PRELIMINARY SEISMIC CONSIDERATIONS FOR PULASKI SKYWAY REHABILITATION PROJECT

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Abstract

The 3.5 mile (5.63 km) long Pulaski Skyway is located in northern New Jersey, serving as a major connection from Newark to New York City. Built in 1932, the Skyway mainline consists of different types of superstructures supported on 108 piers and one abutment. Due to existing bridge condition and funding availability, the entire bridge is under plan and design for major rehabilitation. Seismic retrofit is one of the major rehabilitation actions under plan and has to be considered in conjunction with other rehabilitation actions. To achieve cost-effectiveness, feasibility assessment and preliminary site-specific seismic analysis are conducted. This paper presents background of the bridge rehabilitation plan, seismic considerations, assessment result, and geotechnical seismic vulnerability study that may lead to decision making.

Introduction

The Pulaski Skyway, completed in 1932 and opened to traffic in 1933, is the vital link in the northern New Jersey transportation network, linking Jersey City, South Kearny and Newark. It serves as an express link for car and bus traffic to and from the Holland Tunnel, via Rt.139, carrying ADT of 74,000. The 3.5 mile (5.63 km) long elevated structure is composed of a series of different types of bridges (118 spans in total: 108 for mainline and 10 for east approach) that carries Route 1 & 9 over the Hackensack River, Passaic River, New Jersey Turnpike (I-95), several railroads, local roads, and industry facilities (**Fig.1**).

The inspection reports (NJDOT, 2010) have shown that the Skyway is in need of major repair and rehabilitation due to deterioration that occurred over its lifetime. The bridges are structural deficient (SD) and functional obsolete (FO) that do not meet modern design and safety standards. NJDOT has developed long term strategy for improvement of the entire structure, anticipated to commence in 2015. Due to the high cost and complexity, this major project will be performed in several stages, and concept development and initial/feasibility assessment study are conducted to evaluate project alternatives and select the most cost-effective alternative to advance to the design.

Among many factors for the preservation and rehabilitation plan, structural systems and safety concerns are the major considerations. The objective is to bring the Pulaski Skyway into a good service condition and extend to another life cycle useful for 75 years by addressing structural deficiencies, mitigating vulnerability to seismic

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hazard, modifying functional deficiencies, and improving the overall condition and operational safety of the roadway. This paper will provide brief information on the current structural condition, rehabilitation considerations, and seismic retrofit considerations, through initial site-specific assessment and geotechnical seismic vulnerability study.

Existing Bridge Conditions

The Pulaski Skyway consists of 108 spans (numbered east to west) of elevated bridge structures, including a series of superstructure (**Fig.2**): two 550 ft (168m) through truss main spans with 350 ft (107m) flanking truss spans over Passaic River and Hackensack River, three steel through trusses over railroads in Jersey City (east), deck trusses between the two through trusses, and deck trusses and girder bridges in the remaining spans. The foundation types along the mainline are complicated, including spread footings, pile foundations (concrete and timer), and concrete caissons. The bridge is 3.5 mile (3.63 km) long and 56.5 ft (17.2 m) wide (**Fig.2**) carrying 4 lanes. The original concrete deck slab still remains with several times of resurfacing along the service life, and will be replaced entirely in the near future. Note that ramps and approaches off the mainline will not be included in the discussion of this paper.

Based on the previous inspection reports and suggestions, a series of interim repairs and rehab have been implemented in 1978, 1983-1984 and 2008, such as deck repairs, deck overlay/resurfacing, partial widening, parapet/railing repairs, priority repairs to superstructure and substructure.

The current biennuial inspection report shows that the bridge overall physical condition is “Poor” due to superstructure condition, and overall condition is “Serious” due to low inventory ratings of some truss diagonal and steel girder members. The following is a summary:

- *Components* (**Fig 3** & **Fig.4**):
  - Deck: Poor (due to continued extensive deterioration of underdeck, and spalling of the top of the deck);
  - Superstructure: Poor
  - Substructure: Fair
- *Structurally Deficient* (SD): Poor ratings; Structural steel defects
- *Functionally Obsolete* (FO): Poor geometrics; Low vertical clearance

A further in-depth FEA rating of the entire structure is being performed and result is under evaluation to ensure the final load rating. A special assessment of the primary truss gusset plates was conducted to measure remaining sections and used in this in-depth structural analysis.

Deck and Joints The deck is in overall “poor” condition due to continued extensive deterioration of the underdeck (spalling and cracking with efflorescence), and continued spalling of the top of the deck (**Fig.3**). The loose concrete on the underside of deck has become a safety issue for roadway below. The slide plate and steel finger expansion joints throughout the bridge are in “fair” condition with vertical profile mismatch from ¼” (~6mm) to 1” (~25mm) and one is frozen.

Superstructure The structure steel exhibits areas of moderate to severe rust throughout, with substantial pack rust and local areas of heavy section losses; Severe rusting occurs primarily at the truss lower chords and joints, and in all steel components below the open deck joints and floorbeam cantilever ends. The paint system has generally failed throughout the structure.

Concrete encasement for the stringers and girders typically exhibits minor to moderate deterioration. The on-going maintenance has performed to remove most of the existing encasement from the steel girders and expansion pier bents, both to prevent falling concrete onto roadways and ensure that hidden section losses at the joint areas can be discovered and repaired (**Fig.3**).

Typical deterioration of truss members in through trusses and deck trusses are shown in **Figure 3**. The rate and amount of localized deterioration represents a progressive structural distress that has continued since last biennial inspection.

The truss pin assemblies of upper and lower chords typically exhibit localized material loss due to wear around the exposed surface of the pin between the truss connection plates, or rust. UT testing of primary truss pins and accessible truss pins of other spans was performed. The observation indicated the presence of 3 small localized discontinuities which are acceptable due to corrosion pitting, etc.

Bearings Severe rust, minor section loss, missing nuts, and pack rust between vertical stiffeners are observed at fixed and expansion bearings, but evidence of normal movement are noted at all sliding bearing locations.

Substructure Pulaski Skyway mainline structure was constructed on different types of foundations depending on superstructure type, soil condition and foundation depth. They include spread footings, timber piles, CIP concrete piles, precast concrete square piles, concrete caissons (single or pair), columns/piers are of two types: concrete encased steel piers and reinforced concrete piers with pedestals (**Fig.4**),

The substructure is in “fair” condition. Most deterioration is concrete cracks with heavy efflorescence, spalling, delaminated concrete areas. Many of these defects have previously been repaired with concrete patches and epoxy crack sealant, which is deteriorated/cracked again. The previously added post tension rods and beam collars appear to be limiting crack propagation (typically for Piers 65 to 67 Main Truss over Hackensack River) (**Fig.4**).

Earlier underwater inspection report states that the submerged substructure components are in overall “fair” condition (east portion) due to cracks and spalls in the concrete, or satisfactory (west portion).

Rehabilitation Plan

Several remedial actions are recommended and implemented for interim repairs and short-term rehabilitation based on the biennial inspection report. The structure is on the National Historical Register and a full replacement is not economically feasible, so the final decision was made to rehabilitate Skyway in current configuration. With significant funding available recently, a comprehensive long-term rehabilitation plan has been developed by NJDOT, under the Pulaski Skyway Improvement Program, to ensure the bridge integrity and extend the bridge service life. The following major rehabilitation actions related to the entire bridge structure are planned:

- Deck replacement:
  - Using precast panels with stainless steel reinforcement, and using partially lightweight concrete;
  - Structural response to be monitored by sensors during different construction stages
  - Deck joint replacement and joint reduction/elimination; use of modular joints
- Superstructure rehabilitation:
  - Concrete encasement removal;
  - Replacement of severely corroded members;
  - Structural steel repairs for severely deteriorated members
  - Strengthening members rated low (under consideration dependent on FEA rating results)
- Substructure rehabilitation:
  - Repair of concrete columns/piers, bent caps/strut beams, pedestals and abutments
  - Retrofit of foundations/footings (dependent on seismic retrofit analysis)
- **Seismic retrofit**:
  - Bearing replacement with isolation bearings (under consideration);
  - Soil improvement (under consideration)
  - Other alternatives (seat length; restraints, Etc.)
- Re-painting:
  - Removal of existing lead paint
  - Re-coating

General Seismic Considerations in New Jersey

The seismic histories of New Jersey and New York, as documented by NJDEP, NYGS and USGS, show that many earthquakes have been recorded in the project region between 1973 and 2012. Although it makes NYC and northern NJ region among the highest in frequency of seismic activity in the country, the recent 100 year or so recorded maximum magnitude of earthquake in New Jersey was 5.0 in Richter Scale (VI and VII in MM Scale). Prior to 1990s, design and details of all highway bridges did not take earthquake or security issue into account. This implies that most existing highway bridges, including Pulaski Skyway, have potential to be vulnerable to seismic damage during an earthquake event.

Now NJDOT requires that seismic design and seismic retrofit for standard or ordinary bridges to follow AASHTO “Guide Specifications for LRFD Seismic Bridge Design” (AASHTO, 2011) and FHWA “Seismic Retrofitting Manual for Highway Structures” (FHWA, 2006), respectively (NJDOT, 2009). All NJDOT bridges should initially be considered to be “standard” and designed for “life safety” performance objective considering a seismic hazard corresponding to 7% probability of exceedance in 75 years (approximately 1,000 year return period). However, consideration for increasing bridge Importance Category is permitted and should strictly be based on social/survival and security/defense factoring of the bridge location. If these factors clearly indicate the location’s critical nature, increase of Importance Category and/or Performance Level may be considered. The foundation supporting a bridge structure shall be designed not to experience damage in an earthquake event to prevent from costly inspection and repair work after an earthquake event.

To obtain general seismic information for preliminary design reference purpose, a research was carried out on “Seismic Design Considerations” in New Jersey (Agrawal, Liu & Imbsen, 2011) and statewide Seismic Design Category (SDC) maps were developed for Standard Bridges using AASHTO/USGS 1,000 year hazard map, and Critical Bridges using USGS 2,500 year hazard maps (or using 1.5 time seismic hazard map of 1,000 year return Period), respectively. The SDC maps are developed based on AASHTO Guide Spec procedure, AASHTO/USGS seismic maps, and representative soil classes for each zip code location referring to NJDOT Geologic Survey (NJGS) boring log database (**Fig.5**).

Figure 6 shows the SDC maps for Standard/Ordinary Bridges and Critical Bridges in New Jersey. It can be seen that for standard/ordinary bridges, the SDC is “A” for almost all locations in New Jersey regardless of soil site classes (except some locations requiring site-specific investigation (blue area in the map)), while for critical bridges, SDC “B” possibly exists in some locations (green in Hudson County) due to low soil site classification “E” or “D” (purple or red in **Fig.5** soil site map). Coincidently Pulaski Skyway falls in this area.

In conjunction with the seismic hazard analysis of New Jersey, liquefaction hazard analysis was conducted to assess the liquefaction potential of each zip code. The analysis utilized the Standard Penetration Test (SPT) blow counts of soil and followed the approach by Youd et al. which is one of the approaches suggested by the AASHTO Guide Spec. (AASHTO, 2011). It can be seen from these maps that areas with higher liquefaction hazard are mainly in the northern part of New Jersey especially where the Pulaski Skyway is located. For 2,500 year event, compared to the hazard for 1000-year earthquake, the areas with “medium” liquefaction hazard are now classified as “high”, and some areas with “low” hazard now have “medium” liquefaction hazard.

It is noted that the SDC maps and liquefaction hazard maps for standard or critical bridges are for preliminary design and reference purposes only, since critical or specially important bridges require site specific analysis and the maximum acceleration a_{max} at ground surface that is needed for liquefaction potential analysis that must be obtained using site-specific analysis.

Although Pulaski Skyway is a vital link between the Holland Tunnel/NYC and NJ turnpike, interstate highway and northern Jersey highway system, considering the highway redundancy in the area and high cost of seismic retrofit, it is very important to correctly evaluate the bridge vulnerability, risk of seismic damage and alternative comparison to achieve the most cost-effective solution.

It appears that Pulaski Skyway seismic analysis needs further site-specific analysis, because 1) From above general analysis, Pulaski is located in a relatively high seismic risk area of New Jersey as an important bridge; 2) The bridge is a historical signature bridge as a vital highway link; 3) Seismic retrofit for this existing bridge should follow FHWA Seismic Retrofitting Manual (FHWA, 2006; MCEER, 2006), instead of AASHTO Guide Spec. (AASHTO, 2011) despite their similar philosophy.

It is anticipated that the following questions be addressed from site-specific seismic analysis:

1) What seismic hazard level should be used for seismic analysis to achieve a cost-effective alternative?

2) What is the seismic retrofit demand for the selected seismic hazard?

3) What alternate measures are needed for seismic retrofit for superstructure and substructure?

4) What extensive seismic retrofit should be taken or avoided in conjunction with the entire rehabilitation plan, such as isolation bearings, subsurface soil improvement, pier and foundation retrofit, etc.

Initial Assessment for Seismic Design Criteria

Pulaski Skyway site-specific subsurface soil investigation was conducted, including P.S. Logging testing. Initial site-specific seismic response analysis was performed for three seismic hazard levels of 500, 1,000, and 2,500 year return period to preliminarily understand the seismic retrofit demand for the bridge. From site specific soil investigation result, site class is summarized as: Piers 1 to 41 – Class D; Piers 42 to 63 – Class E; Piers 64 to 98 – Class E; and Piers 99 to 108 – Class D.

Multi-mode response spectrum analysis was performed. The bridge period for main through trusses is 1.71 Sec. for Mode 1 and 1.49 Sec. for Mode 2. The recommended design response spectrum for Pier 64 to 98, covering two main through trusses and deck trusses between and beyond the two main trusses, is shown in **Fig. 8**.

To further investigate relative cost estimates for seismic retrofit, the effect of the three seismic hazard levels on the Capacity (C) to Demand (D) ratio, C/D, are analyzed.

The results for Piers 41 to 100 are plotted in **Figure 9**, presenting elastic moment C/D ratio for concrete pier pedestals (column bases). Pier column results are skipped herein. Most of the concrete piers were found to be deficient for the 2.500-year event, i.e. C/D is less than 1.0. The computed C/D ratio is as low as 0.16 for column (not shown herein) and 0.23 for pier pedestal (column base). According to available drawings, some original pier columns and pedestals appear to be unreinforced. In addition, some piers were subject to net uplift under 2,500 year event without capacity. It is clear that seismic retrofit required for 2,500 year event would be very extensive.

For the 1,000 year event, the computed C/D ratios are significantly higher than 2,500 year event, and most columns and pedestals are adequate (i.e. C/D is equal or greater than 1.0). The lowest C/D is 1.5 for pier column and 0.61 for pedestal, and none of them would be subject to net uplift for the 1,000 year event. Comparing to 2,500 year event, the seismic demand is greatly reduced and hence seismic retrofit required would be less extensive. It is noted that although seismic demand may be lower, due to exiting pier condition and soil site condition, the vulnerability to seismic damage does exist. For 500 year event, computed C/D ratios for pier columns and pedestal are greater than 1.0 for all piers from 41 to 100.

On the other hand, FHWA Seismic Retrofit Manual (FHWA, 2006) is used to find out seismic retrofit requirement for the Pulaski Skyway following the evaluation procedure provided in the Manual. Providing that Bridge Importance is “Essential”, Anticipated Service Life is ASL-3 for 75 years (>50 years), and Upper Level Ground Motion is 1,000-year return period, the minimum performance level would require to be PL2 Operational, which is the performance level Pulaski Skyway bridge is expected. The initial Seismic Retrofit Category (SRC) evaluation comes up with Hazard Level II and SRC B for PL2 and SRC C for PL3 “Fully Operational”.

Geotechnical Seismic Vulnerability Assessment

A comprehensive work (PB, 2013) has been performed for soil logging as deep as 100 ft. (30.5m), deriving shear wave velocity, site-specific seismic response analysis, liquefaction evaluation for 1,000 year return period, and evaluating foundation vulnerability (C/D).

Due to the lack of real earthquake records (acceleration time histories); selected seed histories (NYCDOT) were used for ground motion input in Pulaski Skyway site-specific ground motion analysis. Response spectrum scaling methodology was used to generate the synthetic spectrum-compatible ground motion time histories for site-specific response analysis. The design rock acceleration time histories for both transverse and longitudinal directions were developed for 1,000 year event. The 5% damped, spectrally-matched bedrock motions agreed closely with USGS probabilistic bedrock response spectrum after converting from time history to their corresponding response spectrum. The maximum spectral acceleration results (envelope) of all site response analyses were used to develop the recommended spectrum for design purposes to accommodate the uncertainties in differential soil stratigraphy. Site classes for foundations along the Pulaski Sky are summarized as shown in last Section.

The existing foundations include: 1) Spread footing at Piers 1 to 41; 2) Precast and cast-in-place concrete files at Piers 45 to 63 and Piers 101 to 108 (except Piers 50 and 52) with various pile cap sizes; 3) Timber piles at Pier 42 to 44 and 99 & 100; 4) Batter piles at west abutment; and 5) Single caisson at Piers 64 to 87 & pair caisson at Piers 88 to 98.

The foundation seismic vulnerability evaluation was based on results of liquefaction analysis and foundation nominal resistance analysis for each pier location. These vulnerabilities consist of changes to foundation demand due to seismic loading and effects of liquefaction. Due to the expanded subsurface information obtained in this study, areas susceptible to liquefaction were better defined resulting in fewer piers considered vulnerable to liquefaction than identified during the feasibility assessment phase. Liquefaction effects analysis includes seismic-induced settlement, down drag on deep foundation, and lateral spreading for 1,000 year seismic event.

Capacity-Demand Ratios for each foundation type were summarized. Almost all spread foundations failed in eccentricity (C/D ranging from 0.01 to 1.87) while many failed in sliding (C/D ranging from 0.12 to 8.33). All group pile foundations except one are satisfactory. All deep caissons are satisfactory if no liquefaction effect (C/D >2.0).

Based on the results of the preliminary vulnerability analysis, several foundations exhibited the need for possible remediation, pending investigation with final loading conditions from final design phase.

Several alternative mitigation methods are considered for the existing bridge: 1) to improve the subsurface liquefiable soils around the existing foundations; 2) to retrofit existing superstructures, substructures, and foundations to accommodate the predicted liquefaction and related ground movement; 3) for hazard other than liquefaction in spread footings, to install isolation bearings to mitigate seismic displacement and moment in substructures; to increase footing sizes; to tie-down anchor bolts along each side of footing, and so on. FHWA Manual and other alternatives will be sought to achieve cost-effective solutions in final design.

Summary

The discussion recommends earthquake hazard level of 1,000 year return period to be used for seismic analysis for cost-effectiveness. Although seismic response is not high, the vulnerability to seismic damage does exist due to poor condition of existing bridge and soil condition on the bridge site, which may require a retrofit action. Further study is needed to address the seismic retrofit questions and move on to determine final seismic retrofit design criteria.

Acknowledgments

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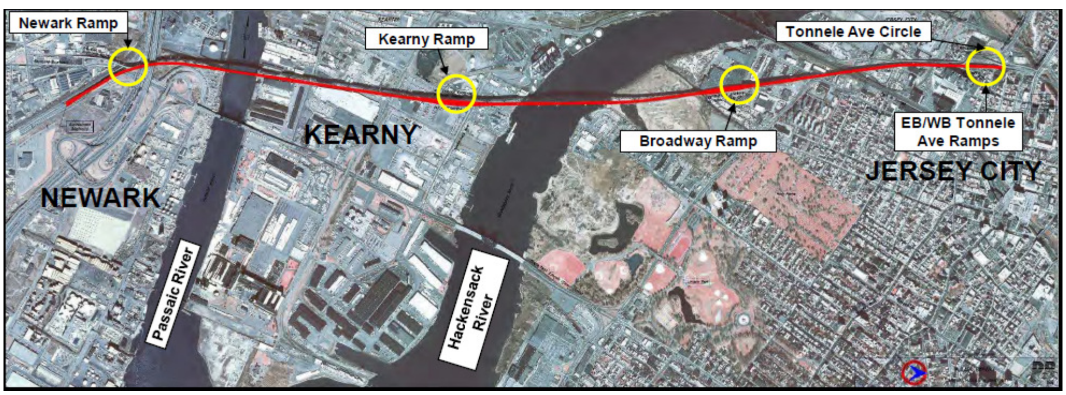
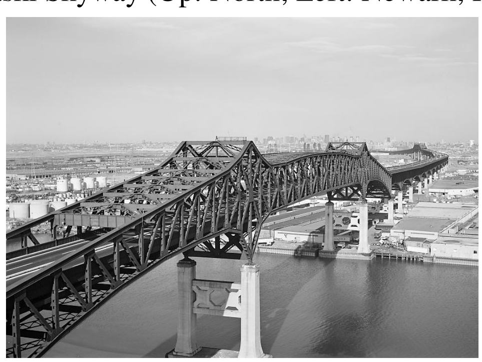
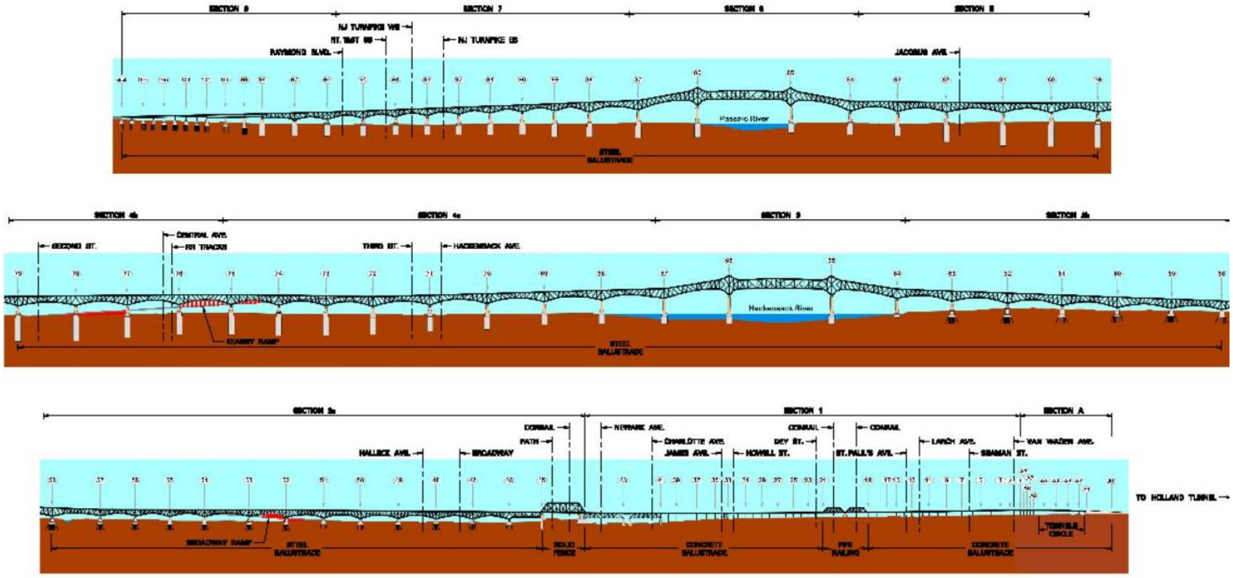


(a) Location of Pulaski Skyway (Up: North; Left: Newark; Right: Holland Tunnel)

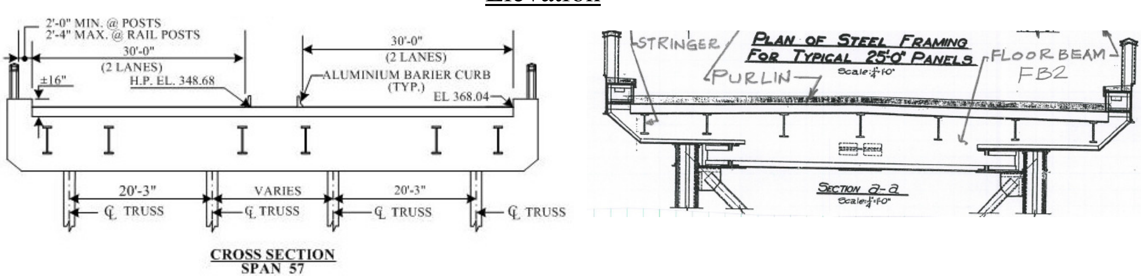


(b) General View of Pulaski Skyway

Figure 1 Pulaski Skyway Location and Overall View



Elevation



Typical Cross Section

Superstructure

Figure 2 Pulaski Skyway Bridge Elevation, Typical Cross Section and Different Superstructure

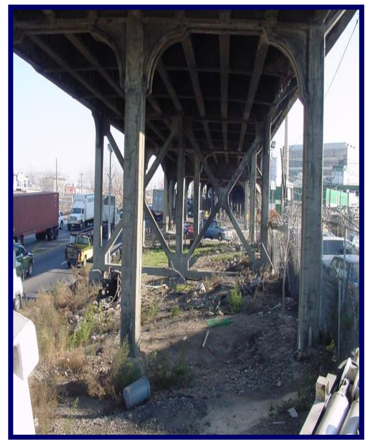
(a) Concrete Deck Spalling (underdeck): Left – Rebar exposure; Right – Cracking with efflorescence (chloride contamination)

(a) Encasement Loss in Stringer/Floorbeam and Steel Girder Encasement Removal

(b) Severe Rust and Section Loss of Truss Chord, Stringer and Floorbeam; Wear Pin Connection Hole and Pack Rust

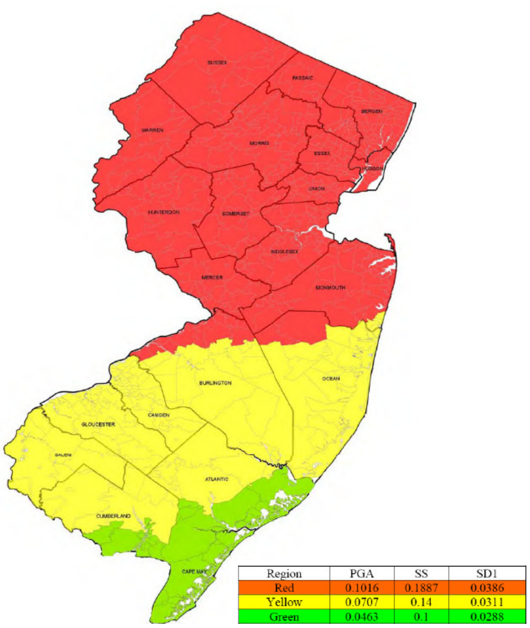
Figure 3 Superstructure Deterioration Examples

(a) Severe Concrete Scaling with Efflorescence in Pier Column, and Chipped Concrete and Expose Rebar at Bearing Seat



(b) Wide Concrete Cracks and Efflorescence in Column Pedestal and Previously Added Post Tension Rods and Beam Collars; Concrete Encasement Cracking

Figure 4 Substructure Deterioration Examples



Region	PGA	SS	SD1
Red	0.1016	0.1887	0.0386
Yellow	0.0707	0.14	0.0311
Green	0.0463	0.1	0.0288

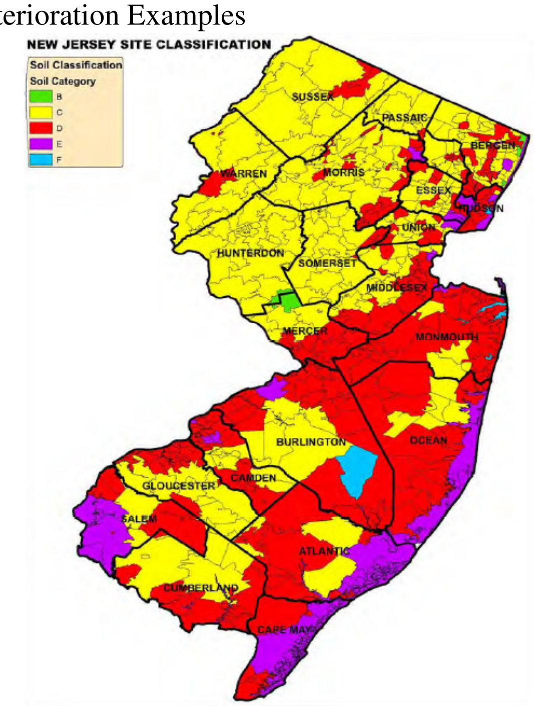


Figure 5 Seismic Map of New Jersey Using AASHTO/USGS 1,000 Year Return Period (left) and Soil Site Classification Map (right)

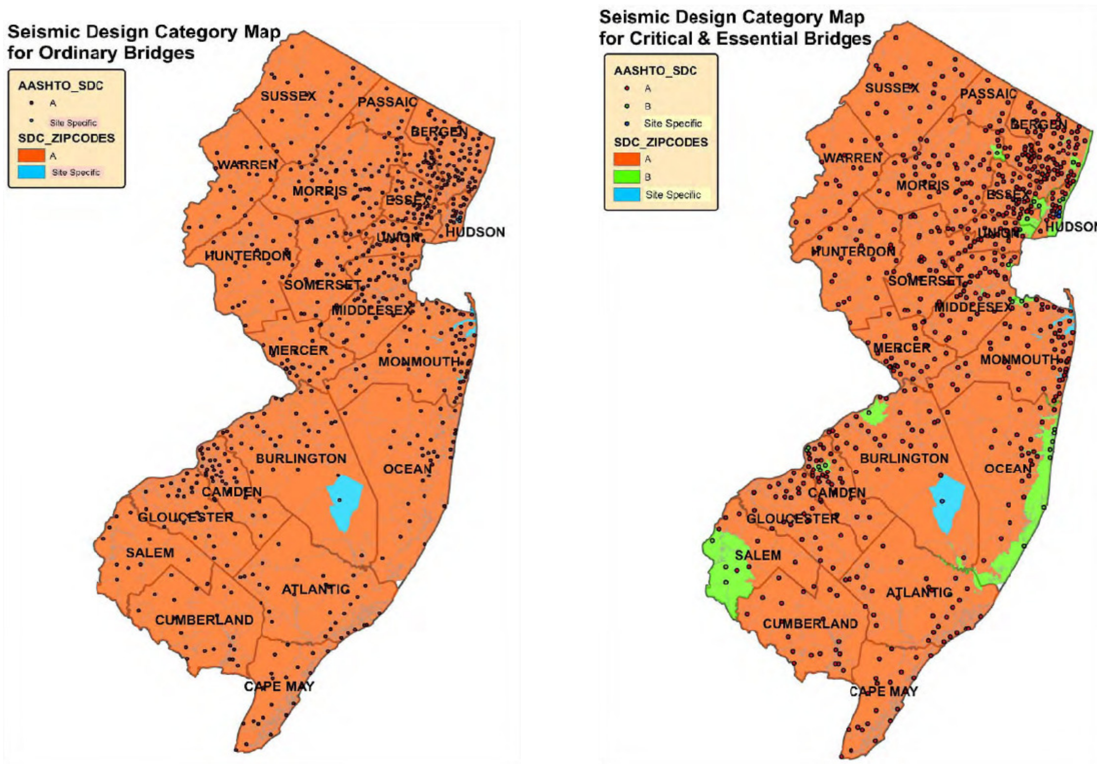


Figure 6 Seismic Design Category Maps for Standard/Ordinary Bridges and Critical/Essential Bridges in New Jersey

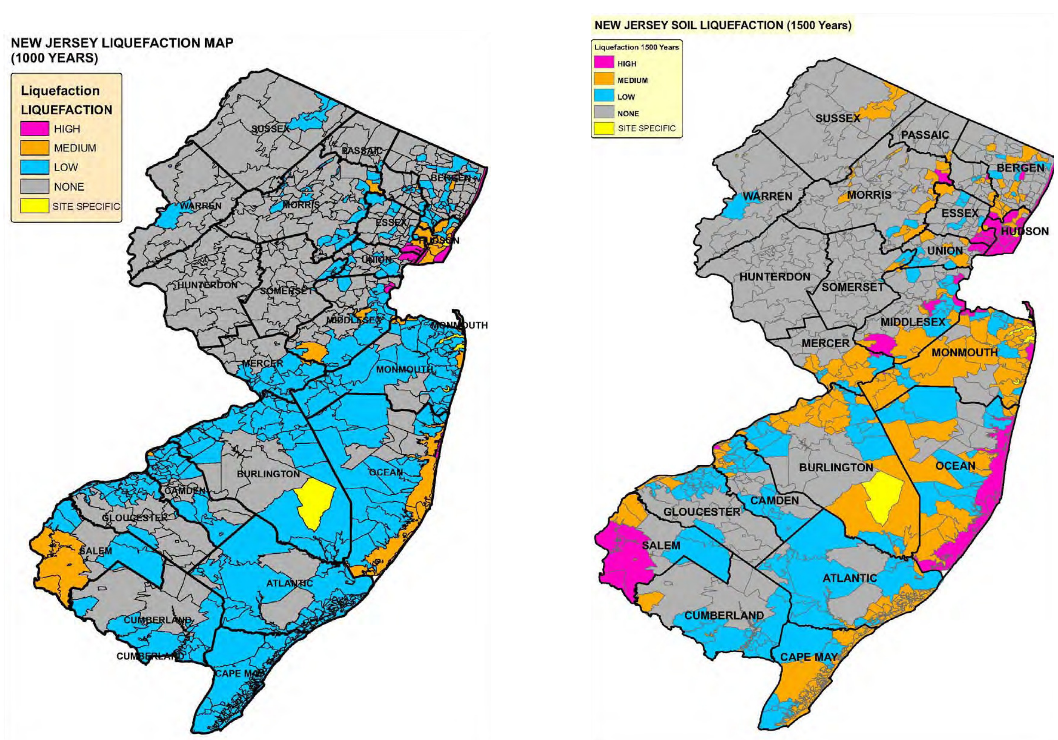


Figure 7 Liquefaction Hazard Map for Standard and Critical Bridges in New Jersey

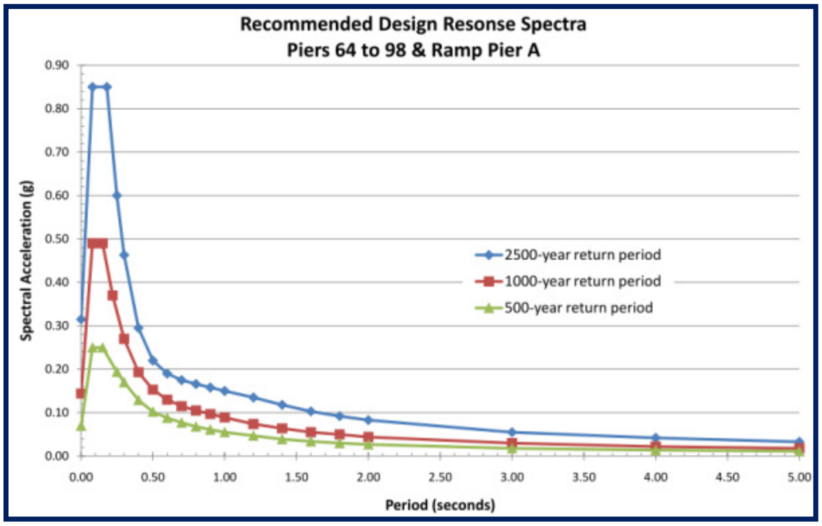


Figure 8 Recommended Design Response Spectrum for Piers 64 to 98

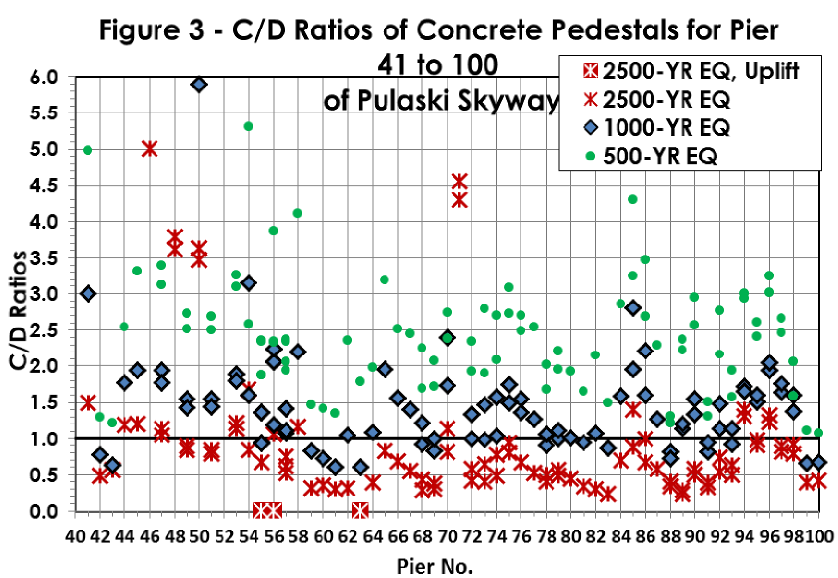


Figure 9 Seismic Capacity to Demand Ratio (C/D) Evaluation for Concrete Pedestals