: cyclic resistance ratio for a different magnitude (MSF)
MSF: magnitude scaling factor

LEGEND	
Above ground water level	Depth < 9.15 m
Below ground water level	Depth > 9.15 m

Granular soil
Clayey soil

--- $(N_1)_{60}$ >= 30 : No liquefaction

*** Factor of Safety F.S.= ($CRR_{7.5}$/CSR)*MSF*$K\sigma$**

Note: factor of safety calculated for 5% fine content

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ANNEXE C

Rapport préliminaire de paléosismologie

Preliminary Report

PALEOSEISMIC INVESTIGATION OF LONG-TERM RATES OF LARGE EARTHQUAKES IN THE CHARLEVOIX AND PROPOSED RABASKA SITE AREAS

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October 9, 2006

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PALEOSEISMIC INVESTIGATION OF LONG-TERM RATES OF LARGE EARTHQUAKES IN THE CHARLEVOIX AND PROPOSED RABASKA SITE AREAS

Executive Summary

Earthquake-induced paleoliquefaction features found along the Gouffre River in the Charlevoix area formed between A.D. 780 and 7590 B.C. (or 1170 and 9540 years B.P.). They indicate that at least one large Holocene paleoearthquake was centered in the Charlevoix area. The event was probably larger than the 1988 **M** 5.9 Saguenay earthquake and possibly larger than the 1925 **M** 6.2 Charlevoix earthquake. No Holocene paleoliquefaction features have been found along the Etchemin or Jacques-Cartier Rivers in the region of the proposed site for the Rabaska LNG terminal. Sediments that are susceptible to liquefaction do occur in the Rabaska site region and are likely to contain a record of large local Holocene earthquakes, if they occurred. Unfortunately, cutbank exposures were relatively poor at the time of reconnaissance. Given the current lack of paleoseismic data for the Rabaska site area, it would be premature to draw comparisons with the paleoseismic record for the Charlevoix area. This investigation has been limited by the small number of rivers surveyed in both areas and by the poor exposure along rivers in the site area. To realize the full potential of this paleoseismic investigation, an additional effort is needed to locate areas where liquefiable sediments and good exposures of Holocene deposits occur and to search those exposures for earthquake-induced liquefaction features. A better understanding of the distribution of paleoliquefaction features will help to assess whether or not the Charlevoix seismic zone extends into the Rabaska site area.

1. INTRODUCTION

This report presents a reconnaissance-level paleoseismic investigation to help define seismic source zones relevant to the design of the proposed Rabaska LNG terminal near Levis, Quebec. This investigation begins to address the question of long-term rates of large earthquakes in the Charlevoix seismic zone and Rabaska site area located about 70 km to the southwest. During this study, rivers are evaluated for presence and exposure of potentially liquefiable Quaternary sediments, searches for earthquake-induced liquefaction features are conducted along selected portions of rivers, sites where liquefaction features do and do not occur are documented, samples of organic material from these sites are dated to provide age control of sediments and liquefaction features, and observations are interpreted in terms of timing, magnitude, and source areas of paleoearthquakes and their implications for seismic source zones.

1.1 Purpose

The purpose of this paleoseismic study is to provide information about the approximate timing, magnitude, and source area of large Holocene earthquakes in the Charlevoix and Rabaska site areas in order to reduce uncertainties in seismic source models used in a site-specific earthquake hazard assessment of the proposed Rabaska LNG terminal site near Levis. As noted in the earthquake hazard assessment of the proposed Rabaska site, the seismicity rate is lower in the site area than the Charlevoix area but higher than in other areas of the St. Lawrence and Ottawa valleys (Atkinson, 2006). In addition, the study points out that the relationship of seismicity to major faults in the Charlevoix region is unclear and the geographical extent of faults responsible for Charlevoix seismicity uncertain. To represent the geological uncertainty of the seismic source zones, several models are considered in the hazard assessment, including one that limits the extent of the Charlevoix source zone to the area of historically high levels of seismicity and another that broadens the Charlevoix source zone to include the area of higher than average levels of seismicity near Levis. The report recommends that a paleoseismic investigation be conducted in the Charlevoix and Rabaska site areas to determine if the recurrence rate of large earthquakes is lower in the site area than in the Charlevoix area. If there were evidence in the Holocene (past 10,000 yr) geologic record for repeated large earthquakes in the Charlevoix area but not in the Rabaska site area, the seismic source model that extends the Charlevoix zone into the site area would receive less weight in the hazard assessment.

1.2 Scope of Work

The primary goals of this study are to evaluate whether or not sites conditions are suitable for paleoliquefaction investigation in the Charlevoix-Rabaska site region and to compare geologic records of strong ground shaking in the Charlevoix and Rabaska site areas during the Holocene. The scope of work includes the following tasks:

- gather background geological and geotechnical information relevant to the project:
- review the background information and develop a plan for field work;
- conduct field work in the Charlevoix-Rabaska site region to select sections of three rivers for reconnaissance and to document the presence and/or absence of liquefiable sediments and earthquake-induced liquefaction features;
- perform radiocarbon dating of organic samples for the purpose of estimating the ages of sediments, liquefaction features, and paleoearthquakes;
- analyze results of field work and radiocarbon dating in terms of timing, magnitude, and source areas of paleoearthquakes and their implications for seismic source zones; and
- prepare this report summarizing results of the paleoseismic study and making recommendations regarding further study.

2. REGIONAL SEISMOTECTONIC SETTING

The proposed site of the Rabaska LNG terminal in Levis, Quebec is located along the Appalachian Front, coincident with the St. Lawrence River in the site area (Figure 1; Douglas, 1969, 1972, 1973). This region has a long and complicated tectonic history punctuated by four major tectonic events. These events include the Grenvillian collision (1100 to 900 Ma), rifting related to opening of the Iapetus Ocean (700 Ma), Taconic orogeny related to closing of the Iapetus Ocean (450 Ma), and a Devonian meteor impact (350 MA) (Kumarapeli and Saull, 1966; Rondot, 1979; Lamontagne et al., 2000). Northwest of the Appalachian Front, faults resulting from these tectonic events can be observed in Precambrian granitic gneisses, granulite, and charnockite (Figure 2). Faults related to Iapetan rifting are parallel to the St. Lawrence River. Under the St. Lawrence River and to the southeast of the front, the Iapetan faults are overlain by several kilometers of folded Cambrian and Ordovician sedimentary strata.

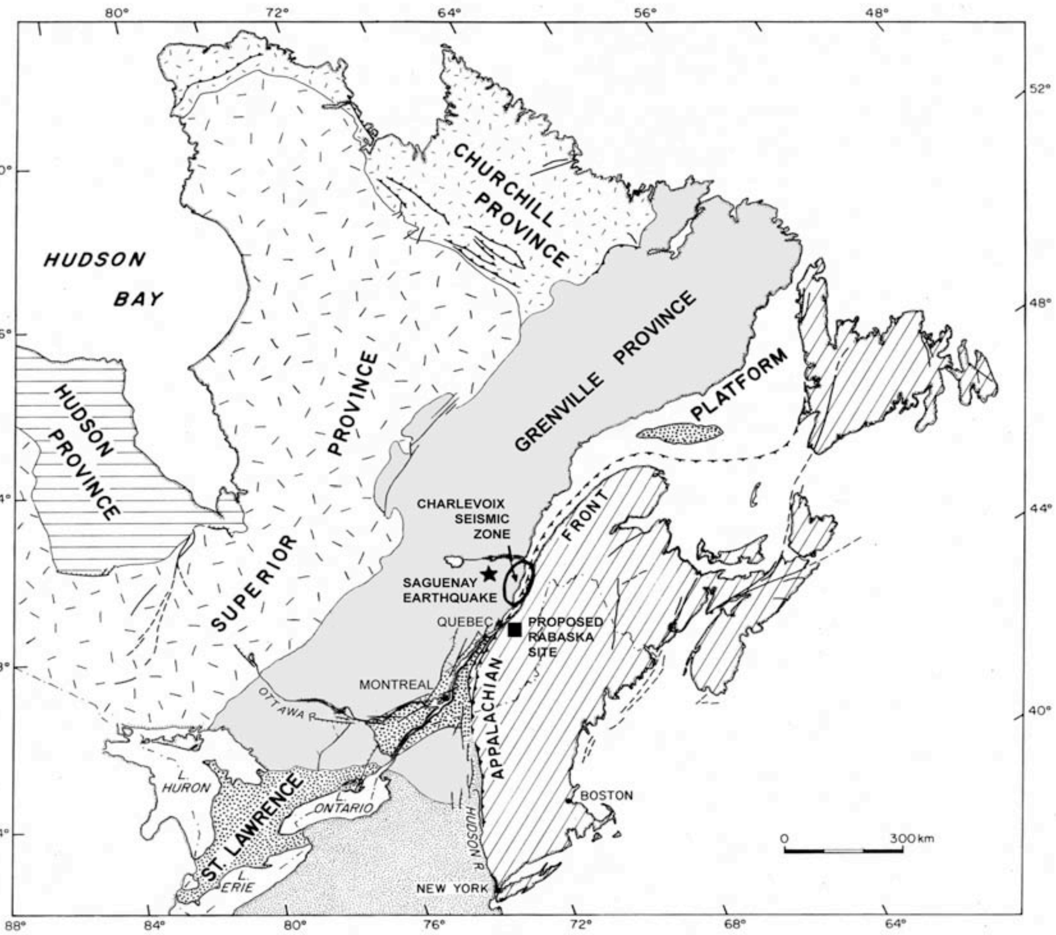


Figure 1. Geological provinces of eastern Canada and northeastern United States, showing locations of Charlevoix seismic zone, 1988 Saguenay earthquake, and proposed Rabaska site in Quebec (from Tuttle et al., 1992; after Douglas, 1973).

The Charlevoix seismic zone is one of the most seismically active areas in eastern North America (e.g., Adams and Basham, 1989; Figure 1). The seismic zone has generated five earthquakes greater than moment magnitude (**M**) 6 since 1663 and thousands of small earthquakes since 1977 (Lamontagne et al., 2000). The Quebec City area, including the Rabaska site, is not as seismically active as the Charlevoix area but is more active than other portions of the St. Lawrence and Ottawa valleys (Atkinson, 2006). Throughout the region it is difficult to correlate earthquakes with specific faults (Figure 2). In the Charlevoix seismic zone, earthquakes occur from the surface to 30 km depth, with most between 7 and 15 km (Lamontagne et al., 2000). In the Appalachian province, earthquakes occur below the Paleozoic strata at depths of 5 to 20 km in Precambrian rock.

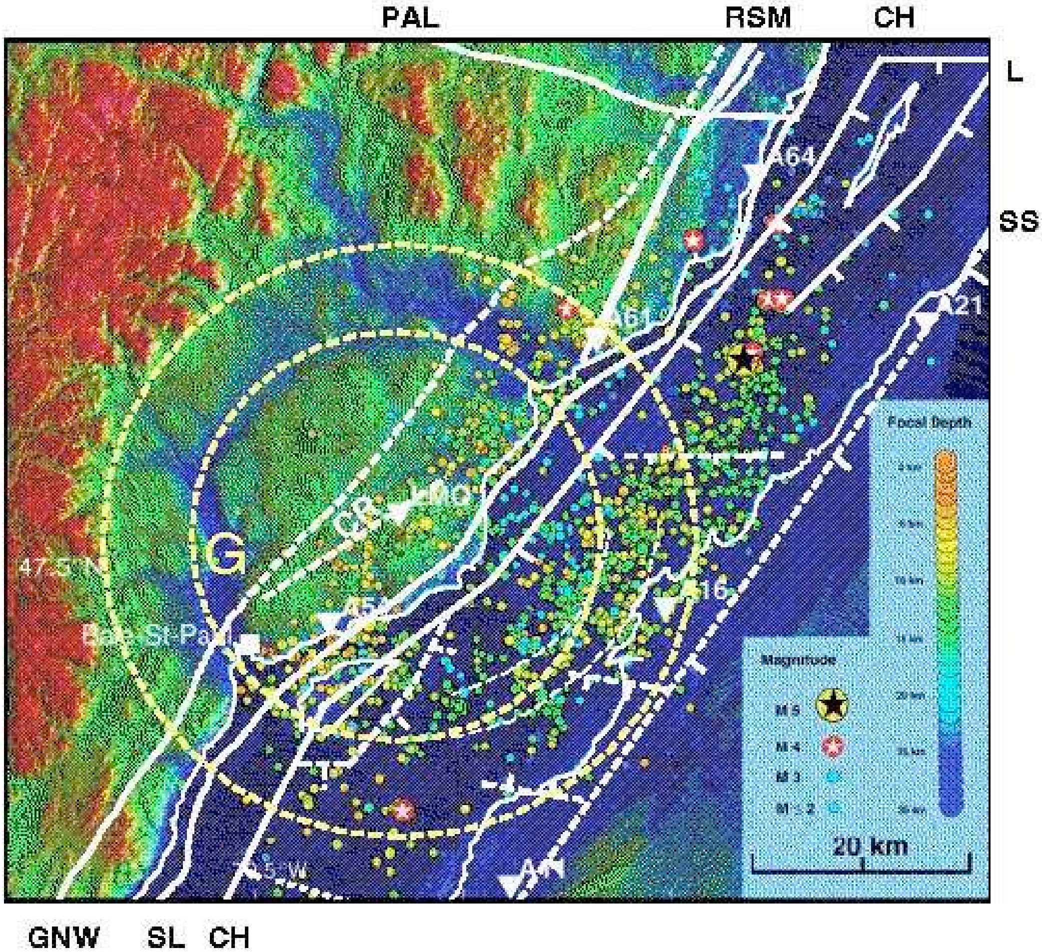


Figure 2. Combined RADARSAT-SAR ortho-image and terrain elevation, showing interpreted structures of Charlevoix seismic zone: RSM - Rang Sainte-Mathilde fault; SL - Saint-Laurent fault; CH - Charlevoix fault; L - Lièvres fault; SS - South Shore fault; G - peripheral graben of the impact structure; CR - crater fault; GNW - Gouffre NW fault; PAL - Palissades fault. Earthquake hypocenters (circles) are from January 1978 to September 1999, with colors reflecting focal depth. White triangles are station locations of the local and national seismograph network (from Lamontagne et al., 2000).

The reason for the high seismicity rate in the Charlevoix area is a topic of much debate. Some investigators think that the earthquakes are caused by reactivation of regional Iapetan faults in a uniform continent-wide stress field (e.g., Adams and Basham, 1989), while others think that the impact crater contributes to the high seismicity rate (e.g., Anglin, 1984; Lamontagne et al., 2000). The two models have significantly different implications for seismic hazards along the St. Lawrence valley. Little is known about the long-term behavior of the faults in the Charlevoix and

Rabaska site areas. Paleoseismology, which attempts to extend earthquake history back in time, has the potential to address this question.

3. PREVIOUS PALEOSEISMIC STUDIES IN THE REGION

Only a few paleoseismic studies, all of limited geographical scope, have been conducted in the Charlevoix-Rabaska site region. Nevertheless, the studies found a geologic record of prehistoric earthquakes that could help to characterize Holocene seismicity and earthquake potential. Studies of sediment cores from a few lakes in the Laurentide Mountains correlated anomalous silt layers with modern and historic earthquakes and attributed other silt layers deeper in the sections to prehistoric earthquakes (Doig, 1990 and 1998). The lake core studies suggested that there had been more frequent earthquakes in the Charlevoix area than in the Saguenay area during the Late Holocene. In addition, studies of landslides in the Saint-Maurice River valley near Shawinigan and in the Gouffre River valley near Baie St. Paul attributed mass movements to the 1663 Charlevoix earthquake and possibly to other events during the Holocene (Figure 2; Desjardins, 1980; Filion et al., 1990).

Accounts of the 1870 and 1925 Charlevoix earthquakes describe ground failure indicative of liquefaction in the Gouffre River valley (Smith, 1966), probably in sandy portions of Holocene fluvial and Late Wisconsin glaciomarine deposits (Rondot, 1972). Investigators excavated a large (2 m high and 10 m in diameter) mound in the Gouffre River valley and interpreted it to be a sand volcano resulting from seismic activity (Chagnon and Locat, 1988). Subsequent excavation of a nearby mound revealed blocks of tilted and folded stratified deposits (Tuttle, 1994). The mound was interpreted as an erosional remnant of a landslide block derived from the nearby hillslope. Radiocarbon dating of organic material buried beneath the block yielded a calibrated date of A.D. 1210-1400, indicating that the landslide occurred after A.D. 1210, possibly during the 1663 Charlevoix earthquake thought to be centered near La Malbaie about 30 km northeast of Baie-Saint Paul (Figure 2).

The 1988 **M** 5.9 Saguenay earthquake, centered about 50 km northwest of the Charlevoix seismic zone in the Laurentide Mountains, induced liquefaction up to 30 km from its epicenter.

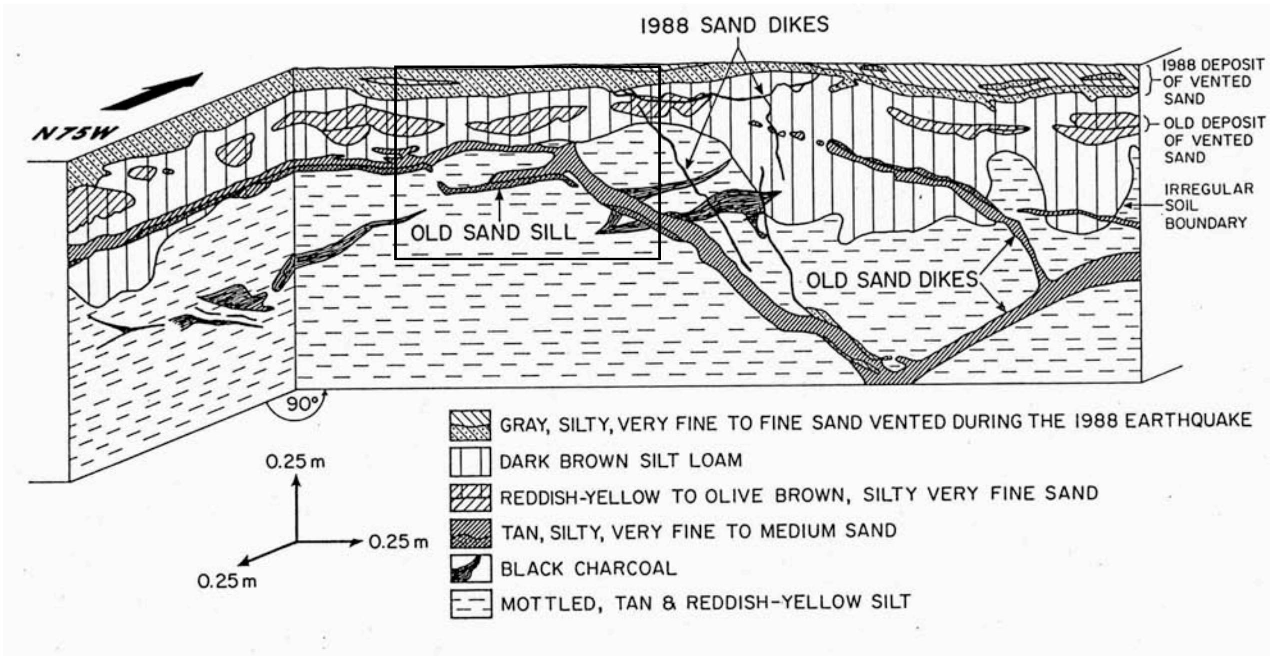
Liquefaction occurred in Holocene fluvial and Late Wisconsin glaciofluvial and glaciolacustrine deposits in the Ferland-Boilleau valley (Figure 1; Tuttle et al., 1990). During excavation and documentation of the liquefaction features, investigators found liquefaction evidence for a prior earthquake (Figure 3; Tuttle et al., 1992; Tuttle, 1994). Radiocarbon dating of the paleoliquefaction features indicated that a large earthquake occurred in the region in A.D. 1420 ± 200 yr. Given the relative size of the two generations of features, the previous event may have been larger or located closer to the Ferland-Boilleau valley than the 1988 Saguenay earthquake.

4. METHODOLOGY

Paleoseismology, or the study of past earthquakes as preserved in the geologic record, extends our knowledge of seismic activity into the prehistoric period, and thereby improves our understanding of the long-term behavior of fault zones or earthquake sources. Paleoseismology has proven especially useful in regions where the historical record of earthquakes is short and where strain rates are relatively low and recurrence intervals of large earthquakes relatively long.

In eastern North America, where surface traces of seismogenic faults are uncommon or difficult to identify, many paleoseismic studies have employed earthquake-induced liquefaction features. Other studies have used landslides, subaqueous slumps, and siltation layers in lacustrine deposits to infer paleoearthquakes, but there is greater uncertainties regarding the triggering mechanism of these features. Notable paleoliquefaction studies include those in the New Madrid seismic zone, source of the 1811-1812 **M** 7.5-8 earthquakes, in the central United States (e.g., Tuttle et al., 1996, 2002, 2005), the Charleston seismic zone, source of the 1886 **M** 7.3 earthquake in South Carolina (e.g., Amick et al., 1990; Talwani and Schaeffer, 2001), and the Wabash Valley seismic zone in southern Indiana and Illinois (e.g., Munson et al., 1997; Obermeier et al., 1993). Established methods for dating liquefaction features and estimating the timing, magnitude, and source areas of paleoearthquakes will be used in this paleoseismic investigation in southeastern Quebec. For more complete reviews of methods used in paleoliquefaction studies, the reader is referred to Obermeier (1996) and Tuttle (2001).

A



B

Figure 3. Two generations of earthquake-induced liquefaction features in Ferland, Quebec (Tuttle, 1994). Older features are more weathered and cut by younger sand dikes. A) Log of liquefaction features including sand blows and sand dikes exposed in excavation. B) Photograph of portion of trench wall outlined by box in A).

During a regional paleoliquefaction study, it is critical to narrowly constrain ages of liquefaction features so that they can be correlated over large distances and used to estimate the source areas and magnitudes of paleoearthquakes. Also, the better the age control on individual liquefaction features, the smaller the uncertainties associated with the timing, and thus recurrence times, of earthquakes. This is especially important where large earthquakes occur fairly frequently or there are multiple earthquake sources. Radiocarbon analysis is the most common dating technique used in paleoliquefaction studies. Analysis of artifacts found in soil horizons bounding liquefaction features can be employed in regions where ceramic and projectile point chronologies are well established. In addition, soil development within liquefaction features and bounding horizons can help to estimate the age of liquefaction features.

Of the various types of liquefaction features, sand blows provide the best opportunity for dating paleoearthquakes. Organic material and cultural artifacts in soil horizons developed in or above sand blows provide minimum age estimates of the features, and thus the event(s). Organic material and cultural artifacts within soil horizons buried by sand blows provide approximate, or at least maximum, age estimates of the event(s). In the case of sand dikes, their maximum ages can be determined by dating the uppermost stratigraphic unit that they crosscut or otherwise deform. Their minimum ages can be determined by dating material that clearly post-dates the liquefaction features, such as intruding roots and cultural pits. Deposits that overlie deformation related to liquefaction or an unconformity that truncates liquefaction features can provide minimum age estimates. Unfortunately, the method of bracketing the age of liquefaction features adds to uncertainties in age estimates. It is not uncommon for age estimates to have uncertainties of a couple of hundred years (using 2-sigma calibrated dates), even in the best of circumstances. Therefore, it is important to examine each site carefully for organic samples that will provide close minimum and maximum dates.

Case studies of many earthquakes around the world have shown that for an event of a given magnitude, liquefaction-related ground failures occur within certain epicentral and fault distances (Ambraseys, 1988). Also, the severity of liquefaction has been found to decrease with distance (Youd and Perkins, 1987). Therefore, the size distribution of liquefaction features can be used to estimate the location and magnitude of earthquakes, so long as variables such as liquefaction susceptibility of sediments, topography, and mechanism of ground failure are taken into account (Tuttle, 2001). Also, local earthquakes that induced liquefaction in the region, such as the **M** 6.2 1925 Charlevoix and **M** 5.9 1988 Saguenay event for Quebec, can serve as calibration events and help to interpret paleoliquefaction features.

Evaluation of scenario earthquakes using liquefaction potential analysis can help to place constraints on locations and magnitudes of paleoearthquakes. This is usually done using either the cyclic-stress method or the energy-stress method. We prefer the cyclic-stress method, also known as the simplified procedure (e.g., Seed and Idriss, 1982; Youd et al., 2001) because it is relatively easy to apply and is suitable for many field and tectonic settings. Using appropriate ground motion relations, peak ground accelerations are estimated for earthquakes of various magnitudes (e.g., **M** 5.5, 6, 6.5, 7.0, and 7.5) at distances of interest from known or suspected sources. Having derived peak ground accelerations, cyclic stress ratios generated by the various scenario earthquakes are calculated. Using empirical relations between cyclic stress ratio and corrected blow counts, it is determined whether or not representative layers at a site would be likely, or not likely, to liquefy. By comparing results of this analysis with field observations, one or more scenario earthquake can be selected that may reflect the locations and magnitudes of paleoearthquakes. During this analysis, minimum values of acceleration (liquefaction threshold) are estimated that may have been experienced during the earthquakes. For distal sites of liquefaction, the values may be close to the actual accelerations. Uncertainties in this method are related to identifying the layer that liquefied and estimating the susceptibility of the sediments at the time of the event.

Given the history of earthquake-induce liquefaction and the presence of sandy Holocene and Late Wisconsin deposits along tributaries to the St. Lawrence (e.g., Bolduc, 2003; Bolduc et al., 2003; and Cloutier et al., 1997), a record of large post-glacial earthquakes is likely to exist in the Charlevoix-Rabaska site region. If so, this paleoseismic investigation has the potential to extend the history of large earthquakes in the region by thousands of years. Possible limitations to this investigation include the distribution of liquefiable sediments, adequate exposure of Holocene and Late Wisconsin deposits, and preservation of features in narrow stream valleys and on floodplains disturbed by farming practices.

5. PROJECT RESULTS

The aim of this reconnaissance-level paleoseismic investigation is to provide information to determine if large Holocene earthquakes have occurred in the Charlevoix and Rabaska site areas in order to reduce uncertainties in seismic source models used in a site-specific earthquake hazard assessment of the proposed Rabaska LNG terminal site near Levis. During this investigation conducted in August and September 2006, we reviewed background information on the surficial geology of the Charlevoix-Rabaska site region, evaluated rivers in the region for presence and exposure of potentially liquefiable Quaternary sediments, searched selected portions of three river for earthquake-induced liquefaction features, documented sites where liquefaction features do and do not occur, dated organic-material from these sites to provide age control of sediments and liquefaction features, and interpreted observations in terms of timing, magnitude, and source areas of paleoearthquakes and their implications for seismic source zones (Figure 4).

5.1 Review of Background Information

Rabaska made available to this project reports by SNC-Lavalin (2006), Technisol (2005a, 2005b), and Terratech (2006) that contain relevant geological and geotechnical information for the Levis area. In addition, we gathered surficial geology maps and scientific articles on Quaternary geology and stratigraphy of the Charlevoix and Rabaska site areas (see References) and downloaded and printed digital topographic maps (1:50,000) produced by Energy, Mines, and Resources Canada. Also, we requested a search of the Quebec Ministry of Transport and Quebec Ministry of Natural Resources databases for previously collected borehole data describing sediment type and standard penetration test blow counts or N, a measure of relative density, for the following rivers: Beaurivage, Du Sud, Etchemin, Gouffre, Jacques-Cartier, Malbaie, Sainte-Anne, and Ouelle (Figure 4). This information would help to characterize the liquefaction susceptibility of sediments along the rivers. In addition, the information could be used to evaluate scenario earthquakes in a later phase of this investigation, if warranted. So far, one geotechnical report has been found for a study of the foundation of retaining walls along Route 138 in Riviere Malbaie (D'Astous, 1996).

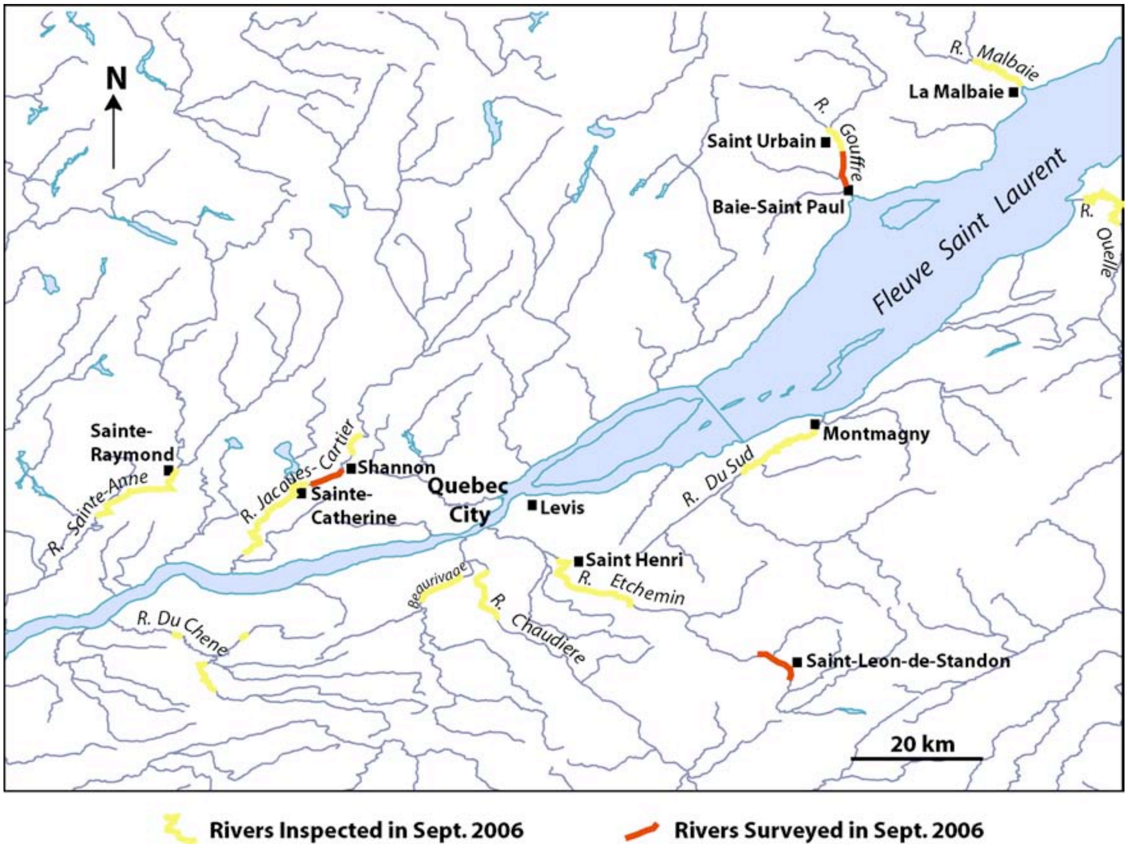


Figure 4. Map of surface hydrology in the Quebec City region (from Quebec Resources Naturelles et Faune; scale1:1,000,000), showing portions of rivers inspected to assess suitability for reconnaissance (yellow) and surveyed to search for earthquake-induced liquefaction features (red).

According to the report by SNC-Lavalin (2006), Quaternary deposits in the Levis area include till, fluvio-glacial sediments, clayey marine sediments, sandy marine sediments, organic sediments, and recent fluvial sediments. Quaternary stratigraphy varies laterally and vertically across the area. The most common stratigraphic sequence is peat over sandy marine sediments over clayey marine sediments. This sequence is observed in the drainage basins of the Etchemin and Chaudiere Rivers. Where bedrock is close to the surface, sandy marine sediments overlie till or rest directly on rock. This is the case in the northern part of the area and along the Chaudiere River. Of the various sediment types, sandy marine sediments and clayey marine sediments cover most of the surface area.

Consistent with the reports mentioned above, a recent surficial geology map of the Levis area shows that Holocene fluvial and Wisconsin marine littoral deposits occur along the Beaurivage and Chaudiere Rivers (Bolduc, 2003; Table 1). Similar deposits and Wisconsin marine deep-water deposits are mapped along the Etchemin River. Surficial geology maps of the Quebec and Saint-Marc-des Carrieres areas depict Holocene fluvial and organic deposits and Wisconsin marine deltaic deposits along the Jacques-Cartier and Sainte Anne Rivers (Bolduc et al., 2003; Cloutier et al., 1997; Table 1). Similar deposits and Wisconsin marine littoral deposits are mapped along the Montmorency River. To the northeast in the Charlevoix area, Quaternary glacial, fluvial, and marine deposits are fairly widespread and have been mapped along both the Gouffre and Malbaie Rivers (Rondot, 1969 and 1972; Table 2). Unfortunately, the different types of Quaternary deposits are not differentiated on these maps of the Charlevoix area. Surficial geology maps have not been identified or reviewed for the southern shore of the Saint Lawrence River southwest or northeast of Levis where the Du Chene, Du Sud and Ouelle Rivers are located (Figure 4). The Quaternary geology is likely to be quite similar to the Levis area where Holocene fluvial and Wisconsin marine deposits are mapped along river courses.

As reported in a geotechnical report for the proposed west option site LNG receiving terminal (Terratech, 2006), granular soils in the upper 16 m are generally compact to dense with standard penetration test blow counts (N) that range from 20 to 50, and with a few values less than 10. N values from 0 to 4 indicate very loose relative density, from 4 to 10 reflect loose density, and from 10 to 30 point to moderate density (Terzaghi and Peck, 1967). Liquefaction susceptibility decreases with increasing relative density or N (Youd and Perkins, 1987). If saturated, sediments with low blow counts (0 to 10) are thought to be susceptible to liquefaction during strong earthquakes. As demonstrated in the liquefaction potential analysis performed by Terratech (2006), the sandy portions of the soil profile with low blow counts at the west option site are susceptible to localized liquefaction at depth, not viewed as a risk for lightly loaded shallow foundations. Geotechnical investigations were conducted also for proposed pipeline crossings of the Beaurivage, Etchemin and Chaudiere Rivers (Technisol, 2005a and 2005b). Near the pipeline crossing on the south side of the Beaurivage River, borehole log F-BE-02 indicates sand with a trace of silt and N values of 3 to 13 from 0 to 2.5 m below the surface, silty sand, sand, and silt, with a trace of clay and N values of 11 to 12 from 2.5 to 5 m depth, similar sediment with an N value of 28 from 5 to 7.25 m, and interbedded silty clay and silty sand with N values of 1 to 5 from 9.1 to 15.5 m depth. A liquefaction potential analysis has not been performed with these data, but low N values for a few of the sandy layers suggest that they are likely to be susceptible to liquefaction. Sediments encountered in boreholes at the pipeline crossings of the Etchemin and Chaudiere Rivers, on the other hand, did not appear to be susceptible to liquefaction. The geotechnical investigations indicate that liquefiable sediments do occur in the Rabaska site area and suggest that a record of large local Holocene earthquakes could exist in the form of liquefaction features, if such events occurred.

According to the geotechnical study for the retaining walls along Route 138 in Riviere Malbaie (D'Astous, 1996), silty sand with traces of gravel and clay and N values ranging from 2 to 9 occurs from 0.3 to 3.8 m below the surface, and silt and sand with traces of clay and N values of 12 to 28 occur from 3.8 to 7.6 m below the surface. Both the upper and lower sedimentary units are within the range of blow counts for which liquefaction is possible if they are saturated and subjected to strong ground shaking. Liquefaction potential analysis would have to be conducted to determine whether or not these sediments are likely to liquefy at various levels of ground shaking.

5.2 Selection of River Sections for Field Inspection

According to compilation of worldwide data on earthquakes that induced liquefaction, saturated Holocene sandy deposits of fluvial, deltaic, and lacustrine origins are highly to moderately susceptible to liquefaction (Youd and Perkins, 1978). Locally, the 1988 Saguenay, Quebec earthquake induced liquefaction in Holocene sandy fluvial and Late Wisconsin sandy glaciofluvial and glaciolacustrine deposits up to 30 km from its epicenter (Tuttle et al., 1990). Therefore, in the study region, Holocene and Late Wisconsin sandy fluvial, deltaic, and lacustrine deposits are likely to be susceptible to liquefaction. In addition, the formation of liquefaction features is likely to be enhanced where the sandy deposits are interbedded with fine-grained deposits, such as silty or clayey fluvial or marine deposits, that would promote the buildup of pore-water pressure during ground shaking.

Meandering rivers often provide exposures of Quaternary deposits. Therefore, we viewed rivers in the region using Google Earth to identify meandering sections. After reviewing maps, papers, and reports mentioned above, we selected meandering river sections for field inspection where Holocene fluvial and Late Wisconsin marine deltaic and littoral deposits had been mapped. These included sections along the following rivers: Beaurivage, Chaudiere, Du Chene, Du Sud, Etchemin, Gouffre, Jacques-Cartier, Malbaie, Montmorency, Ouelle, and Sainte Anne. See Figure 4 and Tables 1, 2 and 3 for a summary of the locations, mapped surficial deposits, and other characteristics of the selected river sections.

5.3 Field Inspection of Rivers

In early September 2006, we inspected selected meandering sections of rivers in the Charlevoix-Rabaska site region where Holocene fluvial and Late Wisconsin marine deltaic and littoral deposits had been mapped (Figures 4 and 5). We inspected the rivers at bridge crossings and along roads that come close to the rivers for cutbank exposure of sediments, canoeing conditions and access points. Unfortunately, cutbank exposure was poor along many of the rivers due to vegetative cover (Figures 6 and 7). Our observations are summarized in Tables 1, 2, and 3.

Figure 5. Photograph of meandering Etchemin River near Saint-Leon-de-Standon.

Figure 6. Photograph of Beaurivage River showing poor cutbank exposure in September 2006. Borehole data indicates that sediments likely to be susceptible to liquefaction occur along river.

Figure 7. Photograph showing cutbank exposure of sandy and silty sediments along Etchemin River in September 2006. Even in meander bends, cutbanks are partially vegetated.

Table 1. Rivers in Rabaska Site Area.

River	Location	Mapped Surficial Deposits	Canoeing Conditions	Exposure
Beaurivage	Near Saint-Etienne-de-Lauzon	Holocene fluvial; Wisconsin marine littoral	Good	Steep banks suggest erosion in bends; vegetated
Chaudiere	Rue des Lilas to Rt. 73 bridge	Mostly Wisconsin marine littoral; some Holocene fluvial	Good to R1 rapids	Low vegetated banks; rock outcrops
Du Chene and tributaries	Near Val-Alain West of Laurier-Station Near Saint Edouard	Unknown; probably similar to Chaudiere and Etchemin	Small creeks; little water Good Rapids	Vegetated banks Grassy, slumpy banks; may have potential Boulders
Etchemin	Near Saint-Jean Chrysostome Saint-Anselme to Saint-Henri Near Saint-Leon-de-Standon	Holocene fluvial Holocene fluvial; Wisconsin marine littoral and deep water Unknown	Easy to R1 rapids Good to R1 rapids Good	Poor exposure; vegetated banks; rocky and boulders Observed in few places; vegetated and rocky banks Vegetated banks; little exposure in bends; sandy
Jacques-Cartier	Upstream from Saint-Gabriel-de-Valcartier Near Shannon and St. Catherine	Modern and Holocene fluvial; Wisconsin marine deltaic Holocene fluvial; Wisconsin marine deltaic	Good to R1 rapids Good to R1 rapids	Vegetated cobbley banks; high banks-sand above cobbles inset in silt Vegetated banks; sandy exposure near Shannon
Sainte-Anne	Near Saint-Raymond Near Sainte-Christine	Holocene fluvial; Wisconsin marine deltaic Holocene fluvial; Wisconsin marine deltaic	Good Good	Vegetated; retaining wall; little exposure; cobbley Some exposure of sandy sediment

Table 2. Rivers in Charlevoix Area.

River	Location	Mapped Surficial Deposits	Canoeing Conditions	Exposure
Gouffre	Bridge crossings St. Urbain, Rt. 138, and Rt. 362 in Baie-Saint Paul	Quaternary glacial, fluvial, or marine	Good to R1 rapids	Few exposures; sand, pebbles, cobbles, and boulders
Malbaie	Clermont bridge to La Malbaie bridge crossing	Quaternary glacial, fluvial, or marine	Good	Low banks - many cobbles; high banks - fine layered sediment
Ouelle	Saint-Pacome to Riviere Ouelle	Unknown; probably Holocene fluvial	Good	Vegetated and rip rap; few sandy exposures downstream from Rt. 20

Table 3. Rivers between Charlevoix and Rabaska Site Areas.

River	Location	Mapped Surficial Deposits	Canoeing Conditions	Exposure
Du Sud	Saint-Francois to Montmagny	Unknown; probably Quaternary glacial, fluvial, or marine	Good	Mostly vegetated; little exposure in bends
Montmorency	Barriere du Seminaire to Ile Enchanteresse	Modern and Holocene fluvial; Wisconsin marine littoral	Rapids; R2 and higher	Rocky

On the basis of mapped surficial deposits and field observations including exposure, we selected the following river segments for an initial phase of reconnaissance: 16 km of the Gouffre River between Rt. 138 and Rt. 362 near Baie-Saint Paul, 12 km of the Etchemin River near Saint-Leon-de-Standon, and 6 km of the Jacques-Cartier River downriver from Shannon.

5.4 River Reconnaissance and Radiocarbon Dating

We conducted reconnaissance of sections of the Gouffre, Etchemin, and Jacques-Cartier Rivers, examining cutbank exposures of Holocene and Wisconsin sediments for earthquake-induced liquefaction features and other co-seismic deformation. For each river, we described exposure and sedimentary conditions for several representative sites. In addition, we described all sites where liquefaction features or soft-sediment deformation structures occur. We measured the size and orientations and described grain-size, degree of weathering or soil development, and stratigraphic context of liquefaction features. We photographed significant features and collected wood and other organic materials for radiocarbon dating. The location of each site was marked on a 1:50,000 scale topographic map and a hand-help global positioning system device was used to measure its position. The information was recorded on site description forms and is summarized in Tables 4, 6, and 7. Beta Analytic, Inc., conducted radiocarbon dating of organic samples for this study. They used the accelerator mass spectrometry (AMS) technique recommended when high precision is desired. The results of the radiocarbon dating are summarized in Tables 5 and 8 and discussed below.

5.4.1 Observations for the Gouffre Rivers

We conducted river reconnaissance along 16 km of the Gouffre River between Rt. 138 and Rt. 362 near Baie-Saint Paul (Figure 8). Exposure along the Gouffre River is good to excellent in river bends, most cutbanks are 6 to 13 m high, and slumping is common. In most exposures, a coarse-grained deposit of sand, pebbles, and cobbles is underlain by a fine-grained deposit of clay, silty clay, or silt with interbeds of sand (Table 4). In several locations, the fine-grained silty clay deposit is underlain by loose fine to medium sand exposed at the base of the cutbank or encountered below the river level with a soil probe. The sedimentary conditions appear to be good for the formation of earthquake-induced liquefaction features. Radiocarbon dating of organic material collected near the base of the cutbank at site GR6 from the silty clay deposit indicates that it was deposited 9410-9540 years B.P. (Table 5). Therefore, sediments exposed along the Gouffre River are old enough to record Holocene earthquakes.

At sites GR1 and GR2, the coarse-grained deposit is overlain by a clayey silt deposit with tilted bedding, interpreted as a landslide deposit. A buried soil/organic-rich layer occurs at the upper contact of the coarse-grained deposit and represents the land surface at the time of mass movement. Radiocarbon dating of samples (GR1-W1, GR2-C1) collected from the buried soil/organic-rich layer at the two sites indicates that the landslide occurred after A.D. 1440 and possible after A.D. 1640 (Table 5). Like others in the Charlevoix region, the landslide could have been triggered by the 1663 Charlevoix earthquake.

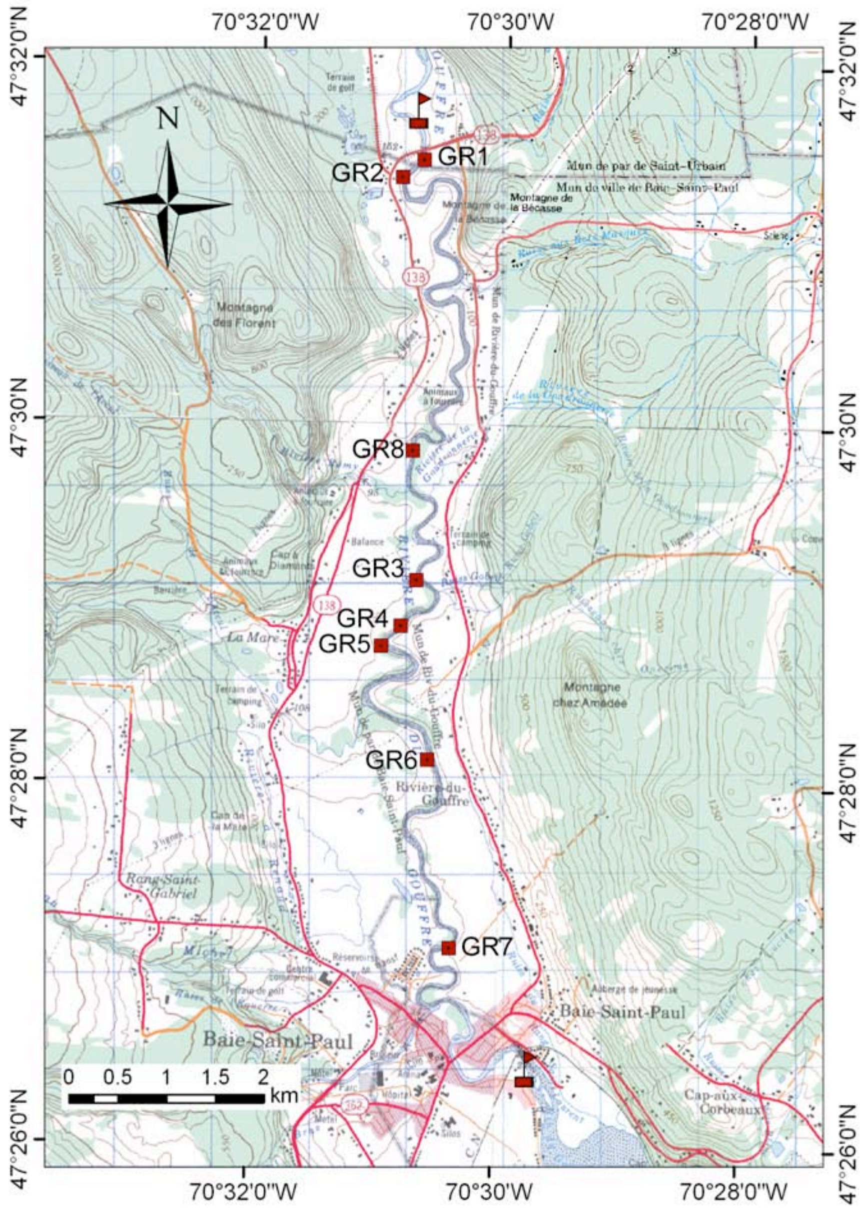


Figure 8. Topographic map of Gouffre River near Baie-Saint Paul showing locations of study sites described in Table 4. Red flags mark beginning and end points of river reconnaissance.

We found eight sand dikes interpreted as earthquake-induced liquefaction features at sites GR3 and GR4, another possible sand dike at site GR8, and a loose sand layer that may be an intruded sill at GR2 (Tables 4 and 6). The three sand dikes at GR3 range from 9 to 15 cm in width. The two larger dikes extend upward through 3 m of a silty clay deposit and terminate in the base of an overlying sandy deposit (Figure 9). The dikes fine upward from coarse sand to silty, very fine sand and contain clasts of silty clay. Small subhorizontal dikes extend from and between the two large dikes about 15 cm below the contact between the silty clay and overlying sand deposit. There is little to no weathering in all three dikes. Many small pieces of organic material occur in root casts or animal burrows in the tips of the two larger dikes. The biological casts post-date the formation of the sand dikes. Radiocarbon dating of a sample of the organic material from one of the casts (GR3-C1) yielded a calibrated date of A.D. 650-780 and provides a minimum age estimate for the sand dikes (Table 5). The date indicates that the sand dikes are prehistoric in age and formed before A.D. 780. A very small sample of organic sediment (GR3-C2) was collected between the two larger sand dikes from the silty clay deposit 17 cm below the contact with the overlying sand deposit. Radiocarbon dating of the sample gave a result of 16,130-15,400 B.C. This date is much older than those from other samples collected along the Gouffre River. The sample was small, mineral in nature, and could have been reworked. The date of 7460-7590 B.C. (or 9410-9540 years B.P.) from GR6-W1, a sample of leaves and other plant material collected only 80 cm above the water level, may more closely reflect the age of the clayey silt deposit. If so, the sand dikes formed between A.D. 780 and 7590 B.C. (or 1170 and 9540 years B.P.). Even so, the age of the sand dikes is not well-constrained.

The five sand dikes at GR4 range from 1 to 3 cm in width (Tables 4 and 6). All of the sand dikes terminate within a silty clay deposit exposed along the base of the cutbank. The small dikes extend no more than 1.1 m above the river level at the time of reconnaissance. All the dikes are composed of very fine sand and exhibit little to no weathering. No organic material for dating was found at this site. Given that they terminate within the silty clay deposit, the sand dikes likely formed since 7590 B.C.

At site GR7, we found only one possible sand dike (Tables 4 and 6). It is 1 cm wide and only 1.25 m in length, pinching out in both directions within a silt deposit at the base of the cutbank. The dike is composed of very fine sand and could be a sand lense within a tilted slump block.

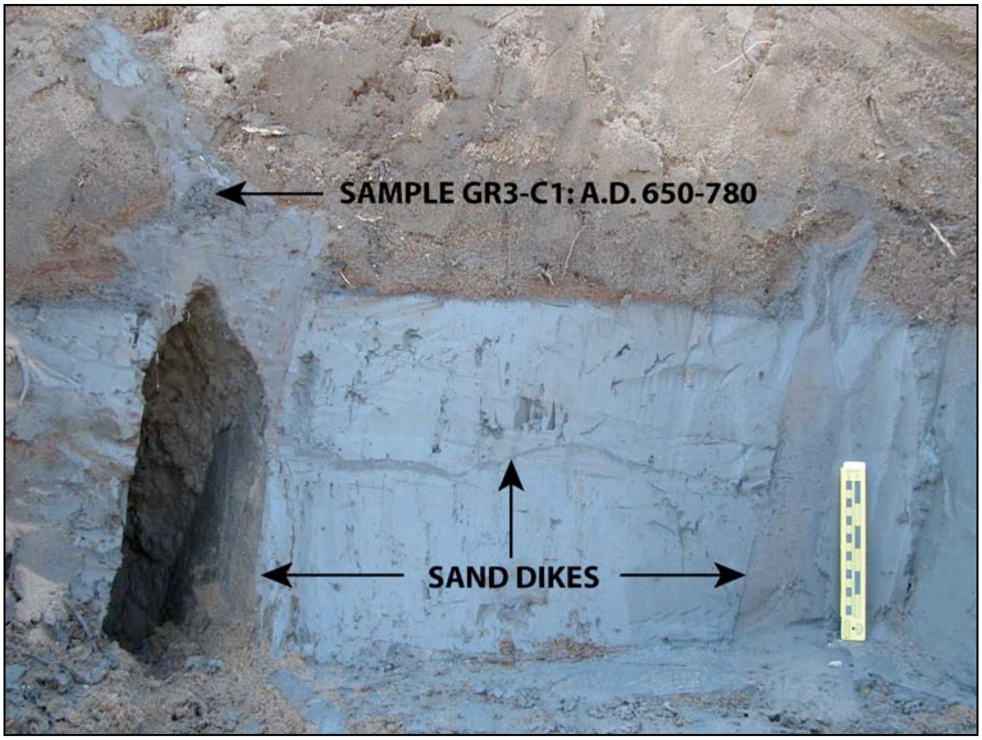


Figure 9. Photograph of sand dikes (left one partially washed out) at site GR3 intruding silty clay deposit and terminating in base of overlying sand deposit. Note small subhorizontal dike extending between larger subvertical dikes. Plant material from root cast or animal burrow in top of sand dike provides minimum age constraint of A.D. 650-780 and indicates that it is prehistoric.

A possible sill at GR2 is 5 cm thick, composed of loose, fine to medium sand, and occurs within a sand layer that fines upward from fine sand to silt (Tables 4 and 6). There is no feeder dike associated with the sill, but there are discontinuous domains of similar sand at one end of the sill. In addition, there is a possible source layer of very loose medium to fine sand at the base of the cutbank below an intervening clay deposit. The possible sill occurs near the top of a coarse-grained deposit and below a landslide deposit described above. A radiocarbon date from charcoal (sample GR2-C1) in the buried soil does not help to estimate the age of the possible sill.

Table 4. Description of Study Sites along the Gouffre River.

Site Number	Latitude (Decimal Degrees)	Longitude (Decimal Degrees)	Liquefaction Features Observed	Exposure	Sediments Observed in Cutbank
GR1	47.5245	70.5112	None	Excellent in scarp of slump failure	Cross-bedded sand ovelies tilted massive to layered clayey silt; underlain by interbedded sand, pebbles, and cobbles with organic layer at upper contact
GR2	47.5228	70.5142	Possible sill	Fair; bank mostly vegetated	Brownish clayey silt overlies generally fining upward sequence of cobbles, pebbles, and sand with organic layer at upper contact; underlain by blue-gray clay and very loose, medium to fine sand
GR3	47.4856	70.5114	Sand dikes	Excellent	Interbedded cross-bedded sand and cobble layers overlies silty clay with interbeds of silty sand and silt
GR4	47.4814	70.5134	Sand dikes	Lower 1.2 m of bank very good; otherwise vegetated	Gray, silty clay
GR5	47.4795	70.5159	None	Very good but slumpy	Slump blocks of sandy, cobbley, and silty deposits; upside down tree in sandy deposit
GR6	47.4691	70.5094	None	Good; bank partly vegetated	Interbedded coarse to silty very fine sand overlies clayey silt with interbeds of sand; organic material in silt deposit low in cutbank
GR7	47.4517	70.5060	None	Excellent	Interbedded silt and sand overlies silt with interbeds of sand becomes massive below
GR8	46.4976	70.5122	Possible sand dike	Excellent	Cross-bedded sand overlies layered silt; some portions appear tilted

Table 5. Radiocarbon Dating of Samples from Study Sites along the Gouffre River.

Sample Number Lab Number	$^{13}C/^{12}C$ Ratio	Radiocarbon Age Yr B.P.[1]	Calibrated Radiocarbon Age Yr B.P.[2]	Calibrated Calendar Date A.D./B.C.[2]	Sample Description
GR1-W1 Beta-221043	-26.4	370 ± 40	310-510	A.D. 1440-1640	Plant material collected 4.5 m below surface from organic layer below tilted silt deposit and at top of interbedded sand, pebbles, and cobbles
GR2-C1 Beta-221504	-26.4	460 ± 40	470-540	A.D. 1410-1480	Charcoal collected 2 m below surface from buried soil below silty clay and above fining upward unit of cobbles and sand
GR3-C1 Beta-221044	-26.0	1320 ± 40	1170-1300	A.D. 650-780	Plant material collected 10 m below surface from top of sand dike
GR3-C2 Beta-221505	-26.6	14790 ± 50	17340-18080	16130-15400 B.C.	Organic sediment collected 10 m below surface or 2.9 m above water level from silty clay; 17 cm below contact with overlying sandy deposit
GR6-W1 Beta-221045	-24.4	8460 ± 60	9410-9540	7460-7590 B.C.	Plant material collected 7.2 m below surface or 80 cm above water level from organic layer within interbedded clayey silt and sand

[1] Conventional radiocarbon ages in years B.P. or before present (1950) determined by Beta Analytic, Inc. Errors represent 1 standard deviation statistics or 68% probability.

[2] Calibrated age ranges as determined by Beta Analytic, Inc., using the Pretoria procedure (Talma and Vogel, 1993; Vogel et al., 1993). Ranges represent 2 standard deviation statistics or 95% probability.

Table 6. Earthquake-Induced Liquefaction Features along Riviere Gouffre.

Site number	Sand Dike or Sill Width (cm)	Strike and Dip of Sand Dikes	Weathering Characteristics	C14 Age Constraint	Age Estimate (A.D.) of Features
GR2	5	Sill; not measured	Some iron-staining but loose	After 7590 B.C.	Holocene
GR3	15 12 9	N24°E, 73°SE N59°E, 78°SE N19°E, 86°NW	Little to none	Before A.D. 780 After 7590 B.C.	Holocene
GR4	3 2 1	N84°W, 88°NE N34°W, 86°NE N33°W, 85°NE	Little to none	After 7590 B.C.	Holocene
GR8	1	Not measured	Little to none	After 7590 B.C.	Holocene

The sand dikes found on the Gouffre River are classic earthquake-induced liquefaction features. The source beds that liquefied must occur below the silty clay deposits intruded by the sand dikes and at least 9 m below the floodplain. None of the observed sand dikes reached the surface. This is not surprising given the thick section of coarse-grained deposit they would have to cross. This leads one to wonder about the liquefaction-related ground failure described during the 1870 and 1925 earthquakes. Why did we not find historic liquefaction features? Do they occur in areas that are currently poorly exposure or upstream from the section searched? Could the paleoearthquake have been stronger than the 1925 earthquake and produced more obvious liquefaction features? Additional study is needed along the Gouffre to answer these questions that may have implications regarding the use of the 1925 earthquake as a calibration event.

5.4.2 Observations for the Etchemin River

We conducted reconnaissance along 12 km of the Etchemin River near Saint-Leon-de-Standon (Figure 10). Exposure along this section of the river is fair in river bends with 50 percent or more of the 2 to 3 m high cutbanks covered by vegetation (Table 7; Figure 7). In most exposures, an interbedded silt and very fine sandy silt deposit containing buried soils overlies a medium to coarse sand or pebbly sand deposit. Radiocarbon dating of plant material from organic layers near the base of the cutbank at three sites (EC2, EC3, and EC6) provides excellent age control for sediments along the river and indicate they were deposited since 7310 B.C. (or 9260 years B.P) (Table 8). Wood collected 1.6 m below the surface from a soil developed in the top of a silt deposit yielded a calibrated date of 2920-3120 B.C. and 3220-3320 B.C. (or 4860-5070 and 5170-5270 years B.P.). The sandy sediments above the soil were deposited since 3320 B.C. (or 5270 years B.P.). The 3-m section of sediments exposed along the Etchemin River in September 2006 are old enough to record earthquakes during the past 9200 years.

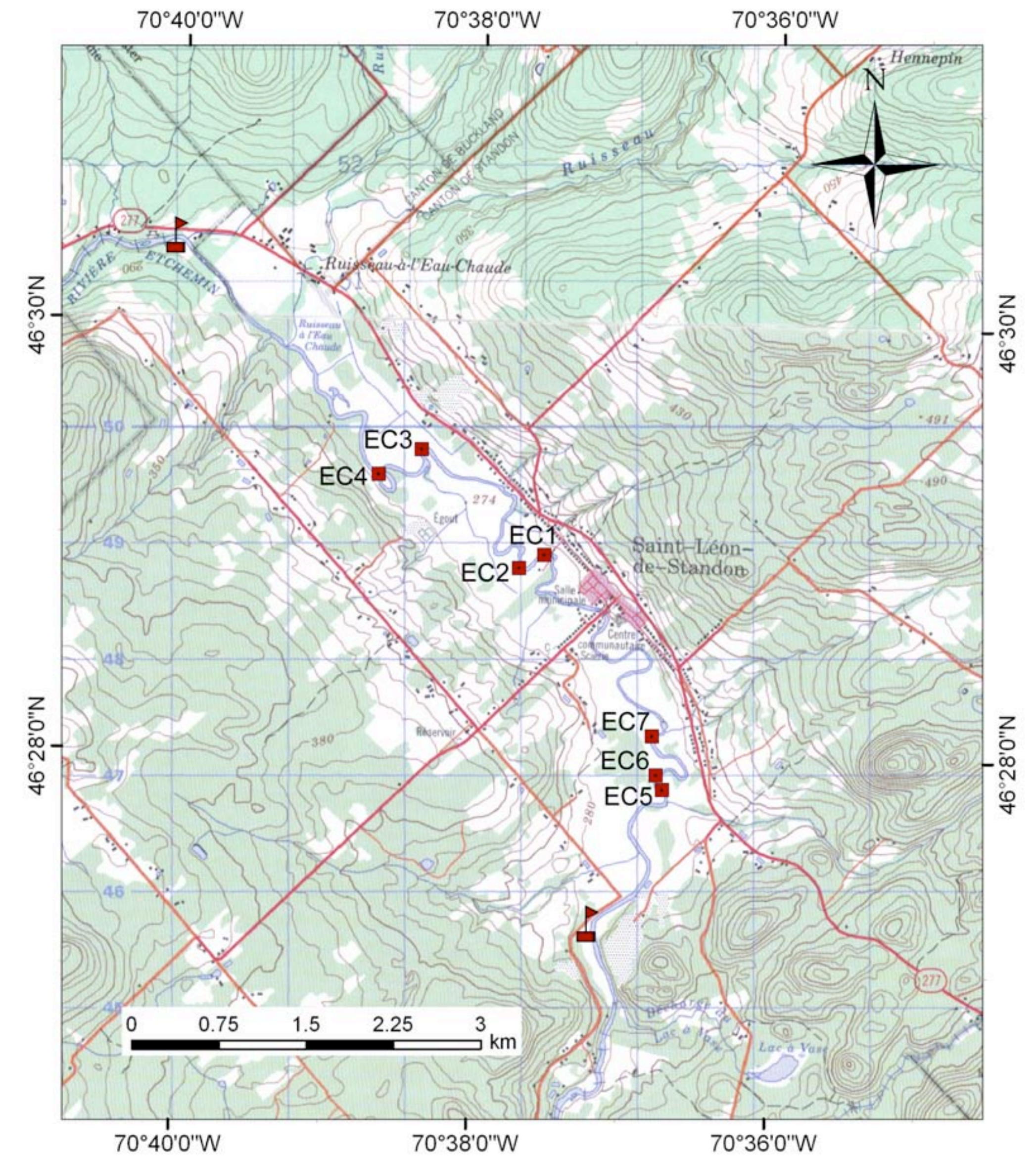


Figure 10. Topographic map of Etchemin River near Saint-Leon-de-Standon showing locations of study sites described in Table 7. Red flags mark beginning and end points of river reconnaissance.

Sedimentary conditions appear to be suitable for the formation and preservation of liquefaction features; yet, we found no sand dike or other unequivocal earthquake-induced liquefaction feature (Figure 11). If exposure had been better, we could be more confident that liquefaction features do not occur in the Holocene sediments near Saint-Leon-de-Standon. We did observed soft-sediment deformation structures at sites EC2 and EC7 (Table 7; Figure 12). Given the age of the sediments along the river, these deformation structures must have formed in the past 9200 years. Certain types of deformation structures such as load casts and heave structures are sometimes attributed to ground shaking (Sims, 1973). Features such as these could help to constrain ground-shaking but would require additional study.

Figure 11. Photograph of weathered silty very fine sand over cross-bedded medium to coarse sand at site EC4. Sedimentary conditions appear suitable for formation of earthquake-induced liquefaction features. So far, no uneqivocal liquefaction feature has been found along Etchemin River.

Table 7. Description of Study Sites along the Etchemin River.

Site Number	Latitude (Decimal Degrees)	Longitude (Decimal Degrees)	Liquefaction Features Observed	Exposure	Sediments Observed in Cutbank
EC1	46.4822	70.6258	None	Fair; banks slumpy and partly vegetated	Interbedded pebbley sand and very fine sandy silt with buried soil horizon overlies pebbley sand
EC2	46.4812	70.6286	Soft-sediment deformation	Fair	Interbedded very fine sandy silt and silt; buried soil
EC3	46.4902	70.6398	None	Good	Interbedded very fine sandy silt and silt with organic and pebbley sand layers
EC4	46.4882	70.6445	None	Excellent	Interbedded very fine sand and silty very fine sand with manganese nodules overlies crossbedded medium to coarse sand
EC5	46.4642	70.6121	None	Fair; banks slumpy and 65% vegetated	Silty very fine sand overlies pebbley sand
EC6	46.4653	70.6128	None	Fair; banks 50% vegetated	Very fine sandy silt overlies silt with buried soils; sand 1 m below water level
EC7	46.4683	70.6134	Soft-sediment deformation	Good	Interbedded very fine sandy silt and silty very fine sand

5.4.3 Observations for the Jacques-Cartier River

We conducted reconnaissance along 6 km of the Jacques-Cartier River downriver from Shannon. (Figure 13; Table 9). Except for the upstream 2 km, the exposure along this section of the river is poor. The banks are heavily vegetated and in places protected from erosion with large boulders or riprap. At sites JC1a and JC1b, cutbank exposures reveal interbedded silty fine sand, sand, and silt. At site JC2, medium sand overlies rhymites of clayey silt and very fine sand. We found organic material for radiocarbon dating only at site JC1b. The sample came from an organic layer within a deposit of interbedded silt and silty very fine sand that appears to be a channel fill of a nearby creek. The sample yielded a calibrated date of A.D. 650-770 (or 1180-1300 B.P.) (Table 8) which represents the age of the channel fill and post-dates the sandy deposit in which it is incised. Further reconnaissance of the river was abandoned due to poor exposure at the time.

Figure 12. Photograph of soft-sediment deformation structure at site EC7 on Etchemin River. Sand diapir suggests upward remobilization of sand and deformation of overlying silt lamination. Alternatively, this may be a flame structure related to depositional processes.

Table 8. Radiocarbon Dates of Samples from Study Sites along the Etchemin and Jacques-Cartier Rivers.

Sample Number Lab Number	**$^{13}C/^{12}C$ Ratio**	**Radiocarbon Age Yr B.P.[1]**	**Calibrated Radiocarbon Age Yr B.P.[2]**	**Calibrated Calendar Date A.D./B.C.[2]**	**Sample Description**
EC2-W1 Beta-221041	-26.8	8130 ± 60	8990-9260	7040-7310 B.C.	Plant material collected 2.5 m below surface from organic layer within interbedded silt and very fine sandy silt
EC3-W1 Beta-221503	-28.1	7550 ± 40	8330-8400	6380-6450 B.C.	Leaf fragments collected 2.4 m below surface from organic layer within interbedded silt and very fine sandy silt
EC6-W1 Beta-221042	-24.0	6860 ± 50	7600-7780	5660-5830 B.C.	Wood collected 2.8 m below surface from outer 1 cm of tree trunk bedded in silt
EC6-W2 Beta-221502	-28.0	4420 ± 40	4860-5070 5170-5270	2920-3120 B.C. 3220-3320 B.C.	Wood collected 1.6 m below surface from paleosol developed in silt
JC1b-W1 Beta-221506	-27.10	1330 ± 40	1180-1300	A.D. 650-770	Plant material including leaf and needles collected 2.8 m below surface from organic layer within interbedded silt and silty very fine sand

[1] Conventional radiocarbon ages in years B.P. or before present (1950) determined by Beta Analytic, Inc. Errors represent 1 standard deviation statistics or 68% probability.

[2] Calibrated age ranges as determined by Beta Analytic, Inc., using the Pretoria procedure (Talma and Vogel, 1993; Vogel *et al*., 1993). Ranges represent 2 standard deviation statistics or 95% probability.

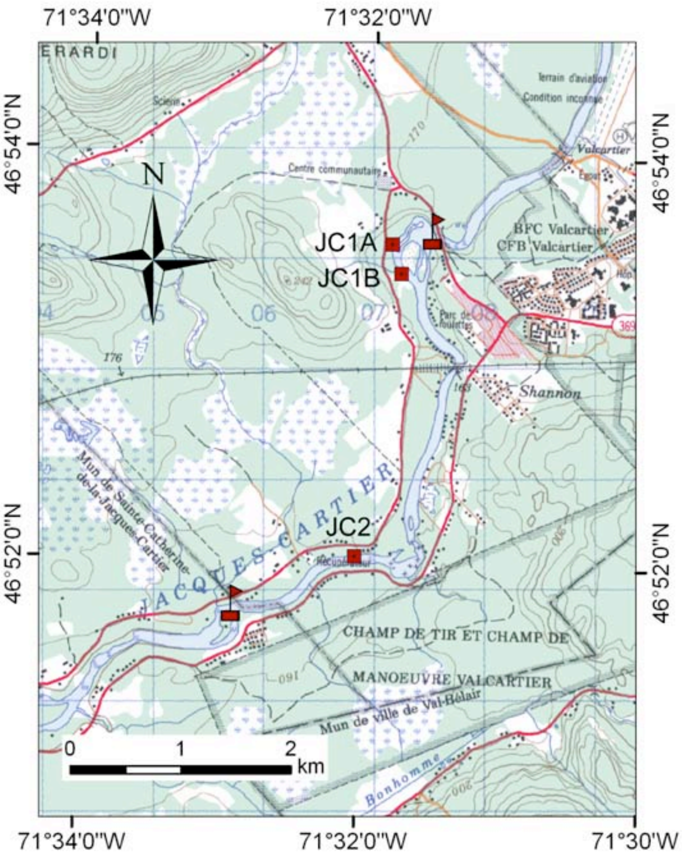


Figure 13. Topographic map of Jacques-Cartier River near Shannon showing locations of study sites described in Table 9. Red flags mark beginning and end points of river reconnaissance.

Table 9. Description of Study Sites along the Jacques-Cartier River.

Site Number	Latitude (Decimal Degrees)	Longitude (Decimal Degrees)	Liquefaction Features Observed	Exposure	Sediments Observed in Cutbank
JC1a	46.8927	71.5308	None	Excellent	Interbedded silty fine sand and medium to fine sand
JC1b	46.8904	71.5296	None	Excellent	Interbedded silty fine sand and medium to fine sand overlies interbedded silt and silty very fine sand; organic layer at base
JC2	46.8673	71.5342	None	Very good; but only 5 m in length	Medium sand overlies rhymites of clayey silt and very fine sand

5.5 Interpretation of Results

Interpretations of field observations and radiocarbon dating are given below. This investigation has been limited by the small number and poor exposure of river sections searched for liquefaction features in the Charlevoix-Rabaska site region.

5.5.1 Charlevoix Area

In the Charlevoix area, sand dikes resulting from earthquake-induced liquefaction occur along the Gouffre River. There is no evidence to suggest that they formed during more than one event, although this possibility cannot be ruled out with the current data. The sand dikes are prehistoric in age and formed between A.D. 780 and 7590 B.C. or 1170 and 9540 years B.P. Additional dating of sediments cut by the prehistoric sand dikes along the Gouffre River could help to further constrain the age of the paleoearthquake.

The geographical distribution of paleoliquefaction features in the Charlevoix area is not known because no other rivers in the area have been searched so far. Therefore, it is not yet possible to determine the location of the paleoearthquake responsible for the liquefaction features, although it was probably centered somewhere in the Charlevoix area. Inspections of rivers suggested that cutbank exposures of Holocene and Late Wisconsin deposits are available along sections of the Malbaie and Ouelle Rivers. Reconnaissance along these rivers may or may not lead to the discovery of additional liquefaction features, but could help to define the distribution of paleoliquefaction features and thus the location of the paleoearthquake.

The Gouffre River sand dikes range up to 15 cm in width. The sand dikes that formed during the 1988 **M** 5.9 Saguenay earthquake are only 1 to 5 cm in width. Assuming liquefaction susceptibility of sediments is similar in the two areas, the larger size of prehistoric sand dikes along the Gouffre River suggests that the responsible earthquake may have been located less than 30 km away or been greater than **M** 5.9. Geotechnical data for the Gouffre River sediments would help to assess their liquefaction susceptibility and to evaluate scenario earthquakes that could explain the occurrence of liquefaction features.

5.5.2 Rabaska Site Area

Exposure of Holocene and Late Wisconsin deposits is poor along the Chaudiere, Etchemin, and Jacques-Cartier Rivers in the Rabaska site area because river banks are heavily vegetated and places protected in places from erosion by boulders. Dams have been built along the rivers to control flood water and they have also reduced the amount of cutbank erosion. Smaller rivers in more rural settings and with fewer flood control measures, such as the Beaurivage, Du Chene, and Du Sud Rivers, might provide better exposures, especially after spring floods and before vegetation has grown on cutbanks.

The meandering section of the Etchemin River near Saint-Leon-de-Standon provided fair exposure of sediments in river bends. Radiocarbon dating indicates that there is a 9200-year sedimentary record along the river. In addition, conditions seem to be favorable for the formation and preservation of liquefaction features. Yet, we found no liquefaction features along the river, suggesting that this area has not been subjected to strong ground shaking for 9000 years. We did find a few soft-sediment deformation structures that might indicate low levels of ground shaking during this time period.

If we could be more confident that earthquake-induced liquefaction features do not occur in the Holocene age sediments near Saint-Leon-de-Standon and if we had geotechnical data confirming that the Etchemin sediments are highly susceptible to liquefaction, it might be possible to discount large paleoearthquakes in the Rabaska site area during the Holocene. If so, this would suggest that the Charlevoix seismic zone does not extend into the site area. For example, the Etchemin River near Saint-Leon-de-Standon is about 50-60 km from the Rabaska site area and 100-140 km from the Charlevoix area. According to Ambraseys' (1988) relation between earthquake magnitude and distance to liquefaction developed from a worldwide database of shallow earthquakes, **M** 6.7 earthquakes can induce liquefaction up to 70 km from their epicenters. Therefore, an earthquake of this magnitude centered in Charlevoix would not be expected to induce liquefaction in highly susceptible sediments near Saint-Leon-de-Standon; whereas, such an event centered near the Rabaska site would. It should be taken into consideration, however, that the 1988 **M** 5.9 Saguenay earthquake induced liquefaction up to 30 km from its epicenter, about 1.5 times farther than would be expected based on Ambraseys' relation. This may be due to higher than average frequency content or stress drop for the Saguenay earthquake compared to the earthquakes used to develop the magnitude-distance relation. If the Saguenay earthquake is typical of Quebec earthquakes, **M** 6.7 earthquakes might induce liquefaction up to 105 km from their epicenters in this region. If this were the case, a Charlevoix earthquake of this magnitude might have a minimal effect on the sediments (perhaps soft-sediment deformation structures) near Saint-Leon-de-Standon; whereas such an event centered near the Rabaska site would have an even greater impact than expected using the standard Ambraseys relation. If geotechnical data were available for the sediments along the Etchemin River near Saint-Leon-de-Standon, liquefaction potential analysis also could be used to evaluate various scenario earthquakes and to place limits on the magnitude of Holocene paleoearthquakes that might have been centered in the Rabaska site area.

5.5.3 Implications for Seismic Source Models

Findings of this study suggest that there was at least one large Holocene paleoearthquake in the Charlevoix area. Unfortunately, poor exposure along the Etchemin, Jacques-Cartier, and other rivers limited our ability to observe the Holocene record in the Rabaska site area. The Etchemin River near Saint-Leon-de-Standon, about 55 km southeast of the Rabaska site, provided a somewhat better glimpse at the Holocene record, where we found no unequivocal liquefaction features. If we could be more confident that earthquake-induced liquefaction features do not occur in the Holocene age sediments near Saint-Leon-de-Standon and along other rivers in the region, and if geotechnical data were available that indicated that the sediments are highly susceptible to liquefaction, we could better ascertain whether or not large Holocene paleoearthquakes have occurred in the Rabaska site area.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Review of surficial geology maps of the region, one geotechnical report for a retaining wall along the Malbaie River, and geological and geotechnical investigations conducted for the environmental impact assessment of the proposed Rabaska terminal and pipeline found that

Holocene and Late Wisconsin deposits that are likely to be susceptible to earthquake-induced liquefaction occur along rivers in both the Charlevoix and Rabaska site areas. We inspected meandering sections of rivers in the region, including the Beaurivage, Chaudiere, Du Chene, Du Sud, Etchemin, Gouffre, Jacques-Cartier, Malbaie, Ouelle, and Saint-Anne Rivers, and selected sections of one river, the Gouffre, in the Chalevoix area and of two rivers, the Etchemin and Jacques-Cartier, in the region of the Rabaska site for an initial phase of reconnaissance.

During survey of the Gouffre River, we found sand dikes resulting from earthquake-induced liquefaction. Dating of organic samples associated with sediments and liquefaction features indicates that they formed during a paleoearthquake between A.D. 780 and 7590 B.C. or from 1170 to 9540 years B.P. The event was likely centered in the Charlevoix area and at least as large as the **M** 5.9 1988 Saguenay earthquake and possibly larger than the 1925 **M** 6.2 Charlevoix earthquake. These findings suggest that at least one large Holocene paleoearthquake occurred in the Charlevoix area.

We found no similar earthquake-induced liquefaction features during surveys of the Etchemin and Jacques-Cartier Rivers in the Rabaska site region; but cutbank exposure was not very good along the Etchemin and was poor along the Jacques Cartier. Additional reconnaissance of well-exposed river cutbanks, either along the same river sections at a time when they are better exposed or along different river sections, is needed to determine whether or not large Holocene paleoearthquakes, similar to the one that affected the Gouffre River, have occurred in the Rabaska site area.

This reconnaissance-level investigation did not uncover enough new information about paleoearthquakes in the Charlevoix-Rabaska region to more clearly define the southwestern limit of the Charlevoix seismic zone. However, the investigation did make some progress towards this goal and the overall approach seems promising. Further progress is contingent on better exposure of Holocene deposits preferably in, but not limited to, the proposed Rabaska site area. Recommendations for additional study follows:

- Repeat surveys of the Etchemin and Gouffre Rivers at a time when cutbank exposure is better than it was in September 2006 to improve confidence in assessment of presence and absence of liquefaction features. Any additional liquefaction features found would be documented and samples collected for radiocarbon dating. Improved dating of paleoearthquakes in the Charlevoix area would help to estimate the rate of recurrence of large earthquakes. It probably would be best to resurvey rivers in early summer of 2007 after spring floods have cleaned banks, river levels have fallen, and before vegetation grows on the cutbanks. Depending on river and weather condition, an attempt could be made in late autumn of 2006.

- Survey portions of the Malbaie and Ouelle Rivers in Charlevoix area and the Beaurivage, Du Chene, and Du Sud Rivers in the Rabaska site area that appear to have reasonably good exposure of suitable Holocene deposits. As above, it probably would be best to wait until summer of 2007 to perform this task, but an attempt could be made in late autumn 2006.
- Extend search for liquefaction features to other rivers with better exposure farther afield but along the St. Lawrence in order to test the hypothesis that the Charlevoix seismic zone has a higher rate of seismicity than the regional Iapetan faults in the Rabaska site area. The search for paleoliquefaction features to the southwest of the Charlevoix seismic zone does not have to be limited to the site area. It is important to find good exposure of Holocene deposits that are themselves, or are underlain by, sediments that are susceptible to liquefaction. The Trois Riviere area, where Holocene fluvial and lacustrine and Wisconsin marine littoral deposits are widespread (Bolduc, 1999), may be an alternative location to search for liquefaction features resulting from large Holocene earthquakes centered near the Rabaska site area.
- Compile and tabulate borehole data of bridge crossings and from other geotechnical studies, including sediment description, depth ranges, and blow counts (N), and water-table depth, along surveyed rivers where earthquake-induced liquefaction features have and have not been found. If geotechnical data cannot be found for areas of interest, such as the Etchemin River near Saint-Leon-de-Standon, consider conducting in situ investigations (standard or cone penetration tests).
- Evaluate scenario earthquakes (~**M** 5, 5.5, 6, 6.5, and 7) in Charlevoix-Rabaska site region using liquefaction potential analysis and comparing results with field observations. This will make it possible to estimate with more confidence the location and magnitude of

paleoearthquakes that may have affected the two areas.

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