Research Status Summary Report

Submitted to

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION

and

Federal Highway Administration

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1 Introduction and Summary

1.1 Background and Motivation

The writers started their research for WVDOT in 2005 and have received three subsequent contracts from WVDOT via the FHWA NDE Center. The third contract ended in October 2008 with a total funding of $377,723. In addition, the FHWA NDE Center and WVDOT spent funds for equipment and in-kind support.

The motivation for the research came about as a result of the Coal Resource Transportation System (CRTS) which was established in WV in 2003. The CRTS is comprised of routes in 15 southern counties in the state that allow permits for shipments up to 120,000 pounds based on the truck configuration. Meanwhile, from the population of over 600 bridges located on CRTS routes, approximately 100 must be posted for loads less than the desired CRTS live load level.

The National Bridge Inventory documents that WVDOT has 1163 cast-in-place reinforced concrete (RC) bridges in its inventory, with an average span of 50 ft and an average age of 70 years. Many of the CRTS bridges were of RC slab, RC Beam-Slab and RC filled-arch types, and older than 70 years. Some of the slab or arch bridges were subsequently widened by placing pre-stressed concrete side beams on the sides of the existing RC bridge. Many of these bridges lacked all or some part of their critical documentation, such as their foundation, substructure and superstructure reinforcing, connection, and bearing details. These documents are necessary for the determination of their safe load carrying capacity. Postings of bridges were being based on engineering judgment and WVDOT was concerned with their load capacity given the potential for repeated overloads by coal haulers.

The following research objectives were therefore identified:

1. To develop and demonstrate a methodology for reliably establishing safe load carrying capacity of aged bridges that are missing critical documentation through structural identification, i.e. the integration of analytical and experimental techniques (FE analysis, Material tests, NDE, Structural load tests)
2. Given the cost and time associated with a detailed investigation of each bridge, investigate screening approaches and associated experimental tools for classifying the bridge population in terms of their relative risk of failure so that detailed experimental and analytical investigation resources are dedicated to those bridges that are considered to have a relatively higher risk of failure.

This overview provides an account of what the writers have actually accomplished towards the above objectives and the most relevant conclusions they have reached in relation to the challenges that the agency faced in managing their bridge stock. It was important that the writers did not work in a detached manner from WVDOT and the FHWA NDE Center. They partnered with these agencies, striving to understand their special circumstances, concerns, interests and capabilities, and worked closely with individuals at the agency so that the generated research products would be acceptable, useful and worth their investment into the research.

1.2 Research Findings

During the three-year research project, the writers visited over a dozen bridges that were considered representative of the WVDOT RC bridge inventory, and performed in-depth studies on five of these bridges. These bridges were RC structures that were over 70 years old, showing various signs of aging, deterioration and damage. Three were single-span filled-arches, one was a single-span slab and one was a multi-span girder-slab system. The third single-span filled arch was widened by adding PC beams on each side. The writers submitted interim reports documenting their work on each of these five test bridges, and provided recommendations related to their repair, retrofit and rating.

Although five bridges out of nearly 1200 aged RC bridges in WVDOT's inventory cannot be considered as a sufficient sample, their investigation led to observations, findings and conclusions that can be qualitatively generalized to apply to the entire population of RC bridges. Here the writers summarize the progress made towards each of the two objectives, and then offer their observations and findings related to the entire inventory of WVDOT (and many other states) aged, cast-in-pace RC bridges that may be missing critical documentation. Various specific research products related to how WVDOT may evaluate bridges based on risk, and how RC filled-arch bridges may be rated in a simple manner by the agency are presented.

The writers are grateful to WVDOT and FHWA for their confidence and support, and hope that these agencies will permit them to complete the work they have initiated.

1.3 Methodology for Establishing Safe Load Capacity

When the writers started their research, there were no examples or guidelines for evaluating the load capacity of aged RC bridges with signs of deterioration and damage, especially when they were also missing documentation. The research led to the following steps that may be recommended for evaluating any cast-in-place short-and-medium span RC bridge missing documentation:

(1) Field observations of the bridge's performance under traffic by experienced bridge research engineers, close-range inspections, interviews of local engineers, and a study of existing documents are required to understand the behaviors that would be expected from the soil-foundation-substructure and superstructure system. Probable failure modes of the system and whether failure may be affected by existing deterioration and damage must also be assessed. An understanding of possible causes of any existing damage and how damage may affect the safety during a proof load test is critical for both data interpretation as well as safety. Experience as a bridge research engineer is essential for this step.

(2) Material sampling and testing, in conjunction with various NDE tools, helps to establish the minimum required information regarding the existence, the layout and the conditions of the reinforcement. Concrete properties and their variability at various areas of the structure must also be determined by core tests and petrographic analyses.

(3) Based on the information collected, a FE model that reflects the existing conditions of the structure should be constructed by an expert following best practices for model development and error screening. Using this model, the behavior under various load levels and the load capacity should be estimated and these simulations used to design the proof-load tests.

(4) A proof level load test should be carried out, which requires special trucks that are fitted on the bridge at various back-to-back or side-by-side configurations while measuring the critical strains and deformations. Proof-level load is defined as the load, if resisted with only working service level stresses and deformations can be considered to correspond to the long-term safe load capacity of the bridge. The writers developed a safe proof load test procedure based on the analyses in (3) and by using 25-50 sensors to monitor critical pulses of the bridge. Instrumentation of regions with damage is critical to determine if any of the damages may increase under repeated live loads as well as whether deterioration and damage may adversely impact the failure mode.

(5) Following the proof load test, a safe load capacity should be established through various means including the calibration of the FE model. Calibrated model serves for further analyses to reveal whether and how long the bridge may serve under loads below the proof load as well as those that may exceed the proof load. Based on these analyses, recommendations regarding how the bridge may be maintained, repaired and/or retrofitted for a long-term service life are formulated.

The writers believe that the effort outlined above in (1)-(5) is currently the best possible approach to evaluate the structural safety of a typical cast-in-place RC bridge missing documentation. The cost of a typical application by the researchers, requiring several weeks of effort by a team of three highly-trained and qualified bridge research engineers, supervised by a highly experienced senior research engineer, is estimated to be in the order of $75K. This amount does not include the in-kind support that is needed from WVDOT.

To put the cost of evaluating aged bridges missing documentation in context, we note that of the five bridges tested, four were found to have capacities several times larger than their posted limits and their posting could be removed as discussed in the following. Repairs to improve their durability and safety performance over a long term were formulated to be applied by WVDOT's in-house capabilities. Given that the high cost of replacing any of these four test bridges, even without considering user costs (cost of detours and congestion to users), the cost of evaluation should be considered feasible.

While four of the test bridges were proven to have a much higher level of safe load capacity relative to their posted levels, one of the bridges did not pass the test. The bridge that was widened by PC beams exhibited permanent deformations that were objectionable for its safe operation even at the posted load level. This bridge served Michigan Avenue in the town of Smithers, and the writers happened to observe much heavier trucks than the posted limits attempting to cross this bridge. The evaluation and the recommendations may have therefore mitigated a potentially dangerous structural failure, and mitigating such a risk is likely much more valuable to the agency than the funds spent on the evaluation.

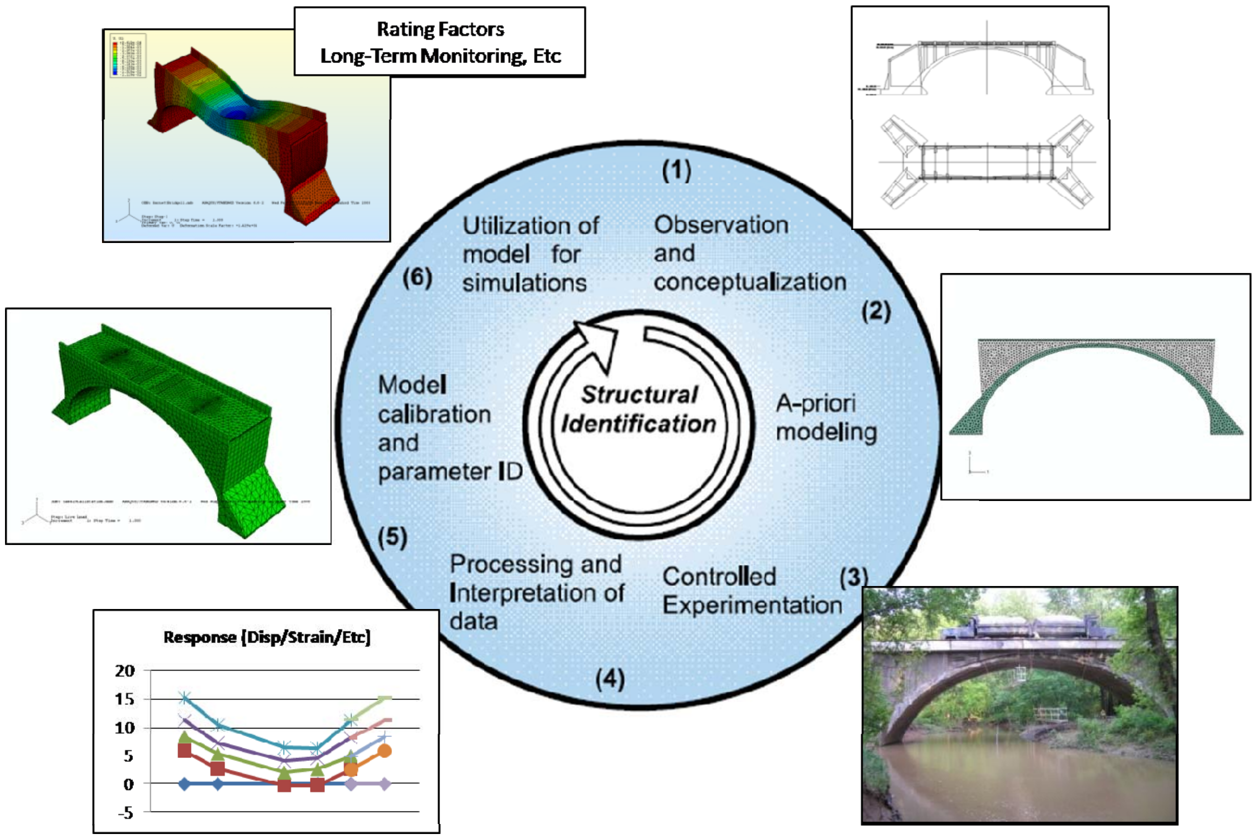


Figure 1-1 - Structural Identification Methodology

1.3.1 Conclusion Related to Objective 1:

The writers submit that there is value in developing and demonstrating a method for evaluating the safe load capacity of aged RC bridges missing critical documentation by leveraging proof-load testing and field-calibrated FE models. Although the average cost of an application, about $75K, appears considerable, a simple economic analysis reveals that this may be a feasible approach at least for bridges that are important for local economy.

Nevertheless the feasibility of a method often depends not only on what it saves but also whether there are funds that are available for applications of the method. It is not feasible for WVDOT to expend time and funds that would be required for proof load testing and evaluating all of its nearly 1,200 cast-in-place RC bridges with an average age of 70 yrs. Further, WVDOT is responsible for over 3,200 steel bridges with an average age of 40 yrs and 2,000 newer PC bridges less than 20 yrs old. The condition rating of several hundred of the steel bridges has been noted as 4 and below 4 in the NBI, placing these into the structurally deficient category.

It follows that WVDOT's concerns for objectively evaluating the actual safe load capacity of its bridges is not just limited to CRTS Bridges, or just to the RC bridges. While the evaluation method based on proof load testing and structural identification by field-calibrated FE models remains as the most reliable approach to evaluating bridges with missing documentation, it is also time-consuming and not yet readily available for implementation by many consultants. Therefore more practical and expedient approaches are required to supplement the proof load method and quickly and rationally prioritize the overall population of bridges. In this manner, proof level load testing will be used sparingly and only applied to a smaller number of highly critical bridges selected by a rational and reliable screening process.

1.3.2 Bridge Screening Concept

During the third year of the research the writers recognized the need for a screening approach that will permit identifying bridges that are at a higher risk of structural failure, and classifying the RC bridge inventory of WVDOT into low risk, medium risk and high risk categories in terms of structural safety performance.

In a similar project for PennDOT, the writers had successfully implemented statistical sampling to RC T-Beam bridges (Catbas et al, 2005). However, Pennsylvania T-Beam bridges were constructed from the same generic set of plans describing beam dimensions and reinforcing details based on span length. Therefore, there were no missing information and these bridges were indeed a type-specific family, lending themselves to statistical sampling based on a few parameters such as span, age, location, traffic, climate, etc.

West Virginia's RC bridges include slab, T-Beam, filled-arch, and filled-arch widened by adding PC beams on the sides. These bridges were not constructed from the same set of plans and many are missing their plans. Therefore, statistical sampling is not feasible without careful scrutiny of the entire population of the 1,200 RC bridges. The writers thought about a screening device that would apply a dynamic impact load at various locations of a bridge that would be comparable to a truck wheel load. This idea was based on early research conducted by Ohio DOT and FHWA, which revealed that it is possible to measure the deflection profile of a bridge under impact by proper instrumentation and data processing.

The writers consulted with WVDOT engineers about possibly leveraging the Falling Weight Deflectometer (FWD) device that is owned by the agency. They received support for modifying the FWD device to explore how dynamic impact testing by this device may be used for screening bridges based on their flexibility. Given that this was one of the objectives of the initial research contract, WVDOT approved this exploration as part of the Third Year contract. However, WVDOT subsequently requested that the FWD device not to be permanently altered and that it should maintain its calibration for testing pavements. Trials and efforts for modifying the FWD device are described further in the following, and these explorations led to the fabrication of an alterable stand-alone device that provided impacts of desired levels with the characteristics that were more suitable for recovering bridge flexibility than the FWD device.

Considerable progress was made towards testing bridges and recovering flexibility by the impact device as described in the following. However, the device needs additional improvements and data post-processing needs to be automated before the device may be provided to WVDOT for the agency to be able to use the device and obtain reliable results. This is a reason that the writers are submitting their request for WVDOT's consideration of the continuation of the research on rapid dynamic testing for bridge screening.

Even after developing a practical bridge impact test device, considerable effort for structuring and classifying the bridge populations within aged RC bridges in West Virginia will be needed. Populations need to be structured first in terms of their structural systems, such as filled-arch, solid slab, T-Beam, etc. Then, bridges in each sub-population should be evaluated based on the risk they pose, as discussed and exemplified in the ensuing sections. Finally, a number of the bridges with higher risk should be selected for proof-testing, while the remainder should be tested by rapid impact test device as a complement to their biennial inspections. This is the most rational and reliable manner of accelerating the evaluation of WVDOT's aged, posted and structurally deficient bridge stock.

One of the limitations in rapid impact testing was that it proved ineffective for testing filled-arch bridges unless the impacts could be applied directly on the main load carrying mechanism, the arch. Given that filled-arch bridges are very difficult to analyze, the writers have developed a simplified procedure for their analysis based on the three specimens that they have tested during this research project. This method was developed so that it may be applied by WVDOT's bridge evaluation engineers, and is described in detail in Appendix C.

1.3.3 Conclusion Related to Objective 2:

A general methodology for re-classifying the aged RC bridge inventory (also applicable to structurally deficient steel and PC bridges) has been formulated during the three year research while the writers evaluated five of WVDOT's posted RC bridges by proof-testing. This methodology requires several analytical and experimental tools that are not common in bridge engineering practice. These tools include a risk-based classification of RC bridges, a rapid impact-based bridge test device that will provide a means of processing data to obtain the deflection basin under impact for bridge flexibility, and a simplified analysis method for filled-arch bridges which exhibit the most complex of the structural behavior mechanisms amongst the various types of RC bridges in the WVDOT's inventory.

These tools have been described and demonstrated further in the following, noting the need for additional work for finalizing the dynamic impact test device and the automation of post-processing its data to generate bridge deflection basins.

2 Research Products

Several tangible and beneficial products resulted from the joint research efforts of FHWA, WVDOT and Drexel University. These products include load ratings, removal of postings, rehabilitation strategies, and some long-term monitoring which all correspond directly to the load testing. The load rating process highlighted the difficulty in rating of arch bridges, which resulted in an arch load rating method. Additionally, the concept of rapid bridge condition assessment using impact excitation was developed and proven to be effective.

2.1 Changes to Bridge Postings

Based on the results of the load tests, four of the five structures were cleared to remain operational without posting, considering minor maintenance and repair. The fifth structure was already slated for replacement, and the investigations indicated that this structure was of immediate concern and should be posted even lower, with all efforts made to replace it as soon as possible.

Figure 2-1 - Barnett Bridge (Posting was Removed)

2.2 Load Ratings

Each bridge tested was load rated based on both the NCHRP Proof Load Test procedures and the developed finite model utilized during testing. These load ratings take into account numerous reductions in uncertainty associated with the structures, providing a much more accurate load rating than typically calculated. With the exception of the Michigan Avenue Bridge, the load ratings all indicated substantial reserve capacity compared with the preliminary ratings calculated based on AASHTO.

Figure 2-2 - Tams Slab Bridge

Figure 2-3 - Amigo Arch Bridge

Figure 2-4 - Barnett Bridge

Figure 2-5 - Michigan Avenue Bridge

Figure 2-6 - Smithers Bridge

2.3 Rehabilitation and Long-term Monitoring

For each structure tested, an individual repair and rehabilitation strategy was developed which focused on the specific vulnerabilities and hazards associated with that structure. These repairs ranged from minor repair of the pier caps on the Smithers Bridge to reconstruction of the deteriorated arch ring at Barnett Bridge. Due to the severity of the deterioration, in conjunction with the observed movement of the spandrel walls at Barnett Bridge in the past, a long-term monitoring system was recommended. This was installed by Drexel personnel and is currently being monitored by WVDOT, replacing a prior manual string-line monitoring system.

Figure 2-7 - Long-term Monitoring

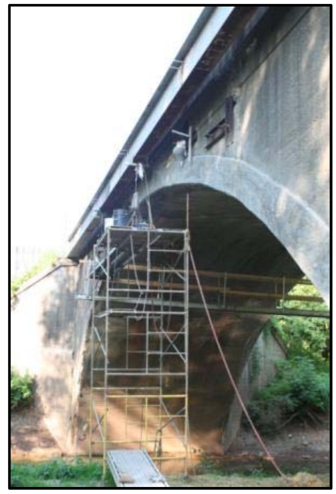


Figure 2-8 - Repair to Barnett Bridge

2.4 Rapid Bridge Condition Assessment

The entire research process indicated to Drexel that there was a need for a method to help define which bridges needed load testing and further investigations. This idea led to the development of the rapid bridge screening concept. The proof of this concept was achieved through the results of the load tests which were compared with dynamic investigations to simulate the methodology of rapid bridge screening using impact testing. While the Falling Weight Deflectometer (FWD) was not fully converted into a bridge screening device due to limitations in allowable modification as well as a shift in the focus of the research mid-project, the concept has been proven as valid for many types of structures. This is summarized in Appendix D – Rapid Bridge Condition Assessment – Proof of Concept.

Figure 2-9 – FWD on the Bridge for Testing

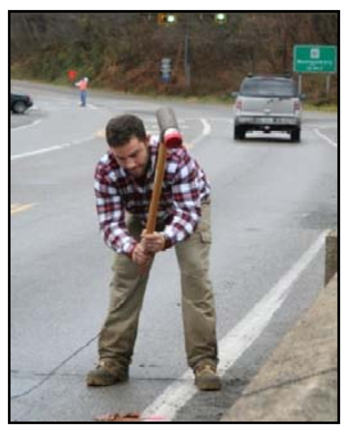
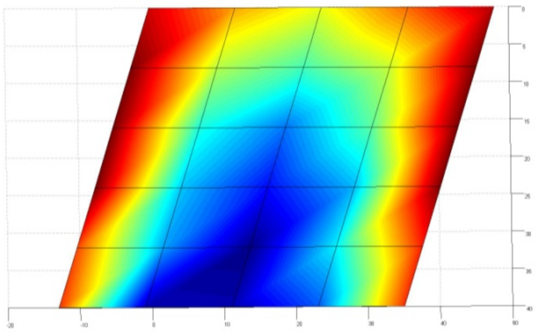


Figure 2-10 – Manual Impact at Smithers Bridge

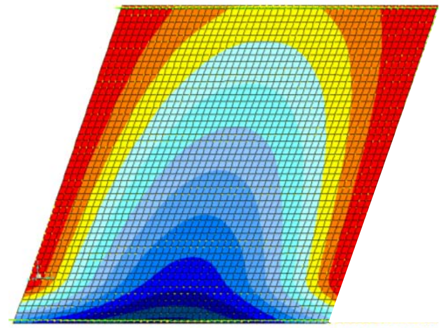
Mode #1

Dynamic Test Results



11.3 Hz

SAP2000 Model Results



11.4 Hz

Figure 2-11 - Rapid Bridge Screening Assessment Proof of Concept at Smithers Bridge

2.5 Arch Load Rating Method

During the load testing process at Barnett Bridge, it became apparent that rapid bridge screening using impact testing was not an applicable technology for filled concrete arch bridges. These structures utilize a mechanism of load distribution that negates a principle assumption in rapid bridge screening; excitation of the main structural components. The impact to the surface of the structure was found to not excite the main structural system. This is due to the inherent damping in the fill material and the overall, complex interactions of the components of a filled arch bridge. It was also discovered that there is immense uncertainty with filled concrete arch bridges in terms of load ratings, making the process highly difficult and variable. Therefore, in an effort to assist in prioritizing which arch structures need further in-depth investigation, a method of load rating for arch structures was developed and is presented in Appendix B – Arch Load Rating Method. This method takes into consideration the geometry of the arch structure, and how that geometry affects the dead load distribution and the transfer of load to the foundations. The method makes several simplifying assumptions which must be verified before application. Used in conjunction with a condition assessment, this method can be used to prioritize arch structures in much the same manner that a rapid bridge screening device could for other types of structures.

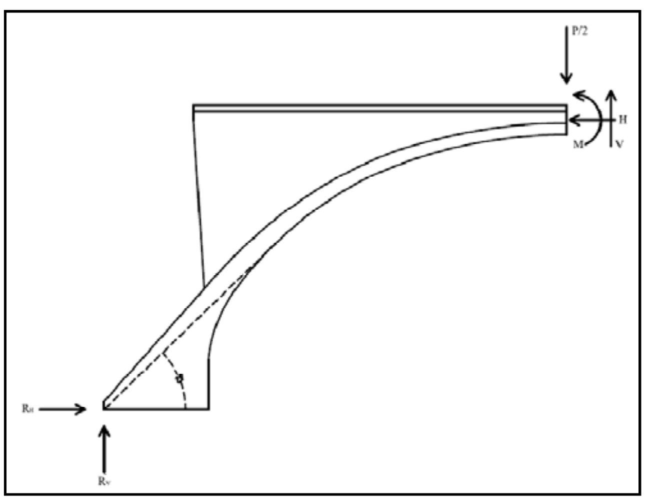


Figure 2-12 - Arch Load Rating Schematic

3 Load Test Lessons Learned

There are several practical lessons which resulted directly from the load testing experiences. These lessons are organized based on the stage of the process to which they are applicable. They are included below:

1. Initial Site Visit
    a. Quantify damage as much as possible
    b. Scout out locations for DAQ
        i. Accurately determine the distance to the DAQ locations from all over the bridge
        ii. Consider power, tent location, extra personnel etc
    c. Determine presence of all utilities
2. Initial Modeling
    a. Develop a suite of models
        i. Simple 1D elements through 3D solids
            1. Quantify model run time and construction effort
        ii. Run parameter sensitivity studies to help guide instrumentation design
3. Equipment Prep
    a. Measure, cut, label and solder cables at the lab based on selected DAQ site
    b. Design cable run layout prior to test
4. Installation and Pre-Test
    a. Connect all sensors and trouble shoot before the day of the test
        i. Schedule extra time for this purpose
    b. Keep detailed daily logs of work completed etc
5. Test Day
    a. Schedule so that down time in static test can be spelled with dynamic testing
    b. Develop simple visuals to present data to outside personnel

Additionally, there would be a great benefit in developing a single database of the population of bridges for which the DOT is responsible. The catalog should be a digital record of all of the assets for which the State maintains and should be searchable and sortable based on numerous parameters. This will allow WVDOT and Drexel to determine which structures are physically "testable."

4 Risk-based Assessment for Prioritization of Load Testing

The overarching lesson learned throughout the load testing conducted over the past 3 years is one of organization and prioritization of the structures that make up the population of bridges in West Virginia. Typically, in the past, the maintenance division sent out a request to the chief engineers of each district asking for potential structures for extensive load testing. The results of this initial survey were reviewed by the maintenance division to pull out the best candidates in their opinion. From that point, a site visit of the remaining structures was conducted by Drexel and WV personnel to select the final structure for testing. Based on the experience of the Drexel personnel on the most recent bridge survey it was determined that many of the structures were not in dire need of in-depth investigation. This is not to say that the test at Smithers was frivolous, as the results of the test aided WVDOT in decision making related to the removal of the posting as well as repairs and maintenance. However, the authors feel that other candidates, similar to Barnett Bridge, may present a larger vulnerability and provide a greater return for WVDOT. This conclusion led to the realization that the selection process should be altered from the ground up to ensure that the selected structures provide the most return. A proposed method of assessment and prioritization based on risk is presented.

The movement towards prioritization considering a risk-based approach is already established. The 110th U.S. Congress considered legislation entitled "National Highway Bridge Reconstruction and Inspection Act" which, amongst other things, would have required states to "assign a risk-based priority for… bridge[s] after consideration of safety, serviceability, and essentially for public use." While this was not passed into law, similar provisions are expected in transportation Bills in the 111th Congress. The details of a risked-based prioritization are not specified in the legislation, but the basic elements are already defined in the literature. The aim of this discussion to highlight the basic framework for objective, quantitative risk-based prioritization of a population of structures for the purpose of making informed decisions about management and funding allocation. The general framework will be viable both in terms of addressing the selection of bridges for extensive testing and investigation, but also for everyday management of the population, assisting in decision-making and allocation of funding for maintenance and repairs. It should be noted that considering the novel state of this approach, changes and improvements resulting from real-life experience and additional exposure to other sources are imminent and necessary.

The proposed approach to prioritization is in essence, a definition of risk as a function of several variables. These are:

1. Hazard – The probability of a hazard, H, occurring = p(H)
2. Vulnerability – the probability of failure (to perform adequately) given a hazard, (H) = p(f|H)
3. Exposure – consequences associated with a failure to perform adequately
4. Uncertainty Premium – a factor to account for the level of uncertainty associated with the selected assessment approach, including the quality control measured employed

Risk then is defined as:

(1) Risk (H) = (Hazard) x (Vulnerability) x (Exposure) x (Uncertainty Premium)

Risk will be quantified by a simple Level I-V scale, which indicates the relative level of risk with Level I being Low Risk and Level V being Severe Risk. This is easy to understand for both engineers and the public alike, and does not induce any unwanted concerns, as the term "structurally deficient" did after the attention on bridges and inspection procedures resulting from the collapse of I-35 in Minneapolis.

In order to apply this methodology to a real population of structures, the variables described above must be quantitatively defined to allow for numerical calculation or risk. Additionally, the framework of the step-by-step process through which the prioritization will take place should be laid out at least in a preliminary manner.

Table 4-1 provides a summary of relevant performance limit states, hazards, vulnerabilities, and exposures for bridges. These are general guidelines which should be specialized based on the population and the governing body (WVDOT) in terms of any particular hazards, vulnerabilities, or exposures.

Table 4-1 - Summary of relevant 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 and Cost	• 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

Figure 4-1 shows the proposed risk-based prioritization framework. The general procedure is to first define the level of risk assessment, which in turn, defines the uncertainty premium which is deemed acceptable. Next the estimation of risk is conducted by defining the hazard, vulnerability, and exposure. The risk level is calculated and if it is acceptable then WVDOT can begin to make informed decisions about management and resource allocation. If the level of risk is unacceptable then an intervention can be conducted, or a more refined risk analysis can be conducted in hopes of achieving an acceptable level.

Table 4-2 shows the uncertainty premium associated with varying degrees of assessment levels. The major deciding factor in the uncertainty premium is whether the risk is computed in an aggregate manner or divided up into individual risks. Computing aggregate risk is both efficient and conservative. However, there is often so much conservatism inherent in this approach that it can drastically over-estimate the actual risk. Therefore, in order to obtain a more realistic, accurate assessment of risk, it is often worthwhile to assess risks based on individual hazards.

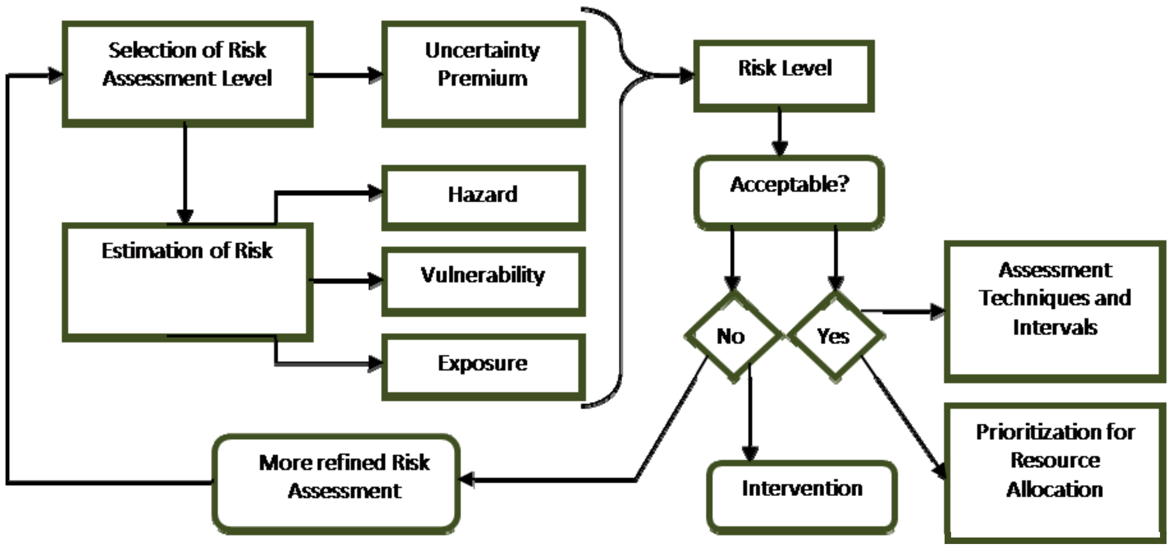


Figure 4-1 - Envisioned Risk-Based Assessment Framework

Table 4-2 - Risk Assessment Levels

Level	Example Approaches	Resolution	Quality Assurance	Uncertainty Premium
1	Visual Insp., Doc. Review	Aggregate Risks	Min Standards	2.5
2	Visual Insp., Doc. Review	Aggregate Risks	Best Practices	2.0
3	Visual Insp., Doc. Review, Anal. Tech.	Individual Risks	Min Standards	1.5
4	Visual Insp., Doc. Review, Anal. Tech.	Individual Risks	Best Practices	1.25
5	Visual Insp., Doc. Review, Anal. and NDE Tech.	Individual Risks	Best Practices	1.0

The assessment levels also reflect the specific approaches and technologies employed. Since the NBIS was initially developed, numerous analytical and experimental technologies that can reduce the uncertainty associated with assessment activities have become available. Further, there are a wide range of successful quality assurance programs that have been developed. To recognize their influence and benefits, assessment levels that take advantage of these developments will have a lower uncertainty premium associated with them. The owner is afforded the freedom to choose from a wide range of assessment approaches, but the standards will explicitly recognize the inherent differences in the resulting uncertainty, and thus will promote the use of best practices and proven technology.

Table 4-3 through Table 4-5 provide an illustration of how hazard, vulnerability and exposure may be quantified for Level 1 and 2 assessments. In this case, the risks are aggregated in four categories: Safety – Geotechnical/Hydraulic; Safety – Structural; Serviceability, Durability, and Maintenance; and Operational and Functional. For each of these categories, the hazard, vulnerability and exposure is given a value of 1-3 based on location, structural and operational attributes, age, etc. In the case of Individual Risk Assessments (Levels 3-5), these categories would be further divided to allow the risks associated with each individual hazard to be assessed independently. The aggregate risks are then computed as shown in Equation 1; and combined by square-root-sum-of-squares to develop the Risk Level. A preliminary scale of Risk Levels is shown in Table 4-6.

Table 4-3 - Preliminary Hazard Values for Level 1 and 2 Risk Assessments

Hazards Considered		Hazard Values 1	Hazard Values 2	Hazard Values 3
Safety: Geo/Hydraulic	Scour; Debris and ice ; Vessel Collision; Seismic - Liquefaction; Settlement; Flood	•Outside of a 500 yr flood plain; •Seismic Design Category A; •Over a non-navigable channel; •Located more than 500 miles from coast; •No potential for scour; •No records of significant earthquake, floods or storm surge;…	•Outside of a 100 yr flood plain •Seismic Design Category B, C •Navigable channel for mid-sized vessels •Located more than 50 miles from coast •A rating of NBI Item 113 (scour) of 7, 5, or 4 •Records of moderate earthquake, floods or storm surge;…	•Within of a 100 yr flood plain; •Observed drift and debris at piers/abutment; history of ice flows in waterway; •Seismic Design Category D, E, F; •Navigable channel for large vessels; •Located within 50 miles from coast; •A rating of NBI Item 113 (scour) of 6, 3, 2, or 1; •Records of significant earthquake, floods or storm surge;…
Safety: Structural	Seismic; Fatigue; Vehicle Collision; Overload; Fire	•Seismic Design Category A; •ADTT less than 500; •Not spanning over a roadway; •Located more than 10 miles from heavy industry; •No history of overloads, collision, earthquake;…	•Seismic Design Category B, C; •ADT less than 10,000; •Spanning over a roadway with ADTT less than 1,000; •Located more than mile from heavy industry; •History of isolated overloads, collision, and moderate earthquakes;…	•Seismic Design Category D, E, F; • ADT more than 10,000; •Spanning over a roadway with ADTT more than 1,000; •Spanning a rail line; •Located less than mile from heavy industry; •History of repeated overloads, collision, and significant earthquakes;…
Serviceability and Durability		•No routine use of deicing salts; •Located more than 100 miles from the coast; •Low number of freeze-thaw cycles; •No history of overloads; …	•Moderate usage of deicing salts; •Located more than 25 miles from the coast; •Moderate number of freeze-thaw cycles; •History of isolated overloads;	•High usage of deicing salts; •Located less than 25 miles from the coast; •Moderate number of freeze-thaw cycles; •History of repeated overloads and permits; …
Operations		•ADTT less than 1,000 and ADT less than 10,000; •No history of fatal accidents; •No history of congestion; …	•ADTT less than 10,000 and ADT less than 50,000; •History of isolated fatal accidents; •History of moderate congestion; …	•ADTT more than 10,000 and ADT more than 50,000; •History of repeated fatal accidents; •History of high congestion; …

Table 4-4 - Preliminary Vulnerability Values for Level 1 and 2 Risk Assessment

Vulnerabilities Considered	Vulnerability Values		
	1	2	3
Safety: Geo/Hydraulic	•Founded on deep foundations or bedrock; •Meets current pier impact and scour protection standards; •No history and no evidence of scour or settlement; …	•Founded on shallow foundations on cohesive soil; •Evidence of minor scour/undermining during past/present underwater inspections; •Pier protection system in good condition; •Superstructure above 100 yr flood level; •Minor tilt of substructure elements; …	•Founded on shallow foundations or non-cohesive soil; •Evidence of moderate to significant scour/undermining during past/present underwater inspections; •Pier protection system missing or in poor condition; •Superstructure below 100 yr flood level; •Significant tilt of substructure elements; …
Safety: Structural	•Meets all current design specs; •Structure displays bi-directional redundancy; •20 years or less since construction or major renewal; •A and B fatigue details; •No evidence of structural damage; •No history of excessive displacements or vibrations; …	•Simply-supported constructed with transverse distribution capabilities; •50 years or less since construction or major renewal; C and D fatigue details; •Minor evidence of structural damage within the critical load path; •Clearance within 6 in of current standard; •History of significant displacements or vibrations; •Substructure elements within 10% of plumb…	•Non-composite construction; •Simply-supported construction with minimal transverse distribution capabilities; •50 years or more since construction or major renewal; •E and E' fatigue details; •Rocker bearings; •Intrinsic force dependency; •Exposed prestressing strands; •Pin and hanger details; •Evidence of structural damage within the critical load path; •Clearance below current standards; •History of excessive displacements or vibrations;
Serviceability and Durability	•No visible cracks; •No evidence of reinforcement corrosion; •Elastomeric bearing; •Joints in good operating condition; •Paint in good condition; •Scuppers are less than 10% clogged …	•Minor local cracking; some evidence of reinforcement and structural steel corrosion; •Paint in moderate condition •Joints with minor evidence of leaking; •Approach displays minor rutting; •Scuppers are between 10-50% clogged …	•Extensive cracking and spalling; •Evidence of wide-spread reinforcement and structural steel corrosion; •Paint in poor condition •Exposed prestressing strands; •Frozen bearings; •Failed expansion joints; •Approach displays significant rutting; •Scuppers are between 50-100% clogged…
Operations	•Roadway approach alignment and bridge geometry up to current standards; •Guard rail and road paint in good condition; •Good ride quality of deck; •Breakdown lane/ shoulders; …	•Lane width within 1 ft of current standards; •Guard rail and road paint in fair condition; •Posted for more than 90% of legal truck weight; •Moderate ride quality of deck •Breakdown lane/ shoulders not present; •Minor rutting of pavement …	•Lane width more than 1 ft less than current standards; •Guard rail and road paint in poor condition; •Posted for less than 90% of legal truck load; •Breakdown lane/ shoulders not present; •Poor ride quality of deck •Significant rutting of pavement…

Table 4-5 - Preliminary Exposure Levels for Level 1 and 2 Risk Assessments

Exposure Considered	Exposure Values		
	1	2	3
Safety: Geo/Hydaulic Safety: Structural	•ADT less than 10000; •Replacement cost less than $2 million; •Not on a critical route (life line, evacuation route); •Detour route less than 5 miles;	•ADT less than 50000; •Replacement cost less than $10 million; •Not on a critical, non-redundant route (life line, evacuation route); •Detour route less than 10 miles;	•ADT more than 50000; •Replacement cost more than $10 million; •On a critical, non-redundant route; •Detour route more than 10 miles;
Serviceability and Durability	•Low maintenance costs; •ADT less than 50,000	•High maintenance and repair costs; •ADT more than 50,000	Not Applicable
Operations	•No history of congestion; •ADT less than 25,000; •ADTT less than 10,000	•Average peak hour delays of more than 10 min; •ADT more than 25,000; •ADTT more than 10,000	Not Applicable

Table 4-6 - Preliminary Risk Levels

Risk Level	Threshold Risk Values
Level V: Severe risk bridges	>40
Level IV: High risk bridges	30-40
Level III: Significant risk bridges	20-30
Level II: General risk bridges	10-20
Level I: Low risk bridges	<10

To translate the Risk Level into appropriate assessment techniques and intervals, a set of minimum requirements and optional assessment programs is needed. A preliminary estimate of this relationship is shown in Table 4-7. The levels of acceptable risk that would trigger more refined risk assessment and relative values of quantification of uncertainty would need to be ‘calibrated” based on many case studies and expert solicitations. The type of investigation completed by Drexel University over the past few years would fit into a management scheme of assessment based on risk, or would stand alone as an intervention in response to a structure of concern. Additionally, some of the tools developed proved conceptually by Drexel will be available to provide Level 3-5 individual risk assessments in order to reduce uncertainty premiums. These include the arch load rating method and the concept of rapid bridge screening.

Table 4-7 - Preliminary Assessment Programs per Risk Level

Risk Level	Mandatory	Option 1	Option 2
Severe	Level 3 / Year	Level 4 / 18 months	Level 5 / 2 years
High	Level 2 / Year	Level 4 / 2 years	Level 5 / 3 years
Elevated	Level 2 / 2 years	Level 4 / 3 years	Level 5 / 4 years
Guarded	Level 1 / 2 years	Level 4 / 4 years	Level 5 / 6 years
Low	Level 1 / 2 years	Level 4 / 4 years	Level 5 / 6 years

Sample assessments are included in Appendix A – Risk Assessment of Two West Virginia Structures.

4.1 Risk-based Assessment for Bridge Management

While the above approach is presented in terms of determining candidates for potential load testing procedures, the application as a basis for bridge management is inherently clear. The approach would remain the same for any sort of resource allocation, from general maintenance to rehabilitation to replacement.

The current approach to management utilized by West Virginia, as well as many other states, is resource allocation based on condition assessment. This focuses all decisions on qualitative assessment of the condition of the structure, and ignores several crucial factors including uncertainty and relevant bridge performance. The frequency of the condition inspections is generally dictated by law; however the procedure for completing them is not uniform between states and contractors. By utilizing the data from inspections for the purpose of establishing risk considering uncertainty, hazards, vulnerabilities and exposures, WVDOT could move toward the performance-based management structure which would better inform decision makers as to which assets truly require funding and/or intervention.

5 Conclusion

The authors submit that the three year research effort provided immediate results for the evaluation of five bridges and several additional products that were designed for future management of the entire reinforced concrete bridge population in West Virginia. The utilization of these products for a rational, expedient and reliable evaluation of each of the 1200 RC bridges in WV has been described. The authors greatly benefited from the opportunity to complete these investigations, including the awarding of three advanced degrees based on the research. The writers are motivated to demonstrate the application of these tools and completing their work on the rapid impact testing for bridge scanning tool so that it may be turned over to WVDOT for in-house use. The authors greatly appreciate the opportunity to complete this effort as well as all assistance provided throughout the past three years, and will be happy to continue serving WVDOT's bridge management needs.

References

(Catbas, F. N., Ciloglu, and Aktan, A. E., "Strategies for Condition Assessment of Infrastructure populations: A Case Study on T-beam Bridges," Structure and Infrastructure Engineering, Vol. 1, No. 3, pp: 221-238, September, 2005, Taylor and Francis).

Appendix A – Risk Assessment of Two West Virginia Structures

The first risk assessment shown in the following sections were carried out using inventory information and information contained in the previous inspection reports, which indicates that little impact on actual inspection procedure may be required to transition to a more rational risk-based approach. The second is a qualitative assessment conducted post-experiment and representing the changes in the uncertainty and vulnerability of the structure.

Barnett Bridge Assessment

The Barnett Bridge was constructed in the 1930's near Parkersburg, West Virginia. The structural form is a filled arch with a span of 90' and a height over the Tygart Creek of 34'. A review of inspection reports indicates that the bridge carries an ADT of 15,000 vehicles. Tygart Creek flows freely under the bridge and there is no indication of scour potential. The creek tends to flood after weather events due to backflow of the Ohio River, to which the Tygart Creek is a tributary. The proximity of the bridge to Parkersburg, as well as Rt. 77 means that a substantial portion of the traffic is trucks. The bridge tends to be congested at peak rush hour times. There is a posting of 32 tons for the state legal truck and signage indicating that trucks should cross one at a time. The following is a summary of specific notes pulled from the inspection reports:

Substructure – No evidence of tipping or settling in either foundation.

Superstructure – The downstream spandrel wall is in fair condition with hairline cracking and weathering. The upstream spandrel is in poor condition with deterioration along the arch ring allowing lateral movement and tipping of the wall. There are large vertical cracks. The arch ring is in fair condition. There are small spalls, with exposed reinforcing, hairline cracks and efflorescence. The edges of the arch ring have heavy deterioration, particularly at the apex. In some cases, there is no bond to the spandrel wall.

Wearing Surface – The wearing surface is in fair condition. There are transverse and longitudinal cracks throughout the overlay, fill settlement, heavy map cracking, voids and patchwork. The curbs are in fair condition, though often covered with asphalt and debris.

Maintenance – The bridge is inspected every 6 months.

Table C 1 - Assessment of Barnett Bridge *Pre-Experiment*

Performance Limit State	Hazard	Vulnerability	Exposure	Uncertainty	Aggregate Risk
Safety: Geo/Hydraulic	1	1	3	2	6
Safety: Structural	3	3	3	2	54
Serviceability and Durability	3	2	1	2	12
Operations	2	2	2	2	16
				Total Risk	58 (Level V)

Table C 2 - Assessment of Barnett Bridge *Post-Experiment*

Performance Limit State	Hazard	Vulnerability	Exposure	Uncertainty	Aggregate Risk
Safety: Geo/Hydraulic	1	1	3	***1***	***3***
Safety: Structural	3	***1***	3	***1***	***9***
Serviceability and Durability	3	2	1	2	12
Operations	***1***	2	2	2	***8***
				Total Risk	18 (Level II)

Smithers Bridge Assessment

The Smithers Bridge was constructed in the 1930's and consists of a skewed, three-span cast-in-place beam construction which spans both a roadway and a creek. The bridge had a recorded ADT of 7600 in 2006. The bridge is one hour west of Charleston, and lies on a CRTS route. This indicates that there is a substantial potential for overload. Congestion does not appear to be a problem for the structure, though there is a traffic light in the region which can cause the backup of traffic onto the structure.

Substructure – Both abutments are in fair condition with some cracking and spalling. The piers are also in fair condition with cracking and delamination in the caps, particularly on the underside.

Superstructure – The deck is in good condition with a few random hairline cracks on the underside. All three spans are in fair condition. Span 1 has some shear cracking, and all three spans have spalling and exposed. There are vertical hairline cracks and some popouts on all beams of all three spans.

Wearing Surface – The wearing surface is in good condition with open cracks of the piers and abutments.

Maintenance – The bridge is inspected every 2 years.

Table C 3- Assessment of Smithers Bridge *Pre-Experiment*

Performance Limit State	Hazard	Vulnerability	Exposure	Uncertainty	Aggregate Risk
Safety: Geo/Hydraulic	1	1	3	2	6
Safety: Structural	2	2	3	2	24
Serviceability and Durability	2	2	1	2	8
Operations	2	3	1	2	12
				Total Risk	29 (Level III)

Table C 4- Assessment of Smithers Bridge *Post-Experiment*

Performance Limit State	Hazard	Vulnerability	Exposure	Uncertainty	Aggregate Risk
Safety: Geo/Hydraulic	1	1	3	***<u>1</u>***	***<u>3</u>***
Safety: Structural	2	***<u>1</u>***	3	***<u>1</u>***	***<u>6</u>***
Serviceability and Durability	2	2	1	2	8
Operations	2	***<u>1</u>***	1	2	***<u>4</u>***
				Total Risk	11 (Level II)

Analysis of Risk-based Assessment

The comparison of the two assessments conducted previously is in agreement with the results of the experimental investigations conducted by Drexel University. The Barnett Bridge represented a level V risk and required immediate intervention, aligning with the course of action taken by the DOT. In this case, the risk was driven by the structural safety, which was brought into question due to visual inspections. The Smithers Bridge, which achieved a level III risk assessment, showed that the risk was driven by structural safety and operational concerns. The easiest way to reduce these risk levels is to reduce the uncertainty premium and vulnerability. This reduction is the chief result of conducting any sort of further investigations, analogous to what Drexel completed over the last several years for WVDOT.

A post-experiment assessment of each structure is presented as well, taking into account the reduction in uncertainty premium and vulnerability resulting from the investigation. The new assessments are both level II, which indicate a fairly low level of risk, especially when compared to the initial assessments.

Overall, applying this type of assessment to the entire population of structures in West Virginia would clearly highlight the structures that should be of major concern for the DOT, like the Barnett Bridge.

Appendix B – Arch Load Rating Method

Background

The arch is a simple, robust system by its nature. However the application of arches in real structures creates a large amount of uncertainty. As a result, accepted load rating procedures and methods can be problematic and unreliable.

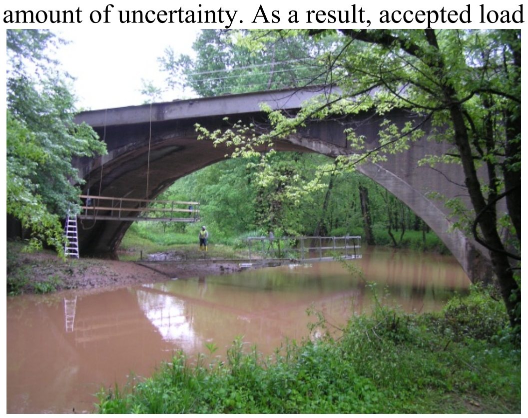


Figure B 1 - Barnett Bridge

These structures were typical construction in the early to mid 20th century and are prevalent throughout the country. The aim of this report is to identify a method which quantifies and minimizes the uncertainty associated with arches, explores the modeling options available, and proposes a methodology for load rating of an arch guided by a cost-benefit approach for decision-making.

This methodology will be presented in the context of a case study of a single WV arch structure, the Barnett Bridge. The Barnett Bridge is a single span concrete arch structure located in Mineral Wells, WV. Barnett is situated just off of Interstate 77 near Parkersburg, WV on Route 14. Parkersburg is a highly industrial area, situated on the Ohio River. The bridge provides the primary access to a large area east of Interstate 77, and as a result, sees very high volumes of traffic. The bridge is comprised of an asphalt wearing surface layered on top of a concrete deck, a layer of fill, and the main structural component, a concrete arch. The arch is of variable thickness, ranging from 17" to approximately 100". The fill is contained by spandrel walls which are tied to the arch through triangular concrete buttresses. The surrounding soil is contained by four wingwalls. The bridge was posted lower than the state legal limit of 80,000 lbs due to deterioration and the resulting unpredictable effects. These postings cause some rerouting of traffic on a detour of approximately 16 miles. This occurred after there was an economic push to local industry to move businesses to the area. The postings created problems for the businesses after trucks were getting ticketed. The district intends to remove the bridge from service within the next 5 years. There is a reroute of Rt. 14 over a new bridge scheduled to be completed by that time, which was part of the reason for the industry push.

Problem Description

Key Sources of Uncertainty

The disconnect between simple structural models and real life structures is especially apparent in arches. In practice, the response and behavior of an arch structure is dependent on numerous factors which all can vary significantly even within a single structure. In a structural model analogous to those used for basic load ratings, the section properties are uniform, the boundary conditions are known and are generally simple, and the only component of interest is the arch portion. In reality, the boundary conditions likely provide some rotational or translational stiffness, and do not behave as perfect pins or rollers. Section and material properties change throughout the structure, and the interaction of the components is both complex and difficult to quantify.

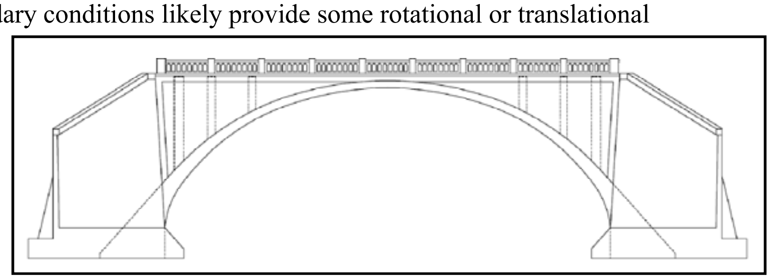


Figure B 2 - Barnett Bridge Plan and Elevation

Boundary Conditions

While the Barnett Bridge had plans available from the original construction in the 1930's, there was no indication of the foundation conditions. The arch expands out to a substantial footing at its base, but the conditions upon which these footings bear are totally unknown. There is a distinct possibility that the conditions on either side of the creek are totally different. Even if foundation plans were available, it is not known if any changes have occurred. The elevation shown here was transferred from the existing drawings of which the DOT was in possession. Note that the full shape of the arch is visible in the elevation, but there is no indication of the soil conditions on which the foundation rests.

Soil Properties and Continuity with Surrounding Roadway

Equally important to understanding the structure is the backfill which bears on the arch and wingwalls of the structure. For aged bridges, there was likely no requirement for the surrounding soil, excluding the portion which the foundations bear directly on. Consequently, contractors could fill the voids required for construction with anything readily available. This soil surrounding the base of an arch structure interacts with the structure to determine the boundary conditions of the bridge. The soil could act as a fixed boundary, totally preventing any movement, or it could allow slippage, sinking and rotation of the footing, acting as springs. It is likely that the behavior is somewhere in between, but the possibilities are infinite.

Continuity Conditions throughout the Structure

In addition to the main load carrying portion of the bridge, the arch, there are several other crucial components. In the case of the Barnett Bridge, there was a deck bearing on a layer of fill which was supported on the sides by spandrel walls, on the ends by the backfilled soil, and underneath by the arch itself. The interaction between these components is often variable, nonlinear and typically unknown.

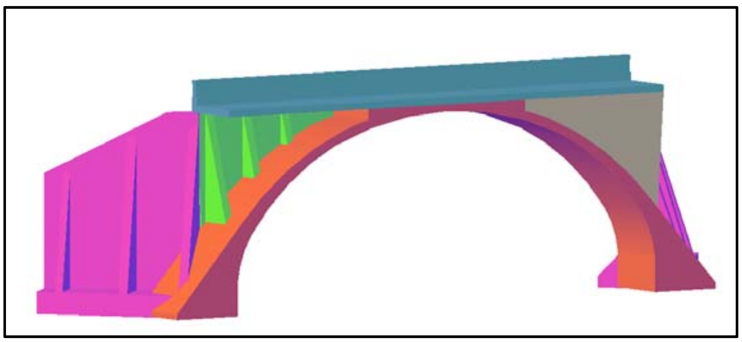


Figure B 3 - Filled Arch Cross-section

Material Properties

There are general specifications of concrete strength available based on the era in which the bridge was constructed, but these values are inherently conservative. This conservatism is required as a result of the practices employed in the early to mid 20th century. Oftentimes the water and aggregate for concrete came from the creek where the bridge was being constructed. ASTM guidelines for concrete and steel properties were in existence, but not always required by engineers yet. Even when a specific concrete design is specified, research has shown there is substantial variability in the resulting strength.

Figure B 4 - Oversize Aggregate

The composition of the fill used in the filled arch bridges was likely given much less consideration than the concrete mixture. The fill was probably considered a lighter alternate option to a solid concrete structure, which would have been more expensive and labor intensive. However, the investigation in Barnett Bridge revealed the importance of the makeup of the fill material in terms of the bridge behavior. The arch exhibited dishing towards the center of the span which indicated that the fill was very soft as compared to the concrete which comprised the arch and spandrel walls. This phenomenon was discussed in the Barnett Bridge Load Rating Report.

Modeling Challenges
The uncertainty described previously is typical of most structures, not just arches. A common solution to aid in reducing this uncertainty for the purpose of load ratings is to create a finite element model. However, building FE models for arch structures has an entirely different subset of difficulties, which can often exacerbate the uncertainty instead of diminishing it.

Coupling of Responses
Real life arch structures do not support only compressive axial loads, but also substantial shear and moment. The coupling of these responses has an effect on the resulting load ratings from any given model, be it an FE model or a hand calculation. An axial compressive force can greatly increase moment capacity for example, but must be contained at the foundations.

Non-prismatic Cross-sections
Many reinforced concrete arch structures, including the Barnett Bridge, have a variable depth cross-section for the main load carrying portion of the bridge, the arch. This is intuitive considering the compressive load increases along the length of the arch, but it does not make modeling easy. This type of variation is common for concrete arch structures.

Heterogeneous Construction
The discussions of the quantification of the uncertainty associated with filled arch structures defined the composition of the components of the structure. This includes the deck, fill, spandrel walls and the arch. These components are inherently redundant and are often complex geometries. For modeling, they are easiest to construct with solid elements, as opposed to frames or shells. Regretfully, frame and shell construction is typically more simple and straightforward for modeling than solids. Therefore the main challenge in modeling an arch bridge for the purpose of load ratings is to reliably represent the structure using the more simple methods.

Load Rating Method
Due to the high level of uncertainty as well as the difficulty in creating simple analytical models which accurately represent the structure, a new rating method will be presented. This method addresses the two problems with arch load rating, uncertainty and modeling difficulty, at their core. Understanding the appropriate depth the load rating effort should utilize is a crucial step in the process which dictates the direction the effort will proceed. Each possible investigation has several levels of depth, which can be selected based on the risk associated with the structure, the level of vulnerability, and the available financial resources. The method presented will be followed through as an example using the Barnett Bridge in Section 4.

Step 1: Define the Geometry of the Structure
Understanding the geometry of an arch structure is crucial for load rating, especially when considering the complex interaction of moment, shear and axial actions. The first step is to collect all relevant documentation on the structure. Hopefully this includes design and as-built drawings, retrofit drawings, and inspection reports. Often only some of these documents are available. From these, a complete understanding of the geometry and condition of the structure can be determined.

In conjunction with whatever documentation can be collected, a field visit to the bridge can be conducted. This allows for comparison of some cursory field measurements to those of the documents. If no documentation was available, more in-depth measurements can be taken to approximate as closely as possible the geometry of the bridge.

In cases where no documentation is available and the geometry of the structure must be specifically known, a complete survey can be conducted with surveying equipment. This is time consuming and labor intensive, but is the best way to determine the geometry is plans are unavailable.

Whatever information can be collected should be combined and put into an AutoCAD drawing. Simplifications to the geometry can be completed in accordance with the guidelines presented in Section 4.

Step 2: Determine the Material Properties
Material properties can be explored to several different levels. At the most conservative level, the assumed properties based on the construction date of the bridge can be applied, assuming that the specified properties are not

given in the available material about the bridge. The inherent problem with specifications is that they generally function as a lower bound only. Often the actual material properties are much more substantial than the specification would indicate.

For this reason, material sampling is often conducted throughout the structure. This process has varying levels of depth as well, which become more reliable with effort. The first level is rapid testing, and is comprised of simple methods like a Schmidt hammer for concrete strength, which requires large amounts of data to provide statistically reliable information. These methods are totally non-destructive and require little to no impact on traffic.

The next level of depth for material testing is a sparse array of in-depth material testing, including cores and rebar samples. The locations sampled should be spread out enough to cover the entire structure, and provide the bare minimum number of samples for the results to be considered statistically relevant. The samples should be tested according to ASTM standards and the data processed to determine a more accurate representation of the material properties.

The final level is a full complement of samples from which very accurate representations of the material properties throughout the structure can be determined. The number of samples required for this would indicate that this level of testing should be considered destructive. This is not ideal and should be avoided unless absolutely required.

Step 3: Verify the Boundary Conditions

During the collection of all documentation and conduction of site visits, careful attention should be paid to the boundary conditions of the structure. Any signs of movement of the base of the arch, deterioration of the wingwalls or the soil behind it, or cracking at the interface to the roadway should be noted. Ideally, the engineer should be comfortable with assuming a simple boundary condition (pin-pin or similar) based on their observations.

Step 4: Development of a Model

Modeling Options

There are four levels of depth for the construction of an FE model. A simple hand model is valid for numerous structural types when calculating load ratings. It is very difficult to account for the factors which greatly affect the structure with this type of model. However if a bridge can rate considering this high level of conservatism, then it is likely that this structure would not be of concern.

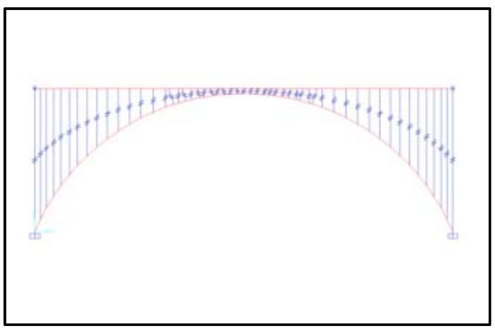


Figure B 5 - 1D Element Model

Taking the simple model from a hand calculation and making it a 1D finite element model with frame elements makes it more versatile and relevant. First off, creation of a simple 1D model is simple with almost any finite element software. Some of this type of software is available online for a nominal fee. Variation of boundary conditions and material properties based on field investigations is more straightforward, and determination of the demand envelopes is very rapid compared to hand analysis. This allows for multiple iterations of the model with varying parameters. There is still a substantial disconnect between the behavior of this type of model and a real structure.

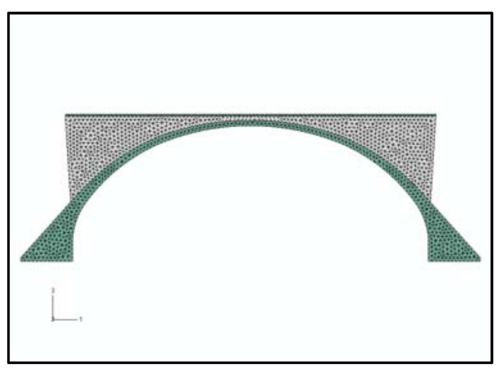


Figure B 6 - 2D Slice Model - ABAQUS

The next step up from the 1D model is a 2D model comprised of shell elements. Within this, the two dimensions could either represent a slice of the entire structure, or an extrusion of the 1D beams into a full thickness model of the arch. Both options are possible with shell elements, and are still fairly simple to create. The full-width extrusion method is technically 3D, but since it is comprised of entirely 2D elements, it is classified as 2D. This method accounts for two-way action which can occur within the arch, but again it neglects the rest of the structure including the fill, spandrels and deck. Due to this discrepancy between model and reality, this is not the best solution for a simple load rating method. A second 2D option is to take a slice of the bridge and build the entire model using shell elements. This is the best approximation which can be done using 1D or 2D modeling, as it only neglects two-way behavior. The best method for modeling of filled arch structures is a complete 3D model comprised of solid elements which can represent complex, variable geometries. A 3D model can account for interactions between components, two-way behavior, variable boundary conditions, and represent dead load more accurately. This was the modeling method used for the load test of the Barnett Bridge. Utilizing this calibrated model and the experimental results, a 2D shell model will be calibrated in order to extrapolate appropriate assumptions which could be made pretest to develop this 2D model for other situations and calculate load ratings. This process is included in Section 4.

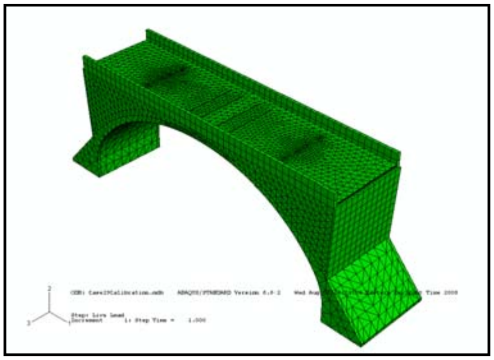


Figure B 7 - 3D Solid Model

The most important benefit of any finite element model is the ability to explore the sensitivity of the structure to any given parameter. This can be accomplished by bounding the responses after a parameter change to the model. Identifying which parameters a model is sensitive to can guide preliminary investigations, load ratings, and testing methods. This is especially important for properties like the fill stiffness, which is difficult to quantify even with experiments.

Analysis of Modeling Methods for Application to Load Rating Method

Hand Calculation
The first investigation completed is a simple hand calculation which is based on extremely conservative assumptions. The major assumption is that dead load has no effect on the live load capacity. In reality, the axial force caused by dead load would greatly increase the capacity of the structure, behaving similar to prestressing. The load resulting from any truck is conservatively combined to a single point load, which is placed at midspan, they divided across the width of the bridge. Another assumption is to ignore any compatibility issues between components of the bridge by not accounting for the fill or deck at all. The arch is the only member which can provide capacity. The hand calculation must also assume a relationship between the live load and the thrust of the arch at its base. This relationship is defined by the angle of line between the bottom outside corner of the footing and the tangent of the bottom of the arch. Only the safest bridges will rate with these assumptions. If some additional capacity is required, dead load can be accounted for using a model from one of the methods described below.

Arch Alone with Frame Elements
The most basic finite element modeling technique is basically an extension of a simple hand calculation. The analysis is of the main structural component only, and represents a unit width of the structure. The model is composed of linear elastic frame elements and pin supports. This is 1° indeterminate, but could be solved easily by hand using any number of methods. However, repeated solution of the same problem with varying properties is not efficient which is why software should be used. This model is clearly very basic and functions as a lower bound. It cannot represent the rest of the componentry that makes up the structure, including the spandrels, deck and fill. These portions can be represented for dead load as linearly varying distributed loads, but their participation as structural components is not accounted for in this model. The varying thickness of the cross-section can be accounted for by defining different section properties individually for each frame element comprising the arch. This is labor intensive, and could be approximated with smeared or averaged properties. This model does not account for any sort fixity at the boundaries or two-way action. With just an arch structure, any loads not at the midpoint must be applied at a point projected down from where the load is applied to the deck in reality. This becomes less and less accurate the further the load is from the centerline, as the arch gets thicker towards the ends, changing the load distribution caused by the fill.

Arch with Springs and Deck – Frame Elements
The next step up from the most basic model described above was the addition of linear springs to represent the fill, connected to frame elements representing the deck. This model at least partially accounts for the structural contribution of the fill and deck to the overall behavior of the structure. It also greatly simplifies application of loads at appropriate locations. The problem in using springs is that the fill is an inherently 3D material which transmits load very differently than a 1D spring. It is possible to develop a spring stiffness which would provide the same

results the actual structure, but this would require testing in order to experimentally determine the stiffness. This would only be relevant for that particular loading and could not be extrapolated either.

Arch Alone with Shell Elements – Extruded Model
In order to compensate for the lack of two-way action from 1D models, the next step was to extrude the 1D frame model of the arch into a 2D shell model. This still neglects the fill and deck, but it does account for load distribution across the width of the structure and the two-way action of the full width arch. Loading this type of model is very difficult, as the loads are projected from the deck, and must be located across the width as well.

Shell Model of Complete Structure – 2D Slice Model
The most effective simple model is the 2D slice of the structure. This model accounts for all of the components of the structure and their respective interactions, with the exception of the two-way action. The in-plane behavior of the shell element is a much better predictor of the 3D behavior of the fill than a linear spring. The variable thickness of the arch is easily modeled simply by defining the outer geometry of the arch and meshing within that area. The material properties for each component can be an average resulting from numerous slices across the width of the structure. That way, the stiffness of the spandrel walls can be included in the fill properties.

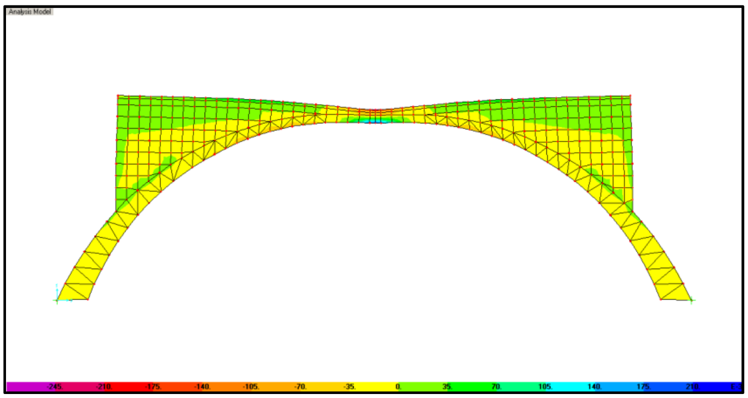


Figure B 8 - SAP2000 2D Slice Model

Solid Structure
The most accurate model for representing a filled arch structure is a complete, solid 3D model. This was created for the Barnett Bridge and used for pretest experimental ratings, and post-test model calibration. It accounts for all the variability of components and interactions as well as two-way action. Loading can be applied directly to the deck and at appropriate locations across the width. This no longer can be considered a simple method.

Comparison
All of the finite element models, including the 2D Slice model created in SAP2000 for this report were compared, under the actual loading which occurred during the load test to determine which methods are the best for simple load rating. The midspan displacement is shown in the following Table. Note that the 2D slice models are the closest with the exception of the calibrated 3D solid model. This indicates that for the simple load rating method, the model should be a 2D slice utilizing as much information as possible from the preliminary investigations.

Table B 1- Model Comparison

Model	2D Frame (SAP)	2D Slice (ABAQUS)	3D Shell Arch (SAP)	2D Slice (SAP)	3D Solid (ABAQUS)	*Experiment*
Midspan Deflection	-.15 in	-.019 in	-.202 in	-0.024 in	-.018 in	*-.017 in*

Step 5: Determine Demand and Capacity
The next step is to determine the live and dead load demands, as well as the capacity. The critical limit state for the Barnett Bridge was flexure at the top of the arch, though other limit states could be investigated.

Live Load Demand
If using the hand calculation method, the demand values come from a solution of equilibrium equations which is presented in Section 4.

Dead Load Axial Force
If required, use a stress analysis of the section to determine the dead load axial force. The stresses must come from a model, as it is too difficult to simplify the dead load distribution for hand calculation. This analysis will include the effects of the axial force caused by the live load as well. Recall that the resulting dead load axial force demand should be in per foot units. The process is presented in Section 4.

Capacity (M_n)
A section analysis approach is used to determine the capacity. The entire cross section is taken as a box beam, transferring shear down the spandrel walls. This is not entirely correct in reality, as load can be transferred directly through the fill. The ability to transfer shear between the deck, fill and arch is unpredictable and incorporating the fill is difficult and potentially unconservative. Therefore the capacity is based only on the box section. This is presented in Section 4.

Step 6: Determine Load Rating Factor
The equation used to determine the load rating factors is a simplified version of the equation utilized by AASHTO for load rating. Factors can applied to the demand and capacity at the engineer's discretion. Also, the dead load demand is often ignored because, based on the geometry, it has a tendency to produce negative moment at the top of the arch.

$$RF = \frac{\varphi \cdot M_n - M_D}{\gamma \cdot M_L \cdot (1 + I)}$$

where:

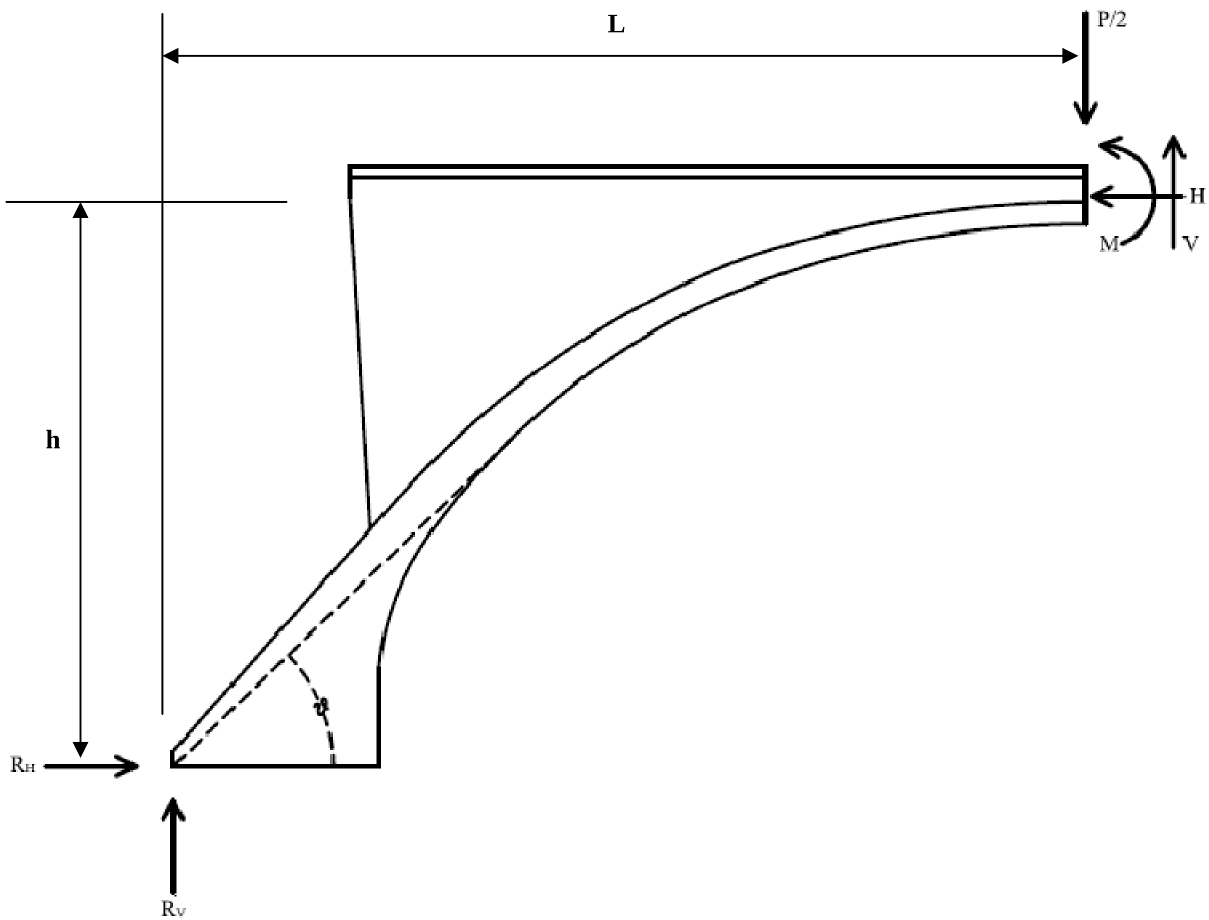
$$I = 0.33$$
$$\gamma = \begin{cases} 1.35\ for\ operating \\ 1.75\ for\ inventory \\ 1.8\ for\ legal\ loads \end{cases}$$
$$\varphi = 0.9$$

If the rating calculated is not at desired level, the engineer has the option of trying to reduce the uncertainty more through the methods described above including material property investigations, more detailed modeling, or a load test.

Appendix C - Sample Load Rating

In the case of the Barnett Bridge, the software package ABAQUS was used to develop both the 2D slice model and the complete 3D model before the load test because of the versatility of the interface. This is not a common software package and not as intuitive as something like SAP2000 or Visual Analysis which is analogous to the software a typical DOT may be using. For the purpose of developing a simple load rating method, first a hand calculation will be completed, and followed by a simple 2D slice model. The final step would be a full 3D model, which was already completed before the load test. The material properties and geometry were determined in the initial preparation for the load test and will be used throughout this load rating.

Hand Calculation Procedure



In the diagram above, P is representative of the total live load due to the rating truck (State Legal 80 k truck), adjusted for this calculation by the following:

$$P = \frac{Truck\ Weight\ \times\ \#\ of\ Lanes}{Width\ of\ Bridge}$$

From symmetry:

$$V = 0$$

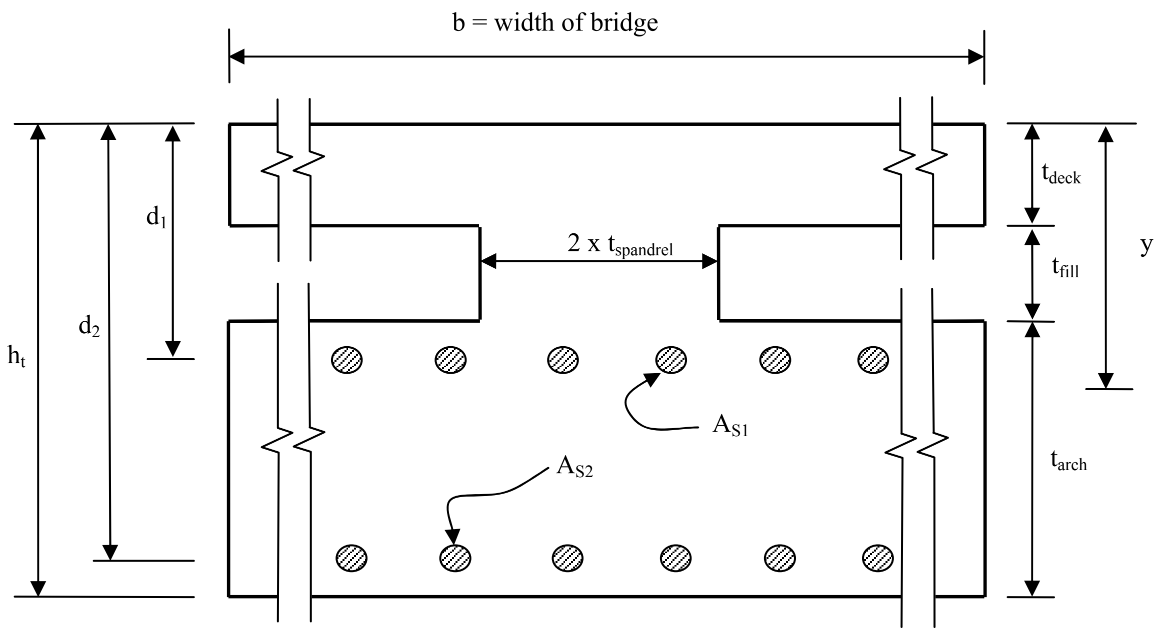
From equilibrium, determine live load moment demand:

$$RV = \frac{P}{2}$$

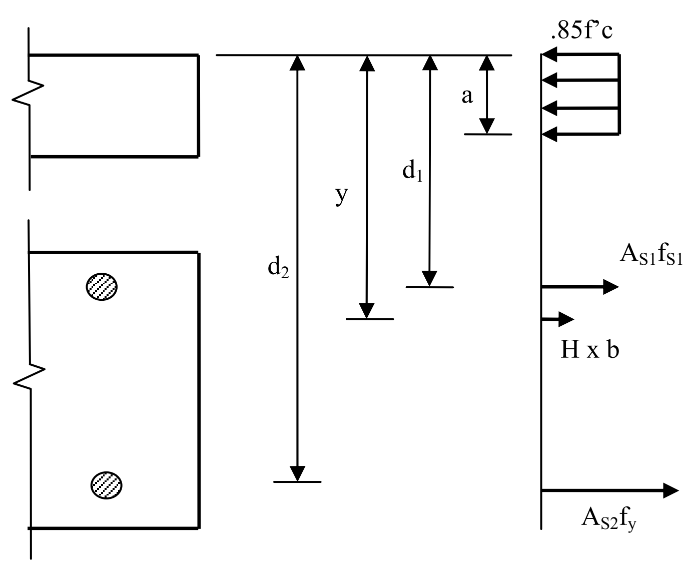
$$R_H = \frac{R_V}{\tan(\theta)} = H\ (per\ foot\ of\ width)$$

$$M_L = R_V \cdot L - H \cdot h$$

Find moment capacity of the cross-section, neglecting dead load effects, by using an I-section analysis including the entire width of the structure:



Assume that compression block is within the depth of the deck, and complete a section analysis, at capacity, using the following stress distribution:



Determine the stress block depth:

$$a = \frac{A_{S1} \cdot \left(\frac{d_1}{d_2}\right) \cdot f_y + A_{S2} \cdot f_y + H \cdot b}{0.85 \cdot f'_c \cdot b}$$

Check that *a* is within the depth of the deck, and then determine moment capacity:

$$M_n = \left(A_{S1} \cdot \left(\frac{d_1}{d_2}\right) \cdot f_y \cdot d_1\right) + \left(A_{S2} \cdot f_y \cdot d_2\right) + (H \cdot b \cdot y) - \left(0.85 \cdot f'_c \cdot b \cdot a\right)$$

Determine a factored load rating (Note that M_D is not included):

$$RF = \frac{\varphi \cdot M_n}{\gamma \cdot M_L \cdot (1 + I)}$$

where:

$$I = 0.33$$
$$\gamma = \begin{cases} 1.35 \; for\ operating \\ 1.75 \; for\ inventory \\ 1.8 \; for\ legal \end{cases}$$
$$\varphi = 0.9$$

Example Calculation for Barnett Bridge – State Legal Load

Given:

$$L = 50'$$
$$h = 35'$$
$$\theta = 44°$$

Find P for a unit width:

$$P = \frac{80\ k\ \times\ 2\ lanes}{22.5'} = 7.11\ k$$

Find reactions at the base of the arch, assuming a pin condition:

$$R_V = \frac{P}{2} = \frac{7.11}{2} = 3.6\ k$$

$$H = \frac{R_V}{\tan{(\theta)}} = \frac{3.6\ k}{\tan(44°)} = 3.72\ k$$

Determine live load demand:

$$M_L = R_V \cdot L - H \cdot h = 3.6\ k\ \cdot 50' - 3.72\ k \cdot 35' = \mathbf{648\ k \cdot in/ft}$$

Determine capacity:

Given:

$$d_1 = 14"$$
$$d_2 = 27"$$
$$f'_c = 9\ ksi$$
$$f_y = 36\ ksi$$
$$A_{S1} = 27\ in^2$$
$$A_{S2} = 27\ in^2$$
$$b = 270"$$
$$y = 16.1"$$

Determine the stress block depth:

$$a = \frac{A_{S1} \cdot \left(\frac{d_1}{d_2}\right) \cdot f_y + A_{S2} \cdot f_y + H \cdot b}{0.85 \cdot f'_c \cdot b} = \frac{27in^2 \cdot \left(\frac{14"}{27"}\right) \cdot 36\ ksi + 27in^2 \cdot 36\ ksi + 3.72\frac{k}{ft} \cdot 22.5'}{0.85 \cdot 9\ ksi \cdot 270"}$$

$$a = 0.76\ in < t_{deck} = 6\ in$$

Determine moment capacity:

$$M_n = \left(A_{S1} \cdot \left(\frac{d_1}{d_2}\right) \cdot f_y \cdot d_1\right) + \left(A_{S2} \cdot f_y \cdot d_2\right) + (H \cdot b \cdot y) - \left(0.85 \cdot f'_c \cdot b \cdot a\right)$$

$$M_n = \left(27\ in^2 \cdot \left(\frac{14"}{27"}\right) \cdot 36\ ksi \cdot 14"\right) + (27\ in^2 \cdot 36\ ksi \cdot 27") + (3.72\ k/ft \cdot 22.5' \cdot 16.1") - (.85 \cdot 9\ ksi \cdot 270" \cdot 0.76")$$

$$\boldsymbol{M_n} = 34058.9\ k \cdot in = \mathbf{1513}\ \boldsymbol{k \cdot in/ft}$$

Determine a factored load rating at operating:

$$RF = \frac{\varphi \cdot M_n}{\gamma \cdot M_L \cdot (1 + I)} = \frac{0.9 \cdot 1513\ k \cdot in/ft}{1.35 \cdot 648\ k \cdot in/ft \cdot (1 + 0.33)} = \mathbf{1.17}$$

If needed, additional investigations could be completed which would factor in the additional capacity which results from the axial force caused by the dead load. In this situation, the dead load demand should be extracted from a 2D slice model of the type described in the previous section.

Finite Element Model Development

Define the Geometry of the Structure

Filled arch bridges have complex geometry which can be simplified without sacrificing the functionality of the arch system. There are several key parameters which define this geometry, but the overall aspect ratio of L to h and the angle, θ, between the bottom outside corner of the arch and the tangent of the underside of the arch are the most important. These define the manner in which the load carried by the arch will be transferred to the foundations. When inputting the geometry to AutoCAD for the purpose of modeling, the top profile of the arch can be modeled by the three points which define the two ends and the apex of the arch. The lower profile of the arch model should be defined by the variation between the thickness at the top of the actual arch and near the bottom before the arch flares out into a footing. The fill area should be defined by the thickness of the layer at the apex of the arch and at the end of the bridge, and should be bounded by a rectangle. The deck should be a rectangle with the thickness dimension determined.

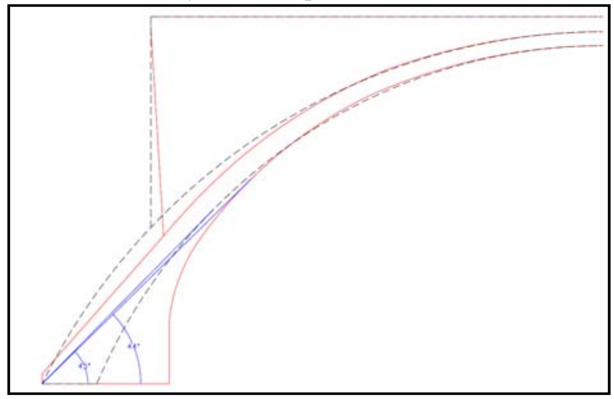


Figure C 1 - Geometry Changes

Utilization of the 2D Slice Model

The geometry above (dashed) was input into SAP2000 and meshed using shell elements. The resulting model was used to determine stresses at the critical cross-section at the top of the arch. From these stress profiles, the dead load axial or moment contributions can be determined. The moment should only be included if it is conservative to do so. In this case, only the axial force present in the arch will be accounted for in the calculation because the moment demand due to dead load is negative, and will act to further increase the capacity of the section.

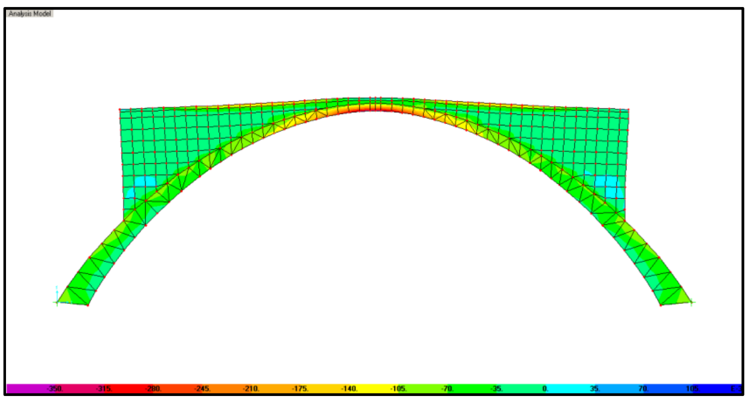
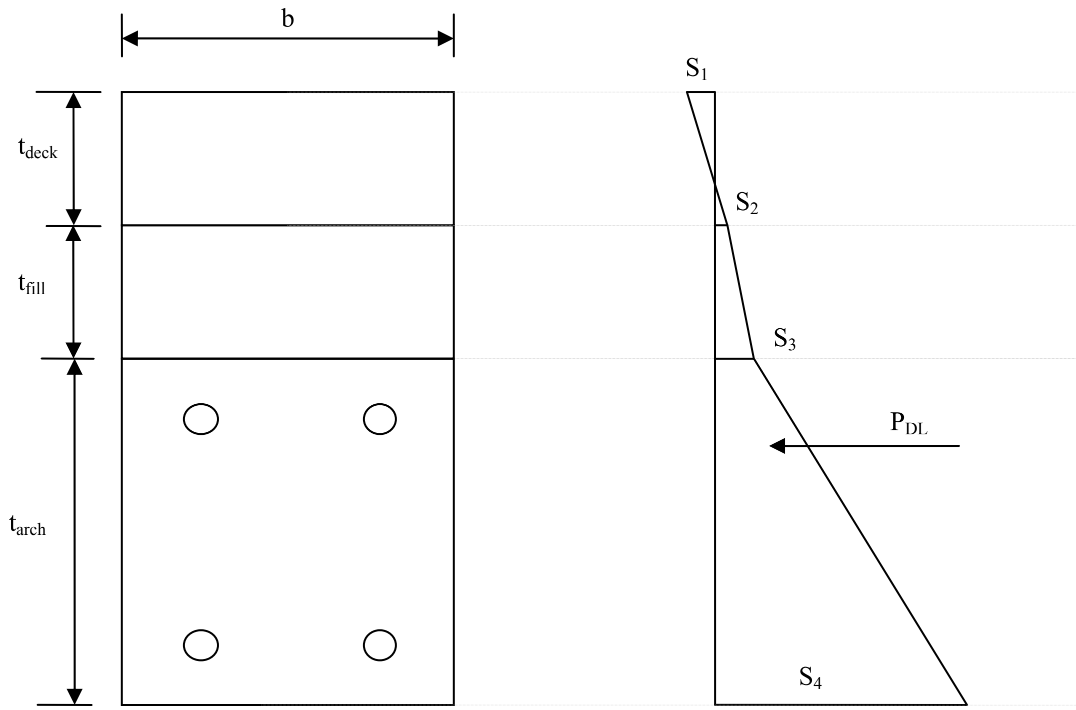


Figure C 2 - Dead Load Stress - SAP2000 2D Slice Model

Dead Load Contribution Utilizing the SAP2000 Model
Recall that the 2D slice model is representative of only a unit width of the entire structure. Determine the dead load demand given a stress distribution across the cross-section:



The magnitude of this resultant force is known based on the geometry of the stress distribution. Integrate the stress across the cross-section to determine the axial force due to dead load.

Example Calculation for Barnett Bridge
Given:

$$S_1 = .0135\ ksi$$
$$S_2 = -.0371\ ksi$$
$$S_3 = -.0639\ ksi$$
$$S_4 = -.3579\ ksi$$

$$P_{DL} = \left(\sum A\right) \cdot b$$

$$P_{DL} = \left(t_{arch} \cdot \left(\frac{S_3 + S_4}{2}\right) + t_{fill} \cdot \left(\frac{S_2 + S_3}{2}\right) + \frac{1}{2} \cdot S_2 \cdot \left(\frac{S_2}{S_2 + S_1} \cdot t_{deck}\right) - \frac{1}{2} \cdot S_1 \cdot \left(\frac{S_1}{S_2 + S_1} \cdot t_{deck}\right)\right) \cdot b$$

$$P_{DL} = \left(17 \cdot \left(\frac{.064 + .368}{2}\right) + 6 \cdot \left(\frac{.037 + .064}{2}\right) + \frac{1}{2} \cdot 0.038 \cdot \left(\frac{.038}{.051} \cdot 6\right) - \frac{1}{2} \cdot 0.014 \cdot \left(\frac{.014}{.051} \cdot 6\right)\right) \cdot 12$$

$$P_{DL} = (3.672 + .303 + .0849 - .0115) \cdot 12$$

$$\boldsymbol{P_{DL} = 48.6\ k/ft}$$

Determine capacity:

Given:

$$d_1 = 14"$$
$$d_2 = 27"$$
$$f'_c = 9\ ksi$$
$$f_y = 36\ ksi$$
$$A_{S1} = 27\ in^2$$
$$\mathrm{A_{S2}} = 27\ \mathrm{in}^2$$
$$b = 270"$$
$$y = 16.1"$$

Determine the stress block depth, assuming dead load axial acts at the centroid:

$$a = \frac{A_{S1} \cdot \left(\frac{d_1}{d_2}\right) \cdot f_y + A_{S2} \cdot f_y + (H + P_{DL}) \cdot b}{0.85 \cdot f'_c \cdot b}$$

$$a = \frac{27in^2 \cdot \left(\frac{14"}{27"}\right) \cdot 36\ ksi + 27in^2 \cdot 36\ ksi + \left(3.72\frac{k}{ft} + 48.6\frac{k}{ft}\right) \cdot 22.5'}{0.85 \cdot 9\ ksi \cdot 270"} = 1.28\ in < t_{deck} = 6\ in$$

Determine moment capacity:

$$M_n = \left(A_{S1} \cdot \left(\frac{d_1}{d_2}\right) \cdot f_y \cdot d_1\right) + \left(A_{S2} \cdot f_y \cdot d_2\right) + \left((H + P_{DL}) \cdot b \cdot y\right) - \left(0.85 \cdot f'_c \cdot b \cdot a\right)$$

$$M_n = \left(27\ in^2 \cdot \left(\frac{14"}{27"}\right) \cdot 36\ ksi \cdot 14"\right) + (27\ in^2 \cdot 36\ ksi \cdot 27") + \left(\left(3.72\frac{k}{ft} + 48.6\frac{k}{ft}\right) \cdot 22.5' \cdot 16.1"\right) - \left(.85 \cdot 9\ ksi \cdot 270" \cdot \frac{1.28}{2}"\right)$$

$$\boldsymbol{M_n} = 50931\ k \cdot in = \mathbf{2263\ k \cdot in/ft}$$

Determine a factored load rating at operating:

$$RF = \frac{\varphi \cdot M_n}{\gamma \cdot M_L \cdot (1 + I)} = \frac{0.9 \cdot 2263\ k \cdot in/ft}{1.35 \cdot 648\ k \cdot in/ft\ \cdot (1 + 0.33)} = \mathbf{1.75}$$

Discussion of Arch Load Rating Method

The above method of load rating makes every attempt to be as conservative as possible in an effort to counteract the violations of equilibrium and compatibility that are required to make the method "simple." Still, because there are so many inherent assumptions which do violate equilibrium and compatibility, the application of the arch load rating method as a legal load rating should not be allowed.

The process of developing this method has clarified to the authors the difficult task of understanding the filled arch system, highlighting the main factors which are of importance for the function of the system. These include the continuity and containment of the fill within the spandrels, the connection of the deck to the spandrels and the arch, and the solidarity of the foundations. Vulnerability in any of these factors could greatly affect the capacity of the structure. For this reason, it is especially crucial to pay close attention to these areas when conducting any initial inspections or documentation reviews.

Given that these areas appear in good condition, the arch load rating method will serve as a tool which can stratify the population of arch bridges based on their geometric vulnerability. The relative ratios of the rating factors will indicate which bridges are of the most concern based on their aspect ratios (h/L) and cross-sectional properties. Bridges which exhibit deterioration in the main factors listed above should be considered more crucial than any those with geometric concerns.

Appendix D – Rapid Bridge Condition Assessment – Proof of Concept

At the inception of the joint research between FHWA, WVDOT and Drexel University, the goal was to load test structures in West Virginia along the Coal Resource Transportation System (CRTS). However, as the CRTS contains thousands of structures, it quickly became apparent that while there was benefit in testing and clearing any single bridge, a larger advantage would result if the bridges could be screened in a rapid, reliable manner. From this, the concept of rapid bridge condition assessment was born. The concept relies on proven experimental testing techniques to extract modal parameters which serve as indicators to the bridges condition relative to any prior tests. These are quantitative values which are not subjective like the data collected during visual inspections. Therefore the results of rapid condition assessment could be coupled with visual inspection data to provide more reliable information to the DOT for decision-making.

Drexel conducted numerous investigations and feasibility studies in relation to the development of rapid bridge condition assessment, described briefly below:

- Initial investigations into Falling Weight Deflectometer (FWD) as a screening device on Amigo and Tams Bridges
- Lab investigations into modal testing using impact excitation
    - Cantilever beam tests
    - Steel I-section tests
    - Parking garage tests (Figure D1)
    - Concrete beam tests (Figure D2)
- Retrofit of FWD to act as rapid screening device
    - Improvement of frequency content of impact (Figure D3)
    - Reduction or elimination of rebound
    - Utilization of geophone sensors currently on FWD
- Testing of Modified FWD
    - Barnett Bridge and Smithers Bridge
- Proof of concept of modal testing for condition assessment
    - Smithers Bridge

The details of these studies and all experimental results are included in several reports submitted to West Virginia and FHWA over the last 4 years. Based on these results, the authors feel that rapid bridge condition assessment using modal data from impact excitation is a proven, viable technique. The best approach to make this concept a reality would be the development of a novel Rapid Load Testing Device (RLTD) which would be designed considering the problems discovered during the modification efforts of the FWD. This device would provide a single, clean impact which would excite a wide band of frequencies. Incorporated within the device would be appropriate sensors and data acquisition combined with automatic data processing which would allow for easy operation of the device. Considering these parameters, the RLTD would fit into the risk-based assessment framework as a method to decrease the uncertainty premium associated with a structure as yet another non-destructive evaluation technique. The RLTD concept was previously presented as a potential pooled funds project to FWHA and WVDOT personnel, though never submitted do to a shift in the focus of the research to particular, problematic structures in need additional investigations and load testing.

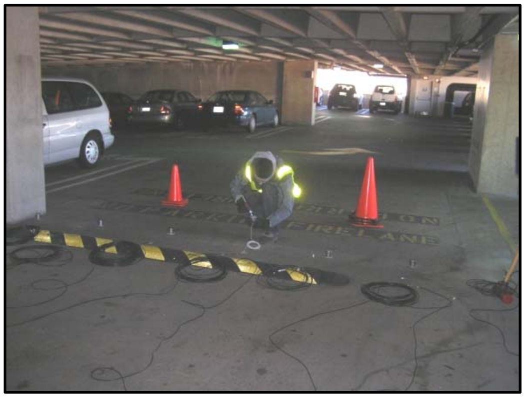


Figure D 1 - Parking Garage Test

Figure D 2 - Concrete Beam Test

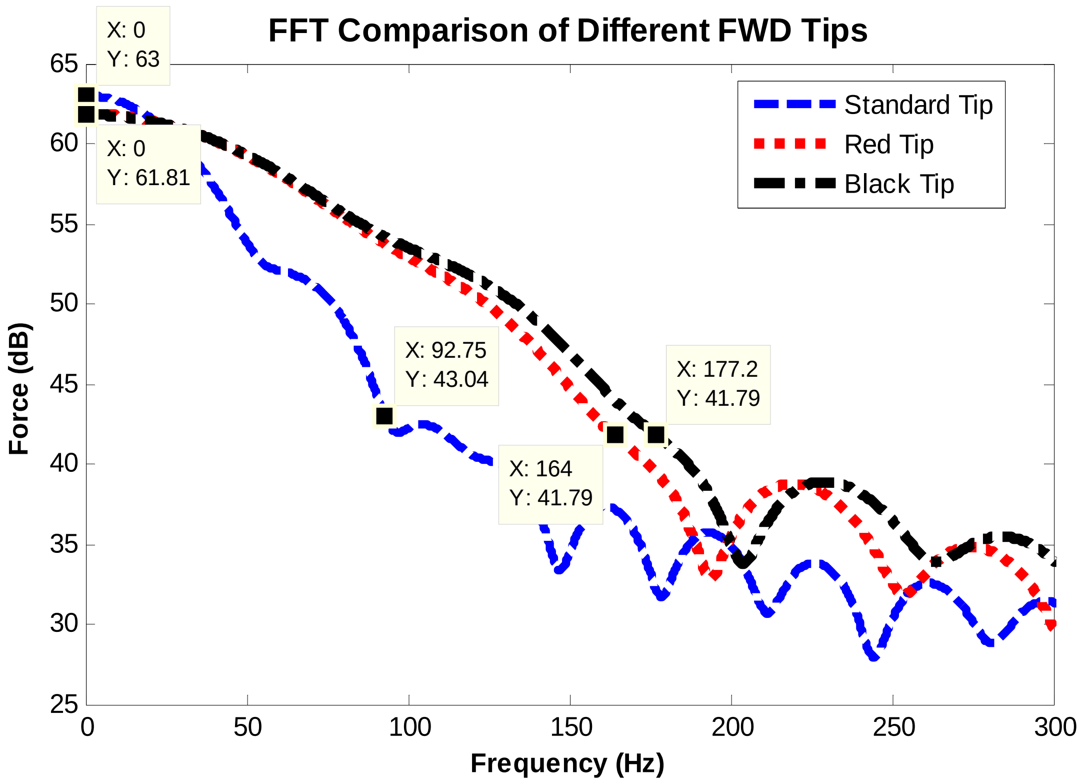


Figure D 3 - Improved Frequency Content of Impact