3.2 Selection of the IBS Test Bridge

In order to properly assess the value of various technologies to help inform decisions regarding the renewal, preservation or replacement of existing bridges, it was crucial to select a bridge that displayed common performance concerns whose root causes and effective mitigation approaches were unknown. It is important to recognize that the goal was not to select the poorest performing bridge (where replacement is probably the only feasible option), but rather to select a bridge with common, significant performance problems that are difficult to address. To accomplish this, the project team worked closely with the New Jersey Department of Transportation (NJDOT) staff to identify a series of candidate bridges.

In all a suite of 15 bridges was identified by the NJDOT as bridges for which there was no clear path forward (i.e. they met the primary goal above). The secondary criteria for the test bridge selection included (1) the significance and number of performance problems, (2) the commonality of the bridge type/form and of the performance problems, (3) the availability of documentation, (4) the significance of load rating and inspection challenges, and (5) the ease of access. To assess these issues for each of the candidate bridges the project team reviewed past inspection reports, examined their relation to the overall bridge population within NJ (through the LTBP Project Bridge Portal), and performed site visits.

Following this criteria sister multi-girder steel stringer bridges that carry US 202/NJ 23 through Wayne, NJ (approximately 30 minutes outside of New York City) were selected (Figure 3-1). These structures were constructed in 1983 and 1984 and currently display very common problems associated with approach settlement, bearing alignment/walking, substantial vibrations and fatigue cracking. In addition, these structures have a variety of skew, partial skew and straight spans that traverse a park, which allows for unrestricted underside access. A more complete description of these structures is provided in Section 3.3.

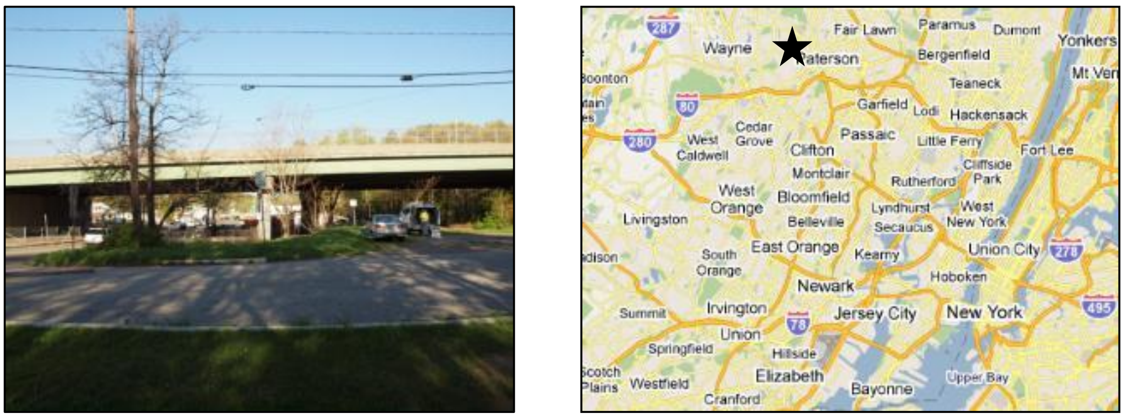

Figure 3-1 - US202/NJ23 International Test Specimen in Wayne, NJ

While in this case the test bridge was selected to best serve the goals of the IBS study, it does represent a very common approach to selecting structures for technology applications. That is, owners are not necessarily interested in testing their “worst” bridge, but rather a bridge that displays significant performance concerns for which the appropriate path forward is unclear. The other, and perhaps more important approach to selecting bridges for technology applications, occurs when a sizeable population of bridges have similar performance problems or there is significant uncertainty as to the best preservation/renewal practices for a population of bridges. In this case, a sample of bridges from the population in question may be selected with the goal of generating results that may be extrapolated and applied to the entire population. While such applications have the potential to pay huge dividends, there are very few entities with the knowledge and ability to select appropriate sample populations (both size and distribution) and then extract generic information from this sample which may be applied to the overall population.

As discussed previously the observation and conceptualization step is of paramount importance as it drives all of the modeling and experimental activities throughout the St-Id. To perform this step for the US 202/NJ 23 Bridge (Figure 3-1) the project team obtained relevant documentation (including design drawings and the previous two inspection reports), mined NBI data to place the bridge in context, performed a preliminary ambient vibration test, and computed nominal rating factors. This all of led to a qualitative risk assessment and ultimately to the development of a series of critical questions to guide the entire St-Id. The details of these steps are provided in the following subsections.

3.3 Overview of the Structure (Review of Design Drawings)

Each of the test bridges (constructed in 1983 and 1984) are comprised of four simply supported spans using a standard steel stringer design of eight girders with variable section properties and geometries including straight, partially skewed and fully skewed spans. From south to north, the bridges span Mountainview Blvd, an open field, train tracks, and an exit ramp. Due to constraints imposed by the railroad, no underside access to Span 3 was provided to the project team at any time during this study. An overview of the general bridge statistics is provided in Table 3-1.

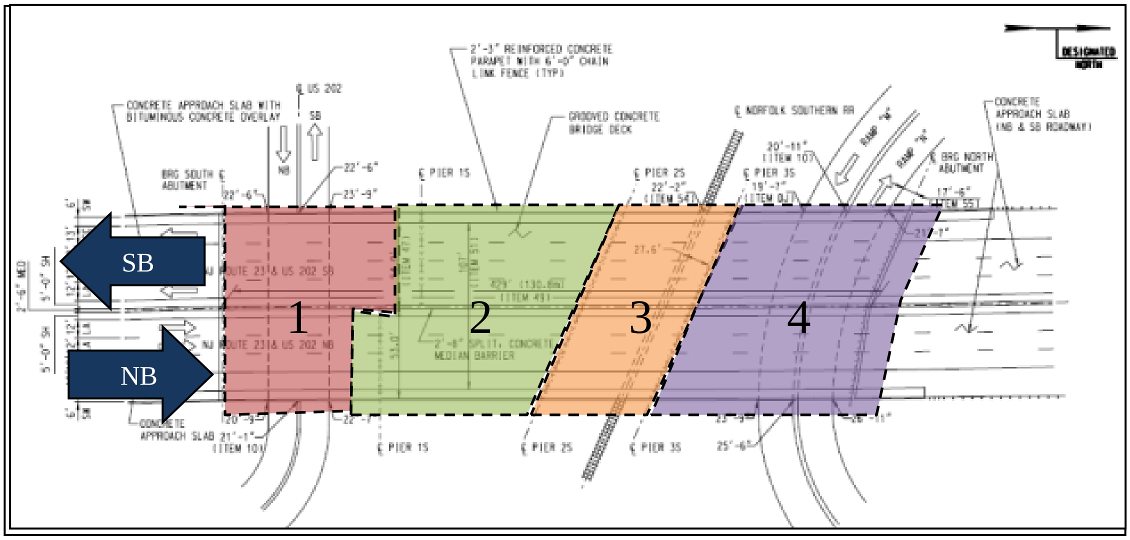

Figure 3-2 - Plan View of Bridge

Table 3-1 - General Bridge Information

Table of Statistics - International Bridge Study Test Structure									
Year Built	1983-1984								
# of Spans	4								
Span #	***Span 1***		***Span 2***		***Span 3***		***Span 4***		***Overall***
Direction	***NB***	***SB***	***NB***	***SB***	***NB***	***SB***	***NB***	***SB***	**-**
Length	105'	130'	130'	130'	70'	70'	130'	114'	429'
Width	61.75'	61.75'	61.75'	61.75'	61.75'	61.75'	61.75'	61.75'	123.5'
Skew Angle	0°	0°	0°/66°	0°/66°	66°	66°	66°/80°	66°	-
Clearance	22+'	22+'	22+'	22+'	22'	22'	22+'	22+'	-
Lanes	3	4	3	4	3	4	3	4	7
Deck Condition Rating			7 – Good (2008)						
Superstructure Condition Rating			5 – Fair (2008)						
Substructure Condition Rating			7 – Good (2008)						

The girders of both bridges are built up members with variable flange thicknesses. The change in flange thickness is a smooth, well-detailed transition that should not cause fatigue issues. The flange thickness varies from 1 in. to 2.5 in. depending on the girder length, with the top flange transitioning once and the bottom flange transitioning twice. This results in up to five different cross-sections on a given girder and adds to the overall complexity and irregularity caused by the varying skew conditions.

The decks of the two bridges were cast using stay-in-place forms which prevent any visual assessment of the condition of the concrete from the underside of the structure. Shear studs were provided in groups of two to three spaced at 15 in. to 21 in. along the girder, depending on the span and location. According to the design drawings, the studs are all 6 in. tall with a ¾ in. diameter.

The bridges contain diagonal wind braces (Figure 3-3) between the fascia and first interior girders on every span which are connected via a 'Category E' gusset-to-girder web detail, discussed in Section 3.4. The diaphragms are a standard truss-type composed of four single angles connected to the girders with bolted connections and gusset plates (Figure 3-4).

Figure 3-3 - Typical Wind Bracing for Fascia Girders

Figure 3-4 - Typical Diaphragms

Each span is simply supported, with pin and rocker bearings and preformed elastomeric compression seal joints.

3.4 Overview of Current Condition (Review of Inspection Reports)

According to the most recent inspection report available, the structures had an overall rating of 5 (Fair) due mainly to the condition of the superstructure. The inspection report pays particular attention to a series of fatigue cracks, which are present at the location of wind bracing-to-girder connections. These ‘Category E’ fatigue details, shown in Figure 3-5, have resulted in a total of 86 fatigue cracks, of which over 40 have propagated into the web. These 40 cracks were arrested by drilling a hole at the tip of the crack and inserting a bolt in the hole. Through the use of dye penetrant testing, it was determined that the arrested cracks have not progressed past where the hole was drilled.

In addition to the fatigue issues, the report notes bearing and joint deterioration and mentions a heavy vibration of the bridge under traffic loads. According to the inspection report, the bridge has an Average Daily Traffic (ADT) of 93,400 with 4% being truck traffic.

Figure 3-5 - Wind Brace Connection

3.5 Relation to Overall Bridge Population

To assess the relation between the test bridges and the overall bridge population with the state of NJ, the Nation Bridge Inventory (NBI) was mined using the LTBP Program's Bridge Portal. The Bridge Portal is a web-based data mining and visualization tool that allows easy access to the complete NBI (along with other data sources) and as such represents a powerful tool for assessing how a specific bridge fits within the overall population. To begin this process, the overall population is structured into a series of sub-populations based on parameters associated with structural form (referred to in NBI as "main design") and material (referred to in NBI as "main material"). It should be mentioned however that in the case of the material parameter the NBI includes 'continuous' as a possible modifier which has more to do with structural form than material. In any case, the sub-population that the test bridges fall within is simply-supported, steel stringer bridges (Figure 3-6), which represents a large portion (42%) of the total number of bridges within N.J.

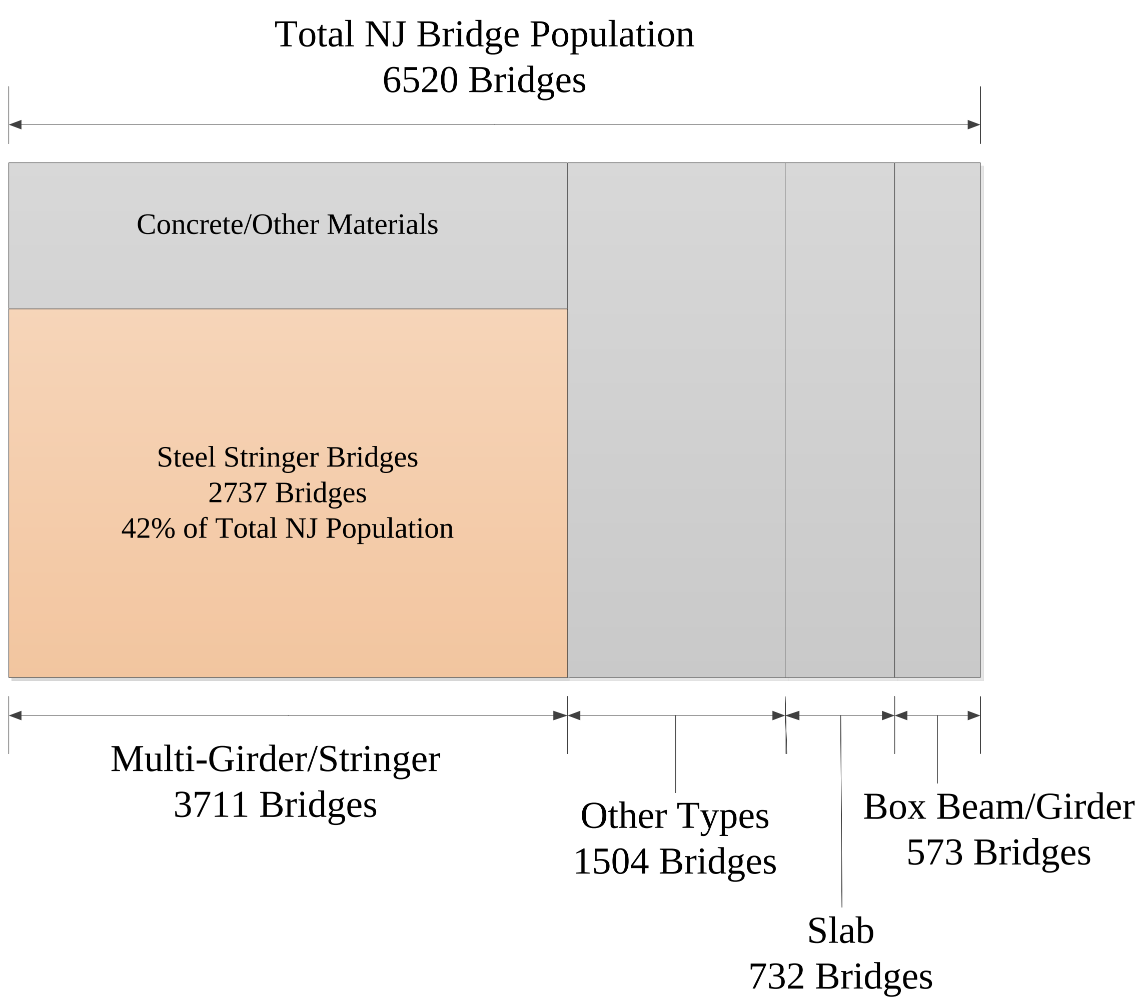

Figure 3-6 - Sub-population of the test bridges (All % of Total Population)

The second step of this process is to examine the distribution of key parameters within the relevant sub-population. This is accomplished by hypothesizing (based on heuristics) which parameters are expected to have the greatest influence on performance. One practical constraint on this step is that all influential characteristics must be represented in the NBI database (or another available database such as PONTIS). For the test bridges the parameters of age and maximum span length were selected, and the number of bridges in the various "bins" chosen for these parameters is shown in Table 3-2.

Table 3-2 - Distribution of the sub-population based on age and maximum span length

		Maximum Span Length (ft)			Total
		L < 60	61 < L < 150	151 < L	
Age (yr)	0 < Y < 20	31	97	14	142
	21 < Y < 40	60	**680**	49	789
	41 < Y < 60	309	775	19	1103
	61 < Y < 80	324	87	0	411
	81 < Y	269	23	0	292
Total		993	1662	82	2737

As shown in bold in Table 3-2, the 'family' of the test bridges contains 680 bridges or just over 10% of the total bridge population within N.J. As a result, it appears that the test bridges belong to a fairly large family of bridges (as defined by form, material, age, and span length). It is illustrative at this point to drill down and identify the ‘demographics’ of the bridge family to understand how the bridges of interest relate. It is useful to summarize this effort as a series of pie charts as shown in Figure 3-7 (with the test bridges shown as orange stars). Of course it is possible (and likely) that some parameters may have coupling effects, i.e., ADTT may become a more important parameter for older bridges. As a result, the “one-dimensional” representation shown may not suffice for all analyses.

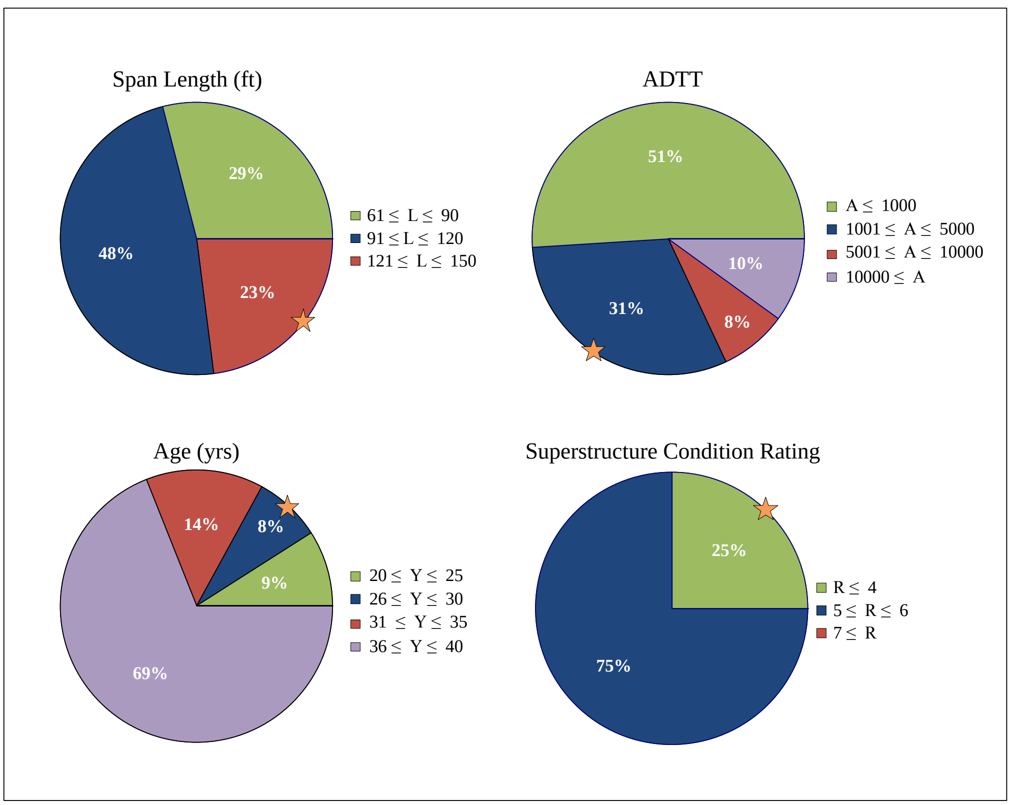

Figure 3-7 Distributions of the family of bridges based on (a) span length, (b) ADTT, (c) age, and (d) deck condition

Examining the pie charts shown in Figure 3-7, several observations about the test bridges can be made. First, while the bridges are quite young compared to the remainder of the family, they appear to be under-performing (based on superstructure rating). This is not surprising as these bridges were identified by NJDOT as outliers. In addition, it is observed that they have rather long span lengths and have a moderate to high ADTT compared to the rest of the family. While such information cannot be used to identify the root causes of the performance problems, it does place the bridges in context and may indicate how the results of the St-Id may be applied to other structures.

3.6 On-site Observations (Initial Field Visit)

During the first site visit, several important observations were made. While walking the span, heavy vibration of all spans was observed, though it was heaviest on Southbound Span 1 (SB1) and Southbound Span 2 (SB2). There was also transverse cracking of the top of the bridge deck at regular intervals visible on SB1. Otherwise the deck condition appeared satisfactory when viewed from above. However, without being able to see the deck from underneath, it is difficult to assess the quality of the deck and deterioration that may be limiting the ability of studs to enforce strain compatibility between the girders and deck. With the exception of the fatigue cracks previously discussed, no major performance problems were noted for the primary elements of the superstructure. This notwithstanding, there was one particular diaphragm connection that was missing two of the original three bolts and could be heard rattling under normal traffic loads.

Bearing condition varied significantly across the structure with some bearings exhibiting almost no corrosion or deterioration, while others appear so deteriorated that replacement may be required. In particular, the fascia bearings showed extensive corrosion, and in one case a fully cracked pintel was observed (Figure 3-8) that allowed the bearing to rotate about its vertical axis.

Figure 3-8 - Fully Cracked Pintel

In addition, the majority of the joints were either filled with debris or bulging up into the roadway. Examination of the piercaps showed that most joints allowed water to drain directly through on top of the pier. In addition, the pier between SB1 and SB2 had a very large vertical crack between the bearings for Girder #1 and Girder #2 (Figure 3-9). There was also some spalling and cracking visible on the piers, as well as extensive map cracking on the abutment walls.

Figure 3-9 - Crack in SB Pier 1

Also worth noting was the fact that noticeable ground vibrations were felt when standing under Spans 2 and 4. According to the design drawings the piers were founded on timber piles, but it was not possible to visually assess their condition. The approach slabs exhibited some settlement directly adjacent to the joint of the exterior spans of the bridge as well.

3.7 Preliminary Data Collection

Although not always required, it is very useful to perform a preliminary data collection effort during the site visit as this provides some quantitative information regarding bridge performance and can greatly help in developing the a priori model and designing the experimental study. Given the concerns surrounding the vibration of the spans, a preliminary ambient vibration monitoring of the bridge was conducted during one of the site visits. This study consisted of recording ten minutes of data from four accelerometers installed on the sidewalk of each span (so no traffic control was required). While a single setup is possible, this particular study was carried out by moving the four accelerometers from span to span and the total data collection effort took less than two hours.

Figure 3-10 and Figure 3-11 show typical acceleration time histories and power spectral densities for Span 2 northbound and south bound respectively. As apparent from these figures, the bridges experience significant acceleration under normal traffic loading (with 0.4 g the highest acceleration recorded). In addition, while both spans (which are nearly identical) have very well defined peaks (indicating low damping ratios), they display distinctly different natural frequencies (as shown in Table 3-3). In addition, the higher frequency modes of Span 2 northbound appear to be participating more than their counterparts for Span 2 southbound. While there are many potential reasons for this (such as the different locations of the sidewalks relative to the skew, or simply different traffic loading during the monitoring), at this point it is important to at least be aware that the vibration responses of the two bridges may have significant differences.

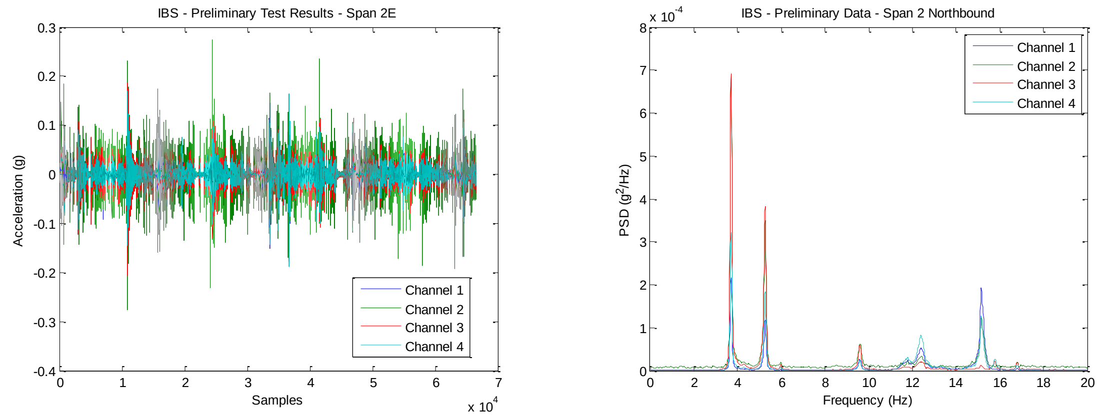

Figure 3-10 - Acceleration Time History and Power Spectral Densities for Span 2 NB

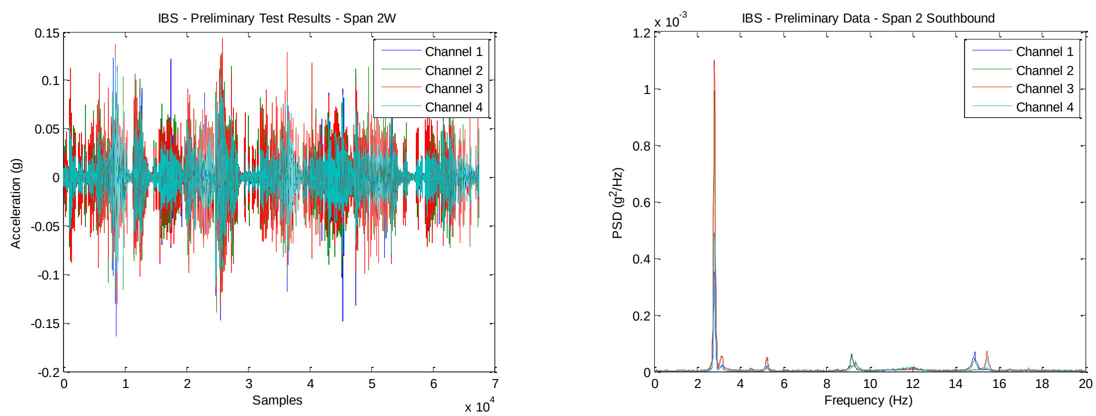

Figure 3-11 - Acceleration Time History and Power Spectral Densities for Span 2 SB

Table 3-3 - Identified Frequencies - Preliminary Data

Span 2 NB	Span 2 SB
3.699	2.808
5.261	3.174
5.981	5.249
9.595	9.167
12.380	14.880
15.150	15.440
20.860	

In addition to providing a quantitative basis for the conceptualization of the apparent vibration problem, this study also served to provide a sound basis for the error screening of the a priori model and the design of the vibration tests. Specifically, these results provide an indication of the bandwidth of interest and the level of ambient vibrations, which are important design parameters for the sensing and data acquisition protocols. In addition, given the large levels of vibration, it was noted that it will be difficult to perform forced vibration testing if the bridge is not completely closed as the excitation systems (impact or shaker) may not provide sufficient energy to overcome the ambient vibration level.

3.8 Preliminary Load Rating

The inspection reports provided by NJDOT include the most critical load rating values calculated using PennDOT's Bar7 Program (PennDOT 2008), which utilizes the Load Factor Rating (LFR) approach. The critical member listed in the inspection report is

Girder #5 on Span 1SB with ratings of 41 tons at inventory and 68 tons at operating for the HS-20 trucks. No access to the actual calculations or the software was provided.

As an independent check on the inspection report, preliminary load ratings were calculated using the AASHTO LRFD Bridge Design Specification (American Association of State Highway and Transportation Officials. 2007). The results of these ratings, in terms of rating factor, are presented in Table 3-4. Note that all ratings were governed by flexure.

Table 3-4 - Load Rating Factors

	HL-93		HS-20	
Span	**Inventory**	**Operating**	**Inventory**	**Operating**
SB1	1.50	1.95	2.21	2.86
SB2 (G8)	1.46	1.90	2.03	2.63
SB3	1.61	2.09	2.05	2.65
SB4	1.58	2.05	2.23	2.90
NB1	1.86	2.41	2.56	3.32
NB2 (G1)	1.29	1.68	1.90	2.46
NB3	1.61	2.09	2.05	2.65
NB4	1.50	1.95	2.13	2.76

As a direct comparison to the ratings provided in the inspection report, SB1 has ratings of 44.2 tons at inventory and 57.2 tons at operating. As these ratings were calculated using the LRFR approach, and not with Bar7 (LFR), it is likely that the discrepancy lies in this difference or in assumptions or simplifications made in either or both rating calculations. In fact, the LRFR calculations indicate that the most crucial member was Girder #1 of

Span 2NB. Incidentally this member has the same section properties as all of the girders on Span 1SB.

3.9 Qualitative Risk Assessment and Development of Critical Questions

Perhaps the most important portion of the first step of an assessment is the identification of potential limit states at which the structure may fail to perform in a satisfactory manner. Given that these limit states may include everything from functional issues through structural safety concerns, it is important that this be done in a risk-based manner. That is, the bridge in question (and the surrounding network, environment) should be examined to identify potential hazards (external inputs to the bridge), vulnerabilities (internal weaknesses), and the consequences associated with failures that may be mobilized by the identified hazards and vulnerabilities. Table 3-5 provides a partial listing of some common hazards, vulnerabilities, and exposures associated with various performance limit states.

Following this procedure a qualitative risk assessment was performed on the test bridges to identify potential risk factors. From a Safety: Geotechnical and Hydraulic view point it was determined that all hazards were quite low, and while the perceptible ground vibrations felt during the site visit indicated some loading of the foundation system, the potential for this to propagate into a safety issue was taken as negligible.

Table 3-5 - Summary of Performance Limit States, Hazards, Vulnerabilities, and Exposures for Bridges

Performance Limit States	Hazards	Vulnerabilities	Exposures

Safety: Geotechnical/ Hydraulic	• Flowing water
	• Debris and ice
	• Seismic
	• Vessel Collision
	• Flood
		• Scour/Undermining
		• Loss of support
		• Soil liquefaction
		• Unseating of superstructure
		• Settlement
		• Overtopping
			• Loss of human life
			• Replacement and repair costs
			• Impact of removal from service related to:
			  • Safety – life line,
			  • Economic
			  • Social – mobility
			  • Defense

Safety: Structural	• Seismic • Repeated loads • Trucks and overloads • Vehicle collision • Fire	• Lack of ductility and redundancy • Fatigue and fracture • Overloads • Details and bearings	

Serviceability, Durability and Maintenance	• Winter maintenance practices
	• Climate
	• Intrinsic Loads
	• Impact (Vertical)
	• Environment
		• Corrosion
		• Cracking/spalling
		• Excessive deflections/ vibrations
		• Chemical attacks/reactions
		• Difficulty of maintenance
			• User costs
			• Maintenance costs
			  • Direct
			  • Indirect – delays, congestion, etc.

Functionality	• Traffic • Special traffic and freight demands	• Network redundancy and adequacy • Geometry and roadway alignment	• Loss of human life and property (accidents) • Economic and social impacts of congestion

In the case of Safety: Structural limit states a few risk factors were noted. The largest hazard noted was the very high traffic and truck traffic loading (nearly 100,000 and 4,000 ADT and ADTT, respectively). While there is no quantitative information on overloaded vehicles, during the site visit numerous heavily loaded trucks were observed crossing the bridge. In addition, three primary vulnerabilities were noted including (1) the observed fatigue cracking at the location of the poor connection details of the wind bracing members, (2) the type (rocker) and significant deterioration of the bearings, and (3) the large single crack noticed in the pier cap, which likely indicates poor reinforcement details (due to the lack of distributed cracking). While the computed rating factors indicate that the bridge does not have a capacity issue, they do not provide any information about the potential for continued fatigue crack propagation, local bearing instabilities, or localized failures due to poor reinforcement details.

From a Serviceability, Durability and Maintenance standpoint, the primary hazards noted were associated with the climate (including freeze-thaw), the use of de-icing chemicals, and the repeated heavy truck loads the bridge experiences. On the vulnerability side, five primary issues were noted, including (1) the excessive vibrations under traffic loading (nearly 0.4g of vertical upwards acceleration), (2) the transverse cracking of the concrete deck, (3) the poorly performing bridge joints, (4) the corrosion of the girder ends and bearing assemblies, and (5) the map cracking within the abutments. While these risk factors are not likely to pose a safety issue, they do impose significant costs due to their negative impact of durability which shortens the service life of the bridge.

Finally, the functionality of the bridge was assessed and four risk factors were identified including (1) significant traffic levels (hazard), (2) approach slab settlement, which caused a significant bump at the beginning of the bridge (vulnerability), (3) relatively poor network redundancy (vulnerability), and (4) high economic importance as evidence by the large traffic levels (exposure). While in many cases it is not possible to use sensor and simulation technologies to improve bridge functionality, it is important to place the safety and serviceability risks in context with functionality when making a decision related to preservation and renewal.