Numerically Modeling Structural Behavior of Precast Three-Sided Arch Bridges for Analysis and Design

by

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Abstract

Foley Products contracted with Auburn University to perform laboratory testing and design validation of three-sided bottomless precast concrete structures being produced. Tests performed included a field test on an active construction project, a laboratory test of a 20 ft clear span structure, and a laboratory test of a 36 ft clear span structure. The field test was performed in Midland, North Carolina on a 42 ft clear span arch structure. The field test was a service load level test performed by backfilling around the structure and driving a truck with a known weight over the bridge and stopping at various locations. The 20 ft clear span and 36 ft clear span laboratory tests were performed in the Auburn University Structural Research Laboratory. Gauges were installed in these structures prior to casting so data could be collected during testing. In both laboratory tests the structures were loaded to failure using hydraulic loading actuators.

Following testing, the computer program SAP2000 was used to develop two structural models. SAP2000 structural models of the 20 ft clear span structure as well as the 36 ft clear span structure was developed to correlate results between the structural analysis performed in SAP2000 and the data and results found during laboratory testing. Nonlinear behavior was accounted for in analysis. Moment hinges were incorporated in the structural model to correlate deflection magnitude in analysis to deflection measurements in the laboratory testing.

Once the SAP2000 structural models were developed and correlated well with the laboratory test results, an evaluation of the design methodology in use by the designers of the Foley Arch was carried out. A structural computer model was developed in RISA 3-D by the designers and used in design of the structure to predict the point of maximum moment. The RISA 3-D model used in design of the Foley Arch was compared to the model developed in SAP2000 to evaluate how well the structure corresponded to the behavior the designers expected.

It was found that the strength of the structures was adequate and that the design methodology being used was reasonable and safe.

Acknowledgments

I would like to thank Foley Products, Dr. Justin Marshall, Dr. Brian Anderson, and Dr. Robert Barnes who have all been a source of knowledge and guidance throughout this project and I thank them for giving me the opportunity to contribute to the research. My research partner, Luke Meadows, has been a pleasure to work with and I wish him all the best in his future pursuits. The entire Civil Engineering staff has provided me with invaluable experience throughout the duration of my education; I respect and appreciate the hard work and commitment to progress I have seen in the department.

I would also like to thank my parents. Since I was young, they have been dedicated to raising me to appreciate the importance of my spirituality, setting priorities, being disciplined, and remaining well rounded and balanced as I matured into an adult. Their persistent efforts to instill these qualities in me coupled with their tremendous example have given me an excellent model to follow as I live my life. Without their support I would not have been able to accomplish what I have at Auburn University.

Finally, I extend my deepest appreciation to my loving wife, Meredith. She has supported me in all that I have endeavored to accomplish and has been a tremendous source of encouragement and support throughout my time at Auburn University. Academic obligations and responsibilities have been a source of tremendous stress on many occasions, but she has always been right by my side, ready and willing to support me in any way. I love her and will forever appreciate all her love, support, and devotion.

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Chapter 1: Introduction and Background

1.1 – Background

Foley Products contracted Auburn University to perform third party analytical and experimental validation of the performance, design, and analysis methods currently in use for three-sided bottomless arch bridges being produced. The project included one field test and two laboratory tests of arches. A structural computer model was developed to correlate to the results of the laboratory tests. The design methodology was evaluated to determine what improvements could be made.

1.2 – Project Objective

The ultimate goal of the project was to validate all aspects of the Foley Arch bottomless bridge. Data from the laboratory tests were used to develop analytical computer models. The models were developed to the point that the analytical results correlated to the results of the laboratory test. The tests and structural computer models could then be used to validate the design methodology used for the arches.

1.3 – Project Scope

The experimental validation of the three sided bottomless arch bridges included one service load level test in a field test and two ultimate load level tests in the laboratory. The two laboratory tests were conducted on a 20 ft clear span structure and a

36 ft clear span structure. Correlation to the results of the laboratory testing was achieved using a structural computer model developed within SAP2000. The design methodology used in the design calculations provided by Foley Products was reviewed and evaluated to determine the effectiveness.

1.3.1 – Field Test

The first test performed was a field test. This test was performed during construction of a precast arch bridge in Midland, North Carolina. This field test was performed on a 42 ft span bridge with a vertical rise of 14 ft. The field test site concept and the test specimen cross section is shown in Figure 1-1 and Figure 1-2, respectively. This field test was utilized to evaluate precast arch bridge sections under a known truck load by installing instrumentation to measure strains. Deflection and earth pressure transducers were installed on-site prior to backfilling and loading the bridge.

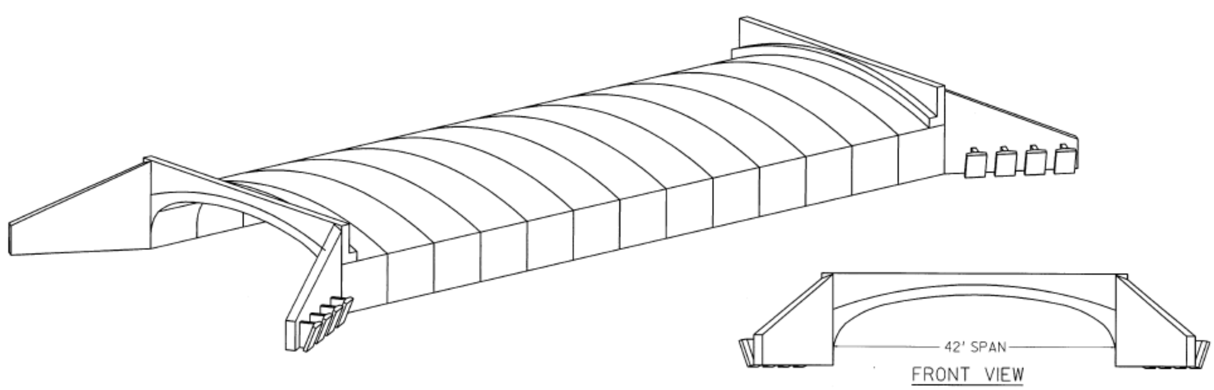

Figure 1-1: Midland, NC Field Test Site Conceptual View

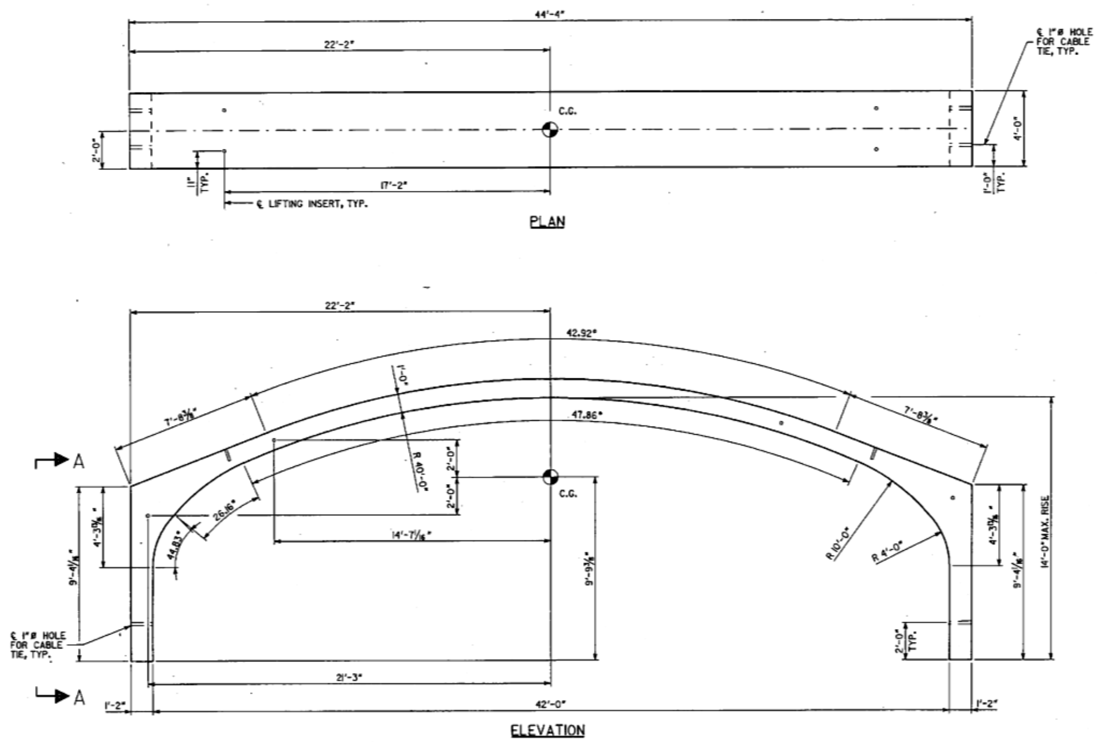

Figure 1-2: Midland, NC Field Test Section

1.3.2 – Laboratory Tests

In addition to a field test, two laboratory tests were performed to evaluate bridge behavior up to ultimate loading conditions. The first laboratory test section was a 20ft span bridge with an 8ft vertical rise. This laboratory test specimen is shown in Figure 1-3. The second laboratory test section was a 36 ft span bridge with a 9 ft vertical rise. This test specimen is shown in Figure 1-4. Strain gauges were installed on reinforcing bars that were added to the reinforcing cage of the bridge prior to casting of the section. The two tested sections were meant to be on the bottom and top of the design range spectrum of the precast sections available for production. Therefore, the range of behavior could be effectively evaluated by testing the two different sections.

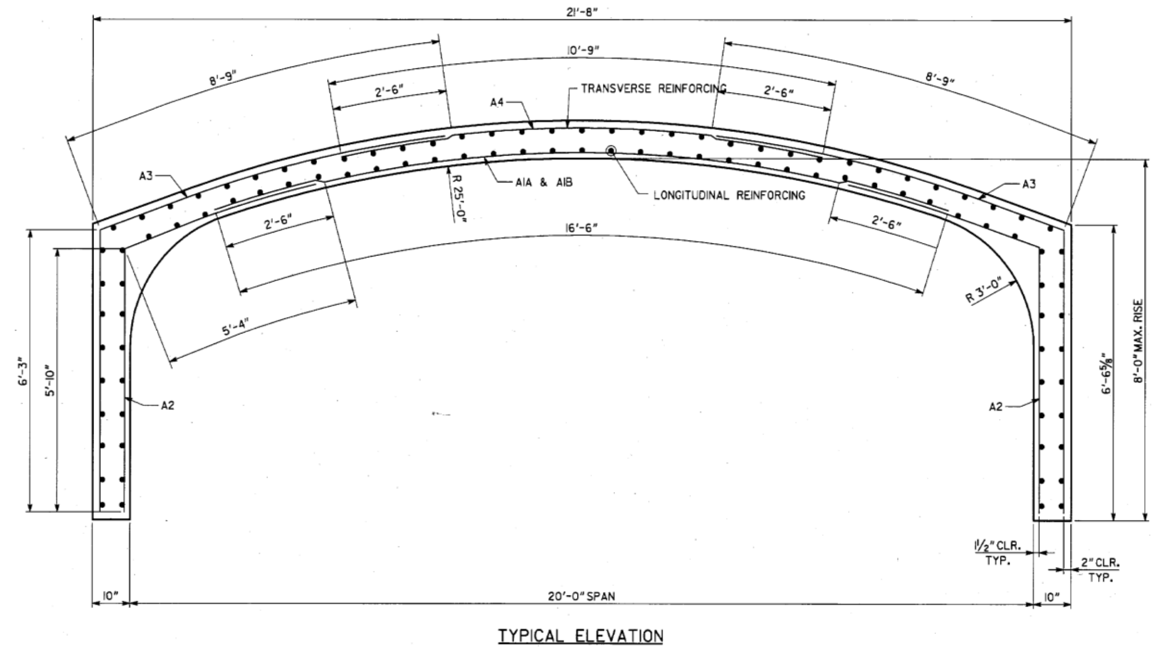

Figure 1-3: Laboratory Test – 20 ft Span Section

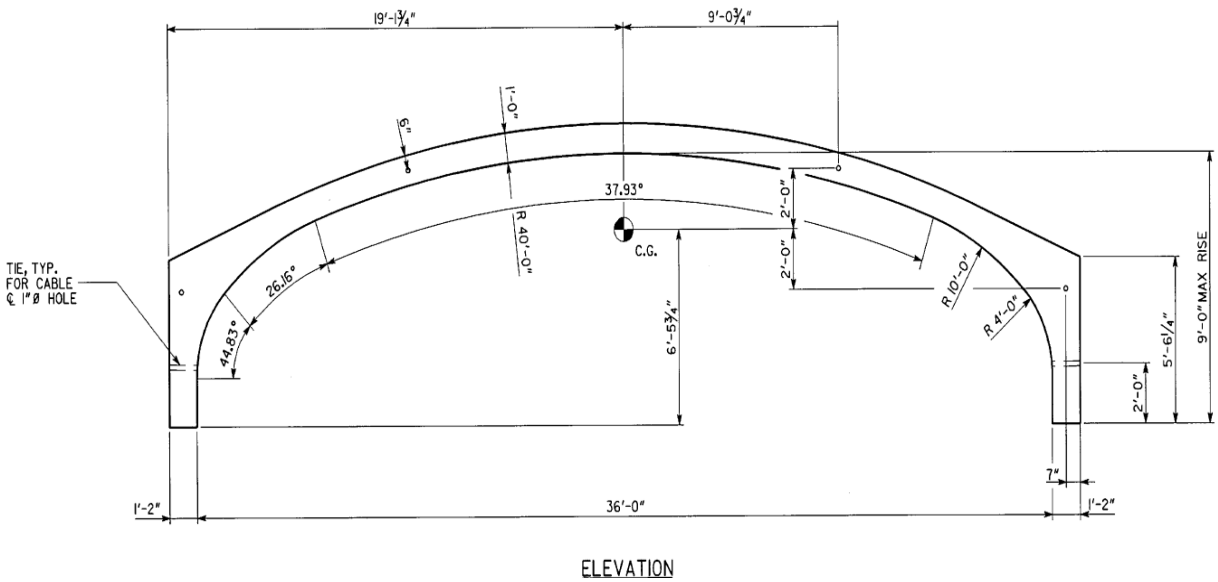

Figure 1-4: Laboratory Test – 36 ft Span Section

1.3.3 – Computer Modeling and Design Methodology

Following the field and laboratory testing, a computer model of the Foley Arch structure was created in SAP2000 to correlate the results of the laboratory testing to the analysis results of the computer model. This was used to determine if any economic or serviceability improvements could be reasonably recommended and/or implemented in the design or production of the structures. The design methodology was investigated by evaluation of the design calculations provided by Foley products.

1.4 – Outline of Thesis

This paper describes the testing performed in the field and laboratory tests; however, primary focus is placed on details of the analytical computer models that were developed based on the structures tested. Also discussed is the correlation of these models to test results and an evaluation of the design methodology used in design calculations provided by Foley Products for review.

Chapter 2 discusses the literature review that was conducted to understand and benefit from the work previously done related to testing and modeling of precast arch bridge structures. This assisted in planning for instrumentation needed. It also helped in developing how to effectively carry out and improve upon past experiments related to field testing. This in turn gave insight into what would be effective and useful in the laboratory testing. Work previously performed related to analytical modeling of precast arch structures was also reviewed to understand effective methods used in the past for modeling purposes.

In Chapter 3 the experimental tests are discussed. The field and laboratory testing procedure is reviewed and illustrated. This includes installation of gauges prior to casting of the test specimen and installation of other instrumentation prior to testing of the specimen. The field test loading was done by stopping a truck with a known weight at various locations. The laboratory testing allowed a more controlled environment and setup for test monitoring. Hydraulic loading actuators were used at three locations for testing in the laboratory. It was necessary to design various steel members to create a system that would allow loading of the structure. These designs are summarized and explained. During the two laboratory tests the specimens were loaded to failure. The loading progression of the specimen during testing provided numerous points of comparison to develop an analytical model that correlated to actual behavior. Those points of comparison and critical moments measured in the specimen are outlined and then discussed with the modeling summary.

The typical application of a three-sided arch bridge culvert consists of backfilling around and above the structure. Therefore, lateral restraints were designed for use during the first laboratory test to simulate the lateral earth pressure of the soil around the specimen. These designs and considerations in design are discussed in detail.

Chapter 4 explains in detail the analytical computer models developed in the program SAP2000 version 14 of the test specimens. The goal was to correlate the model analysis and results to the laboratory testing results. As with any structural analytical model, an appropriate level of detail needed to be achieved to produce appropriately accurate and reliable results. However, this is also balanced with computation and analysis time. One of the test specimens was on lower end of the range of spans typically produced and the other test specimen was on the higher end of the range of spans typically produced. The model was generated using the properties attained by examining the construction documents produced for the test section as well as physical inspection and measurements taken before and after casting.

Since the test specimens were loaded to ultimate failure in the laboratory setting, it was essential to include nonlinear material behavior in the computer modeling of the structure. Therefore, research on the manner in which SAP2000 V14 analyzes nonlinear behavior was performed in order to decide the most appropriate way in which to model the structure using that software. Concrete sections were assigned within SAP and a combination of prismatic and non-prismatic frame elements were used. Various changes throughout the modeling process are discussed as adjustments were made to better correlate to the actual behavior of the structure.

When the modeling was complete, the objective was to have good correlation to test results and be able to constructively review the design methodology. The design methodology and modeling used by the designer was reviewed to determine whether the design was effective. This review and assessment of the design methodology is discussed in Chapter 5. Recommendations were made regarding any changes deemed necessary or appropriate based on the testing and modeling performed.

To conclude, a summary of the paper is provided, conclusions made are discussed, and recommendations for further research are provided.

Chapter 2: Literature Review

2.1– Precast Arch Structures

Precast concrete members are increasingly used as a result of their time and cost efficiency. Ensuring a safe, but efficient, design with such a streamlined production and broad application becomes ever more important to companies utilizing this procedure. To ensure safe designs, controlled tests have been and continue to be performed on precast members. The precast arch bridge discussed herein is one such precast structure that has seen increased implementation on projects in the United States.

Companies including, but not limited to, Foley Products, CON/SPAN, and Con Arch are businesses that have utilized the arch bridge system for overpasses or underpasses, culverts, vaults, or combinations thereof. The result of widening interest in the precast arch bridge market has led to broader and creative applications of this time and cost effective product. This industry has also seen a considerable amount of investments in research and testing to better understand the performance, serviceability, and safety of arch structures. This chapter will review a number of such research projects, the objectives of those projects, and conclusions of the research.

2.2 – McGrath and Mastroianni (McGrath and Mastoianni 2002)

This project was related to the field testing and subsequent development of computer models for two 28 ft span reinforced concrete arch culverts. The field test data included arch deflections, pressures at the interface of the structure and backfill material, concrete strain, and reinforcing bar strain. The structure was cast and backfilled with the native material excavated from the site. Two types of loading were simulated for the field testing procedure, tandem-axle load conditions and HS20 load conditions. Both arch designs under consideration carried the maximum load that the equipment being used could apply. In both tests, loading was terminated due to bearing failure under the load plates. No failure or yielding of the reinforcement occurred up to the point of highest loading. No significant cracking had occurred in either of the structures until well beyond the ultimate factored HS20 load was achieved. The performance of both of the designs was found to be excellent during the field tests. Each structure carried a live load in excess of 240 kips without failing. The factored design load for the HS20 truck was 87 kips. When comparing the live load carried by the structure with no failure, it was apparent the structure had more than adequate strength capacity. Cores were cut from the tested structure and it was determined that the reinforcement at the crown of the structure had not yielded at the maximum load applied during the test.

A two dimensional structural model was developed using the computer program CANDE (Katona and Smith 1976) after completing the field tests. This software was a finite-element soil structure interaction program that incorporated nonlinear soil properties as well as nonlinear concrete behavior. A three dimensional structural model was developed using the computer program ABAQUS (ABAQUS 2006). ABAQUS is a general purpose finite-element program with nonlinear capabilities. The same in-plane mesh was used for both the two and three dimensional models. The three dimensional model was an extrusion of the two dimensional cross section to a total length of 70 ft. Results of the testing and modeling indicated that deflections and strains were often underestimated in the three dimensional computer model. Adjustments were made to the model to decrease the tensile strength limit which accounted for various parameters such as sustained loads, losses in concrete strength over time, and material variability in the concrete. Changing the concrete model parameters had a significant effect on the ability of the model to accurately predict structure behavior to correlate to testing results. The researchers concluded that internal forces could be well predicted even if the concrete modulus or cracking characteristics were not fully known. The three dimensional model developed within ABAQUS accurately predicted design forces but deflection and deformations were only correlated with test results by altering the concrete properties within the model. The two dimensional model developed within CANDE exaggerated the predicted deflection and deformation.

After testing, McGrath and Mastroianni concluded that “the performance of the standard and special test structures met AASHTO criteria for performance, 3-D analysis provided good predictions of the culvert performance and should be a useful design tool, and modifications or further study should be undertaken to correct the prediction of 2-D analysis to make them suitable for design for live loads” (McGrath and Mastroianni 2002).

2.3 – Zoghi and Farhey(Zoghi and Farhey 2006)

This project involved a field test of a 36 ft span bridge. During the field test, backfill up to 3 ft of cover was placed over the bridge. Load was applied in 20 kip increments, and data was collected at each increment. After applying the full capacity of the jack, 200 kips, there was only 1.5 in of vertical deflection and no yielding or damage to the structure. There was no lateral displacement at the bottom of the legs. Idealizing the structure to be pin supported at the bridge foundation was reasonable. Measured deflections at the top of the legs were around ¼ in. at most.

A two dimensional structural model of the bridge was developed in the computer program CANDE. The material properties of the concrete, reinforcing steel, and soil from the field testing were incorporated in the model.

The model results were compared to that of the field tests. It was found that the soil pressures at the top of the vertical legs of the structure correlated well. The soil pressures at the bottom of the vertical legs correlated well up to the ultimate design load of 57 kips. However, beyond this design load, the finite element model over predicted earth pressures. The vertical displacement predicted by the model correlated well up to the same design load of 57 kips; beyond the design load the model was sufficiently accurate but not as good as the lower range of loading. The horizontal displacements predicted by the model correlated well to the field test up to the design load of 57 kips, beyond which the deviation between the model and field test grew but was reasonably sufficient.

Some of the conclusions drawn from the research were that "the reliability of the analysis is highly dependent on the accuracy of the soil models and properties that are used for the given site conditions, the contribution of earth pressures is relatively small under normal operating loads, an extreme overload capacity can be developed by mobilizing the passive earth pressures along the side walls without concern for local failures, very small deflections were recorded at both design and factored loads," and the ultimate load capacity of the bridge was not achieved (Zoghi and Farhey 2006).

2.4 – Zoghi and Hastings (Zoghi and Hastings 2000)

On a project prior to this one, discussed in Section 2.3, testing of full scale live load testing had been carried out on a 36 ft precast reinforced concrete arch culvert but the ultimate capacity could not be achieved due to the load exceeding the capability of the hydraulic jack during the test.

> "The objectives of this second phase of the research project were two fold. First, the failure mode of the test culvert was investigated utilizing a more rigorous analysis based on the modified CANDE finite-element computer program. Second, the soil-structure interaction characteristics of long-span precast concrete arch culverts were analyzed under a wide variety of site conditions using the previous full scale load test results as a benchmark" (Zoghi and Hastings 2000).

In the previous testing of a 36 ft span, the vertical and horizontal deflections, crack widths, and surrounding earth pressures were monitored as the structure was loaded in 10 kip increments. The findings of that test verified the structure performed satisfactorily and exceeded required design load specifications. The same project verified a CANDE model developed correlated very well with actual deflections and soil pressure readings collected during testing.

The failure mode could not be established from this previous testing since the hydraulic jack reached its limit. Therefore, part of the second phase of this project was to identify the structure's failure mechanism using a modified CANDE finite-element computer program. Beam-column elements were used to model the culvert structure behavior and quadrilateral or triangular elements were used for the surrounding soil. A step by step procedure of analysis was used, with backfill incrementally being added into the model and monitoring the behavior with this increasing backfill. Laboratory tests were used to determine at what point the steel was expected to yield and the point at which the concrete was expected to crush. Using this information, CANDE runs were performed to find the applied loads corresponding to the steel yielding and concrete crushing points. The yielding of steel was found to occur at a total load of approximately 235 kips. The concrete was found to begin crushing at approximately 250 kips. "The point, at which sudden change of structural response took place, was considered to be the ultimate failure. Based on this methodology and the concept of soil-structure interaction, the ultimate load that arch culvert structure can withstand is approximately 340 kips [at a deflection of 7 in]. This is an impressive 13 times greater than the design load of 26 kips" (Zoghi and Hastings 2000).

2.5 McGrath, Selig, and Beach (McGrath, Selig, and Beach 1996)

This research was related to a field test of a 36 ft span arch bridge. Using 1 ft of cover, a live load test was performed by driving a three-axle truck over the bridge and stopping at certain locations. The load on the truck was intended to represent an HS-25+30% AASHTO specified load truck. After this initial load test, 3 ft of cover was put on the bridge and measurements were taken. Additional deflection measurements were taken 6, 12, 18, and 24 months after the completion of the test. It was found that the structure reacted proportionally very little to the live loading. The maximum vertical deflection under the live load was 0.1 in. When the 2 ft of additional cover was added on top of the system, the reaction was larger than the live load, but all the displacement and pressure values were relatively small.

The computer analysis program CANDE was used for analysis. An attempt was made to match the measured properties of the actual soil in the computer model. Researchers found that the analysis agreed generally well with the test results; however, the actual live load deflections were lower than that predicted by the model. This relationship was true of the soil pressures as well.

2.6 – Beach (1988)

This research was related to load testing of a 19 ft span bridge. A cover depth of 1 ft was used for the test. Load was applied in increments of 6.2 kips. The ultimate load of the bridge was reached at a load of 133.4 kips. At this point in the loading, the structure began to deflect a considerable amount without increasing the resisted load. The midspan deflection increased to approximately 2.75 in before the unit failed at midspan. Near the corners and at midspan the tension steel yielded and snapped with the concrete never crushing in compression. The structure maintained a load carrying capacity of 133.5 kips after failure, assumed to have been due to the backfill pressure supporting the legs. Also noted was the appropriateness of treating the legs as pin supported at their base.

The software analysis program CANDE was used to correlate a model to the test results. The model analysis gave good correlation to measured deflections, though the theoretically predicted values were higher. The CANDE model was run without any backfill or cover and it was found that "its load-deformation curve was nearly identical until the loading reached 4.5 times the service load, at which time the effects of the support from the backfill were realized." The researchers concluded that "because of the relatively high stiffness of the culvert, the predominant effect in this test was the structural behavior of the precast unit" and they speculated that the soil-structure interaction would have more effect on longer spans and higher fills.

2.7 – Summary

The projects discussed in this chapter highlight important research related directly to the research and testing done in the Foley Arch project discussed throughout the rest of this thesis. This previous work allowed insight into issues faced in the past with testing arch structures, such as not reaching the ultimate load of the structure due to limitations of the loading devices and difficulty in correlating computer model analysis results to testing results. Regarding modeling, considerations and assumptions of previous research allowed a more thorough and confident approach to modeling the Foley Arch. This previous work allowed an overall more educated and effective approach to various aspects of the project. As a result the project goals were more successfully met.

Chapter 3: Experimental Testing

3.1 – Introduction

Three experimental tests were performed to evaluate the Foley Arch structure. A field test was performed at an actual construction project to evaluate the behavior of the structure under service loads. A laboratory test was performed on a 20 ft clear span Foley Arch. Laboratory test loading continued up to failure of the structure. A second laboratory test was performed on a 36 ft clear span Foley Arch. This laboratory test loading also continued up to failure of the structure. The purpose of the discussion in this chapter is to give enough detail to reasonably convey what tests were performed in order to develop a computer model that correlated to the results of the laboratory tests. Meadows (2012) has supplied more detail on test procedure and results.

3.2 – Field Test

The first experimental test performed was a field test at an active project in Midland, NC. The sections being installed on site had a clear span of 42ft. The objective of this field testing was to evaluate the behavior of a structure under service level loads.

3.2.1 – Field Test Gauges

Vibrating wire strain gauges were installed on the inside face of three individual arch cross sections. Two of the three sections had five gauges installed on their inside face. One of the three sections had the same five gauges installed on the inside face as well as six additional gauges on the exterior face. The locations where the strain gauges were installed are shown in Figure 3-1. The section with the additional gauges on the exterior face also had earth pressure cells installed on the vertical walls of the section. The same section with additional gauges also had two displacement wire potentiometers; one was placed at midspan to measure vertical displacement and the other on a vertical wall to measure horizontal displacement. These were installed on the interior face of the section. The placement and layout of the earth pressure cells and displacement wire pots can be seen in Figure 3-2.

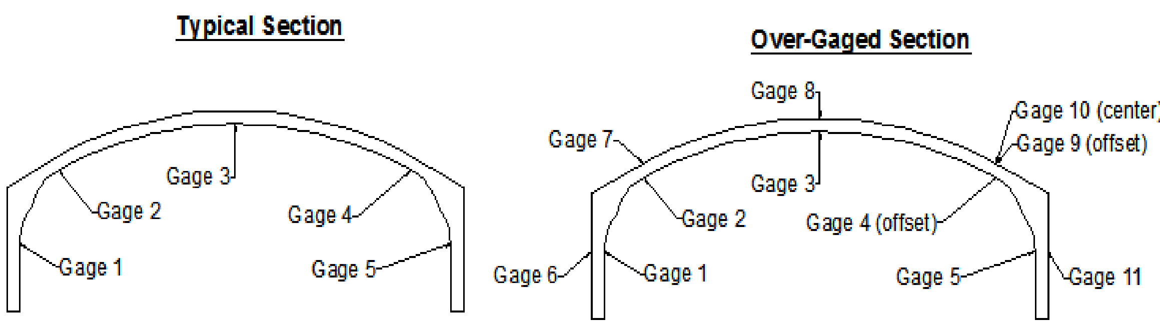

Figure 3-1: Field Test Strain Gauge Layout

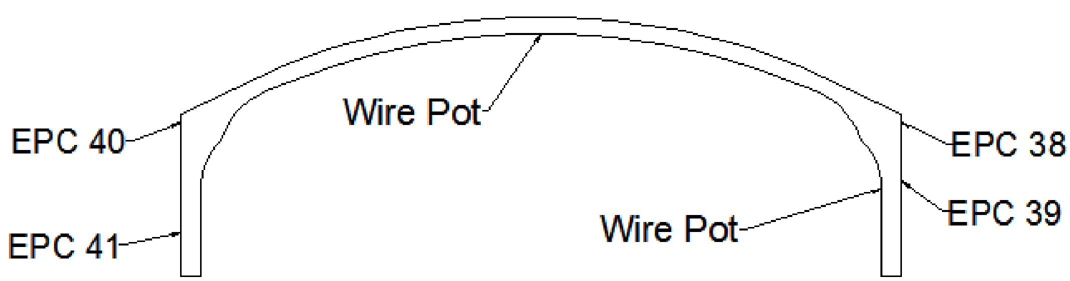

Figure 3-2: Field Test Earth Pressure Cell and Wire Pot Layouts

3.2.2 – Field Test Loading

The load test was performed by using a loaded three axle dump truck with a gross vehicle weight of 56,280 lbs. The truck was incrementally driven across the bridge after backfill had been place and compacted around and on top of the bridge. The trucks axles were positioned and held over specific points on the bridge in order to allow the gauges to record several measurements. The highest strains were recorded when the centerline of the rear axles was placed over the midspan of the bridge. A photo of the truck used to load the bridge at various locations is shown in Figure 3-3. Its position in Figure 3-3, midspan, was the location that induced the greatest measured response.

Figure 3-3: Field Testing – Three Axle Dump Truck at Midspan

3.2.3 – Field Test Results and Behavior

The behavior of the bridge was similar to what was expected. Negative moments developed at the corners of the legs and positive moments developed at midspan. The lateral earth pressure measured near the top of the wall confirmed the arch deflected away from the centerline of the bridge. The lateral pressure was relatively zero at the bottom of the wall, as the bottom was restrained from moving laterally.

3.3 – Laboratory Tests

Two laboratory tests on Foley Arch specimens were performed at the Auburn University Structural Research Laboratory (SRL). The first laboratory test specimen was a 4 ft wide 20 ft clear span precast arch section as shown in Figure 3-4. The vertical rise of the 20 ft span section, at its tallest point, was 8ft clear.

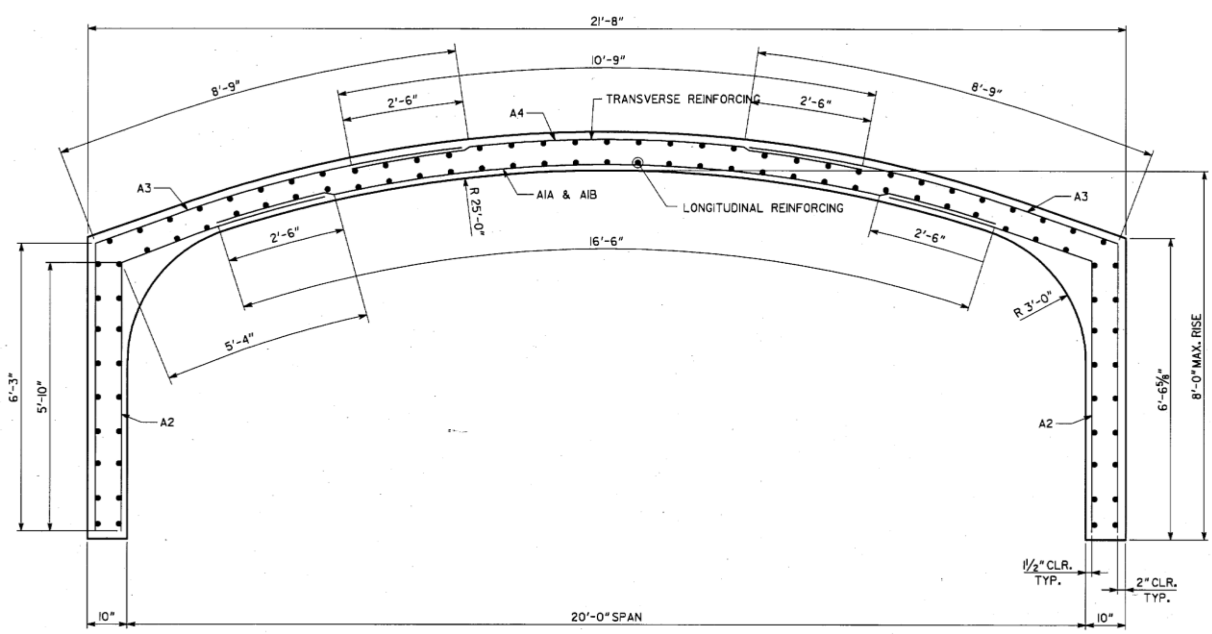

Figure 3-4: Laboratory Test Specimen – 20 ft Clear Span Section

The second laboratory test specimen was a 4 ft wide 36 ft clear span precast arch section as shown in Figure 3-5. The vertical rise of the 36 ft span section, at its tallest point was 9 ft clear. The arch bridge sections to be tested were cast at Foley Products' casting yard in Winder, GA.

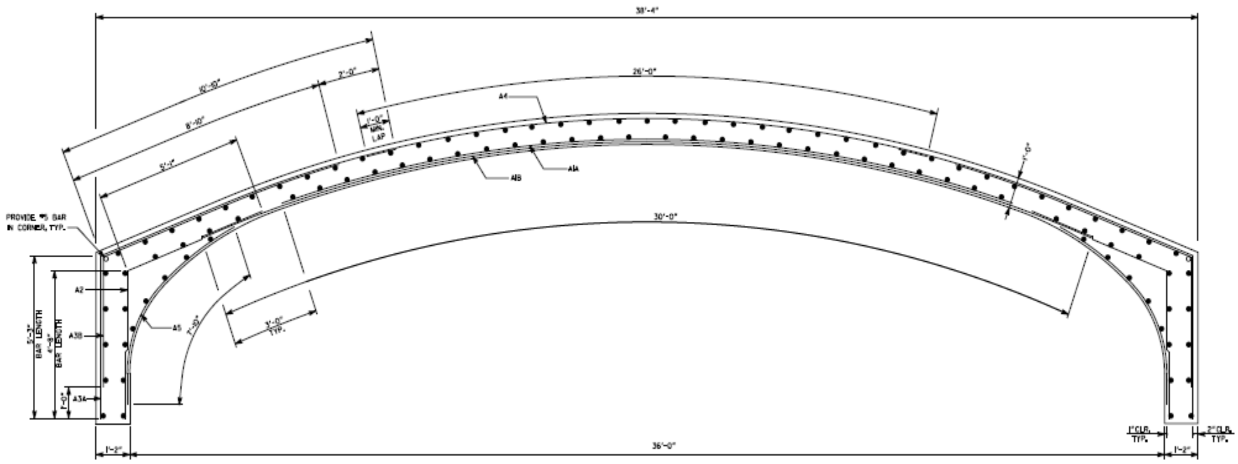

Figure 3-5: Laboratory Test Specimen – 36 ft Clear Span Section

3.3.1 – Laboratory Test Gauges

Prior to the casting date strain gauges were installed on multiple reinforcing bars. This was done by grinding the deformations of the reinforcing bar off, completely smoothing the surface of the bar, and installing gauges with adhesive and protective coverings. These gauged bars were taken to the casting yard and installed on the reinforcing cage of the structure. The bars were placed at locations of anticipated critical stress. A schematic diagram is shown in Figure 3-6 of the approximate location of the reinforcing bar mounted strain gauges. On the 20 ft span test specimen, the corner gauges were an average of 55 in from the edge of the wall, the midspan gauges were placed as close as possible to midspan, and the gauges placed in the legs of the structure were an average of 45 in from the base of the legs. The specific layout of the gauges on the 20 ft clear span test specimen is shown in Figure 3-6.

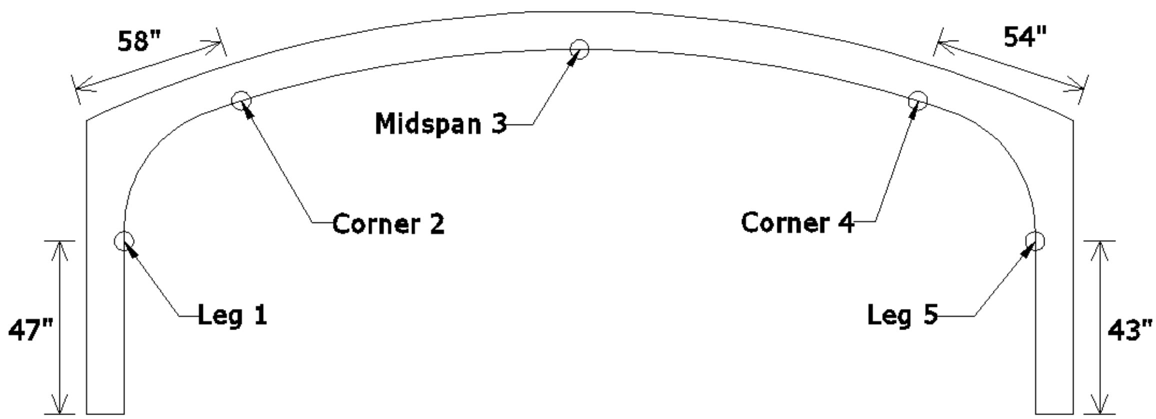

Figure 3-6: Gauge Layout – 20 ft Clear Span Test Specimen

On the 36 ft span there were two sets of gauges placed near the corners; one set of corner gauges was approximately 22 in from the edge of the wall, and the second set was approximately 75 in from the edge of the wall. As with the 20 ft span, the midspan gauges on the 36 ft span were placed as close as reasonably possible to the midspan point of the arch and the gauges installed in the legs of the structure were approximately 20 in from the base of the legs. The specific layout of the gauges on the 36 ft clear span test specimen is shown in Figure 3-7.

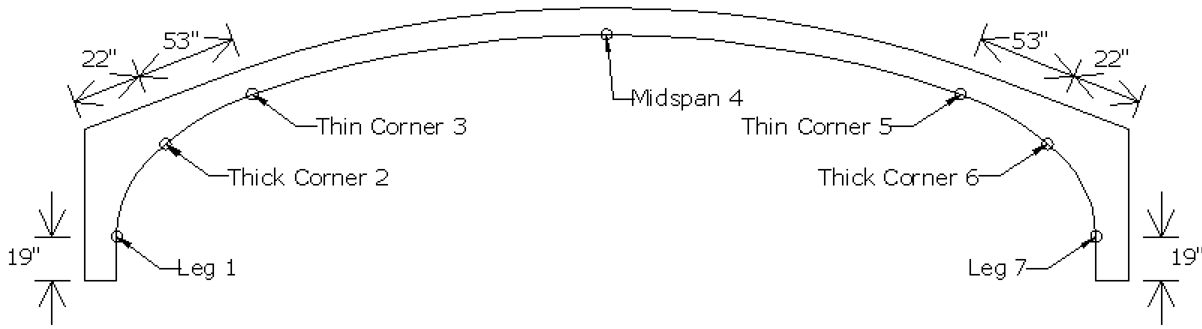

Figure 3-7: Gauge Layout – 36 ft Clear Span Test Specimen

There were three bars/gauges placed across the width of the section at each critical location of interest on the 20 ft clear span test specimen and at the midspan location of the 36 ft clear span test specimen. There were two bars/gauges placed across the width of the section at all locations but midspan of the 36 ft clear span test specimen. A set of gauges was placed on each vertical wall of the structure just prior to the arch forming on the interior face. A set of gauges was placed on the arch portion of the section approximately where a constant thickness of the arch was reached; this was done at both ends of the arch. One additional set of gauges was placed near the quarter span point of the 36 ft clear span specimen. A set of gauges was placed in the arch directly at midspan of the structure. The installation of the reinforcing bar mounted strain gauges to the reinforcing cage of the structure can be seen in Figure 3-8 and Figure 3-9.

Figure 3-8: Reinforcing Cage of Precast Section Prior To Casting

Figure 3-9: Securing Wires of Strain Gauged Reinforcing Bar

Concrete strain gauges were also installed on the face of the concrete after delivery of the test specimen to the laboratory. These were placed at the same stress locations as the strain-gauged rebar. A set of two gauges were placed on the compression side of the concrete surface at each location of interest. A set of gauges was placed on the interior face of each vertical wall of the structure just prior to where the arch formed. A set of gauges was placed on the interior surface of the arch portion of the section approximately where a constant thickness of the arch was reached; this was done on both ends of the arch. A set of gauges was placed on the arch directly at midspan on the exterior face of the concrete. The diagrams of Figure 3-6 and Figure 3-7 shows the approximate locations where the concrete strain gauges were mounted on the compression face of the specimen.

During the 20 ft clear span test, three wire potentiometer deflection measurement devices were used during testing. One measured vertical deflection at midspan and was mounted to a platform that was constructed for that purpose. This midspan wire pot can be seen in Figure 3-10. The other two wire pots were installed on the lateral restraint frames constructed and placed on each end of the structure to measure horizontal displacement. The discussion in this section is limited solely to instrumentation of the structure, see Section 3.3.3 for a discussion of the lateral forces and restraints designed for testing. A wire pot installed on the lateral restraint frame is shown in Figure 3-11.

Figure 3-10: Midspan Wire Pot Deflection Gauge Installed

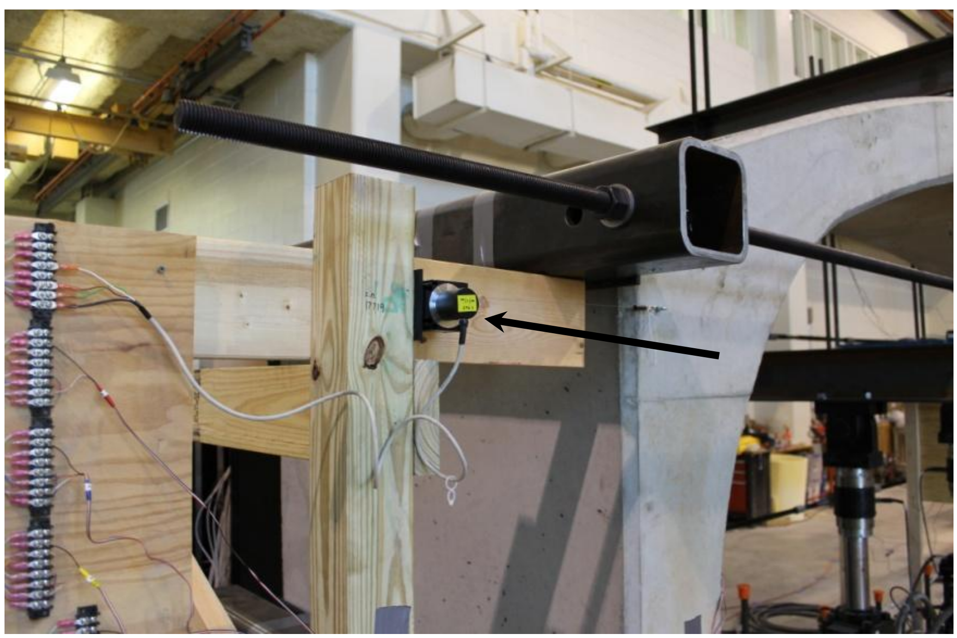

Figure 3-11: Wire Pot Installed on Lateral Restraint Frame

3.3.2 – Laboratory Test Loading Configuration

Three hydraulic actuators were used for the laboratory testing. One was placed at midspan. The two other loading actuators were offset from the middle actuator by 4 ft on either side. A preliminary estimate of ultimate capacity of the structure, including a margin of safety on the estimate, enabled an initial analysis of forces expected at multiple locations. Therefore, appropriately conservative design of the steel members used in the loading configuration was possible. The design of the loading system allowed loading of the structure to be applied from above while having the loading actuators anchored to the floor below the specimen. The loading configuration setup used on the 20ft span structure is shown in Figure 3-12. The loading actuators were anchored to the strong floor of the Auburn University SRL using three-quarter inch thick steel plates. The actuators were in tension as they attached and pulled down on an I-beam that spanned a distance greater than the width of the bridge which in turn was attached to another I-beam placed on top of the bridge. I-beams on top of the bridge had neoprene padding placed between the steel and the concrete section to avoid bearing directly on the concrete. The two outer I-beams, at approximately third points of the structure, also had roller type supports between the neoprene padding and the I-beam to achieve a loading condition approximately vertical from the actuators.

Figure 3-12: Loading Actuator Configuration

3.3.3 – Lateral Soil Pressure Considerations

Three-sided bottomless arch bridge culverts are typically loaded both vertically *and* laterally due to backfill material. These lateral forces occur due to the pressure from the soil against the structure's vertical legs. Lateral pressure develops as the structure experiences heavy vertical loading and lateral deflection at either end occurs. This pressure was calculated for typical conditions given the amount of backfill and the dimensions of the structure. To reasonably approximate the behavior of the structure with the presence of this confining pressure on the bridge, a lateral restraining system was designed to be used during the laboratory testing of the 20 ft clear span specimen. The system, after exploring multiple options, involved an angle mounted to the base plate under each leg of the structure, to restrain any lateral movement outwards at the bottom of the legs. This was appropriate as the typical installation calls for a key in the foundation for the bridge leg to be placed in to restrict lateral motion at the base of the leg.

The range of fill depth considered on the 20 ft test section was 2 ft to 10 ft. An approximate soil unit weight of 100 pcf and a lateral earth pressure coefficient (K_o) of 0.3 was used in calculations. The range of lateral earth pressure at the top of the approximately 6 ft vertical walls of the bridge was calculated to be 60 psf to 300 psf. The range of pressures at the bottom of the vertical walls was calculated to be 246 psf to 486 psf. The minimum of the range of pressures is based on 2 ft of fill over the structure and the maximum of the range of pressures is based upon 10 ft of fill over the structure.

To observe the influence of lateral earth pressure during service loads, a design was developed to place lateral restraints on the exterior faces of the 20 ft clear span section to apply confining pressure. This system induced behavior similar to that produced by passive pressure of the soil backfill in typical application. Two HSS 6x6x1/2 sections were used to apply lateral confinement/load to the section. One HSS was placed at the top of each vertical leg of the structure. The other HSS section was placed at approximately mid-height of the vertical leg of the structure.

A set of threaded steel rods on both sides of the wall spanned the length of the section and attached the HSS sections to each other. These rods were spliced at two locations to allow the insertion of a shorter section of threaded steel rod that was instrumented with a strain gauge. Having a gauged rod on each set of threaded rods allowed adjustment of the tension in each span of rods to the desired amount. The amount of tension to place in the bar was a function of the anticipated horizontal soil pressure experienced by the bridge at a given fill depth. The minimum pressures calculated were for the minimum fill depth of 2 ft and the maximum pressures calculated were for the maximum fill depth of 10 ft. Intermediate fill depths were used to calculate pressures at given points. The calculated earth pressure distribution at various fill depths on the vertical walls of the 20 ft clear span section is shown in Figure 3-13.

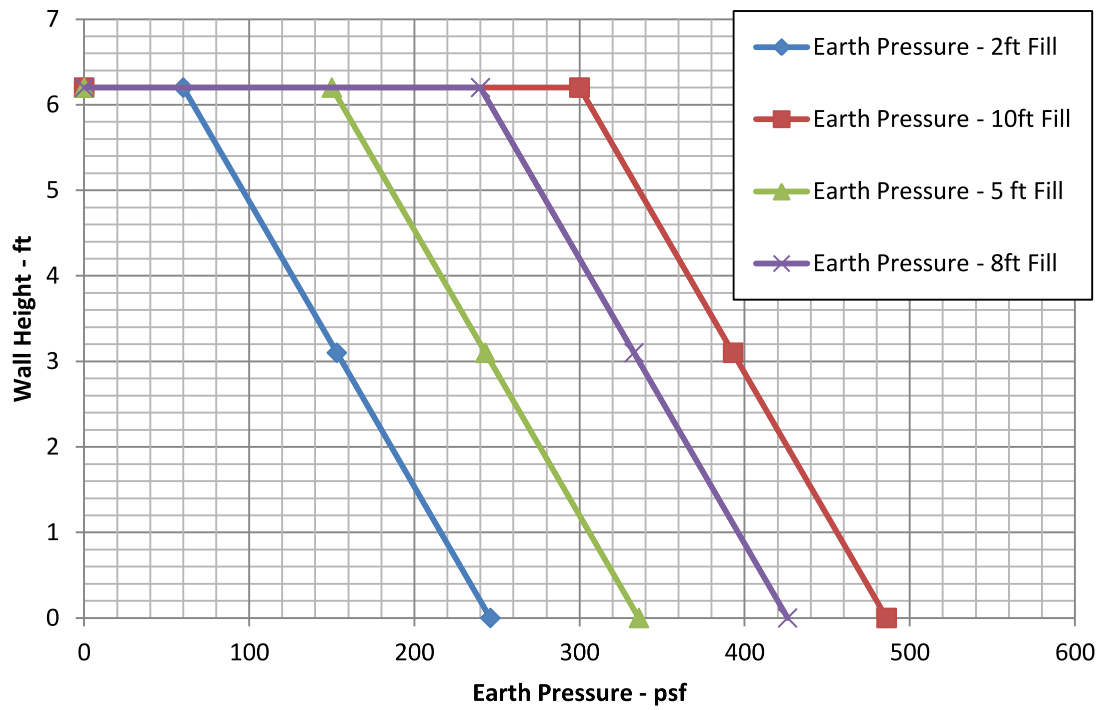

Figure 3-13: Horizontal Earth Pressure Distribution at Various Fill Depths

To calculate the force required in each bar to reach an equivalent loading and behavior as the horizontal earth pressure in application, the relationship between axial force and strain gauge reading had to be determined for each of the threaded rods that had strain gauges placed on them. This was accomplished by using a testing machine to apply tensile load and take readings on the strain measured by the gauge on the threaded rod at small increments. Multiple tests were performed for each of the gauged bars to produce an average axial load to strain gauge reading relationship

A general view of the entire structure with the laterally restraining system in place is shown in Figure 3-14. Figure 3-15 shows a closer view of the HSS sections as they were placed on the wall of the structure. The force required in the threaded rods to create the laterally restrained condition was based on calculations for an approximately median fill depth of 5 ft. With 5 ft of fill on the 20 ft clear span test specimen, the calculated lateral earth pressure at the top of the wall was 150 psf. The pressure calculated at the bottom of the vertical wall of the structure was 336 psf. Using the assumption that the pressure distribution along the wall is approximately linear, the average of the pressures at the bottom and top of the wall provide the pressure at the middle of the wall as 243 psf. The HSS sections were placed approximately at the top and middle of the vertical walls of the test specimen during testing. To calculate the required force in the rods that were spanning between the HSS sections on either end of the structure a tributary area for the top and middle HSS section was used. The HSS at the top of the wall was assumed to account for the pressure on the top quarter of the wall, approximately 1.5 ft. Accounting for the linearly increasing pressure along the wall's depth by averaging the pressure across the tributary depth, the top HSS should apply a distributed load of 260 lb/ft across the width of the section. With a threaded rod on each end of the HSS to provide this confining force, the axial force required in each of the rods on the top HSS was approximately 520 lbs. The middle HSS was assumed to account for pressure on the middle half of the wall, approximately 3 ft. Again accounting for the linearly increasing pressure along the wall’s depth by averaging the pressure across the tributary depth, the middle HSS should apply a distributed load of 730 lb/ft across the width of the section. With a threaded rod on each end of the middle HSS to provide this confining force, the axial force required in each of the rods on the middle HSS was approximately 1,460 lbs.

Figure 3-14: Structure with Lateral Restraint System Installed

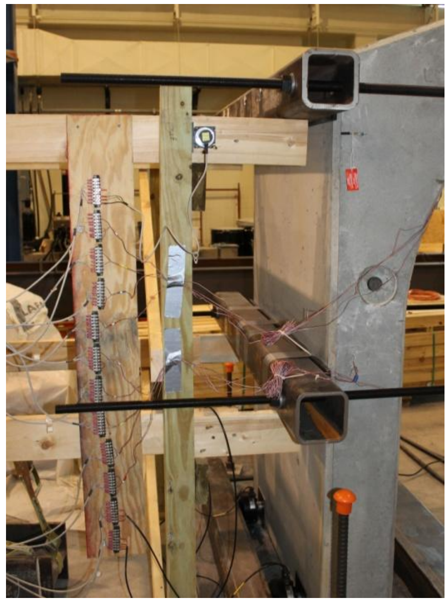

Figure 3-15: Detailed View of Laterally Restraining HSS Sections Installed

3.3.4 –Test Results and Behavior

While the field test provided useful information related to service-level load behavior, it was not used directly for correlation with the computer model developed after testing. Meadows (2012) has provided a detailed discussion of field testing results.

The test structures behaved very much as anticipated throughout the service load range and higher load conditions. The maximum moments were seen at midspan and at the corners of the arches. The maximum vertical deflections were seen at midspan of the structure. The maximum lateral deflection was seen in the corners of the structure. While the magnitude of values varied significantly between the two, the general behavior of the 20 ft clear span test specimen and the 36 ft clear span test specimen was similar. While the failure mode is of interest and was recorded in each of the test specimens, this subject was not the focus of this paper; Meadows (2012) discusses the failure modes observed during testing. The focus of this work was to monitor the ultimate capacity of the structure and to then correlate a structural model constructed in SAP2000 to those test results. Therefore, the remaining portions of this chapter discuss the results of the lab test critical to correlation of the SAP2000 structural model. Meadows (2012) discusses the laboratory testing results in detail.

Using the strain data from gauges installed on the 20 ft clear span arch prior to casting, the internal moments were calculated at critical locations. The moments calculated in the 20 ft clear span arch at midspan and at both ends of the arch are plotted against total load in Figure 3-16. In addition, displacements were monitored using wire pots at midspan and approximately 56 in from the edge (corners) of the 20 ft clear span arch. These displacements are plotted against total load in Figure 3-17.

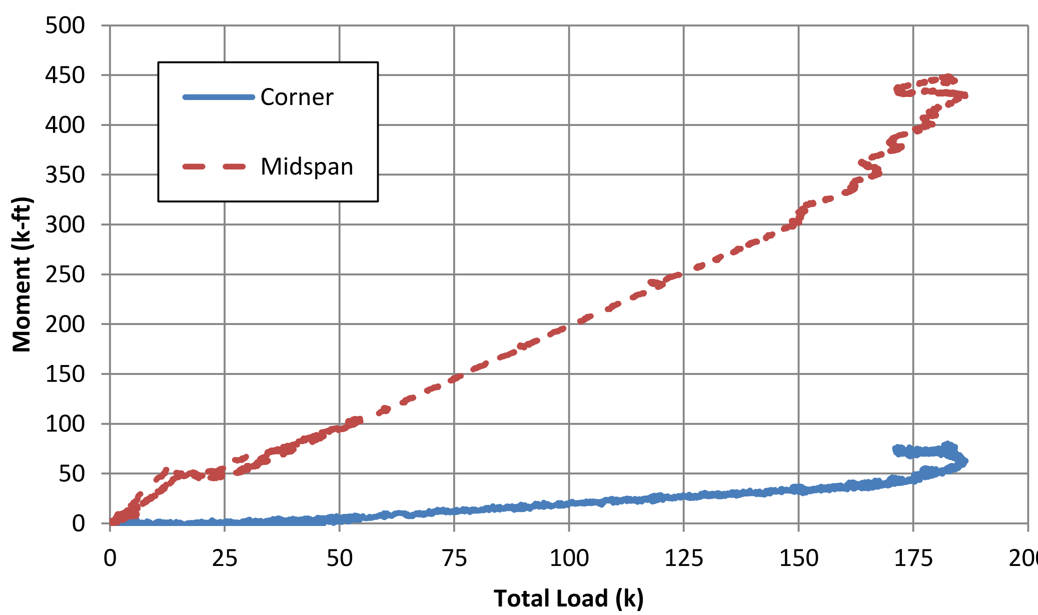

Figure 3-16: Internal Moments in 20 ft Clear Span Laboratory Test

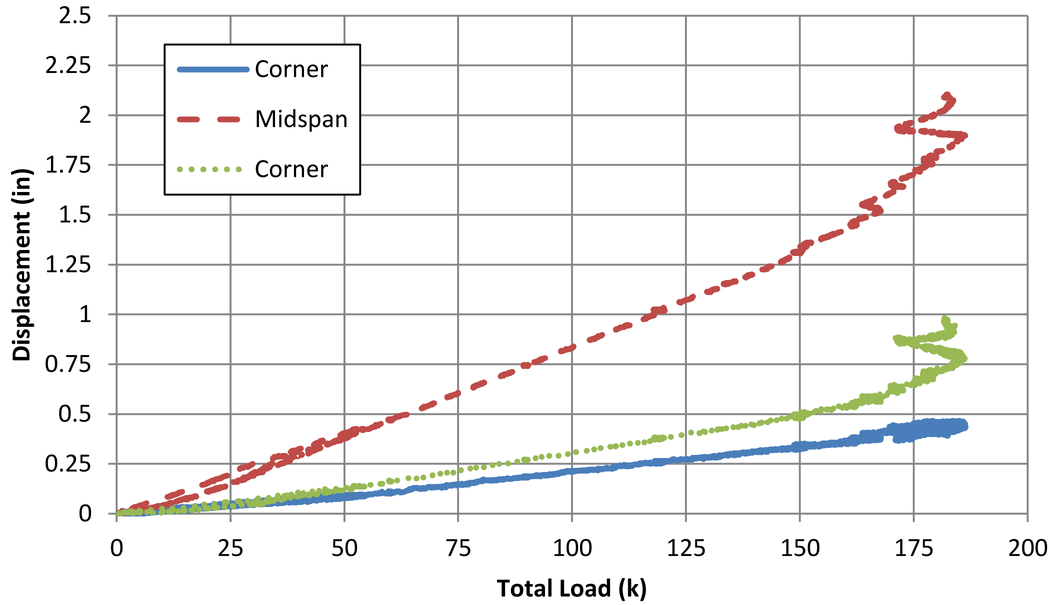

Figure 3-17: Displacements in 20 ft Clear Span Laboratory Test

Similar to the 20 ft span, the strain data readings in the 36 ft span were used to calculate the moments at various locations along the structure. The moments were calculated in the 36 ft clear span arch at midspan, approximately 75 in from the edge of the wall (corner) up the arch, and approximately 22 in from the edge of the wall (thick corner) up the arch. These moments are plotted against total load in Figure 3-18. In addition to moments being calculated using the monitored strain gauges measurements, vertical displacements were monitored using wire pots at midspan and lateral displacement was monitored at the corners of the 36 ft clear span arch. These displacements are plotted against total load in Figure 3-19.

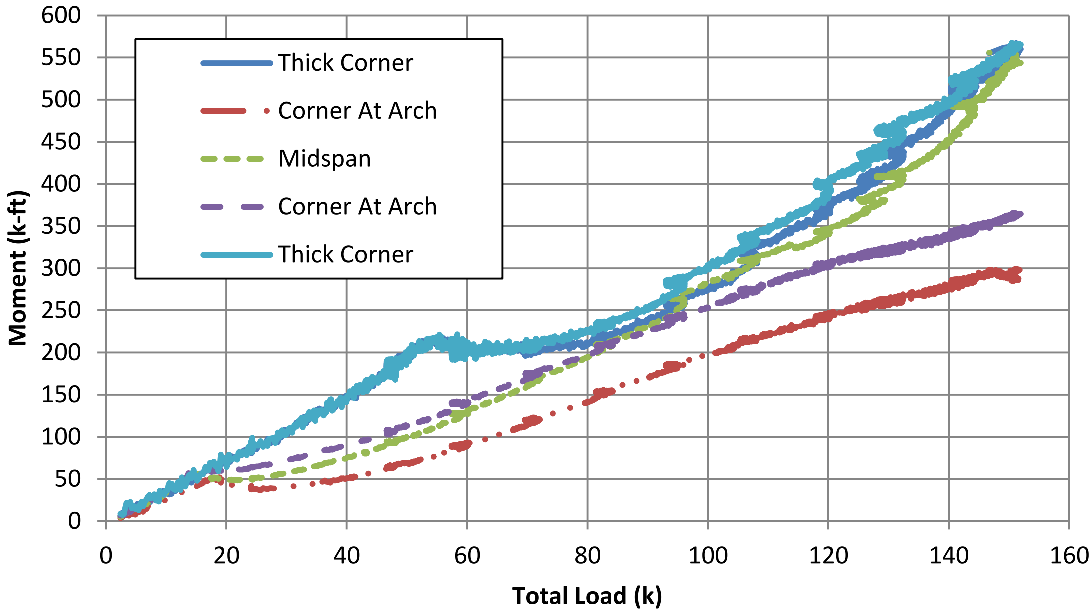

Figure 3-18: Corner Moments in 36 ft Clear Span Laboratory Test

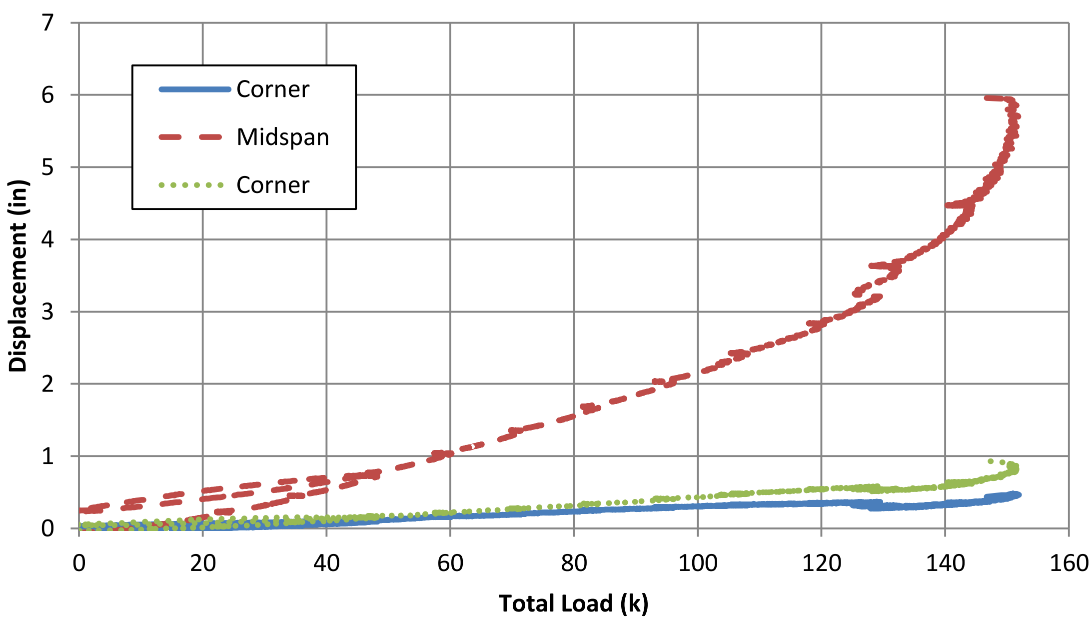

Figure 3-19: Displacements in 36 ft Clear Span Laboratory Test

3.4 – Conclusions

The laboratory test yielded results that were conceptually compatible with the anticipated behavior. The highest moments being developed at midspan and at the corner of the arch section along with the highest deflections being measured at midspan were all logical. The data collected and analyzed provided a solid comparison point to move forward with the project in developing a structural computer model in SAP2000. This process will be discussed in the following chapter.

Chapter 4: Computer Modeling

4.1 – Introduction

The objective in creating a structural computer model was to correlate the results of the model to those found during the laboratory testing described in Chapter 3. The computer software program used for development of the structural model was SAP2000. Since the structure was loaded to failure during the laboratory tests, the range of behavior the model needed to correlate to included nonlinear behavior of the structure. Therefore, SAP2000's (SAP2000) nonlinear analysis capabilities were of particular importance in choosing it to develop the structural model. SAP2000 was chosen in place of other programs that offered more soil-structure interaction capabilities because it was determined that the soil pressure had a minimal effect on the behavior of the bridge at its ultimate capacity. SAP2000's ability to model reinforced concrete was easier to implement and researchers on the project had previous experience with SAP2000.

4.2 – Geometric Properties

Geometric dimensions based on the construction documents and physical measurements provided a reference point for which to construct the structural model. A barrel shell element was created from the new model templates within SAP2000. This allowed for creation of the geometric arch portion of the structure. The appropriate length, roll down angle, radius, axial divisions, and angular divisions were input. The barrel shell created was a three dimensional object. However, the model of the arch was to be a 2-D frame element. Therefore, all objects aside from the end cross section of the barrel shape were deleted and the grid lines and points of the one cross section of the barrel shape were used to create the arch shape of the Foley Arch being modeled.

Once the basic arch shape was created, the legs of the arch were manually drawn in. Grid lines and grid points were added at the appropriate locations below the arch to allow the legs of the arch to be drawn in. The desired number of divisions of the legs dictated the location of the grid lines and points used to draw in the vertical legs. The number of divisions was based on various parameters including how much and at what rate the geometry of the cross section of the laboratory test specimens changed when approaching the corner of the arch. All elements were located along the geometric centerline of the structure.

4.3 – Cross Sectional Element and Material Properties

Using the concrete frame section generator within SAP2000, the cross sectional dimensions and reinforcing details were assigned to the frame elements. While a majority of the frame element could appropriately be assigned as uniform prismatic rectangular sections throughout the length of the element, the corner sections of the structure were in reality much stiffer due to a non uniform section throughout their length. To represent this in the model, non-prismatic frame sections were assigned to these corner elements. The geometric properties of these elements linearly change throughout their length to two distinctly defined cross sections assigned at each end of the element. The prismatic and non-prismatic sections used in the 20 ft clear span model are conceptually shown in Figure 4-1. The prismatic and non-prismatic sections used in the 36 ft clear span model are conceptually shown in Figure 4-2.

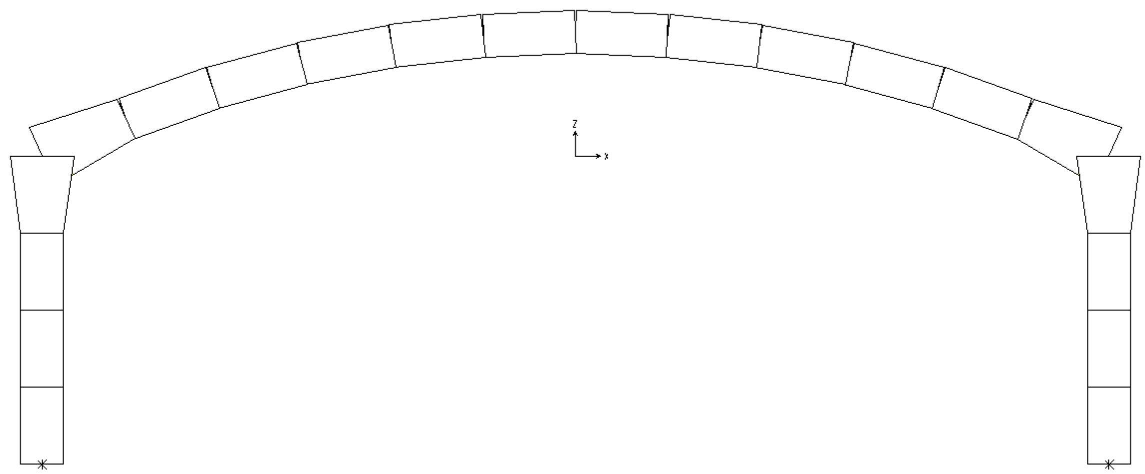

Figure 4-1: 20 ft Clear Span – SAP2000 Elements

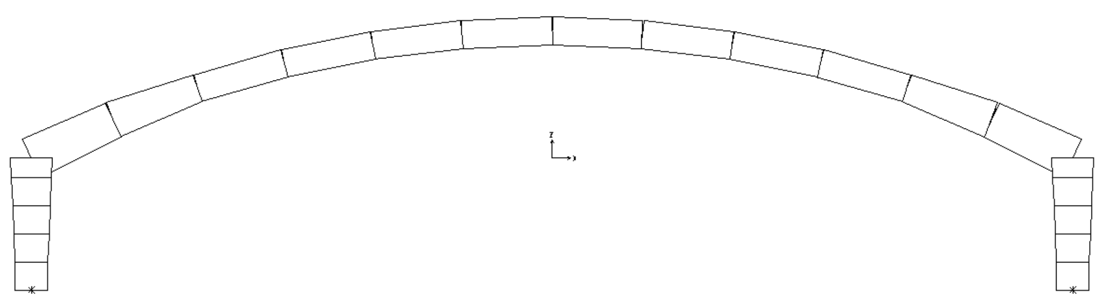

Figure 4-2: 36 ft Clear Span – SAP2000 Elements

The material properties used in the SAP2000 model were a combination of default material properties built into the software and appropriate adjustments to these default properties based on tests performed using material samples taken from the structures tested. Concrete cylinders made from the same batch of concrete used for each of the two arches tested in the Auburn University laboratory were used for modulus of elasticity and compressive strength tests. Reinforcing bar samples were taken from the 36 ft clear span structure for testing. The expected yield strength and ultimate strength of the reinforcing bar were found using these tests.

The tests for modulus of elasticity performed on the concrete cylinders were in accord with ASTM C 469-02 (ASTM). The modulus of elasticity of the concrete used in the 20 ft clear span structure at the time of testing was 5,950 ksi. The compressive strength of the concrete used in the 20 ft clear span structure at the time of testing was 12,500 psi. The modulus of elasticity of the concrete used in the 36 ft span at the time of testing was 4,100 ksi. The compressive strength of the concrete used in the 36 ft clear span structure at the time of testing was 7,040 psi. These concrete material properties were incorporated in the models for the respective 20 ft clear span and 36 ft clear span models. These properties were manually defined within SAP2000. Figure 4-3 shows one of the concrete cylinders in the testing machine after having been tested for ultimate compressive strength.

Figure 4-3: Concrete Cylinder Test

Tests were performed on reinforcing bar samples from the 36 ft clear span structure for use in the computer model of both the 20 ft clear span and 36 ft clear span. The yield strength of the reinforcing steel was 65 ksi. The rupture strength of the reinforcing steel was 80 ksi. These material properties were manually incorporated into the default material settings for A615Gr 60 steel within SAP2000.

4.4 – Loading Configuration

To simulate the behavior in the laboratory testing within the SAP2000 model of the 20 ft clear span structure and the 36 ft clear span structure, the loading configuration was setup to be the same. Therefore, the point loading configuration used in the SAP2000 structural computer model geometrically matched that of the laboratory testing load setup.

Three point loads on the 20 ft and 36 ft clear span SAP2000 structural models were set up at locations imitating those used in the laboratory tests. The load cases incrementally increased the magnitude of each of these three point loads by 10 kips, a 30 kip total increase per load case. The lowest magnitude load case in both models was 20 kips per point load, 60 kips total load. The highest magnitude load case for the 20 ft clear span model was 64 kips per point load, 192 kips total load. The lowest magnitude load case in the 36 ft model was 20 kips per point load, 60 kips total load. The highest magnitude load case in the 36 ft model was 51 kips per point load, 153 kips total load.

4.5 – Nonlinear Analysis

As the laboratory testing carried out loading up to ultimate capacity of the structure, it was necessary to include nonlinear effects in the computer analysis model. This was accomplished in SAP2000 through the use of M3 hinges assigned to various frame elements throughout the structure in conjunction with nonlinear load cases.

4.5.1 –User Defined M3 Hinges

User defined M3 type hinges were decided upon after considering the benefits and data comparison of various models with different types of potential hinges within SAP2000. M3 hinges are dependent on moments at the assigned hinge location. Fiber hinges were experimented with to account for axial load effects. However, axial forces ended up being negligible. Therefore, flexural behavior, or M3 forces, was the dominant contributing factor to increased deformation. User defined M3 hinges accounted for these forces well and produced results with the best correlation those of the actual results in the laboratory. As the cross sectional properties of the structures were very well known from the construction documents, reinforcing detail specifications, and physical measurements taken in the laboratory, it was possible to perform detailed cross sectional analysis calculations for critical cross sections to define the needed range of data points necessary to utilize user defined M3 hinge properties, as opposed to automatically generated M3 hinge properties. The cross sectional properties used in calculating hinge parameters are shown in Table 4-1. The cracking, yielding, and nominal moment capacities and corresponding curvatures were calculated for three different cross sections corresponding to the desired hinge locations. A limiting steel stress of 78 ksi was imposed when calculating the ultimate moment capacity of the cross sections. This was based on the data collected from running tensile tests on reinforcing bar samples from the test specimens. A moment-curvature type property definition was used in defining the hinge properties. The frame hinge property data form for the various M3 hinges allows for definition of five distinct displacement control parameters using a user defined value for moment and a value for curvature. These points are conceptually illustrated in Figure 4-4.These points are labeled A through E. Point A corresponded to the unstressed origin point. Point B was setup to correspond to the calculated cracking moment with a curvature of zero. Point C corresponded to the yield moment and yield curvature. Point D and E corresponded to the ultimate moment capacity and ultimate curvature. These displacement control parameters for the hinges of the 20 ft and 36 ft clear span SAP2000 structural models are shown in Table 4-2 and Table 4-3, respectively. Figure 4-5 and Figure 4-6 are graphical representations of the hinge definition point values given in Table 4-2 and Table 4-3, respectively.

Table 4-1: Cross Sectional Properties Used In Hinge Parameter Calculations

Span	Span Location	Cross Sectional Property					
		b (in.)	h (in.)	d (in.)	A_s (in^2)	d' (in.)	A'_s (in^2)
20 ft	Mid	48	10	8.04	8.02	2.18	1.26
	Quarter	48	10	7.82	2.52	1.68	1.26
	Corner	48	22	19.82	2.52	13.68	1.26
36 ft	Mid	48	12	10.62	5.34	2.13	1.27
	Quarter	48	12	9.64	4.60	1.20	3.04
	Corner	48	28	25.64	4.60	8.15	4.30

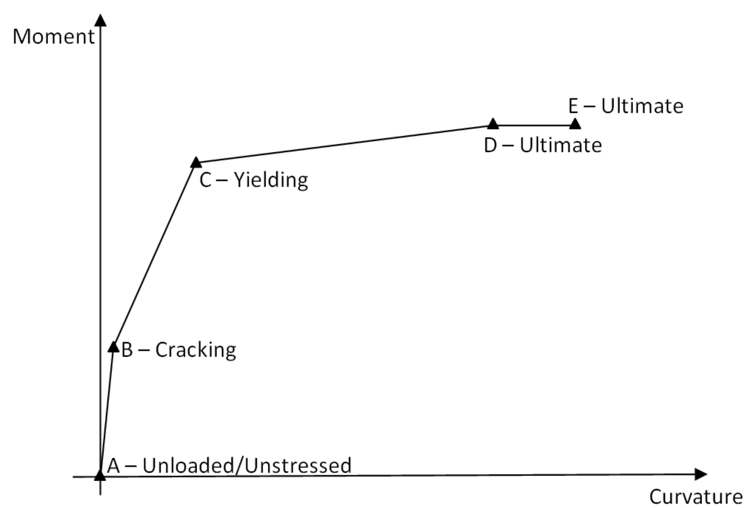

Figure 4-4: SAP2000 Hinge Property Definition Points

Table 4-2: Displacement Control Parameters of Hinges – 20 ft Clear Span Model

Hinge Definition Point	Hinge Location					
	Midspan		Quarter Span		Corner	
	M (k-ft)	Ø (in^{-1})	M (k-ft)	Ø (in^{-1})	M (k-ft)	Ø (in^{-1})
A	0	0	0	0	0	0
B	62	0	58	0	277	0
C	308	0.000428	99	0.000367	299	0.000134
D	388	0.001555	135	0.003375	430	0.003375
E	388	0.001555	135	0.003375	430	0.003375

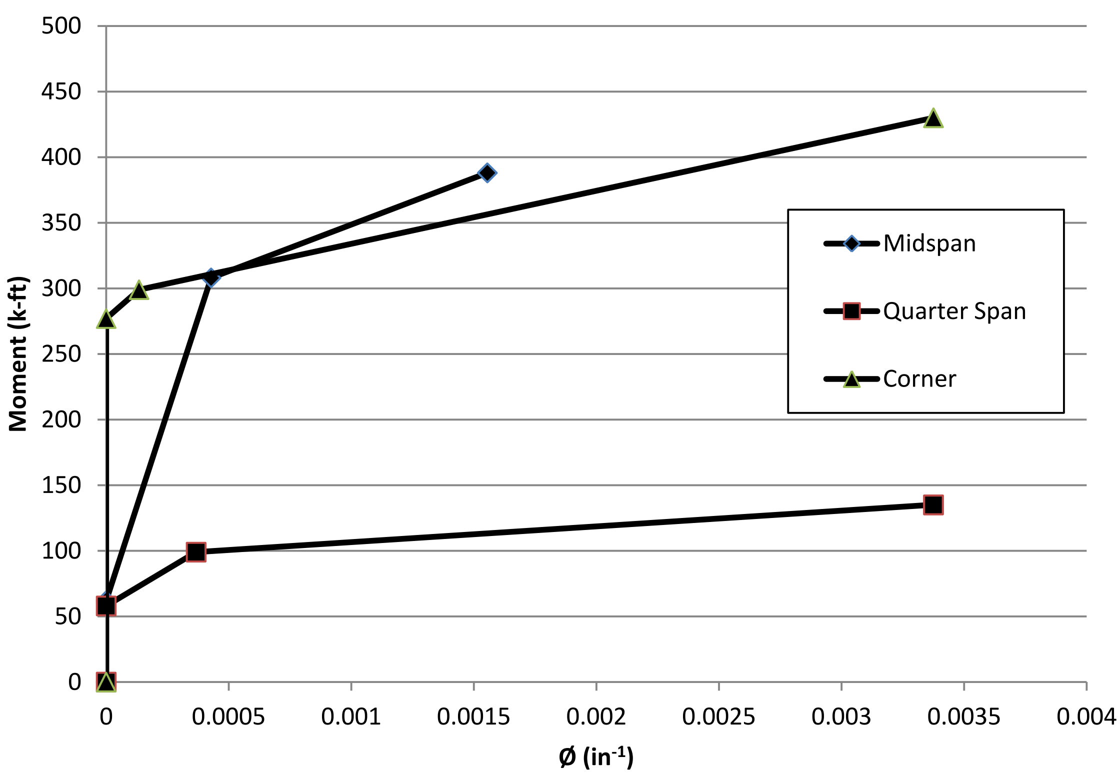

Figure 4-5: Displacement Control Parameters of Hinges – 20 ft Clear Span Model

Table 4-3: Displacement Control Parameters of Hinges – 36 ft Clear Span Model

Hinge Definition Point	Hinge Location					
	Midspan		Quarter Span		Corner	
	M (k-ft)	Ø (in^{-1})	M (k-ft)	Ø (in^{-1})	M (k-ft)	Ø (in^{-1})
A	0	0	0	0	0	0
B	68	0	66	0	346	0
C	276	0.000299	217	0.000321	614	0.000109
D	343	0.001439	269	0.001978	924	0.000867
E	343	0.001439	269	0.001978	924	0.000867

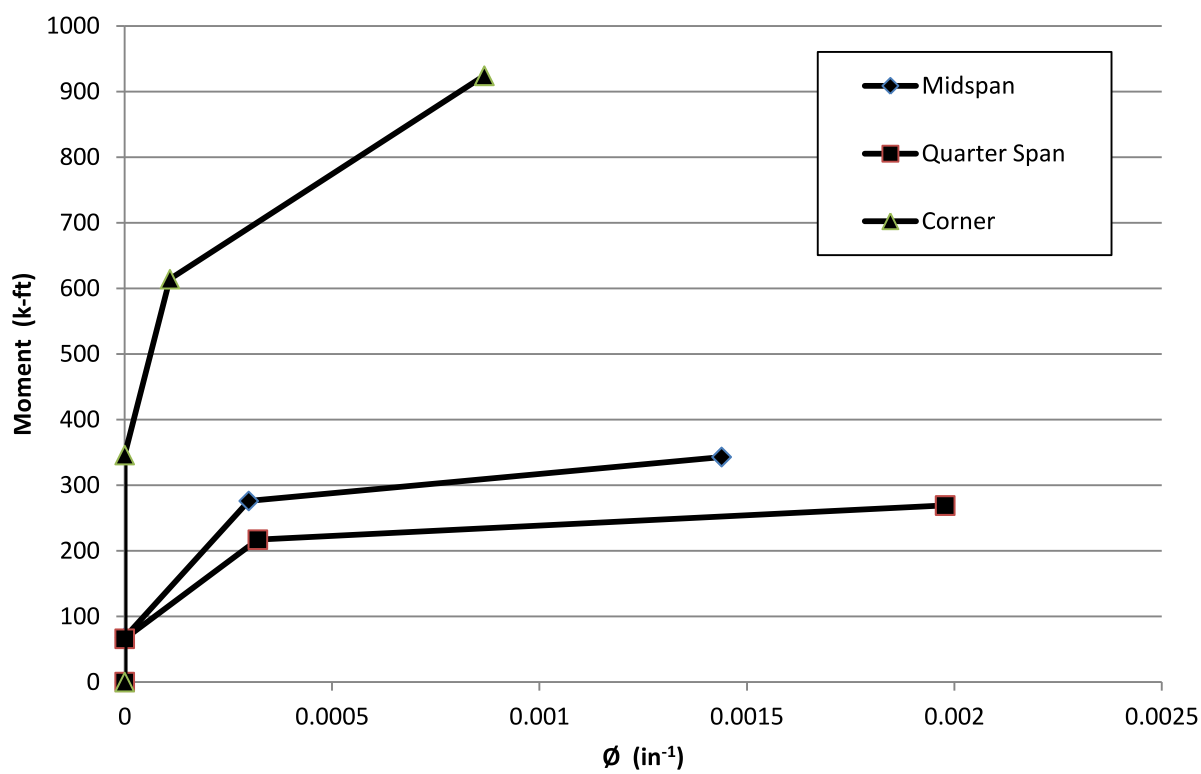

Figure 4-6: Displacement Control Parameters of Hinges – 36 ft Clear Span Model

As can be seen primarily in the moment capacity defined in the hinge property definition, the additional size/thickness of the concrete cross section at the corners of the structure resulted in larger moments at cracking, yielding, and ultimate loading conditions. This was appropriate as the corner sections were physically thicker and stiffer. Therefore, stiffer hinges were needed in the model at those corner locations. An isotropic hysteresis type was used for the hinge definition. The moments and curvatures values calculated and used in the definition of points A through E were used directly; therefore a scaling factor of 1.0 for moment and curvature was defined.

The location of the hinges in the SAP2000 model matched the highest concentration of observed cracking and rotation during the laboratory testing of both the 20 ft clear span and 36 ft clear span structures. The location of the hinges in the 20 ft clear span and the 36 ft clear span SAP200 structural model were (a) the corners of the structure, (b) approximately quarter span of the structure, and (c) midspan of the structure. Since the midspan location of the structure was at the intersection of two individual elements, a hinge was assigned to both elements at the extreme end of each of those two elements. This assured continuity and symmetric behavior at this location. Therefore, a total of six user defined M3 hinges were assigned in the 20 ft clear span SAP2000 structural model. The hinges and their locations in the 20 ft clear span model and the 36 ft clear span model are shown in Figure 4-7 and Figure 4-8, respectively.

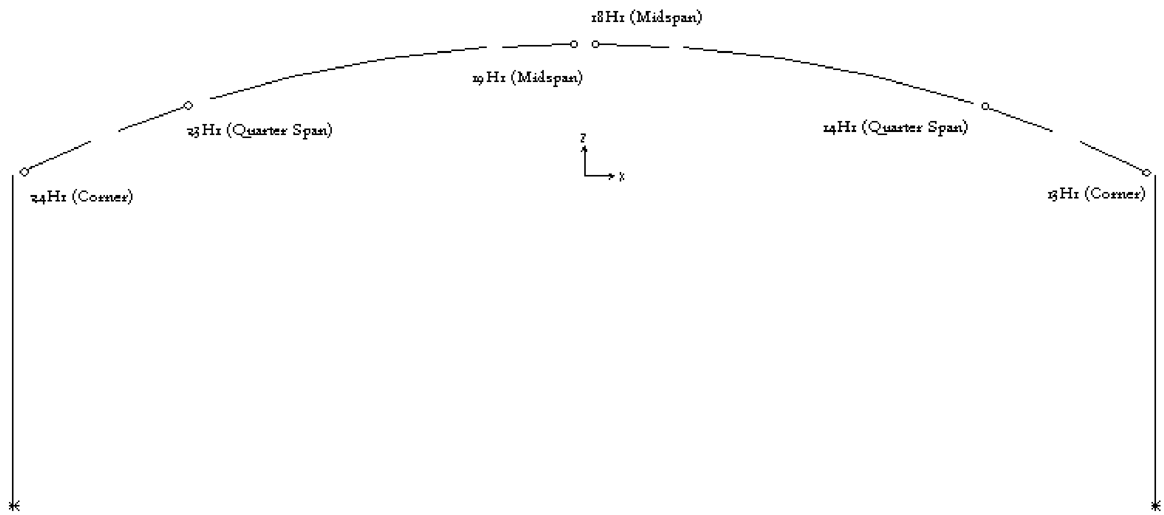

Figure 4-7: Hinge Locations and Labels – 20 ft Clear Span Model

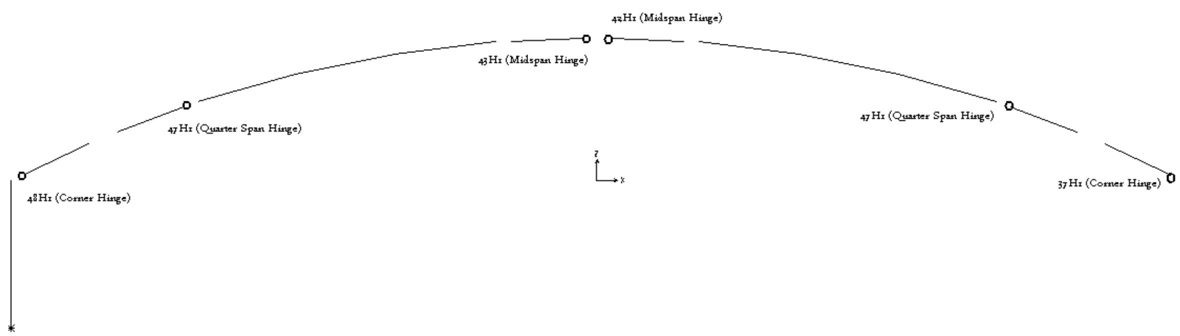

Figure 4-8: Hinge Locations and Labels – 36 ft Clear Span Model

Failure modes are not predicted by SAP2000. However, the “failure point” of the structure was assumed to have occurred when a drastic, unrealistic increase in deflection occurred in the model analysis. For the 20 ft clear span model, this drastic increase in displacement occurred around 65 kips per loading location, 195 kips total load. This was very close to the ultimate total load in testing of the 20 ft clear span of 185 kips. The deflection in the 20 ft clear span SAP2000 model increased from 1.839 in. at midspan with 192 kips total load to thousands of inches at midspan with 195 kips total load. The hinges near the quarter span point were closest to their rotational capacity at the 192 kips total load case than either the midspan hinges or the corner hinges. This indicates that theoretical failure that occurs slightly above a total load of 192 kips; the exact location of the hinge closest to its rotational capacity is not necessarily a reliable prediction of where failure will occur since the hinge properties are all user defined based on cross sectional analysis; even though in the case of the 20 ft clear span the highest hinge rotation did correspond to the laboratory specimen's failure location. However, the ultimate load capacity of the structure could be reasonably predicted based on the comparison of the 20 ft model capacity to laboratory specimen capacity.

For the 36 ft clear span model, the drastic increase in displacement occurred at approximately 52 kips per loading location, or a total load of 156 kips. This was very close to the ultimate total load in testing of the 36 ft clear span of approximately 150 kips. The deflection in the 36 ft clear span SAP2000 model drastically increased from 3.501 in. at a total load of 156 kips to thousands of inches around a total load of 156 kips. The hinges at midspan were closer to their rotational capacity than the hinges at the quarter span points or the corners. However, similar to the 20 ft clear span case, the exact location of the hinge closest to its rotational capacity is not necessarily a reliable prediction of where failure will occur since the hinge properties are all user defined based on cross sectional analysis; this unreliability was evident in the 36 ft clear span test where failure occurred near the quarter span point and not at midspan. However, the ultimate load capacity of the structure was reasonably predicted based on a comparison of the 36 ft model capacity to the laboratory specimen capacity.

4.5.2 –Nonlinear Load Case

SAP2000 allows the incorporation of multiple load cases for desired loading conditions and settings for each individual load case. The load cases used in the 20 ft clear span structural model were all nonlinear static load cases beginning from an unstressed state. Load patterns were assigned to the load cases. These load patterns corresponded to the magnitude of load application being considered in each load case. Six load patterns were assigned. All assigned loads were live loads with a self weight multiplier of zero. The loads were assigned to joints corresponding closest to the actual load location used in the laboratory testing. The smallest load intensity included three point loads with a 20 kip magnitude; a total load of 60 kips. The load intensity increased by 10 kips per load location up to 60 kips per load location; a total load of 180 kips. The final stable load pattern in the 20 ft clear span model was 64 kips per load location; a total load of 192 kips.

During testing of the 20 ft clear span structure it was necessary to use two steel angles, one directly on either side of the middle actuator to reach the ultimate loading capacity of the bridge. A photo of this configuration is shown in Figure 4-9.

Figure 4-9: Alternative Load Application for 20 ft Clear Span Test Specimen

The hydraulic actuators reached their maximum capacity before the ultimate load was reached. This slightly different loading configuration was also incorporated into the SAP2000 model of the 20 ft clear span structure. However, this alternative load application configuration made essentially *no difference* in the moment diagram or other results as compared to a load case with just three point loads. Therefore, all final results for the 20 ft clear span model reported herein correspond to a load case utilizing three load application points.

The load application control was typically set up to apply the full load, or force controlled analysis, with a monitored displacement in the vertical direction at the joint corresponding to midspan. However, at various stages of model development it was advantageous to run a displacement controlled analysis to better observe the behavior of the model structure at various loading increments. This displacement controlled analysis proved useful in identifying potential improvements to details, especially hinge properties, of the model and overall progression of the model in developing good correlation to laboratory results.

It was necessary to save multiple states of the analysis results in order to view detailed results of the hinge behavior throughout load application during analysis. Once the model was in the later stages of development, the final stages of the analysis were of more interest than incremental stages and therefore the analysis was set to include only the final state of analysis.

The default nonlinear parameters and settings incorporated in SAP2000 were primarily used, as opposed to user defined nonlinear settings. The hinge unloading method was to unload the entire structure. The solution control default values were all adequate for the analysis performed.

The structure under consideration was composed of reinforced concrete. As is true of most concrete structures, loading resulted in flexural cracking of the concrete. Cracked sections have a decreased moment of inertia which directly decreases the stiffness of the overall structure. Prior to this cracking, the moment of inertia of a given cross section is often represented simply by the gross moment of inertia of the section. However, after cracking has occurred, it becomes necessary to adjust the moment of inertia to an effective moment of inertia for cross sectional analysis purposes since part of the section is cracked and therefore not contributing to the stiffness of the section. It was necessary to account for this change in section stiffness in the SAP2000 structural models by the use of an I3 property modifier. Calculating this property modifier was done by calculating the effective moment of inertia as discussed in Section 9-4, particularly Equation 9-10b, of Wight and MacGregor's "Reinforced Concrete – Mechanics and Design."

$$I_e = I_{cr} + (I_g - I_{cr}) * (M_{cr}/M_a)^3$$

The I3 multipliers to be incorporated into the SAP2000 model were found by dividing the effective moment of inertia by the gross section moment of inertia. Property modifiers were calculated for and assigned to two different sections of elements. A middle portion of elements and the quarter span portion of elements. The middle portion of elements included four total elements, two on either side of midspan. The quarter span portion of elements included the rest of the elements making up the arch section of the structure except for the most exterior element in the corner on each end of the arch. Since the effective moment of inertia is a function of the actual moment at a cross section under the applied loading, each load case had a different I3 property modifier. The I3 property modifiers used in the 20 ft clear span model and 36 ft clear span model are shown in Table 4-4.

Table 4-4: I3 Property Modifiers

Total Load	SAP2000 Model			
	I3 Property Modifier – 20 ft		I3 Property Modifier – 36 ft	
	Middle Section	Quarter Section	Middle Section	Quarter Section
0	1.0	1.0	1.0	1.0
60	0.45	0.43	0.43	0.44
90	0.36	0.27	0.36	0.30
120	0.34	0.26	0.35	0.27
150	0.34	0.26	0.34	0.26
180	0.34	0.26	N/A	N/A
195	0.34	0.26	N/A	N/A

4.6 Model Analysis Results

The objective of developing a SAP2000 structural model was to correlate the results of the model with those of the laboratory testing. The primary means of comparison used to evaluate how well the model correlated to the laboratory results and to further develop the model were moment diagrams and deflections.

The moments and deflections were recorded at locations of interest after running each of the load cases. Table 4-5 shows the SAP2000 20 ft clear span model results. Table 4-5 shows the SAP2000 36 ft clear span model results.

Table 4-5: 20 ft Clear Span SAP2000 Structural Model Results

Load Case	**Total Load (kips)**	**Midspan Moment (k-ft)**	**Corner Moment (Interior of 2nd Element From Corner) (k-ft)**	**Lateral Deflection (in)**	**Midspan Deflection (in)**
0	**0**	0	0	0	0
20	**60**	97	58	0.06	0.29
30	**90**	146	84	0.12	0.57
40	**120**	203	105	0.16	0.82
50	**150**	267	120	0.22	1.11
60	**180**	331	132	0.32	1.54
64	**192**	376	123	0.37	1.84

Table 4-6: 36 ft Clear Span SAP2000 Structural Model Results

Load Case	Total Load (kips)	Midspan Moment (k-ft)	Corner At Arch Moment (Middle of 2nd Element From Corner) (k-ft)	Thick Corner Moment (Middle of 1st Element At Corner) (k-ft)	Lateral Deflection (in)	Midspan Deflection (in)
0	**0**	0	0	0	0	0
20	**60**	123	133	172	0.14	0.83
30	**90**	193	193	252	0.25	1.59
40	**120**	253	253	333	0.36	2.26
50	**150**	315	315	416	0.52	3.38
53	**159**	339	321	424	0.54	3.50

The moment diagrams and displacements were found for each load case of the 20 ft clear span model and the 36 ft clear span model in order to compare the results to those recorded during the laboratory testing. Figure 4-10 shows a plot of the midspan moment versus the total load for the 20 ft clear span SAP2000 model compared to the midspan moment versus the total load for the 20 ft clear span test specimen. Figure 4-11 shows a plot of the midspan vertical deflection versus total load for the 20 ft clear span SAP2000 model compared to midspan deflection measured during the laboratory testing of the 20 ft clear span test specimen. Figure 4-12 shows a plot of the corner moment versus total load for the 20 ft clear span SAP2000 model compared to the corner moment measured during testing of the 20 ft clear span test specimen. Figure 4-13 shows a plot of the lateral deflections at the corners of the arch versus total load for the SAP2000 model analysis compared to the laboratory testing of the 20 ft clear span.

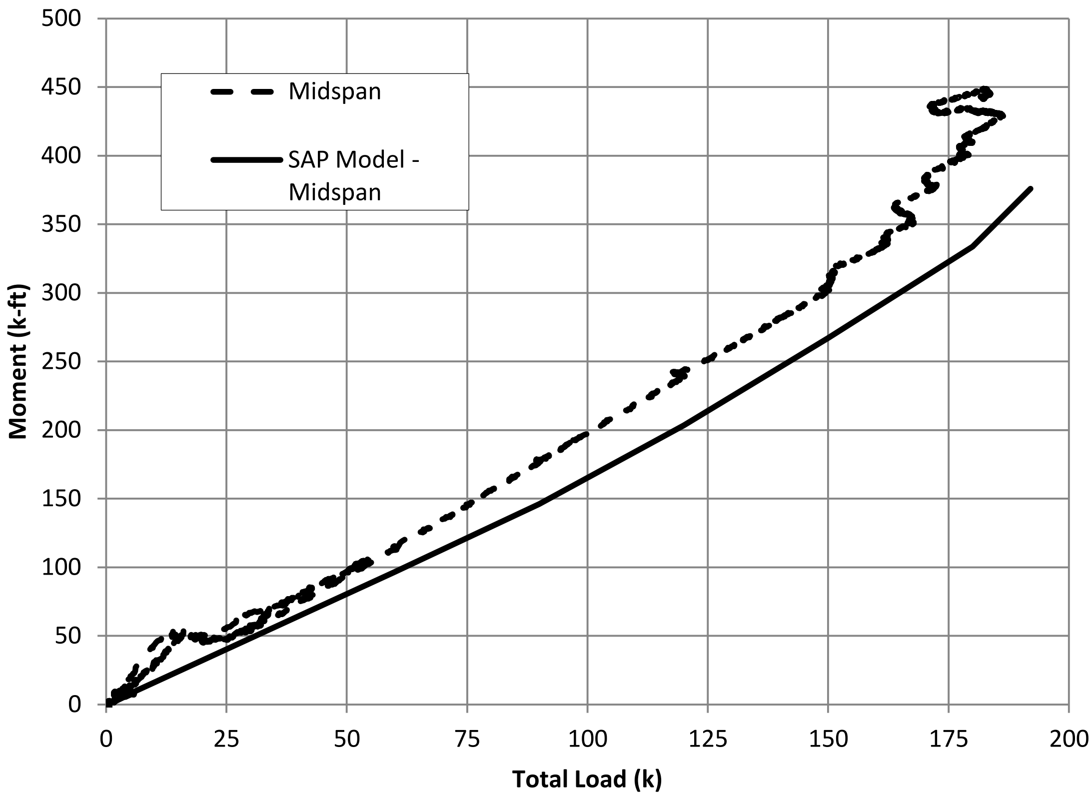

Figure 4-10: Model versus Laboratory Specimen – 20 ft Span Midspan Moment

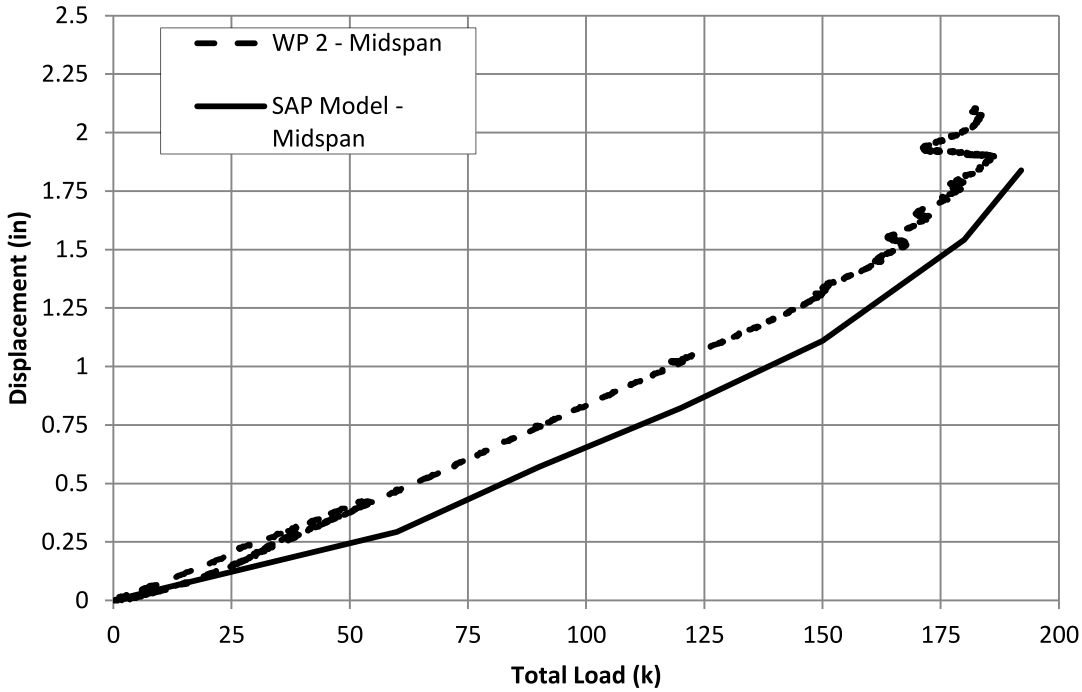

Figure 4-11: Model versus Laboratory Specimen – 20 ft Span Midspan Displacement

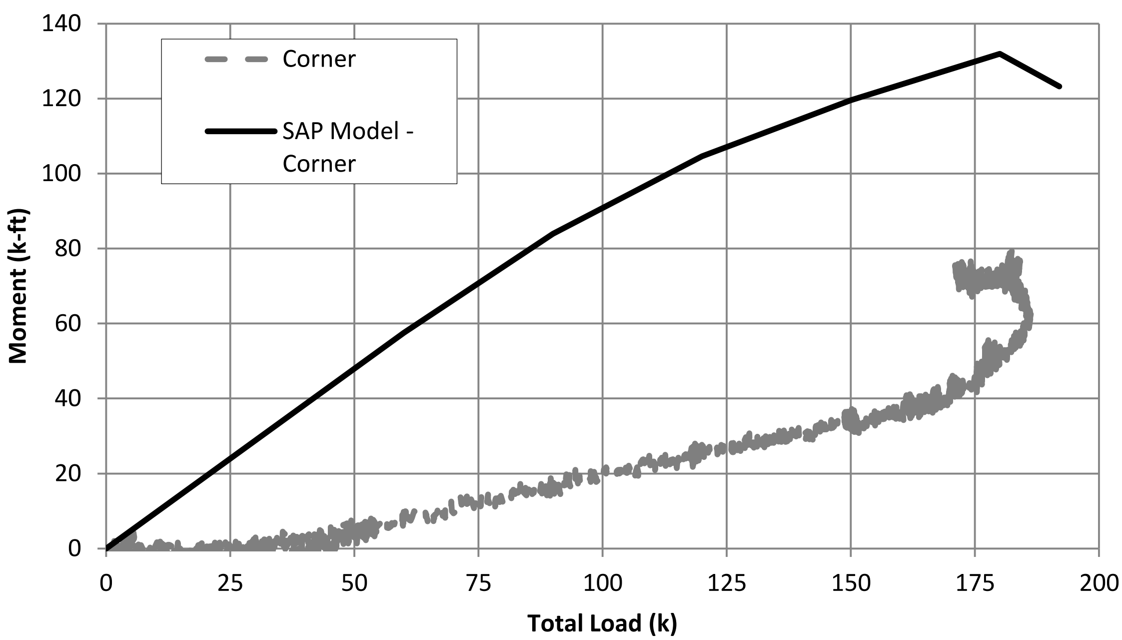

Figure 4-12: Model versus Laboratory Specimen – 20 ft Span Corner Moment

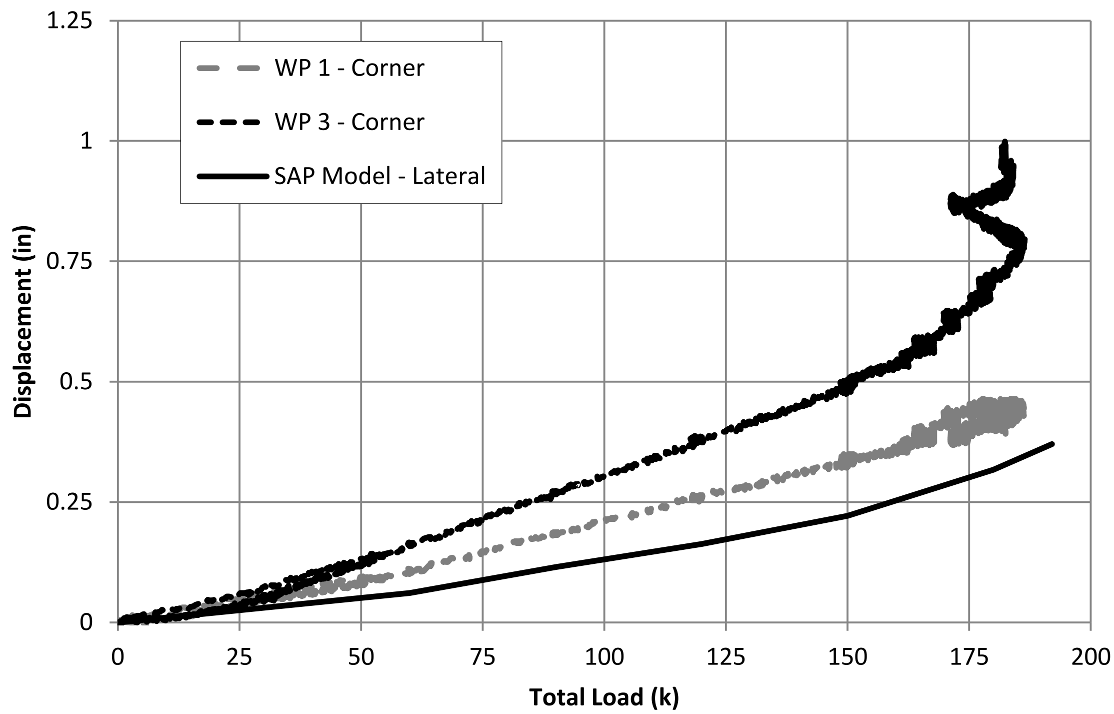

Figure 4-13: Model versus Laboratory Specimen – 20 ft Span Corner Displacement

The midspan moment versus total load plot for the 20 ft clear span SAP2000 model has less than 20 percent difference than the laboratory throughout the range of loading. The SAP2000 model midspan moment was slightly lower than the 20 ft clear span laboratory test midspan moment throughout the range of testing.

The midspan displacement versus total load plot for the 20 ft clear span SAP2000 model is approximately 10 percent lower than the laboratory throughout the range of loading. The SAP2000 model midspan displacement was slightly lower than the 20 ft clear span laboratory test test midspan displacement throughout the range of testing.

The corner moment versus total load plot for the 20 ft clear span SAP2000 model is significantly higher than the laboratory throughout the range of loading. Of the properties recorded for comparison between the SAP2000 model and laboratory specimen, this is the only parameter that was overestimated in the 20 ft SAP2000 model compared to the laboratory testing.

The corner displacement of the 20 ft clear span test specimen was slightly different for the two corners. The 20 ft clear span SAP2000 model was approximately 20 to 30 percent lower than the 20 ft clear span laboratory test specimen corner deflection.

Figure 4-14 shows a plot of the midspan moment versus total load for the 36 ft clear span SAP2000 model compared to the midspan moment versus total load for the 36 ft clear span test specimen. Figure 4-15 shows a plot of the midspan vertical deflection versus total load for the 36 ft clear span SAP2000 model compared to midspan deflection measured during the laboratory testing of the 36 ft clear span test specimen. Figure 4-16 shows a plot of the quarter span moment versus total load for the 36 ft clear span SAP2000 model compared to the quarter span moment measured during testing of the 36 ft clear span test specimen. Figure 4-17 shows a plot of the lateral deflections at the corners of the arch versus total load for the SAP2000 model analysis compared to the lateral deflections measured during laboratory testing of the 36 ft clear span. Figure 4-18 shows a plot of the corner moment versus total load for the 36 ft clear span SAP2000 model compared to the laboratory test data.

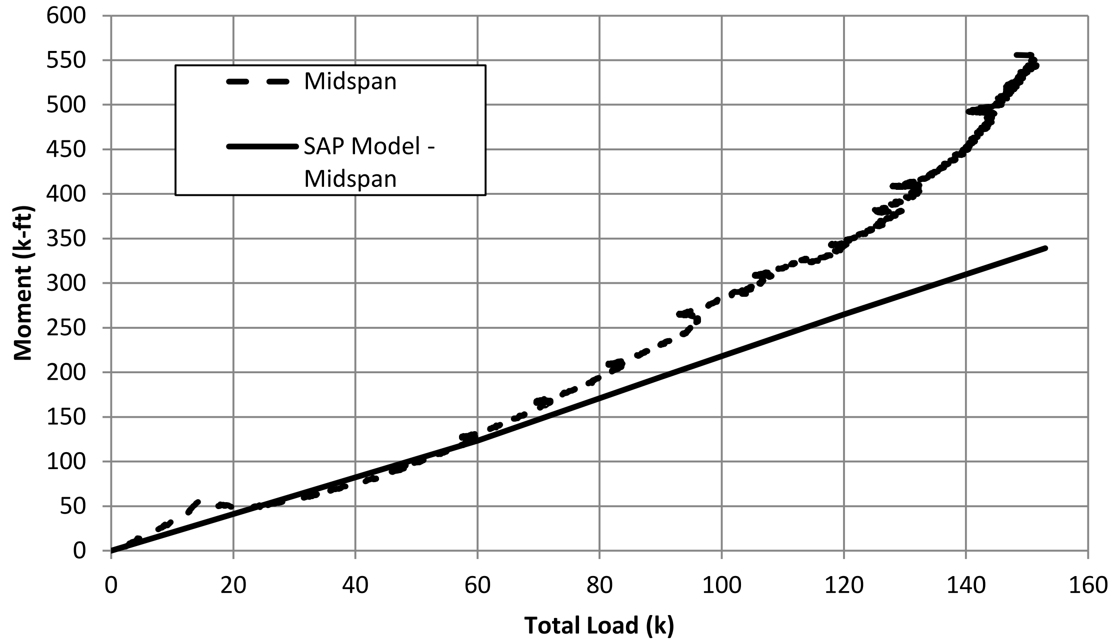

Figure 4-14: Model versus Laboratory Specimen – 36 ft Span Midspan Moment

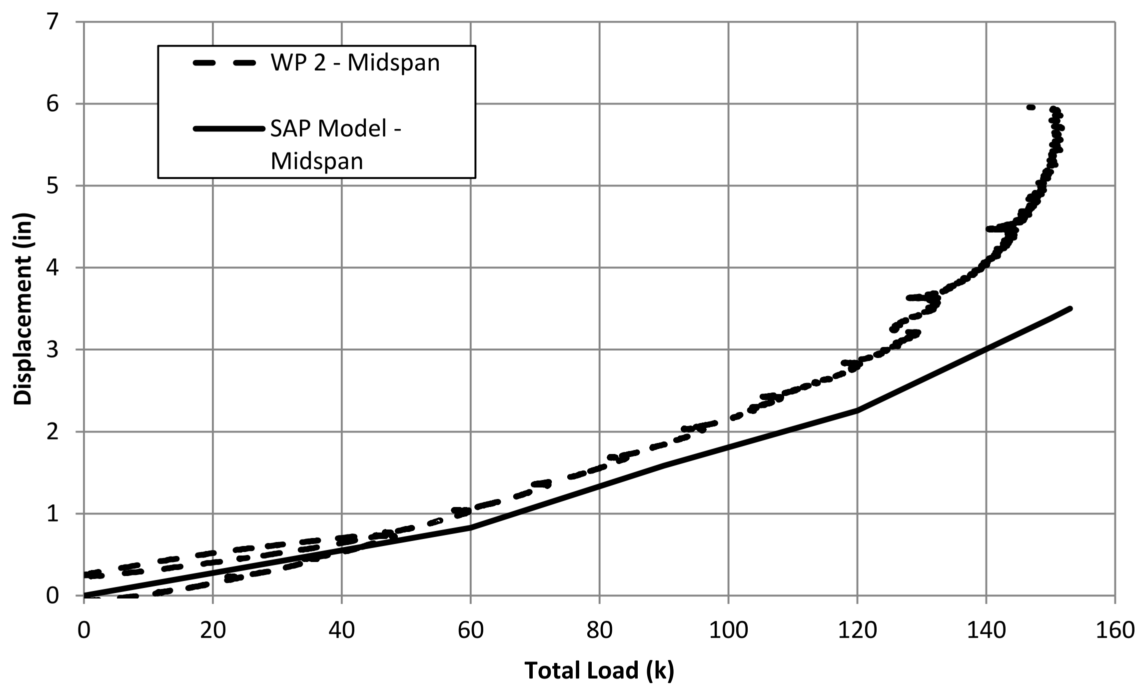

Figure 4-15: Model versus Laboratory Specimen – 36 ft Span Midspan Displacement

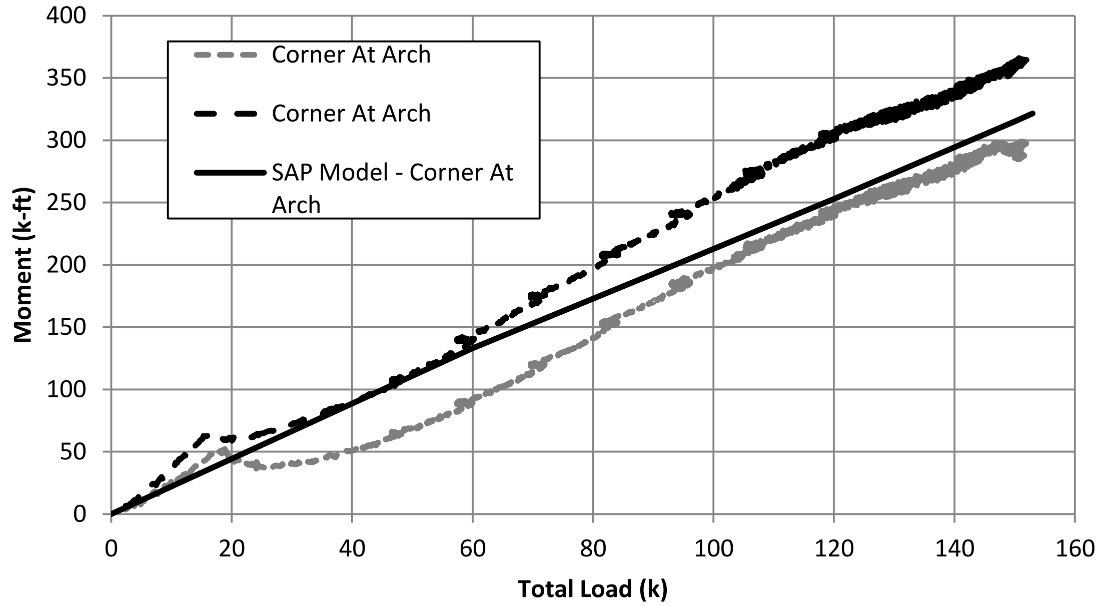

Figure 4-16: Model versus Laboratory Specimen – 36 ft Span Corner Moment

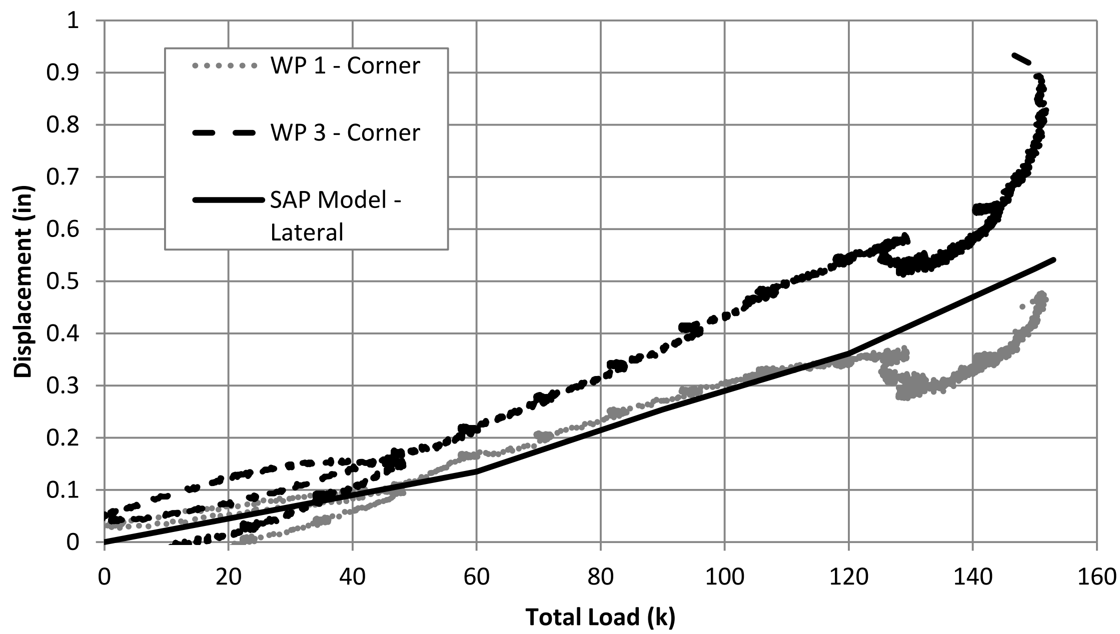

Figure 4-17: Model versus Laboratory Specimen – 36 ft Span Corner Displacement

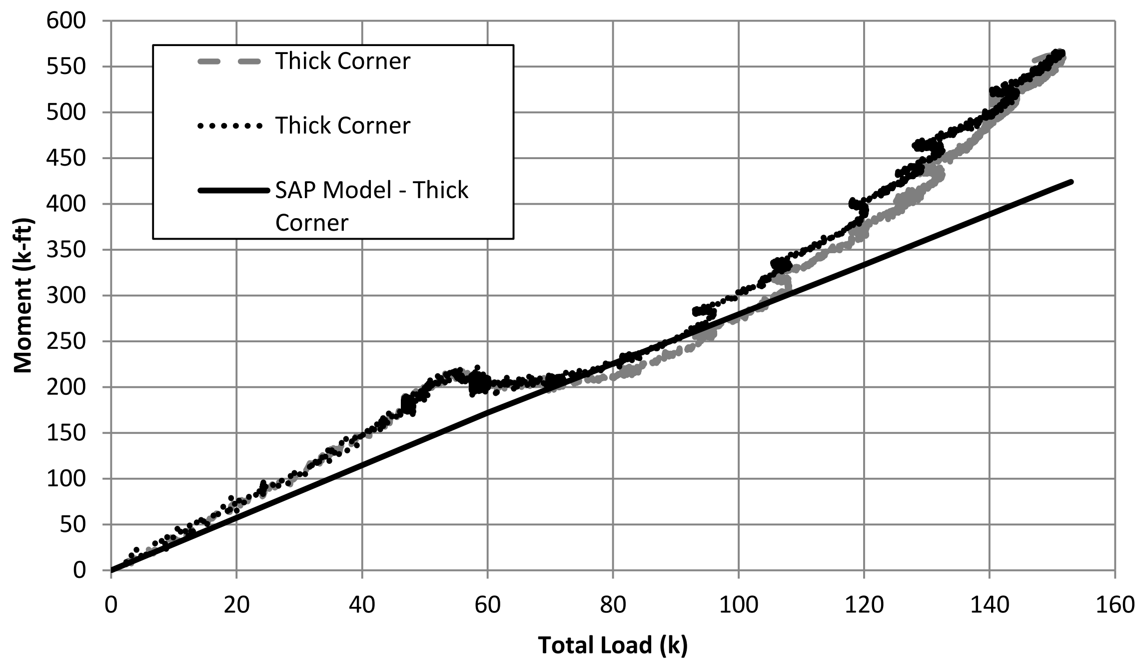

Figure 4-18: Model versus Laboratory Specimen – 36 ft Span Corner Moment

The midspan moment generated by the 36 ft clear span SAP2000 model was less than 5 percent different than the laboratory specimen data throughout the first 40 percent of the load range. After the first 40 percent of the load range, the 36 ft clear span SAP2000 model gradually increased in its underestimate of the midspan moment. At the ultimate load the 36 ft clear span SAP2000 model was approximately 40 percent less than the midspan moment of the 36 ft clear span test specimen. The average difference between the SAP2000 model and the laboratory test specimen was approximately 25 percent.

The midspan displacement generated by the 36 ft clear span SAP2000 model was approximately 5 percent different than the laboratory specimen data throughout the first 70 percent of the load range. For the final 30 percent of the load range, the 36 ft clear span SAP2000 model gradually increased its underestimate of the midspan displacement. At the ultimate load the 36 ft clear span SAP2000 model was approximately 30 percent less than the laboratory test specimen midspan displacement. The average difference between the 36 ft clear span SAP2000 model and36 ft clear span laboratory test specimen for the final 30 percent of the load range was approximately 15 percent.

The two corner at arch moments measured in on the 36 ft clear span test specimen were different by approximately 30 to 40 percent. The corner arch moment generated in the 36 ft clear SAP2000 model fell, in between the two different corner at arch moments measured in the laboratory. Therefore, the SAP2000 model was approximately equal to the average of the two laboratory measured corner at arch moments throughout the load range.

The two corner displacements measured in on the 36 ft clear span test specimen were different by approximately 20 to 25 percent. The corner displacement generated in the 36 ft clear span SAP model fell, on average, right in line with the lower of the two measured laboratory test specimen corner displacements. Therefore, the SAP2000 model was approximately equal to the lower of the two measured corner displacements in the laboratory and 20 to 25 percent less than the higher of the two laboratory measured corner displacements throughout the load range.

The two thick corner moments measured in on the 36 ft clear span test specimen were different by less than 5 percent throughout the range of loading. The thick corner moment generated in the 36 ft clear SAP2000 model fell within approximately 5 to 10 percent of the two different thick corner moments measured in the laboratory for the first

75 percent of loading. For the last 25 percent of the loading range, the 36 ft clear span SAP2000 model underestimated the thick corner moment by an average of approximately 15 to 20 percent

4.7 - Conclusions

For the various load combinations, the SAP2000 structural models provided good correlation of the moments and deflections seen during the laboratory testing. This was especially true throughout the first three quarters of the loading range. Near the ultimate capacity of the arches the model underpredicted, with the exception of the model overpredicting the corner moment of the 20 ft clear span arch, the moments and deflection by 15 to 20 percent. Based on the results, the SAP2000 structural models provided a reasonable starting point for assessment of the design and performance of the arches which will be discussed in the following chapter.

Chapter 5: Design Methodology

5.1 – Introduction

Foley Products provided the design methodology and calculations used by the engineer of record to evaluate the effectiveness of the arch structures' design. The designs were evaluated based upon laboratory testing results described in Chapter 3 and the detailed SAP2000 model described in Chapter 4.

The same methodology was used in design of both the 20 ft clear span structure and the 36 ft clear span structure. The computer program RISA 3-D (RISA Technologies) was used to evaluate various load combinations. The RISA model analysis indicated the highest moment in the structure was at midspan. In the design calculations provided, this midspan moment was used to perform standard reinforced concrete design to specify the required reinforcing.

The effectiveness of the design methodology is discussed in this chapter. Details of the design methodology are provided. The RISA model used in design is compared to the SAP2000 model that correlated to laboratory results. An overall evaluation of the design is provided based on a comparison to laboratory testing and the corresponding SAP model. The ratio of demand to capacity at specific arch sections, namely midspan, was used to determine the effectiveness of the design.

5.2 – General Methodology

The expected loads on the structure were calculated to begin design. Dead load, live load, and lateral loads were calculated. The dead load was a function of the anticipated depth of fill. Live loads were calculated based on the 17th Edition of the American Association of State Highway and Transportation Officials Standard Specification for Highway Bridges HS20 design truck (AASHTO). Allowance was made for impact loading as well as its distribution through a given depth of soil medium based on AASHTO provisions. Lateral loads, or pressures, were calculated based on the depth of fill above the point of interest. The vertical walls of the structure were the main location of interest in calculating the lateral soil pressure. This lateral pressure was calculated based on the unit weight of the soil being approximated at 120 pcf. The lateral pressure is based on the active case with a coefficient of lateral earth pressure equal to 0.3.

The analysis program RISA was used to create a structural model to determine the demand forces. The gravity and lateral loads were incorporated in the model. The controlling moment from the load cases, including load factors, considered in RISA was used to perform cross sectional analysis in designing the section. Details of the RISA model and cross sectional analysis are discussed in Section 5.3.

5.3 – Analysis and Modeling

The RISA structural model used in the design of the Foley Arch was a two dimensional model that was based on common assumptions and practice in structural engineering. The 20 ft clear span RISA model was composed of 34 frame elements. All these elements were rectangular and prismatic beam frame elements. The concrete material defined in the 20 ft clear span RISA model was 5000 psi compressive strength. The reinforcing steel yield strength was 60 ksi. Support conditions in the model consisted of pin supports, no moment resistance, at the base of the vertical structure legs. The extruded view of the members is shown in Figure 5-1. The loads shown in Figure 5-2 are after a load factor of 1.3 was applied to the vertical distributed load, the active lateral load, and the live load within RISA. As can be seen in Figure 5-2, springs with appropriate spring constants, not specified in the design calculations, were used to simulate soil passive pressures as the bridge deflected into the soil. The computer analysis was then run and the moment diagram shown in Figure 5-3 was output by RISA.

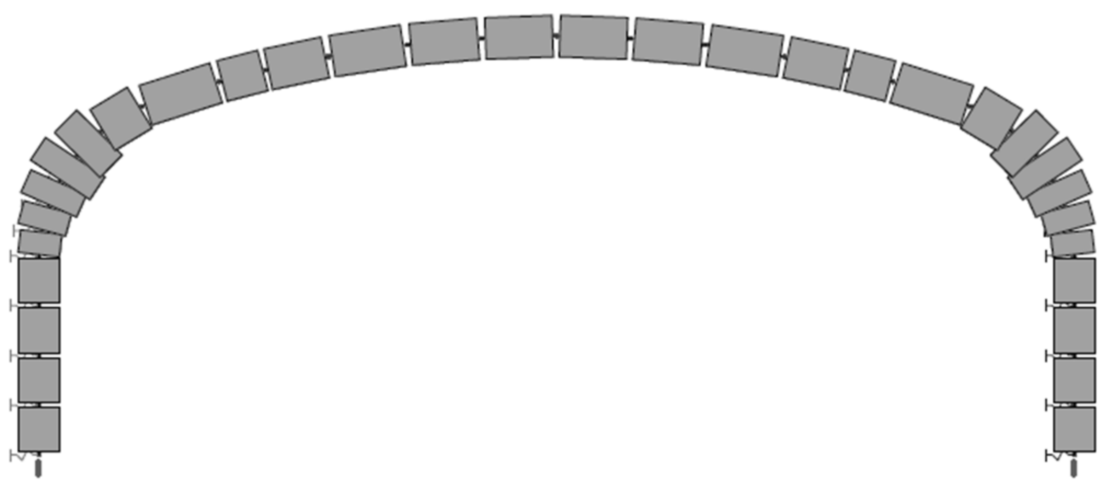

Figure 5-1: Extruded View – 20 ft Span RISA Model

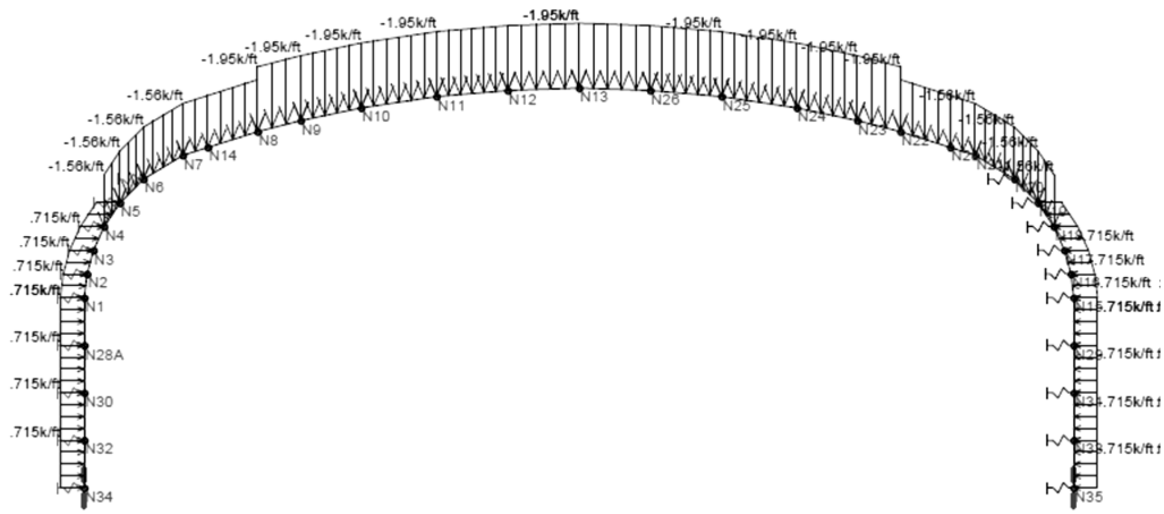

Figure 5-2: Factored Loads – 20 ft Span RISA Model

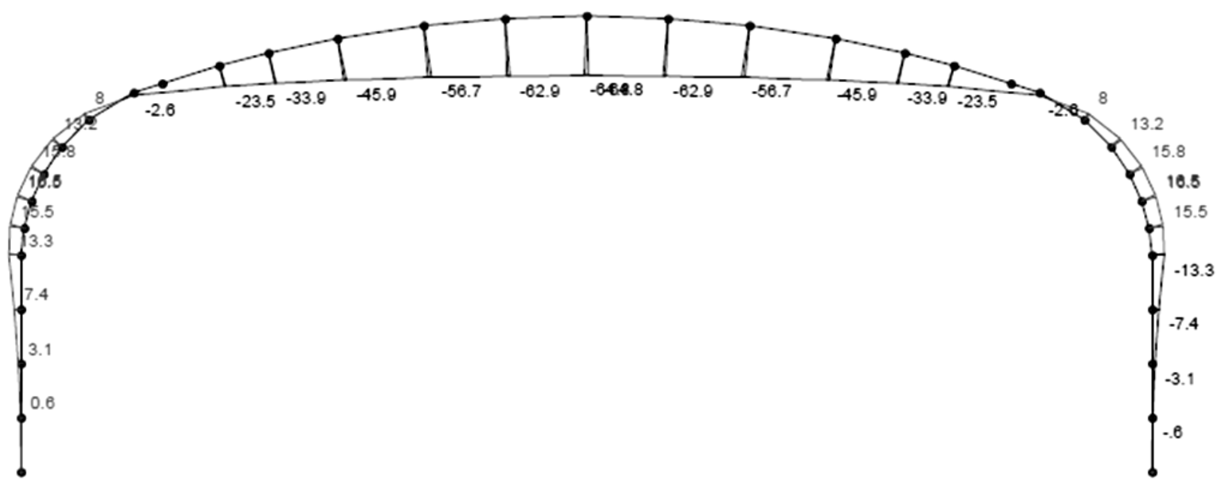

Figure 5-3: Moment Diagram – 20 ft Span RISA Model

The maximum moment used in the cross sectional analysis and design provided was taken at midspan of the structure in Figure 5-3. The moment at this location in the 20 ft span location is 65 k-ft per foot of arch width. All calculations related to reinforced concrete design were in accordance with Section 8.16 of the 17th Edition AASHTO

Standard Specification for Highway Bridges provisions. Based on the demand of 65 kip-ft, the required area of steel for a representative 1 ft wide strip was calculated to achieve the needed capacity. The design calculations were performed for a 1 ft width strip and therefore, to make comparisons with the test structures, they were multiplied by 4 since the bridge tested in the laboratory and modeled in SAP2000 was a 4 ft wide section.

Similar properties and methodology were used in the design of the 36 ft clear span structure. A two dimensional RISA structural model was developed for design purposes. The 36 ft clear span structure RISA model was composed of 42 frame elements. All these elements were rectangular prismatic beam frame elements. The concrete material defined in the 36 ft clear span RISA model was 6000 psi compressive strength. The reinforcing steel yield strength was 60 ksi. A view of the 36 ft span members and the schematic division of beam elements is shown in Figure 5-4. The 36 ft span loads in the RISA model shown in Figure 5-5 are dead loads and the loads shown in Figure 5-6 are live loads plus impact loading. The moment diagram shown in Figure 5-7 is for the controlling factored load combination; this moment diagram was generated within RISA after having run the analysis.

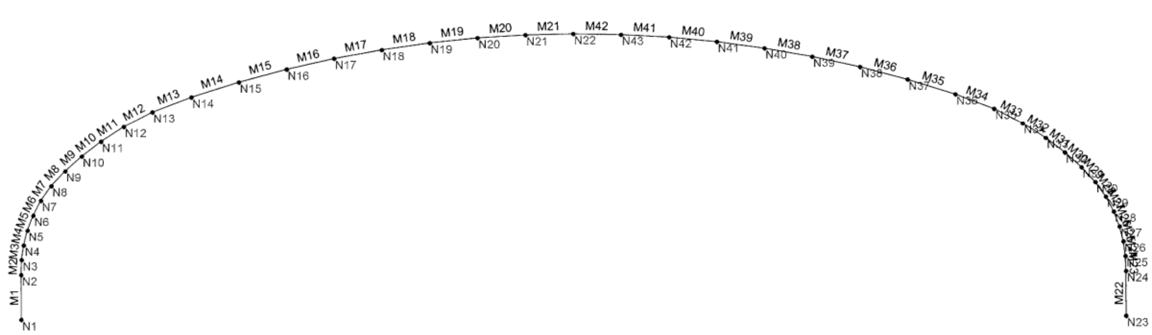

Figure 5-4: Beam Elements – 36 ft Span RISA Model

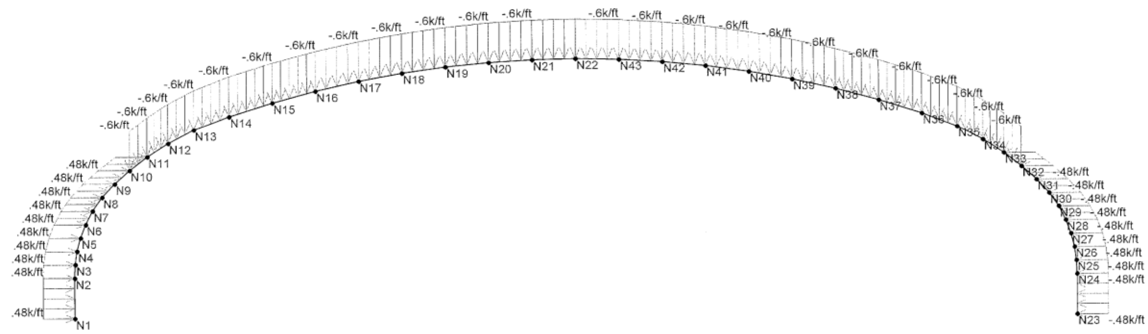

Figure 5-5: Dead Load – 36 ft Span RISA Model

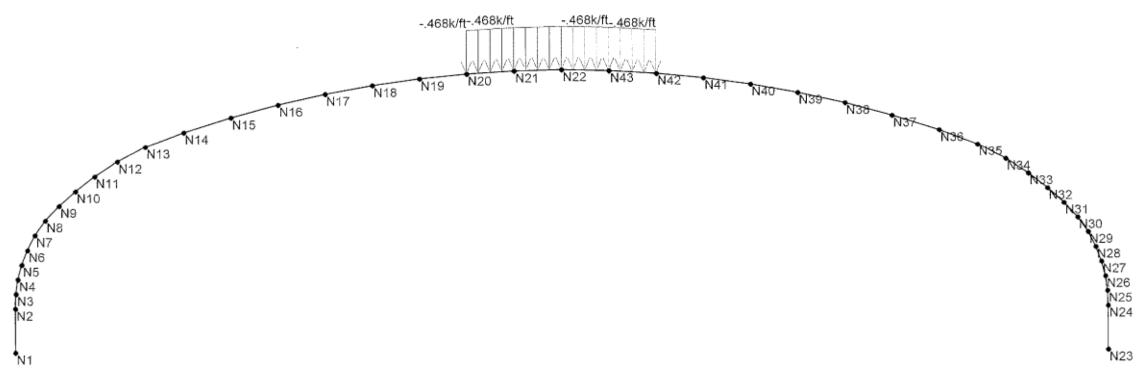

Figure 5-6: Live Load + Impact – 36 ft Span RISA Model

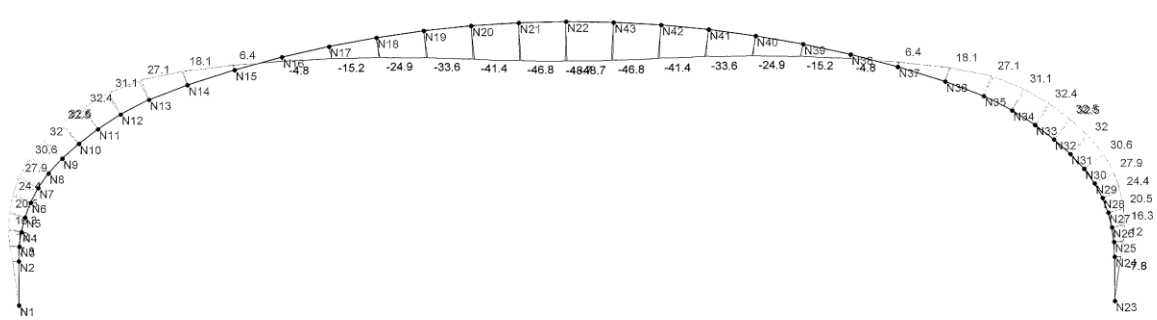

Figure 5-7: Moment Diagram – 36 ft Span RISA Model

The maximum moment used in the cross sectional analysis and design provided was taken at midspan of the structure in, Figure 5-7. The moment at this location in the 36 ft RISA model is 48.7 k-ft per foot of arch width. As was the case with the 20 ft clear span design calculations, all calculations for the 36 ft clear span related to reinforced concrete design were in accordance with Section 8.16 of the 17th Edition AASHTO Standard Specification. Based on the demand of 48.7 kip-ft, the required area of steel for a representative 1 ft wide strip was calculated to achieve the needed capacity. The design calculations were performed for a 1 ft width strip and therefore, to make comparisons with the test structures, they were multiplied by 4 since the bridge tested in the laboratory and modeled in SAP2000 was a 4 ft wide section.

The design calculations were used to collect and record various properties of the Foley Arch as it was designed and modeled in RISA. The required flexural steel area at the midspan cross section was examined; the amount provided in construction of the structure fluctuated slightly with the steel area of actual reinforcing bars. A typical compressive strength of the concrete was designed for. The standard reinforcing bar yield strength was used. These properties were used in standard specified analysis procedures to calculate the nominal flexural moment capacity of the cross section under consideration. These properties are shown in Table 5-1 and are from the design calculations provided by Foley Arch.

Table 5-1: Design Calculation Properties

Structure Clear Span	Design Calculations					
	1 ft Width Flexural Steel Area, A_s (in^2)	Equivalent 4 ft Width Steel Area, A_s (in^2)	Compressive Strength, f'_c (psi)	Steel Yield Strength, f_y (ksi)	1 ft Width Nominal Moment Capacity, M_n (k-ft)	Equivalent 4 ft Width Nominal Moment Capacity, M_n (k-ft)
20 ft	2.40	9.6	6,000	60	76.7	307
36 ft	1.22	4.88	6,000	60	54.4	218

The design moment capacity for the midspan section of the 20 ft clear span structure was 307 k-ft. The design moment capacity for the midspan section of the 36 ft clear span structure was 218 k-ft. One of the reasons for the high design moment capacity of the 20 ft clear span structure is the fact that it was designed for 10 ft of fill. The 36 ft clear span structure was designed for half the fill of the 20 ft structure, 5 ft. Depending on the depth of fill being designed for, the dead load can easily be, and often is, the largest contributing factor to the demand load on the structure. In the two structures under consideration herein, the design loads due to the soil fill were substantially greater than the live loads resulting from AASHTO specified bridge loading requirements. Therefore, the design is heavily dependent on the depth of fill being designed for.

A moment distribution comparison between the RISA models and the SAP2000 models was made. The factored design loads put on the structure in the RISA model were for a 1 ft wide strip of the bridge. Therefore, the values from the 1 ft wide strip calculations and modeling could be multiplied by the width of the desired section. Since the test structures were 4 ft wide, the factored loads form the RISA models were multiplied by 4 to get the equivalent loads to put into the SAP2000 model for comparison. There were three sections of load magnitudes to consider. These were the horizontal loads applied on the vertical legs of the structure simulating lateral earth pressure, vertical loads on the exterior ends of the arch simulating primarily dead load of the soil on the structure, and the loads on the middle portion of the arch simulating a combination of both dead load of the soil on the structure as well as the required live load. The factored design loads input to the SAP2000 20 ft clear span model are shown in Figure 5-8. The resulting moment diagram for this load case on the 20 ft clear span in SAP2000 is shown in Figure 5-9. Figure 5-8 and Figure 5-9 are of half the structure for clarity. The moment diagram is symmetric about the midspan point of the structure.

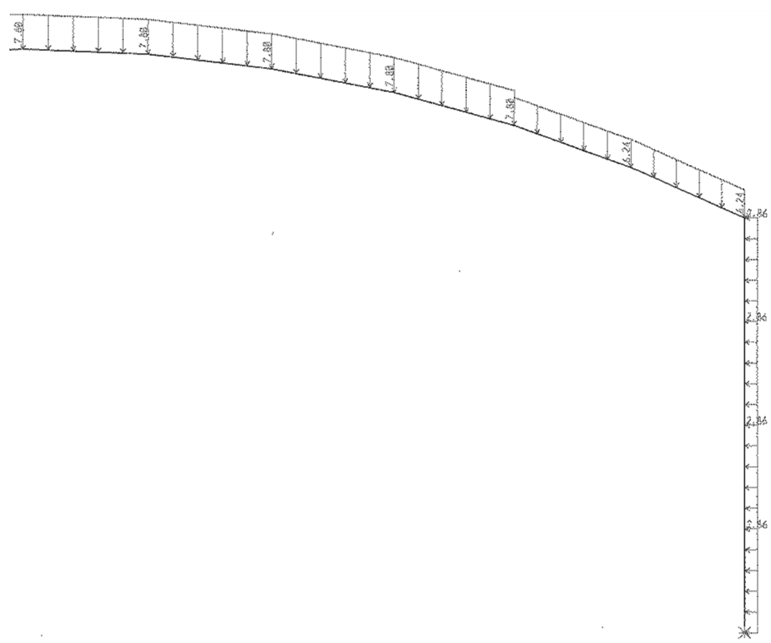

Figure 5-8: Equivalent Factored Design Loads on 20 ft SAP2000 Model

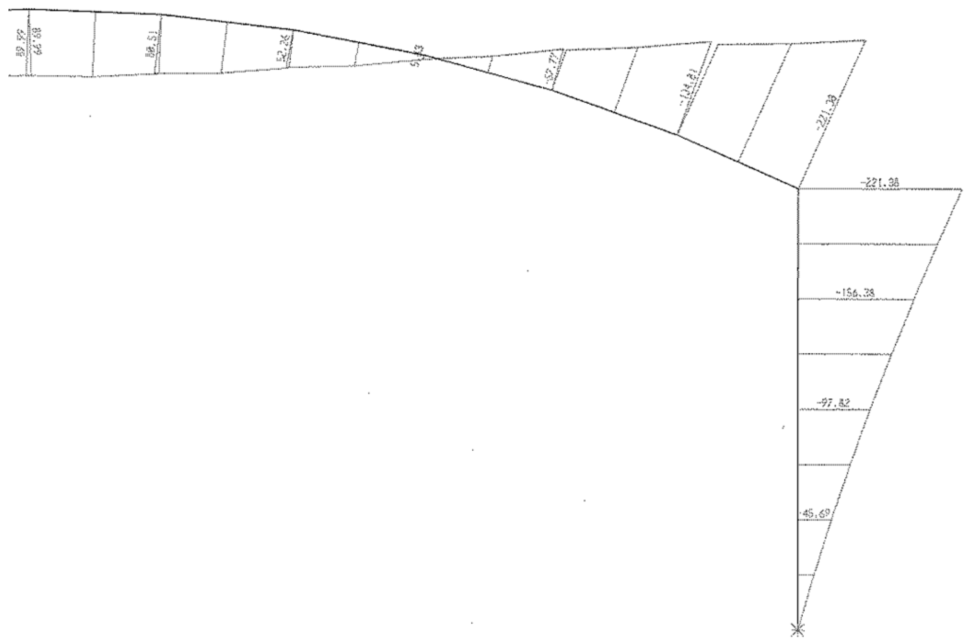

Figure 5-9: Moment Diagram of Factored Design Loads on 20 ft SAP2000 Model

The moment at midspan of the SAP2000 model under the factored design loads was approximately 90 k-ft. The moment at the corners of the arch of the SAP2000 model under the factored design loads was approximately 222 k-ft.

The equivalent factored design loads input to the 36 ft clear span SAP2000 model are shown in Figure 5-10. The resulting moment diagram for this load case on the 36 ft clear span in SAP2000 is shown in Figure 5-11. Figure 5-10 and Figure 5-11 are of half the structure for clarity. The moment diagram is symmetric about the midspan point of the structure.

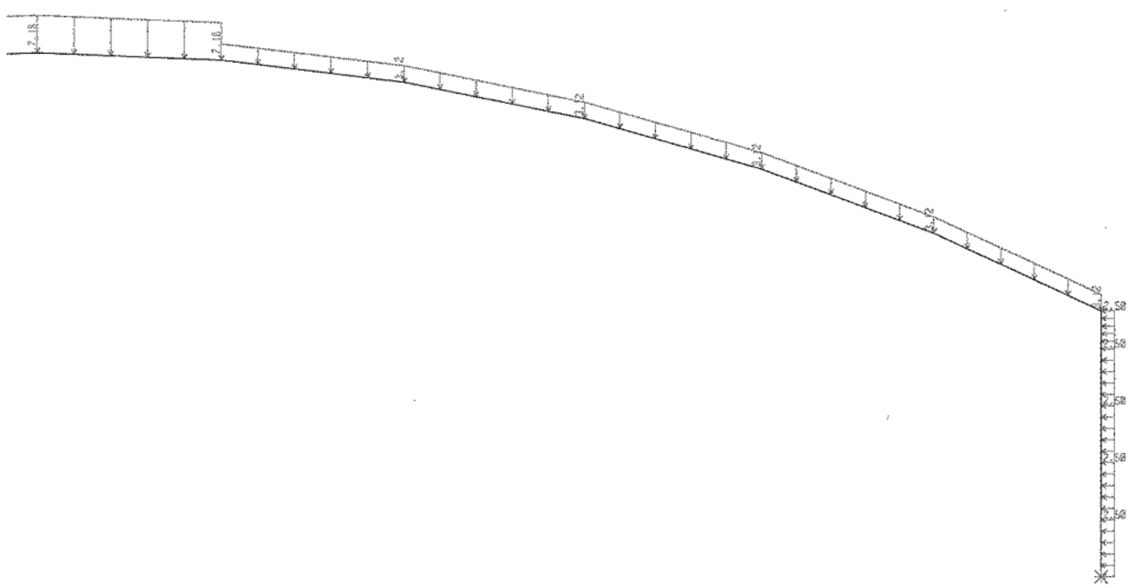

Figure 5-10: Equivalent Factored Design Loads on 36 ft SAP2000 Model

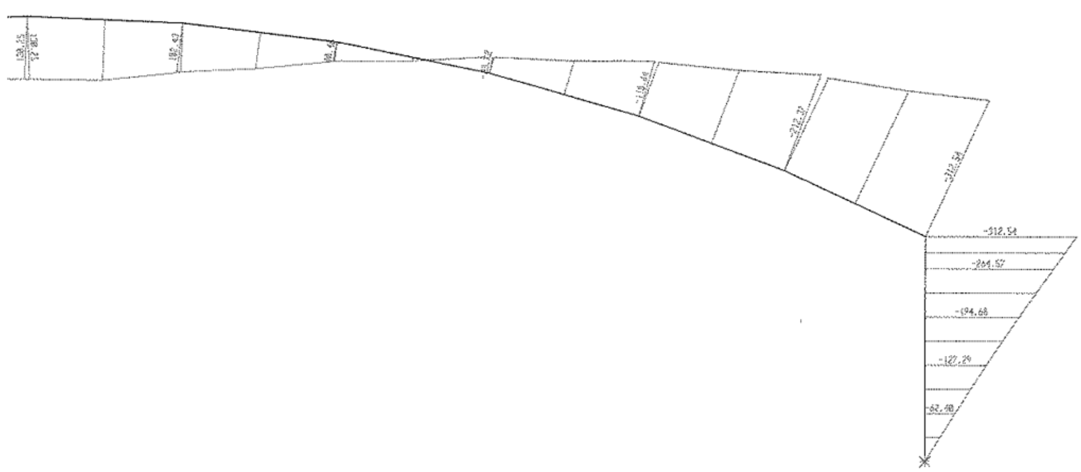

Figure 5-11: Moment Diagram of Factored Design Loads on 36 ft SAP2000 Model

The moment at midspan of the 36 ft SAP2000 model under the factored design loads was approximately 130 k-ft. The moment at the corners of the 36 ft SAP2000 model under the factored design loads was approximately 312 k-ft.

A comparison was made between the moment diagrams output by RISA and by SAP2000 with the same design loading configuration applied. The moment diagram of the RISA model of the 20 ft clear span with the factored design loads applied was shown in Figure 5-3. This can be compared to the moment diagram of the SAP2000 model of the 20 ft clear span with the factored design loads applied in Figure 5-9. The moment diagram of the RISA model of the 36 ft clear span with the factored design loads applied was shown in Figure 5-7. This can be compared to the moment diagram of the SAP2000 model of the 36 ft clear span with the factored design loads applied in Figure 5-11. A summary of the comparison between moment magnitudes of the RISA model to the moment magnitudes of the SAP2000 models is shown in Table 5-2.

Table 5-2: Comparison of RISA Model Moments to SAP2000 Model Moments

Location	**RISA Model – Equivalent 4 ft Wide Section**		**SAP2000 Model**	
	20 ft Clear Span Moment (k-ft)	**36 ft Clear Span Moment (k-ft)**	**20 ft Clear Span Moment (k-ft)**	**36 ft Clear Span Moment (k-ft)**
Midspan	260	196	90	130
Corner	68	130	222	312

The moments from the RISA model and the SAP2000 model do not correlate well. In the RISA model the midspan moment was the maximum. However, in the SAP2000 model the corner moments were the maximum. One reason this could be the case is that the corner section members in the SAP2000 model are slightly more massive than those in the RISA model, therefore increasing the stiffness and moment flow to those areas. Another possible source of difference is the incorporation of various springs by the designers in the RISA 3-D models. The SAP2000 model also made use of non prismatic elements at the corner sections while the RISA model used all prismatic elements assigned along a slight different centerline than the SAP2000 non prismatic frame elements. Both nonlinear and linear analyses were performed in SAP2000. Under the design loads being considered, the moment diagrams in both the linear and nonlinear case were the same magnitude. This indicates that nonlinear effects did not have a significant impact in the distribution of moments throughout the range of loading.

5.4 – Experimental Results versus Design

Midspan cross sectional analysis was the only cross sectional analysis provided in the design calculations and therefore the midspan will be used for comparison to the laboratory testing and SAP2000 model. This comparison provides a means of evaluating the overall effectiveness, see Section 5.5, of the design methodology currently used for the Foley Arch.

Table 5-3 shows an approximate average between the laboratory test results and the SAP2000 model results for the 20 ft and 36 ft clear span structures. The average ultimate moment shown in the last column of Table 5-3 can be seen in Figure 4-10 and Figure 4-14 for the 20 ft and 36 ft clear span structure, respectively. It shows the cross sectional flexural steel area in the lab specimens, the compressive strength of the concrete specimen samples, and the yield strength of the flexural steel used in the 20 ft clear span structure and the 36 ft clear span structures tested in the laboratory and modeled in SAP2000.

Table 5-3: Test Structure Midspan Properties

Structure Clear Span	Test/Model Structure			
	Flexural Steel, A_s (in^2)	**Compressive Strength, f'_c (psi)**	**Steel Yield Strength, f_y (ksi)**	**Ultimate Midspan Moment Capacity, M_u (kip-ft)**
20 ft	8.02	12,506	65	448
36 ft	5.34	7,044	65	550

Of particular interest from Table 5-3, is the compressive strength of the concrete in the 20 ft clear span test specimen. It was significantly higher that the design strength of 6000 psi as shown in Table 5-1. This may raise some concerns related to directly comparing the design calculations and associated moment capacity to the test specimens and corresponding SAP2000 model results.

A comparison between the structure as it was designed and analyzed in RISA to the structure tested in the laboratory and the SAP2000 model was of primary interest at this stage of the project. As was discussed earlier in this section, the midspan cross sectional properties and capacity was the critical location shown in detail in the design calculations provided. For comparison, a ratio of test properties to design properties was used as shown in Table 5-4. The test properties used in the ratio is an average between the laboratory test specimen and SAP2000 model. The design properties, as shown in

Table 5-1, are based on the details contained in the design calculations provided by Foley.

Table 5-4: Ratio of Design Properties to Test/Model Specimen Properties

Structure Clear Span	$\frac{A_{s,Test}}{A_{s,Design}}$	$\frac{f'_{c,Test}}{f'_{c,Design}}$	$\frac{f_{y,Test}}{f_{y,Design}}$	$\frac{M_{u,Test}}{M_{u,Design}}$
20 ft	0.835	2.08	1.08	1.46
36 ft	1.09	1.17	1.08	2.52

The 20ft clear span structure and the 36ft clear span structure exceeded the required design strength. The structures are safe as designed under the test conditions. It appears that the 20 ft clear span test specimen had slightly less steel than called for in the reinforcement details reviewed in the design calculations. At the time of testing the 20 ft clear span concrete had reached a much higher compressive strength than was designed for. The effect the small reduction in cross sectional steel had on the structure was explored using the 20 ft clear span SAP2000 structural model. It was found that the difference in ultimate capacity was small enough that it could be neglected for the purpose of evaluating the design methodology. The effect the increased compressive strength of concrete had on the structure was also explored using the SAP2000 model. Similarly, it was found that the difference in ultimate capacity was small enough that this difference was not a source of concern. Based on the ratio of an average between the laboratory test and the model results to the design calculations, the 20ft clear span specimen had a 30 percent higher moment capacity at midspan than the design calculations for the same structure indicated. Based on the ratio of an average between the laboratory test and the model results to the design calculations, the 36ft clear span specimen had an 84 percent higher moment capacity at midspan than the design calculations. Both designs, the 20 ft clear span and 36 ft clear span, were reasonably effective. The 36 ft clear span is slightly higher than an ideal overstrength, but not to an exorbitant or overly uneconomical point.

5.5 – Conclusions

It was concluded from evaluation of the 20 ft clear span structure and the 36 ft clear span structure that the designs were effective and the design methodology was reasonably well representative of actual behavior of the structure when tested. The designs are safe but not exceedingly overdesigned. It was also concluded that the SAP200 models and the RISA models did not have the same maximum moment locations under the factored design loads in the design calculations provided by Foley due to a combination of thicker corner sections in the SAP2000 models as well as springs incorporated into the RISA model that were not in the SAP2000 models. While there were slight differences in the RISA and SAP2000 models when analyzed under approximately equivalent loads, this did not impact the determination of the overall capacity of the structure. It was concluded that the way the design methodology is carried out is safe and effective.

Chapter 6: Summary, Conclusions, and Recommendations

6.1 – Summary

Foley Products contracted Auburn University to perform third party analytical and experimental validation of the performance, design, and analysis methods. The goal of the project was to validate all aspects of the Foley Arch bottomless bridge. The experimental validation of the three sided bottomless arch bridge included one serviceability field test during an existing project's construction and two laboratory tests which included loading the structure to its ultimate capacity. The two laboratory tests were conducted on a 20 ft clear span structure and a 36 ft clear span structure. Correlation to the results of the laboratory testing was achieved using a structural computer model developed within SAP2000. The design methodology used in the design packages provided by Foley Products was reviewed and evaluated to determine its effectiveness. This field test was utilized to evaluate precast arch bridge sections under a known truck load by installing instrumentation to measure reinforcing bar and concrete strains. Deflection and earth pressure transducers were installed on-site prior to backfilling the bridge. This test was during construction of a precast arch bridge project in Carrabus County, North Carolina.

In addition to a field test, two laboratory tests were performed to evaluate bridge behavior up to ultimate loading conditions. The first laboratory test section was a 20ft span bridge with an 8ft vertical rise. The second laboratory test section was a 36ft span bridge with a 9ft vertical rise.

Following the field and laboratory testing, a computer model of the Foley Arch structure was created in SAP2000 to correlate the results of the laboratory testing to the analysis and results of the computer model. This was used to determine if any economic or serviceability improvements could be reasonably recommended and/or implemented in the design or production of the structures.

6.2 – Conclusions

The conclusions reached based on the research performed were:

- The laboratory testing results were conceptually compatible with the anticipated behavior.
- The highest moments were developed at midspan and at the corner of the arch section along with the highest deflections being measured at midspan.
- The data collected and analyzed provided a solid comparison point to move forward with the project in developing a structural computer model in SAP2000.
- The SAP2000 structural models provided good correlation of the moments and deflections seen during the laboratory testing. This was especially true throughout the first three quarters of the loading range.
- Near the ultimate capacity of the arches the model under predicted, with the exception of the model over predicting the corner moment of the 20 ft clear span arch, the moments and deflection by 15 to 20 percent.

- The designs were effective and the design methodology was reasonably well representative of actual behavior of the structure when tested.
- The designs are safe but not exceedingly over designed.
- The SAP2000 models and the RISA models did not have the same maximum moment locations under the factored design loads in the design package provided by Foley.
- The difference between the SAP2000 models and RISA models did not require a change in the design methodology.

6.3 – Future Research

While the service load range test performed in the field and the ultimate loading behavior test performed in the laboratory both provided a solid basis for analysis of the behavior and design of the Foley Arch structure, there are some additional considerations that could result in an even better understanding of the structures behavior. A full scale test set up under fill material that is typically used in construction would allow a better understanding of how exactly the fully distributed lateral pressure effects the initial condition of the structure. If a full scale field test were performed all the way to ultimate loading conditions, it would be possible to trace this active pressure influence and any developing passive pressures as the structure deflects.

The structural computer models developed using SAP2000 well represented the behavior of the structure in the laboratory. However, if the above full scale field test were performed up to ultimate loading conditions, a structural model that incorporates detailed soil properties and soil structure interaction would be even more representative of true behavior of the structure in its application.

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