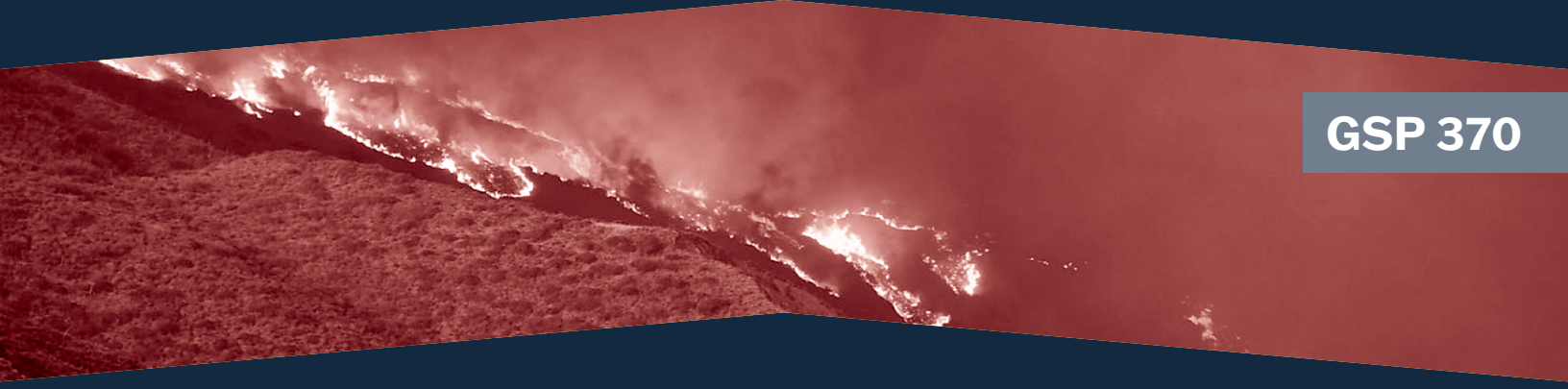


GSP 370

Geo-Extreme 2025



Geotechnical Earthquake Engineering



Selected Papers from Geo-Extreme 2025
Long Beach, California • November 2–5, 2025

EDITED BY

Christopher L. Meehan, Ph.D., P.E., BC.GE;
James A. McKelvey III, Ph.D., P.E., BC.GE;
Adda Athanasopoulos-Zekkos, Ph.D.;
Miguel A. Pando, Ph.D., P.Eng.; Nina Stark, Ph.D.;
and Ming Xiao, Ph.D., P.E., BC.GE

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GEOTECHNICAL EARTHQUAKE ENGINEERING

SELECTED PAPERS FROM GEO-EXTREME 2025

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Preface

The Embankments, Dams, and Slopes (EDS) Technical Committee of the ASCE Geo-Institute organized a timely and interesting specialty conference in 2021 in Savannah, Georgia, entitled *Geo-Extreme 2021: Geotechnical Engineering for Extreme Events*. This 2021 conference was intended as an inaugural specialty conference, and the beginning of a series of planned conferences organized by the EDS committee seeking to address the many societal challenges that are posed by extreme events such as: earthquakes, hurricanes, landslides, debris flows, wildfires, tornadoes, extreme rainfall events, blizzards, and other extreme loading events caused by changing weather patterns. This 2021 conference built on three previous slope stability related conferences that were organized by the EDS committee, two in 1966 and 1992 at the University of California at Berkeley and one in 2013 in San Diego. Strong impetus for that meeting was provided by a number of very significant extreme events in the United States, starting with Hurricanes Katrina in 2005 and Sandy in 2012 followed by Hurricanes Harvey, Irma, and Maria in 2017, and then Hurricanes Laura, Delta, and Zeta in 2020, which all caused widespread damage to communities at large, and particular damage to natural slopes and engineered structures that are the purview of the EDS Committee.

Since the 2021 conference, the onslaught of extreme loading events in the United States has continued unabated, with significant loss of life and severe financial consequences to communities that are unprecedented. The impact of major hurricanes including Ian, Ida, Idalia, Helene, and Milton has been strongly felt in coastal communities but also ranging far inland. Extreme rainfall and flood events have caused damages averaging in excess of a billion dollars annually, particularly in California. Severe storms, winds, and flooding also battered Kentucky heavily in 2025. Wildfires brought significant destruction and caused numerous deaths in Hawaii's Lahaina fire of 2023, the Smokehouse Creek fire in Texas and Oklahoma in 2024, and in California with the Park Fire in 2024 and wildfires in Southern California in 2025. The 2023 fire season was also particularly significant in Canada, which saw 45,700,000 acres of land burn across ten provinces and three territories.

Following these extreme events, the EDS Committee has proposed and organized the current specialty conference, *Geo-Extreme 2025: Geotechnical Engineering for Extreme Events*. The goal of this conference is to continue to draw attention to the pressing problems posed by extreme events, and to identify the critical role that geotechnical engineers and the civil engineering profession as a whole can play in mitigating the effects of these events, both before they occur, as well as after – that is, "building back safer" in a way that benefits both the built and natural environments.

Historical records show that the number of billion-dollar extreme events are on the rise. Mitigating the consequences of future extreme events requires geo-professionals taking timely actions, including evaluation of existing earthen structures, design and construction of protective structures in areas where an increased probability of damage is expected, refinement of existing geotechnical designs, and development of new adaptive measures and risk assessment tools. To achieve these objectives, the Geo-Extreme 2025 conference will comprise a multi-disciplinary forum between geotechnical engineers and other professionals from related disciplines, such as scientists, various

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types of engineers, emergency managers, resilience and sustainability investigators, insurance experts, and policy makers, to provide solutions that properly cope with and address the wide variety of extreme events that can occur.

There are five volumes containing the full collection of papers presented at Geo-Extreme 2025: Geotechnical Engineering for Extreme Events, which was held on November 2-5, 2025, in Long Beach, California. These five volumes are subtitled: (1) Remote Sensing, Instrumentation, Big Data, and Decision Making, (2) Geotechnical Earthquake Engineering, (3) Civil Infrastructure, Geomaterials, and Case Histories, (4) Different Climates and the Coastal Zone, and (5) Permafrost, Wildfires, and Other Emerging Challenges in Geotechnical Engineering, respectively. These five volumes include over 130 contributions that span 25 subtopic areas of the conference, as noted in the subsequent tables of contents for each volume. Each of the five volumes are Geotechnical Special Publications (GSPs), containing papers that were reviewed in accordance with ASCE GSP standards. Each paper was subjected to technical review by two or more independent peer reviewers, and publication required concurrence by at least two peer reviewers.

The Editors gratefully acknowledge the immeasurable contributions of the countless individuals who made Geo-Extreme 2025 and these published proceedings possible, especially the reviewers, session chairs, organizing committee members, and ASCE staff listed in the acknowledgements.

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The conference and its published proceedings would not have been possible without the diligent, coordinated effort of numerous people, especially the reviewers who provided detailed feedback on papers and the session chairs who made decisions and provided editorial assistance on selected topics. The editorial team is also profoundly grateful to Dr. Daniel Pradel and Dr. Navid Jafari (Conference Co-Chairs), the Technical Program Committee (listed below), Mr. Brad Keelor, Ms. Aurora Phlegar, and the ASCE publications team for their crucial roles in the production of these proceedings.

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Probabilistic and Deterministic Approach to Assess Liquefaction Potential at Selected Sites Using SPT Data

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ABSTRACT

In this study, five major sites were assessed across Windsor-Essex Region in the Province of Ontario, Canada, to determine if an earthquake event with the probability of exceedance 2% in 50 years, i.e., a return period of 2,475 years, may lead to liquefaction. The simplified procedure suggested by Idriss and Boulanger was used to determine the liquefaction factor of safety and probability. The cyclic stress ratio (CSR) and cyclic resistance ratio (CRR) are depth functions; assessment was done with respect to depth for at least 3 borelogs per site. Sites were assessed for magnitude 6.0, 7.0, and 8.0 earthquakes. Results show that sites with higher groundwater tables, low *N* values, and low fine contents are prone to liquefaction, with probabilities ranging from 60% to 100%. The factor of safety in some scenarios was below 0.40. Extensive assessments should be conducted to prevent structural damage and loss of capital and lives.

INTRODUCTION

During soil liquefaction, soil deforms continuously under low to no residual resistance, building high pore water pressures. Very low or negative effective pressure occurs due to static and cyclic stress applications during an earthquake. Liquefaction is a catastrophic geotechnical event, causing structural failure, lateral spreading, slope sliding, and flooding, leading to billions in damages. Research identifies high hydrostatic pore pressure from cyclic shear stresses and high groundwater tables as primary causes. Cyclic stresses, mainly from shear wave propagation, compact cohesionless soil, transferring stress to pore water and reducing grain stress. When pore pressure reaches a critical level, sand grains deform significantly. Loose sand sees a rapid increase in pore pressure, causing excessive deformation. If sand deforms without resistance, it liquefies, offering no structural support.

The evaluation of soil liquefaction phenomena and related risk due to ground failure associated with earthquakes remains a challenging and complex process in geotechnical earthquake engineering. Identifying susceptibility to liquefaction before construction is crucial, as failure can significantly damage superstructures, foundations, underground structures, and substructures, leading to catastrophic consequences, casualties, and capital loss. Liquefaction susceptibility methods developed after the 1964 Niigata Earthquake in Japan, where damages exceeded $1 billion, mainly due to widespread liquefaction events. This disaster drew researchers' attention, highlighting the need for effective evaluation methods to mitigate losses. For vital structures like nuclear power plants and earthen dams, damage can be irreversible, making liquefaction assessment even more critical. A deterministic approach defines liquefaction susceptibility by the factor of safety, the ratio of Cyclic Resistance Ratio (CRR) to Cyclic Stress Ratio (CSR), while a probabilistic approach evaluates the probability of liquefaction when the

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factor of safety is below 1.41. Many researchers have contributed to advancing knowledge of earthquake-induced liquefaction.

The most widely used methods to assess liquefaction potential are the Cone Penetration Test (CPT) and Standard Penetration Test (SPT). Other methods, including the shear wave velocity test, Backer penetration test (BPT), and large penetration test (LPT), are used in special cases but less frequently than CPT and SPT. Both tests have advantages and disadvantages. SPT is widely standardized, cost-effective, easier to conduct than CPT, requires less skilled labor, and has extensive global data availability. The Niigata earthquake spurred liquefaction susceptibility studies, with Kishida (1966) using SPT data from affected areas. A breakthrough came in 1984-1985, when Youd et al. further studied liquefaction potential. Seed & Idriss (1971) introduced the "Simplified Procedure" which underwent multiple modifications in 1979, 1982, and 1985, with over 20 updates until Idriss & Boulanger's (2008) final modification, standardizing parameters at equivalent, effective pressure, σ_v' = 1 atm and Magnitude of earthquake, M_w = 7.5. Researchers such as Douglas et al. (1981), Shibata & Teparaksa (1988), Stark & Olson (1995), Suzuki et al. (1995), Robertson & Wride (1997), and Moss et al. (2006) developed SPT-based methods. With increased SPT & CPT data quality and quantity, confidence in earthquake-induced liquefaction evaluation has grown. Green et al. and Ballegooy et al. (2014) studied liquefaction events in the 2010-2011 Canterbury earthquakes in New Zealand, while Tokimatsu et al. and Cox et al. (2012-2013) analyzed the 2011 Tohoku earthquake in Japan. Green et al. presented 50 high-quality case studies, reinforcing SPT-based liquefaction analysis and enabling future mapping of liquefaction-prone zones.

This article uses the Idriss & Boulanger (2014) method, a stress-based approach utilizing Standard Penetration Test data, soil laboratory data, and ground motion parameters to determine CSR and CRR. The method, incorporated into the Italian seismic design code (N.T.C. 2008), does not consider liquefaction probability. Earthquake-induced stresses are derived from Peak Ground Acceleration (PGA), with hazard evaluation limited to PGA values of 0.10g or higher.

ANALYSIS FRAMEWORK

Earthquake Induced Cyclic Stress Ratio (CSR): The Cyclic Stress Ratio induced by earthquake ground motion represents the stresses caused by cyclic loads which trigger liquefaction in saturated loose cohesionless soil stratum. The CSR, at a given depth, z, within the soil profile, is usually expressed as a representative value equal to 65% of the maximum cyclic shear stress ratio, i.e.

$$\text{CSR}_{M,\,\sigma v'} = 0.65\left(\frac{\tau_{max}}{\sigma'_v}\right)$$

Here, τ_{max} is the maximum earthquake induced shear stress and σ_v' is the vertical effective stress for a specific moment magnitude and in-situ σ_v'. The choice of reference stress level 65% was selected by Seed & Idriss in 1967 and has been in effect since then. τ_{max} can be analyzed from the dynamic response of the earthquake but in Simplified Liquefaction Procedure (SLP), CSR is expressed as:

$$\text{CSR}_{M,\,\sigma v'} = 0.65\left(\frac{\sigma_v}{\sigma'_v}\right)\left(\frac{a_{max}}{g}\right) r_d \left(\frac{1}{MSF}\right)\left(\frac{1}{K_\sigma}\right)$$

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where σ_v is the vertical total stress at depth z, a_{max}/g is the maximum horizontal acceleration at the ground surface, i.e. Peak Ground Acceleration (PGA, available from Earthquake Canada Database), r_d is the shear stress reduction factor considering dynamic response of the soil profile, MSF is the Magnitude Scaling Factor and K_σ is the Overburden Correction factor.

Shear stress reduction factor (r_d) can be calculated analytically based on site specific soil layer thickness and stiffness or alternatively, Liao & Whitman in 1986 suggested following equations for stress reduction factor (r_d):

$r_d = 1.0 - 0.00765\ z$ for $z \leq 9.15$ m
$r_d = 1.174 - 0.0267\ z$ for $9.15\ m < z \leq 23$ m
$r_d = 0.774 - 0.008\ z$ for $23\ m < z \leq 30$ m
$r_d = 0.5$ for $z > 30$ m

The magnitude scaling factor, MSF, is used to correct the induced Cyclic Shear-stress Ratio (CSR) during an earthquake event of magnitude M to an equivalent CSR for an earthquake magnitude, M = 7.5, calculated as:

$$MSF = \frac{CSR_M}{CSR_{M=7.5}}$$

However, Idriss (1999) re-evaluated the MSF as:

$$MSF = 6.9 \exp\left(-\frac{M}{4}\right) - 0.058$$

Boulanger and Idriss (2004) found that overburden stress effects on the Cyclic Resistance Ratio (CRR) and proposed the following empirical equation to calculate K_σ:

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma vo'}{Pa}\right) \leq 1.1$$

The coefficient C_σ is expressed in terms of $(N_1)_{60}$:

$$C_\sigma = \frac{1}{\left(18.9 - 2.55\sqrt{(N_1)60}\right)} \leq 0.3$$

Where, $(N_1)_{60}$ is the corrected blow count.

Cyclic Resistance Ratio (CRR): The Cyclic Resistance Ratio (CRR) is based on the penetration resistance offered by soil strata correlated to CPT or SPT. In SPT, several corrections are applied before reaching to final N60 value are summarized by Idriss & Boulanger (2008, 2014) namely water table, standardized energy correction, rod length, hole diameter, sampler corrections and finally overburden stress correction is applied. All these correction tables are mentioned in the following subsections.

$$N_{60} = \frac{E_H\ C_B C_S C_R N}{0.60}$$

Here, N_{60} is the corrected SPT N-value for field procedures, E_H is hammer efficiency, C_B is derived from borehole diameter, C_S varies based on type of sampler, C_R is the rod length correction and N in the equation is the measured SPT N-value in the field.

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These corrections are applied irrespective of soil type. After applying these corrections, the overburden correction and water table correction is applied specially for cohesionless soils. Peck et. al. (1974) formulated the standard pressure of 100 kN/m^2, such as the field SPT N_{60} values to be corrected further for overburden using equation:

$$(N_1)_{60} = C_N \times N_{60}$$

In the equation, Overburden pressure correction factor $C_N = 0.77 \log (\frac{2000}{\sigma'_0})$. Here, σ_0' is in kN/m^2 or kPa.

Furthermore, researchers have suggested applying water table correction to account for water table effect in field. Terzaghi & Peck in 1948 suggested that water table correction should be calculated as:

$$(N_1)_{60,\text{ corrected}} = 15 + \frac{1}{2}((N_1)_{60} - 15)$$

Determination of Cyclic Resistance Ratio (CRR) requires fine content in each layer of soil and corrected SPT $N_{60,\text{ corrected}}$ values. Equation suggested by Idriss & Boulanger (2006) for CRR is:

$$\text{CRR} = \exp\left(\frac{(N1)60cs}{14.1} + \left(\frac{(N1)60cs}{126}\right)^2 - \left(\frac{(N1)60cs}{23.6}\right)^3 + \left(\frac{(N1)60cs}{25.4}\right)^4 - 2.8\right)$$

$$\text{Here, } (N_1)_{60,cs} = (N_1)_{60,\text{ corrected}} + \Delta (N_1)_{60}$$

$\Delta (N_1)_{60}$ is the correction for fines content (FC) present in soil and expressed as:

$$\Delta (N_1)_{60} = \exp\left(1.63 + \left(\frac{9.7}{FC}\right) - \left(\frac{15.7}{FC}\right)^2\right)$$

Deterministic Approach for Susceptibility of Liquefaction: A comparison of CRR and CSR values is defined as a safety factor value that implies soil susceptibility to liquefaction hazards. If the value of the safety factor $(FS)_{Liq}$ is less than 1 then the soil layer is considered vulnerable to liquefaction potential.

$$(\text{FoS})_{\text{Liq}} = \frac{CRR_M}{CSR_M}$$

Both CRR & CSR vary with depth and soil properties, hence the potential of liquefaction corresponds to depths within soil layers.

Probabilistic Approach for Liquefaction: The probability of liquefaction occurring is a statistical indicator to analyze the severity hazard for the given factor of safety. The resistance offered by soil and earthquake induced stresses are cyclic in nature and engineers should not rely solely on Factor of Safety approach. Hence, Sonmez (2003) came up with empirical equation to address the likelihood of liquefaction occurring defined as Probability of Liquefaction (P_L), calculated as:

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$$P_L(z) = \frac{1}{1+\left(\frac{FS_{Liq}}{0.96}\right)^{4.5}}, \text{ for } FS_{Liq} \leq 1.411 \;\&\; P_L = 0, \text{ for } FS_{Liq} > 1.411$$

Probability of Liquefaction (P_L) gives insight of likelihood of liquefaction event during an earthquake, which may be helpful in planning the remedial measures and estimate the extent of damage to plan.

STUDY AREA

The soil in any region is the reflect the factors affecting the formation of it. Soil in Windsor-Essex region is found to be glacial lakes deposits. A large stratum of sand is visible on the west end of old Sandwich West town near the new Gordie-Howe International Bridge (GHIB) site, which is the first site selected for this study. The other three sides in the City of Windsor were considered near the downtown Windsor Riverside, as the SPT records suggest that over the years the silty clay has been replaced partly by sandy granular deposit from Detroit River sediments and cutting-filling process for riverfront development works. Hence, it becomes important to study the area since engineering fill is loose in nature compared to natural deposit, saturated due to higher water table is prone to liquefaction. One site, Golfview located in Leamington, ON was also considered for the study as the water table effect can be compared in liquefaction analysis. All the locations are shown on the map below.

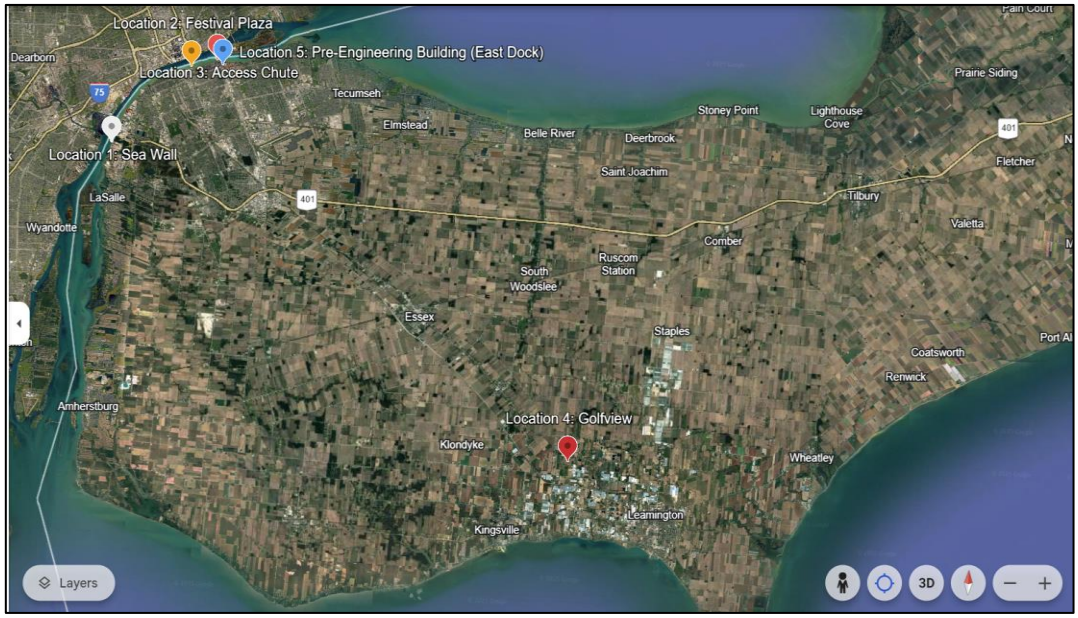


Figure 1: Selected Sites across Windsor- Essex Region for Liquefaction Potential Study (Source: Google Earth)

List of Selected Sites:

- Location 1: Seawall at Riverside, Windsor (Close Proximity of Gordie-Howe International Bridge)
- Location 2: Festival Plaza, Riverside at Downtown Windsor

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- Location 3: Access Chute near Riverside Drive E (Close proximity of a giant distillery)
- Location 4: Golfview housing to be constructed in Leamington, ON
- Location 5: Pre-Engineered Building at East Dock, Riverside Drive, Windsor, ON

DATA COLLECTION

The SPT N-value, soil classification, and majority of parameters were obtained from geotechnical soil exploration reports by Soil & Materials Engineering Inc., C.T. Soil Group of Companies in Windsor, Ontario. Data was approved by Professional Geotechnical and Geological Engineers with four decades of experience in the region. Fine content, unit weight, and moisture content were analyzed by C.T. Soil & Materials Testing Laboratory, approved by MTO and CCIL.

The stratigraphic and interpreted boundaries in the testholes were obtained from 35 mm diameter samples, retrieved from hollow stem auger boreholes. The subsurface conditions mentioned in tables 2 through 6 are defined as Fill consists of a mixture of brown/red sand with gravel and organic inclusions, while some contain a mix of sandy clay fill with gravel, along with traces of asphalt and saturated pockets of sand. GWS is a well graded crushed limestone with fine to medium grained sand. Lithology defined as SP is poorly graded, fine to medium grained sand with some silt. Silty Clay (CL-ML) is desiccated to plastic clay with silt, embedded sand and gravel, along with sand lenses and varying consistency at various depths. At some sites, Silty Sand (SM) encountered can be identified as saturated, fine to medium grained sand with non-plastic silt.

The study considers earthquake magnitudes from 6.0 to 8.0 Mw. Parameters such as unit weight, water table, N-values (uncorrected and corrected), correction factors, vertical stresses, fine content, stress-reduction factor, Cyclic Resisting Ratio, Cyclic Stress Ratio, Factor of Safety, and Probability of Occurrence change with depth. Site classification was based on uncorrected N-values for conservative safety observations.

Liquefaction Possibilities only exists in granular soil as discussed above, hence even if the empirical equations are only applicable to granular soils with less fines and not applicable for cohesive soils i.e. clayey soil with higher fine content.

Seismic parameters, including spectral acceleration and Peak Ground Acceleration (PGA) for 2% probability of exceedance in 50 years, were sourced from 2020 National Building Code of Canada (NBCC) tool of Earthquake Canada Database. This online tool offers spectral acceleration (Sa) values at various periods (e.g., 0.2s, 0.5s, 1.0s, 2.0s) and peak ground acceleration (PGA) for any location in Canada. Users can input a specific site location and select parameters such as the probability of exceedance (e.g., 2% in 50 years for this study) and site conditions (e.g., V_{s30} values or Site Class). Table 1 below shows the seismic parameters for study.

An acceleration response spectrum shows the maximum acceleration response of a series of single-degree-of-freedom (SDOF) systems with different natural periods (or frequencies), subjected to a specific ground motion or design input. Figure 2 shows the acceleration response spectrum for all five study locations at 5% damping (typical for most structures) and various periods of up to 10 seconds. Since the locations 1 through 3 and 5 are near by the change is very negligible and overlapping with each other in Figure 2 below.

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Table 1: Seismic Parameters for Study

Site Locations	Earthquake Site Classification	Peak Ground Acceleration
Location 1: Seawall at Riverside Dr	E	0.131g
Location 2: Festival Plaza at Downtown Riverside Dr	E	0.130g
Location 3: Access Chute near Riverside Dr E	E	0.130g
Location 4: Golfview housings, Leamington	D	0.151g
Location 5: Pre-Engineered Building, Riverside Dr	D	0.120g

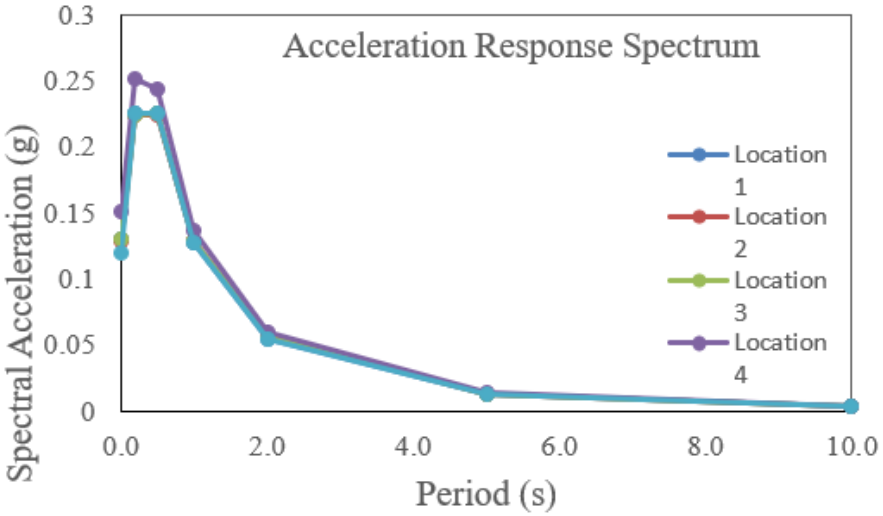


Figure 2: Acceleration Response Spectrum of Selected Locations (Source: Earthquake Canada)

RESULTS & DISCUSSION

Location 1: Seawall at Riverside Drive West

The seawall is located at Riverside West in City of Windsor. This site is in very close proximity to the new Gordie-Howe International Bridge (GHIB) project. Hence, studying liquefaction becomes important here. In table 2, necessary values are given for magnitude 6.0, 7.0 & 8.0 earthquake.

The water table was at 0.35m, which makes the site saturated as it is adjacent to Detroit River. Hence, loose sandy soil when subjected to earthquake may induce the liquefaction phenomenon. The site was covered with Clayey topsoil fill followed by layers of crushed well graded gravel with sand and some traces of silt. During magnitude 6.0 earthquake, factor of safety at 7.90m in layer of Poorly Graded Sand (SP) with 5% fines is 0.70 and probability of occurrence is 77%. As the magnitude of earthquake events increases, the factor of safety decreases for all layers and in magnitude 7.0 & 8.0 earthquake events probability is 92% & 97% respectively.

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Location 2: Festival Plaza at Downtown Riverside Drive West

This site is very close to Windsor- Detroit Underwater Tunnel Crossing. It houses many large structures, important government buildings, large public gatherings, and highly dense and well-developed infrastructure.

The water table was at 0.30m, which makes the site saturated as it is adjacent to Detroit River. The results show that soil layers from 4.55m (15.0ft) to 23.45m (77.0ft) have high probability of liquefaction but in such scenario, it is not considered to be vulnerable to liquefaction. The reason behind this is the lithology at these depths. liquefaction will not occur in this type of soil. However, at greater depth of 23.45m (77.0ft) and beyond the sand layer shows 98%, 99% & 100% probability of liquefaction during magnitude 6.0, 7.0 & 8.0 earthquake event.

Table 2: Site Properties for Location 1: Seawall at Riverside West, Windsor

Depth	Blow Count, N	Unit Weight, kN/m^3	Fines (%)	Lithology	Liquefaction Possibility
0.30	7	16.0	65	Fill	NO
1.05	19	15.2	65	Fill	NO
1.80	9	15.2	65	Fill	NO
2.55	7	16.1	10	GWS	YES
3.25	24	17.3	10	GWS	YES
4.05	12	18.01	10	GP	YES
4.80	11	17.8	10	GP	YES
6.30	12	17.1	10	GP	YES
7.90	5	17.9	5	SP	YES

Table 3: Liquefaction Analysis for Location 1: Seawall at Riverside West, Windsor

Depth	Earthquake Magnitude 6.0		Earthquake Magnitude 7.0		Earthquake Magnitude 8.0	
	FS_{Liq}	P_L	FS_{Liq}	P_L	FS_{Liq}	P_L
0.30	3.25	0.00	2.50	0.00	1.92	0.00
1.05	16.86	0.00	12.98	0.00	9.97	0.00
1.80	3.61	0.00	2.78	0.00	2.14	0.00
2.55	1.79	0.00	1.38	0.17	1.06	0.39
3.25	>100	0.00	82.35	0.00	63.21	0.00
4.05	2.27	0.00	1.75	0.00	1.34	0.18
4.80	2.10	0.00	1.62	0.00	1.24	0.24
6.30	1.98	0.00	1.53	0.00	1.17	0.29
7.90	0.70	0.77	0.60	0.92	0.4	0.97

Location 3: Access Chutes at Riverside Drive East

This is the data for Access Chutes structures to be built on the East side of Windsor Riverside. The lithology was Granular till 3.8m followed by Silty Clay all the way up to 10.0m depth. The water table at this location was 0.75m.

At this site, shallow liquefiable sands and granular soils may pose issues; soil replacement or appropriate densification could be more cost-effective.

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Table 4: Site Properties for Location 2: Festival Plaza, Downtown Riverside, Windsor

Depth	Blow Count, N	Unit Weight, kN/m^3	Fines (%)	Lithology	Liquefaction Possibility
2.30	26	16.00	8	Sand Fill	YES
2.50	28	15.20	3	Sand Fill	YES
3.05	41	15.20	4	Sand Fill	YES
4.55	2	16.10	73	CL-ML	NO
6.10	5	17.30	75	CL-ML	NO
7.65	3	18.01	89	CL-ML	NO
8.10	5	17.80	96	CL-ML	NO
9.15	4	17.1	911	CL-ML	NO
10.65	6	16.9	78	CL-ML	NO
23.45	1	14.95	4	SP	YES

Location 4: Golfview Site in Leamington County

The formation of this area is mainly sand with low fines. The soil exploration suggests the same, due to distance from the water source and high farming activity in and around the site, the water table was found to be at 8.1m.

It can be seen from the data, due to low ground water table ensures no saturation and high n-values suggests more denser and stiffer the soil. Blow count N >15 suggested that soil is dense to very dense condition. Here, the range is nearly 20 and above which makes it safer naturally. The purpose of taking this location in study was to examine and represent how the water table makes impact on liquefaction potential of any soil strata.

Table 5: Liquefaction Analysis for Location 2: Festival Plaza, Downtown Riverside, Windsor

Depth	Earthquake Magnitude 6.0		Earthquake Magnitude 7.0		Earthquake Magnitude 8.0	
	FS_{Liq}	P_L	FS_{Liq}	P_L	FS_{Liq}	P_L
2.30	>100	0.00	>100	0.00	>100	0.00
2.50	>100	0.00	>100	0.00	>100	0.00
3.05	>100	0.00	>100	0.00	>100	0.00
4.55	0.60	0.89	0.46	0.96	0.35	0.99
6.10	0.93	0.53	0.72	0.79	0.55	0.92
7.65	0.70	0.81	0.54	0.93	0.41	0.98
8.10	0.88	0.59	0.68	0.82	0.52	0.94
9.15	0.76	0.74	0.59	0.90	0.45	0.97
10.65	1.00	0.45	0.80	0.73	0.60	0.90
23.45	0.40	0.98	0.30	0.99	0.20	1.00

Location 5: Pre-Engineered Building at Riverside Drive East

This is an important pre-engineered building for a corporation with Site Classified as category D. At this site of the east dock of Windsor, ground Water Table level at 0.35m. The

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liquefaction study shows that sand fill and poorly graded sand (GP) layers are prone to liquefaction, with decreasing Factor of Safety (FS) as earthquake magnitude increases. Clayey silt (CL-ML) layers are non-liquefiable. The lowest FS (0.48) occurs at 4.55m depth during an 8.0 magnitude earthquake, indicating high susceptibility.

Table 6: Site Properties for Location 3: Access Chutes. Riverside Drive E, Windsor

Depth	Blow Count, N	Unit Weight, kN/m^3	Fines (%)	Lithology	Liquefaction Possibility
1.80	6	17.00	5	GWS	YES
2.30	16	16.00	7	GWS	YES
3.05	2	16.25	11	GWS	YES
3.80	7	16.30	9	SP	YES
4.55	20	16.25	78	CL-ML	NO
6.10	17	16.25	85	CL-ML	NO
7.65	3	16.25	86	CL-ML	NO
9.15	4	16.25	49	CL-ML	NO

Table 7: Liquefaction Analysis for Location 3: Access Chutes. Riverside Drive E, Windsor

Depth	Earthquake Magnitude 6.0		Earthquake Magnitude 7.0		Earthquake Magnitude 8.0	
	FS_{Liq}	P_L	FS_{Liq}	P_L	FS_{Liq}	P_L
1.80	1.20	0.27	0.92	0.55	0.71	0.80
2.30	4.54	0.00	3.50	0.00	2.68	0.00
3.05	0.57	0.92	0.44	0.97	0.33	0.99
3.80	1.04	0.41	0.80	0.69	0.62	0.88
4.55	>100	0.00	>100	0.00	>100	0.00
6.10	10.31	0.00	7.94	0.00	6.09	0.00
7.65	0.69	0.82	0.53	0.94	0.41	0.98
9.15	0.78	0.72	0.60	0.89	0.46	0.97

Table 8: Site Properties for Location 4: Golfview Project, Leamington, ON

Depth	Blow Count, N	Unit Weight, kN/m^3	Fines (%)	Lithology	Liquefaction Possibility
1.50	13	16.00	12	SP-SM	YES
3.05	30	16.00	6	SP-SM	YES
4.55	30	16.25	7	SM	YES
6.10	28	16.30	5	SP	YES
7.65	29	16.25	26	SP	YES
9.15	21	16.25	87	CL-ML	NO
10.65	18	16.25	15	SM	YES
12.2	22	16.25	36	SM	YES
13.75	31	16.70	13	SM	YES

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Table 9: Liquefaction Analysis for Location 4: Golfview Project, Leamington, ON

Depth	Earthquake Magnitude 6.0		Earthquake Magnitude 7.0		Earthquake Magnitude 8.0	
	FS_{Liq}	P_L	FS_{Liq}	P_L	FS_{Liq}	P_L
1.50	3.65	0.00	2.81	0.00	2.16	0.00
3.05	>100	0.00	>100	0.00	97	0.00
4.55	46.52	0.00	35.83	0.00	27.5	0.00
6.10	11.49	0.00	8.85	0.00	6.79	0.00
7.65	23.73	0.00	18.28	0.00	14.03	0.00
9.15	6.46	0.00	4.98	0.00	3.82	0.00
10.65	3.78	0.00	2.91	0.00	2.24	0.00
12.2	8.67	0.00	6.68	0.00	5.13	0.00
13.75	81.6	0.00	62.8	0.00	48.2	0.00

Table 10: Site Properties for Location 5: Pre-Engineered Building, East Dock, Windsor

Depth	Blow Count, N	Unit Weight, kN/m^3	Fines (%)	Lithology	Liquefaction Possibility
0.30	28	16.00	12	Sand Fill	YES
0.90	12	16.00	16	Sand Fill	YES
1.50	17	17.00	7	GP	YES
2.30	14	17.23	5	GP	YES
3.05	16	16.99	26	GP	YES
3.80	15	17.10	13	GP	YES
4.55	4	17.00	15	GP	YES
6.10	7	21.22	82	CL-ML	NO
7.65	9	21.59	88	CL-ML	NO
9.15	18	21.20	76	CL-ML	NO
10.65	5	22.01	84	CL-ML	NO

Table 11: Liquefaction Analysis for Location 5: Pre-Engineered Building, East Dock, Windsor

Depth	Earthquake Magnitude 6.0		Earthquake Magnitude 7.0		Earthquake Magnitude 8.0	
	FS_{Liq}	P_L	FS_{Liq}	P_L	FS_{Liq}	P_L
0.30	>100	0.00	>100	0.00	>100	0.00
0.90	4.00	0.00	3.00	0.00	2.00	0.00
1.50	10.65	0.00	8.20	0.00	6.30	0.00
2.30	2.96	0.00	2.28	0.00	1.75	0.00
3.05	9.79	0.00	7.54	0.00	5.79	0.00
3.80	3.67	0.00	2.83	0.00	2.17	0.00
4.55	0.82	0.67	0.63	0.87	0.48	0.96
6.10	1.42	0.00	1.09	0.36	0.84	0.65
7.65	1.60	0.00	1.2	0.23	1.0	0.50
9.15	5.20	0.00	4.00	0.00	3.1	0.00
10.65	1.10	0.38	0.80	0.67	0.6	0.87

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CONCLUSION

From the literature review and case studies across the selected locations, there are multiple critical factors that must be considered when drawing conclusions regarding soil liquefaction behavior and mitigation. The most important factor in liquefaction is the water table level. Due to the presence of a high water-table, effective stress decreases, and pore water pressure builds up, making it difficult for sand layers to resist liquefaction. N-values evaluate the soil compactness of coarse-grained soils, is another important factor in the liquefaction phenomenon. Loose sand layers with N-values less than 10 are more susceptible to liquefaction. Additionally, soil layers with fine content of less than 25% should be carefully analyzed. These are typically pure sands or possess the properties of pure sands and may be easily liquefied due to the absence of cohesion. Most sites across the Windsor riverside fall into category "E," which is classified as the most seismic hazardous site conditions and need to be considered for liquefaction potential during construction of heavily loaded structures.

Although the likelihood of a magnitude 6.0 or higher earthquake event in Windsor is very rare, appropriate precautions should still be considered as lower magnitude earthquake can also damage the structure severely.

Several techniques can be employed to improve the resistance of soils against earthquake-induced liquefaction. The selection of a suitable method depends on various factors, including the cost and importance of the structure, the required level of precaution, the depth of the liquefaction-prone layers, and the availability of materials and skilled workers. Remedial measures include soil compaction techniques such as dynamic compaction, vibro-compaction, and vibro-replacement. Dewatering methods like electro-osmosis consolidation can also be used to mitigate the liquefaction susceptibility. Chemical admixture techniques, including jet grouting and deep mixing, are effective in enhancing soil strength. Other soil improvement methods such as the use of reinforcement, geomembranes, and geotextiles can also be applied. In some cases, complete replacement of liquefiable soil or avoiding construction in high-risk areas may be the most effective approach.

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