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Site-Specific Earthquake Ground Motions for the 12th Street and 14th Street Viaducts on Route 139 in Jersey City, New Jersey

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Site-Specific Earthquake Ground Motions for the 12th Street and 14th Street Viaducts on Route 139 in Jersey City, New Jersey

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ABSTRACT

The 12th and 14th Street Viaducts, which carry Route 139, are located west of the Holland Tunnel in Jersey City, New Jersey. The two viaducts, supported on pile foundations, provide access to and from the Holland Tunnel (which connects New York City and New Jersey) and are considered critical structures. The New Jersey Department of Transportation (NJDOT) has selected DMJM+HARRIS to perform design services including rehabilitation and seismic retrofit for the 12th Street and 14th Street Viaducts.

The subsurface conditions play a major role in evaluating the structures behavior under a seismic event. At the project site, the subsurface conditions consist of the following six strata: Loose to Medium Dense Sand (Fill), Peat/Organic Silt, Normally Consolidated Silty Clay, Silt/Sand, Weathered Rock and Bedrock. The magnitude of the seismic event will influence the response of these soil strata. The purpose of this paper is to present the results of a site-specific earthquake ground motions on rock. A probabilistic seismic hazard analysis (PSHA) was developed for this site. In addition, the paper will also present the results of site response analysis performed to compute ground motions at footing level and their associated response spectra. Values of soil amplification between the spectral acceleration of the footing level and the bedrock response spectra will be calculated and presented. A comparison of the resulting data to the current design spectra as defined in the 1998 New York City Department of Transportation (NYCDOT) Seismic Design Guidelines is presented.

In order to determine the effects of the site soils on bedrock motions, it was required to define a soil profile with layer thickness and dynamic soil properties. The dynamic soil properties for the soil profiles were derived using a correlation with shear wave velocities obtained from cross hole surveys and from a correlation with standard penetration tests. Fourteen (14) soil profiles with two hazard levels having a return period of 500 years (probability of exceedence of 10% in 50 years) and a return period of 2500 years (probability of exceedance of 2% in 50 years) will be compared to the NYCDOT Class C, D and E soil profiles, which are appropriate for this site. The site response spectra were developed for the two level seismic events, using the computer program SHAKE. Nonlinear soil behavior is approximated by equivalent linear techniques implemented in SHAKE.

PROJECT BACKGROUND

DMJM+HARRIS, Inc. was selected by New Jersey Department of Transportation (NJDOT) to perform seismic retrofit for 12th and 14th Streets Viaducts. The two Viaducts are located within the confines of Jersey City, New Jersey just west of the access to and from the Holland Tunnel. Both of these viaducts serve as primary access to and from the Holland Tunnel, which connects Jersey City to Manhattan. The 12th Street Viaduct (constructed in 1926) carries eastbound traffic towards the Holland Tunnel while the 14th Street Viaduct (constructed in 1948) carries westbound

Fig. 1. Site Location.

traffic away from the Holland Tunnel. The viaducts run east to west for a distance of approximately 2000 feet. See Figure 1.

The 12th Street and 14th Street Viaducts in Jersey City are classified as critical structures. Therefore, the two structures were evaluated for two levels seismic return periods. The 500-year (10% in 50 years) and 2500 year (2% in 50 years) return periods.

The performance criteria for the 500-year event is that the viaducts will not collapse, there will be no damage to the primary elements, minimal repair, and that there will be full access to normal traffic within a few hours. While the performance criteria for the 2500-year return period is that the bridge will not collapse, limited access for emergency vehicles and full service within months.

The cost associated with seismic retrofitting of both the substructure and superstructure of the two existing viaduct structures made a site-specific analysis essential. The site-specific analysis was performed by Geomatrix Consultant whereas geotechnical investigation, soil profiles, and dynamic soil properties were performed by DMJM+HARRIS.

FIELD INVESTIGATION

A total of 84 soil borings were drilled for the 12th Street and 14th Street Viaducts along Route 139. Each soil boring that was drilled terminated 10 feet into rock. Eight of these boreholes were used at four locations for seismic cross hole surveys. These surveys were used to determine the shear wave and compressional wave velocities along the project site. The cross hole surveys were performed by Geophysics GPR International, Inc. on March 2000.

The typical cross hole survey consisted of two boreholes, at approximately 10 feet apart. One borehole was used for lowering the seismic source hammer, which produced the seismic energy waves and the second borehole was used to take readings with a 3D geophone. Based on the travel time of the shear (S) and compressional (P) waves with distance the shear (Vs) and compressional (Vp) wave velocities were computed.

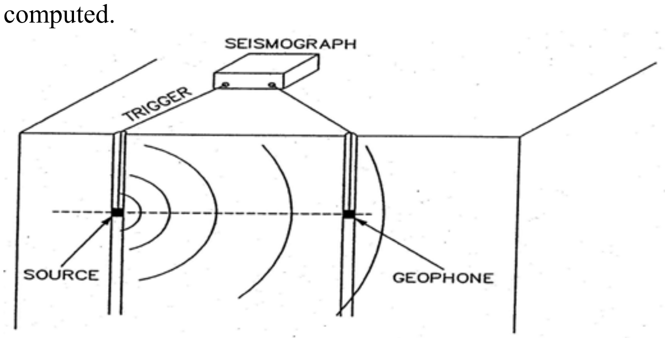

Fig. 2. Seismic cross hole survey

SUBSURFACE CONDITIONS

In general the subsurface profile consisted of five (5) distinct strata underlain by rock. The following describes the subsurface data starting from ground surface:

(1) Fill:
This stratum consists of fill material that varies from a loose to dense sand and gravel to construction debris. This layer begins at the ground surface and extends to a maximum depth of seventeen (17) feet below the ground surface.

(2) Peat/Organic Silt:
This is a thin layer, which underlies the fill material. This layer is approximately five (5) feet in thickness throughout the site and covers the natural soils.

(3) Normally Consolidated Silty Clay:
The thickness of this material varies throughout the site, with standard penetration values of basically weight of hammer. The maximum thickness of this layer is fifty (50) feet at the center of the project.

(4) Non Plastic Silt/Sand:
This layer is very dense with N Values ranging from 14 to refusal. This layer varies in thickness from six (6) feet to fifty (50) feet at the eastern most point of the project.

(5) Decomposed Rock:
This layer is encountered above the bedrock and follows the bedrock contours.

(6) Bedrock:
The bedrock in this area consists of diabase and sandstone. The diabase was recovered predominately on the west side of the project and the sandstone on the east side of the project. The depth to bedrock varied from 2 feet at the west end with the rock line sloping rapidly at approximately 2 horizontal to 1 vertical to a maximum depth of 110 feet towards the center of the project. The rock then rises gently going east to an approximately depth of 50 feet. The RQD's in the rock formations vary from poor in some of the sandstone samples to excellent in the diabase samples. See Figure 3 for a general soil profile of the project.

The groundwater at the project site was monitored for several months with monitoring wells installed during the subsurface investigation. The water level was generally five (5) feet below the ground surface.

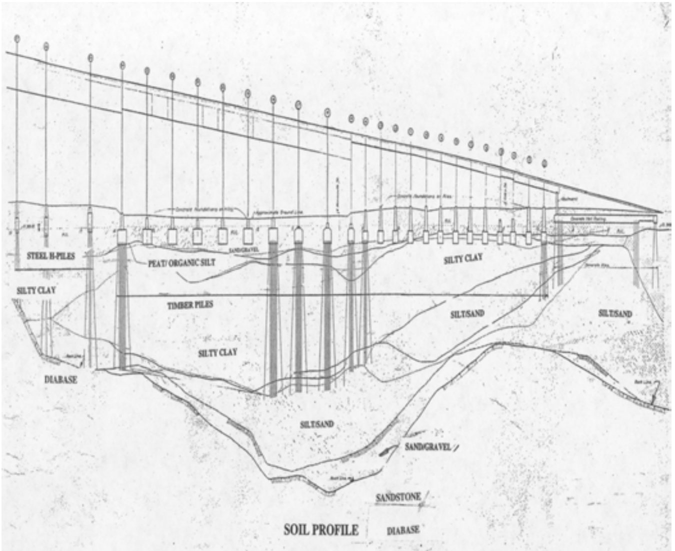

Fig. 3. Soil Profile

LABORATORY TESTING

Laboratory testing was performed on specific samples to determine soil classification and engineering properties of the soils, such as shear strength, unit weight and dynamic soil properties that are required for a site specific analysis.

In order to perform the site-specific analysis dynamic soil properties needed to be determined such as the small strain shear modulus Gmax and small strain Young's Modulus Emax.

These values were determined from empirical correlations:

$$Gmax = \rho \times V_s^2 \qquad (1)$$

ρ = mass density of soil
V_s = Shear Wave Velocity

$$Emax = 2(1+\nu)Gmax \qquad (2)$$

ν = Poison's Ratio

Table 1 Dynamic Soil Properties

Soil Description	γ (lbs/ft^3)	Gmax (ksf)	Vs (ft/s)
Sand (Fill)	110	1984	760
Peat/.Org Silt	65	1399	830
Silty Clay	94	2172	860
Silt/Sand	119	2924	890
Sand/Gravel	125	4658	1090
Bedrock	187	6892	1090

PROBABILISTIC SEISMIC HAZARD ANALYSIS

The intensity of the ground motions, which may be generated, was identified using a Probabilistic Seismic Hazard Analysis (PSHA). The ground motion parameters were evaluated using an attenuation relationship using synthetic models of ground motion. This is due to the lack of data to characterize strong ground motions along the east coast of the United States. Sources were identified that were capable of producing the ground motion and the minimum and maximum earthquake was obtained along with frequency. Results of the PSHA were used to determine the peak horizontal and vertical ground motion.

In the PSHA the following needs to be identified:

1) Determine the source to produce the ground motion.
2) The maximum and minimum magnitudes associated with each source
3) Attenuation relationship for each source.
4) The probability of exceeding the ground motion parameter for a specified time.

Three synthetic earthquakes were selected to represent the hazard return period. The following earthquakes were selected:

Table 2 Earthquake Characteristics

Group	Body Wave Mag (mb)	Distance (km)
1	5.7	17
2	6.1	60
3	7.0	300

From these events three synthetic accelerograms were developed to evaluate the duration of strong shaking for the bedrock for both the 500-year and 2500-year events. See Figures 4 and 5.

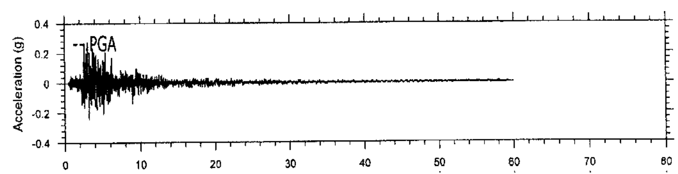

Fig.4a. 2500-year event Group 1

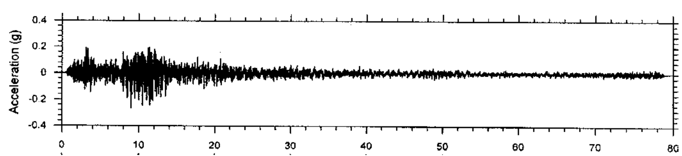

Fig.4b. 2500-year event Group 2

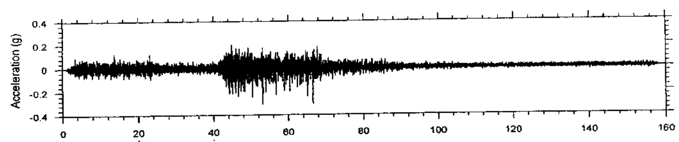

Fig.4c. 2500-year event Group 3

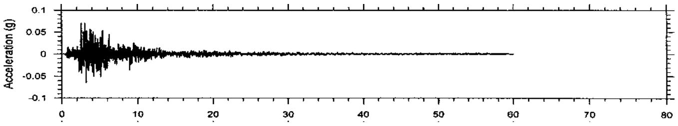

Fig.5a. 500-year event Group 1

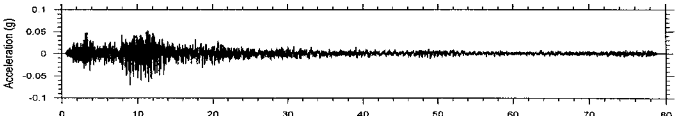

Fig.5b. 500-year event Group 2

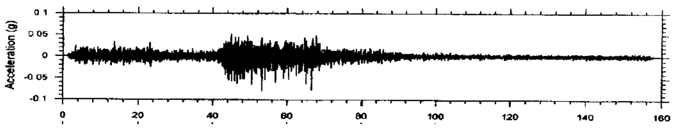

Fig. 5c.500-year event Group 3

The PSHA computes the probability of exceeding various levels of ground motions parameter over a given time.

RESPONSE SPECTRA FOR ROCK

From the (PSHA) a hard rock response spectra was obtained. The characterization of hard rock is defined as competent rock having a shear wave velocity greater than 3000 ft/sec. Based on the low shear wave velocities that were obtained from the subsurface investigation, the site response for hard rock was scaled down to a soft rock using factors that were obtained from the US Geological Survey. The comparison of hard rock to soft rock is shown in Figure 6.

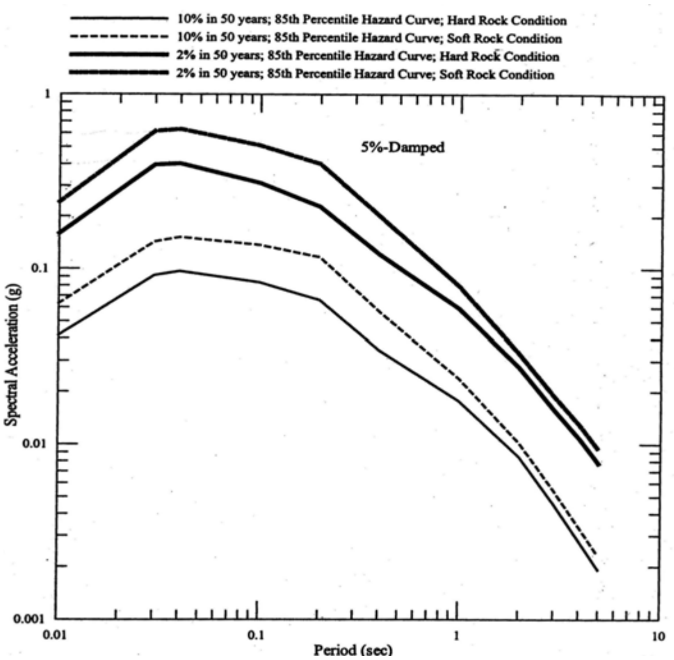

Fig.6. Comparison of Soft Rock to Hard Rock

SITE RESPONSE ANALYSIS

The site response analysis was performed for both the 500 and 2500-year events. The site response analysis was performed to compute ground motions at existing footing levels along with its response spectra. The response spectrum was obtained for periods ranging from 0 to 5 seconds and with a damping ratio of 5%.

A one dimensional equivalent linear seismic site response analysis of horizontally layered soil deposits using SHAKE was performed. The rock motions developed were used as input motions to the response spectra. Due to the presence of soft soils at the site, it is anticipated that some level of amplification will take place. The amplification can be contributed to the thickness of the overburden soils in the local soil profile and the bedrock ground motion.

Because of the varying soil conditions on the project site fourteen (14) idealized soil profiles (Table 3) were used to determine response spectrum for both seismic events.

Table 3 Idealized Soil Profiles

12th Street Viaduct		14th Street Viaduct	
Profile	Bent	Profile	Bent
A	E Abutment	I	E Abutment
B	1-8	J	2-15
C	9- 13	K	15- 18
D	13- 19	L	18- 24
E	19- 23	M	24- 26
F	23 – 27	N	18-24
H	31- 32		

Amplification for the horizontal design spectral acceleration between the footing level and soft rock response spectra were calculated. These values are presented in Table 4 for the 500-year event and in Table 5 for the 2500-year event.

Table 4 Amplification Factor 500-Year Event

	Profiles							
Period	A	B	C	D	E	F	G	H
0.010	1.90	1.75	1.75	1.90	1.75	1.75	1.75	1.51
0.200	2.56	2.39	2.39	2.56	2.39	2.39	2.39	1.16
1.000	1.67	1.67	1.88	2.58	2.71	2.08	1.67	1.67

	Profiles					
Period	I	J	K	L	M	N
0.010	1.90	1.90	1.75	1.90	1.75	1.90
0.200	2.56	2.39	2.39	2.56	2.39	2.39
1.000	2.08	2.08	2.71	2.08	1.88	2.08

Table 5 Amplification Factor 2500-Year Event

	Profiles							
Period	A	B	C	D	E	F	G	H
0.010	1.41	1.41	1.20	1.29	1.29	1.29	1.37	1.58
0.200	2.12	2.12	1.82	1.95	1.95	1.95	2.07	1.19
1.000	1.63	1.63	1.88	2.75	3.00	2.25	1.88	1.13

	Profiles					
Period	I	J	K	L	M	N
0.010	1.29	1.45	1.33	1.49	1.49	1.49
0.200	2.29	2.19	2.00	2.24	2.24	2.24
1.000	2.00	2.00	2.88	2.00	2.00	2.00

NYCDOT SEISMIC DESIGN GUIDELINES

In 1998 the New York City Department of Transportation (NYCDOT) issued Seismic Criteria Guidelines for highway structures which applied to both new bridges and bridges being rehabilitated. The guidelines were developed by an expert Panel for the DOT under the guidance of the consultant firm of Weidlinger Associates. The following areas were included in the guidelines: New York City, Rockland, Westchester, and Nassau Counties in New York and Passaic, Bergen, Essex, Hudson, Union and Middlesex Counties in New Jersey.

Amplification due to the soil profiles was generalized by classifying the sites into 5 classifications A through E. The classification relates the soil conditions to amplification. The classification criteria were based on shear wave velocity, standard penetration and undrained shear strength.

The following table generalizes the criteria for the design spectra as per NYCDOT.

Table 6 Soil Classes for Design Spectra

Soil Class	Description	Vs(ft/sec)	Su(ksf)	N
Soil A	Hard Rock	>5000	--	--
Soil B	Firm to Hard Rock	2500-5000	--	--
Soil C	Dense Soils/Firm Rock	1200-2500	2	>50
Soil D	Stiff Soils	600-1200	1-2	15-50
Soil E	Soft Soils	<600	<1	<15
Soil F	Special Investigation Soft Soils		--	--

RESULTS

A comparison was performed between the NYCDOT response spectrum and the site specific performed for this project.

The response spectra for hard rock from the site specific study shows that the NYCDOT response spectra is more conservative for periods greater than 0.03 seconds and less conservative than the site specific response spectra for periods less than 0.03 seconds for both the 500 year and 2500 year events. See Fig 7.

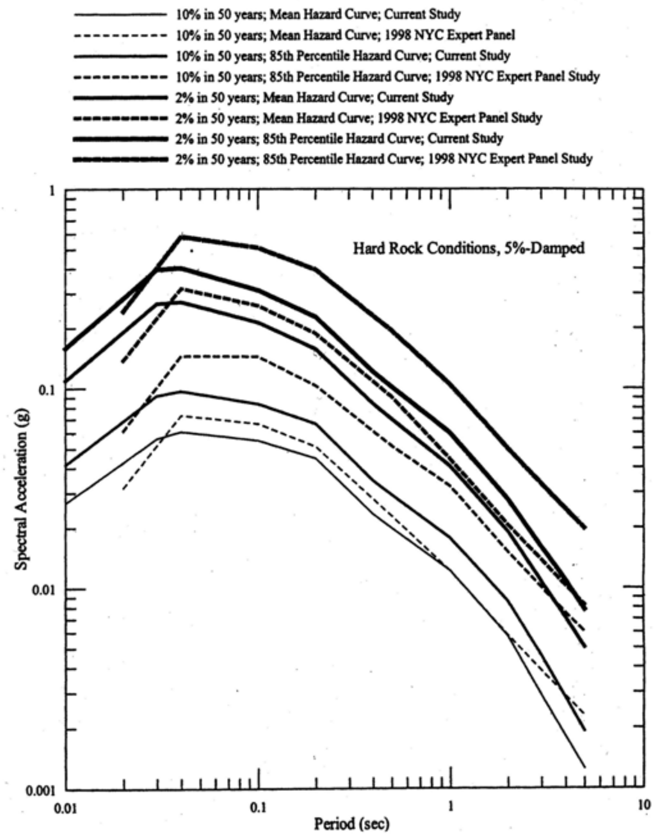

Fig.7. Response spectra for hard rock

Several idealized soil profiles from the project site were also compared to NYCDOT soil classification. In this paper, because of the limited printing space, only Idealized soil profiles C, D, E and F were used for comparison purpose.

The following are the idealized soil profiles

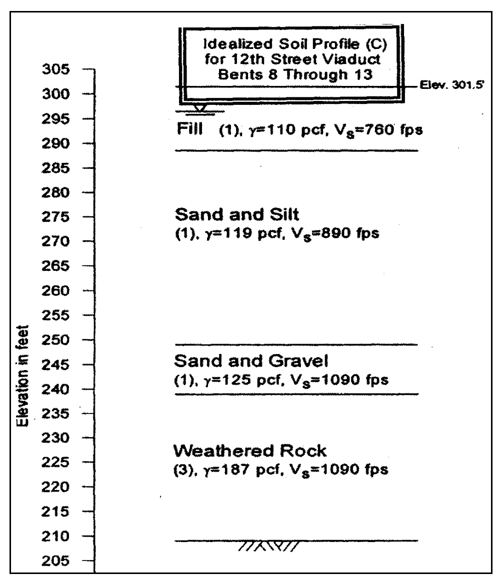

Fig. 8. Idealized Soil Profile C

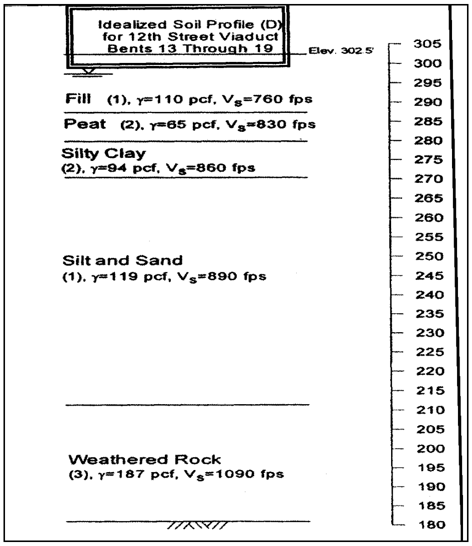

Fig. 9. Idealized Soil Profile D

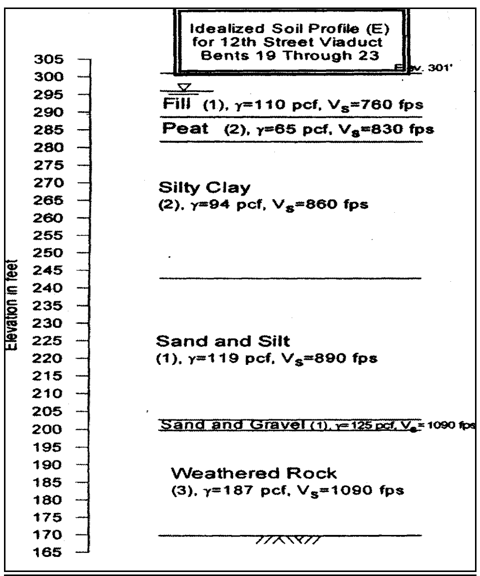

Fig. 10. Idealized Soil Profile E

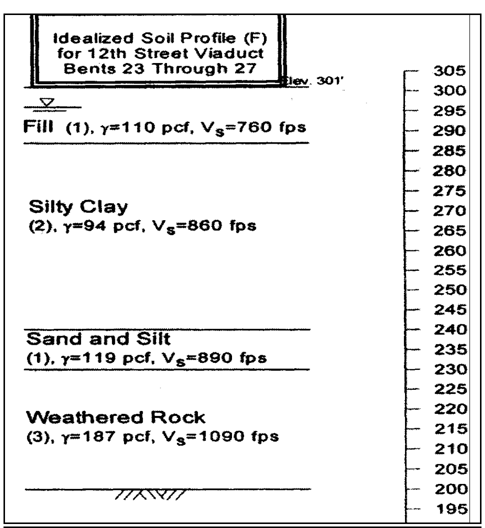

Fig. 11. Idealized Soil Profile F

The idealized soil profiles E and F are comparable to NYCDOT profiles D and E. This is determined from the soil classification, the shear wave velocity and the undrained shear strength. For the 500 year and 2500 year events the NYCDOT response spectra are more conservative throughout.

On the other hand, idealized soil profiles C and D relate better with soil classification D from NYCDOT. This was also determined from soil classification, the shear wave velocity and the standard penetration values. The spectrum are comparable for the 500-year event up to a period of 0.05 seconds then the NYCDOT spectra is more conservative at higher periods. For the 2500-year event the NYCDOT spectra is more conservative throughout. See Figures 12 and 13 for comparison.

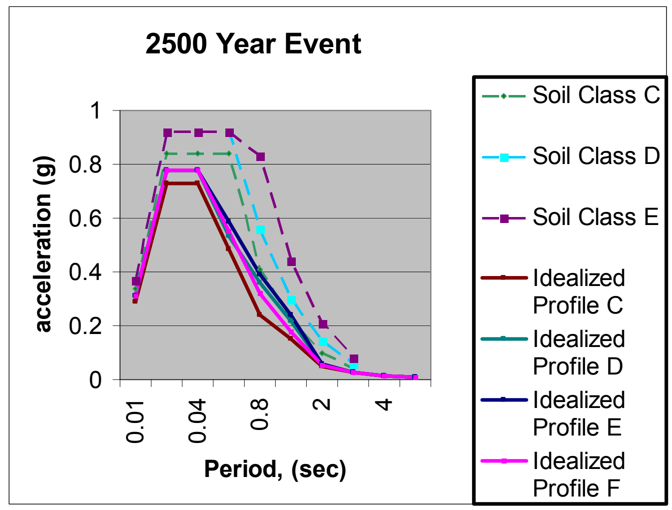

Fig. 12. Comparison of Response Spectrum for 2500-yr event

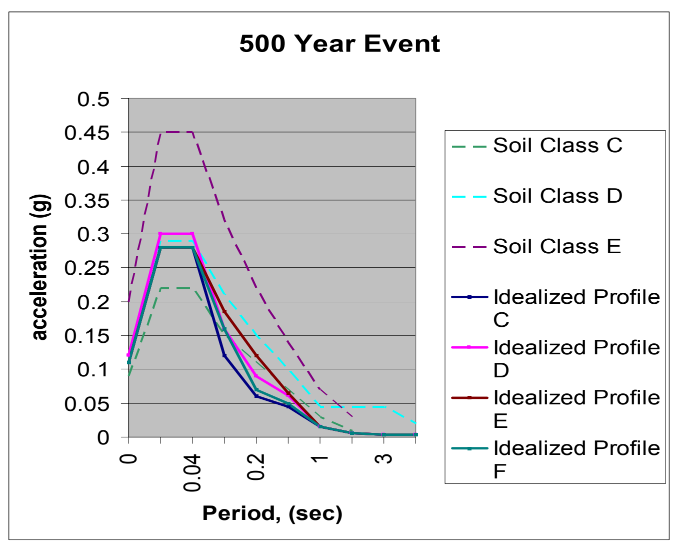

Fig. 13. Comparison of Response Spectrum for 500-yr event

CONCLUSIONS

The project presented in this paper is located in Jersey City, Hudson County, New Jersey. The paper presented the results of site-specific earthquake ground motion on rock and at footing levels. The Design Engineer is always faced with the challenge of selecting the appropriate response spectra for the retrofit design. These values are very critical in evaluation soil liquefaction potential and determining lateral forces on the structures. Current Standard Codes available within the geographical region of the project presented here include: AASHTO, New York City Department of Transportation, and Uniform Building Code.

The results of site-specific analysis proved to be less conservative than available Standard Codes and provided cost saving in terms of foundation retrofit for the project presented. Performing site-specific analysis may not be as complicated as some may expect. Successful analysis requires identifying: subsurface conditions, thickness of each stratum, engineering properties and shear wave velocity for each layer. This information can be obtained during the subsurface investigation program required for the project. Owners and Engineers are getting more familiar with the procedures of the site-specific analysis. In recent years some Standard Codes have elected to requiring to perform a site-specific analysis particularly at sites with soft soils and for large-scale projects.

With seismic criteria becoming more widely used on the east coast a site-specific analysis could be an efficient method to reduce rehabilitation costs on existing structures and proposed structures. The cost of a site-specific study may be minimal to the overall construction costs that may be saved.

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