SPR PROJECT # C-09-04

VERIFICATION / DEVELOPMENT OF SEISMIC DESIGN SPECIFICATIONS FOR DOWNSTATE ZONE

FINAL REPORT
JULY 2014

Principal Investigators

Anil K. Agrawal, Ph.D., P.E.
The City College of New York
Department of Civil Engineering
New York, NY 10031
Tel. (212) 650-8442

Huabei Liu, Ph.D.
The City College of New York
Department of Civil Engineering
New York, NY 10031
Tel. (212) 650-8007

University Transportation Research Center
New York State Department of Transportation
(UTRC/NYSDOT)

DISCLAIMER

The contents of this report reflect the views of the author who is responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the New York State Department of Transportation, the New York City Department of Transportation, the United States Department of Transportation, or the Federal Highway Administration. This report does not constitute a standard, specification, regulation, product endorsement, or an endorsement of manufacturers.

DISCLAIMER

This report was funded in part through grant(s) from the Federal Highway Administration, United States Department of Transportation, under the State Planning and Research Program, Section 505 of Title 23, U.S. Code. The contents of this report do not necessarily reflect the official views or policy of the United States Department of Transportation, the Federal Highway Administration or the New York State Department of Transportation. This report does not constitute a standard, specification, regulation, product endorsement, or an endorsement of manufacturers.

EXECUTIVE SUMMARY

The New York City Department of Transportation (NYCDOT) Seismic Design Guidelines Report was updated in September 2008 by Weidlinger Associates to reflect current state-of-the-art knowledge. The NYCDOT seismic design guidelines are for use in the Downstate Zone which consists of New York City, Rockland County, Westchester County and Nassau County. NYSDOT has adopted the AASHTO LRFD Seismic Design Specifications for the Upstate Zone. The NYCDOT Seismic Design Guidelines Report (September 2008) proposed for use in the Downstate Zone has some key differences with the current AASHTO LRFD Bridge Design Specifications. The main objectives of this project has been to (i) carry out independent assessment of the approach proposed in the New York City Department of Transportation (NYCDOT) Seismic Design Guidelines Report (September 2008) by evaluating the methodology and assumptions used in the development of the report and by critically commenting on the results of the report, (ii) determine if the methodology and assumptions used are acceptable, (iii) determine the effects of the NYCDOT Seismic Design Guidelines Report by developing examples of critical and essential bridges for soil types A to E showing comparisons of NYCDOT and AASHTO LRFD spectra curves (iv) estimate the design and construction cost differences using NYCDOT and AASHTO LRFD Guide Specifications for the seismic design of bridges and (v) develop New York State Blue Pages to be used with the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

Numerous changes in the NYCDOT seismic design guidelines have been proposed and incorporated. Some of the significant changes in the Weidlinger Associates report are (i) use of 1000-Yr instead of 500-Yr earthquake as the lower level earthquake for the design of critical bridges, (ii) generation of response spectra for 1000-Yr return period earthquakes using response spectra for other return periods, (iii) use of 1000-Yr earthquake for the design of noncritical bridges while giving the owners the option of using 1500-Yr earthquake, (iv) disallowance of waiver for the site-specific analysis, (vi) use of shear wave velocity for seismic site characterization and design, (vi) determination of seismic design category based on SD_1, (vi) Site classification based on site-specific data, especially for critical bridges and (vi) response spectrum analysis using only two horizontal components (i.e., vertical component shouldn't be included in the response spectrum analysis).

While these changes have improved the applicability and effectiveness of seismic design of bridges in the downstate region, one of the critical issues identified by the research team couldn't be resolved. The horizontal design spectra of Rock Class A and Rock Class B are directly obtained from the VHR spectra by applying constant ratios, 1.15 and 1.65 respectively for Rock Class A and Rock Class B, to spectral accelerations at all periods. The site factor of 1.65 for Rock Class B was based on an analysis suite that included only one profile of very thick soft rock (3000 ft), whereas a relatively large number of thin soft rock profiles were considered. The consultants believe that these factors are too conservative and should be developed based on a profile suite that considers a broader range of profile depths, with the results averaged according to weights assigned to the various possible profiles. While this work was out of scope for this project, the consultants recommend NYCDOT / NYSDOT to carry out the proposed work so that unnecessary conservatism in the seismic design of bridges can be minimized, resulting in significant economical savings.

A comparative cost analysis of NYCDOT and AASHTO LRFD (or USGS in case of 2500-Yr earthquake) shows that the response quantities of a bridge subjected to NYCDOT spectra are generally higher than those using AASHTO LRFD (or USGS spectra) for earthquakes of all return periods. However, maximum increase has been seen for the soil site C where response quantities using NYCDOT spectra are amplified by 2.69 to 3.15 over those obtained by using AASHTO LRFD or USGS spectra of identical return periods. However, relative increase in costs, including design and construction, of using the NYCDOT spectra is not very significant as compared to the total cost of the bridge construction or rehabilitation. Overall, increase in construction cost to satisfy seismic design requirements will be higher when NYCDOT design spectra are used.

Detailed NYSDOT blue pages have also been developed for the implementation of the seismic design guidelines for bridges in downstate region. These blue pages already incorporate changes proposed by the research team and they will need to be adopted by NYSDOT to use the NYCDOT seismic design Guidelines in conjunction with AASHTO Bridge Seismic Design Guide Specifications.

STATEMENT ON IMPLEMENTATION

The main outcome of this project is the peer-reviewed New York City Department of Transportation Guidelines for Seismic Design of Bridges in the Downstate Region of New York State. The report is the outcome of extensive peer review of seismic design guidelines developed for the New York City Department of Transportation by Weidlinger Associates. The main outcome of the peer review has been incorporated into the seismic design guidelines for the downstate region that will be used in conjunction with NYSDOT Blue Pages. Hence, the outcome of this project is going to be implemented for seismic design of new bridges and retrofit of existing bridges in the downstate region.

<table>
<tr><td>1. Report No. C-09-04</td><td>2. Government Accession No.</td><td colspan="2">3. Recipient's Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle
Verification/Development of Seismic Design Specifications for Downstate Zone</td><td colspan="2">5. Report Date: July 2014</td></tr>
<tr><td colspan="2">6. Performing Organization Code</td></tr>
<tr><td colspan="2">7. Author(s)
Anil Agrawal, Huabei Liu, Jonathan Stewart, Hongfan Wang, Yuzhen Han, Roy Imbsen and Michael Abrahams</td><td colspan="2">8. Performing Organization Report No.</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address
The City College of New York
Department of Civil Engineering
New York, NY 10031</td><td colspan="2">10. Work Unit No.</td></tr>
<tr><td colspan="2">11. Contract or Grant No.</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address
NYS Department of Transportation
50 Wolf Road
Albany, New York 12232</td><td colspan="2">13. Type of Report and Period Covered Final Report</td></tr>
<tr><td colspan="2">14. Sponsoring Agency Code</td></tr>
<tr><td colspan="4">15. Supplementary Notes</td></tr>
<tr><td colspan="4">16. Abstract
The New York City Department of Transportation (NYCDOT) Seismic Design Guidelines Report was updated in September 2008 by Weidlinger Associates to reflect current state-of-the-art knowledge. The NYCDOT seismic design guidelines are for use in the Downstate Zone which consists of New York City, Rockland County, Westchester County and Nassau County. NYSDOT has adopted the AASHTO LRFD Seismic Design Specifications for the Upstate Zone. The NYCDOT Seismic Design Guidelines Report (September 2008) proposed for use in the Downstate Zone has some key differences with the current AASHTO LRFD Bridge Design Specifications. The main objectives of this project has been to (i) carry out an independent assessment of the proposed New York City Department of Transportation (NYCDOT) Seismic Design Guidelines Report (September 2008) by evaluating the methodology and assumptions used in the development of the report and by critically commenting on the results of the report, (ii) determine If the methodology and assumptions used are acceptable, (iii) determine the effects of the NYCDOT Seismic Design Guidelines Report by developing examples of critical and essential bridges for soil types A to E showing comparisons of NYCDOT and AASHTO LRFD spectra curves (iv) estimate the design and construction cost differences using NYCDOT and AASHTO LRFD Guide Specifications for the seismic design of bridges and (v) develop New York State Blue Pages to be used with both the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for LRFD Seismic Bridge Design. Numerous changes in the NYCDOT seismic design guidelines have been proposed and incorporated. Recommendations for future improvement of the guideline have also been provided.</td></tr>
<tr><td>17. Key Words: Seismic Design of Bridges, AASHTO LRFD Guide Specifications, NYCDOT Seismic Design Guidelines</td><td colspan="3">18. Distribution Statement</td></tr>
<tr><td>19. Security Classif. (of this report): None</td><td>20. Security Classif. (of this page) None</td><td>21. No. of Pages:104</td><td>22. Price</td></tr>
</table>

ACKNOWLEDGMENTS

The research presented in this report has been sponsored by New York State Department of Transportation (NYSDOT) through FHWA SPR Project No. C-09-04. Special thanks go to the members of the NYSDOT Technical Working Group (TWG) consisting of Rajesh Taneja (Project Manager), Abdur Razzaq (NYCDOT), Art Yannotti (NYSDOT), Diego Munoz (NYSDOT), Gary Douglas (NYSDOT), Michael Bergmann (NYSDOT), and Daniel Byer (FHWA) for providing supervision and feedback during the peer review process. Support and feedback of Mr. Richard Marchione of NYSDOT, Mr. Udaya Dommaraju and Mr. Anil Vyas of NYCDOT have been instrumental during the execution of the peer review work.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1 : REVIEW OF THE NYCDOT SEISMIC DESIGN GUIDELINES REPORT

1.1 Introduction

Considering the specific geological condition in the New York City area and the new development in seismic hazard analysis, Weidlinger Associates and consultants prepared the *NYCDOT Seismic Design Guidelines Report (September 2008)* based on the 2002 report *Seismic Hazard for New York* by Risk Engineering and the 2003 Edition *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures.* The Guidelines Report differs from the 1998 NYCDOT guidelines on many critical aspects, including new Very-Hard-Rock (VHR) seismic hazard for NYC region, new soil and rock site classification methods, new generic design spectra at soil sites as a function of three parameters based on a fully probabilistic approach, and more detailed guidance on soil liquefaction, spatial variation and site specific studies. These new aspects are based on certain theory, methodology or assumption, all of which require critical assessment. Based on a preliminary review of the 2008 NYCDOT guidelines, eight sub-tasks were identified to review the *NYCDOT Seismic Design Guidelines Report (September 2008)*. Work carried out under these subtasks is described in sections 1.2 to 1.9 of this report.

1.2 Bridge Classification and Performance Criteria

The section "Bridge Classification and Performance Criteria" in the NYCDOT Guidelines Report is not very much different from that in the 1998 NYCDOT guidelines. The major aspects of this section include a) the specific descriptions of the bridge classification and damage level; b) a new 1500-yr return period seismic event for Non-Critical Bridges (Essential and Other); and c) requirements on site specific study. Accordingly, this section reviews and evaluates these aspects.

Bridge Classification

The Guidelines Report classifies all NYCDOT bridges into Critical, Essential and Other Bridges. The research team thinks that this classification is also relevant to the NYSDOT bridges in downstate New York.

The research team agrees with the definitions of Critical and Essential Bridges and definitions of performance levels in section 2.1 in the Guidelines Report.

Performance Requirements

The research team agrees with two-level design earthquakes for critical bridges and the associated performance requirements. However, the use of 1500-yr earthquake for Essential and Other Bridges doesn't seem to be necessary. Instead, 1000-yr earthquake with a performance level of "Reparable Damage" for essential bridges and a performance level of "Significant Damage" (as per the definitions in section 2.1 of the Guidelines Report) for other bridges can be adopted. The rationale for using a 1000-yr earthquake is discussed in the next section.

Design Earthquake of 1000-yr Return Period vs. 1500-yr Return Period

Following the rejection of NCHRP 12-49 by AASHTO because of extremely conservative design, Task 193 under NCHRP 20-07 was initiated to explore the development of acceptable seismic guideline. The final recommendations of this task formed the basis of the AASHTO Seismic Guide Specifications. As per NCHRP 20-

07/Task 193, "Selection of a lower return period for Design is made such that Collapse Prevention is not compromised when considering historical large earthquakes. This reduction can be achieved by taking advantage of sources of conservatism not explicitly taken into account in current design procedures. These sources of conservatism are becoming obvious based on recent findings from both observations of earthquake damage and experimental data." Reduction here refers to the use of 1000-yr return period instead of 2500-yr return period, as recommended for other bridges in NCHRP 12-49.

Table 1.1 shows some sources of conservatism that are not accounted for during the design and construction, but they contribute to increase in resistance of bridge components during an earthquake. Considering this conservatism in the design and construction, seismic design was decreased from 2500-yr return period earthquake to 1000-yr return period earthquake for collapse prevention, or significant damage performance, as per AASHTO Seismic Guide Specifications (2009). Overall, the AASHTO Seismic Guide Specifications (2009) contains a safety factor of around 1.5 based on conservatism reported in Table 1.1 with the understanding that hinging mechanism will contribute to energy dissipation before collapse during earthquakes equal to or greater than 1000-yr return periods. With these sources of conservatism in place, the research team believes that it is not necessary to design essential and other bridges using an earthquake larger than 1000-yr return period. Based on the same logic, if a performance level of reparable damage is required, it is not necessary to adopt a design earthquake larger that 1000-yr return period for essential bridges.

Table 1.1: Identified sources of conservatism

Source of Conservatism	Safety Factor
Computational vs. Experimental Displacement Capacity of Components	1.3
Effective Damping	1.2 to 1.5
Dynamic Effect (i.e., strain rate effect)	1.2
Pushover Techniques Governed by First Plastic Hinge to Reach Ultimate Capacity	1.2 to 1.5
Out of Phase Displacement at Hinge Seat	Addressed in Task 3

The CUNY research team has investigated the response of a three-span bridge with 0.92 sec period subjected to both NYCDOT and AASHTO earthquakes of different periods. The bridge has two piers of cross-section 1.4 m x1.4 m with 4.02 m and 7.5m height, denoted as bents 2 and 3, respectively. Capacities of these piers based on pushover analysis are shown in Table 1.2 below. Table 1.3 below shows displacement demands on bridge piers during AASHTO and NYCDOT earthquakes of different return periods and C/D ratios for the case of NYCDOT hazards for the bridge on Soil Type D. It is observed that the bent 2 has a C/D ratio of 0.99 during 2500 Yr earthquake, i.e., it is likely to suffer significant damage during a 2500 Yr earthquake event, even though it hasn't been designed for this earthquake.

It should be noted that bridge piers are based on older seismic design detailing for a region outside downstate region. Even then, the bents are likely to survive 2500 Yr earthquake with significant damages. Displacement capacities of two piers are likely to be significantly higher, if they are redesigned as per current AASHTO Guide Specs, since the seismic detailing corresponding to SDC B will be applicable. This detailing is based on $\mu = 2$, which is close to essentially elastic behavior. Hence, ultimate capacity based on the pushover analysis will be significantly higher than that in Table 1.3. These results clearly demonstrate that the use of 1000-Yr earthquake for the design of non-critical bridges is sufficient to sustain 2500 Yr earthquake with significant damages. This performance level is acceptable because of the bridge being non-critical.

Table 1.2: Displacement capacities for bents of the bridge

Bent	Pushover Analysis
2	0.0968 (m)
3	0.1665 (m)

Table 1.3: Displacement demands and C/D ratios for the case of NYCDOT spectra.

Return Period	Displacement Demand (NYCDOT)	Displacement Demand (AASHTO)	C/D Bent 2 (NYCDOT)	C/D Bent 3 (NYCDOT)
2500	3.86 in (0.0980m)	2.34 in (0.0595m)	0.99	1.70
1500	2.71 in (0.0689m)	-	1.40	2.41
1000	1.97 in (0.0500m)	1.44 in (0.0366m)	1.94	3.33

In short, the research team believes that essential and other bridges should be designed using the same design earthquake, but with different performance requirements. Since the proposed guidelines for downstate New York will be used to modify the corresponding items in AASHTO Seismic Guide Specifications (2009) but the structural design requirements will be the same, it is rational to use design earthquake of 1000-yr return period. However, design spectra shouldn't be based on AASHTO Seismic Guide Specifications (2009). Rather, design spectra should sufficiently take into account the seismic sources and ground characteristics in the region and also be consistent with the lower one of the two-level design earthquakes (i.e., 1000-yr return period and 2500-yr return period) for critical bridges.

Design Earthquake of 1000-yr Return Period for Downstate New York

Seismic hazard for bridge design in New York City area should consider the seismic sources and ground characteristics in the region, which is reflected in the Very-Hard-Rock (VHR) motions adopted by NYCDOT in 2004 for site-specific analysis. The research team believes that earthquake motions of 1000-yr return period and 2500-yr return periods based on the VHR seismic hazard should be used to design critical bridges. The team also believe that these particular characteristics of seismic hazard should be taken into account in the design of essential and other bridges using response spectra of 1000-yr return period.

The response spectra for 1000-yr return period in AASHTO Seismic Guide Specifications (2009) have been developed without properly taking into account rock characteristics in the eastern USA, as pointed out in the Risk Engineering Report (2002) and the Guidelines Report Commentary (Weidlinger 2008). As compared in the Guidelines Report Commentary (Weidlinger 2008), at short period, the NEHRP response spectra are much lower than those of NYCDOT.

Another reason why AASHTO 1000-yr motions cannot be directly applied to design essential and other bridges in downstate New York comes from the different definitions of site classes in AASHTO Seismic Guide Specifications (2009) and in the Guidelines Report. The Guidelines Report adopted a three-parameter site classification, based on which, suits of ground motions were proposed for the design of critical bridges. The research team agrees with this site classification method and the associated methodologies to develop the ground motions for critical bridges. Hence, seismic motions of 1000-yr return period, which are consistent with the classification method and the motions for critical bridges, should be used to design essential and other bridges.

Unfortunately, response spectra for 1000-yr return period for different site conditions have not been developed either by Risk Engineering (2002) or Weidlinger (2008). Based on the communications with Risk Engineering, the CUNY research team has developed 1000 Yr design spectra through interpolation of 500-Yr, 1500-Yr and 2500-Yr return period design spectra.

For comparison purpose, a suit of horizontal response spectra were interpolated from the 500-yr, 1500-yr and 2500-yr spectra in the Guidelines Report. They are labeled as "Interpolated 1000-yr spectra" in Figs. 1.1 and 1.2 for site classes D and E. These figures show comparison of these interpolated spectra with those from AASHTO Seismic Guide Specifications. It can be seen that the interpolated curves are generally higher than those of AASHTO curves at short period. The difference is still quite significant for the period range of 0.5 sec to 1.0 sec, which is relevant to many short span bridges. In addition, set of interpolated curves for soil over deep rock of any type are higher than those of AASHTO curves even in the higher period range.

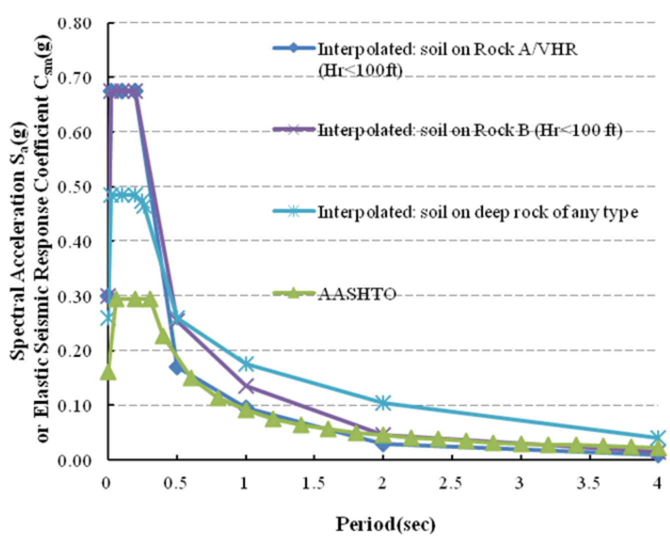


Fig. 1.1 Comparison of horizontal response spectra of 1000-yr return period: site class D

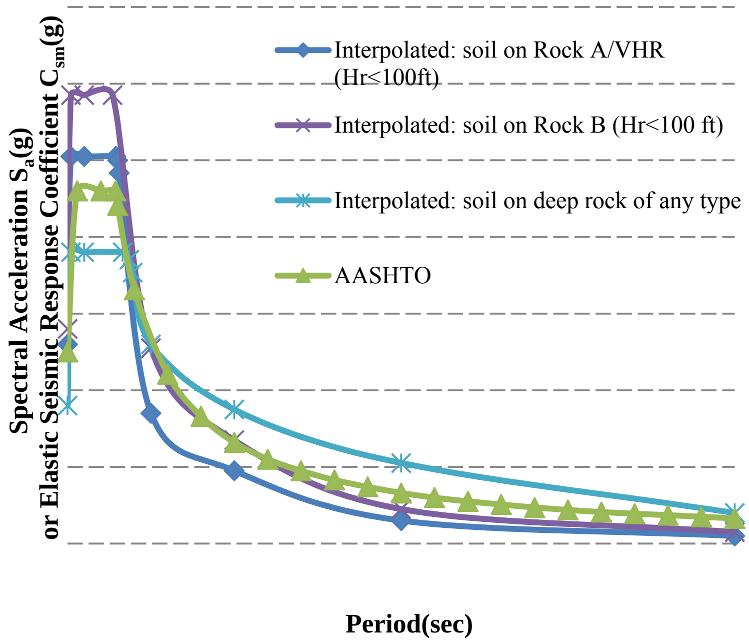


Fig. 1.2 Comparison of horizontal response spectra of 1000-yr return period: site class E.

In summary, 1000-yr response spectra developed by the CUNY research team should be should be used for the design of essential and other bridges in the region. The AASHTO 1000-yr response spectra shouldn't be used for design in the NYS downstate region.

Requirement of Sit Specific Study for Critical Bridges

The Guidelines allow the owner of a facility to waive the requirement for site specific study of critical bridges. The research team believes that this allowance should be removed (i.e., owner shouldn't be allowed to waive), so that higher confidence on the seismic hazard based on site-specific analysis can be identified, as it is needed for the design of critical bridges.

Summary

The research team suggests classifying the NYSDOT bridges in the region as Critical, Essential and Other Bridges. They agrees that critical bridges shall be design against two levels of earthquake, 1000-yr return period and 2500-yr return period, respectively, with performance levels of minimal damage and reparable damage, respectively. Essential and other bridges should be designed for seismic motions of 1000-yr return period. However, 1000-yr return period design spectra based on AASHTO Seismic Guide Specifications (2009) cannot be used directly. Rather the 1000 Yr design spectra developed through interpolation based on 500-Yr, 1500-Yr and 2500-Yr return period design spectra should be used. NYSDOT has agreed to adopt these two suggestions.

The research team thinks that owners should not be allowed to waive the requirement for site specific study of critical bridges. NYSDOT has accepted this recommendation.

1.3 Classification of a Site as a Rock or Soil Site

Depth of bedrock

In the proposed New York City Department of Transportation Guidelines Report, rock sites and soil sites are differentiated according to the depth of soil H_r, which is the distance between the ground surface and the surface of bedrock. If H_r is smaller than 15 ft, the site is classified as a rock site; otherwise, the site is classified as a soil site. The classification is based on an assumed minimum burial depth of 5 ft for a footing or pile cap. In the AASHTO Guide Specifications for LRFD Seismic Bridge Design, rock sites are defined somewhat differently, as having 10 ft or less of soil between the bedrock surface and bottom of the spread footing or pile cap. The 10 ft in the AASHTO Specifications (2009) and the 5 ft minimum burial depth yields the H_r = 15 ft in the proposed NYCDOT Guidelines.

While we recognize that these depths are somewhat arbitrary, our recommendation is to not introduce a new arbitrary depth that deviates from AASHTO specifications. The thickness of soil over bedrock affects ground motions, and the magnitude of change and its frequency-dependence varies continuously with depth. Either the proposed criterion (Hr = 15 ft) or the original AASHTO criterion (10 ft of soil between the bottom of footing and surface of bedrock) will result in abrupt change of deign ground motions, and the proposed criterion in the Guidelines Report does not improve the accuracy of design ground motions relative to the established AASHTO criterion.

Figs. 1.3 and 1.4 compare the response design spectra on rock and soil sites for two earthquake return periods: 1500 year and 2500 year as proposed in the NYCDOT Guidelines Report. The differences are substantial, as can be expected, and the spectral accelerations at all periods on soil sites (Hr < 100 ft) are much higher than those at the rock sites due to soil amplification. The same trend is also observed for the 500-year earthquake.

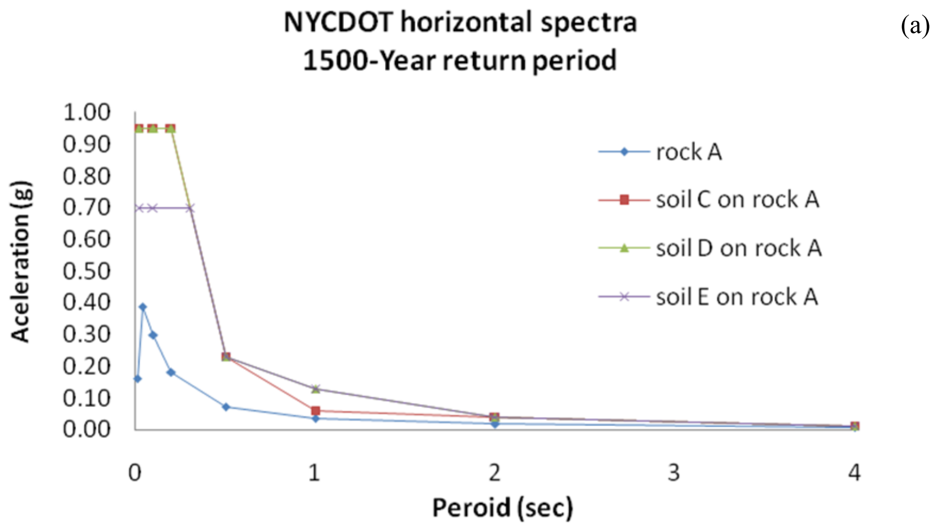
Since the depth of footing can be larger than 5 ft in many cases, the proposed criterion may even impose unnecessary seismic demand on a bridge if its footing is close to the bedrock (smaller than 10 ft) but Hr > 15ft, as can be seen from Figs. 1.3 and 1. 4.

Summary

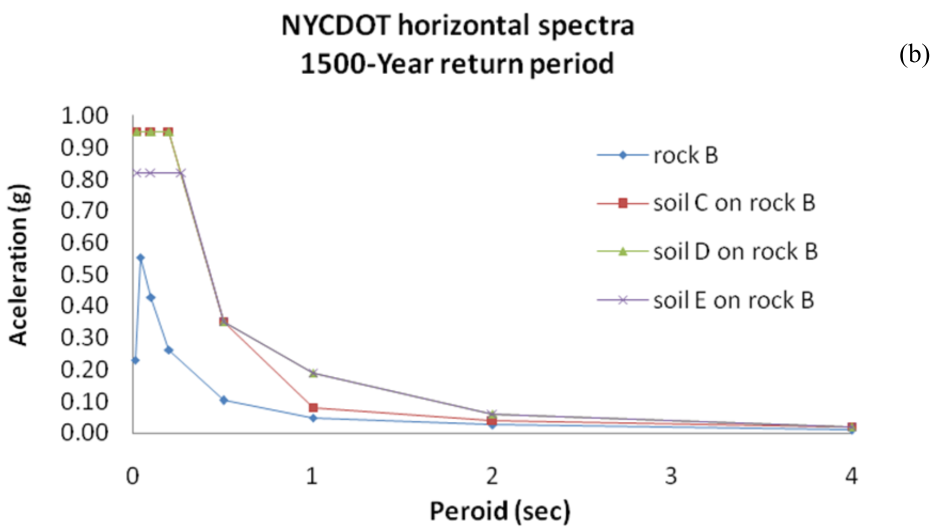
Based on observations presented above, it is recommended that the new guideline adopt the following criterion specified in the AASHTO Specification:

“A site is classified as a soil site if there is more than 10 ft of soil between the bottom of spread footing or pile cap and the surface of bedrock; otherwise it is classified as a rock site”.

This suggestion has been accepted by NYSDOT.



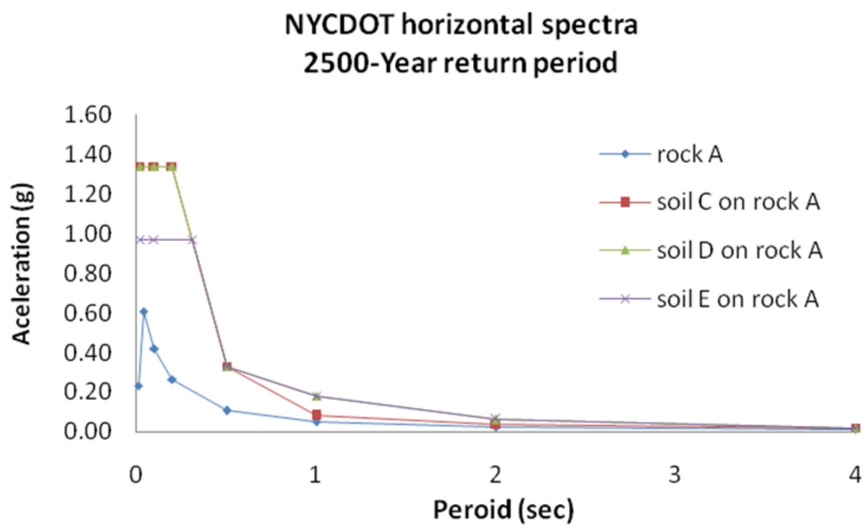
(a)



(b)

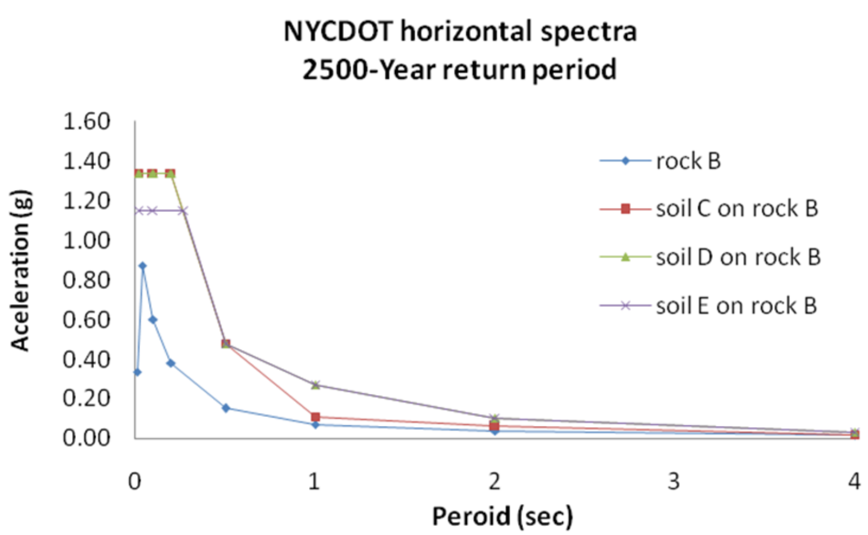
Fig. 1.3 Comparison of response design spectra of rock and soil sites for the 1500-year earthquake: (a): rock class A and soil on rock class A; (b) rock class B and soil on rock class B.

(a)



(a)

(b)



(b)

Fig. 1.4 Comparison of response design spectra of rock and soil sites for the 2500-year earthquake: (a): rock class A and soil on rock class A; (b) rock class B and soil on rock class B.

1.4 Rock Classes and Rock Generic Horizontal Design Spectra

In the proposed New York City Department of Transportation Guidelines Report, Section 5 addresses the definitions of rock classes and rock generic horizontal design spectra. Outcrop rock and bedrock in the region are classified into categories, based on the average shear wave velocity in the upper 20 feet of the rock profile ($\overline{V}_{s20}$): Very Hard Rock (VHR, $\overline{V}_{s20}$>9000 ft/sec), Rock Class A (5000 ft/sec <$\overline{V}_{s20}$≤9000 ft/sec), and Rock Class B (2500 ft/sec <$\overline{V}_{s20}$≤5000 ft/sec). The horizontal design spectra of Rock Class A and Rock Class B are directly obtained from the VHR spectra by applying constant ratios, 1.15 and 1.65 respectively for Rock Class A and Rock Class B, to spectral accelerations at all periods.

The concerns of the research team have been related to constant ratios representing the site effects and the level of documentation for the probabilistic seismic hazard results used to develop the spectra in Figs. 6-8 of the Guidelines Report.

<u>Existence of Deep Softer Rock Profiles in the Region</u>

According to the USGS Geological map of New York City, the rock under the soil in most of the five boroughs is metamorphic, but in some locations, sedimentary rock is found. Sedimentary rock can be much softer than metamorphic rock, which provides the possibility of the existence of thick soft rock over VHR.

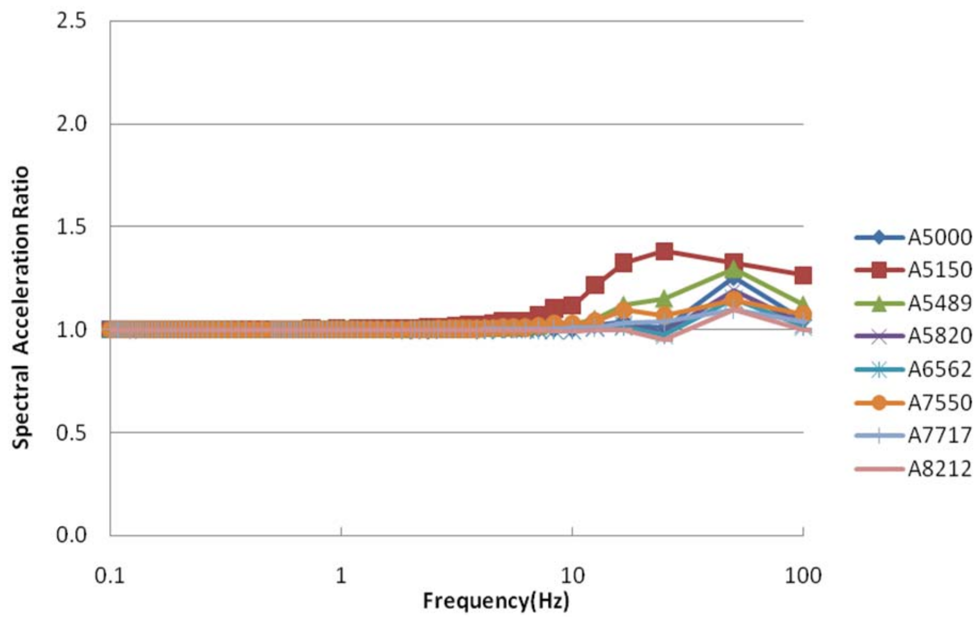
The Commentary of the Guidelines Report acknowledges the possibility of very thick soft rock (> 100 ft) in the region. However, there is a paucity of data showing its depth or stiffness profile. Parson Brinckerhoff (PB), one member of the research team, found from its consulting experience that the depth of Class B rock in the sandstone profile below the Hudson River is less than 100 ft. They conclude that the possibility of very thick soft rock in the NYC area is small.

<u>Effects of Soft Rock Site Effects on Horizontal Design Spectra</u>

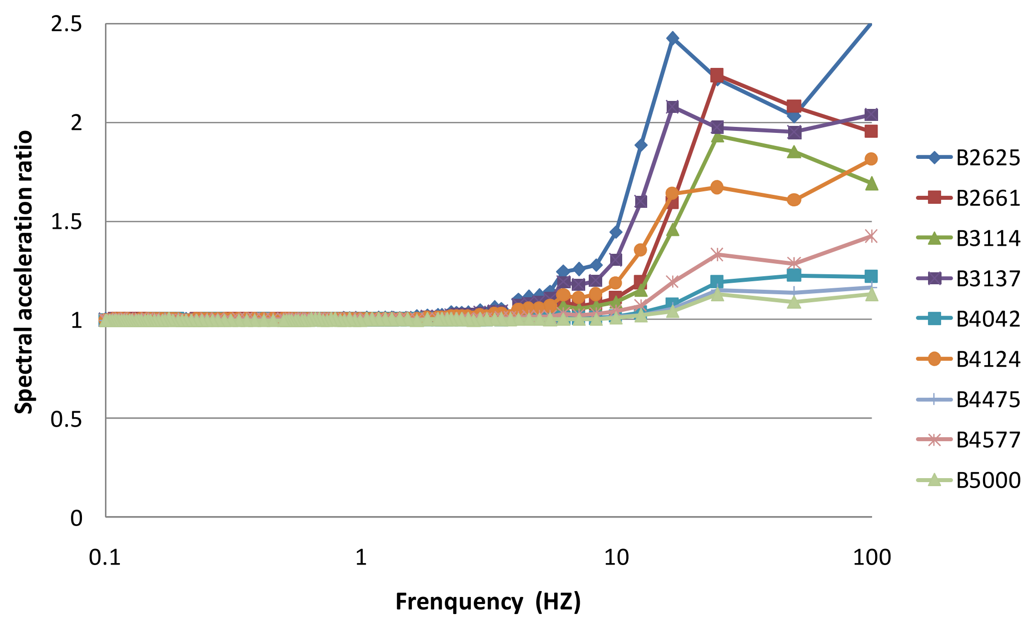
The average shear wave velocity of eastern US crust is reported as 9300 ft/sec (Risk Engineering, 2002), which falls within the VHR category. However, softer rock ($\overline{V}_{s20}$ < 9000 ft/sec) generally occurs above VHR, introducing a site effect on ground motion relative to the VHR reference condition. The level of site amplification and its frequency-dependence depend on the thickness of the soft rock layer and its stiffness profile.

In the Guidelines Report, 1D ground response analyses using Random Vibration Theory were conducted for profiles of Rock Classes A and B with thicknesses ranging from 20 ft to 100 ft on top of VHR. As expected, the results showed that spectral accelerations are amplified only at high frequencies for small soft rock thickness (i.e., 20-100 ft); site amplification relative to VHR was essentially unity for low frequencies. The research team conducted additional 1D ground response analyses by the Proshake software, using the mean profiles provided in the Commentary of the Guidelines Report and the VHR input ground motions provided by Risk Engineering (2002). The standard modulus reduction and damping curves for rock included with Proshake (Schnabel et al. 1972) were used in the analysis. As shown in Fig. 1.5, similar conclusions were obtained, with site amplification for the 2500 year earthquake occurring principally at frequencies greater than approximately 7 Hz. The profile name in Fig. 1.5 represents the $\overline{V}_{s20}$ value for the profile. The research team also investigated the effect of the input ground motion amplitude on the site amplification by repeating the analyses for the lower 500 year and

1500 year earthquake return periods. As shown in Fig. 1.6, those weaker input motions do not change the amplification characteristics significantly. Results of these analyses as well as those from the Commentary of the Guidelines Report indicate site amplifications at high-frequencies of 2.0 or more for Site Class B and up to 1.5 for Site Class A.

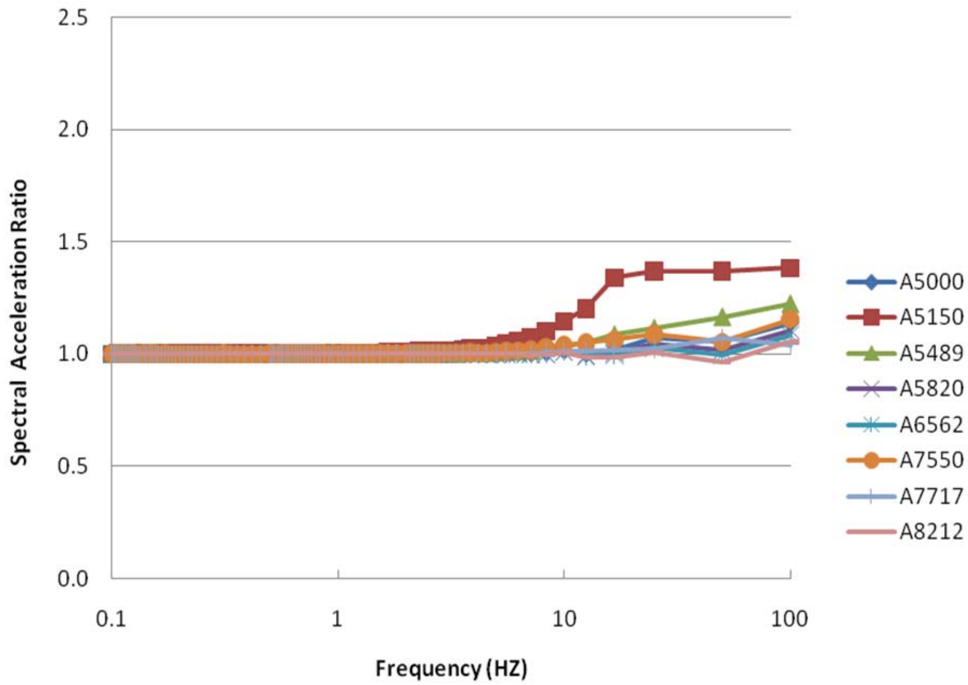


(a)

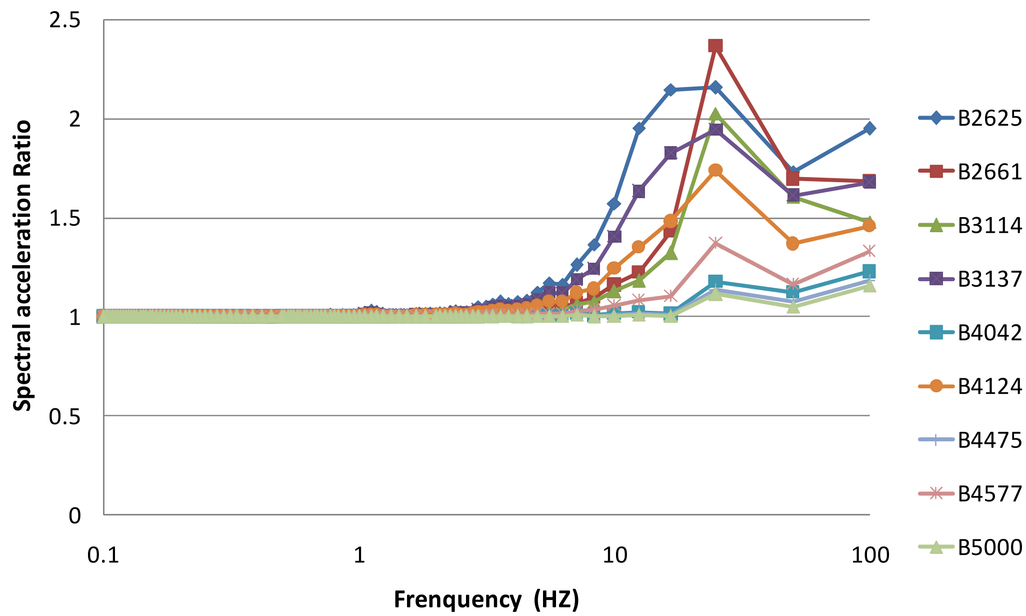


(b)

Fig. 1.5 Spectral ratio as a function of frequency for the 2500-year earthquake: (a) rock class A / VHR; (b) rock class B / VHR



(a)



(b)

Fig. 1.6 Spectral ratio as a function of frequency for the 1500-year earthquake: (a) rock class A / VHR; (b) rock class B / VHR

The trend of site factor versus frequency is reversed for thick soft rock profiles above VHR. Since no measured data is available, the existence of any thick soft-rock profile can only be assumed. The Commentary describes analysis of an assumed 3000 ft thick soft rock profile that falls within Rock Class B. The results show site amplifications up to 1.8 at low frequencies (< 1.0 Hz) and values spread around unity at high frequencies (> 10 Hz). The research team conducted Proshake analyses on the same profile described in the

Commentary as well as several other assumed Rock Class B profiles shown in Fig. 1.7. As shown in Fig. 1.8, the results are consistent with those in the Commentary, but also show the dependence of site amplification on the thickness of soft rock and other details of the stiffness profile.

Similar analyses were conducted for rock profiles of various thicknesses above VHR falling within Rock Class A. Fig. 1.9 shows the assumed profiles. Fig. 1.10 shows trends of site amplification with frequency similar to those for Rock Class B, but with smaller amplification levels.

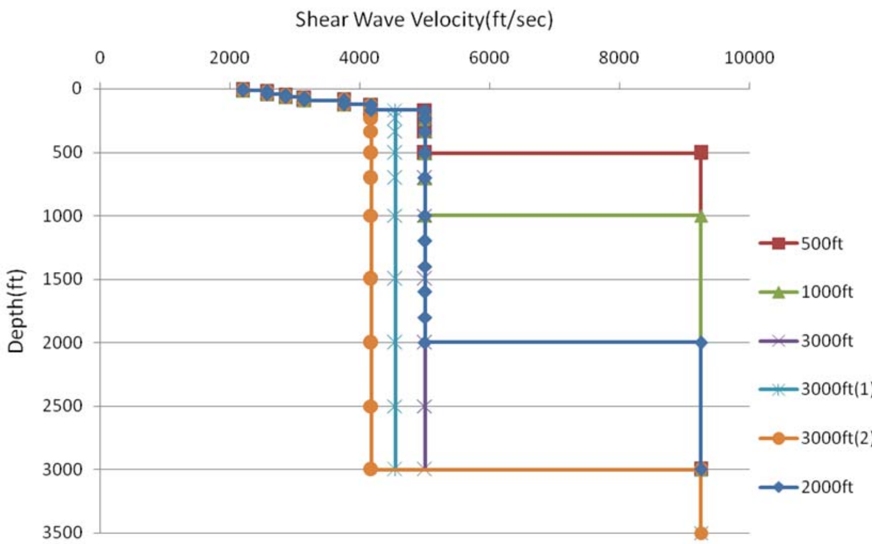


Fig. 1.7 Deep rock class B profiles analyzed

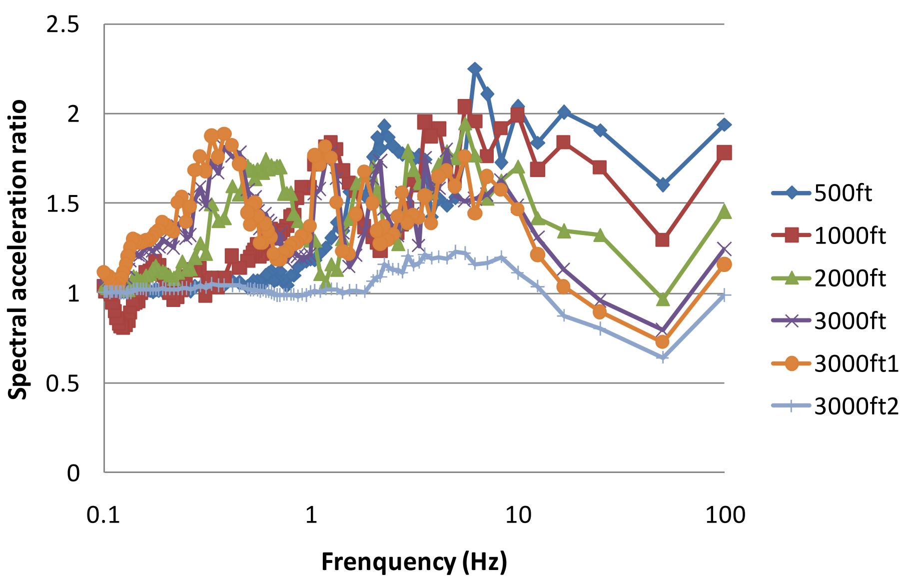


Fig. 1.8 Spectral acceleration ratio for the rock class B profiles anaylzed (2500-year earthquake)

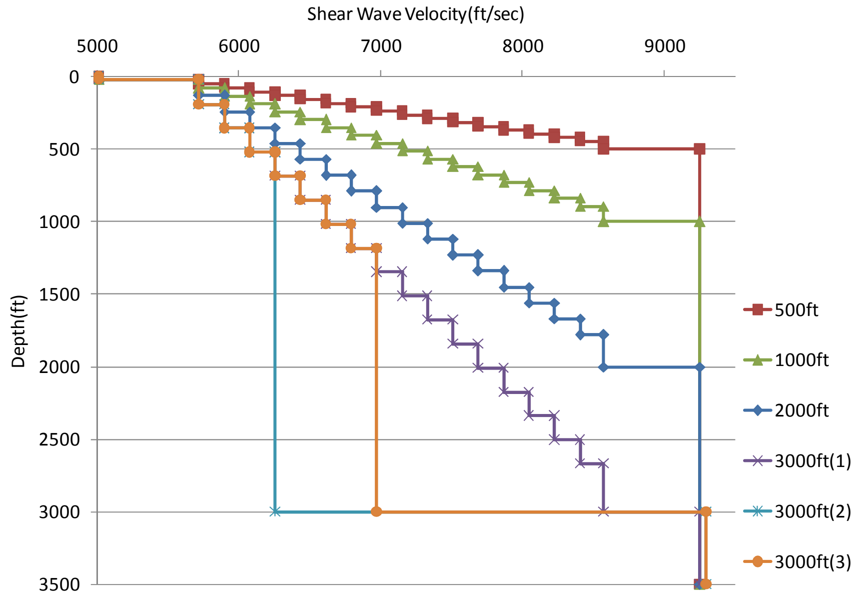


Fig. 1.9 Deep rock class A profiles analyzed

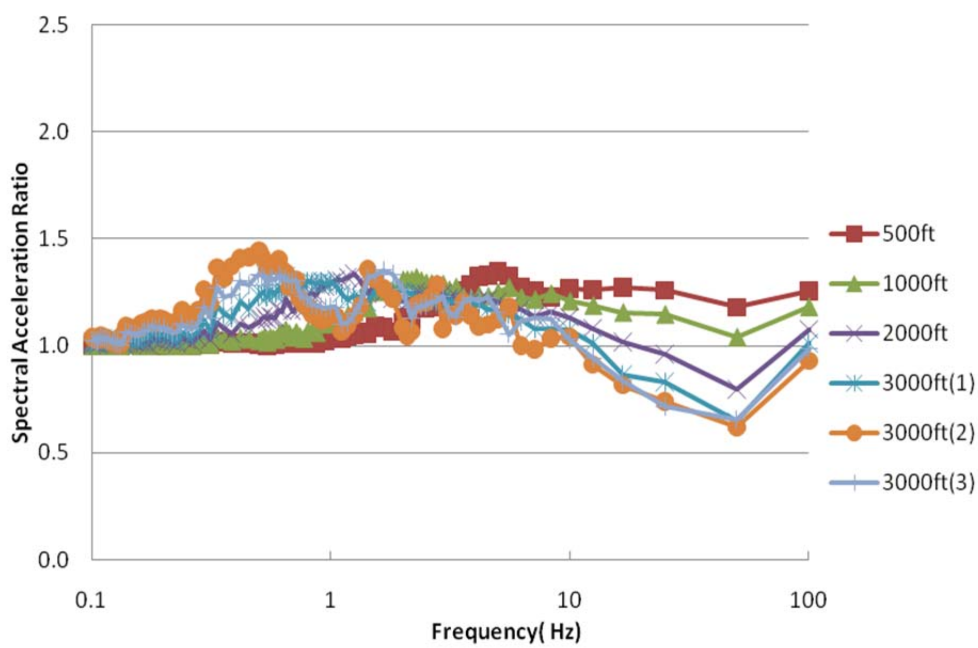


Fig. 1.10 Spectral acceleration ratios for the rock class A profiles analyzed (2500-year earthquake)

<u>Unification of Rock Class A/VHR under soil when Hr < 100ft</u>

The Guidelines Report unifies the horizontal spectra for Rock Class A/VHR under soil when Hr < 100 ft. The research team conducted a series of ground response analysis and found that the unification was reasonable. When shear wave velocity of bedrock was close to 5000 ft/sec, which is the lower bound of Rock Class A, the horizontal ground spectra were not significantly different from that when the shear wave velocity is 9000 ft/sec.

Generic Horizontal Design Spectra for Rock Class A and Rock Class B

Since profile data for deep soft rock over VHR is not available at present, it seems appropriate to adopt a conservative approach. Since thin soft rock tends to amplify motions at high frequencies, thick soft rock tends to amplify motions at low frequencies, and the amplification levels for the thick and thin profiles at their respective frequencies are not very different, it seems appropriate to envelope those responses with a single amplification ratio that applies across all frequencies, as suggested in the Guidelines Report.

The use of a single amplification ratio is also adopted by the AASHTO Specifications as well as the NEHRP Provisions. In both of those standards, the reference rock condition is V_{s30m}=760 m/s (which is considerably softer than the VHR condition of 9300 ft/s = 2800 m/s). For Rock Classes A and B in those standards (which are defined as 30 m velocities, in contrast to the 20 ft velocities in the NYCDOT document), the site factors are taken as 1.0 for Class B and 0.8 for Class A both for low and high frequencies. Those factors cannot be readily compared to the site factors in the NYCDOT document, because of the different reference conditions (VHR for NYCDOT; V_{s30m}=760 m/s for NEHRP).

Our concern is that the proposed site factor of 1.65 for Rock Class B was based on an analysis suite that included only one profile of very thick soft rock (3000 ft), whereas a relatively large number of thin soft rock profiles were considered. As can be seen from the results in Fig. 1.8, site factors for thick soft rock conditions exhibit significant variability, and an amplification factor determined largely by an assumption of deep soft rock as thick as 3000 ft may be too conservative. We suggest that the site factors be developed based on a profile suite that considers a broader range of profile depths, with the results averaged according to weights assigned to the various possible profiles. If the envelope of those analysis results indicates different amplification levels across the frequency band, the selection of a single factor should be weighted towards low frequencies, which are of the greatest interest for bridges.

The same suggestion applies to site factors for Rock Class A. No thick Class A profile was analyzed by Weidlinger and Consultants while developing the proposed ratio of 1.15. According to the preliminary result in Fig. 1.10, site factors could be considerably larger, even at the low frequencies of interest to bridges. Another observation is the difference between Rock Class A motion and Rock Class B motion. The ratio is 1.15/1.65 = 0.7 according to the Guidelines Report, while both AASHTO and NEHRP adopt a ratio of 0.8. Resolution of these issues is importation to ensure adequate safety as well as economy during the seismic design/retrofit of bridges in the downstate region.

Concern Related to PSHA Documentation

In the presentation of the VHR hazard analysis, a list of source models is given but very little is said about them. This is important insofar as the spatial invariance of the hazard. Is nothing known about source locations in NY so that they must be assumed randomly located? Also, more documentation is needed on the epistemic uncertainty in the source models -- is it only represented by model-to-model variations or do the models themselves have variations in key parameters (such as b value or M_{max})?

Another issue is the component of ground motion considered in the analysis. Are hazard results given as the maximum horizontal motion or the geometric mean of two horizontal components?

Summary

Use of a single ratio to scale the VHR motion for either Rock Class A or Rock Class B may be justified, but the adopted values should be supported by additional analysis. Particularly, how to determine various possible profiles and their associated weights requires in-depth investigations. However, NYSDOT has decided not to carry out further analysis to resolve this issue. Regarding the concerns on PSHA documentation, they have been sufficiently addressed by the Weidlinger's team in its response to the comment.

1.5 Soil Site Characterization and Soil Generic Horizontal Design Spectra

Section 6 of the 2007 NYCDOT Seismic Design Guidelines Report addresses soil site characterization and soil generic horizontal design spectra. Soil site characteristics are described by Site Classes (C, D, E and F) on the basis of average soil properties and generic horizontal design spectra are proposed for Site Class C, D and E based on probabilistic seismic hazard analysis (PSHA) for reference rock combined with the results of one-dimensional site response analyses.

The concerns of the research team relate to details in the site classification procedures (Sections 6.1-6.3), dynamic soil properties used in the site response analyses (Section 6.4), and issues related to the PSHA and the procedures used for merging the PSHA results with the site response analysis results (Section 6.4)

Criteria for Site Classification

The criterion of soil class characterization is modified from the criterion adopted in NEHRP (2010). The research team has the following comments on this criterion:

1) The research team believes that the use of shear wave velocity Vs over SPT blowcount N or undrained shear strength S_u should be more strongly encouraged. The generic horizontal design spectra were developed based on shear wave velocity, which is defined at very low strain level, while N and S_u are both soil properties at large strain. Recent efforts (e.g., Brandenberg et al. 2010) to correlate these two parameters with shear wave velocity showed that the relations have high dispersion. Hence, the site characterization using N or S_u introduces significant additional uncertainty to the ground motion analysis. The research team believes that the use of Vs for site characterization should be required at least for critical bridges.

2) The symbol of $\overline{V}_{s100}$ representing average shear wave velocity is confusing, since it is not necessarily the average Vs down to 100 ft, depending on the depth of bedrock according to section 6.2 of the Guidelines Report.

Definitions of Soil Class Parameters

The definitions of soil class parameters in the Guidelines Report are also modified from NEHRP (2003). The average soil properties consider only the soil layers if Hr < 100 ft, while NEHRP (2003) includes all the materials down to a depth of 100 ft, regardless of whether those materials are soil or rock. The NEHRP site factors are based on data from California, where the site profiles typically do not show a strong impedance contrast at the rock/soil contact. For applications in NYC, where the rock is very stiff, the revised definitions in the Guidelines Report appear to be reasonable.

The research team has a minor concern regarding the exploration depth for sites having very thick soft/medium stiff clays (H > 120 ft). We suggest that the Guidelines be clearer on the depth of subsurface exploration where such conditions may exist. If a continuous soft/medium clay layer (s_u<1000 psf, or Vs < 600 ft/sec) extends below 100 ft depth, the subsurface exploration should continue until the base of this layer is encountered or the thickness of clay layer is already 120 ft.

Steps for Classifying Soil Sites as Class C, D, E or F

The Guidelines Report is not clear on the site classification using $\overline{N}$, $\overline{N}_{ch}$ or $\overline{S}_u$. For example, if a site has alternate cohesionless and clayey soil layers, should all the three average properties be obtained and used in the site classification? Is it mandatory or optional?

The research team believes that the Guidelines should be unambiguous on the classification steps. It recommends that Vs be used instead of N or Su to improve the accuracy of site classification. We recommend that the use of Vs be required for the design of critical bridges.

Ground Motion models

In the discussion of PSHA for VHR in Section 3 of the Commentary (Pages 8-9), a series of ground motion models that were used are listed. In Section 6 of the Commentary, reference is made on p 31 and 33 to omega-squared source model and CEUS parameters (repeated on p 33 of the commentary). So, there is a contradiction in the ground motion models used in the VHR PSHA that needs to be corrected.

Base case profiles

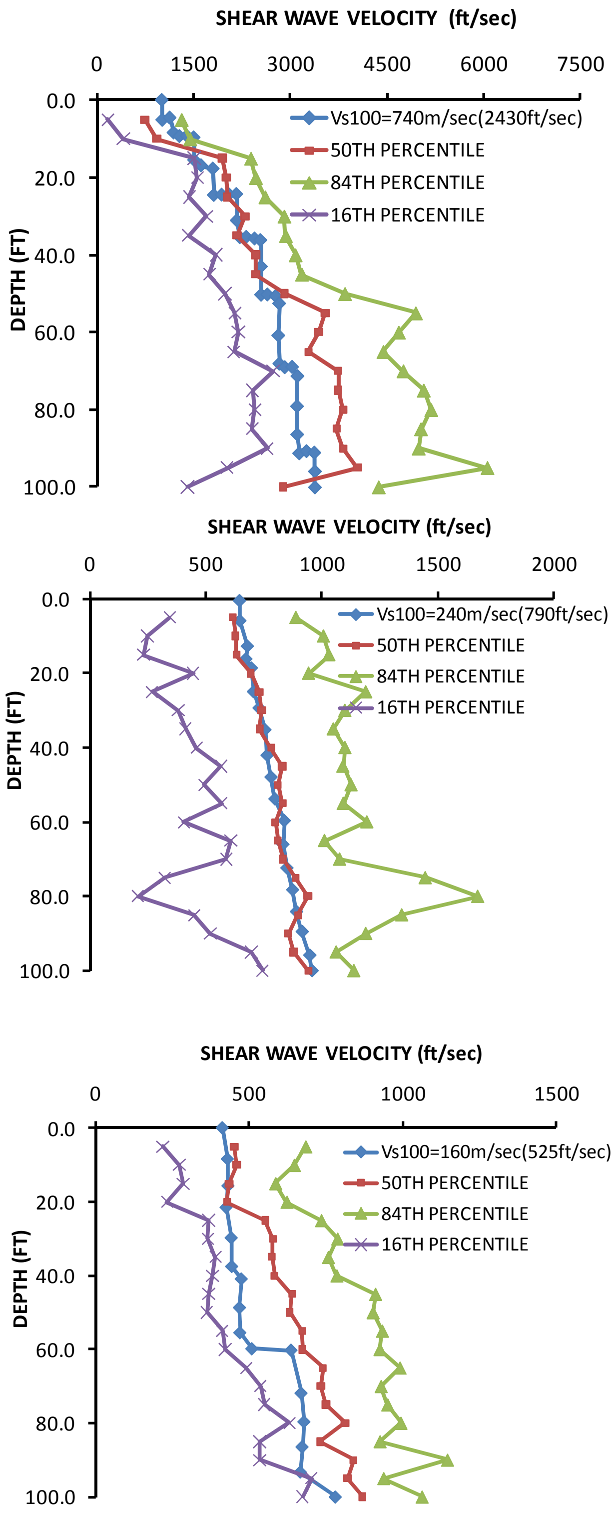
The base case profiles for soils used in the RVT analyses have been adjusted from measured velocities in predominately tectonically active regions (i.e. from profiles in LA and SF areas in California). The adjustment is based on a limited number of NYC profiles (2 Class-C sites, 31 Class-D sites, and 9 Class-E sites). In order to verify the Vs profiles described in the Commentary (Page 28-29), the research team compiled Vs data from a series of geotechnical reports from NYSDOT, NYCDOT and Parsons Brinckerhoff.

Due to the fact that shear wave measurement for bridge projects started only in recent years in the region, only a few boreholes with Vs can be found in these reports. The research team collected 71 relevant boreholes, of which 7 are Class C sites, 36 are Class D sites, 21 are Class E sites, and 7 are Class F sites. The depth of boreholes ranges from 30 ft to more than 600 ft. Most of the shear wave velocities were measured by cross-hole methods, with few by down-hole, seismic cone penetration or SASW methods. The results from different methods were given identical weight in the data analysis.

Fig. 1.11 compares the median Vs profiles from these boreholes with the original base case profiles, which are labeled in the figures as "Vs100=…". Only data from the top 100 ft was compared. Also shown in Fig. 1.11 are profiles of 16- and 84-percentile velocities. It can be seen that the base-case profiles from the Guidelines Report Commentary are similar to the medians from our analysis in Site Classes C and D. The differences are larger for site class E.

In summary, we believe that the assumed base-case profiles adopted in the RVT analysis are reasonable.

Thickness of soils over bedrock in the region



(a) (b) (c)

Fig. 1.11 Comparison of soil profiles: (a) site class C (7 profiles); (b) soil class D (34 profiles); (c) soil class E (24 profiles)

While developing the base-case profiles, soil deposits ranging from 25 ft to 2000 ft were assumed by Weidlinger and Consultants. The research team investigated whether this assumed range is consistent with regional geologic conditions using local surface geological maps (e.g., Baskerville 1992, 1994) and reports related to soil thickness over bedrock (e.g., Collins et al. 2002; USGS 2010).

It was found from these sources that the thickness of surficial deposits varies considerably in the region. In downstate New York, which includes New York City, Nassau County, Westchester County and Rockland County, the bedrock is mainly overlain by surficial deposits left by continental glacier ice sheets. The most abundant surficial material is glacial till, which consists of a mixture of clay, silt, sand, gravel and boulders and its thickness ranges from zero at bedrock outcrops to 2000 ft.

Most of Queens and Brooklyn have very thick surficial deposits of unconsolidated material overlying bedrock. These deposits range in composition from clay to gravel and in thickness from null to more than 1000 feet. However, very deep soil deposits up to 2000 ft thick was found in Nassau County. Overlying the bedrock is a wedge of unconsolidated gravel, sand, silt and clay deposits. The thickness of the wedge of these deposits increases from null where bedrock outcrops along the north shore in Queens, up to about 2,000 ft along the south shore barrier.

The information obtained from available sources is not comprehensive; the research team was not able to find general surficial geological information of Westchester County. However, based on what was collected, it seems that the assumption of soil thickness up to 2000 ft in the Guidelines Report Commentary is reasonable.

<u>Plasticity Index (PI) of clays</u>

In the RVT analyses by Weidlinger and Consultants, clay layers were assumed to have a Plasticity Index (PI) of 40. Although the research team does not have adequate first hand data to draw a definite conclusion, it believes that it is more appropriate to consider a range of PI to cover the complicated soil condition in the region. In fact, according to the geotechnical reports the research team obtained from NYSDOT and NYCDOT, the majority of investigated clay layers have PI<40. On the other hand, based on the consulting experience of Parsons Brinckerhoff, PI > 40 is typical for the thick organic silt and clay deposits in the Flushing Meadow area in Queens (including the LGA airport area), and underneath the Hudson River (where an average PI of 60 is frequently encountered).

<u>Allowed lack of site-specific data for Critical Bridges</u>

The Guidelines allow the owner a facility to waive the requirement for site specific study and assume site class D for the purpose of ground motion analysis. This should not be allowed. The owner should not be allowed to waive the need for site-specific study, and site classification should always be based on site-specific data, especially for critical bridges.

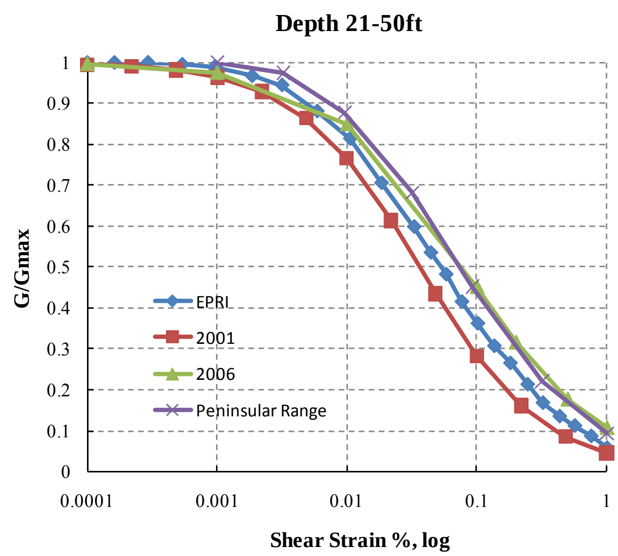
<u>Modulus reduction and damping curves</u>

Modulus reduction and damping (MRD) curves for the RVT analyses were based on those by EPRI (1993), Peninsular Range (Silva et al. 1997) and Vucetic and Dobry (1991). In order to take into account the dependence of clay MRD curves on confining pressure, the Vucetic and Dobry (1991) curve for a PI of 50 was used for clay layers 51 ft to 200 ft deep, while that for a PI of 100 was used if the depth is greater than 200 ft.

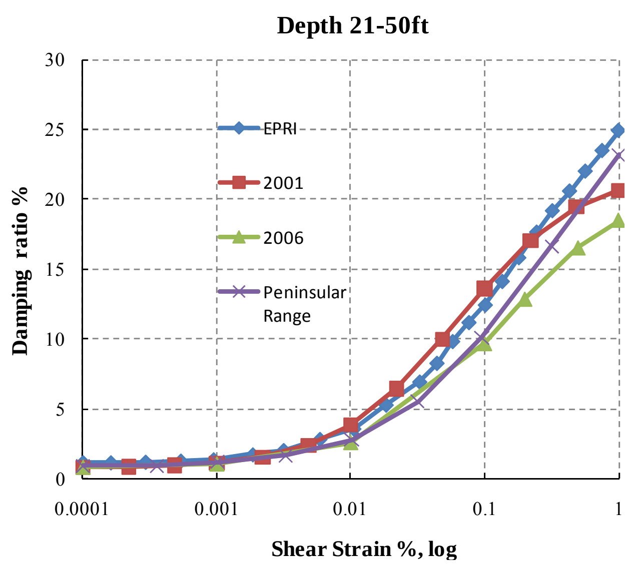
The research team has a concern that these curves may be out of date. Confining pressure has long been recognized as important for granular soils, which was considered in the EPRI (1993) and the peninsular range curves. Recent MRD models for clays also include a confining pressure effect (Darendeli, 2001; Zhang et al., 2006), but this effect is not considered in the Vucetic and Dobry (1991) curves.

In Figs. 1.12-1.14, we compare the MRD curves adopted in the RVT analyses (from EPRI, Vucectic and Dobry, and Silva et al., 1997) to those in Darendeli (2001) and Zhang et al. (2006). For the purpose of these comparisons, soil age was taken as Quaternary (required for the Zhang et al., 2006 model).

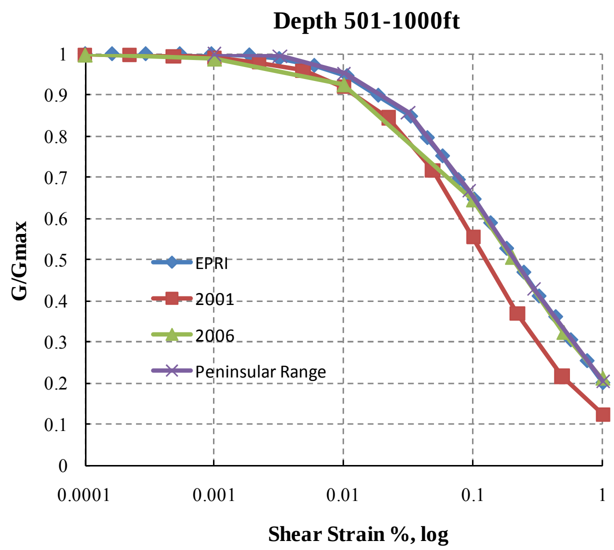
The influence of confining pressure on the Darendeli (2001) and Zhang et al. (2006) curves were converted to equivalent depths assuming a constant unit weight of 125 lb/ft^3 and a constant value of $K_0 = 0.5$. The three sets of curves are then compared as a function of depth, as shown in Fig. 1.12. Overall the four sets of curves are not significantly different.



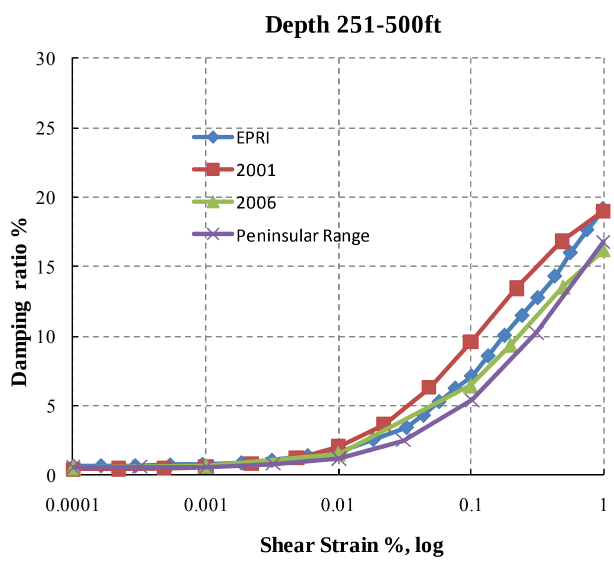
(a)



(b)

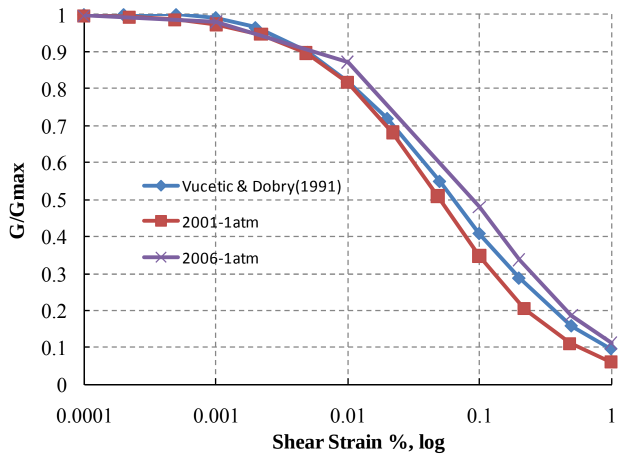


(c)

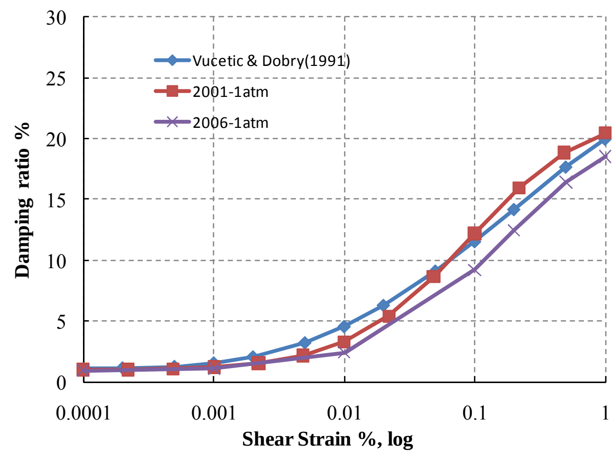


(d)

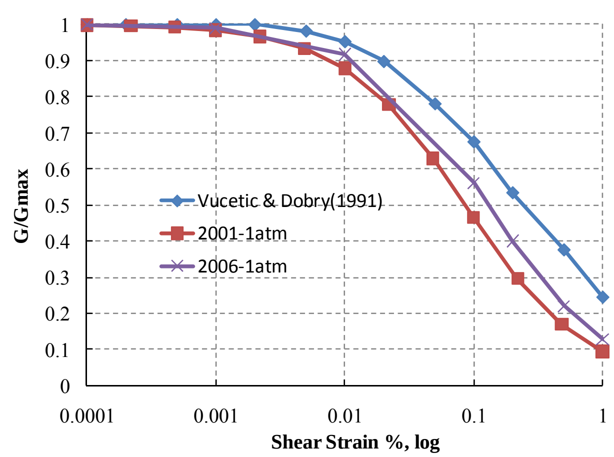
Fig. 1.12 Comparison of selected MRD curves of granular soils from EPRI (1993), peninsular range (Silva et al. 1997), Darendeli (2001) and Zhang et al. (2006): (a) MR curves: 21-50ft; (b) D curves: 21-50 ft; (c) MR curves: 251-500 ft; (d) D curves: 251-500 ft


G/Gmax
Shear Strain %, log
Vucetic & Dobry(1991)
2001-1atm
2006-1atm
0.0001
0.001
0.01
0.1
1

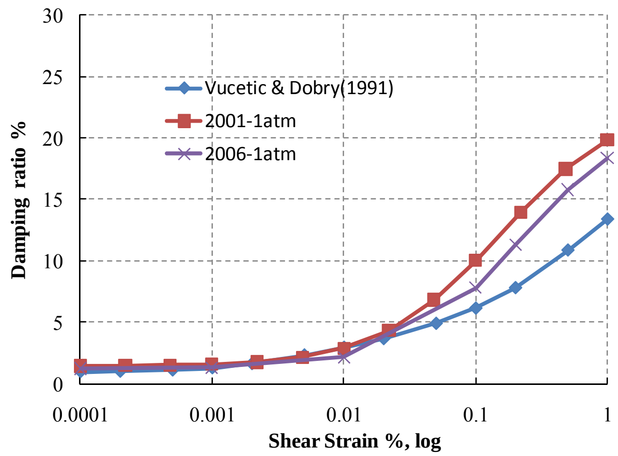
(a)


Damping ratio %
Shear Strain %, log
Vucetic & Dobry(1991)
2001-1atm
2006-1atm
0.0001
0.001
0.01
0.1
1

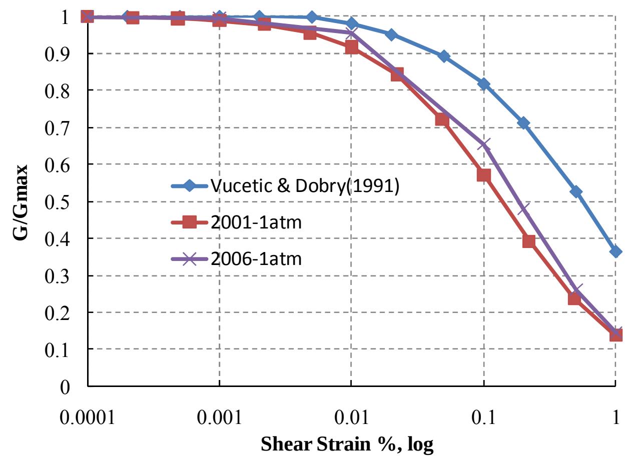
(b)


G/Gmax
Shear Strain %, log
Vucetic & Dobry(1991)
2001-1atm
2006-1atm
0.0001
0.001
0.01
0.1
1

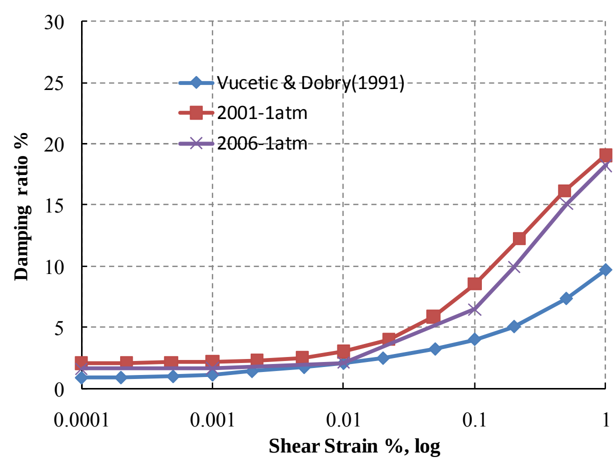
(c)



(d)

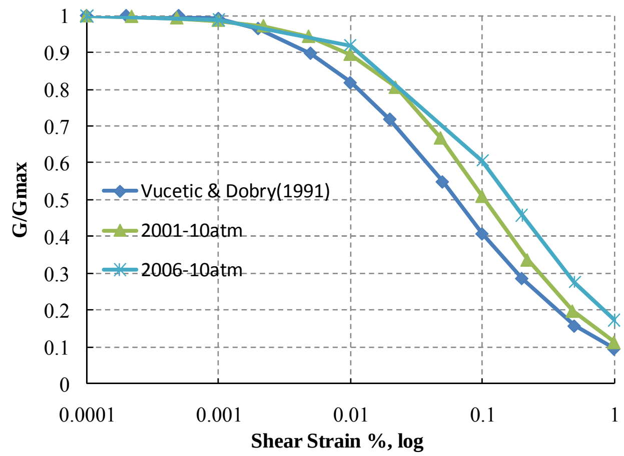


(e)

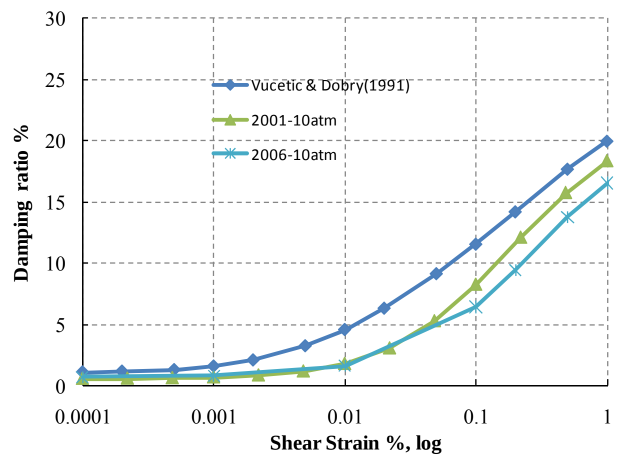


(f)

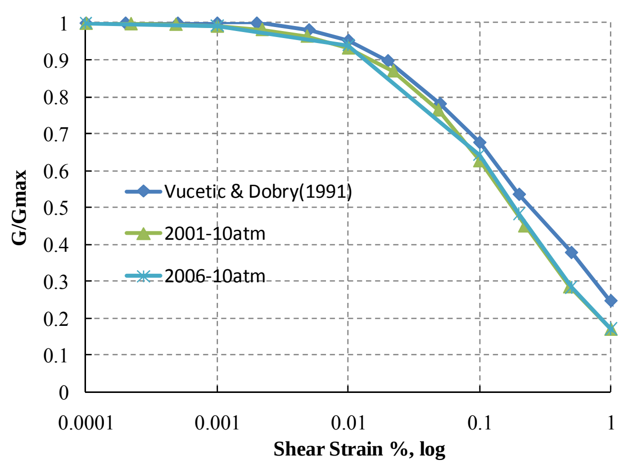
Fig. 1.13 Comparison of selected MRD curves of clays at 100 kPa from Vucetic and Dobry (1991), Darendeli (2001) and Zhang et al. (2006): (a) MR curves: PI = 15; (b) D curve: PI = 15; (c) MR curves: PI = 50; (d) D curve: PI = 50; (e) MR curves: PI = 100; (f) D curve: PI = 100


G/Gmax
Shear Strain %, log
Vucetic & Dobry(1991)
2001-10atm
2006-10atm
0.0001
0.001
0.01
0.1
1

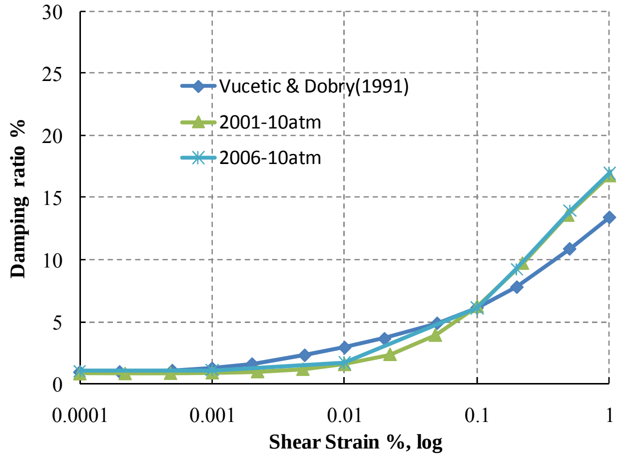
(a)


Damping ratio %
Shear Strain %, log
Vucetic & Dobry(1991)
2001-10atm
2006-10atm
0.0001
0.001
0.01
0.1
1

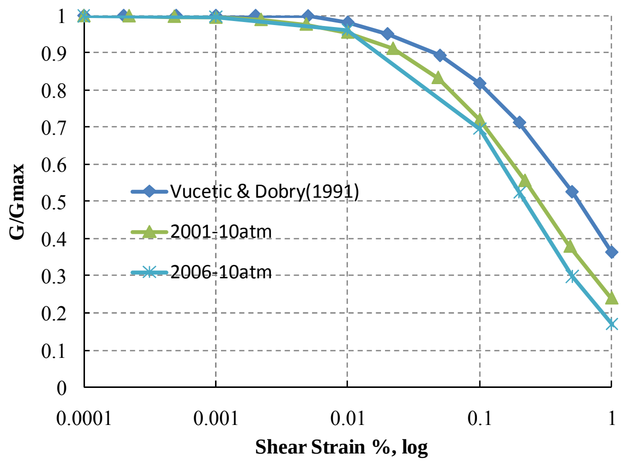
(b)


G/Gmax
Shear Strain %, log
Vucetic & Dobry(1991)
2001-10atm
2006-10atm
0.0001
0.001
0.01
0.1
1

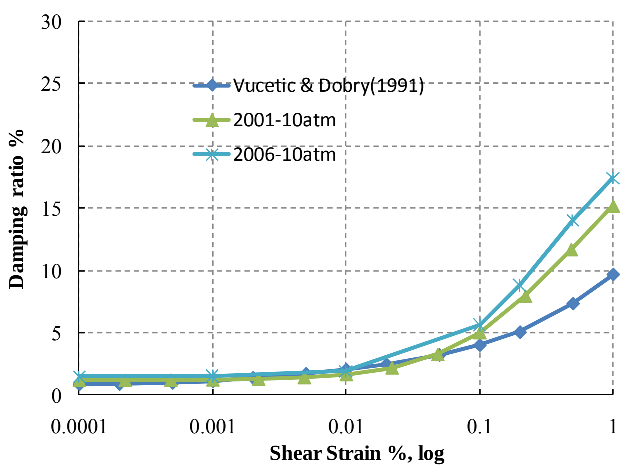
(c)



(d)



(e)



(f)

Fig. 1.14 Comparison of selected MRD curves of clays at 1000 kPa from Vucetic and Dobry (1991), Darendeli (2001) and Zhang et al. (2006): (a) MR curves: PI = 15; (b) D curve: PI = 15; (c) MR curves: PI = 50; (d) D curve: PI = 50; (e) MR curves: PI = 100; (f) D curve: PI = 100

Figs. 1.13 and 1.14 show the comparison of selected MRD curves for clays. For clay soils, the difference is larger when PI is large, but the difference decreases when high confining pressure is considered in the curves by Darendeli (2001) and Zhang et al. (2006). It is also noted that in the RVT analysis, deep clays under large confining pressure assumed the MRD curves with large PI according to Dobry and Vucetic (1991) to compensate the effect of confining pressure, which is reasonable as can be seen in Fig. 1.14.

To investigate the possible effects of these differences, the research team selected three boreholes for the collected geotechnical reports, which belong to Soil Class C, D and E, respectively. Their seismic responses were compared using different MRD curves. These three profiles consisted of both granular soil and clay layers, the depth of which to the bedrock was 90 ft (Class C), 600 ft (Class D) and 70 ft (Class E), respectively. Both Class A and Class B bedrocks were assumed in the analysis. The Class A or Class B bedrock was assumed to be 100 ft thick overlaying Very Hard Rock (VHR). The horizontal motions for VHR, which were established by Risk Engineering (2002) and adopted by NYCDOT in 2004, were input at VHR. Fig. 1.15 shows the analyzed soil profiles over bedrock.

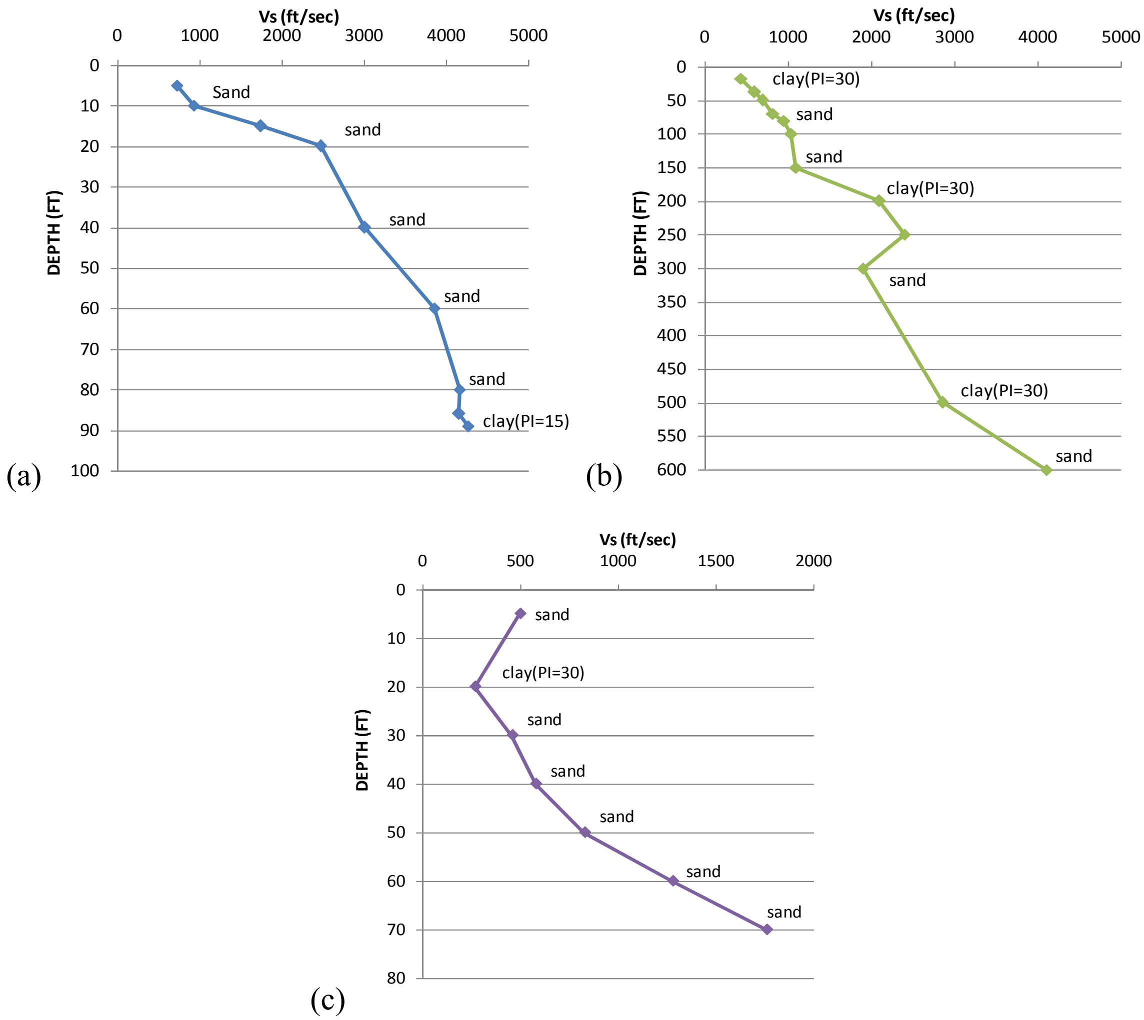
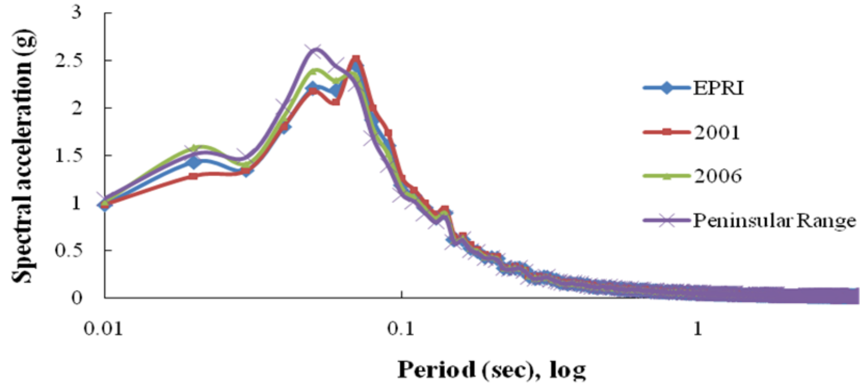
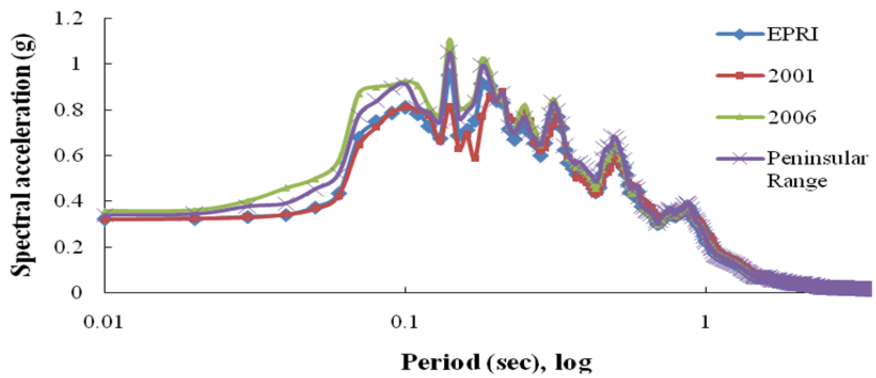


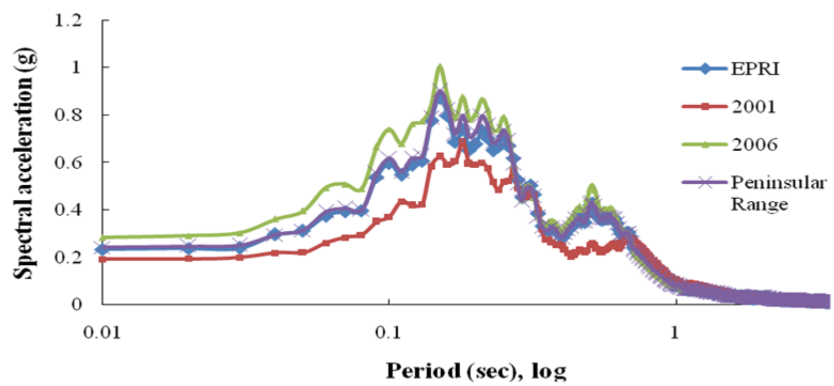
Fig. 1.15 Three soil profiles analyzed: (a) class C profile; (b) class D profile; (c) class E profile



(a)



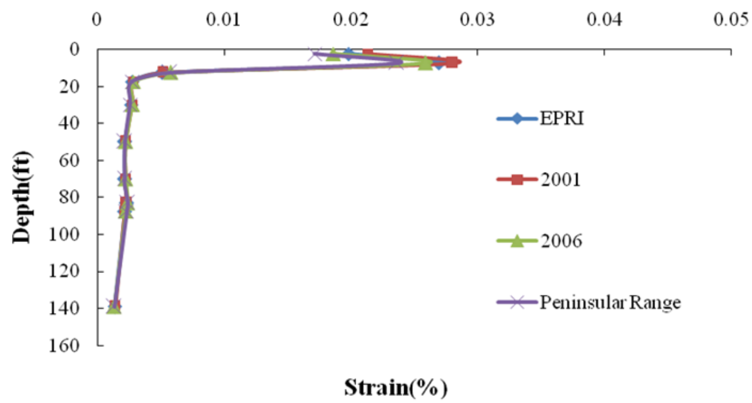
(b)



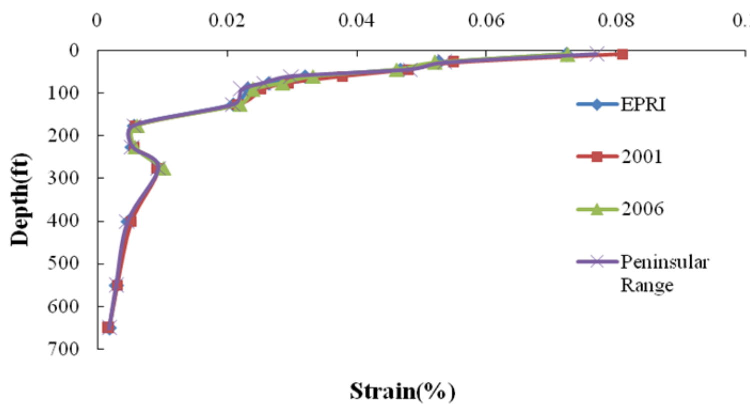
(c)

Fig. 1.16 Comparison of horizontal response spectra under 2500-year earthquake: class A bedrock and 5% damping: (a) class C profile; (b) soil class D; (c) soil class E

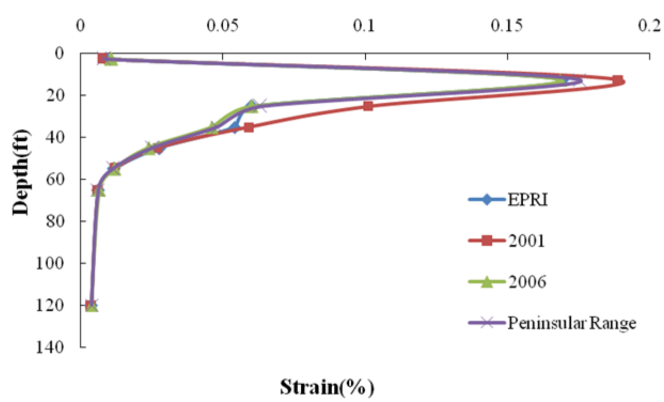
Fig. 1.16 shows some comparisons of horizontal response spectra at the ground surface under the 2500-year earthquake. The 1500-year earthquake was also analyzed and the difference among the three sets of results using different MRD curves was similar to those shown in Fig. 1.16. Overall the differences were small, and as expected, are only apparent at short periods. Fig. 1.17 shows the corresponding maximum shear strains developed in the profiles. The maximum shear strains were generally smaller than 0.2%, which explains the small differences among the three sets of results because the MRD curves are similar at small strains. The small shear strains also indicate that equivalent linear analysis methods are acceptable for developing the horizontal response spectra in the area. The Commentary to the Guidelines should add a note to this effect.



(a)

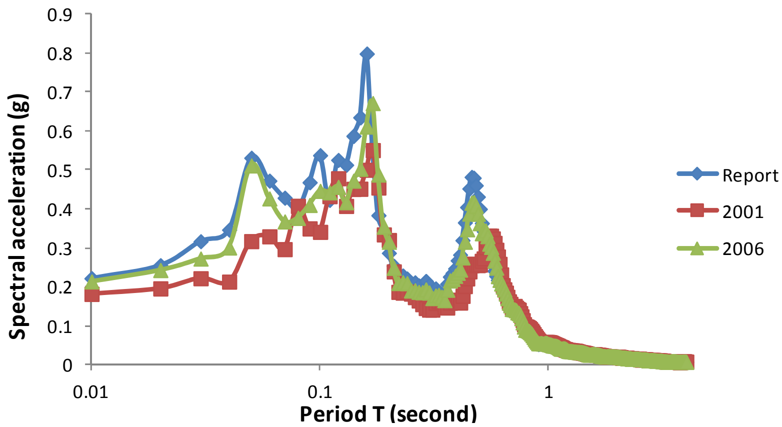


(b)

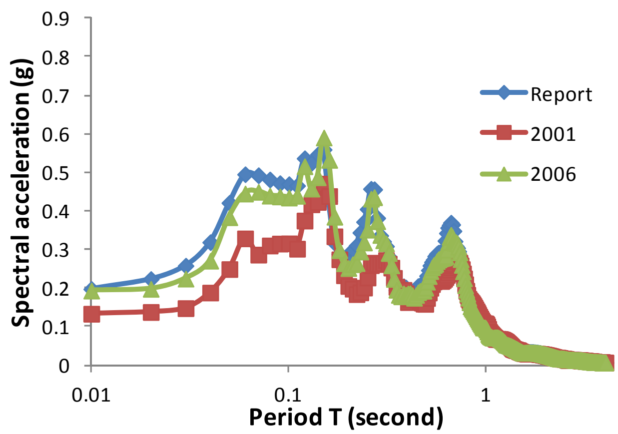


(c)

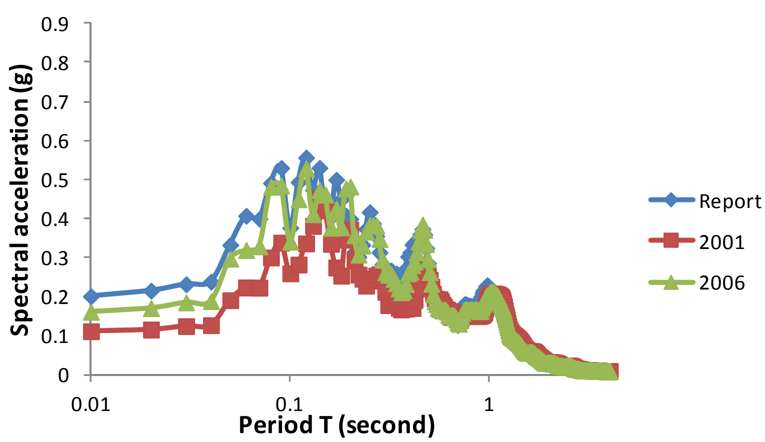
Fig. 1.17 Maximum shear strain in the soil deposits under 2500-year earthquake: (a) soil class C; (b) soil class D; (c) soil class E



(a)



(b)



(c)

Fig. 1.18 Comparison of horizontal response spectra of assumed class E sites with mainly high plastic clays (PI = 40) under the 1500-year earthquake: (a) 50 ft; (b) 100 ft; (c) 200 ft

Three extreme cases were also analyzed to check the effects of using different MRD curves. Three Class E profiles were analyzed having soil depths of 50 feet, 100 ft and 200ft, and mainly highly plastic clays (PI = 40). The bedrock was assumed to be Class A rock. Fig. 1.18 shows the results under the 1500-year earthquake. The large difference in spectral accelerations occurs in the high frequency range; it was small for frequencies below than 5 Hz. The difference is somewhat larger under the 2500-year earthquake, but it was also mainly at high frequencies.

The analysis results indicated that the MRD curves adopted in developing the Guidelines Report are not likely to yield very different horizontal response spectra in the period range around or above T = 1.0 sec, which is more relevant to the seismic design of bridges in the NYC region compared to more recently published curves. In another word, the research team agrees that the Weidlinger Team employed reasonable MRD curves in developing the Guidelines Report.

Deep softer bedrock beneath soils

A deep Rock Class B as thick as 3000 ft was considered in the RVT analysis to accommodate possible deep sedimentary rock beneath soils. As discussed in the Section 1.4, the existence of very thick soft rock is not likely in the region. The possibility is even smaller if thick overlying soils exist above bedrock, since the modulus of weak rock increases with confining pressure (e.g. MacBeth 2004). Some clarification is needed in this case.

Generic horizontal design spectra for Soil Class E under 500-yr earthquake

The Guidelines Report proposes a much smaller amplification on Class-E sites than AASHTO (2009) at high frequencies, which contradicts the AASHTO Guide Specifications (2009). In AASHTO (2009), spectral acceleration at Class-E sites is larger than that at Class-C or D sites when the amplitude of bedrock motion is small, regardless of period range. The research team has a concern if more conservative generic spectra for Site Class E under an earthquake of 500-yr return period are needed.

The research team carried out a series of ground response analysis and compared the horizontal response spectra of Site Class C, D and E. The base profiles provided in the Guidelines Report were used but different depths of soil over bedrock were considered. Both Class A and Class B bedrocks were analyzed.

Although there are some variations, overall the horizontal spectra obtained from these analyses shared similar characteristics with Figs. 16 – 18 of the Guidelines Report. The spectral acceleration at short period was smaller for Site Class E.

The research team believes that this trend comes from the characteristics rock motion in this area. The VHR motions accepted by NYCDOT in 2004 have much higher dominant frequency than the AASHTO motions obtained from the program AASHTO GM2.1 accompanying AASHTO (2009). Bedrock motions with such characteristics can result in smaller spectral acceleration at short periods on soft soils (Soil Site Class E) even if the input magnitude is small. Our study therefore showed that Figures 16-18 of the Guidelines Report are reasonable.

Issues requiring clarifications

It is not clear if the integration of site response into Probabilistic Seismic Hazard (PSH) is done using Bazzurro and Cornell (2004)'s convolution procedure or the procedure that modifies the median and sigma inside the hazard integral. This should be described, as it is fundamental to the development of the generic spectra.

Site amplification factors should be compared to empirical estimates (e.g. Choi and Stewart 2005). It is needed to verify what is reported on p 46 of the Guidelines Commentary, indicating that results conform to info in recent literature.

Finally, the kappa values that were used to develop the Uniform Hazard Spectrum (UHS) should also be compared with more recent estimates from Compbell (2009).

Summary

Through extensive analysis, the research team in general agrees that, except some minor issues that need to be addressed or clarified, overall the content in section 6 of the Guidelines Report is reliable.

These minor issues include:

1) In the discussion of PSHA for VHR in Section 3 of the Commentary (Pages 8-9), a series of ground motion models that were used are listed. In Section 6 of the Commentary, reference is made on p 31 and 33 to omega-squared source model and CEUS parameters (repeated on p 33 of the commentary). The inconsistency should be clearly explained.
2) It is not clear if the integration of site response into PSH (Probabilistic Seismic Hazard) is done using Bazzurro and Cornell (2004)'s convolution procedure or the procedure that modifies the median and sigma inside the hazard integral.
3) The research team believes that the possibility of large thickness of soft rock in the NYC area is small.
4) The research team suggests that the Guidelines be clearer on the depth of subsurface exploration where such conditions may exist. If a continuous soft/medium clay layer (su<1000 psf, or Vs < 600 ft/sec) extends below 100 ft depth, the subsurface exploration should continue until the base of this layer is encountered or the thickness of clay layer is already 120 ft.
5) The symbol of $\overline{V}_{s100}$ representing average shear wave velocity is confusing. The research team suggests using just $\overline{V}_s$ to represent average shear wave velocity.
6) The research team believes that the use of shear wave velocity Vs over SPT blow count N or undrained shear strength Su should be more strongly encouraged.
7) Use of Vs is recommended to be required for the design of critical bridges.
8) The Guidelines allow the owner an option to waive the requirement for site specific study and assume site class D for the purpose of ground motion analysis. This should not be allowed.
9) The kappa values that were used to develop the Uniform Hazard Spectrum (UHS) should also be compared with more recent estimates from Compbell (2009).
10) Site amplification factors should be compared to empirical estimates (e.g. Choi and Stewart 2005).

Among them, items (1) to (4) have been resolved by the Weidlinger's team, while items (5) to (8) have been accepted by the NYSDOT. However, the Weidlinger's team has declined the last two comments. Since the research team does not have access to the data used in developing the Guidelines Report, they will not be incorporated in the Blue Pages.

1.6 Vertical Motions and Generic Design Spectra for Rock and Soil Sites

Section 7 of the 2008 NYCDOT Seismic Design Guidelines Report addresses vertical motions and generic design spectra for rock and soil sites. According to the Commentary, the vertical generic design spectra were obtained based on both RVT analysis and empirical ground motion models. Equal weights were assigned to the analytical and empirical ratios between vertical and horizontal spectral accelerations (the V/H ratios).

While the research team believes that more weight should be given to the empirical V/H models (e.g. Bozorgnia and Campbell 2004), the final V/H ratios proposed in the Guidelines Report seems to be consistent with the most recent empirical data (Bozorgnia and Campbell 2004).

Regarding the analysis of vertical ground motion, because it was related to horizontal motion, all of the research team's comments on soil properties and other issues affecting the horizontal ground motions, which have been discussed in detail in the Section 1.5, will also affect the vertical motion.

The research team also wonders why the most recent paper on empirical V/H ratios by Bozorgnia and Campbell (2004) was not cited. The Commentary cited their earlier paper (Campbell and Bozorgnia 2003). This concern has been resolved by the Weidlinger's team in its response to the comment.

The 2008 NYCDOT Seismic Design Guidelines Report provides generic design spectra in vertical direction for rock and soil sites. However, the Guidelines don't provide clear guidance on the use of generic vertical spectra for response spectrum analysis. The research team recommends that the generic vertical spectra should not be used for response spectrum analysis. Rather they should be used for reference purposes for comparing spectra of generated ground motions or for generating ground motion time histories using appropriate software.

1.7 Site Liquefaction

Section 8 of the proposed NYCDOT Guidelines suggests a procedure for a preliminary evaluation of the liquefaction potential at a site. This suggested procedure is the simplified method presented in Youd et al. (2001). This procedure is basically the same as that in AASHTO (2009). According to the NYCDOT Guidelines, a site should be classified as Site Class F if found to be liquefiable based on the preliminary analysis. In such situations, a site specific analysis of ground motions using an appropriate nonlinear code is required according to Section 9. The review presented in the following paragraphs focuses on two critical issues related to the simplified procedure: (i) reliability of the empirical procedure and (ii) the earthquake magnitude M_w to be used in the empirical analysis.

<u>Liquefaction Susceptibility and Triggering Analysis Procedures</u>

The procedure is similar to that originally developed by Seed and Idriss (1971), which has been updated a number of times, with the most recent incarnation being Youd et al. (2001). The Youd et al. (2001) document presents the findings of a group of experts from two workshops in 1996 and 1998. Since its publication, the procedure has until recently represented the state-of-practice throughout North America and much of the world.

Following a number of major earthquakes in the 1990s that have provided important additional case studies, along with other supporting research, substantial improvements have been introduced to the simplified approach, some of which were acknowledged in the Commentary of the Guidelines Report (Cetin et al. 2004; Bray and Sancio 2006; Moss et al. 2006, and Boulanger and Idriss 2006). Other modifications include those in Juang et al. (2002, 2006) and Cetin and Ozan (2009). Two especially important aspects of these revisions are as follows:

1. There is now broad consensus that the criteria for evaluating liquefaction susceptibility of soils with plastic fines in Youd et al. (2001) should no longer be used (i.e., the "Chinese criteria" of Seed and Idriss, 1982). Two methods based on

index tests have replaced the Chinese criteria, which are presented in Bray and Sancio (2006) and Boulanger and Idriss (2006). For critical projects, higher-level material specific testing should be used to judge liquefaction susceptibility (e.g., Chu et al., 2008).

2. For liquefaction triggering analysis, the additional case histories significantly improve upon the older criteria in Youd et al. (2001) in several important respects:
   a. Stress-reduction (r_d) values in Youd et al. (2001) are based on limited site response analyses from Seed and Idriss (1971), which are now known to be biased. An important practical implication of those r_d values is that they affect the position of data points that establish the CSR-$(N_1)_{60}$ triggering relationship. The r_d relations in the newer publications (Idriss and Boulanger, 2006; Cetin et al., 2004; Moss et al. 2006) are based on many more site response analyses and largely avoid this problem.
   b. The relations between cyclic resistance ratio (CRR) and penetration resistance (from SPT or CPT) have been improved. The new relations are more reliable for CSRs greater than about 0.25 due to the inclusion of new case histories.
   c. Other improvements to the 2001 procedure include the overburden correction factor (K_σ), the magnitude scaling factor (MSF), the overburden correction for penetration resistance (C_N), and the fines correction term (FC).

These modifications are discussed in detail by Cetin et al. (2004), Idriss and Boulanger (2004, 2006, 2008), and Moss et al. (2006).

We agree that empirical liquefaction susceptibility and triggering procedures based on conventional exploration methods and index properties of soil can still be used, but recent developments on the liquefaction susceptibility criteria and other important parameters should be taken into account. The liquefaction susceptibility of soil with plastic fines should be evaluated according to Bray and Sancio (2006) and Boulanger and Idriss (2006) criteria, and for critical projects on the basis of more advanced testing as described for example in Chu et al. (2008). We also recommend use of the more recent liquefaction triggering parameters in lieu of the now-dated Youd et al. (2001) methods, providing that they are used consistently. In other words, the Idriss and Boulanger method should be used with the various improvements recommended by Idriss and Boulanger (2004, 2006, 2008). Likewise, if the Cetin et al. (2004) method is going to be used, it should be used in its entirety.

Earthquake magnitude M_w

In the evaluation of liquefaction potential using the simplified procedure, the magnitude M_w is included mainly to take into account the number of loading cycles or the duration of strong ground shaking in an earthquake (e.g., Seed and Idriss 1982; Youd et al. 2001; Idriss and Boulanger, 2006; Cetin et al., 2004).

In the proposed NYCDOT Guidelines, a magnitude $M_w = 6.25$ is suggested for critical bridges for a preliminary evaluation of liquefaction potential. This magnitude is 6.0 for non-critical bridges. The earthquake magnitudes for the two levels of seismic hazard are not strictly based on the disaggregations of the corresponding PGAs. The disaggregation at T = 0.1 sec, as shown in Fig. 1.19, was used in the Guidelines Report to obtain the M_w for the 2500-year hazard. The magnitude suggested for the 1500-year earthquake was not based on the disaggregation at any period; it was simply assumed to be 6.0 based on M_w = 6.25 for the 2500-year earthquake.

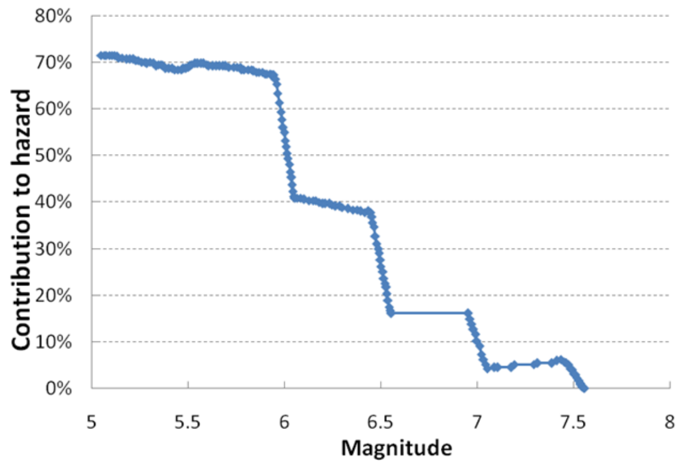


Fig. 1.19 Magnitude contribution to 2500-year seismic hazard at T = 0.1 sec. in New York City. **[From Risk Engineering Report (2002)]**

Both the AASHTO Specifications (2009) and the FHWA Seismic Retrofitting Manual for Highway Structures (2006) use mean magnitude from disaggregation analysis of peak ground acceleration (PGA). Hence, there are two differences from the NYCDOT Guidelines Report: (1) the ground motion intensity parameter that is the basis for the disaggregation is PGA (AASHTO, FHWA) instead of 0.1 sec Sa (NYCDOT) and (2) the selected magnitude is specifically defined as the mean in the AASHTO/FHWA approach, whereas the selected magnitude in NYCDOT is less clearly associated with a particular statistical attribute of the disaggregation. We recommend the use of PGA disaggregation because PGA is the ground motion parameter used in liquefaction triggering analysis. We also recommend explicitly stating the basis for the selected magnitude given a disaggregation result.

Many magnitude earthquakes contribute to the design PGA, and the appropriate magnitude for use within the distribution has been the subject of recent research. Kramer and Mayfield (2005, 2007) and Mayfield et al. (2010) showed that magnitudes larger than the mean magnitude often control the liquefaction potential, and for cases where a single magnitude is to be applied in combination with the design PGA, the 70-80th percentile magnitude is preferred to the mean.

Summary

The research team has the following recommendations on site liquefaction issue:

1) We recommend that the criteria for evaluating liquefaction susceptibility of soils with plastic fines in Youd et al. (2001) should no longer be used (i.e., the “Chinese criteria” of Seed and Idriss, 1982). Two methods based on index tests have replaced the Chinese criteria, which are presented in Bray and Sancio (2006) and Boulanger and Idriss (2006).

2) We also recommend use of the more recent liquefaction triggering parameters in lieu of the now-dated Youd et al. (2001) methods, providing that they are used consistently. In other words, the Idriss and Boulanger method should be used with the various improvements recommended by Idriss and Boulanger (2004, 2006, 2008). Likewise, if the Cetin et al. (2004) method is going to be used, it should be used in its entirety.

3) We recommend for both levels of seismic hazard (2500-year earthquake or 1000-year earthquake) (1) that disaggregations of PGAs be used in lieu of disaggregations on spectral ordinates and (2) that 70-80th percentile moment magnitudes be selected from the distribution found in the PGA deaggregations instead of the mean magnitude. The magnitude should also be larger than or equal to 6.0.

The first and second recommendations above have been accepted by the NYSDOT. Regarding the third recommendation, based on the Weidlinger's team's response, the research team can accept the use of Mw=6.25 for liquefaction analysis of critical bridges, as it is not expected that the deaggreations of PGA and Sa at 0.1 second would result in large difference. For non-critical bridges, if the 1000-yr motion is adopted, Mw=6.0 for liquefaction analysis should also be sufficient.

1.8 Site Specific Studies

Section 9 of the 2008 NYCDOT Seismic Design Guidelines Report addresses site specific studies for bridges, with sections on general requirements for analyses (Section 9.1), representation of spatially variable ground motions (9.2), and special considerations for critical and non-critical bridges (Sections 9.3-9.4, respectively). Site specific studies are required for all critical bridges and for non-critical bridges on Site Class F. The research team agrees with this requirement. Our concerns are related to details regarding how these site-specific studies are performed, as described in the following sections.

Horizontal Motions for Site Classes A and B

We have concerns with the period-independent site factors for Site Classes A and B. These concerns are described in the Section 1.4.

We also suggest that site specific ground response analysis be allowed to obtain the horizontal motions for Rock Classes A and B, provided that shear wave velocity measurement proves the existence of Very Hard Rock (VHR) below the layer of Rock Class A or Rock Class B.

Vertical Ground Motions

From its extensive consulting experience, Parson Brinckerhoff (PB), one member of the research team, found that design vertical response spectra in the soil may not be very different from the design vertical response spectra in the underlying rock. The V/H ratio will be further discussed in the Section 1.6.

Site Specific Studies

This section calls for geotechnical ground response analyses to evaluate site effects. The requirement in this section is consistent with the basic principles as per Section 3.4.3 of Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2009) and Section 3.10.2.2 of AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) but considers the special situation in New York City. The research team agrees with the Weidlinger's team on the general aspects of this section but we are concerned at the lack of guidance on soil property measurements, parameter selection, and analysis procedures. We appreciate that guidelines should not be overly prescriptive, but some general direction and minimum requirements should be provided, perhaps in the commentary. Some specific issues that should be discussed include:

1) Shear wave velocity measurement: The Guidelines Report recommends use of a "best estimate" V_s from in situ measurements to represent the shear wave velocity profile for site-specific ground response analysis without specifying the test methods. We

recommend the Commentary list standardized test methods by ASTM or AASHTO that can be used.

2) Selection of modulus reduction and damping (MRD) curves: The Guidelines Report currently provides no suggestions on the selection of MRD curves. We recommend the curves selected for analysis be based on material-specific testing following standard ASTM or AASHTO procedures or that they be selected from the literature. If the latter approach is taken, the MRD model should, at the very least, account for the effects of plasticity index and confining pressure on MRD curves for non-organic soils. Plasticity index is essential, because it separates soils by mineral composition, which is well known to have first-order effects on MRD characteristics (Vucetic and Dobry, 1991). Confining pressure has long been recognized as important for sands (Iwasaki et al., 1978; EPRI, 1993), but is now recognized as also affecting gravels (Rollins et al., 1998) and low plasticity clays (Darendeli, 2001). Examples of models that include both effects are Darendeli (2001) and Zhang et al. (2006), and additional models will become available by 2011 as part of the NGA-East project. Kishida et al. (2009) similarly provide a useful model for peaty organic soils.

3) Selection of computer code (EL/NL, and type of NL): The Guidelines Report does not specify the computer codes for site-specific ground response analysis. While we recognize that a highly prescriptive approach to this problem would be problematic (e.g., if a particular program is recommended, it might be superseded or become unavailable over time), we nonetheless recommend that some general guidelines be provided, perhaps in the Commentary, on some of the general considerations associated with the selection of an analysis method. This is critical because different codes may yield very different computed responses, especially if layers of the soil are susceptible to liquefaction.

   a. We recommend that the Guidelines Report provide some guidance on the use of equivalent-linear (EL) versus nonlinear (NL) codes. Because of their computational efficiency and relatively small number of required parameters, EL codes are typically the default approach in practice, but are problematic for problems involving large shear strains, including liquefaction problems. NL codes fall into two categories based on their soil material models. Many NL codes utilize relatively simple cyclic stress-strain relationships, which are characterized by a backbone curve and a series of rules that describe unloading-reloading behavior, pore-water generation, and cyclic modulus degradation. Other NL codes utilize advanced constitutive models incorporating yield surfaces, hardening laws, and flow rules to describe the behavior of soils under seismic shaking. The relatively simple NL codes have the advantage of fewer input parameters than the advanced procedures, but the simplified procedures cannot produce potentially significant liquefaction effects, such as within-cycle dilation.

   b. If soil liquefaction is expected to be triggered based on the results of calculations from Chapter 8 and the liquefiable soils will not be mitigated, then ground motions at the ground surface should be evaluated using nonlinear effective stress analyses. We recommend that the guidelines provide a clear recommendation in this regard. The alternative of using equivalent linear analysis with reduced shear wave velocity, as suggested in the Guidelines Report's Commentary, should be discouraged. The soil parameters for nonlinear effective stress analysis should be strictly calibrated using the procedures specified in the corresponding user manuals. A series of parametric runs is recommended to capture the uncertainties of the analysis.

c. Among the effective stress nonlinear codes, the ones using cyclic stress-strain relationships do not track within-cycle pore pressure changes, only relatively gross overall pore pressure generation. Accordingly, those NL methods cannot account for the dilatant behavior of cohesionless soils following phase transformation, which generally occurs at large shear strains (e.g. Zeghal and Elgamal 1994; Bonilla et al. 2005; Liu and Song 2006). Some relatively advanced NL codes such as OpenSees can simulate these effects, but this requires additional model parameters. The choice of the appropriate code for a given application is highly site specific and general guidelines cannot be provided at this time. We recommend that the guidelines provide commentary on these issues.

d. If soil liquefaction is not expected to be triggered, the choice of EL or NL analysis depends on level of shear strain in soil (Stewart and Kwok, 2008).

Consideration of Parametric Variability in Evaluation of Horizontal Design Spectrum

The Guidelines Report addresses the topic of evaluating parametric variability in the analysis of site response by recommending that analyses be performed for the best estimate Vs profile and ±20% variations on the Vs profile. We find this recommendation to be reasonable, but we recommend that this section be re-named and significantly expanded to address variability from MRD curves and alternate input motions. In addition, guidance on how the horizontal design spectrum is obtained from the results of these analyses is needed in the Guidelines Report.

Variability in MRD curves becomes important when soil is sheared at large strains (e.g., Rathje et al., 2010). There are numerous references that could be used to estimate variations on MRD curves, including Darendeli (2001) and Zhang et al. (2008). We suggest that guidelines be provided to accommodate this source of variability in site response.

In order to consider the randomness of ground motion, the research team believes that, as a minimum, three input ground motions compatible with the rock design spectrum (VHR, Site Class A or Site Class B) should be used in ground response analyses. These multiple input motions should be used in combination with multiple sets of MRD curves and multiple Vs profiles.

The final issue that should be addressed in this section is the manner by which the ground response analysis results are merged with the UHS (uniform hazard spectrum) for rock to produce the estimated soil site design ground motions. Several alternatives are available, the most common of which is a hybrid approach in which the probabilistic rock spectrum is deterministically modified by soil/rock site factors determined by ground response analysis. Goulet and Stewart (2009) showed that this hybrid approach is unconservative relative to formally integrating the site response into the probabilistic analysis. Bazzurro and Cornell (2004) and Baturay and Stewart (2003) discuss several approaches by which the probabilistic integration can be performed. We recommend that this issue be discussed in the Commentary.

The horizontal design spectrum should also satisfy the minimum two-third rule if peer review is waived by the owner, i.e., the final design soil horizontal spectrum shall be equal to or greater than two thirds of the corresponding site generic spectrum. This requirement is consistent with that in AASHTO Guide Specifications (2009) and is only a minimum requirement.

Liquefaction Effects on Ground Motions

The Guidelines Report calls for the engineer to perform site response without liquefaction, calculate the seismic demand, and then evaluate triggering potential. If liquefaction is triggered, additional site response analyses are performed considering pore pressure generation. Response history analysis of bridges is performed using ground motions computed with and without pore pressure generation. The envelope of spectra from both approaches is recommended for use.

There is a bias in the assessment of seismic demand for liquefaction problems if the Youd et al. (2001) triggering procedure is used with site-specific analysis of shear stresses. See discussion in the Section 1.7 for additional details on this issue.

As described above for Section 9.1.3, many NL ground response analysis codes have relatively simple pore pressure models that do not account for within-cycle pore pressure variations, which can lead to pronounced dilation spikes in ground motion. Rather, pore pressure generation in these codes simply degrades the backbone curve, producing a "base isolation" effect that is non-physical. We recommend that the Commentary be expanded to address this issue. In fact, the guidelines indicate that short period motions always decrease. This is the case despite Fig. C9.5 of the Guidelines Report Commentary showing higher spectra at short periods (< about 0.3 sec) than what is predicted from EL analysis without pore pressure effects.

Time History Analysis of Bridges.

It is now recognized that the representations of ground motions with UHS provides an overly conservative estimate of seismic demand, because each spectral ordinate represents an extreme case of loading (for a long return period), and those extremes are unlikely to occur simultaneously at all spectral periods. A more realistic spectrum is the conditional mean spectrum (CMS, see Baker and Cornell, 2006), which matches the UHS at the period of interest (typically the first mode period) but has a more realistic spectral shape at other periods. We recommend that CMS be given as an alternative target spectrum for bridges subject to relatively advanced response history analysis. Discussion of this point should be added to the Guidelines Report and Commentary.

Spatial Variation of Ground Motions

The Guidelines Report indicates that spatially variable ground motions must be considered for critical, essential, or other bridges. The accompanying CDROM has sets of such motions. Guidelines encourage use of these motions, not site-specific spatially variable motions. The motions are for VHR conditions.

We recommend that the commentary include some discussion of how these motions were developed. The guidelines should not discourage the use of site-specific analysis of spatially variable ground motions, which considers various factors such as wave passage effect, attenuation effect and others at the specific location.

500-year earthquake

The research team believes that evaluating the response of a critical bridge for serviceability should be conducted. The corresponding design spectrum should still be obtained through site specific ground response analysis.

Peer-review

The Guidelines Report indicates peer-review "shall be done," but the requirement can be waived by the owner. It appears that the 2/3 rule is waived if peer review is done, but this is not directly stated.

Allowing the peer-review requirement to be waived by the owner would appear to remove the "teeth" from this provision. We recommend this provision be re-visited. The guidelines should make a clearer statement on waiving 2/3 rule when peer review is performed.

<u>Summary</u>

The research team has the following recommendations:

1) The horizontal design spectrum from site specific analysis should also satisfy the minimum two-third rule if peer review is waived by the owner, i.e., the final design soil horizontal spectrum shall be equal to or greater than two thirds of the corresponding site generic spectrum.
2) The Commentary should include some discussion of how the spatially varied ground motions were developed.
3) Conditional mean spectrum (CMS, see Baker and Cornell, 2006) should be given as an alternative target spectrum for bridges subject to relatively advanced response history analysis.
4) Site specific response of critical bridges for serviceability should be conducted for 500 Yr earthquakes.
5) Section 9.14 of the Commentary should be significantly expanded to address variability from MRD curves and alternate input motions. In addition, guidance on how the horizontal design spectrum is obtained from the results of these analyses is needed in the Guidelines Report.
6) Some general direction and minimum requirements should be provided in the Commentary on soil property measurements, parameter selection, and analysis procedures.
7) The Commentary should be expanded to address the issue of within-cycle pore pressure variations, which can lead to pronounced dilation spikes in ground motion.
8) Site specific ground response analysis should be allowed to obtain the horizontal motions for Rock Classes A and B, provided that shear wave velocity measurement proves the existence of Very Hard Rock (VHR) below the layer of Rock Class A or Rock Class B.

Among these recommendations, items (1) to (2) have been resolved by the responses of the Weidlinger's team, while items (3) to (8) have been accepted by the NYSDOT.

1.9 Seismic Performance Zones

The Proposed NYCDOT Guidelines Report uses both S_{D1} (Long period component) and S_{DS} (short period component) to determine appropriate seismic design category. It also requires a minimum Seismic Design Category (SDC) B for all bridges in New York City. While a minimum SDC B is appropriate for bridges in Down State New York, use of both S_{D1} and S_{DS} is significant departure from the AASHTO Seismic Design Specifications (2009) that utilized only S_{D1} to determine seismic design category for the bridge.

The research team doesn't see any rational justification to amplify seismic design category based on short period component since a SDC for a bridge determines its type of detailing and analysis approach. Based on AASHTO Seismic Guide specifications, most of bridges in the downstate region are likely to be in SDC B, which should be sufficient for the level of seismic activity anticipated for the region. Actual design details for SDC B will still depend on the design spectra. If the bridge has short period component, the seismic force for the design of members and connections may still be governed by the short period region of the spectra.

Since seismic characteristics of the NYC area has already been accounted for through prescribed spectra (which are already amplifying the response by a factor between 1.5 and 3 over those of AASHTO spectra), there is no valid rationale to further amplify the seismic resistance based on short period components. Including short period components in the selection of SDCs will unnecessarily place the bridge into higher seismic design category, requiring higher level pushover analysis, which essentially depends on the longer period component. Current AASHTO SDCs also include the effects of short period components in the form of more stringent design recommendations for connection forces.

Therefore, the research team recommends that the Seismic Design Categories should be based on 1-sec period spectra acceleration S_{D1}, as shown in Table 1.4.

Table 1.4: Criterion of seismic design category

Seismic Design Category (SDC)	S_{D1}
B	$S_{D1} \leq 0.30$
C	$0.30 < S_{D1} \leq 0.50$
D	$0.50 < S_{D1}$

This recommendation has been accepted by the NYSDOT.

1.10 Summary of Recommended Changes and Resolutions

Overall, the research team has identified 30 items based on an extensive review of the Guidelines Report, as described in previous sections. Table 1.5 below summarizes these items, the response from Weidlinger Associates and the resolution by NYSDOT.

Table 1.5: Summary of items identified by the research team

S.N.	Issue Raised by CCNY	Recommendation by CCNY	Feedback From Weidlinger	Resolution by NYSDOT	Subsection
1	Use of 1000-Yr Earthquake for Essential and Other Bridges	Recommended. 1000 Yr spectra can be interpolated from 500, 1500 and 2500 Yr spectra	Reject		1.2
2	Generic horizontal design spectra for Rock Class A and Rock Class B	Recommends generation of site factors for Rock Classes A and B	Reject	Reject	1.4
3	Kappa Values Used for Developing UHS	The kappa values should also be compared with more recent estimates from Compbell (2009).	Reject	Accept but no data	1.5
4	PGA Deaggregations	Recommends the use of	Resolved		1.7

		deaggregation of PGA to determine moment magnitude in liquefaction analysis.			
5	Soil Property Guidance	Recommends inclusion of several soil property guidance in the commentary	Reject	Accept	1.8
6	Variability from MRD Curves	Recommends addressing variability from MRD curves	Reject	Accept	1.8
7	With-in Cycle Pore Pressure Variations	Recommends addressing within cycle pore pressure variations in the commentary	Reject	Accept	1.8
8	Source Models in VHR Analysis	Description on source models needs to be included in the commentary	Resolved		1.4
9	Epistemic Uncertainty in Source Models	Guideline should have more information on epistemic uncertainty in source models.	Resolved		1.4
10	Basis of Hazard Results	Guideline should have more information on this for future reference purposes.	Resolved		1.4
11	Inconsistency in documentation of ground motion models	Identified inconsistency should be addressed.	Resolved		1.5
12	Integration of Site Response into PSH	The approach used to do this should be documented.	Resolved		1.5
13	Site Amplification Factors	These factors should be compared to their empirical estimates.	Reject	Accept but no data	1.5
14	Literature on V/H Ratios	Most recent literature should be cited.	Resolved		1.6
15	Spatial Variation in Ground Motions	Approach used to introduce spatial variation in ground motions should be included in the commentary.	Resolved		1.8
16	Waiver of Site-Specific Analysis for Critical Bridges	Such waiver shouldn't be allowed.	Reject	Accept	1.5
17	Use of shear wave velocity for site characterization	Use of shear wave velocity should be encouraged for site characterization.	Reject	Accept	1.5
18	Use of Shear Wave Velocity for Design	Shear velocity V_s be required for the design of critical bridges	Reject	Accept	1.5
19	Allowing lack of site-specific data for Critical Bridges	Site classification should always be based on site-specific data, especially for critical bridges.	Reject	Accept	1.8
20	Liquefaction Susceptibility of Soils With Plastic Fines	Criteria for evaluating liquefaction susceptibility of soils with plastic fines in Youd et al. (2001) should be replaced.	Reject	Accept	1.7
21	Liquefaction Susceptibility Analysis	This analysis should be conducted on the basis of more recent studies.	Reject	Accept	1.7
22	Horizontal Motions for Rock Classes A and B	Recommended to generate these through site specific analysis if shear velocity measurements show existence of VHR below these sites	Reject	Accept	1.8
23	Two Third Rule for Site-Specific Analysis	2/3rd rule should be applicable if peer review is waived by the	Reject	Accept	1.8

		owner.			
24	Conditional Mean Spectrum	Conditional mean spectrum should be used as an alternative target spectrum for relatively advanced response history analysis	Reject	Accept	1.8
25	Site Specific Analysis for 500-year earthquake	Site specific analysis should be conducted to obtain 500-Yr earthquake design spectrum	Reject	Accept	1.8
26	Seismic Design Category	SDC should be based on SD1 as in AASHTO Guide Specifications.	Reject	Accept	1.9
27	Thickness of soil over rock	Inconsistency w.r.t. AASHTO should be removed.	Reject	Accept	1.3
28	Existence of deep softer rock in the region	Possibility of large thickness of soft rock in the NYC area is small.	Resolved		1.5
29	Symbol of average shear wave velocity	The symbol for average shear wave velocity is confusing.	Reject	Accept	1.5
30	Inconsistency in site classification approach	This inconsistency is in AASHTO Seismic Guide Specification and should be addressed in NYCDOT guideline.	Resolved		1.5

CHAPTER 2 : COMPARATIVE STUDY OF SEISMIC DESIGN OF BRIDGES USING AASHTO LRFD AND NYCDOT DESIGN SPECTRA

2.1 Introduction

An extensive comparative analysis between response quantities of a bridge subjected to AASHTO and NYCDOT spectra has been carried out to investigate comparative magnitude of key response quantities of a representative bridge. For this purpose, a 3-span I-shape steel continuous bridge has been modeled in SAP2000. A detailed description of the bridge is presented in the following.

2.2 Bridge Description and Modeling

The example bridge is a conventional multi-span bridge having multiple continuous spans using multiple I-shape girders and piers. It has fixed bearings at one of the piers and expansion bearings at the remaining piers and at both abutments. Fig. 2.1 shows plan and elevation view of the bridge.

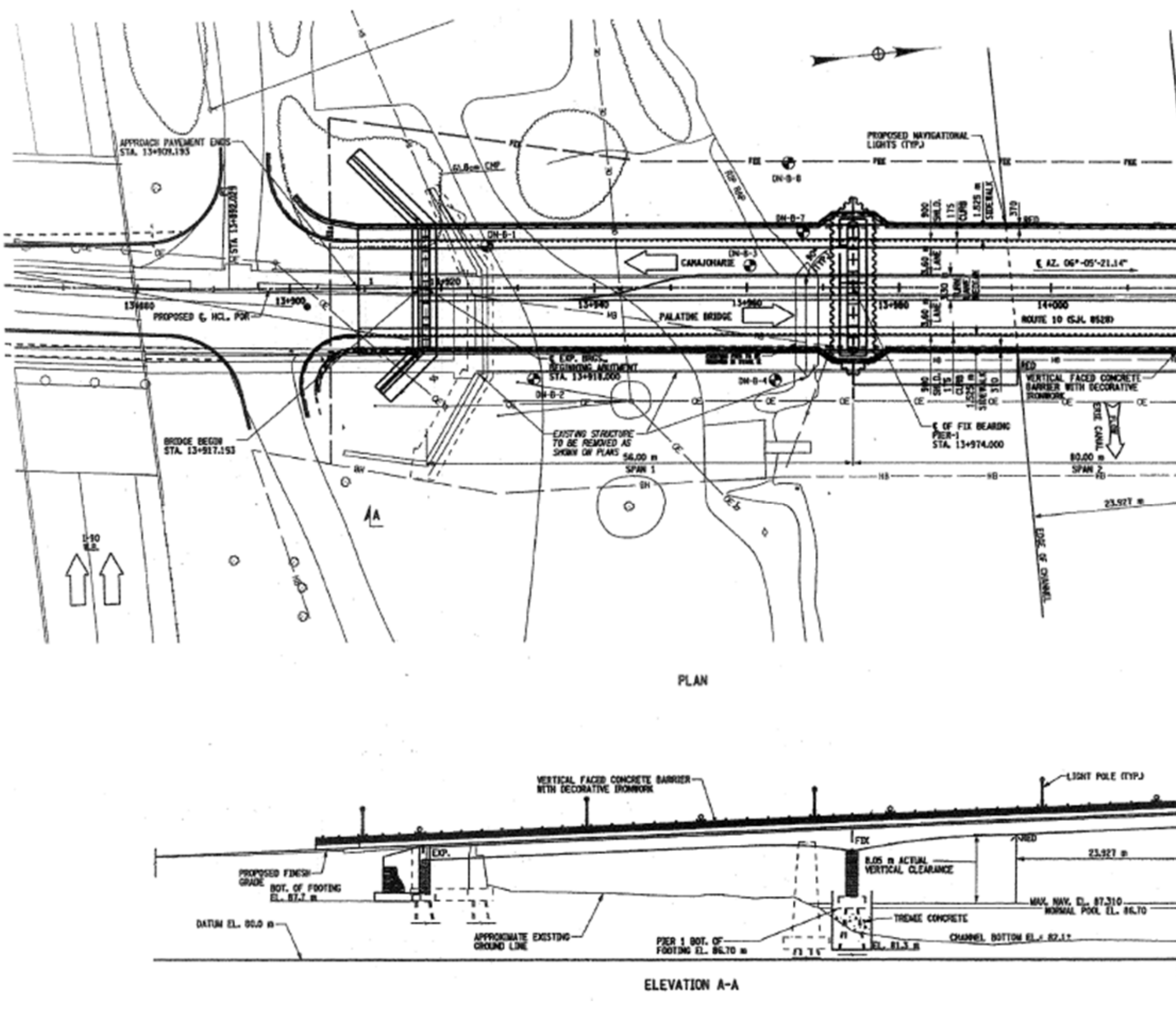


Fig. 2.1 Plan and elevation view of the bridge

Fig. 2.2 shows 3-D model of the bridge in SAP2000. All components of the bridge except bearings are modeled using beam-column frame elements. Model dimensions and results are shown in the units of kN and m. A detailed description of modeling of different components is presented in the following.

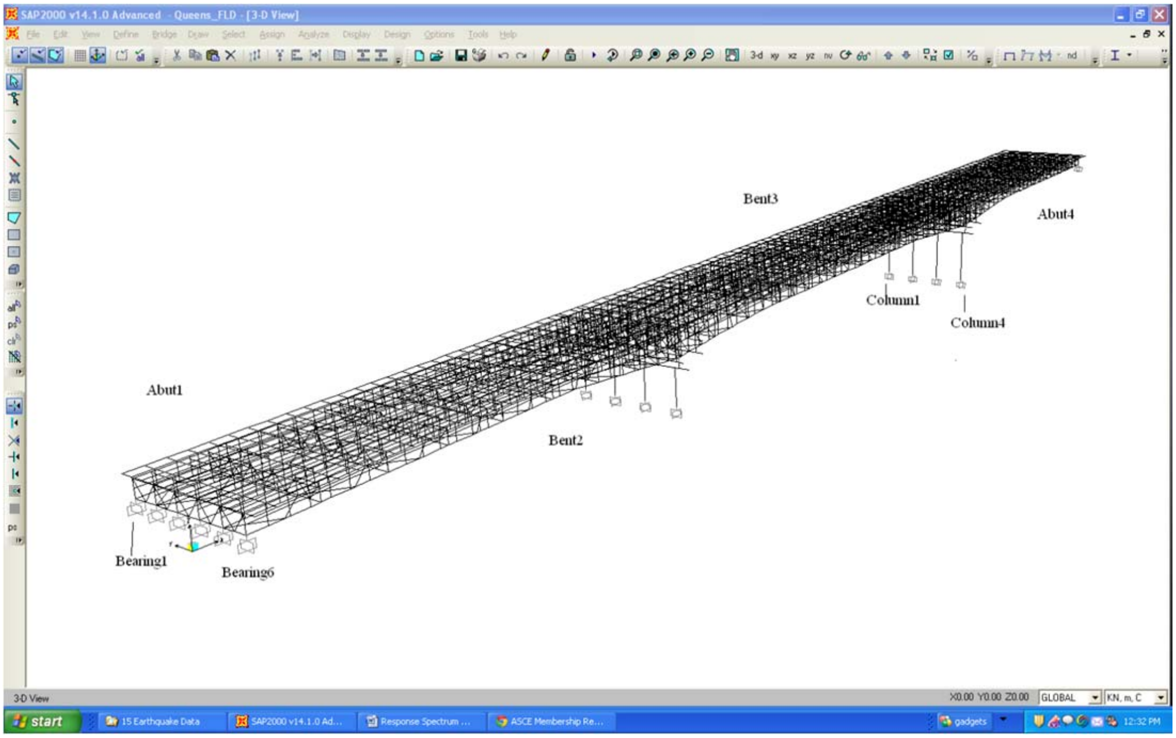
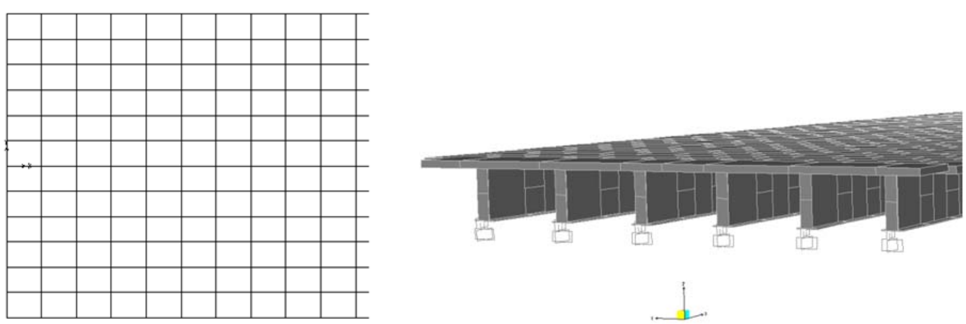


Fig. 2.2 Whole bridge 3-D finite element model using SAP2000

<u>Superstructure</u>

Concrete Deck: The concrete deck is 16.44 m wide and 0.24 m thick. It is supported by 13 beams, each of a section of 1.2646 m × 0.24 m. These longitudinal beams are connected to each other through transverse beams of sections 2 m× 0.24 m with zero mass at every 2 m along the entire bridge decks. Fig. 2.3(a) shows plan view of the deck. Fig. 2.3(b) shows 3-D view of the deck.



(a) 2-D (x-y) Plan View of the Deck (b) 3-D Bridge Deck Model

Fig. 2.3 Grid beam model of the bridge deck

I-Shape Steel Girder: The depth of I-shape girders along the longitudinal direction of the bridge span varies. Flanges of the girders are modeled by one continuous beam at the top and bottom of the girders and the web is modeled by one horizontal beam and one vertical beam without mass. These flanges and webs are connected to each other at cross nodes as shown in Fig. 2.5. Fig. 2.4 shows the 3-D view of the girder flanges, webs having different sectional depths, and intermediate stiffener for the girders.

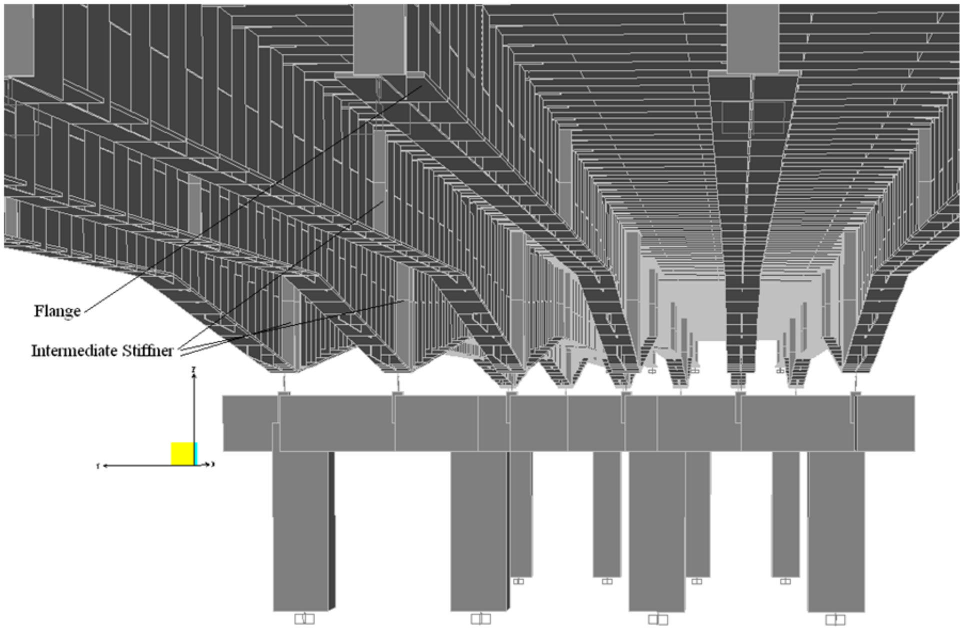


Fig. 2.4 3-D view of underside of the bridge

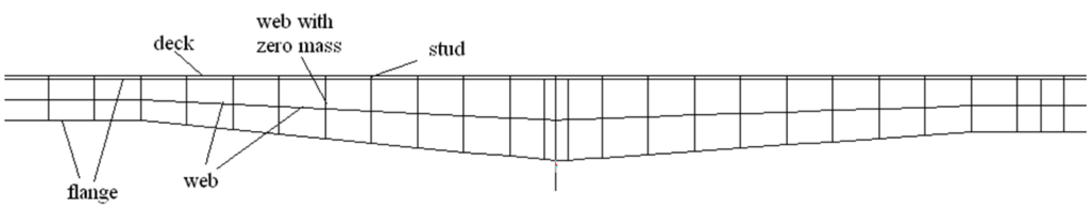


Fig. 2.5 I-shape girders in the bridge

Studs: The bridge deck is rigidly connected to I-Shape girders through studs. In SAP2000, studs are modeled by rigid beam-column elements at each cross node between two components as shown in Fig. 2.6.

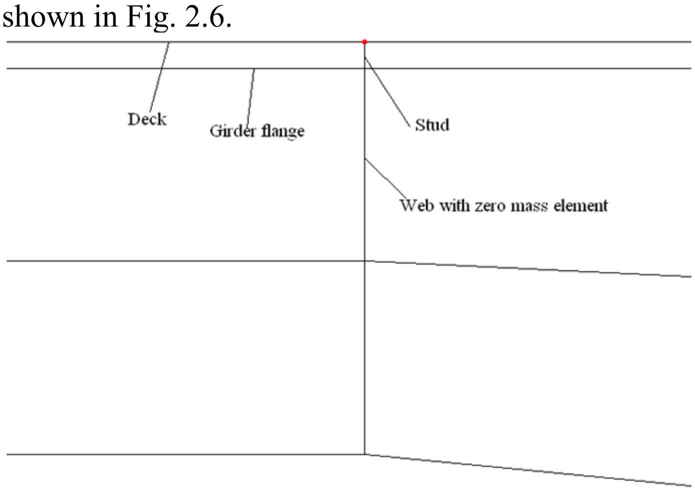


Fig. 2.6 Modeling of studs in the bridge deck

Stiffener: Intermediate stiffeners in longitudinal girders, as illustrated in Fig. 2.5, have different sectional properties presented in Table 2.1. Bearing stiffeners are installed at each end abutment and at each pier. The intermediate stiffeners are installed at 25.3m from the Abutment 1 at section A in the Fig. 2.7. Stiffeners are installed at the 24.6m from Bent 2 in the section D. All sectional properties of the components of the superstructures are symmetric about center of section D.

Table 2.1 Section properties of stiffeners

	Depth of the Stiffener	With of the Stiffener
Bearing Stiffener at Abutments	0.040	0.33
Bearing Stiffener at Pier	0.040	0.33
Intermediate Stiffener	0.018	0.20

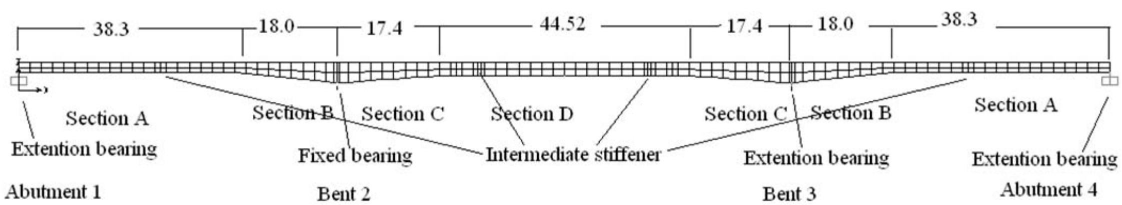


Fig. 2.7 Intermediate stiffeners I the longitudinal girders

Bracing: In order to prevent out-of-plane buckling of girders, typical cross frames are installed as bracing system between two girders. Each section has different section properties. Fig. 2.8 shows the sectional properties of typical cross bracing systems at the both end of abutments. Table 2.2 shows the cross bracing properties in bridge span section A and B in Fig. 2.8. The cross bracing systems, i.e., type T1, T2 and T3 are installed in the span as a one set. The distance between end of the abutment and type T1 in Table 2.2 is 1.75 m, the distance between type T1 and type T2 is 3.50 m, and the distance between type T2 and T3 is 1.75 m. These T1, T2, and T3 types as a one set are installed between girders at intervals of 7.273 m from Abutment 1 to Bent 2 in the Fig. 2.8. In case of the bridge sections C and D also, type T1, T2, and T3 are used, however at distances 1.818 m, 3.637 m, and 1.818 m, respectively. Lateral-diagonal bracings (WT6x115) are also installed between girder 1 and 2, between 2 and 3, and between 5 and 6. The layout for the lateral-diagonal bracing is shown in Figs. 2.9 and 2.10.

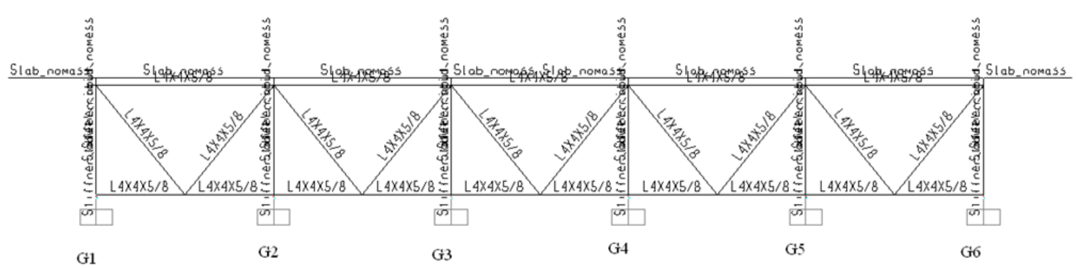


Fig. 2.8 Properties of the cross section at both ends of abutments

Table 2.2: Cross section properties at intermediate span of the bridge

Type	level	G1-G2	G2-G3	G3-G4	G4-G5	G5-G6
T1	Upper bracing	W8x58	W12x65		W12x65	
	Diagonal bracing					
	Lower bracing					
T2	Upper bracing	W8x58	W12x65		W12x65	
	Diagonal bracing					
	Lower bracing					
T3	Upper bracing	L5x5x3/8	L5x5x3/8			L5x5x3/8
	Diagonal bracing	L4x4x3/8	L4x4x3/8	L4x4x3/8	L4x4x3/8	L4x4x3/8
	Lower bracing	L4x4x3/8	L4x4x3/8	L4x4x3/8	L4x4x3/8	L4x4x3/8

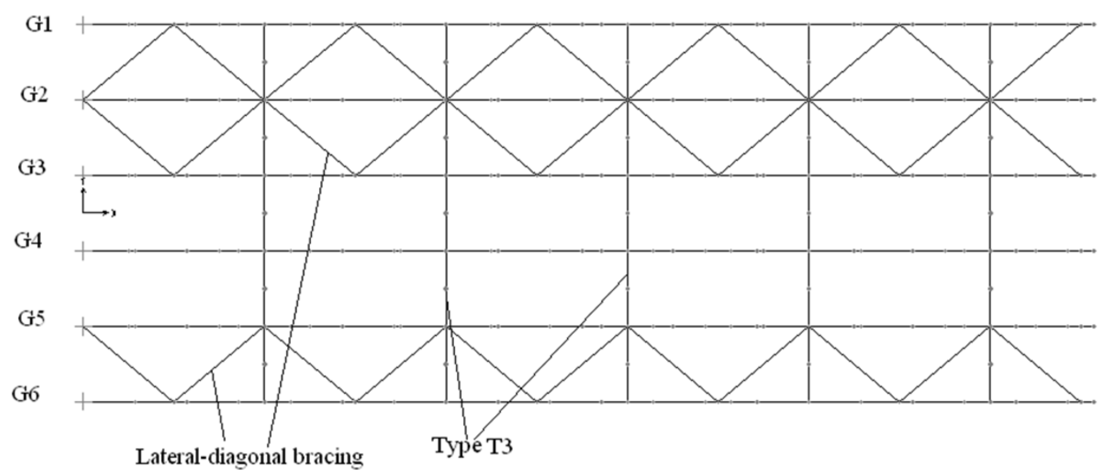


Fig. 2.9 The layout of the lateral-diagonal bracing at the bottom flange of the I-shape girder

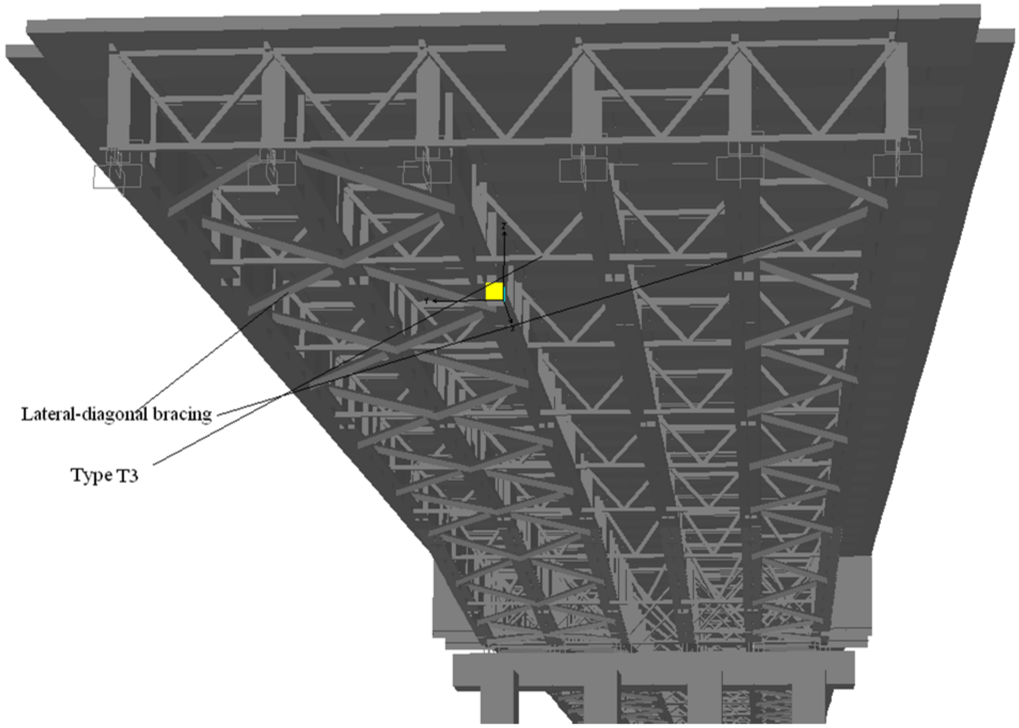


Fig. 2.10 Layout of lateral-diagonal bracing in the superstructure of the bridge

Substructure

Bearings: Fixed bearings are installed at the Bent 2, as shown in Fig. 2.7, at the bottom flanges of six I-shape girders. Expansion bearings are installed at the Abutment 1, 4 and Bent 3. Table 2.3 below shows details of bearing properties. These bearings are modeled using nonlinear link element shown in Fig. 2.11. Table 2.3 below shows properties of bearings. In Table 2.3, KF1F1, KF2F2, and KF3F3 are stiffness coefficients in longitudinal, transverse, and vertical directions, respectively. Likewise, KM1M1, KM2M2, and KM3M3 are rotational stiffness coefficients about longitudinal, transverse, and vertical directions, respectively.

Table 2.3 Cross sectional properties of bearings

	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
Abutment 1	1209	1.E+10	1.E+10	254	1.E+10	254
Bent 2	1.E+10	1.E+10	1.E+10	2572	0	2572
Bent 3	5661	1.E+10	1.E+10	2566	1.E+10	2566
Abutment 4	1209	1.E+10	1.E+10	254	1.E+10	254

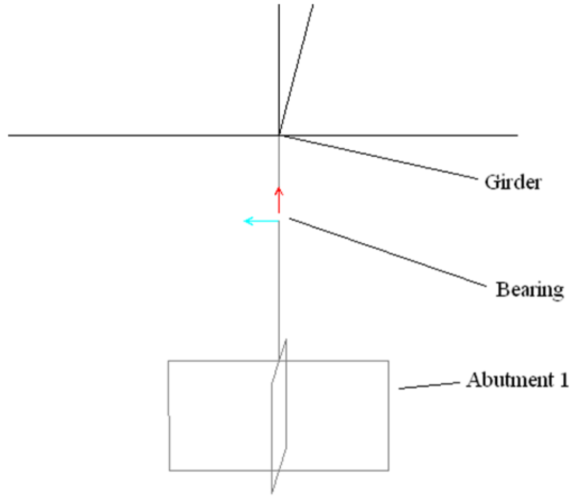


Fig 2.11 Nonlinear link element for modeling of bearings

Bent: Cap type bent is used at the bent 2 and 3. These bents are 7.4 m in length and are made of concrete beams with a cross-section of 1.4 m x1.5 m.

Pier: Rectangular reinforced concrete columns are used as piers. The sectional properties of columns are: 1.4 m x1.4 m with 4.02 m height for the bent 2 and 7.5m height for the bent 3, as shown in Fig. 2.12.

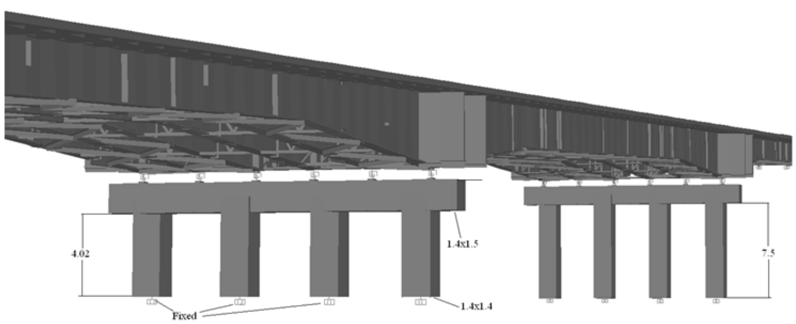


Fig. 2.12 Details of the substructure

Abutment: A detailed modeling of abutments hasn't been carried out. Fixed restraints are used under abutment bearings.

<u>Earthquake Design Spectra</u>

The analysis of bridge model has been carried out using the following earthquake design spectra:

- AASHTO 1000 Yr Spectra
- USGS 2500 Yr Spectra
- NYCDOT 500 Yr Spectra
- NYCDOT 1000 Yr Spectra
- NYCDOT 1500 Yr Spectra
- NYCDOT 2500 Yr Spectra

NYCDOT spectra correspond to “Soil on top of deep rock of any type- $H_r > 100$ ft”. Plots of these spectra are shown in following figures. It is noted that the spectral accelerations for Site Class C spectra for NYCDOT are much higher than those of Site classes D and E in the short period range.

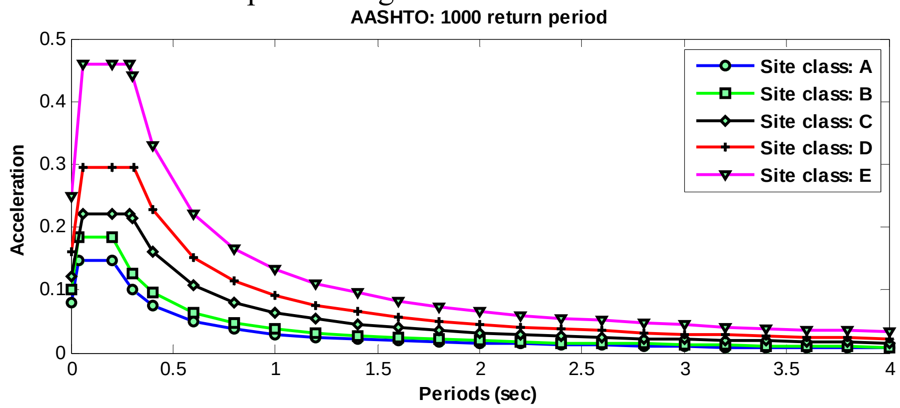


Fig. 2.13 Design spectra: AASHTO 1000 yr. return period

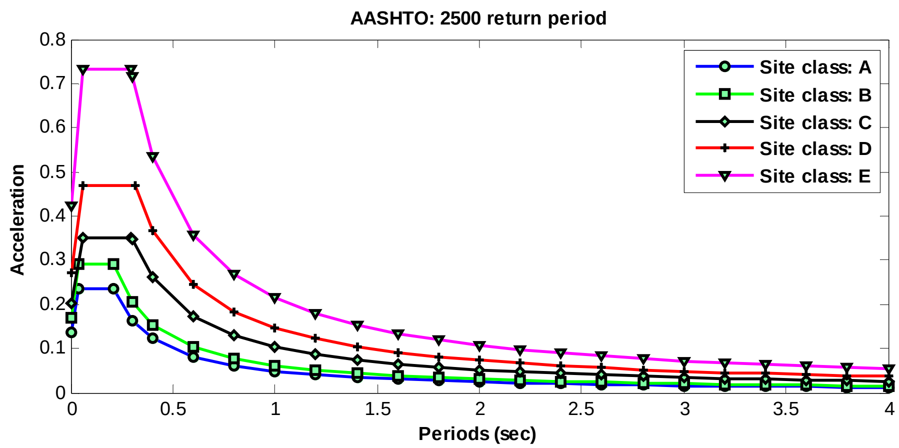


Fig. 2.14 Design spectra: USGS 2500 yr. return period

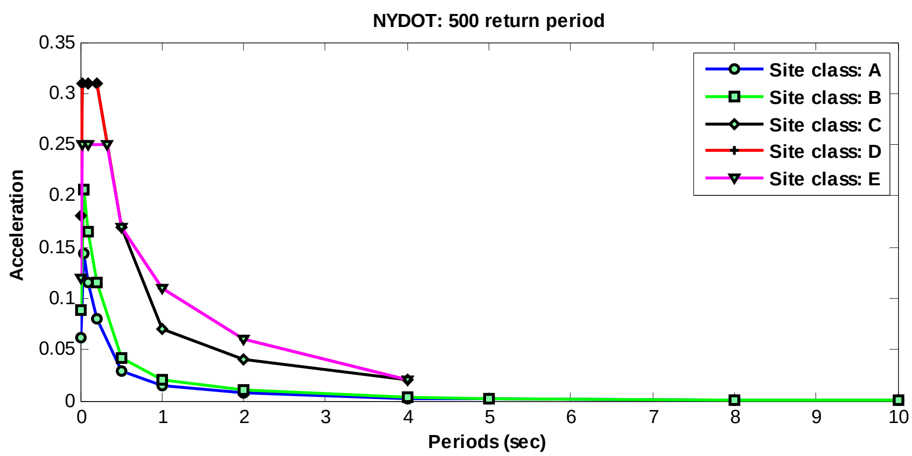


Fig. 2.15 Design spectra: NYCDOT 500 yr. return period

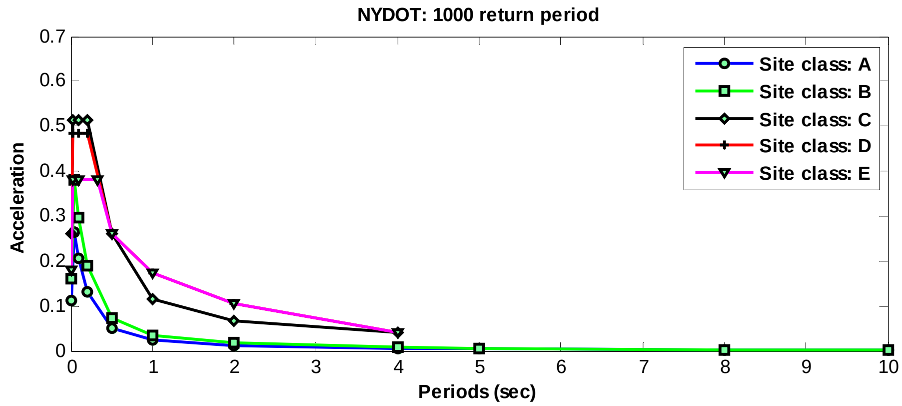


Fig. 2.16 Design spectra: NYCDOT 1000 yr. return period (based on interpolcation)

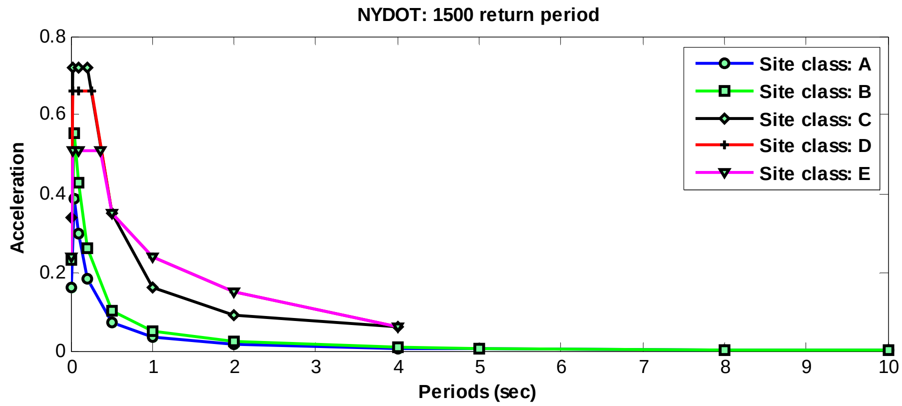


Fig. 2.17 Design spectra: NYCDOT 1500 yr. return period

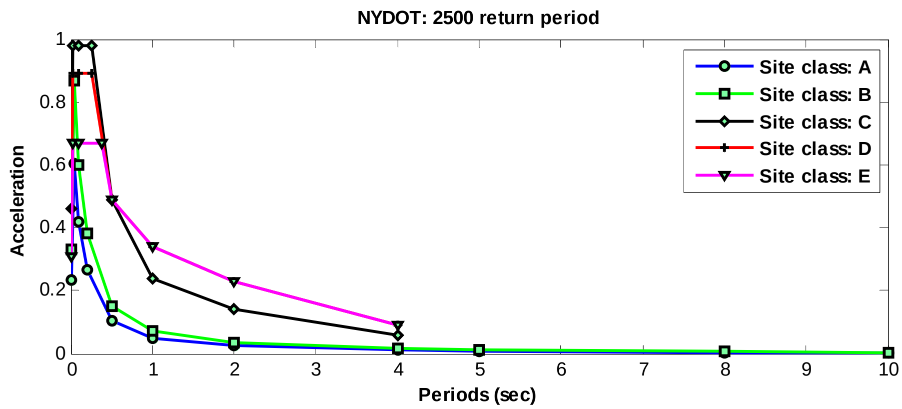


Fig. 2.18 Design spectra: NYCDOT 2500 yr. return period

2.3 Numerical Results

Table 2.4 below shows periods of different modes of the bridge. It is observed that the period of the first mode is 0.928 second. This is a bending mode. Periods of second and third mode are 0.604 and 0.507 seconds. The “B”, “L”, “TO”, “V”, and “TR” mean bending, longitudinal, torsional, vertical, and transverse mode, respectively.

Table 2.4 Natural periods of different modes

Mode	Periods (sec)
1	0.928(B)
2	0.604(L)
3	0.507(L)
4	0.506(TO)
5	0.448(V)
6	0.446(TR)
7	0.410(TO)
8	0.387(TO)
9	0.295(B)
10	0.273

Response of the bridge has been calculated using design spectra in Figs. 2.13 to 2.18. Maximum values of important response quantities, e.g., displacements, moments, shear, axial forces and torsion at bents and abutments, have been calculated to show comparisons between AASHTO and NYCDOT guidelines.

Design of Noncritical Bridges: As per AASHTO Seismic Guide Specifications, noncritical (essential and others) bridges are to be designed using 1000 Yr. return period earthquake. The draft version of 2008 NYCDOT guideline proposes to design noncritical bridges using 1500 Yr. return period NYCDOT spectra. During the review of Task 1, the consultants have proposed that noncritical bridges may be designed using 1000 Yr. NYCDOT spectra. Hence, Tables 2.5 and 2.6 below show comparisons between response quantities obtained using 1000 Yr. AASHTO, 1500 Yr. NYCDOT and 1000 Yr. AASHTO, 1000 Yr. NYCDOT Spectra, respectively. In order to study the increase in seismic response quantities because of the adoption of 2008 Draft NYCDOT Guidelines, a ratio between response quantities for 1500 NYCDOT and 1000 Yr. AASHTO spectra has been calculated for each soil type. Fig. 2.19 shows the range plot of the ratios for each of the soil types. The figure shows maximum and minimum values of ratio (considering all response quantities in Table 2.5) and mean values of ratios of all response quantities in Table 2.5. It is observed that the ratios vary from 1.2 to 1.43 for Soil Type A, 1.35 to 1.57 for Soil Type B, 2.69 to 3.15 for Soil Type C, 1.96 to 2.23 for Soil Type D and 1.14 to 1.47 for Soil Type E. Hence, response quantities using 1500 Yr. return period earthquakes are observed to increase for all soil types. However, most significant increase over those obtained by 1000 Yr. AASHTO ground motions is observed in case of Soil Type C by a factor in the range of 2.69 to 3.15. This is because of local ground motion characteristics in the downstate region. As seen from Fig. 2.18, NYCDOT spectra for soil type C has significantly higher spectral accelerations in the period range of 0.2 to 0.5 seconds, as compared to other soil type. This is because of dominant short period components in very hard rock ground motion in the region. Natural periods of modes higher than 1 of the bridge are predominantly in the short period range and are affected directly by the short period characteristics of the ground motion.

Since 2009 AASHTO Seismic Guide Specifications recommends using 1000 Yr. return period earthquake for noncritical bridges, comparisons have also been carried out by using 1000 Yr. NYCDOT design spectra (obtained through interpolation), as shown in Table 2.6. Similar to the case described above, ratios of response quantities for 1000 Yr. NYCDOT and 1000 Yr. AASHTO spectra for different soil types have been calculated. Fig. 2.20 shows of maximum, minimum and mean values of ratios for different soil types. It is observed that the ratios increase in the range of 2 to 2.25 for Soil Type C and around

1.5 for Soil Type D. For soil types E, the ratios are in the range of 1.0. Hence, design forces using 1000 Yr. NYCDOT spectra are going to be equal to or larger than those using 1000 Yr. AASHTO spectra with a maximum amplification by a factor of 2.5 in case of Soil Type C. This amplification represents local soil condition effects that are not included in 2009 AASHTO Seismic Guide Specifications. The results also show that the 1000 Yr. NYCDOT spectra meet the minimum requirement of 2009 AASHTO Seismic Guide Specifications.

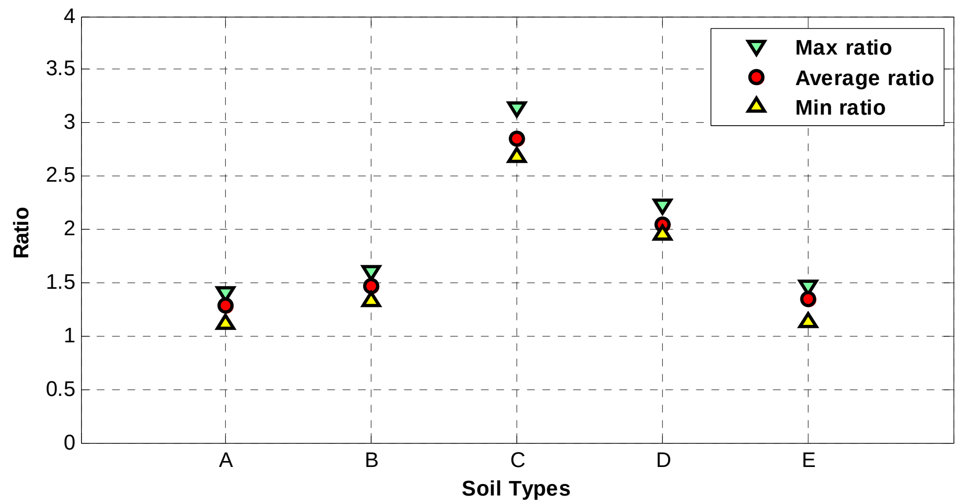


Fig. 2.19 Plot of range of ratios of response quantities using 1500 yr. NYCDOT and 1000 yr. AASHTO spectra for different soil types

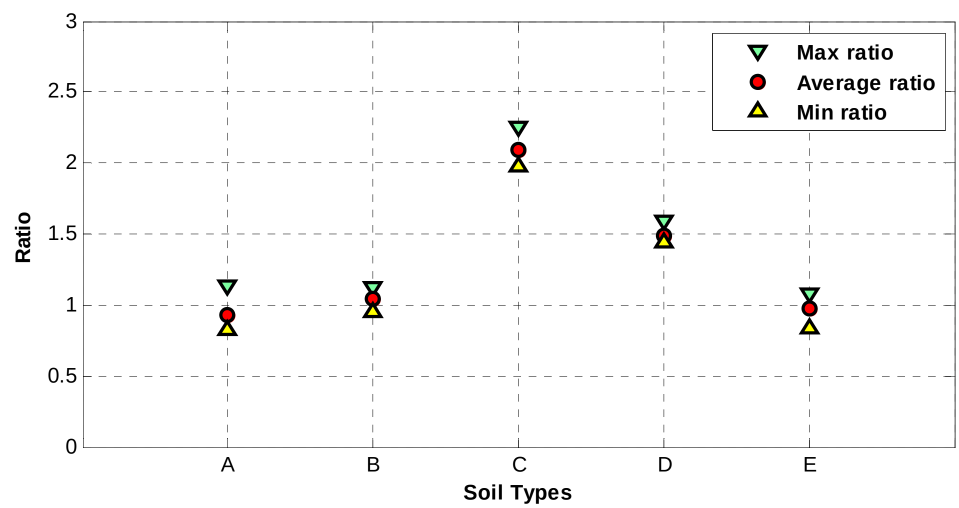


Fig. 2.20 Plot of range of ratios of response quantities using 1000 yr. NYCDOT and 1000 yr. AASHTO spectra for different soil types

Design of Critical Bridges: As per 2008 NYCDOT guideline, critical bridges are recommended to be designed for 500 Yr. return period earthquake for elastic response and for 2500 Yr. return period earthquake for repairable damage. Although AASHTO Seismic Guide Specifications don't provide any specific criteria for critical bridges, we are using 2500 Yr. return period spectra from USGS website for comparison. Tables 2.7 and 2.8 show comparisons between response quantities obtained using 1000 Yr.

AASHTO, 500 Yr. NYCDOT and 2500 Yr. USGS and 2500 Yr. NYCDOT Spectra, respectively.

Fig. 2.21 shows the plot of range of ratios of response quantities obtained by using 500Yr. NYCDOT and 1000 Yr. AASHTO spectra. It is observed that the ratio is less than 1 (i.e., response quantities using 500 Yr. NYCDOT spectra are smaller than those using 1000 Yr. AASHTO spectra) for Soil Types A, B and E and it is higher than 1 for Soil Types C and D. Fig. 2.22 shows the plot of range of ratio of response quantities obtained by using 2500Yr. NYCDOT and 2500 Yr. USGS spectra. It is observed that the ratios of response quantities for different soil types are similar to ratios of response quantities for 1500 NYCDOT and 1000 Yr. AASHTO spectra with maximum ratio being approximately 2.62 for soil type C (i.e., response quantities using 2500 Yr. NYCDOT spectra are amplified by a factor of 2.5 as compared with response quantities using 2500 Yr. USGS spectra)

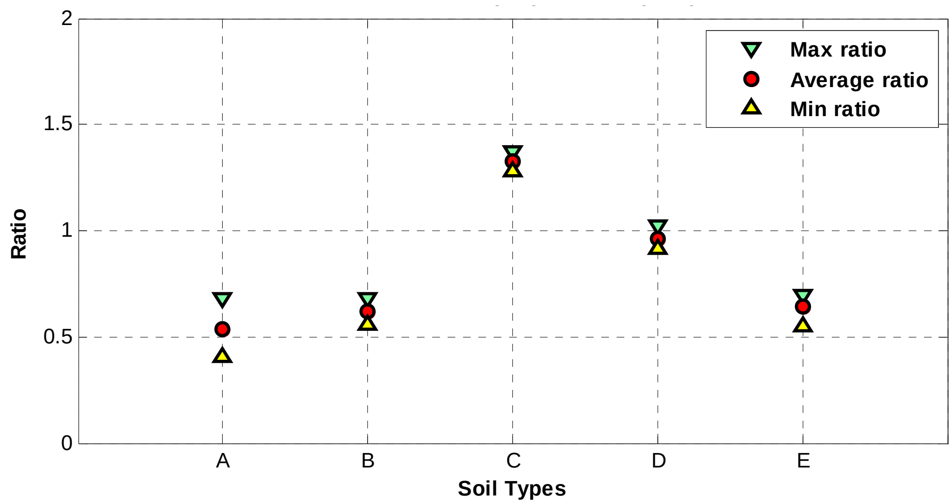


Fig. 2.21 Plot of range of ratios of response quantities using 500 yr. NYCDOT and 1000 yr. AASHTO spectra for different soil types

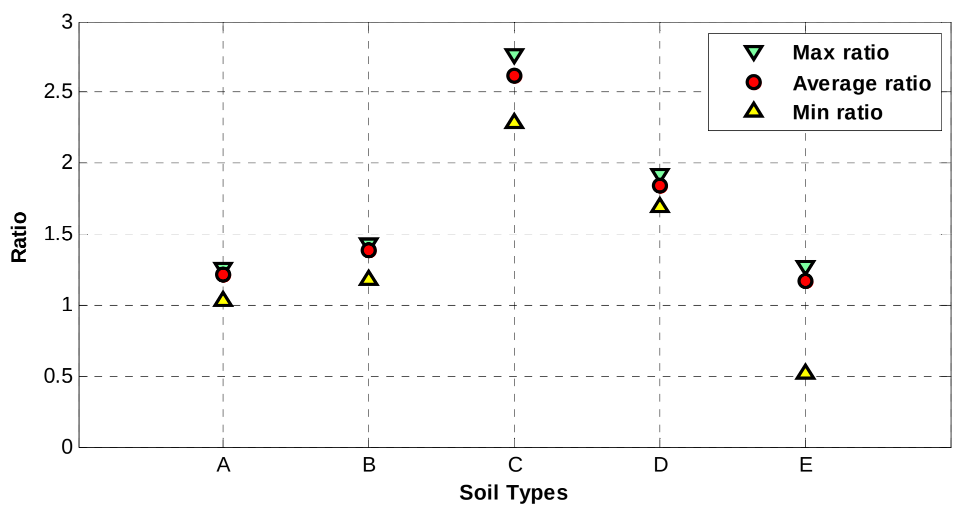


Fig. 2.22 Plot of range of ratios of response quantities using 2500 yr. NYCDOT and 2500 USGS spectra for different soil types

2.4 Summary

A comparative study of response quantities obtained using NYCDOT and AASHTO spectra has been carried out so that an understanding about increase in costs because of NYCDOT guideline can be developed. It is observed that the maximum increase in response quantities using the NYCDOT spectra occurs for Soil Type C. For this soil type, increase in response quantities for noncritical bridges is in the range of 2.69 to 3.15 and it is in the range of 2.5 for critical bridges (when subject to 2500 Yr. return period earthquake). Amplifications in response quantities for soil type C are because of predominance of short period components in the very hard rock ground motion of the downstate region. Overall, using AASHTO and NYCDOT design spectra of the same return period, while response quantities for soil sites A, B and E are in close range of 1 to 1.5, there is much higher amplification for soil types C and D using NYCDOT spectra in comparison to AASHTO spectra.

Table 2.5: Comparisons between response quantities using 1000 yr. AASHTO and 1500 yr. NYCDOT spectra (all quantities in units of kN, m)

	Soil Class Return period: 1000 for AASHTO and 1500 for NYCDOT (years)									
Item	A		B		C		D		E	
	AASHTO	NYCDOT	AASHTO	NYCDOT	AASHTO	NYCDOT	AASHTO	NYCDOT	AASHTO	NYCDOT
Bents and abutments displacements										
Long. disp. (m)	0.0035	0.0043	0.0044	0.0062	0.0075	0.0209	0.0106	0.0216	0.0155	0.0216
Trans. disp. (m)	0.0017	0.0024	0.0022	0.0034	0.0037	0.0106	0.0052	0.0106	0.0076	0.0106
Column forces at Bent 2										
Long. Shear (kN)	385	458	484	654	819	2225	1156	2264	1689	2261
Long. Moment (kN.m)	2173	2579	2686	3685	4622	12466	6520	12765	9528	12754
Trans. Shear (kN)	312	415	391	594	609	1789	848	1769	1257	1674
Trans. Moment (kN.m)	678	1076	1015	1540	1587	4651	2210	4602	3273	4362
Axial (kN)	548	733	686	1050	1092	3191	1525	3169	2251	3037
Torsion (kN.m)	55	72	69	103	105	309	145	304	216	286
Column forces at Bent 3										
Long. Shear (kN)	31	38	39	54	67	181	94	188	137	188
Long. Moment (kN.m)	203	260	259	371	439	1250	618	1300	904	1299
Trans. Shear (kN)	240	330	301	472	510	1468	719	1469	1049	1466
Trans. Moment (kN.m)	1039	1424	1301	2038	2202	6338	3104	6344	4528	6329
Axial (kN)	563	768	704	1099	1193	3431	1682	3438	2454	3431
Torsion (kN.m)	6	8	8	12	12	36	17	35	25	33
Bearing forces at Abutments										
Shear22 (kN)	4	5	5	8	9	25	13	26	19	26
Shear33 (kN)	164	205	206	294	261	821	351	777	543	618
Axial (kN)	69	87	87	124	123	360	172	357	254	339
Moment22 (Kn.m)	25	30	31	44	39	122	52	116	81	92
Moment33 (kN.m)	4	5	5	7	8	22	12	24	17	24
Torsion	0.0	0.1	0.1	0.1	0.1	0.2	0.1	0.2	0.2	0.2

(kN.m)										
Bearing forces at Bent 2										
Shear22 (kN)	288	337	355	481	599	1630	844	1656	1234	1650
Shear33 (kN)	291	389	365	557	569	1672	792	1653	1173	1564
Axial (kN)	247	331	309	474	498	1451	697	1446	1027	1397
Moment22 (kN.m)	148	197	185	282	289	849	402	839	595	794
Moment33 (kN.m)	85	101	107	144	180	489	253	497	370	495
Torsion (kN.m)	0.2	0.2	0.2	0.4	0.4	1	0.5	1.1	0.7	1.0
Bearing forces at Bent 3										
Shear22 (kN)	17	19	21	28	35	94	50	99	73	99
Shear33 (kN)	158	217	198	310	335	965	472	966	689	964
Axial (kN)	199	270	249	387	421	1210	594	1214	867	1211
Moment22 (kN.m)	41	55	50	79	85	244	120	245	175	210
Moment33 (kN.m)	122	155	162	222	275	745	388	834	566	834
Torsion (kN.m)	0.7	0.9	0.8	1.3	1.4	4.1	1.9	4.1	2.9	4.1

Table 2.6: Comparisons between response quantities using 1000 yr. AASHTO and 1000 yr. NYCDOT spectra (all quantities in units of kN, m)

	Soil Class Return period: 1000 for AASHTO and 1000 for NYCDOT (years)									
Item	**A**		**B**		**C**		**D**		**E**	
	AASHTO	**NYCDOT**	**AASHTO**	**NYCDOT**	**AASHTO**	**NYCDOT**	**AASHTO**	**NYCDOT**	**AASHTO**	**NYCDOT**
Bents and abutments displacements										
Long. disp. (m)	0.0035	0.0030	0.0044	0.0044	0.0075	0.0155	0.0106	0.0160	0.0155	0.0160
Trans. disp. (m)	0.0017	0.0017	0.0022	0.0024	0.0037	0.0077	0.0052	0.0075	0.0076	0.0075
Column forces at Bent 2										
Long. Shear (kN)	385	324	484	464	819	1650	1156	1676	1689	1673
Long. Moment (kN.m)	2173	1822	2686	2614	4622	9301	6520	9454	9528	9443
Trans. Shear (kN)	312	296	391	425	609	1301	848	1260	1257	1195
Trans. Moment (kN.m)	678	769	1015	1103	1587	3384	2210	3281	3273	3116
Axial (kN)	548	524	686	751	1092	2324	1525	2258	2251	2164
Torsion (kN.m)	55	51	69	74	105	224	145	217	216	204
Column forces at Bent 3										
Long. Shear (kN)	31	27	39	39	67	134	94	138	137	138
Long. Moment (kN.m)	203	184	259	264	439	924	618	958	904	957
Trans. Shear (kN)	240	235	301	337	510	1073	719	1047	1049	1035
Trans. Moment (kN.m)	1039	1015	1301	1456	2202	4634	3104	4522	4528	4471
Axial (kN)	563	547	704	785	1193	2510	1682	2455	2454	2428
Torsion (kN.m)	6	6	8	8	12	26	17	25	25	24
Bearing fo5rces at Abutments										
Shear22 (kN)	4	4	5	5	9	19	13	19	19	19
Shear33 (kN)	164	147	206	211	261	589	351	558	543	458
Axial (kN)	69	62	87	89	123	263	172	257	254	244
Moment22 (Kn.m)	25	22	31	31	39	88	52	83	81	68
Moment33 (kN.m)	4	3	5	5	8	16	12	17	17	17
Torsion (kN.m)	0.0	0.0	0.1	0.1	0.1	0.2	0.1	0.2	0.2	0.1
Bearing forces at Bent 2										
Shear22 (kN)	288	238	355	341	599	1208	844	1226	1234	1221
Shear33 (kN)	291	278	365	399	569	1216	792	1176	1173	1115
Axial (kN)	247	236	309	339	498	1058	697	1030	1027	995
Moment22 (kN.m)	148	140	185	202	289	617	402	597	595	566
Moment33 (kN.m)	85	71	107	102	180	362	253	368	370	366
Torsion (kN.m)	0.2	0.2	0.2	0.3	0.4	0.8	0.5	0.8	0.7	0.7
Bearing forces at Bent 3										
Shear22 (kN)	17	14	21	20	35	70	50	73	73	73
Shear33 (kN)	158	154	198	222	335	705	472	688	689	681
Axial (kN)	199	192	249	276	421	886	594	868	867	859
Moment22 (kN.m)	41	39	50	56	85	179	120	174	175	172
Moment33 (kN.m)	122	110	162	158	275	544	388	614	566	613
Torsion (kN.m)	0.7	0.7	0.8	0.9	1.4	3.0	1.9	3.6	2.9	2.9

Table 2.7: Comparisons between response quantities using 1000 yr. AASHTO and 500 yr. NYCDOT spectra (all quantities in units of kN, m)

	Soil Class Return period: 1000 for AASHTO and 500 for NYCDOT (years)									
Item	**A**		**B**		**C**		**D**		**E**	
	AASHTO	**NYCDOT**	**AASHTO**	**NYCDOT**	**AASHTO**	**NYCDOT**	**AASHTO**	**NYCDOT**	**AASHTO**	**NYCDOT**
Bents and abutments displacements										
Long. disp. (m)	0.0035	0.0017	0.0044	0.0026	0.0075	0.0101	0.0106	0.0104	0.0155	0.0104
Trans. disp. (m)	0.0017	0.0010	0.0022	0.0015	0.0037	0.0049	0.0052	0.0049	0.0076	0.0049
Column forces at Bent 2										
Long. Shear (kN)	385	189	484	272	819	1074	1156	1093	1689	1092
Long. Moment (kN.m)	2173	1065	2686	1529	4622	6056	6520	6166	9528	6163
Trans. Shear (kN)	216	179	391	256	609	814	848	814	1257	784
Trans. Moment (kN.m)	678	463	1015	663	1587	2119	2210	2120	3273	2045
Axial (kN)	548	315	686	451	1092	1458	1525	1460	2251	1420
Torsion (kN.m)	55	31	69	44	105	140	145	140	216	134
Column forces at Bent 3										
Long. Shear (kN)	31	16	39	23	67	86	94	90	137	90
Long. Moment (kN.m)	203	108	259	155	439	599	618	623	904	623
Trans. Shear (kN)	240	141	301	201	510	679	719	679	1049	679
Trans. Moment (kN.m)	1039	607	1301	869	2202	2930	3104	2930	4528	2933
Axial (kN)	563	227	704	469	1193	1589	1682	1591	2454	1593
Torsion (kN.m)	6	4	8	5	12	16	17	16	25	16
Bearing forces at Abutments										
Shear22 (kN)	4	2	5	3	9	12	13	13	19	13
Shear33 (kN)	164	90	206	129	261	357	351	357	543	301
Axial (kN)	69	38	87	54	123	165	172	667	254	160
Moment22 (Kn.m)	25	13	31	19	39	53	52	386	81	45
Moment33 (kN.m)	4	2	5	3	8	11	12	240	17	11
Torsion	0.0	0.0	0.1	0.0	0.1	0.1	0.1	0.1	0.2	0.1

(kN.m)										
Bearing forces at Bent 2										
Shear22 (kN)	288	139	355	200	599	785	844	799	1234	797
Shear33 (kN)	291	167	365	240	569	760	792	760	1173	732
Axial (kN)	247	142	309	203	498	665	697	166	1027	653
Moment22 (kN.m)	148	85	185	73	289	386	402	53	595	371
Moment33 (kN.m)	85	42	107	60	180	236	253	11	370	239
Torsion (kN.m)	0.2	0.1	0.2	0.1	0.4	0.5	0.5	0.1	0.7	0.5
Bearing forces at Bent 3										
Shear22 (kN)	17	8	21	12	35	45	50	47	73	47
Shear33 (kN)	158	92	198	120	335	446	472	446	689	446
Axial (kN)	199	115	249	165	421	561	594	563	867	563
Moment22 (kN.m)	36	23	50	34	85	113	120	113	175	113
Moment33 (kN.m)	122	65	162	93	275	352	388	396	566	395
Torsion (kN.m)	0.7	0.4	0.8	0.6	1.4	1.7	1.9	1.9	2.9	1.9

Table 2.8: Comparisons between response quantities using 2500 yr. USGS and 2500 yr. NYCDOT spectra (all quantities in units of kN, m)

	Soil Class Return period: 2500 for USGS and 2500 for NYCDOT (years)									
Item	**A**		**B**		**C**		**D**		**E**	
	USGS	**NYCDOT**	**USGS**	**NYCDOT**	**USGS**	**NYCDOT**	**USGS**	**NYCDOT**	**USGS**	**NYCDOT**
Bents and abutments displacements										
Long. disp. (m)	0.0057	0.0063	0.0070	0.0091	0.0122	0.0294	0.0172	0.0302	0.0251	0.0302
Trans. disp. (m)	0.0028	0.0035	0.0035	0.0050	0.0059	0.0152	0.0084	0.0147	0.0122	0.0146
Column forces at Bent 2										
Long. Shear (kN)	626	673	782	968	1327	3129	1873	3169	2733	3165
Long. Moment (kN.m)	3529	3791	4411	5451	7489	17634	10570	17872	15418	17854
Trans. Shear (kN)	504	608	630	872	982	2564	1368	2439	2027	2298
Trans. Moment (kN.m)	1310	1576	1637	2261	2557	6664	3566	6349	5280	5990
Axial (kN)	855	1074	1106	1542	1761	4582	2462	4379	3634	4184
Torsion (kN.m)	88	105	110	151	168	440	234	418	348	392
Column forces at Bent 3										
Long. Shear (kN)	51	56	64	80	107	257	152	263	222	263
Long. Moment (kN.m)	335	382	418	549	710	1767	1004	1820	1464	1495
Trans. Shear (kN)	389	484	486	694	642	2115	1164	2039	1698	2028
Trans. Moment (kN.m)	1678	2088	2098	2997	3561	9132	5027	8804	7332	8759
Axial (kN)	909	1126	1136	1617	1930	4939	2725	4774	3974	4757
Torsion (kN.m)	10	12	13	17	20	50	28	48	41	45
Bearing forces at Abutments										
Shear22 (kN)	7	8	9	11	15	36	21	37	30	37
Shear33 (kN)	265	300	332	429	416	1150	558	1051	866	819
Axial (kN)	113	127	141	181	199	513	277	493	410	465
Moment22 (Kn.m)	40	45	49	64	62	171	83	156	129	97
Moment33 (kN.m)	6	7	8	10	14	32	19	33	28	122
Torsion (kN.m)	0.07	0.07	0.07	0.11	0.12	0.33	0.18	0.32	0.26	0.30
Bearing forces at Bent 2										
Shear22 (kN)	460	495	498	711	970	2292	1368	2317	1997	2308
Shear33 (kN)	471	569	574	817	916	2398	1277	2279	1892	2145
Axial (kN)	398	485	588	696	804	2085	1126	2000	1659	1929
Moment22 (kN.m)	239	289	299	414	465	1217	648	1157	960	1089
Moment33 (kN.m)	138	148	172	213	291	687	410	695	599	692
Torsion (kN.m)	0.3	0.4	0.37	0.5	0.59	1.5	0.83	1.37	1.22	1.41
Bearing forces at Bent 3										
Shear22 (kN)	27	28	34	41	57	138	81	139	118	138
Shear33 (kN)	255	318	319	456	542	1390	765	1340	1116	1334
Axial (kN)	321	396	402	569	682	1740	878	1689	1404	1681
Moment22 (kN.m)	65	81	81	116	137	352	194	340	283	338
Moment33 (kN.m)	203	228	262	328	445	1063	629	1172	917	1170
Torsion (kN.m)	1.0	1.3	1.4	1.9	2.3	5.9	3.2	5.6	4.7	5.6

CHAPTER 3 : COMPARATIVE COST ANALYSIS

3.1 Introduction

A comparative analysis of response quantities presented in the previous chapter shows that the response quantities using NYCDOT spectra are significantly amplified with respect to those using AASHTO LRFD spectra, particularly for soil site C. A further comparative cost analysis using the response quantities presented in the previous chapter has been carried out to quantitatively assess the relative cost of using NYCDOT spectra over AASHTO LRFD spectra.

3.2 Comparative Cost Analysis Approach

Using the model discussed in the previous chapter, estimation of the increased design and construction cost associated with the NYCDOT Seismic Design Guidelines Report with respect to the AASHTO LRFD Bridge Design Specifications have been carried out. Based on bridge drawings, Pier 1 is modeled as fixed pier and Pier 2 is modeled as an expansion pier in the Midas 3D model. Using the finite element of the bridge, multi-mode elastic response spectrum analysis has been used to carry out seismic analysis. Results of the analyses performed in two orthogonal directions (longitudinal and transverse) have been combined using Square Root of Sum of Squares (SRSS) method, in lieu of the 30% combination rule referenced in the AASHTO LRFD Bride Design Specifications.

The forces from the analysis have been applied to the columns and footings to determine the required modifications, if any, to accommodate Site Classes A, B, C, D, and E for 500, 1000, 1500 and 2500-Yr return periods. The columns have been analyzed using spColumn software (www.structurepoint.org) with an assumed minimum reinforcing steel ratio of 1%. For the pile foundations, an Excel spreadsheet was used to compute the axial pile loads. The bridge plans indicate that the design capacity of the piles is 1390 kN, which is approximately 9 ksi for the size of pile indicated on the plans. This capacity was assumed to be at the service load. In order to determine the ultimate pile load capacities with a safety factor of 2, it was assumed that the construction control of the bridge will include an approved static load test. Based on these assumptions, it was estimated that the ultimate total pile load capacity (compression) was 2780 kN; the ultimate frictional pile load capacity was 2085 kN, (75% of total capacity is assumed to be frictional capacity); and the ultimate tension pile load capacity was 695kN (one-third of the ultimate frictional capacity). In cases where the analysis indicated that the pile capacities were exceeded, the pile cap was revised so that they were within the design capacity.

It is noted that the foundations for the bridge are pile supported. It is likely that if this particular bridge was designed for the different soil conditions, different foundation systems might have been used to optimize costs. For example, a spread footing might be a more economical and suitable foundation type for Site Class A, hard rock, Site Class B, rock, or Site Class C, very dense soil and soft rock. It is also noted that the design spectra was applied at the foundation level, and that a more rigorous analysis of a bridge with a pile supported foundation would likely include the piles down to their point of fixity and apply the appropriate spectra at the point of pile fixity.

3.3 Comparative Costs

Design Costs

The design costs would be similar for all cases, since the level of analysis will be the same and the only difference will be the input spectra.

Construction Costs

There were no modifications required for Pier 2 (an expansion pier) for all load cases. Table 3.1 below shows the construction cost increase for the response spectra for different site classes. This cost increase has two components: Extra column rebar cost in cases where required reinforcement ratio exceeds 1% and extra concrete and rebar cost in cases where piles are rearranged and footing size is increased. Zero cost increase stands for the case where 1% column rebar ratio and the footing size and pile layout, as shown in drawings, are sufficient. Unit prices for footing concrete and rebar are taken from NYSDOT Weighted Average Item Price (WAIP) Report, January 1, 2012 to December 31, 2012.

It is observed from Table 3.1 that there was no construction cost increase for soil sites A and B for earthquakes of all return period (except for the NYCDOT 2500 where there is additional cost of $16,800 with respect to existing design). For soil sites C, D and E, construction costs for the bridge designed using NYCDOT spectra are generally higher than those using AASHTO spectra. However, it is observed that the maximum increase of $198,000 is for Soil Class E using NYCDOT 2500 Yr spectra. This increase, considering the construction cost of the bridge, is very insignificant. Hence, overall, construction costs increases because of the proposed NYCDOT seismic design guidelines are generally minimal.

Table 3.1: Added construction costs

Soil Type	A	B	C	D	E
NYCDOT 500	0	0	$24,000	$24,000	$26,000
NYCDOT 1000	0	0	$69,000	$69,000	$71,000
AASHTO 1000	0	0	$9,000	$5,000	$71,000
NYCDOT 1500	0	0	$119,000	$121,000	$123,000
NYCDOT 2500	0	$16,800	$196,000	$196,000	$198,000
USGS 2500	0	0	$43,000	$93,000	$161,000

For a comparative evaluation of cost increases because of the adoption of NYCDOT seismic guidelines, cost increases in Table 3.1 have been expressed in terms of ratios of NYCDOT 1000/AASHTO 1000, NYCDOT 1500/AASHTO 1000, NYCDOT 1500/NYCDOT 1000 and NYCDOT 2500/USGS 2500. For example, ratio of cost increase for NYCDOT 1000/AASHTO 1000 for soil type C has been calculated by dividing $69,000 by $9,000 for the soil type C in Table 3.2.

For Soil Type E, it is observed that the ratio of cost increases because of NYCDOT spectra with respect to those of AASHTO (or USGS) spectra are between 1 to 2. In particular, there is no cost increase with respect to AASHTO 1000 Yr return period spectra, if NYCDOT 1000 Yr spectra are adopted for noncritical bridges and there is a cost increase of 73% if NYCDOT 1500 Yr spectra is adopted for such bridges. For critical bridges on soil sites C and D, cost increases because of using NYCDOT 2500 instead of USGS 2500 are 4.6 and 2.10 times, respectively. For noncritical bridges on soil sites C and D, ratio of cost increases because of adopting NYCDOT 1500 instead of NYCDOT 1000 is around 1.7.

Table 3.2: Comparative added construction costs

Soil Type	A	B	C	D	E
NYCDOT 1000/AASHTO 1000	-	-	7.67	13.8	1.0
NYCDOT 1500/AASHTO 1000	-	-	13.22	24.20	1.73
NYCDOT 1500 / NYCDOT 1000	-	-	1.72	1.75	1.73
NYCDOT 2500/USGS 2500	-	-	4.60	2.10	1.23

It should be mentioned that the cost increases described above are based on the analysis of a specific bridge with minimum foundation and section (concrete, rebars) details. It is possible to optimize the seismic design of a bridge on a particular soil type such that the cost increases with respect to AASHTO response spectra are minimal. For example, improving seismic resistance of a new or existing bridge through appropriate bearings can be sufficient for 1000 Yr to 2500-Yr earthquakes. This is most likely going to be the case during design or retrofit of bridges. Hence, additional construction costs of adopting NYCDOT seismic guidelines can be considered minimal with respect to construction or retrofit costs of the bridge.

CHAPTER 4 : DEVELOPMENT OF NYSDOT BLUE PAGES

4.1 Introduction

New York State Department of Transportation has adopted AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for LRFD Seismic Bridge Design and needs to implement the guidelines for the downstate New York state region through New York State's blue pages. This section presents blue pages developed to implement the guidelines on the seismic design of bridges in the downstate region.

4.2 Blue Pages for the Implementation of the Guidelines

Blue pages for the seismic design guidelines for the downstate region are presented in pages 66 to 87 of this report.

TABLE OF CONTENTS

The following is a list of articles that have been created as "Load and Resistance Factor Design (LRFD) Blue Page" and are incorporated into the NYSDOT LRFD Bridge Design Specifications – for AASHTO LRFD Bridge Design Specification – Fifth Edition 2010 with 2010 Interim Revisions.:

3.10 EARTHQUAKE EFFECTS: EQ

3.10.1 General

Delete the first paragraph of Article 3.10.1 and replace it with the following:

Bridge Operational Categories are defined in Article 3.10.5. Bridges defined as "Other" shall be designed to have a low probability of collapse but may suffer significant damage and disruption to service when subject to earthquake ground motions that have a 7 percent probability of exceeding in 75 years. Partial or complete replacement may be required. Higher levels of performance may be used with the authorization of the bridge owner.

Add the following after the last paragraph:

For bridges in New York City, Rockland County, Nassau County and Westchester County (defined as Downstate Zone), in addition to seismic analysis requirements of Section 3.10 of LRFD Bridge Design Specification, the requirement of Appendix A including Article A3.10 (Appendix to Article 3.10) and Article A4.7 (Appendix to Article 4.7) shall also be followed. For Seismic Analysis and Design Procedures, all remaining sections of LRFD Bridge Design Specifications along with NYS Blue Pages shall be followed.

Appendix A is based on the 2014 NYCDOT Seismic Design Guidelines for Bridges in Downstate Region referred to as "NYCDOT SDGBDR 2014". This study sets forth different seismic performance and design requirements for this area of the state. The major differences are in the design response spectrum, design return period and site classification definitions.

The 2014 Guidelines constitute a significant improvement over the 1998 Guidelines that they will replace. In addition to providing considerably more detail and guidance to the engineers on aspects such as soil liquefaction, spatial variation and site-specific studies, the proposed classification of soil sites reflects much better the specific geotechnical conditions of NYC. The 1998 Guidelines were essentially an adaptation of national site provisions that were necessarily very general rather than specific. The conditions in NYC, where a sharp contrast between soil and rock is often present, and where the depth to rock is often less than 100 ft, offered the possibility of introducing these parameters in a practical way. This possibility has been realized in the Guidelines with the help of extensive analyses. The generic seismic design spectra for bridges included in the Guidelines now depend on depth to rock, class of rock under the soil and on the soil class.

3.10 EARTHQUAKE EFFECTS: EQ

3.10.2.1 General Procedure

Delete the fourth paragraph of the Commentary to Article 3.10.2.1 and replace it with the following:

Values for the peak ground coefficient (PGA) and the spectral coefficients (S_s and S_l) for a return period of about 1000 years (design earthquake) are also available on the *USGS 2007 Seismic Parameters CD*, which is included with this book. Values for the ground coefficient (PGA) and the spectral coefficients (S_s and S_l) for a return period of about 2500 years are available at the USGS web site. Coefficients are given by the longitude and latitude of the bridge site, or by the zip code for the site.

3.10 EARTHQUAKE EFFECTS: EQ

3.10.5 Operational Classification

Add the following after the last paragraph:

SEISMIC PERFORMANCE CRITERIA

Critical Bridge: A Critical Bridge must provide immediate access after the lower level (functional) event and limited access after the upper level (safety) event and continue to function as a part of the lifeline, social/survival network and serve as an important link for civil defense, police, fire department and/or public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge that crosses a critical route whose collapse would block the critical route should also be classified as critical, if there is no readily accessible detour around the site such as at an interchange on a limited access highway.

It is expected that relatively few bridges will be classified as critical. Critical bridges would generally be limited to those on life safety routes in an urban area or on the approaches to an urban area. Critical Bridges would also be located on routes to a defense facility that has limited access. Bridges on limited access highways in rural areas would generally not be classified as critical unless they are major structures. Designation of a bridge as critical is at the discretion of the Regional Director. The designation of a Critical bridge is to be documented in the design report and included in the Site Data Package.

Critical bridges shall be analyzed for two earthquake hazard design levels: a lower level event (functional evaluation/design level) having a 7% probability of being exceeded in 75 years (1000 years Return Period) and an upper level event (safety evaluation/design level) having a 2% probability of being exceeded in 50 years (2500 year Return Period). Critical bridges will have a site specific analysis unless waived by the DCES. In the case of long span bridges, the effects of spatial variation on the seismic ground motions must also be considered.

Critical Bridges shall survive the upper level event (2500 years Return Period) with repairable damage (see definition of damage levels). Traffic access following this event may be limited; within 48 hours for emergency/defense vehicles and within months for general traffic. After the lower level event (1000 years (Return Period) the bridge shall suffer only minimal damage (see definition of damage levels). Access after this event shall be immediate to all traffic with an allowance of a few hours for inspection.

Essential Bridge: An Essential Bridge must provide at least limited access after the one hazard level evaluation/design earthquake event and serve as an important link for civil defense, police, fire department and/or public disaster situation after the event, providing a continuous route. A bridge that crosses an essential route whose collapse would block the essential route should also be classified as essential, if there is no readily accessible detour around the site such as at an interchange on a limited access highway.

Essential bridges should include those on interstate highways and others of importance as designated by the Regional Director. The designation of an Essential Bridge is to be documented in the design report and included in the Site Data Package.

Essential bridges shall be analyzed for a single earthquake hazard design level event having a 7% probability of being exceeded in 75 years (1000 years Return Period). Essential bridges shall survive the design event with repairable damage. (see definition of damage levels) Access following the seismic event may be limited: one or two lanes shall be available within 3 days for emergency vehicles, full service within months.

Other Bridges: All bridges not classified as Critical or Essential shall be classified as Other Bridges. Other bridges shall be analyzed for a single earthquake hazard design level event having a 7% probability of being exceeded in 75 years (1000 years Return Period). Other bridges may suffer significant damage (see definition of damage levels) although collapse shall not occur. The designation of Other Bridge is to be documented in the design report and included in the Site Data Package.

Damage Levels – Definitions

- ***Minimal Damage:*** The Bridge should essentially behave elastically during the earthquake, although minor inelastic response could take place. Post earthquake damage should be limited to narrow flexural cracking in concrete and masonry elements. There should be no permanent deformations to structural members. Only minor damage or permanent deformations to non-structural members should take place.
- ***Repairable Damage:*** The extent of damage should be limited so that the structure can be restored to its pre-earthquake condition without replacement of structural members. Inelastic response may occur resulting in: concrete cracking, minor cover spalling and reinforcement yielding; minor yielding of structural steel members; some damage to secondary members and non-structural components; some damage to masonry. Repair should not require complete closure of the bridge. Permanent offsets should be small and there should be no collapse.
- ***Significant Damage:*** There is no collapse, but permanent offsets may occur. Extensive cracking, major spalling of concrete and reinforcement yielding, cracking of deck slab at the shear studs, may force closure for repair. Similar consequences could result from yielding or local buckling of steel members. There could be yielding of member connections, fracture of limited number of bolts/rivets, serious damage to secondary structural members and non-structural components, as well as to masonry. In sites with significant ground lateral spreading due to liquefaction, large inelastic deformations might be induced to piles. Liquefaction could also result in excessive differential settlements. Partial or complete replacement may be required in some cases.

Insert the following articles in between Article 3.10.10 and Article 3.11

3.10.11 CRITERIA FOR SEISMIC RETROFITTING OF BRIDGES PROGRAMMED FOR REHABILITATION

3.10.11.1 GENERAL

Existing bridges, programmed for rehabilitation shall be evaluated for seismic vulnerability. The evaluation should assess options and costs of seismic retrofit measures, necessary to eliminate or mitigate such failure vulnerability.

For the evaluation and upgrading the seismic resistance of existing highway bridges, FHWA's "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges" (January 2006, Publication No. FHWA-HRT-06-032) is referenced. The provisions of this manual shall apply to highway bridges of conventional steel and concrete construction with spans not exceeding 500 feet. The Owner shall specify and/or approve appropriate provisions for nonconventional construction and for bridges with spans exceeding 500 feet.

C 3.10.11.1

Conventional bridges include those with slab, beam, box girder, or truss superstructures, and single or multiple-column piers, wall-type piers, or pile-bent substructures. In addition, conventional bridges are founded on shallow or piled footings or shafts. Nonconventional bridges include suspension bridges and bridges with truss towers or hollow piers for substructures and arch bridges.

3.10.11.2 SEISMIC PERFORMANCE CRITERIA

The strengthening of existing bridges to the same earthquake resistance as currently required of new bridges is not always practical or cost effective. It is therefore, the intent of these criteria to upgrade elements to be retrofitted to "new bridge" seismic criteria where feasible.

The following changes to the FHWA's Seismic Retrofitting Manual shall be applied:

- Bridges shall be evaluated only for the upper level earthquake ground motions with a 7 percent probability of exceedance in 75 years corresponding to a return period of about 1000 years.
- Critical and Essential bridges shall be combined in one category and will be evaluated under 'Essential Bridge' as defined in the Retrofit Manual.
- "NORMAL" bridges shall be considered the same as "STANDARD" bridges as defined in the Retrofit Manual.
- Design Response Spectrum shall be constructed as per 3.10.4.1 incorporating site factors as per 3.10.3.2.
- Minimum support length requirements are to be calculated as per 4.7.4.4.

C 3.10.11.2

The intent of the retrofitting guideline is to follow the new FHWA Seismic Retrofitting Manual incorporating the latest AASHTO design response spectrum and minimum design requirements.

3.10.11.6 BRIDGES WITH SPECIAL CONDITIONS

Certain bridge types or bridge details are particularly sensitive to seismic forces. When such conditions are identified on bridges programmed for rehabilitation, it would be prudent to consider additional retrofit measures or structure replacement. The location and "importance category" of the structure should be key considerations in this decision.

The following conditions are particularly sensitive to seismic forces:

- Single or individual column pier supports.
- High, slender pier columns (when slenderness ratio exceeds 60).
- Large skews, generally in excess of 45˚, with substandard support lengths.
- Severe curvature, where the subtended arc angle exceeds 75˚.
- Unusual geometry causing portions of the structure to be significantly different in stiffness than the main structure, or which results in unusual support of framing details.
- Hinges or seated connections in suspended superstructures.
- Non-redundant load path superstructures.
- Bridges with weak substructure subject to Liquefaction.

Bridges incorporating any of the above conditions should be evaluated for seismic vulnerability (capacity-demand ratios), as outlined in the current version of FHWA's "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges".

It should be noted that only a few Seismic Performance Zone '1' or Seismic Design Category 'A' bridges, incorporating the conditions noted above, will be candidates for extensive analysis or retrofit actions. However, the presence of these conditions should be acknowledged and considered in the project scoping phase.

A3.10 GUIDELINES FOR ADDITIONAL EARTHQUAKE EFFECTS FOR DOWNSTATE ZONE

Downstate Zone: The counties of Bronx, Kings, New York, Queens, Richmond, Nassau, Rockland and Westchester as shown in Figure A3.10-1.

A3.10.1 General

The ***NYCDOT SDGBDR 2014*** provides criteria for the analysis, evaluation, design and retrofit of bridges in the Downstate Zone. The objectives of the *Guidelines* are:

1. To provide minimum seismic design criteria for bridges considering the need to protect the general public by minimizing the earthquake-related risk to life.
2. To improve their capability to function during and after earthquakes.

The design earthquake ground motion levels specified herein could result in both structural and non-structural damage. For most bridge systems designed and constructed or retrofitted according to the *Guidelines,* structural damage from the design earthquake ground motion would be repairable. It is expected that the damage from the design earthquake ground motions would not be so severe as to preclude continued function of the bridge. The actual ability to accomplish these goals depends upon a number of factors including site conditions, the structural type and configuration of the bridge, construction materials, and as-built details of construction.

The following criteria identify minimum requirements for seismic design. Each bridge presents a unique set of design challenges. The designer must determine the appropriate methods and level of refinement necessary to design and analyze each bridge on a case-by-case basis. The designer must exercise judgment in the application of these criteria. Situations may arise that warrant detailed attention beyond what is provided in the Guidelines. It is the prerogative of each bridge owner to decide under which cases these *Guidelines* would need to be modified to suit specific circumstances.

The ***NYCDOT SDGBDR 2014*** includes requirements related to the following:

- Bridge Classification and Performance Criteria
- Very Hard Rock Spectra and Time History Records
- Classification of a Site as a Rock or Soil Site
- Rock Classes and Rock Generic Horizontal Design Spectra,
- Soil Site Characterization and Soil Generic Horizontal Design Spectra
- Vertical Motions and Generic Design Spectra for Rock and Soil Sites
- Site Liquefaction
- Site Specific Studies

As in the case of any rehabilitation project, judgment should be exercised in assessing options and costs of seismic retrofit measures, and to incorporate into the rehabilitation plans those retrofit measures deemed warranted for eliminating or mitigating such seismic vulnerabilities.

A3.10.2 Seismic Hazard

The seismic hazard at a bridge site shall be characterized by the acceleration response spectrum for the relevant site class. The acceleration spectrum shall be determined using either the General Procedure specified in Section A3.10.2.1 or the Site Specific Procedure specified in Section A3.10.2.2.

The Seismic Hazard for the Downstate Zone (see Article A3.10 and Figure A3.10-1) has been quantified in the form of 5% damped horizontal Uniform Hazard Spectra (UHS) for four earthquake return periods, 500 (10% probability of being exceeded in 50 years), 1000 (7% probability of being exceeded in 75 years), 1500 (3% probability of being exceeded in 50 years) and 2500 (2% probability of being exceeded in 50 years) years. The horizontal UHS are presented in Table A3.10.2-1 as coefficients corresponding to spectral accelerations in terms of g, the acceleration of gravity. The spectra in Table A3.10.2-1 represent an 85th percentile of ground motions corresponding to each one of the four return periods (median plus one standard deviation level). The motions are for Very Hard Rock (VHR) in NYC, typical of the eastern United States (US), with a shear wave velocity of at least 2.83 km/sec (approximately 9,000 ft/sec). This 2.83 km/sec shear wave velocity is an average of eastern US continental crust. UHS in the horizontal direction for softer rock conditions are introduced later in this Article. UHS in the horizontal direction for other soil site conditions are presented in Article A3.10.4.

Vertical UHS for Very Hard Rock, softer rock conditions, as well as for soil site conditions, are included in the *NYCDOT SDGBDR 2014* and shall be used according to Article A3.10.2.1.

Very Hard Rock ground motion time history records developed to match the horizontal spectral accelerations are presented in Table A3.10.2-1 and corresponding vertical spectral accelerations are available, in digital form from the NYSDOT Office of Structures website, for the 500-yr, 1500-yr and 2500-yr earthquake return periods. Three sets (labeled E1, E2 and E3) of multiple support ground motion time-histories, for 500-year, 1500-year and 2500-year earthquake return periods were derived for use as inputs to seismic analyses, (as input to the bridge dynamic analyses in the time domain or as input to the soil dynamic site response analyses). When time history records corresponding to the 1000-yr earthquake return period are needed for ground response analysis, they can be obtained by matching the corresponding Very Hard Rock response spectra using available computer codes. Three sets of time-histories should be generated. Providing three sets (E1, E2 and E3) of time histories for each return period takes into account the uncertainties in the earthquake excitation and variations in the non-linear response of bridge components. In addition, each one of these three sets incorporates the effects of spatial variation along 21 hypothetical piers on Very Hard Rock spaced at 100 m (328 ft), and extended over a straight line having a total length of 2 km. Sets E1, E2 and E3 are to be used as the basis for spatial variation analyses of long-span bridges as required in Article A4.7.4.3.4b and described in the *NYCDOT SDGBDR 2014*.

These Very Hard Rock response spectra and time history records may be used either for the structural dynamic analysis of the bridge (design of the bridge) in the case of a bridge at a rock site, or as rock input to the soil in dynamic site response analyses. Whether used as input to the bridge analyses or as input to the soil site response analyses, these spectra and time histories shall be assumed to be located at the surface of an outcrop of Very Hard Rock (VHR).

Design Acceleration Response Spectra (5% damped) and associated acceleration time histories in the horizontal direction for Rock Classes A and B (see Section A3.10.3b), shall be obtained by one of the following approaches:

1) Modifying the corresponding available UHS spectra and records on Very Hard Rock (see Table A3.10.2-1);
2) Site-specific ground response analysis if the depth and properties of softer rock over VHR are known.

Specific definitions of Rock Class A and B are provided in Section A3.10.3b. The procedures for the modification of the Very Hard Rock motions are as follows (see Table A3.10.2-2):

- Horizontal Design Acceleration Response Spectrum and associated acceleration time histories for Rock Class A. Multiply horizontal UHS on Very Hard Rock (see Table A3.10.2-1) by a factor of 1.15; multiply the horizontal acceleration time histories on Very Hard Rock by a factor of 1.15.
- Horizontal Design Acceleration Response Spectrum and associated acceleration time histories for Rock Class B. Multiply horizontal UHS on Very Hard Rock (see Table A3.10.2-1) by a factor of 1.65; multiply the horizontal acceleration time histories on Very Hard Rock by a factor of 1.65.

The corresponding coefficients for spectra on Rock Classes A and B, already multiplied by 1.15 and 1.65, are included in Table A3.10.4-1 (1000 year), Table A3.10.4-2 (1500 year), Table A3.10.4-3 (2500 year) and Table A3.10.4-4 (500 year) return period earthquakes.

These horizontal spectra and time histories on Rock Class A or B, may be used either for the structural dynamic analysis of the bridge (design of the bridge) in the case of a bridge at a rock site, or as rock input to the soil in dynamic site response analyses. Whether used for design of the bridge or as input to the soil site response analyses, these spectra and time histories shall be assumed to be located at the surface of an outcrop of Rock Class A or B (see Figure A3.10.3a-1).

Figure A3.10-1 Downstate Zone

A3.10.2.1 General Procedure

For design values, the spectra shown in Section A3.10.4 based on Site Classification of Section A3.10.3 shall be used for both longitudinal and transverse direction of a structure.

A3.10.2.2 Site Specific Procedure

A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed for the following conditions.

- *When required by Article 3.10.2 of AASHTO LRFD Bridge Design Specifications.*

- All bridge categories with Site Class F
- All "Critical" bridges with Soil Sites or Rock under Soil Sites as defined in Sections A.3.10.3b and A.3.10.3c.

A site-specific procedure may be performed for any bridge site at the discretion of the Bridge Owner and shall be documented in the design report.

Site specific studies shall be performed using as input the time acceleration histories described in Article A3.10.2 and satisfying procedures and requirements provided by the *NYCDOT SDGBDR 2014*. General provisions of Article 3.10.2.2 of AASHTO LRFD Bridge Design Specifications are also applicable. The time histories are also available in digital form on NYSDOT Office of Structures website

Liquefaction design requirements of AASHTO LRFD Article 10.5.4.2 shall be used in conjunction with "Downstate Zone" provisions stated below. The provisions stated below shall govern over any requirements of AASHTO LRFD Article 10.5.4.2. Specifically, the evaluations of liquefaction potential of these Guidelines shall be performed irrespective of the Seismic Performance Zone assigned to the bridge.

Soil liquefaction assessment shall be required for those "Downstate Zone" bridges where the geotechnical investigation indicates the presence of potentially liquefiable saturated soils and following the specific requirements for Critical and Non-Critical bridges stated below. This assessment shall include:

1) possible occurrence of liquefaction,
2) effect of liquefaction on the dynamic ground motions and spectra used for design; and
3) effects of liquefaction-induced soil deformations and decreases in the stiffness and strength of the soil on the performance and capacity of the foundations and structure of the bridge.

Items (1) and (2) above shall be addressed according to the *NYCDOT SDGBDR 2014*. Item (3) may follow recommendations of the *NYCDOT SDGBDR 2014* or a more updated state of the practice as approved by the Bridge Owner. The methodology adopted for item (3) shall be documented in the design report.

The following provides specific requirements for the evaluation of liquefaction potential for Critical and Non-Critical Bridges:

Critical Bridges*:* a site-specific study to select the design spectra is required, regardless of Soil Class. Liquefaction assessment shall be performed including items (1), (2), and (3) mentioned above.

Non-Critical Bridges: The Soil Class, depth to the rock surface, and Rock Class under the soil, shall be determined in accordance with Sections A3.10.3.a and A3.10.3b. When determining the Soil Class, the potential for liquefaction should be initially ignored. If the site is classified as Soil Class F for reasons other than liquefaction (see Section A3.10.3c), then generic spectra may not be used for design, and a site-specific study needs to be conducted with liquefaction potential being evaluated as part of the site response analyses. For Non-Critical Bridges (Essential and Others) that do not include a site-specific study, an evaluation of liquefaction potential shall be performed for the 1000-year earthquake according to the *NYCDOT SDGBDR 2014*, using the generic PGA values listed in Table A3.10.2.2-1 and earthquake magnitude, M = 6.0. Optionally the owner may also require an evaluation of liquefaction potential for the 1500-year using the generic PGA values listed in Table A3.10.2.2-1 and earthquake magnitude, M = 6.0.

Possible outcomes of this liquefaction evaluation are:

- No liquefaction will occur during the design earthquake. In this case, the effects of liquefaction do not have to be considered in the design.
- Liquefaction will occur during the design earthquake. In this case, the site classification shall be switched to Soil Class F and a site-specific study shall be conducted. The potential for liquefaction shall be reevaluated using the site specific results. If liquefaction is estimated to occur in the site-specific study, site response analyses taking into account the effect of liquefaction must be performed according to the *NYCDOT SDGBDR 2014*.

If the engineer decides to conduct a site-specific study to define the design ground motions for a Non-Critical Bridge, instead of using the generic design spectra from Article A3.10.4, then the liquefaction evaluation shall be performed according to the *NYCDOT SDGBDR 2014* considering the results of the site specific study.

A3.10.3 Site Effects

Site classes specified herein shall be used in the General Procedure for Seismic Hazard (Design Response) Spectrum specified in Section A3.10.4.

A site is classified as a soil site if there is more than 10 ft of soil between the bottom of spread footing or pile cap and the rock surface; otherwise it is classified as a rock site. The specific definitions for rock surface determination are shown in Section A3.10.3a shown below. Rock sites are classified as Rock Site Class VHR, A, or B, as described in Section A3.10.3b below. Soil sites are classified as Soil Site Class C, D, E or F, as described in Section A3.10.3c.

A3.10.3a *Definitions of Rock Surface Determination*

The depth of the rock surface below the ground surface of the site, is labeled H_r in these Guidelines (see Figure A3.10.3a-1).

The "rock surface" is defined as the shallowest depth for which the following three conditions are concurrently satisfied:

1. The geotechnical material in the 20 ft immediately below the rock surface is either rock material or cemented or very dense soil with an average shear wave velocity $\overline{V}_{s20} > 2{,}500$ ft/sec (see Table A3.10.3a-1 and Figure A3.10.3a-1).

In the case of cemented or very dense material, determination that the rock surface has been reached, for the purposes of these Provisions, shall be based only on the actual measured $\overline{V}_{s20} > 2{,}500$ ft/sec in the 20 ft below the assumed rock surface. In the absence of shear wave velocity measurements, the cemented or very dense material shall be considered to be a soil layer above the rock surface. In the case of rock material, determination that the rock surface has been reached shall be based preferably on shear wave velocity measurements in the 20 ft below the assumed rock surface. However, for competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured; otherwise, it shall be assumed that the rock surface has not yet been reached and the highly fractured and weathered rock shall be considered to be a soil layer above the rock surface.

2. Values of V_s at each individual sublayer, within the 20 feet below the rock surface, shall be at least 2,000 ft/sec.

3. Based on the geotechnical characteristics at the site, the engineer can reasonably assume that the V_s profile below the rock surface will not decrease below $\overline{V}_{s20} = 2{,}500$ ft/sec, with V_s eventually increasing with depth until the very hard rock elevation ($V_s > 9{,}000$ ft/sec) is reached.

Determining that the rock surface has been reached, requires calculating $\overline{V}_{s20}$, which is defined as follows (see Figure A3.10.3a-1):

$$\overline{V}_{s20} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}} \quad \text{where} \quad \sum_{i=1}^{n} d_i = 20 \text{ ft.}$$

The symbol i refers to any one of the sublayers within the 20 feet immediately below the rock surface, with measured shear wave velocity V_{si} and thickness d_i, from 1 to n (where n is the total number of sub-layers). Each V_{si} must be at least 2,000 ft/sec.

A3.10.3b *Rock Site & Rock Under Soil Sites Classification*

1) Rock Sites

For a Rock Site, the Rock Class shall be defined as follows (see Table A3.10.3b-1):

- VHR: Very Hard Rock with measured averaged shear wave velocity, $\overline{V}_{s20} > 9{,}000$ ft/sec.
- Rock Class A: Hard Rock with measured averaged shear wave velocity, $5{,}000 < \overline{V}_{s20} \leq 9{,}000$ ft/sec.
- Rock Class B: Rock material or cemented or very dense soil with averaged shear wave velocity $2{,}500 < \overline{V}_{s20} \leq 5{,}000$ ft/sec.

In the case of cemented or very dense material (e.g., very dense glacial till), classification as Rock Class B shall be based only on shear wave velocity measurements. In the absence of shear wave velocity measurements, the cemented or very dense material shall be considered to be a soil layer above the rock surface, as defined in Section A3.10.3a.

In the case of rock material, classification as Rock Class B shall be based preferably on shear wave velocity measurements. However, for competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the highly fractured and weathered rock shall be considered to be a soil layer above the rock surface.

Assignment of either Rock Class A or VHR shall be based on either in-situ shear wave velocity measurements, or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing.

If the measured shear wave velocities indicate Rock Class B, but Rock Class A (or VHR) is found by drilling at a depth below the rock surface not greater than 40 ft, the engineer shall have the option to classify the site as Rock Site Class A (or VHR) instead of Rock Site Class B, for the purpose of selecting the generic spectra in Article A3.10.4 (see Figure A3.10.3b-1). The decision to drill deeper than 20 feet below the rock surface is optional (see Section A3.10.3a).

2) Rock Under Soil Sites

For a Soil Site, with H_r < 100 ft, the selection of design spectra in Article A3.10.4 and the selection of Peak Ground Acceleration (PGA) for liquefaction evaluation (see Article A3.10.2.2), require classification of the rock under the site as either Rock Class A/VHR or B. The same definition of $\overline{V}_{s20}$ (see Figure A3.10.3a-1), and the same ranges of $\overline{V}_{s20}$ given for Rock Sites in Section (1) above, shall be used. Classification of Rock Under Soil Sites is as follows (see Table A3.10.3b-1):

- Rock Class A/VHR: Hard to Very Hard Rock with measured shear wave velocity, $\overline{V}_{s20} >$ 5,000 ft/sec.
- Rock Class B: Rock material or cemented or very dense soil with shear wave velocity $2{,}500 < \overline{V}_{s20} \leq 5{,}000$ ft/sec.

In the case of cemented or very dense material (e.g., very dense glacial till), classification as Rock Class B shall be based only on shear wave velocity measurements. In the absence of shear wave velocity measurements, the cemented or very dense material shall be considered to be a soil layer above the rock surface.

In the case of rock material, classification as Rock Class B shall be based preferably on shear wave velocity measurements. However, for competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the highly fractured and weathered rock shall be considered a soil layer above the rock surface.

Assignment of Rock Class A/VHR shall be based on either in-situ shear wave velocity measurements, or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing.

If $H_r < 100$ ft and the measured averaged shear wave velocity $\overline{V}_{s20}$ indicates Rock Class B directly beneath the soil, but Rock Class A/VHR is found by drilling, at a depth below the rock surface not greater than 40 ft, the engineer shall have the option to specify that the soil profile is on top of Rock Class A/VHR instead of Rock Class B (see Figure A3.10.3b-1), for the purpose of selecting the generic spectral accelerations (see Article A3.10.4), and the Peak Ground Acceleration (PGA) for liquefaction evaluation (see Article A3.10.2.2). To exercise this option, drilling must continue at least 20 ft below the Rock Class A/VHR surface shown in Figure A3.10.3b-1.

A3.10.3c Soil Sites Classification

1) Soil Site Classes C,D,E,F

A soil site is defined as one where the distance between the rock surface and the bottom of spread footing or pile cap is greater than 10 ft.

Soil Site Classes shall be characterized on the basis of average soil properties. The sites of Critical Bridges shall only be classified using the soil shear wave velocity, V_s. The preferred soil property used for the Non-Critical Bridges is also the soil shear wave velocity. Other alternative soil properties that may be used for Non-critical bridges are the Standard Penetration Test (SPT) resistance, N, for cohesionless soils, N_{ch}, and the undrained shear strength, s_u, for cohesive soils. The corresponding average values V_s, $\overline{N}$, $\overline{N}_{ch}$ and $\bar{s}_u$ are specifically defined in Table A3.10.3c-1. The Soil Classes used to classify a Soil Site are defined as follows:

- **Soil Class C**: Very dense soil with shear wave velocity, $1{,}200 < \overline{V}_s \le 2{,}500$ ft/sec or with either standard blow count $(\overline{N}$ or $\overline{N}_{ch}) > 50$, or undrained shear strength $\bar{s}_u > 2{,}000$ psf. Soil above the rock surface which happens to have a $\overline{V}_s > 2{,}500$ ft/sec due to the presence of high shear wave velocity cemented or very dense soil layers, but does not meet the requirements of a rock site, as specified in Sections A3.10.3a and A3.10.3b , shall be classified as Soil Class C
- **Soil Class D**: Stiff soil with $600 \le \overline{V}_s \le 1{,}200$ ft/sec or with $15 \le (\overline{N}$ or $\overline{N}_{ch}) \le 50$ or $1{,}000 \le \bar{s}_u \le 2{,}000$ psf.
- **Soil Class E**: A soil profile with $\overline{V}_s < 600$ ft/sec, or any profile at a soil site with more than 10 feet of soft clay defined as soil with plasticity index PI > 20, water content, $w \ge 40\%$ and $\bar{s}_u < 500$ psf, or any profile with $\overline{N}$ or $\overline{N}_{ch} < 15$, or $\bar{s}_u < 1{,}000$ psf.
- **Soil Class F**: Soils requiring site-specific evaluations:
  1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. The site of a Critical Bridge will be classified as Soil Class F if it will liquefy during the 2500-year earthquake, as determined according to Article A3.10.2.2 . The site of an Essential or Other Bridge will be classified as Soil Class F if it will liquefy during the 1000-year earthquake, as determined according to Article A3.10.2.2 .
  2. Peats and/or highly organic clays (H > 10 ft of peat and/or highly organic clay, where H = total thickness of soil layers with those characteristics).
  3. Very high plasticity clays (H > 25 ft with PI > 75).
  4. Very thick soft/medium stiff clays (H > 120 ft) with $\bar{s}_u < 1{,}000$ psf.

Table A3.10.3c-1 summarizes the classification of Soil Sites.

2) *Definitions of Soil Class Parameters*

The definitions presented in this Section, apply to soil profiles where the rock surface as defined in Section A3.10.3a, locates more than 10 ft below the bottom of spread footing or pile cap. If the depth of rock surface $H_r < 100$ ft, only the properties of the soil between the ground surface and the rock surface are used. If $H_r > 100$ ft, only the properties of the soil between the ground surface and 100 ft depth are used. Therefore, definition of the Soil Class at a soil site does not require subsurface exploration deeper than 100 ft (Figure A3.10.3c-1), except in cases of very thick soft/medium stiff clays (i.e. Site Class F). Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to *n* at the bottom, where there are a total of *n* distinct layers down to the depth H_r or down to a depth of 100 feet, depending on the case. The symbol *i* refers to any one of the layers between 1 and *n*.

V_{si} is the shear wave velocity in ft/sec.

d_i is the thickness of any layer between 0 and 100 ft.

$\overline{\mathrm{V}}_s$ is:

$$\overline{\mathrm{V}}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\mathrm{V}_{si}}}$$

where $\sum_{i=1}^{n} d_i$ shall not be greater than 100 ft. If the depth to rock, H_r, is less than 100 feet, then $\sum_{i=1}^{n} d_i = H_r$.

N_i is the Standard Penetration Resistance determined in accordance with ASTM D 1586, as directly measured in the field without corrections, and shall not be taken greater than 100 blows/ft.

$\overline{N}$ is:

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

where N_i and d_i are for cohesionless soil and cohesive soil layers within a total depth not exceeding 100 ft. Rock shall not be included. If the depth to rock, H_r, is less than 100 ft, then: $\sum_{i=1}^{n} d_i = H_r$.

For cohesionless soils only, $\overline{N}_{ch}$ is:

$$\overline{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

Where $\sum_{i=1}^{m} d_i = d_s$ is the total thickness of cohesionless soil layers. Use only d_i and N_i for cohesionless soils within a total depth not exceeding 100 ft, from 1 to *m*.

s_{ui} is the undrained shear strength in psf, and shall be determined in accordance to ASTM D2166 or D 2850. It shall not exceed 5,000 psf.

$\overline{s}_u$ is,

$$\overline{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

where $d_c = \sum_{i=1}^{k} d_i$ is the total thickness of cohesive soil top layers within a total depth not exceeding 100 ft, from 1 to *k*, and $d_s + d_c \leq 100$ ft

PI is the plasticity index determined in accordance to ASTM D 4318.
w is the moisture content in percent, determined in accordance to ASTM D 2216.

3) *Steps for Classifying Soil Site as Class C, D, E or F*

The steps described below are to be taken after establishing that the site is a Soil Site rather than a Rock Site (see Sections A3.10.3a and A3.10.3b). The definitions of $\overline{V}_s$, $\overline{N}$, $\overline{N}_{ch}$ and $\overline{s}_u$, used below, are given in Part 2 of Section A3.10.3c.

Step 1: Check in Part 1 of Section A3.10.3c for the four categories of Soil Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Class F.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft where a soft clay layer is defined by: $\bar{s}_u < 500$ psf, $w \geq 40\%$, and *PI* > 20. If these criteria are satisfied, classify the Site as Class E.

Step 3: Categorize the site using one of the following three methods with $\overline{V}_s$, $\overline{N}$, $\overline{N}_{ch}$ and $\bar{s}_u$ computed in all cases as specified in Part 2 of Section A3.10.3c.:

i. $\overline{V}_s$ for the soil layers, computed from the ground surface down to a depth of H_r, with a maximum depth of 100 ft if $H_r > 100$ ft ($\overline{V}_s$ method). Classify the site as Soil Class C, even if $\overline{V}_s$ > 2,500 ft/sec, when this is due to the presence of high shear wave velocity cemented or very dense intermediate soil sub-layers above the rock surface (see Section A3.10.3a).

ii. $\overline{N}$ for the soil layers computed from the ground surface down to a depth of H_r, with a maximum depth of 100 ft if $H_r > 100$ ft ($\overline{N}$ method).

iii. $\overline{N}_{ch}$ for cohesionless soil layers (*PI* < 20), and $\bar{s}_u$ for cohesive soil layers (*PI* > 20), with both $\overline{N}_{ch}$ and $\bar{s}_u$ computed from the ground surface down to a depth of H_r with a maximum depth of 100 ft if $H_r > 100$ ft ($\bar{s}_u$ method).

V_s data are required for site classification of Critical Bridges. For Non-critical bridges, where reliable V_s data (i.e.: based on field measured V_s data) are available for the site, $\overline{V}_s$ shall be used to classify the site. If the $\overline{N}$, $\overline{N}_{ch}$ and $\bar{s}_u$ criteria, listed above, are used and two different Soil Classes are obtained for the site, both Soil Classes shall be used in Article A3.10.4 to select the soil horizontal spectra, the vertical generic spectra required according to Article A3.10.2.1 and the Peak Ground Acceleration (PGA) for liquefaction evaluation (see Article A3.10.2.2). In such a case, the envelope of both curves shall be used for design. The envelope curve is obtained from the higher spectral acceleration coefficient corresponding to different periods shown in the Spectral Coefficient Tables included in Article A3.10.4 for horizontal design spectra and the ***NYCDOT SDGBDR 2014*** for vertical design spectra.

A3.10.4 General Procedure - Seismic Hazard (Design Response) Spectrum

The 5-percent-damped generic horizontal design spectra shall be obtained from Tables A3.10.4-1 to A3.10.4-4 for Rock Sites VHR, A and B described in Section (1) below, and Tables A3.10.4-5 to A3.10.4-8 for Soil Site Class C,D,E described in Section (2) below.

1) Generic Horizontal Design Spectra for Rock Site Classes VHR, A and B

Non-Critical Bridges (Essential or Others) founded on a Rock Site, at a minimum, shall be analyzed using the 1000-yr Return Period (7% probability of being exceeded in 75 years) rock generic 5% damped spectra presented in Table A3.10.4-1. The owner also has the option to analyze these bridges using the 1500-yr Return Period (3% probability of being exceeded in 50 years) rock generic 5% damped spectra presented in Table A3.10.4-2

Critical Bridges founded on a Rock Site, shall be analyzed using the following spectra:

- 2500-yr Return Period (2% probability of being exceeded in 50 years) rock generic 5% damped spectra presented in Table A3.10.4-3
- 1000-yr Return Period (7% probability of being exceeded in 75 years) rock generic 5% damped spectra presented in Table A3.10.4-1

Optionally, the engineer may decide to conduct a site-specific study (see Article A3.10.2.2) including Soil-Structure Interaction effects for a Critical Bridge at a Rock Site, using the rock input motions (see Article A3.10.2). In that case, the seismic design will be based on the results of the site-specific study.

2) Soil Generic Horizontal Design Spectra for Soil Site Class C, D, and E

The soil generic horizontal design spectra for Non-critical and Critical Bridges are presented in Tables A3.10.4-5 (1000 yr Return Period), A3.10.4-6 (1500 yr Return Period), A3.10.4-7 (2500 yr Return Period), and A3.10.4-8 (500 yr Return Period) as a function of three parameters:

- The depth to the rock surface (H_r), as defined in Section A3.10.3a
- The Rock Class (A/VHR or B) immediately underneath the rock surface, as defined in Section A3.10.3b
- The Soil Site Class (C, D or E), as defined in Section A3.10.3c

Non-Critical Bridges (Essential or Others) founded on a Soil Site Class C, D or E, at a minimum, shall be analyzed using the 1000-yr Return Period (7% probability of being exceeded in 75 years) soil generic 5% damped horizontal spectra presented in Table A3.10.4-5. The owner also has the option to analyze these bridges using the 1500-yr Return Period (3% probability of being exceeded in 50 years) soil generic 5% damped horizontal spectra presented in Table A3.10.4-6.

If insufficient data are available to classify a soil profile under a non-critical bridge, Soil Site Class D shall be assumed. For periods less than 0.5 seconds, spectral accelerations for "Soil Class D on top of Rock Class B, $H_r < 100$" shall be used. For periods larger than 0.5 seconds, spectral accelerations for "Soil Class D on top of Deep Rock of Any Type" shall be used.

A Critical Bridge at a Soil Site shall be analyzed based on the results of a site-specific study conforming to Article A3.10.2.2. The generic horizontal spectra described above are used as references in the site-specific study.

A3.10.5 Operational Categories and Seismic Performance Criteria

A3.10.5.1 Performance Criteria and Seismic Hazard

Bridges in the New York City and surrounding areas should be designed to meet the performance criteria outlined in Table A3.10.5-1. This Table summarizes the relationship between bridge importance and performance requirements. In all cases, collapse is not permitted. More rigorous analysis as recommended herein and satisfying the minimum requirements of Article 4.7.4.3.1 shall be performed unless otherwise required by the Bridge Owner. Seismic Performance criteria given herein address the safety and functional performance of a bridge during and after an earthquake. They are defined in terms of the bridge's post seismic service level and the extent of damage.

Bridges should be classified by the agency having jurisdiction, as “critical”, “essential” or “other” meeting the following requirements:

Critical Bridge: A Critical Bridge must provide immediate access after the lower level (functional) event and limited access after the upper level (safety) event and continue to function as a part of the lifeline, social/survival network and serve as an important link for civil defense, police, fire department and/or public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge that crosses a critical route whose collapse would block the critical route should also be classified as critical, if there is no readily accessible detour around the site such as at an interchange on a limited access highway.

It is expected that relatively few bridge will be classified as critical. Critical bridges would generally be limited to those on life safety routes in an urban area or on the approaches to an urban area. Critical Bridges would also be located on routes to a defense facility that has limited access. Bridges on limited access highways in rural areas would generally not be classified as critical unless they are major structures. Designation of a bridge as critical is at the discretion of the Regional Director. The designation of a critical bridge is to be documented in the design report.

Critical bridges shall be analyzed for two earthquake hazard design levels: a lower level event (functional evaluation/design level) having a 1000 years Return Period; and an upper level event (safety evaluation/design level) having a 2500 years Return Period. For Critical bridges at Soil Sites (see Article A3.10.3), site-specific soil effects and, if necessary, soil-structure interaction must be considered. The required site-specific soils study shall comply with Article A3.10.2.2. A multimode spectral analysis must be used to establish vulnerability for either event. The seismic retrofit for the 2% in 50 years probability of exceedance event must be confirmed by either a multimodal spectral analysis augmented by non-linear static (pushover) analysis, or by nonlinear time history analysis.

For *Critical* bridges at Rock Sites (Section A3.10.3b), the generic spectra specified in Section (1) of Article A3.10.4 may be used for design; alternatively, the engineers shall have the option of conducting a site-specific study including Soil-Structure Interaction effects, using the rock input motions according to Article A3.10.2.2. In the case of long-span bridges, the effects of spatial variation on the seismic ground motions must also be considered in accordance to the requirements of Article A4.7.4.3.4b.

Critical Bridges shall survive the upper level event (2% probability of being exceeded in 50 years) with repairable damage (see definition of damage levels in Article A3.10.5.2). Traffic access following this event may be limited: within 48 hours for emergency/defense vehicles and within months for general traffic. After the lower level event (7% probability of being exceeded in 75 years), the bridge shall suffer no damage to primary structural elements and minimal damage to other components (see definition of damage levels in Article A3.10.5.2). Access after this event shall be immediate to all traffic with an allowance of a few hours for inspection.

Essential Bridge: An Essential Bridge must provide at least limited access after the one hazard level evaluation/design earthquake event and serve as an important link for civil defense, police, fire department and/or public disaster situation after the event, providing a continuous route. A bridge that crosses an essential route whose collapse would block the essential route should also be classified as essential, if there is no readily accessible detour around the site such as at an interchange on a limited access highway.

Essential bridges should include those on interstate highways and others of importance as designated by the Regional Director. The designation of an Essential Bridge is to be documented in the design report.

Essential bridges shall survive the design event with repairable damage (see definition of damage levels in Article A3.10.5.2). Access following the seismic event may be limited: one or two lanes shall be available within 72 hours for emergency vehicles, and full service within months.

Other Bridges: Other bridges are those not classified as Critical or Essential. Bridges classified as Other may suffer significant damage (see definition of damage levels in Article A3.10.5.2) although collapse shall not occur**.** It is recommended that the damage be in visible and pre-selected areas unless prohibitive by structure type or cost. In this case, a feasible solution shall be demonstrated and documented in the design report. Extended closures are acceptable.

Essential and Other bridges (also labeled together *Non-critical* Bridges), at a minimum, shall be analyzed for a single earthquake hazard level having 7% probability of being exceeded in 75 years (1000 years Return Period). The owner also has the option to analyze these bridges for a single earthquake hazard level having 3% probability of being exceeded in 50 years (1500 years Return Period). The seismic ground motions specified in these guidelines are defined in terms of generic design spectra described in Article A3.10.4. For a Rock Site, the generic design spectra are specified in Section (1) of Article A3.10.4. For a Soil Site, the generic design spectra are specified in Section (2) of Article A3.10.4. If the Essential and Other bridge is on a Soil Site classified as Soil Class C, D, or E (see Section A3.10.3c), the Engineer shall have the option of conducting a site specific study accounting for the local subsurface conditions, to produce site-specific acceleration ground surface spectra and corresponding time histories, for use in the design instead of the generic spectra. A site-specific study accounting for the local subsurface site conditions shall be required for a Soil Site classified as Soil Class F (see Section A3.10.3c). For an Essential bridge at a Rock Site, the Engineer shall have the option of conducting a site-specific study including the effects of Soil-Structure Interaction. Any required or optional site-specific study at a Soil or Rock Site shall comply with Article A3.10.2.2. In the case of long-span bridges, the effects of spatial variation on the seismic ground motions must be considered in accordance with Articles A3.10.2 and A4.7.3b.

A3.10.5.2 Damage Levels – Definitions

Bridge component detailing or retrofit shall be such that the damage caused by an earthquake would be in a controlled pattern in order to allow for a desired post event service level.

- Minimal Damage: The Bridge should essentially behave elastically during the earthquake, although minor inelastic response could take place. Postearthquake damage should be limited to narrow flexural cracking in concrete and masonry elements. There should be no permanent deformations to structural members. Only minor damage or permanent deformations to non-structural members should take place.

- Repairable Damage: The extent of damage should be limited so that the structure can be restored to its pre-earthquake condition without replacement of structural members. Inelastic response may occur resulting in: concrete cracking, minor cover spalling and reinforcement yielding; minor yielding of structural steel members; some damage to secondary members and non-structural components; some damage to masonry. Repair should not require complete closure of the bridge. Permanent offsets should be small and there should be no collapse.
- Significant Damage: There should be no collapse, but permanent offsets may occur. Extensive cracking, major spalling of concrete and reinforcement yielding, cracking of deck slab at the shear studs, may force closure for repair. Similar consequences could result from yielding or local buckling of steel members. There could be yielding of member connections, fracture of limited number of bolts/rivets, serious damage to secondary structural members and non-structural components, as well as to masonry. In sites with significant ground lateral spreading due to liquefaction, large inelastic deformations might be induced to piles. Liquefaction could also result in excessive differential settlements. Partial or complete replacement may be required in some cases.

A3.10.6 Seismic Performance Zones

Delete Article 3.10.6 and replace it with the following:

Each bridge shall be assigned to one of the three Seismic Performance Zones (2, 3 or 4), determined by the One Second Period Spectral Acceleration, S_{D1}, of the horizontal design spectrum, in accordance with Table 3.10.6-1 NYC-Seismic Performance Zones below (Seismic Performance Zone 1 is not applicable to NYC):

Table 3.10.6-1 NYC – Seismic Performance Zones

Acceleration Coefficient, S_{D1}	Seismic Performance Zone
	Seismic Performance Zone 1 Not applicable to NYC
$S_{D1} \leq 0.30g$	2
$0.30g < S_{D1} \leq 0.50g$	3
$0.50g < S_{D1}$	4

The One Second Period Acceleration S_{D1} shall be obtained as the spectral acceleration at the period T = 1.0 second. The horizontal design spectra of Critical Bridges shall be obtained from site-specific study, except for all rock sites (see section A3.10.3b) where site specific study is optional. The horizontal design spectra of Non-Critical Bridges may use the generic ones specified in Article A3.10.4 depending on the site class.

A4.7.4.3.4b Acceleration Time Histories

Delete Article 4.7.4.3.4b and replace it with the following:

When conducting Time History Analyses of a bridge, at least three acceleration time histories, compatible with the horizontal and vertical design spectra selected shall be used for each of the three orthogonal components of the design seismic motions (two horizontals and one vertical). All three orthogonal components shall be input simultaneously. The selection of these input acceleration time histories and actual number of analyses to be conducted, shall be such that the response of the bridge accounts conservatively for:

- Effect of uncertainty in the earthquake excitation, which may be especially significant for the non-linear response of bridge components. At least three site-response analyses shall be conducted; using three different input excitations (E1, E2 and E3, see Article A3.10.2).
- Effect of uncertainty in the soil properties (which requires at least three site-response analyses, V_s best estimate ±20% (see Article A3.10.2.2).
- When applicable, effect of unknown depth to the rock surface, H_r, which requires at least two site-response analyses (see Section A3.10.3a).
- When applicable, the effects of liquefaction as described below.
- When applicable, the effects of spatial variation as described below.

All effects mentioned above shall be treated according to the ***NYCDOT SDGBDR 2014.*** If needed for time-history analysis of the bridge, in addition to the horizontal time histories calculated in the dynamic site-response analyses, it may also be necessary to generate acceleration time histories by matching the selected design horizontal spectrum on soil. This might result in particular from the requirement to comply with the two thirds rule, or due to the need to envelope response spectra corresponding to non-liquefied and liquefied soil configurations. In such cases, horizontal acceleration time histories may be obtained from matching the final design soil horizontal spectrum, using appropriate commercially available software as approved by the Owner. If needed for time-history analyses of the bridge, acceleration time histories may be obtained from matching the design soil vertical spectrum, using appropriate commercially available software as approved by the Owner.

The vertical design spectrum for the design earthquake will be obtained by multiplying the site-specific final design soil horizontal spectrum and PGA by the appropriate period dependant V/H ratios tabulated in the ***NYCDOT SDGBDR 2014.***

If it is determined that liquefaction occurs (see Article A3.10.2.2), the bridge site shall be analyzed for two configurations: (i) *non-liquefied configuration*, where the site is analyzed assuming no pore pressure buildup and no liquefaction; and (ii) *liquefied configuration*, where the site is reanalyzed assuming that liquefaction occurs in the liquefiable soil layers.

The ground motions, spectra and corresponding time histories, calculated with both non-liquefied and liquefied configurations, may be considered conservatively when developing the design ground motions, by enveloping the spectra calculated with both configurations. This simpler conservative option shall be considered carefully as it may have significant consequences on the design of the bridge and shall be approved by the Owner. If needed for time-history analysis of the bridge, in addition to the time histories calculated in the site response analyses, it may also be necessary to generate time histories from matching the final design horizontal spectrum on soil as applicable in this Article and referenced in the NYCDOT SDGBDR 2014.

When establishing the rock motions for a long-span bridge, be it Critical, Essential or Other, the spatial variation of ground motions along the length of the bridge shall be considered. Three sets (E1, E2 and E3) of time history records for each return period are available in digital form from NYSDOT Office of Structures website (see Article A3.10.2). Each set incorporates the effects of spatial variation of the seismic waves travelling though the rock medium, along 21 hypothetical piers/stations on Very Hard Rock spaced at 100 m (328 ft), and extended over a straight line having a total length of 2 km.

The engineer shall establish the ground motions for any long-span bridge length (up to 2 kilometers), by selecting an appropriate subset of piers/stations time histories whose locations match those of the bridge foundations. It is recommended to include pier/station #11 (the reference one) in the subset of ground motions selected.

In site-specific analyses with non-uniform site foundation conditions, the local rock and soil conditions should be incorporated pier by pier by appropriate modification of the Very Hard Rock horizontal and vertical time histories, as applicable in this Article and referenced in the ***NYCDOT SDGBDR 2014.***

5.10 DETAILS OF REINFORCEMENT

5.10.11 Provisions for Seismic Design

5.10.11.3 Change the title to the following:

> "Seismic Zones 1 and 2"

5.10.11.3 Replace the 1st paragraph with the following:

> "The requirements of 5.10.11.4 shall be taken to apply to bridges in Seismic Zones 1 and 2, except that the area of longitudinal reinforcement shall not be less than 0.01 or more than **0.04** times the gross cross-section area, A_g."
>
> For the minimum number of longitudinal reinforcing bars in a column of Seismic Zone 1 bridges, reduced effective area approach as explained in Article 5.7.4.2 shall be used.

CHAPTER 5 : CONCLUSIONS AND RECOMMENDATIONS

The main objective of this project has been to carry out an independent and objective assessment of the NYCDOT Seismic Design Guidelines Report (September 2008) proposed for use in the downstate zone of the New York State. This guidelines report has some key differences with the current AASHTO LRFD Bridge Design Specifications. As a result of the peer review, several significant changes have been proposed and incorporated in the NYCDOT seismic design guidelines report. The revised report is titled as "Seismic Design Guidelines for Bridges in Downstate Region, July 2014". Some of the significant changes incorporated in the 2008 Guidelines report are (i) 1000-Yr instead of 500-Yr earthquake as the lower level earthquake for the design of critical bridges, (ii) Generation of 1000-Yr response spectra for the Downstate Region, (iii) 1000-Yr earthquake for the design of noncritical bridges while giving the owners the option of using 1500-Yr earthquake also, (iv) Disallowance of waiver for the site-specific analysis, (vi) Use of shear wave velocity for seismic site characterization and design, particularly for critical bridges, (vi) Determination of seismic design category based on SD_1, (vi) Site classification based on site-specific data, especially for critical bridges and (vi) Response spectrum analysis using only two horizontal components (i.e., vertical component shouldn't be included in the response spectrum analysis; time history analysis needs to be done when the response in vertical direction is required).

While a majority of issues identified by the research team have been resolved, revisions of site factors for Rock Class A and Rock Class B based on more representative rock profiles in the downstate region couldn't be resolved because of the work being out of scope. The research team recommends NYCDOT/NYSDOT to carry out this work to ensure safety and economy of seismic design of bridges in the downstate region.

A comparative cost analysis of NYCDOT and AASHTO LRFD (or USGS in case of 2500-Yr earthquake) shows that although response quantities using NYCDOT spectra over those using AASHTO LRFD spectra are amplified, particularly for soil site C, relative increase in cost (both design and construction) of using the NYCDOT spectra is not very significant as compared to the total cost of the bridge construction or rehabilitation.

Detailed NYSDOT blue pages for the implementation of the seismic design guidelines for bridges in the downstate region needs to be adopted by NYSDOT to use the NYCDOT seismic design Guidelines in conjunction with the AASHTO Bridge Seismic Design Guide Specifications.

REFERENCES

1. 2003 Edition NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, prepared by the New Building Seismic Safety Council for the Federal Emergency Management Agency, Washington D.C.
2. 2010 Edition NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, prepared by the New Building Seismic Safety Council for the Federal Emergency Management Agency, Washington D.C.
3. AASHTO (2007). AASHTO LRFD Bridge Design Specifications, 4th Edition.
4. AASHTO (2009). Guide Specifications for LRFD Seismic Bridge Design, 1st Edition.
5. Baker JW and Cornell CA (2006). Spectral shape, epsilon and record selection. Earthquake Engineering and Structural Dynamics, 35(9): 1077-1095.
6. Baskerville CA (1992). Bedrock and Engineering Geologic Maps of Bronx County and Parts of New York and Queens Counties, New York. USGS Misc. Invest. Series Map I-2003, 1:24,000.
7. Baskerville CA (1994). Bedrock and Engineering Geologic Maps of New York County and Parts of Kings and Queens Counties, New York and parts of Bergen and Hudson Counties, New Jersey. USGS Misc. Invest. Series Map I-2306, 1:24, 0000.
8. Baturay, M.B. and Stewart, J.P. (2003). Uncertainty and bias in ground motion estimates from ground response analyses. Bull. Seism. Soc. Am., 93 (5), 2025-2042.
9. Bazzurro P, Cornell CA (2004). Nonlinear soil-site effects in probabilistic seismic hazard analysis. Bulletin of the Seismological Society of America, 94: 2110-2123.
10. Bazzurro, P. and Cornell, C.A. (2004). Nonlinear soil-site effects in probabilistic seismic-hazard analysis, Bull. Seism. Soc. Am., 94, 2110 -2123.
11. Bonilla LF, Archuleta RJ, Lavallée D (2005). Hysteretic and dilatant behavior of cohesionless soils and their effects on nonlinear site response: Field data observations and modeling. Bulletin of the Seismological Society of America, 95(6): 2373-2395.
12. Boulanger RW, Idriss IM (2006). Liquefaction Susceptibility Criteria for Silts and Clays. ASCE Journal of Geotechnical and Geoenvironmental Engineering. 132(11): 1413-1426.
13. Bozorgnia Y, Campbell KW (2004). The vertical-to-horizontal response spectral ratio and tentative procedures for developing simplified V/H and vertical design spectra. Journal of Earthquake Engineering, 8(2): 175-207.
14. Brandenberg SJ, Bellana N, Shantz T (2010). Shear wave velocity as function of standard penetration test resistance and vertical effective stress at California bridge sites. Soil Dynamics and Earthquake Engineering, 30: 1026-1035.
15. Bray JD and Sancio RB (2006). Assessment of the liquefaction susceptibility of fine- grained soils. ASCE Journal of geotechnical and geoenvironmental engineering, 132(9): 1165–1177.

16. Campbell KW (2009). Estimates of Shear-Wave Q and κ0 for unconsolidated and semiconsolidated sediments in Eastern North America. Bulletin of the Seismological Society of America, 99(4): 2365-2392.

17. Campbell KW, Bozorgnia Y (2003). Updated near-source ground motion (attenuation) relations for the horizontal and vertical components of peak ground acceleration and acceleration response spectra. Bulletin of the Seismological Society of America 93: 314-331.

18. Cetin KO, Ozan C (2009). CPT-Based Probabilistic Soil Characterization and Classification. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 135(1): 84-107.

19. Cetin KO, Seed RB, Der Kiureghian A, Tokimatsu K, Harder LF, Kayen RE, Moss RES (2004). Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 130(12):1314–1340.

20. Choi Y, Stewart JP (2005). Nonlinear site amplification as function of 30 m shear wave velocity. Earthquake Spectra, 21(1): 1-30.

21. Chu DB, Stewart JP, Boulanger RW, Lin PS (2008). Cyclic softening of low-plasticity clay and its effect on seismic foundation performance. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 134(11):1595–1608.

22. Collins PA, Orio C, Smiriglio S (2002). Geothermal Heat Pump Manual. NYC Department of Design & Construction.

23. Darendeli M (2001). Development of a new family of normalized modulus reduction and material damping curves. Ph.D. Thesis, Department of Civil Engineering, University of Texas, Austin.

24. Electric Power Research Institute (EPRI) (1993). Guidelines for determining design basis ground motions. Volume 1-5: Qualification of seismic source effects: EPRI Report TR-102293.

25. FHWA (2006). Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges.

26. Goulet, C.A. and Stewart, J.P. (2009). Pitfalls of deterministic application of nonlinear site factors in probabilistic assessment of ground motions. Earthquake Spectra, 25 (3), 541-555.

27. Idriss IM, Boulanger RW (2004). Semi-empirical procedures for evaluating liquefaction potential during earthquakes. in Proceedings of the 11th International Conference on Soil Dynamics and Earthquake Engineering, and 3rd International Conference on Earthquake Geotechnical Engineering, Vol. 1, pp 32-56. Stallion Press, Hawthorne, California.

28. Idriss IM, Boulanger RW (2006). Semi-empirical procedures for evaluating liquefaction potential during earthquakes. Soil Dynamic and Earthquake Engineering, 26: 115-130.

29. Idriss IM, Boulanger RW (2008). Soil liquefaction during earthquakes. Monograph MNO-12. Earthquake Engineering Research Institute, Oakland, California.

30. Iwasaki T, Tatsuoka F, Takagi Y (1978). Shear modulus of sands under torsional shear loading. Soils and Foundations, 18(1): 39-56.

31. Juang CH, Fang SY, Khor EH (2006). First-order reliability method for probabilistic liquefaction triggering analysis using CPT. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 132(3): 337–350.

32. Juang CH, Jiang T, Andrus RD (2002). Assessing probability based methods for liquefaction potential evaluation. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 128(7): 580–589.

33. Kishida T, Wehling TM, Boulanger RW, Driller MW, Stokoe KH II (2009). Dynamic properties of highly organic soils from Montezuma Slough and Clifton Court. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 135(4): 525-532.

34. Liu H, Song E (2006). Working mechanism of cutoff walls in reducing uplift of large underground structures induced by soil liquefaction. Computers and Geotechnics, 33(4~5): 209-221.

35. MacBeth C (2004). A classification for the pressure-sensitivity properties of a sandstone rock frame. Geophysics, 69(2): 497-510.

36. McKenna F, Fenves GL (2001). The OpenSees command language manual, version 1.2. Pacific Earthquake Engrg. Research Center, Univ. of Calif., Berkeley. (http://opensees.berkeley.edu).

37. Moss RES, Seed RB, Kayen RE, Stewart JP, Kiureghian AD, Cetin K (2006). CPT-based probabilistic and deterministic assessment of in situ seismic soil liquefaction potential. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 132(8): 1032-1051.

38. Rathje EM, Kottke AR, Trent WL (2010). Influence of input motion and site property variabilities on seismic site response analysis. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 136(4): 607-619.

39. Risk Engineering, Inc. (2002). Seismic Hazard for New York City Final Report. To Weidlinger Associates.

40. Rollins KM, Evans MD, Diehl NB, Daily WD III (1998). Shear modulus and damping relationships for gravels. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 124(5): 396-405.

41. Schnabel PB, Lysmer J and Seeed HB (1972). SHAKE: A computer program for earthquake response analysis of horizontally layered sites. Report No. EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley.

42. Seed HB, Idriss IM (1971). Simplified procedure for evaluating soil liquefaction potential. Journal of Geotechnical Engineering Division ASCE, 97(9): 1249–1273.

43. Seed HB, Idriss IM (1982). Ground motions and soil liquefaction during earthquakes. Earthquake Engineering Research Institute Monograph, Oakland, Calif.

44. Silva WJ, Abrahamson N, Toro G, Costantino C (1997). Description and validation of the stochastic ground motion model. Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573.

45. Stewart JP, Kwok AOL (2008). Nonlinear seismic ground response analysis: Code usage protocols and verification against vertical array data. Proceedings Geotechnical Earthquake Engineering and Soil Dynamics IV, Geotechnical Special Publication 181, Paper No. 1.

46. USGS (2010). Overburden Thickness and Generalized Distribution of Glacial and Alluvial Deposits in Rockland County, NY.

47. Vucetic M, Dobry R (1991). Effects of soil plasticity on cyclic response. ASCE Journal of Geotechnical Engineering, 117(1): 89-107.

48. Weidlinger Associates (2008). NYCDOT Seismic Design Guidelines for Bridges – Commentary.

49. Youd TL et al. (2001). Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF Workshops on evaluation of liquefaction resistance of soils. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 127(10): 817-833.

50. Zeghal M, Elgamal AW (1994). Analysis of site liquefaction using earthquake records, J. Geotech. Eng. 120, no. 6, 996–1017.

51. Zhang J, Andrus RD, Juang CH (2006). Normalized shear modulus and material damping ratio relationships. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 131(4):453-464.

52. Zhang J, Andrus RD, Juang CH (2006). Normalized shear modulus and material damping ratio relationships. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 131(4):453-464.

53. Zhang J, Andrus RD, Juang CH (2008). Model uncertainty in normalized shear modulus and damping relationships. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 134(1):24-36.