Geotechnical Seismic Design in New England

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ABSTRACT

Earthquakes are not commonly associated to New England when compared to more seismically active regions of the United States. However, recent updates to seismic design codes and earthquake hazards mapping have significantly impacted geotechnical design and corresponding site development for projects including highway structures. A point of interest for this paper is the comparison of the current seismic design standards by the American Society of Civil Engineers (ASCE) to that of the American Association of State Highway and Transportation Officials (AASHTO).

Geology within New England is quite diverse including alluvium sand, sensitive marine clay, heterogenous glacial till, and a wide range of sedimentary, metamorphic, and igneous bedrock. Overburden thickness, or depth to bedrock, can also fluctuate greatly within a short distance resulting in a range of subsurface variability and uncertainty.

To investigate subsurface conditions, the current state of practice for geotechnical investigations within New England consist predominately of test borings. To improve geotechnical investigations within New England for seismic design, changes to explorations are necessary. These changes may include the use of geophysical testing such as multichannel analysis of surface waves (MASW) and seismic cone penetration testing ($SCPT_u$). Improved exploration methods will result in less conservatism for evaluating site classification and liquefaction potential and facilitate proper seismic code application to geology within New England. Discussion includes a case history showing comparison of the changes in seismic hazards mapping along with a comparison of exploration methodologies.

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INTRODUCTION

Earthquakes are not commonly associated to New England when compared to more seismically active regions of the United States such as the west coast, predominately California. Additional areas of elevated seismic activity include the middle of the country along the Mississippi River valley, coastal areas of South Carolina, and Alaska. Still, seismic activity has and will continue to occur within New England. The challenge for local design code officials, geologists, and engineers is the consideration of to what effect and risk do earthquakes have on existing and future infrastructure within New England?

That answer may still be up to debate. However, continual updates in engineering standards with provisions to seismic design are suddenly impacting local projects. While the provisions to seismic code and design are likely derived from more seismically active areas, the impacts are being felt local to New England. This is particularly important to geotechnical investigations where seismic loads and liquefaction potential were previously considered an afterthought or low risk. Evidence of this includes the widespread presence of large masonry brick buildings present within the heart of many older cities and towns. Most of these are constructed or retrofitted without geotechnical or structural considerations of any seismic loading given their age. For new projects built to updated standards, the seismic design criteria may actually govern the foundation and soil related buildability and construction method considerations previously reserved for bearing capacity and settlement limitations.

So why the issue? One reason is that many original communities in New England are located near the coastline or along river valleys because the early modes of transportation and source of energy were by water. Unfortunately, many of these areas are where the local geology consists of marine sediment such as soft clay or river valley alluvium such as loose sand and silt. The increase in mapped peak ground accelerations, recognized in newer standards, and unfavorable geology has created a challenge for geotechnical consultants within New England.

GEOTECHNICAL SEISMIC DESIGN CODES

The determination of appropriate seismic site classification and associated hazards are often necessary as part of reporting requirements for geotechnical investigations. The results are used for structural design of foundations for bridges, buildings, towers, and other similar structures. Reporting requirements generally include the following:

- Slope Instability
- Liquefaction Potential
- Total and Differential Settlement
- Surface Displacement Due to Faulting, Lateral Spreading, or Lateral Flow

The methodology for determining geotechnical seismic design parameters varies depending on the code applied. For most highway related projects, the AASHTO LRFD Bridge Design Specifications are adopted by individual state departments of transportation. State and city building codes adopt the International Building Code (IBC) which uses the recommendations provided in ASCE 7.

Is there much difference between the standards? Presently, this is one of the bigger challenges for local geotechnical engineers to sort out because seismic design standards vary between editions. Until recently, most states within New England utilized 2009 IBC, which reference the seismic design maps and procedures of ASCE 7-05. The maps and procedures were significantly modified for the recently adopted IBC 2015 which references ASCE 7-10. To date, new changes are established under ASCE 7-16 that will apply to future editions of IBC.

For highway projects the AASHTO Bridge Design Specification 8th Edition was released in 2017 replacing the 2007 AASHTO 4th Edition that was to be adopted by individual state departments of transportation. Additional publications by the Federal Highway Administration include, the LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations released in 2011, and the LRFD Seismic Analysis and Design of Bridges released in 2014. The current design manual editions for the New England States include:

- Maine DOT Bridge Design Guide (August 2003)
- New Hampshire DOT Bridge Design Guide (January 2015)
- Vermont AOT VTrans Structures Design Manual 5th Edition (2010)
- Massachusetts DOT LRFD Bridge Manual (2013)
- Connecticut DOT Bridge Design Manual (2003)
- Rhode Island DOT LRFD Bridge Design Manual (2007)

The use and details of each of these documents are beyond the scope of this paper. In summary, it appears most of the local DOT bridge design manuals still reference older methodology and mapping for determining seismic parameters as compared to the newer standards and mapping provided by ASCE 7 referenced in IBC.

SESIMIC DESIGN MAPPING

The National Earthquake Hazards Reduction Program (NEHRP) is a multi-agency program with focus on reducing losses due to earthquakes in the United States. The seismic design codes used by AASHTO, ASCE, and IBC generally adopt standards established from the NEHRP provisions. Interactive mapping programs for code related seismic design parameters are provided through the United States Geological Survey (USGS) website.

Deterministic Peak Ground Acceleration

The peak ground acceleration mapping by NEHRP includes provisions in 2003, 2009, and 2015 calculated as the largest 84th percentile geometric mean peak ground acceleration. From 2003 to 2015, the peak ground acceleration has increased significantly for sites located within central to southern Maine and New Hampshire and eastern Massachusetts. Below are graphic results of the corresponding changes for a select list of 15 locations within New England:

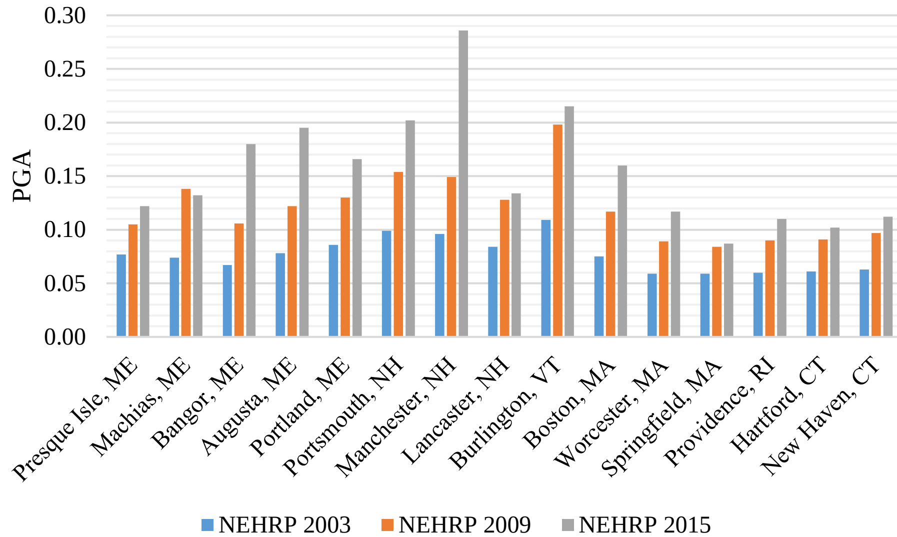

Figure 1a – Peak Ground Acceleration (Site Class B)

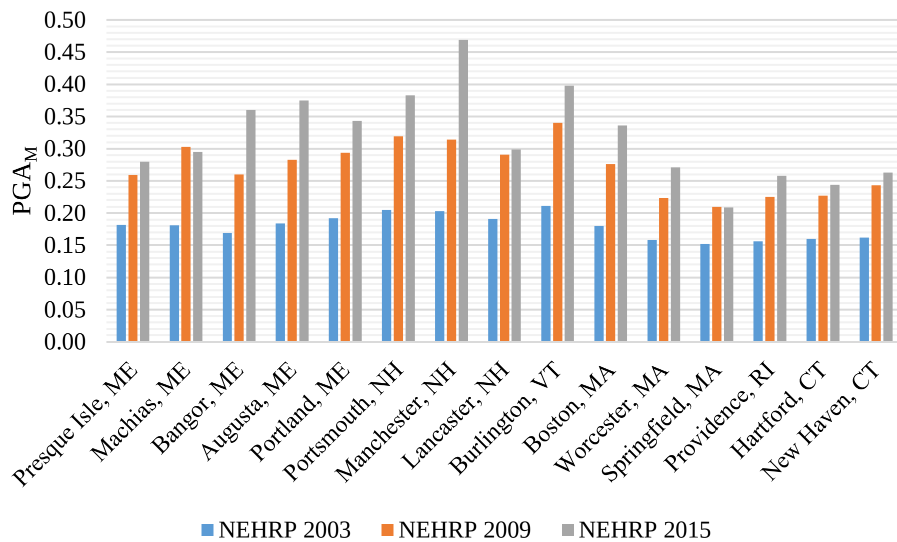

Figure 1b – Peak Ground Acceleration (Site Class E)

When comparing results of Figures 1a and 1b, it is important to note the increase in deterministic peak ground acceleration PGA_M from 2003 to 2015 ranging from 150% to 300%.

Probabilistic Method – Peak Ground Acceleration

The probabilistic geometric mean peak ground acceleration is commonly determined from hazard mapping provided by the United States Geological Survey (USGS). It is common for the designer to select a site specific peak ground acceleration determined as having a 2% probability of exceedance in 50 years.

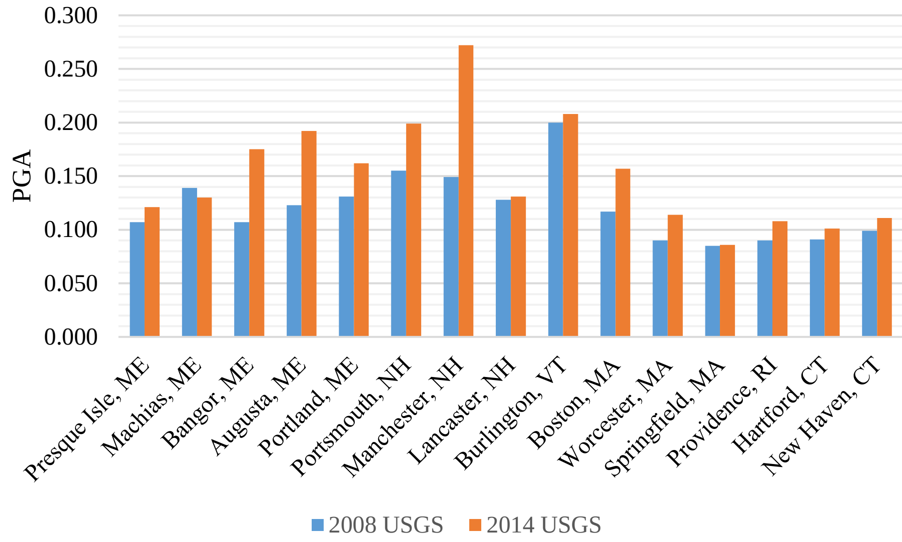

Figure 2 – USGS Hazards Mapping PGA (2% in 50 Years)

The probabilistic peak ground acceleration PGA does not account for soil strata such as shallow bedrock, stiff soils, or soft soils in comparison to the deterministic peak ground acceleration PGA_M which does account for the subsurface soil profile. Comparison of the mapping from 2008 to 2014 shows a 25% to 85% increase in southwestern Maine, southeastern New Hampshire, and northeastern Massachusetts.

Earthquake Magnitude

The earthquake magnitude a quantitative measurement of earthquake size derived from maximum ground shaking measured by a seismograph. The magnitude for a site can be estimated using the USGS Hazards Mapping for a specific location.

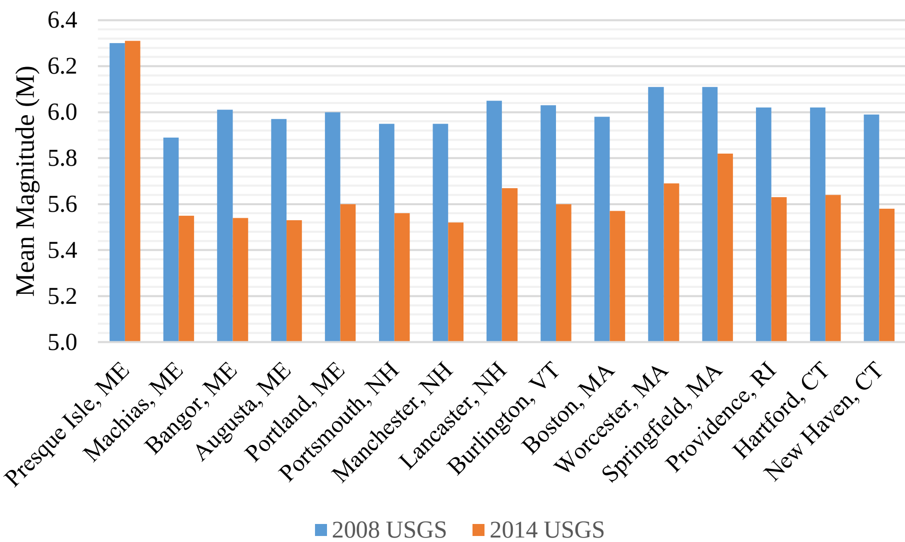

Figure 3 – USGS Hazards Mapping Magnitude (2% in 50 Years)

From observation of Figures 2 and 3, we can see a general increase in peak ground acceleration (PGA) from mapping in 2008 to 2014, however a decrease in earthquake magnitude. One can conclude the updated mapping has recognized an increase in the ground acceleration intensity but a decrease in the earthquake energy. In general, the mean magnitude has decreased from 6.0 to 5.5 for locations within New England. Historic earthquakes within New England include magnitudes of 5.5 or greater are summarized on Table 1.

Table 1 - Historical Earthquakes (5.5 Magnitude or Greater)

Location	Date	Magnitude
Central, NH	June 11, 1638	6.5
Newbury, MA	November 10, 1727	5.6
Cape Ann, MA	November 18, 1755	6.2
Eastport, ME	March 21, 1904	5.9
Whittier, NH	December 20 & 24, 1940	5.5

It might be reasonable to consider the topography and geology within New England having variability within a short distance. The Appalachian Mountain range extends through the central portion of New England. Glacial till and bedrock reside within higher elevations. River valley and flood plain alluvium reside at lower elevations along with widespread marine sediments towards the coastline extending to the Atlantic Ocean. Thus, the effects of variable topography might shorten duration of earthquake shaking. However, the presence of localized deeper alluvial and marine sediments would increase ground acceleration during earthquake.

Site Specific – Peak Ground Acceleration

In retrofitting existing structures and design of new structures, engineering judgement will be necessary to evaluate risk and application of peak ground accelerations. Presently IBC utilizes the procedure specified under ASCE 7. In determining the site-specific PGA for design, ASCE 7-10 states the designer may use the lesser of the probabilistic PGA (2% in 50 years) or deterministic PGA_M (84^{th} percentile) but not less than 80% of the deterministic PGA_M.

AASHTO design guides, as provided in the LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, permit the use of either the probabilistic or deterministic methods. Furthermore, AASHTO concludes the probabilistic approach incorporates all possibilities with respect to earthquake location, magnitude, and ground motion attenuation, producing a weighted average to estimate seismic activity. Thus, the probabilistic approach is often considered an appropriate basis for making rational design decisions about risk versus benefit. However, it is suggested that for critical structures of high importance utilizing deterministic methods or both may be most appropriate to evaluate site specific seismic risk.

In summary, it appears the utilization of probabilistic or deterministic PGA is still a fundamental difference between AASTHO and IBC design standards in current practice. To further assess seismic risk, AASTHO design standards incorporate hazard levels for bridge structures ranging from significant to minimal damage with consideration of bridge importance. Consideration needs to be given for an acceptable return period for seismic risk from earthquake.

The return period commonly ranges from 1,000 to 2,500-years with an expected life span ranging from 50 to 100 years for a bridge structure. The selection of a site-specific return period and hazard level for the structure will influence the peak ground acceleration applied for design.

SEISMIC SITE CLASSIFACTION

Determination of the seismic site classification is based on the results of a subsurface investigation using test borings or piezocone penetration testing. Alternatively, site classification is based on geophysical testing such as spectral analysis of surface waves (SASW), multichannel analysis of surface waves (MASW), downhole and/or crosshole shear wave velocity testing. The procedure for determining site classification adopted by ASCE Standard 7 and AASHTO 2009.

SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Figure 4 – Standard for Site Classification

The procedure defines a site as having a profile of 100 feet with classification ranging from A (hard rock) to E (soft soils) utilizing shear wave velocity v_s. Alternative methods for classifications C through E include the use of the standard penetration resistance (N) from test borings and/or undrained shear strength (s_u) from laboratory testing of cohesive soils.

The classification of A and B are assigned where the thickness of soil between bottom of foundation and surface of competent bedrock is 10 feet or less. Classification A is permitted when verified by onsite shear wave velocity testing or with testing and knowledge of similar geology along with confidence in competent bedrock. Classification B is applied where determined by shear wave velocity testing or where bedrock is estimated as competent with moderate weathering and fracturing. Classification of C is applied where bedrock is considered soft or more highly fracture and weathered unless verified by shear wave velocity testing.

The classification of C, D, or E are determined for the site-specific subgrade profile to 100 feet by using one of the three methods; average shear wave velocity, average standard penetration test (granular, cohesive, and rock), or individual use of standard penetration test for cohesionless layers with undrained shear strength for cohesive layers to 100 feet.

Most commonly the subgrade is evaluated using the N method for combined cohesionless, cohesive, and rock conditions for the profile of 100 feet. Value for N is determined for cohesionless and cohesive layers using the standard penetration test. Where rock is encountered, N is applied for the rock layer as being equal to 100.

Alternatively, the subgrade soil can be evaluated using N_{ch} for granular layers and undrained shear strength s_u for cohesive layers. The limit for cohesionless layers is an N value of 100 and for cohesive layers is 5,000 psf. Determination between cohesionless and cohesive soils is a plastic limit value of 20. Classification is determined base on the lesser value of the two individual methods.

Where soft clay soils are present a classification of E is assigned where soil profile 10 feet or greater in thickness includes; a plasticity index greater than 10, moisture content equal to or greater than 40, and an undrained shear strength of less than 500 psf.

The classification of F is applied where any of the following are present:

- Liquefiable soils, quick/highly sensitive clays, and collapsible weakly cemented soils.
- Peats and/or highly organic clays with thickness greater than 10 feet.
- High plastic clays greater than 25 feet with plasticity index greater than 75.
- Thick soft clays greater than 120 feet with undrained shear strength below 1,000 psf.

GEOTECHNCIAL EXPLORATIONS

Test Borings w/SPT Sampling

It is common practice for geotechnical investigations in New England to be conducted using conventional test borings. Test borings are performed by hollow stem auger or by rotary wash with driven casing. Sampling is conducted using the standard penetration test SPT to collect split spoon samples and to estimate the in-situ density of soils. In-situ field vane shear tests can be performed to obtain estimates of undrained shear strength. Thin wall tube samples can be collected for soft cohesive samples to perform laboratory testing in determining undrained shear strength. Rock core samples can be collected to determine rock type and quality.

Geology within New England is quite diverse and includes; loose alluvial sand, sensitive soft marine clay, heterogenous glacial tills, and a wide range of sedimentary, metamorphic, and igneous bedrock. Overburden thickness also fluctuates from loose soil to hard till over bedrock commonly within the upper 100-foot profile. The use of test borings provide versatility by an ability to collect data for a range of subsurface conditions common to New England.

Despite the versatility, the application of data collected from conventional test borings is generally a poor application for site classifications of D, E, and F. When below groundwater, sand can undergo upheave and disturbance during performance of standard penetration testing unless borehole hydrostatic pressure is maintained. This is commonly done with rotary wash and occasionally the use of drilling muds. Additionally, the standard penetration test is a poor measurement of cohesive strength, especially for soft clays common to marine deposits.

Seismic Piezocone Penetration Testing ($SCPT_u$)

An alternative to test borings for investigating the subsurface conditions is the performance of seismic piezocone penetration testing ($SCPT_u$). $SCPT_u$ is performed by a cone on the end of a series of rods pushed into the ground at a constant rate (2 cm/s) to obtain near continuous measurements of the resistance to penetration of the cone. Parameters obtained include cone resistance (q_c), sleeve friction (f_s), and piezocone pore pressure (u_2). The results are interpreted to obtain soil type and soil parameters for engineering design. Shear wave velocity tests are performed at select intervals, typically 1 meter (3-feet).

The in-situ shear wave velocity profile (V_s) can be obtained from shear wave testing performed during $SCPT_u$. Correlation for standard penetration resistance N and undrained shear strength S_u can be obtained independently from the same $SCPT_u$ test from cone penetration resistance for further evaluation of appropriate seismic site classification. The ability to match 3 independent methods of analysis utilizing one exploration provides the engineer with greater accuracy and less conservatism. The near continuous data acquisition by $SCPT_u$ provides enhanced profiling of the soil stratum and better identification of sub-layering.

Multichannel Analysis of Surface Waves (MASW)

Gaining in popularity are multichannel analysis of surface waves (MASW) surveys conducted to measure shear wave velocity profile (V_s) and to map stratum layering. MASW surveys are non-invasive and performed using a series of 24 or more geophones along a straight alignment at the ground surface. An energy source such as a steel plate and sledge hammer with a trigger switch are used to develop surface waves recorded at low frequency (1 to 30 H_z). The results are collected through a data acquisition system and then processed by dispersion properties to determine V_s profiles in 1D for depth and 2D for depth and location.

The advantage of conducting MASW surveys is the direct site measurement of shear wave velocity for determining appropriate seismic site classification. Additionally, approximate stratum layering between soils types and bedrock are possible from MASW data or additional high-resolution reflection and/or refraction surveys. Still, test borings and/or cone penetration testing should be performed to verify soil strata, depth to groundwater and/or bedrock, and for consideration of liquefaction potential.

Laboratory Testing

For most geotechnical investigations, laboratory testing is conducted on samples obtained from test borings. Laboratory tests determine both index and strength properties and can be helpful for estimating seismic site classification. Performance of index testing is required in determining the special requirement of site class E for soft clays where the undrained shear strength for cohesive soils is below 500 psf and thickness is greater than 10 feet.

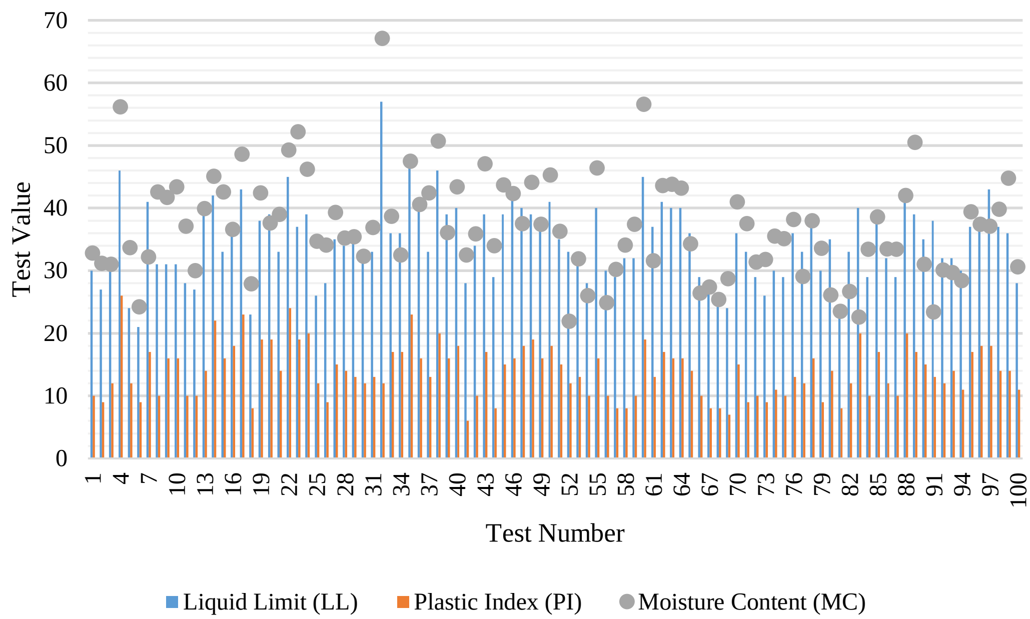

Figure 5 – Atterberg Limit Tests for Presumpscot Formation

Figure 5 shows summary of 100 Atterberg limit tests conducted for marine deposit (soft clay) further described as the Presumspcot Formation for various sites in Maine. The average liquid limit (LL) is 35 with a range of 57 to 21. The average plastic index (PI) is 14 with a range of 26 to 6. The average moisture content is 37 with a range of 67 to 22. In summary, the Presumpscot Formation comprises of lean clay with variable silt classifies as CL in accordance with the Unified Soil Classification System. Further, the lean clay is generally considered to have moderate to high sensitivity.

In comparison to the index requirements for site class E, the range of values are both below and above the criteria for essentially the same geologic formation. Thus, detail profiling of the undrained shear strength and index values is recommended for soft clay deposits.

LIQUEFACTION POTENTIAL

The LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations publication defines sites as having a Seismic Hazard Level as follows:

Table 2 – Seismic Hazard Level

Hazard Level	Using $S_{D1} = F_vS_1$	Using $S_{DS} = F_aS_s$
I	$S_{D1} \leq 0.15$	$S_{DS} \leq 0.15$
II	$0.15 < S_{D1} \leq 0.25$	$0.15 < S_{DS} \leq 0.35$
III	$0.25 < S_{D1} \leq 0.40$	$0.35 < S_{DS} \leq 0.60$
IV	$0.40 < S_{D1}$	$0.60 < S_{DS}$

Table 3 below presents data for 15 select sites within New England mapped with site class E soils using AASHTO 2009 seismic parameters.

Table 3 - AASHTO 2009 (Class E)			
Location	**PGA**	**S_{D1}**	**S_{DS}**
Presque Isle, ME	0.077	0.184	0.439
Machias, ME	0.074	0.138	0.368
Bangor, ME	0.067	0.153	0.362
Augusta, ME	0.078	0.159	0.404
Portland, ME	0.086	0.156	0.429
Portsmouth, NH	0.099	0.155	0.473
Manchester, NH	0.096	0.157	0.466
Lancaster, NH	0.084	0.178	0.447
Burlington, VT	0.109	0.198	0.552
Boston, MA	0.075	0.135	0.377
Worcester, MA	0.059	0.133	0.326
Springfield, MA	0.059	0.132	0.326
Providence, RI	0.060	0.122	0.318
Hartford, CT	0.061	0.128	0.330
New Haven, CT	0.063	0.122	0.331

The seismic accelerations in Table 3 are mapped as hazard level I using the criteria for spectral acceleration of 1 second (S_{D1}) and hazard levels of II and III using criteria for spectral acceleration of 0.2 second (S_{DS}) based on AASHTO mapping of 2009 for site class E. The discrepancy and relatively low peak ground accelerations suggests earthquakes within New England are considered to be short in duration and magnitude but higher in initial intensity as expected for soil class E. The discrepancy also suggests the criteria of Table 2 do not apply well to the mapped accelerations for New England currently published by AASHTO 2009.

For hazard levels I and II, the peak ground acceleration and earthquake magnitudes are 0.14g and 6.0 or less, respectively. Liquefaction potential for hazard levels I and II are considered low thus liquefaction analysis is not required. Further criteria are provided for determining the need for liquefaction analysis of hazard level III. As part of the AASHTO criteria, liquefaction analysis is not required for hazard level III where mean magnitude is less than 6.0. Hazard level IV sites are strictly required to have liquefaction analysis performed but are not mapped within New England. The mean magnitude in New England is generally mapped near 6.0 based on 2008 data and 5.5 using 2014 data by the USGS, as shown on Figure 3. Thus, evaluation of liquefaction potential is not generally considered necessary based on the screening criteria used by the hazard levels of I, II, and III. In comparison, ASCE 7-10 requires all sites having a soil class of D, E, or F be evaluated for liquefaction potential regardless of magnitude.

Screening Criteria of Granular Soils

Sandy soils are defined as being susceptible to liquefaction for sites of level III based on having corrected standard penetration tests (N_{60}) below 30 or normalized cone penetration resistance (q_c) below 160 ksf. Additionally, liquefaction analysis is not needed where groundwater is at a depth of 50 feet below grade or deeper. An exception exists where the mean magnitude is between 6.0 and 6.4 with an SPT N_{60} value below 20.

Screening Criteria of Cohesive Soils

Clayey soils are defined as being highly sensitive and susceptible to liquefaction for sites of level III and IV based on having all of the following:

- Liquid Limit below 40
- Moisture Content/Liquid Limit Ratio > 0.9
- Liquidity Index > 0.6
- SPT N_{60} below 5 or CPT resistance q_c below 50 ksf

The criteria are applicable for seismic hazard levels III and IV which are not common to New England. Still, soils satisfying the screening requirements as having liquefaction potential exist within geology local to New England. As an example, Table 4 shows a summary of 100 Atterberg limit tests conducted for Presumpscot Formation (soft clay) as provided in Figure 5. Results are summarized on Table 4 for comparison to the criteria outlined above.

Table 4 – Summary of Results for Atterberg Limits (Presumpscot Formation)

Index Value	Average	Maximum	Minimum
Moisture Content (MC)	37	67	22
Liquid Limit (LL)	35	57	21
Plastic Index (PI)	14	26	6
Liquid Index (LI)	1.2	2.2	-0.1
Ratio MC/LL	1.1	1.4	0.6

Comparison of the test results suggest possible conformance to the index criteria for clayey soils as being highly sensitive. Based on local experience, SPT N_{60} values for soft clay of the Presumpscot Formation are commonly below 2, CPT resistance q_c below 15 ksf, with an undrained shear strength of 1,000 psf or less. Additionally, the general properties of Presumpscot Formation are similar to Boston Blue Clay and higher elevations of Pleistocene Lake deposits in Vermont and New Hampshire.

In summary, because of the hazard levels determined through mapped accelerations of AASHTO 2009 and the mean magnitudes provided by USGS, liquefaction analysis is not generally triggered through the screening criteria. Still, sandy soil having low fines content and located below groundwater yielding SPT N_{60} values below 30 are commonly present in alluvial deposits prevalent in New England. Additionally, marine clays may also meet the requirements for being highly sensitive yielding liquefaction or potential for shear strength reduction during earthquakes. Thus, despite not satisfying the screening requirements, engineering analysis for liquefaction potential should still perhaps be checked using other published methodologies to better determine risk.

CASE EXAMPLE – RAILWAY FACILITY

The project consisted of a new site development for a 50,000 ft^2 steel frame building within a former railroad yard used for storage and minor maintenance for passenger rail service in Brunswick, Maine. Additional development included approach and descent railroad lines and pavement access drives for the facility. Preliminary geotechnical investigation included the performance of 4 test borings utilizing rotary wash drilling advanced to depths of 30 to 40 feet below ground surface (bgs). Further geotechnical investigation included 24 shallow test borings to depths of 10 to 20 feet bgs along with 4 seismic cone penetration tests ($SCPT_u$) to depths of 45 to 80 feet bgs. The subsurface conditions consisted of the following:

- (0 to 4 ft) *Existing Fill* - Sand with Variable Gravel, Silt, and Coal Ash
- (4 to 30/50 ft) *Marine Regressive Delta Deposit* – Sand with Variable Silt
- (30 to 60 ft CPT-1) (50 to 80 ft CPT-2) *Presumpscot Formation* – Silty Clay
- (4 to 8 ft) *Groundwater Depth* – Sand & Gravel Aquifer at 10 to 50 gpm Yield

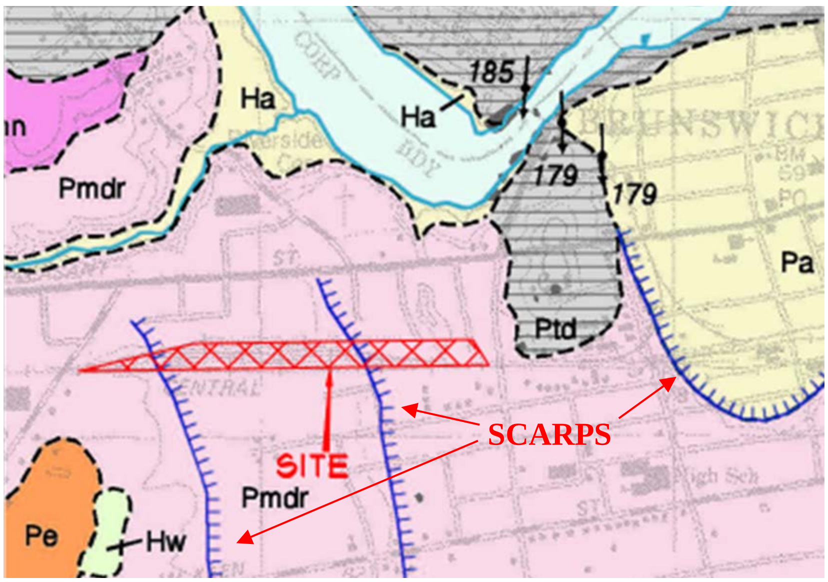

Figure 6 – Geological Mapping by Maine Geological Survey

Geologic scarps are mapped within the marine regressive delta deposit by the Maine Geological Survey as potentially located between the cone penetrometer CPT-1 and CPT-2 tests performed at a horizontal spacing of 200 feet. The geologic scarps represent a shift or division between historic stream channels. The findings suggest a scarp of 20 feet in elevation where the transition of sand to clay shifts from 30 to 60 feet bgs at CPT-1 to 50 to 80 feet bgs at CPT-2.

A challenge for geotechnical design of the facility was the presence of variable sand and silt located below a relatively shallow groundwater table overlying undulating layers of silty clay. Topography for the site was relatively flat requiring minimal grading for cuts or fills. Bearing capacity and estimated settlements of the upper soils were determined sufficient for the building foundations, railway tracks, and pavement areas based on the results of the shallow test borings and soils laboratory testing. Limitation for conventional site development was the determination of seismic site classification along with potential for liquefaction by earthquake.

Results of preliminary test borings conducted using rotary wash with split spoon sampling are compared to the correlated SPT-N_{60} values determined by seismic cone penetration tests ($SCPT_u$) shown below on Figure 7.

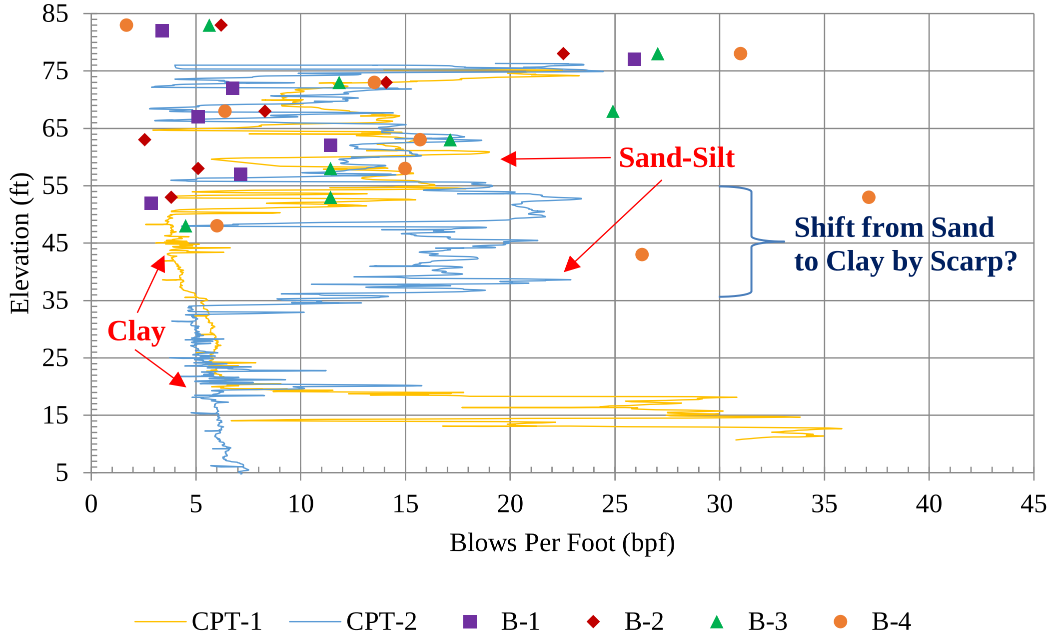

Figure 7 – Standard Penetration Test (SPT N_{60})

The average SPT-N_{60} value obtained from test borings performed from elevations 85 to 45 feet for the upper sand-silt layer is 12. The average correlated N_{60} determined from the cone penetration tests (CPT-1 and CPT-2) is also 12 showing agreement between methodologies. The average SPT-N_{60} for the complete profile from elevation 75 to 5 feet from the cone penetration tests is 12 with a range from 4 to 35.

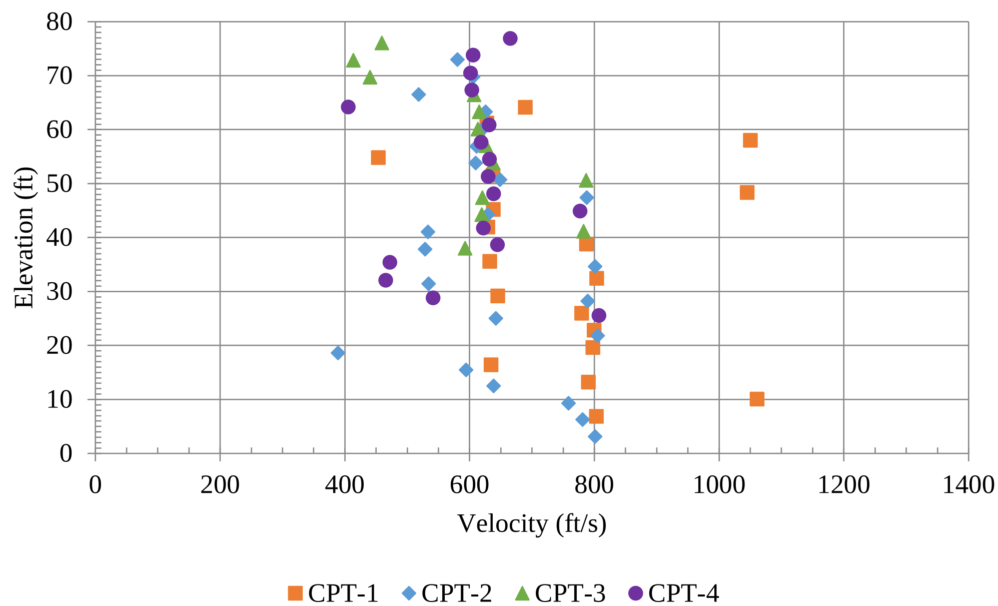

Figure 8 – Shear Wave Velocity (V_s)

The shear wave velocity obtained during performance of seismic cone penetration tests ($SCPT_u$) resulted in a range of 390 to 1,060 ft/s with an average of 665 ft/s. In determining seismic site class per Figure 4, a classification of E is determined by use of the N method (N < 15) where the average N_{60} value for the profile is 12. However, by use of shear wave velocity (V_s) where the average for the profile is 665 ft/s ($V_s > 600$ ft/s) the site is classified as D. The undrained shear strength (S_u) estimated from cone penetration tests for the underlying Silty clay range from 1,000 to 2,000 psf with an over-consolidation ratio (OCR) of 1.5 to 3.0 precluding the special requirements for soft clay soils ($S_u < 500$ psf).

Determination between site class D and E can significantly impact the applied peak ground acceleration used for liquefaction analysis. Below is a list of variable mapped peak ground acceleration for the site using deterministic and probabilistic methods.

Table 4 – Deterministic Method for PGA			
Reference	**PGA**	**PGA_M (Class D)**	**PGA_M (Class E)**
2009 AASHTO	0.079	0.104	0.164
2003 NEHRP	0.118	0.123	0.186
2009 NEHRP	0.122	0.189	0.283
2015 NEHRP	0.170	0.248	0.348

Table 5 – Probabilistic Method for PGA		
USGS	**PGA**	**Mean Magnitude**
2008	0.123	6.0
2014	0.168	5.6

At the time of design for the project, local and current codes included ASCE 7-05 along with AASHTO 2009. The peak ground acceleration for site class D determined by 2009 AASHTO was 0.104g and for ASCE 7-05 utilizing NEHRP 2003 was 0.123g. Additionally, the probabilistic peak ground acceleration obtained from the 2008 USGS hazards mapping was 0.123g at a mean earthquake magnitude of 6.0. For design of the project, a peak ground acceleration of 0.123g at a magnitude of 6.0 was used.

Liquefaction potential was evaluated using the results from the seismic cone penetration tests ($SCPT_u$) and methodology provided by Robertson, et al, in Guide to Cone Penetration Testing for Geotechnical Engineering 5th Edition. The methodology utilizes the fundamental equation for liquefaction analysis of balancing the soil strength cyclic resistant ratio (CRR) and the earthquake forces as cyclic stress ratio (CSR). The factor of safety against liquefaction under design earthquake loading is fundamentally determined by the ratio of CRR/CSR. The soil strength is estimated by cone penetration resistance (q_c) along with adjustment based on soil type such as variability of fines and effective stress parameters. The earthquake force is estimated by the design peak ground acceleration along with adjustments of magnitude, deposit thickness, and effective stress parameters. Use of CPT data provides a more rigorous evaluation for liquefaction potential due to the near continuous profiling data along with in-situ measurements for cone resistance, friction ratio, and drainage properties from piezometer pore pressure for estimating soil behavior type.

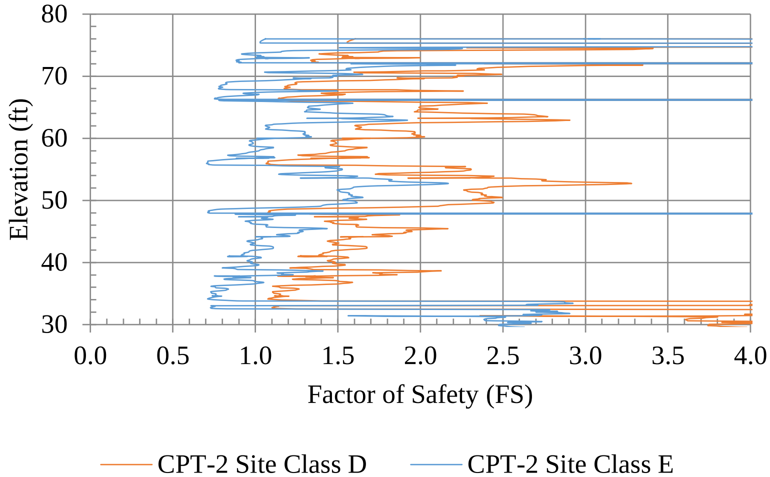

Figure 9a – Shear Wave Velocity (V_s) by NEHRP 2003

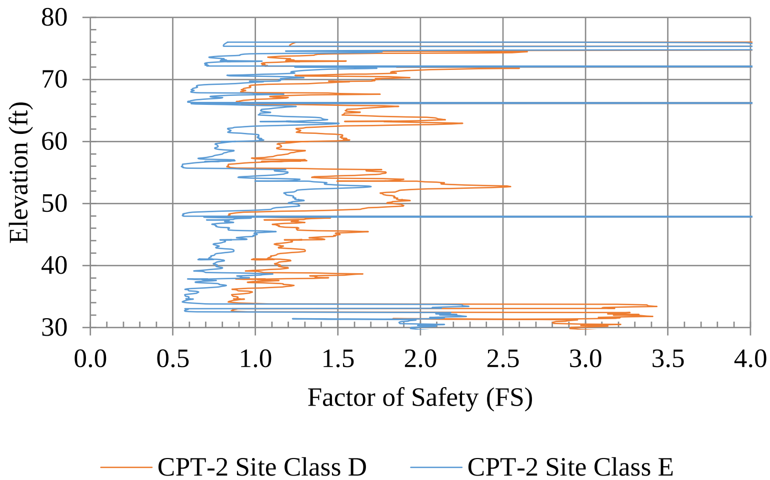

Figure 9b – Shear Wave Velocity (V_s) by NEHRP 2009

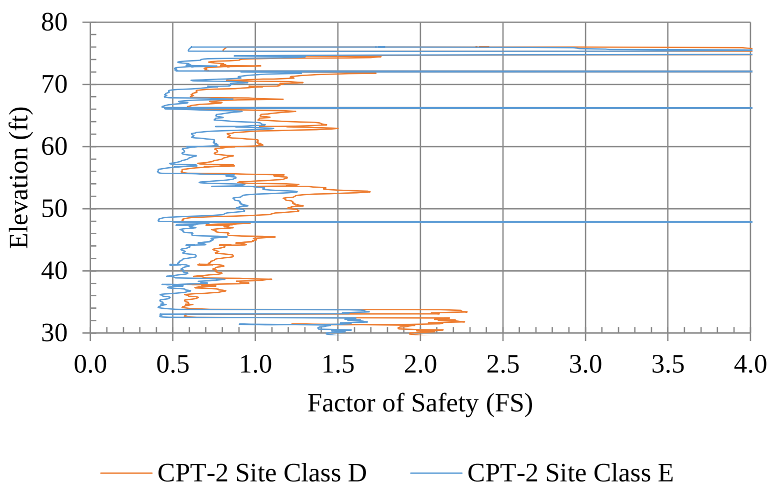

Figure 9c – Shear Wave Velocity (V_s) by NEHRP 2015

Reviewing the results for liquefaction potential, the profile is fully resistant from liquefaction using NEHRP 2003 site class D, and slightly susceptible using site class E. The potential for liquefaction increases using NEHRP 2009 for both site class D and E where the peak ground acceleration is increased but magnitude is decreased from 6.0 to 5.6. The potential for liquefaction becomes widespread using NEHRP 2015 for both sites classed D and E due to the increase peak ground acceleration.

The results compare the same profile of CPT-2 simply adjusting the earthquake parameters for peak ground acceleration and magnitude. Based on this comparison, it appears likely that many sites located within New England that are marginal or slightly above susceptible to liquefaction may eventually, under future seismic mapping and associated codes, be deemed risk to widespread liquefaction which can significantly affect foundation design, construction methods employed, and overall site development feasibility.

CLOSURE

Let's recap to the question of, to what effect and risk do earthquakes have on existing and future infrastructure within New England? The findings of this paper show a comparison of the past, present, and future mapping recommendations by NEHRP that recognizes an increase in peak ground accelerations suggesting an increase in risk. Conversely, the mapping also recognizes a decrease in earthquake magnitude suggesting the general risk of earthquake may actually remain the same and simply the application of seismic design parameters is evolving.

For engineering design, the determination of seismic site classification, hazard risk, and liquefaction potential is dependent on the code applied such as AASHTO, ASCE, and IBC. In current practice, it appears better awareness for evaluating liquefaction potential is provided by ASCE as compared to AASHTO but at perhaps a more conservative approach when using traditional exploration methods such as test borings. Determining a site specific seismic site classification by method of shear wave velocity may reduce conservatism when a result of higher site classification is determined as compared to the traditional method of N from test borings.

The case example further shows the sensitivity of increase peak ground acceleration with an increase liquefaction potential. Because of this, the updated mapping recommendations by NEHRP may result in an increase use of ground improvements such as stone columns, deep dynamic compaction, or even alternative pile support foundations. Improving geotechnical investigations to better predict liquefaction potential and determine site seismic classification will be necessary. In closing, the points of emphases for this paper include the following:

- Updates in seismic mapping by NEHRP will increase design peak ground acceleration.
- Variations in codes will bring discrepancy for geotechnical seismic design risk.
- Techniques such as $SCPT_u$ and MASW should be considered for site class determination.
- Sand deposits for both seismic class D and E may still present liquefaction potential.
- Clay deposits should be evaluated for special case conditions and level of sensitivity.

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