Final Report

Load Rating of Masonry Arch Bridges and Culverts

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Submitted to

Ohio Department of Transportation
Bureau of Research and Development
25 S. Front St.
Columbus, OH 43216

State Job No. 14569(0)
Contract No. 7763

January 31, 1996

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ABSTRACT

The State of Ohio has 139 inventoried masonry arch bridges and many more culverts, in road and highway service. Many of these structures have been identified as historically significant, making their replacement difficult, and their load-rating and rehabilitation imperative. In this study a procedure for load rating masonry arch bridges is developed. The procedure follows the practice of the AASHTO Manual for the Maintenance Inspection of Bridges, Load Factor Method. The geometry of the structure is determined by field measurement, and appropriate material properties are determined by inspection or testing. A unit width of the structure is analyzed using commercially available place frame analysis software. The moments and axial forces in the arch ring under combinations of load and rating factors are compared to estimates of the capacity of the masonry to determining inventory and operating ratings. Field tests of seven structures under various configurations of truck loading have been used to validate the analytical model. The proposed analysis and rating procedure is applied to the structures in this study.

Keywords: Load-rating, masonry, arch, deck arch, stone, bridge
1. STATEMENT OF THE PROBLEM

The State of Ohio has 139 inventoried masonry arch bridges, and many more culverts, in road and highway service. Many of these structures have been identified as historically significant, making their replacement difficult, and their load-rating and rehabilitation imperative. Section 106 of the National Historic Preservation Act of 1966 requires a review process whenever Federal funds or permits are used in a project impacting a National Register-eligible structure, and Section 4F of the U.S. Department of Transportation (U.S. DOT) Act of 1966 mandates a special effort to preserve National Register-eligible structures. In compliance with these acts, the Ohio Department of Transportation (ODOT) has determined that over 30 masonry arch bridges are National Register-eligible, and, as required by the State Historic Preservation Office, has set aside an additional "reserve pool" of structures against the contingency that an eligible structure loses its integrity.

The technology of a masonry arch structure is different from that of a contemporary structure, so that visual inspection, structural analysis, and load-rating techniques applicable to modern structures may not be applicable to these older structures. Moreover, most masonry structures are owned by counties and municipalities, who have limited resources for the development of analysis procedures. In order to accurately determine load ratings and repair strategies, it is necessary to develop a quick, reasonably accurate analysis procedure, validated by experimental evidence, which can incorporate information from visual inspection and physical testing.

1.1 RESEARCH OBJECTIVES

The principal tasks in this study were the surveying of a sample of the masonry arch structures in the State of Ohio, the development of analytical methods for rating arch structures, the experimental validation of the analytical models, and the writing of the report.
1.1.1 Task 1-Literature Search and Review

Literature on analysis and assessment of masonry arch bridges was reviewed, particularly reports on the ultimate strength tests of masonry arch bridges performed in the UK. The UK studies were obtained and reviewed carefully for applicability to the U.S. conditions of climate, inspection frequency, vehicle weights, and U.S. codes and standards. The results of this review facilitated the development of a direction for this research.

1.1.2 Task 2-Selection of Bridges for Study

The bridges available for study within the State of Ohio were surveyed, first through written inventory and inspection reports and finally through field surveys. The owners were contacted to identify which bridges would be available for this study. A minimum of four bridges representative of age, span, and condition were chosen, at least one of which was fundamentally undamaged.

1.1.3 Task 3-Measurement and Crack Inventory

The bridges selected for study were surveyed in detail and carefully measured. Cracks and other visible damage were thoroughly surveyed and documented in as unbiased a manner as possible.

1.1.4 Task 4-Structural Modeling

The bridges selected for testing were modeled in two ways. A finite element computer model was developed using a commercially available finite element computer program. An elastic analysis was conducted at lower load ranges, and plastic effects due to sliding of blocks and plastic deformations of the bridge fill were considered for overloads. In the initial phases, plane stresses were considered for a unit width of the roadway, but transverse and three dimensional effects were considered as found to be necessary. The displacement results from this analysis were compared with displacements observed during the testing program for validation of the finite element model.

The structure was also considered from the point of view of limit states analysis. Upper bounds on the load causing collapse, or the repeated loads causing incremental collapse were found for various choices of a collapse mechanism. Plausible collapse or incremental collapse modes can be inferred by comparing the magnitude of the upper bound load associated with each collapse mode.
The plausible collapse modes can be compared with the evidence of damage found at the bridge, through superficial logging of cracks and through recording of irreversible deformations under loads during the testing program. These results were also compared with the results obtained from the finite element analysis. The results of both analyses were compared with available test data from field tests and scale model studies from the UK and elsewhere (Hendry et al. 1985, 1986).

For both methods, selected structural models were analyzed to determine the sensitivity of the calculated collapse loads to changes or uncertainties in the parameters, such as fill density, coefficient of friction, and elastic modulus.

1.1.5 Task 5-Structural Testing

In an exploratory testing program, two of the bridges selected for study were instrumented and tested under known legal loads. Each bridge was instrumented and sand trucks with known axle loads were run across the bridge. The spread of the arch ring at the abutments, distortions of the arch ring, and relative displacements across critical crack locations were measured. The data were analyzed for evidence of irreversible displacements and collapse modes consistent with any such displacements. The data were then reduced for comparison with the results from the analytical program. Later, the full sample set of four or more bridges were tested under static truck loading. The bridges were instrumented at critical locations determined from the analytical modeling and the exploratory testing program.

1.1.6 Task 6-Final Report

The results of analytical modeling were compared to experimental observations from this program, as well as tests performed in the UK and elsewhere in Europe, and validated analytical models were proposed. Conclusions were inferred regarding the susceptibility of different bridge configurations to different collapse modes, and the identification of these collapse modes on the basis of visual observations. This report includes a draft of a procedure for load rating masonry arch structures, taking into account evidence from visual inspections and physical evidence from testing of in situ material. The procedure is in the Load Factor format of the AASHTO Manual for the
Maintenance Inspection of Bridges (AASHTO, 1983). In addition to the load-rating guide, this report presents sample calculations applying limit states analysis to the load rating of prototype structures.

1.2 PROPOSED LOAD-RATING PROCEDURE

The main product of this research project is a load-rating procedure for masonry arch bridges. The proposed procedure is described in detail in chapter 6. A detailed example of the application of the procedure is provided in section 6.6, and a summary is given in section 6.7.
2. LITERATURE REVIEW

There are a variety of approaches to explain the behavior of masonry arches, including plastic (or mechanism) analyses using the line of thrust concept, finite element models, and small and full-scale tests on masonry arches. Modifications and enhancements continue to be made on these basic concepts, to better predict the failure criteria of a masonry arch bridge. Page (1993) presents a thorough review of the state-of-the-art of masonry arch bridge assessment.

2.1 Line of Thrust

The line of thrust can be constructed by finding the axial thrust $P$ and bending moment $M$ at each joint of an arch. The axial thrust and moment can also be represented by a statically equivalent eccentric thrust, with eccentricity $e = M/P$. The thrust line passes through the point representing the eccentricity of the thrust at each joint. The line of thrust was first described by Robert Hooke in 1676 who stated “as hangs the flexible line, so but inverted will stand the rigid arch.” Harvey (1988) restates this as “The line of thrust may be described as the line at which a flexible member of infinite strength would be in equilibrium, albeit unstable, with the load system imposed.”

Paraphrasing Hooke, a cable under a given set of loads will take a characteristic shape. An example of this would be the “V” shape a line will take when a weight is applied at its midpoint. If such a line were somehow made rigid in this shape and turned upside-down, it would define the line of thrust for that specific load configuration. For an arch to remain in equilibrium under the same load (meaning an arch with a point load at its crown), it would have to contain the line of thrust within the arch ring.

2.2 Mechanism Behavior of a Masonry Arch

As long as the line of thrust does not touch the intrados and extrados of the arch ring four times, the arch is stable for that set of loads. This is the lower bound theorem of plasticity applied to the masonry arch. If the line of thrust does touch the edge of the ring four (or more) times, hinges form at those locations and the arch will fail as a mechanism. An upper bound on the
collapse load can also be found by finding the load in equilibrium in any four hinge collapse mechanisms (Heyman, 1982). The lower and upper bound theorems for arch analysis are illustrated in Figure 2.1.

![Figure 2.1: Lower and upper bounds for collapse load for the masonry arch.](image)

In the application of a plastic analysis, certain assumptions are made regarding the properties of a masonry arch. First, the voussoirs cannot slide with respect to each other. Second, the masonry has no tensile strength. Third, the masonry has an infinite compressive strength. The first and third of these assumptions are not necessarily conservative, but Heyman (1982) claims they are so close to the reality of masonry arch behavior at low stresses that they are acceptable statements. The second assumption is safe, since the very low tensile capacity of the mortar (if it exists at all) cannot be relied upon (Heyman, 1982). From the above statements, it can be seen that, for a given set of loads, there is a minimum arch ring thickness that can envelop the entire thrust line. The ratio of actual arch thickness to the minimum arch thickness is
Heyman’s "geometrical factor of safety." Thus, a factor of three means the arch is three times thicker than necessary to avoid a mechanism failure.

2.3 Three-Hinge Analysis

Smith et al. (1990) proposed that when the centering is removed from a masonry bridge, the arch will naturally take on a determinate three-hinge condition. This can be caused by shortening in the arch ring, spread of the abutments, or both. They maintain that hinges often form at the extrados at the crown and at the intrados at or near the springings, defining the position of the line of thrust. Their analysis begins by assuming three likely positions for hinge formation. The thrust line is found, and if it strays outside the arch ring, a new set of hinge positions must be tried. The applied load is increased and thrust lines are drawn until they touch the edge of the ring at the required four points. The load associated with such a thrust line is the collapse load. However, they also make an attempt to better represent the state of the stone at the hinges. Heyman's assumption that the compressive strength of the blocks is infinite is obviously not accurate. This creates an infinitely small area of contact between the blocks at the hinge. Since this means the stresses at the hinge are infinitely high, Smith et al. propose that the hinge forms over a contact surface due to the localized crushing of the block corners. This surface extends a distance defined very simply as the local thrust divided by the crushing stress of the blocks. A second line of thrust can be drawn passing through mid-depth of the contact surface. The two thrust lines now define a depth of material required to transmit the compressive stresses through the arch. Both lines must be within the dimensions of the arch, and the collapse load is therefore reduced.

2.4 Brittle Hinge Model

Discarding Heyman's assumption that the masonry is of infinite strength, a more rational method of modeling the hinges was sought. The simple linear model first proposed by Smith et al. was improved by Taylor and Mallinder (1993), who performed a series of tests to verify their brittle hinge model. Figure 2.2 shows the improved hinge conceptually. Pairs of calcium silicate bricks were loaded with compressive thrusts near the edge of their joint interface. The stress-
strain curves were plotted as the hinge formed and then crushed under the increased loading. The specimens exhibited a definite plastic response prior to failure, with a distinct falling branch at the end of the curve. Taylor and Mallinder incorporated this brittle hinge response in their mechanism analysis of the masonry arch. The masonry was given a finite compressive strength, which was used to determine the stress-strain behavior of the hinge. This modification eliminated the need for Heyman's non-conservative infinite compressive strength assumption.

![Figure 2.2: Brittle hinge model.](image)

2.5 Effect of Fill on Line of Thrust

Both Smith's et al. and Harvey's mechanism analyses include the horizontal component of the fill around the arch in determining the line of thrust. When the horizontal forces of the fill on the arch ring are accounted for, the shape of the line of thrust in the arch can be altered dramatically. Harvey and Smith (1987) showed that this effect is especially important in arches with low span/depth ratios, such as semicircular arches. Such arches were long considered to be less than ideal, since it was difficult to turn the thrust line vertically downward to stay within the
arch ring. When only the vertical component from the fill is considered, the thrust line takes on a parabolic shape, which is not easily contained within a semicircular ring. When a reasonable angle of internal friction was assumed for the soil, however, the resulting horizontal forces on the arch caused the line to take a more circular path. Thus, arches with steep haunches, such as gothic arches or Roman semicircular arches are shown to be as structurally sound as shallower shapes. The fact that many such steep arches have stood for centuries bears this out.

2.6 Elastic Analysis

While the plastic methods begun by Heyman have been the main focus of recent arch studies, Bridle and Hughes (1990) have used Castigliano’s elastic method to achieve the same goal. Castigliano used the strain energy in the arch ring to determine moments and forces in the arch. He was concerned with the state of stress within the stones, something Heyman’s plastic analysis could not address. Bridle and Hughes took Castigliano’s techniques and used a computer program to determine the stresses at sections along the arch ring under a given load configuration. Wherever tension existed, that section of the arch was neglected and a new arch profile was obtained. That process repeated until convergence, when the effective arch ring entirely in compression was obtained.

After convergence, another load increment was imposed on the arch, and the process was repeated. The load was gradually increased until convergence could not be obtained. Failure to converge was shown to coincide with the type of mechanism analyses described earlier. This energy method lends itself to improvements such as realistic soil dead loads, and it does not require unrealistic assumptions such as infinite compressive strength.

2.7 Finite Element Models

The iterative “thinning” of the arch ring used by Bridle and Hughes (1990) is an important concept in the finite element approaches to the masonry arch problem. Choo et al. (1991) used tapered beam elements, along with the thinning effective arch ring iterative process to converge to a solution. In addition to the assumption that tensile regions do not contribute to the stiffness, this finite element approach also locates regions of high compressive stress, and neglects them as
well. Thus, the effective arch ring is now defined as the region which is not in tension and has not yielded in compression. Horizontal fill elements represent the passive resistance of the soil around the masonry arch. This model accurately models the response of full-scale tests of masonry arch bridges.

Loo and Yang's (1991) finite element procedure incorporated several additional concepts that are unique. The material cracking in the arch ring was examined in more detail than in the Choo et al. model. A von Mises failure envelope is developed for two-dimensional stresses. Stress-strain curves for a variety of failure conditions are used to more accurately represent the state of stress in the arch ring during loading. Rather than distinguishing between the masonry and the mortar, however, average properties of the entire masonry/mortar assembly are used. The horizontal and vertical forces on the arch ring from the fill are found using a second finite element model. This other model replaces the arch/fill interface with a series of hinge supports. The horizontal and vertical reactions found at these supports from the weight of the fill elements can then be applied to the standard finite element model of the arch ring.

2.8 Field Tests on Masonry Arch Bridges

Numerous tests have been performed on masonry arch bridges in recent years, both on actual out-of-service bridges and laboratory models. The results of these tests have been used to verify the analysis techniques described previously. These tests have helped researchers understand the behavior of masonry arch bridges up to failure. They have also shown characteristics in masonry arch bridges that are still difficult to model with any accuracy.

2.9 Snap-Through Failure

The phenomena of "snap-through" failures in masonry arch bridges have been observed in some tests. This occurs when an arch forms three hinges, then fails suddenly prior to formation of a fourth hinge. Often material crushing failure causes this to happen. Some model tests by Royles and Hendry (1991) failed by snap-through buckling, with limited material crushing. A full-scale test to failure on a bridge in the United Kingdom also failed this way, at a load approximately two-thirds of that predicted by Harvey's mechanism analysis (Harvey, 1988). Such
failures are not predictable by current analysis, and typically occur at loads less than the failure loads calculated by mechanism or FEM methods.

2.10 Spandrel Wall Contribution

The contribution of the spandrel walls to arch stability has not yet been mentioned but has been seen in tests to greatly increase the failure load of certain masonry arch bridges. In model tests conducted on similar bridges (span/depth=2.0) with and without spandrel walls, an increase of 72 percent in the failure load was recorded because of the presence of spandrels (Melbourne and Walker, 1988). In much the same way as the fill, spandrel walls prevent mechanisms from forming in the arch ring. In addition, if the spandrel wall is well-bonded to the arch stones, it can prevent the opening of hinges, further strengthening the arch ring.

2.11 Relative Effect of Span/Rise Ratio

As stated earlier by Harvey and Smith (1987) in relation to fill contribution, the effect of the spandrel walls on structural stability is markedly greater for steep arches such as semicircles. For a very general case, Royles and Hendry (1991) maintain that for a span/rise ratio of 2.0, the relative strength of an arch vault could increase five times by the addition of fill, and nine times by adding fill and spandrel walls. It is interesting to note that the increase in relative strength from the presence of fill only to the presence of both fill and spandrels is about 80%, close to the 72% found by Melbourne and Walker (1988). For a span-to-depth ratio of 6.4, however, the respective increases are only 20 percent and 50 percent. More precise methods of incorporating this additional strength into an analysis procedure are underway.

2.12 Multi-Span Arches

The behavior of multi-span masonry arches requires a new series of tests and analysis procedures. Gilbert and Melbourne (1994) began this work on three-span models. Global mechanism failures occurred on all three tests, with seven hinges forming in the arch rings and in the piers. Two additional findings were made in these tests: The collapse loads were lower in the three-span bridges than for similarly proportioned single-span bridges, and the collapse load was
nearly 30 percent lower for a load at the crown than for a load at the traditional quarter-span weak point.

2.13 Codified Bridge Analysis Procedures

Two references on masonry arch bridge analysis prepared by the United Kingdom Department of Transport (1993a, 1993b) include a Design Manual and an Advice Note. These include a general standard for the assessment of masonry arch bridges and a detailed description of two alternative methods for the analysis and assessment of structures. The recently adopted Ontario Highway Bridge Design Code (1991) also contains provisions for the load rating of masonry arch bridges.

2.13.1 MEXE Method

The first method presented in the Advice Note is known as the modified Military Engineering Experimental Establishment (MEXE) method, developed in the 1940s for quick assessment of bridges during World War II. This method relies on a nomograph, shown in Figure 2.3, to establish axle loading, modified by a series of empirical factors that reduce the loads based on observed defects in the structure. Although the method is suitable for quick assessments of a masonry arch structure, it has been found to be imprecise and unconservative in certain instances. Due to its extreme simplicity, it is still the preferred method among local bridge owners in the UK.

2.13.2 Alternative Method

The Design Manual and Advice Note also contain language and conditions permitting any rational method that properly assesses the bridge for the collapse limit state, under the condition that a very conservative overload factor of 3.4 be used. In general, this language refers to methods employing collapse load analysis, as outlined in Chapter 2 of this report. The implementation of these provisions could be either through mechanism analysis, and the search for a least upper bound collapse load, or the use of equilibrium methods, combined with an understanding of the limits of the material strength of the structure. This latter general method is described in a detailed procedure in the Advice Note. It is suggested that the centerline of the arch ring be divided into at least 10 straight line elastic elements, and subjected to truck wheel
loads. The wheel loads are to be applied to a 0.30m square area at deck level and distributed through the fill at a slope of 2 vertical to 1 horizontal, down to the extrados of the arch. The arch is to be modeled as an elastic frame, pinned at the base, and the stresses resulting from dead and live loads are to be examined from the point of view of elastic analysis. This method is illustrated in Figure 2.3, which shows the general outlines of the frame to be used in the analysis method, and Figure 2.4, which describes the distribution of the wheel loads through the fill. This is the general outline of the method that is proposed to be adopted in this report. However, exception is taken to some of the details of the foregoing method, and modifications are proposed and incorporated into the recommendations of Chapters 5 and 6.

Figure 2.3: Alternative method: division of arch ring into straight elements.
2.13.3 Canadian Bridge Code

The Ontario Ministry of Transport (1991) promulgates a comprehensive Ontario Highway Bridge Design Code, which incorporates provisions for the assessment of masonry arch bridges. This code includes analysis for a collapse limit state, using a live load factor of no more than 1.55 and an undercapacity factor of no less than 0.70. Analysis for a serviceability limit state is given, relying on a procedure much like the foregoing alternative method, using overload and undercapacity factors of unity, and using tables of allowable stresses in the mortar. These allowable stresses are in the range of 50-200 psi—reasonable values for allowable mortar stresses, but quite conservative for allowable stresses in unit masonry structures considering the combined strength of the mortar and the units. A statement in the preamble to the Bridge Code strongly discourages the use of the serviceability limit state for the posting of structures, and recommends that this limit state only be used for the determination of load carrying capacity of abutments, piers, and retaining walls.

The ultimate limit state analysis is to be based on the upper bound procedure, and requires the ability to find the least upper bound on the collapse load. Although this can be implemented,
it requires an understanding of limit states principles not generally taught to engineers, and relies heavily on the ability to find a least upper bound.
3. TESTING APPARATUS AND PROCEDURE

3.1 Introduction

This chapter describes the field tests performed on seven masonry arch bridges for this study. The testing equipment and setup are explained in detail, as is the procedure followed during each test. This procedure included lab preparation, field setup, the testing itself, and post-test data reduction. The resulting graphs of truck location vs. displacement are included in Appendices A through G.

3.2 Bridges Under Study

The field tests were conducted on seven masonry arch bridges in all. Figure 3.1 illustrates bridge arch geometry as it relates to the data in table 3.1. Table 3.1 lists the dimensions of each structure.

![Diagram of arch geometry]

Figure 3.1. Arch geometry.
Table 3.1 Dimensions and relevant data for bridges in test program (see Figure 3.1).

<table>
<thead>
<tr>
<th></th>
<th>Cemetery</th>
<th>Jones</th>
<th>Oberlin</th>
<th>Vernon</th>
<th>Vincent</th>
<th>Xenia</th>
<th>Elyria</th>
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<td>4734173</td>
<td>4703278</td>
<td>4200632</td>
<td>4234073</td>
<td>2900335</td>
<td>4770242</td>
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<td>Span (in)</td>
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<td>240</td>
<td>456</td>
<td>180/181</td>
<td>382</td>
<td>800</td>
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<tr>
<td>Height (in)</td>
<td>120</td>
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<td>102</td>
<td>108</td>
<td>54/53</td>
<td>56</td>
<td>212</td>
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<td>2.63</td>
<td>2.35</td>
<td>4.22</td>
<td>3.33/3.41</td>
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<td>12</td>
<td>24</td>
<td>30°</td>
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<td>Roadway Width</td>
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<td>16'-0&quot;</td>
<td>22'-10&quot;</td>
<td>50'</td>
<td>16'-4&quot;</td>
<td>80'-0&quot;</td>
<td>22'-0&quot;</td>
</tr>
<tr>
<td>Arch Barrel Width</td>
<td>25'-0&quot;</td>
<td>22'-1&quot;</td>
<td>28'-10&quot;</td>
<td>64'</td>
<td>18'-4&quot;</td>
<td>99'-2&quot;</td>
<td>28'-0&quot;</td>
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<tr>
<td>Fill Over Crown (in)</td>
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<td>12</td>
<td>0/0</td>
<td>0</td>
<td>48</td>
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<tr>
<td>Angle of Embrace</td>
<td>162°</td>
<td>149°</td>
<td>161°</td>
<td>102°</td>
<td>124°/122°</td>
<td>89°</td>
<td>141°</td>
</tr>
<tr>
<td>No. of Voussoirs</td>
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<td>27</td>
<td>43</td>
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<td>Limestone</td>
<td>Sandstone</td>
<td>Sandstone</td>
<td>Limestone</td>
<td>Limestone</td>
</tr>
<tr>
<td>Date of Test(mm-dd-yy)</td>
<td>7-19-94</td>
<td>7-20-94</td>
<td>7-21-94</td>
<td>10-6-94</td>
<td>5-16-95</td>
<td>10-17-95</td>
<td>10-19-95</td>
</tr>
</tbody>
</table>

Three similar bridges in Lorain County were tested first. Two of the structures are located on Cemetery Road and Jones Road, respectively, south of Oberlin, and the third is in Oberlin itself. These bridges will henceforth be referred to as the Cemetery, Jones, and Oberlin bridges. All three are segmental, single-span bridges made up of cut-stone voussoirs. The mortar joints are too thin to measure accurately. The particulars regarding dimensions can be found in Table 3.1.

The fourth bridge tested is in Mt. Vernon, Ohio. It was built in 1892. One end span of the five-span structure was instrumented for the test. It is segmental in shape, with 31 regular cut stones making up the arch ring. The span is 39 ft and the rise is 9 ft. The arch ring is 2 ft thick. The fill is approximately 12 inches deep above the keystone.

A short, two-span bridge located on Vincent Road in rural Knox County, near the town of Howard, was tested. A particularly wide structure in Xenia was tested under ambient loading to aid in the determination of transverse distribution of wheel loads. A large, two-span structure located on Mussey Ave. in Elyria was the final bridge tested in this program. The seven bridges are illustrated at the end of this chapter.
3.3 Test Preparation and Procedure

A reference frame was built underneath each bridge from 14-gauge slotted steel angle sections. Each frame was aligned parallel to the roadway overhead. Linear variable differential transformers (LVDT’s) were attached to the frame, and were aligned so radial displacements would be measured. Exceptions were the two instruments at the abutments, which were aligned to measure horizontal displacements. Additional instruments were also placed at locations away from the reference frame to monitor spandrel movements, transverse arch ring movements, and crack movements. Table 3.2 lists the locations of the arch ring LVDT’s and Table 3.3 lists the locations of these additional LVDT’s.

Table 3.2  Location of arch ring transducers, θ (see Figure 3.2).

<table>
<thead>
<tr>
<th>LVDT</th>
<th>Cemetery</th>
<th>Jones</th>
<th>Oberlin</th>
<th>Vernon</th>
<th>Vincent’</th>
<th>Xenia’</th>
<th>Elyria</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>horizontal</td>
<td>horizontal</td>
<td>horizontal</td>
<td>horizontal</td>
<td>60°</td>
<td>90°</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>154°</td>
<td>142°</td>
<td>18°</td>
<td>horizontal</td>
<td>90°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>137°</td>
<td>129°</td>
<td>32°</td>
<td>54°</td>
<td>90°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>121°</td>
<td>116°</td>
<td>45°</td>
<td>64°</td>
<td>120°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>101°</td>
<td>103°</td>
<td>76°</td>
<td>74°</td>
<td>horizontal</td>
<td>horizontal</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>90°</td>
<td>90°</td>
<td>90°</td>
<td>90°</td>
<td>horizontal</td>
<td>160°</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>73°</td>
<td>77°</td>
<td>105°</td>
<td>103°</td>
<td>55°</td>
<td>125°</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>62°</td>
<td>64°</td>
<td>134°</td>
<td>113°</td>
<td>90°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>50°</td>
<td>51°</td>
<td>143°</td>
<td>126°</td>
<td>120°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>26°</td>
<td>44°</td>
<td>156°</td>
<td>horizontal</td>
<td>horizontal</td>
<td>90°</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>horizontal</td>
<td>horizontal</td>
<td>horizontal</td>
<td>55°</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Distance, d</td>
<td>5 ft</td>
<td>5 ft</td>
<td>15 ft</td>
<td>15 ft</td>
<td>5 ft 4 in</td>
<td>13 ft 0 in</td>
<td></td>
</tr>
</tbody>
</table>

1. LVDT’s 1-5 in span 2, LVDT’s 6-10 in span 1
2. See Figure 3.3 for locations of LVDT’s at Xenia
3. d=3 ft 8 in
Figure 3.2: Transducer geometry.
(a) Cemetery Road
(b) Jones Road
(c) Oberlin
(d) Mt. Vernon
(e) Vincent
(f) Elyria
Table 3.3 Locations of additional LVDT’s.

<table>
<thead>
<tr>
<th>LVDT</th>
<th>Mt. Vernon Bridge</th>
<th>Vincent Road Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>below near springing 15 ft from edge</td>
<td>in line with LVDT 8, 2'-6&quot; towards spandrel</td>
</tr>
<tr>
<td>11</td>
<td>spanning crack in top of near spandrel wall</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>below far springing 15 ft from edge</td>
<td>in line with LVDT 7, 2'-6&quot; towards spandrel</td>
</tr>
<tr>
<td>13</td>
<td>at crown, 25 ft from edge</td>
<td>in line with LVDT 4, 2'-6&quot; towards spandrel</td>
</tr>
<tr>
<td>14</td>
<td>at crown, 5 ft from edge</td>
<td>in line with LVDT 3, 2'-6&quot; towards spandrel</td>
</tr>
<tr>
<td>15</td>
<td>below near springing 5'-6&quot; from edge</td>
<td>At spandrel wall above center pier at level of road</td>
</tr>
</tbody>
</table>

A data acquisition system was set up in the testing vehicle and connected to the signal conditioners and LVDT’s. The truck location potentiometer was set up on a sawhorse. The bridge was measured and sketched with all pertinent dimensions. The roadway was marked in 10-ft increments, starting at the crown. With the instruments in place and responding properly, the loaded trucks were then brought to the bridge. Table 3.4 gives the truck axle weights and dimensions for all five bridge sites.

Table 3.4 Loaded truck axle weights and dimensions.

<table>
<thead>
<tr>
<th></th>
<th>Half Load</th>
<th>Full Load</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Jones, Cemetery, Oberlin:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Front Axle</td>
<td>9,160 lbs</td>
<td>14,280 lbs</td>
</tr>
<tr>
<td>Rear Axle(s)</td>
<td>21,100 lbs</td>
<td>34,640 lbs*</td>
</tr>
<tr>
<td>Axle Spacing</td>
<td>12 ft.</td>
<td>15 ft.</td>
</tr>
<tr>
<td>Truck #1</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Vernon:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Front Axle</td>
<td>15,800 lbs</td>
<td>16,380 lbs</td>
</tr>
<tr>
<td>Rear Axle(s)</td>
<td>44,220 lbs*</td>
<td>44,640 lbs*</td>
</tr>
<tr>
<td>Axle Spacing</td>
<td>14.8 ft.</td>
<td>14.8 ft.</td>
</tr>
<tr>
<td><strong>Vincent</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Front Axle</td>
<td>13,480 lbs</td>
<td>15,780 lbs</td>
</tr>
<tr>
<td>Rear Axle(s)</td>
<td>28,560 lbs*</td>
<td>38,140 lbs*</td>
</tr>
<tr>
<td>Axle Spacing</td>
<td>14.8 ft.</td>
<td>14.8 ft.</td>
</tr>
<tr>
<td><strong>Elyria</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Front Axle</td>
<td>14,400 lbs</td>
<td>14,200 lbs</td>
</tr>
<tr>
<td>Rear Axle(s)</td>
<td>31,200 lbs*</td>
<td>35,100 lbs*</td>
</tr>
<tr>
<td>Axle Spacing</td>
<td>15.25 ft.</td>
<td>15.25 ft.</td>
</tr>
</tbody>
</table>

* = rear axles are tandem axles spaced 4.5 feet apart. Axle spacing is measured from steering axle to front tandem
3.4 Testing Procedure

As the truck began to move, the truck’s potentiometer voltage triggered the data acquisition system, which started collecting voltage data from the LVDT’s on the bridge. For all runs, data were collected at a rate of 100 scans per second. A total of 4,000 scans were made per channel. Thus, each run was made within a 40-second window of data acquisition. Each test was completed at least three times and assigned a code. The tests completed at each structure are summarized in Table 3.5.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Truck</th>
<th>Span</th>
<th>Location</th>
<th>No. of Tests</th>
<th>Test Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cemetery</td>
<td>Half</td>
<td>Half</td>
<td>Left</td>
<td>2*</td>
<td>HLH01-03</td>
</tr>
<tr>
<td></td>
<td>Half</td>
<td>Whole</td>
<td>Left</td>
<td>3</td>
<td>HLW01-03</td>
</tr>
<tr>
<td></td>
<td>Half</td>
<td>Half</td>
<td>Right</td>
<td>3</td>
<td>HRH01-03</td>
</tr>
<tr>
<td></td>
<td>Half</td>
<td>Whole</td>
<td>Right</td>
<td>3</td>
<td>HRW01-03</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Half</td>
<td>Left</td>
<td>3</td>
<td>FLH01-03</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Whole</td>
<td>Left</td>
<td>3</td>
<td>FLW01-03</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Half</td>
<td>Right</td>
<td>3</td>
<td>FRH01-03</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Whole</td>
<td>Right</td>
<td>3</td>
<td>FRW01-03</td>
</tr>
<tr>
<td>Jones</td>
<td>Half</td>
<td>Half</td>
<td>Center</td>
<td>3</td>
<td>HLHS1-3</td>
</tr>
<tr>
<td></td>
<td>Half</td>
<td>Whole</td>
<td>Center</td>
<td>3</td>
<td>HLWS1-3</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Half</td>
<td>Center</td>
<td>3</td>
<td>FLHS1-3</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Whole</td>
<td>Center</td>
<td>3</td>
<td>FLWS1-3</td>
</tr>
<tr>
<td>Oberlin</td>
<td>Half</td>
<td>Half</td>
<td>Center</td>
<td>4</td>
<td>FLHSC1-4</td>
</tr>
<tr>
<td></td>
<td>Half</td>
<td>Whole</td>
<td>Center</td>
<td>4</td>
<td>HLWSC1-4</td>
</tr>
<tr>
<td></td>
<td>Half</td>
<td>Half</td>
<td>Left</td>
<td>1*</td>
<td>HLHSL1-3</td>
</tr>
<tr>
<td></td>
<td>Half</td>
<td>Whole</td>
<td>Left</td>
<td>2*</td>
<td>HLWSL1-3</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Half</td>
<td>Center</td>
<td>3</td>
<td>FLHSC1-3</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Whole</td>
<td>Center</td>
<td>3</td>
<td>FLWSC1-3</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Half</td>
<td>Left</td>
<td>3</td>
<td>FLHSL1-3</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Whole</td>
<td>Left</td>
<td>3</td>
<td>FLWSL1-3</td>
</tr>
<tr>
<td>Mt. Vernon</td>
<td>1</td>
<td>Whole</td>
<td>Edge</td>
<td>3</td>
<td>EDGE1-3</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>Whole</td>
<td>Center</td>
<td>3</td>
<td>CENTER1-3</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>Whole</td>
<td>Middle</td>
<td>3</td>
<td>MIDDLE1-3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Whole</td>
<td>n/a</td>
<td>3</td>
<td>TWO1-3</td>
</tr>
<tr>
<td>Vincent</td>
<td>Half</td>
<td>Single</td>
<td>Center</td>
<td>3</td>
<td>HLSS01-03</td>
</tr>
<tr>
<td></td>
<td>Half</td>
<td>Double</td>
<td>Center</td>
<td>3</td>
<td>HLDS01-03</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Single</td>
<td>Center</td>
<td>3</td>
<td>FLSS01-03</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>Double</td>
<td>Center</td>
<td>4</td>
<td>FLDS01-03</td>
</tr>
<tr>
<td>Elyria</td>
<td>1</td>
<td>Single</td>
<td>Right</td>
<td>4</td>
<td>RT01-04</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>Double</td>
<td>Right</td>
<td>3</td>
<td>RU01-04</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>Single</td>
<td>Left</td>
<td>2</td>
<td>LT01-02</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>Double</td>
<td>Left</td>
<td>3</td>
<td>LU01-03</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Double</td>
<td>Both</td>
<td>3</td>
<td>BU01-03</td>
</tr>
</tbody>
</table>

* = bad data reduced the number of usable test runs
3.5 Data Reduction

The data were read from a spreadsheet file created in the field, and the voltage data from each channel were multiplied by the appropriate voltage-to-displacement calibration factor. Using a Parks-McClellan low-pass digital filter, the displacement data series were then filtered to reduce the effect of electrical "noise." A 0.1 Hz cut-off frequency and 5 Hz roll-off were specified. The filter effectively removed the ambient high-frequency electrical noise from the data, while retaining the character of the data itself. A data decimation procedure kept every twentieth data point, and discarded the remaining points. The initial offset was reduced to zero by subtracting the average of the first eight scans from all of the subsequent data.

![Diagram of transducer locations: Xenia bridge.]

Figure 3.3. Transducer locations: Xenia bridge.

3.6 Results

3.6.1 Observations Regarding Test Data

The displacements measured in the field tests provided a wealth of information on the behavior of the bridges under service loads. Tables 3.6 through 3.9 summarize the maximum and final displacements measured for each test. The values for abutment and crown displacements are
shown for all bridges, as well as additional values for crack widths and spandrel wall movements at the Mt. Vernon bridge.

Table 3.6 Maximum and final displacements for Lorain County, Ohio bridges.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Maximum Displ. (in (10^{-4}))</th>
<th>Final Displ. (in (10^{-4}))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Near Abut</td>
<td>Crown</td>
</tr>
<tr>
<td>Cemetery</td>
<td>(11)</td>
<td>(6)</td>
</tr>
<tr>
<td>HLH</td>
<td>-19</td>
<td>156</td>
</tr>
<tr>
<td>HLF</td>
<td>-26</td>
<td>172</td>
</tr>
<tr>
<td>HRH</td>
<td>-11</td>
<td>57</td>
</tr>
<tr>
<td>HRF</td>
<td>-14</td>
<td>58</td>
</tr>
<tr>
<td>FLH</td>
<td>-23</td>
<td>248</td>
</tr>
<tr>
<td>FLF</td>
<td>-30</td>
<td>245</td>
</tr>
<tr>
<td>FRH</td>
<td>-20</td>
<td>100</td>
</tr>
<tr>
<td>FRF</td>
<td>-18</td>
<td>88</td>
</tr>
<tr>
<td>Jones</td>
<td>(11)</td>
<td>(6)</td>
</tr>
<tr>
<td>HH</td>
<td>-26</td>
<td>125</td>
</tr>
<tr>
<td>HF</td>
<td>-31</td>
<td>111</td>
</tr>
<tr>
<td>FH</td>
<td>-40</td>
<td>150</td>
</tr>
<tr>
<td>FF</td>
<td>-43</td>
<td>145</td>
</tr>
<tr>
<td>Oberlin</td>
<td>(1)</td>
<td>(6)</td>
</tr>
<tr>
<td>HCH</td>
<td>-1</td>
<td>185</td>
</tr>
<tr>
<td>HCF</td>
<td>-1</td>
<td>203</td>
</tr>
<tr>
<td>HLH</td>
<td>-1</td>
<td>52</td>
</tr>
<tr>
<td>HLF</td>
<td>-2</td>
<td>49</td>
</tr>
<tr>
<td>FCH</td>
<td>-1</td>
<td>188</td>
</tr>
<tr>
<td>FCF</td>
<td>-1</td>
<td>241</td>
</tr>
<tr>
<td>FLH</td>
<td>-1</td>
<td>69</td>
</tr>
<tr>
<td>FLF</td>
<td>-1</td>
<td>88</td>
</tr>
</tbody>
</table>

Table 3.7 Maximum displacements in Mt. Vernon bridge, in \(10^{-4}\)

<table>
<thead>
<tr>
<th>Test</th>
<th>Near Abut (2)</th>
<th>Crown Near Middle (13)</th>
<th>Crown at Frame (9)</th>
<th>Crown Near Edge (14)</th>
<th>Far Abut. (10)</th>
<th>Spandrel Crack (11)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OR</td>
<td>0</td>
<td>-79</td>
<td>119</td>
<td>151</td>
<td>-4</td>
<td>-15</td>
</tr>
<tr>
<td>OS</td>
<td>0</td>
<td>106</td>
<td>170</td>
<td>92</td>
<td>0</td>
<td>-8</td>
</tr>
<tr>
<td>O2</td>
<td>0</td>
<td>137</td>
<td>147</td>
<td>61</td>
<td>0</td>
<td>-4</td>
</tr>
<tr>
<td>TT</td>
<td>-3</td>
<td>197</td>
<td>301</td>
<td>237</td>
<td>-11</td>
<td>-24</td>
</tr>
</tbody>
</table>

32
Table 3.8 Maximum displacements in Vincent Road bridge, in $\times 10^{-4}$.

<table>
<thead>
<tr>
<th>Test</th>
<th>Near Abut.</th>
<th>Crown</th>
<th>Pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Half load</td>
<td>-61</td>
<td>363</td>
<td>-58</td>
</tr>
<tr>
<td>Full load</td>
<td>-78</td>
<td>539</td>
<td>-87</td>
</tr>
</tbody>
</table>

Table 3.9 Maximum displacements in Elyria bridge, in $\times 10^{-4}$.

<table>
<thead>
<tr>
<th>Test</th>
<th>Near Abut.</th>
<th>Crown</th>
<th>Pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Right</td>
<td>-3</td>
<td>57</td>
<td>-2</td>
</tr>
<tr>
<td>Left</td>
<td>-4</td>
<td>62</td>
<td>-1</td>
</tr>
<tr>
<td>Both</td>
<td>-8</td>
<td>133</td>
<td>-1</td>
</tr>
</tbody>
</table>

Appendices A through G display the deflections measured in each bridge in graphs of truck location versus displacements. The first run of each test is included in the Appendices, although each test typically consisted of three or four identical runs. In all cases, subsequent runs provided nearly identical responses to the first run and, therefore, the first run can be considered to be representative of the specific test conditions.

3.6.2 Range of Variation in Responses

The similar size and shape of the three Lorain County bridges allow observations to be made about the range of deflection responses. All three bridges were tested using the same two trucks. The maximum displacements given for these bridges in Table 3.6 show a general degree of consistency among all three structures. For comparison purposes, the maximum crown displacement at the crown for the half-loaded truck, left edge of roadway, half span test at the Cemetery Road bridge will be used as a baseline. This value was 0.0156 in. The same test at the Jones Road bridge resulted in a maximum displacement of 0.0125 in, in a 20 percent decrease from the Cemetery Road bridge test. At the Oberlin bridge, the equivalent test caused a maximum crown displacement of 0.0185 in, a 19 percent increase. For the fully-loaded truck test at Cemetery, the maximum crown displacement was 0.0248 inch. At Jones and Oberlin, the equivalent displacements were 0.0150 inch and 0.0241 inch, respectively. These were 40 percent and 3 percent decreases, respectively.
From the above percentages, a general range of displacements can be seen for a given bridge type and loading pattern. For the three similar bridges in Lorain County, variations of up to 40 percent are seen.

3.6.3 Linearity of Displacements With Respect To Loads

Since two trucks of different axle weights were used for all but the Mt. Vernon bridge, the relationship between deflections and loads can be observed. If the load-deflection curve begins to exhibit nonlinearities, the formation of hinges in the arch ring is likely. While this is not necessarily an indication of impending failure, it is a sign that a significant portion, possibly one-quarter to one-third, of the bridge's capacity is being approached. In the test to failure on the Bridgemill bridge in the UK, the load-deflection curve for displacement at the crown remained linear up to about one-third of the eventual failure load. After the load was increased beyond one-third of the failure load, the deflections increased more rapidly (Hendry et al., 1985). The authors tentatively concluded that the onset of nonlinear response indicated that one-third of the capacity of the bridge had been reached.

To examine the linearity of responses, Table 3.10 relates the increase in axle loads to the average increase in maximum crown displacements for the tests at each bridge.

<table>
<thead>
<tr>
<th></th>
<th>Half Loaded Truck Axle Weight (lbs)</th>
<th>Fully Loaded Truck Axle Weight (lbs)</th>
<th>Increase in Axle Load, percent</th>
<th>Avg. Increase in Max. Crown Displ., percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cemetery Road</td>
<td>21,100</td>
<td>34,840</td>
<td>165</td>
<td>157</td>
</tr>
<tr>
<td>Jones Road</td>
<td>21,100</td>
<td>34,840</td>
<td>165</td>
<td>125</td>
</tr>
<tr>
<td>Oberlin</td>
<td>21,100</td>
<td>34,840</td>
<td>165</td>
<td>133</td>
</tr>
<tr>
<td>Vincent</td>
<td>28,560</td>
<td>38,140</td>
<td>133</td>
<td>148</td>
</tr>
</tbody>
</table>

For the three similar bridges in Lorain County, the responses do not indicate nonlinear behavior. The ratio of deflection increases was slightly less than the ratio of axle weight increases. This is primarily because of the different axle configurations between the half-loaded and the fully loaded truck. All half-loaded trucks had single-axles. The fully loaded trucks in Lorain County, however, had tandem rear axles spaced 4.5 ft apart. This wider distribution of loads in the fully loaded trucks produced smaller magnitudes of the crown displacements than
would be expected from an equally heavy single axle load. The two trucks used at the Vincent Road bridge had identical wheel configurations, and the percentage increase in response is greater than the percentage increase in load.

The test series at the Mt. Vernon bridge and in Elyria, previously presented in table 3.6 and 3.7, also provide the opportunity to examine the linearity of displacements with respect to the applied live loads. Separate test runs were conducted with a single truck in the outside lane and in the second lane. For each of these test runs, one of the wheel lines of the truck passed continuously above the reference frame. Another test run was completed with both trucks moving in two traffic lanes at the same time, essentially combining the effects of the previously described two tests. Table 3.7 shows that the maximum crown displacements for the outside lane and second lane runs were 0.0119 inch and 0.0147 inch, respectively. The sum of these two displacements is 0.0266 in. The two-truck run caused a maximum displacement of 0.0301 in, however, almost 12 percent greater than the sum of the two one-truck tests. The two-truck test induced a small non-linear deflection response in the arch at the crown. Similarly, at Elyria, the sum of the effect on the crown of the left hand and the right hand tests is 0.0110 inches, while the crown displacement was 0.0115 inches when subjected to both loads at once, indicating a linear load-displacement response.

3.6.4: Evidence of Mechanism Formation

Because of the very low tensile strength of the masonry-mortar assemblage, joint cracks are common in masonry arch bridges. While some cracking is inevitable and often poses no significant structural weakness, certain cracks can identify possible modes of failure. Cracking patterns can separate specific portions of the bridge superstructure from each other, creating discrete groups of voussoirs. A careful mapping of such cracks can lead to a better understanding of the behavior of a given masonry arch bridge.

The crack in the approach spandrel wall of the Mt. Vernon bridge is an example of a crack that provides some insight into the response of the bridge. This crack, monitored by LVDT 11, ran from the top of the parapet into the near spandrel wall. It opened as much as 0.0024 inch during the course of the truck passes. As each truck moved to the far end of the arch span, the
crack gradually stopped opening and began to close again. When the truck had completely passed over the first span and onto the next arch span, the crack had closed slightly from its original position.

The behavior of LVDT 14, when compared to LVDT 11, reveals a connection between the two regions of the bridge. LVDT 14 measured the vertical displacement at the crown, 5 feet from the edge of the bridge. This occurred when the truck was at or near the middle of the span. When the truck moved onto the second span, however, the crown displaced upward slightly. This caused a corresponding decrease in the crack width. Figure 3.4 compares the displacements at the crown and at the spandrel crack. Table 3.7 shows that the maximum vertical displacement at the crown for LVDT 14 was consistently ten times the maximum crack width as measured by LVDT 11. The truck location is the distance from the crown to the center of the rear tandem, as shown in the inset figure.

The spandrel wall and arch ring at the approach abutment are acting as a unit. As a load is applied to the arch ring, this unit rotates about a point at or near the approach abutment. A clockwise rotation induces a downward displacement at the crown, and an opening of the crack in the spandrel. A counterclockwise rotation causes the reverse effects. Since truck passes over the entire bridge are causing both effects in rapid succession, the crack is opening and closing again regularly. Such behavior exacerbates the cracking problem, and can provide weaknesses in the global structure. The presence of major cracks could lead to failure modes such as sliding of voussoirs relative to each other.


**Figure 3.4.** Combined output of transducers 11 and 14: Mt. Vernon bridge.

*This figure illustrates the crack opening (CH11) and the displacement of the crown (CH14) for all truck positions.*

### 3.6.5 Multi-span Bridges

The profiles of the two-span Vincent Road bridge and the Elyria bridge under crown loading, shown in Figures 3.5 and 3.6, indicate the influence of the adjacent span. In general, the interior pier and the second span are more compliant than an abutment and result in slightly greater displacements than for an equivalent single span structure. It is especially curious that the crown displacements of the 15 foot span Vincent Road bridge are over twice as great as those of the 50 foot span Elyria structure. This is attributable to the larger fill mass of the bridge in Elyria (4 feet of fill above the crown as opposed to no fill above the crown at Vincent Road). Moreover, the extreme rigidity of the Elyria bridge appears to indicate the presence of haunching.
Figure 3.5. Displacement profile of Vincent Road Bridge.  
This figure shows the displacement at all points of the arch ring transducers through both spans for a truck placed in each span.

Figure 3.6. Displacement profile of Elyria bridge. 
This figure shows the displacement at all transducers in both spans for two trucks at the midpoint of one span.
3.6.6 Lateral Wheel Load Distribution

Although it was not possible to obtain weighed trucks for testing the bridge at Xenia, the bridge was tested under ambient loading: that is, a series of readings was taken for truck passages during normal traffic events. The bridge at Xenia is unusually wide--100 feet--and has a relatively small fill depth at the crown--one foot. The instrumentation included four instruments at the arch crown across the width of the north side of the bridge, which permitted observations of the distribution of the loading across the width of the structure. Figure 3.7 illustrates a typical result for a truck located in a lane from x-coordinates -12 to 0. The loads appeared in general to be quite widely and uniformly distributed, especially away from the stiffening influence of the spandrel wall at x=-34.

![Diagram showing truck location and displacement](image)

**Figure 3.7.** Profile across width of crown: semi-trailer in right lane.

*This figure shows the crown displacements across the width of the structure for a single truck placed between -12 and 0.*

Figures 3.8 through 3.14 illustrate the bridges in the testing program.
Figure 3.8. Cemetery Road bridge.

Figure 3.9. Jones Road bridge.
Figure 3.10. Oberlin bridge.
Figure 3.11. Mt. Vernon bridge.
Figure 3.12. Vincent Road bridge.
Figure 3.13. Xenia bridge.

Figure 3.14. Elyria bridge.
4. FINITE ELEMENT MODEL OF MASONRY ARCH BRIDGE

This chapter describes the finite element procedure that was created to duplicate the displacements found in the field tests. The element types and relevant input quantities are explained.

4.1 Masonry Arch Finite Element Mesh

To adequately duplicate the behavior of a system of discrete blocks under fill, five element types were needed. Isoparametric elements were used for the voussoirs, while gap and hinge elements provided the necessary mesh connectivity between blocks. Cable elements were used to simulate the resistance to arch movements provided by the fill. Spring elements were placed at the abutments to control the amount of abutment spread under loads. Figure 4.1 shows the arch mesh.

![Figure 4.1. Arch finite element mesh.](image-url)
4.2 Load Distribution Though Fill Procedure

The truck axles that pass over a masonry arch under fill can be thought of as applying point loads to the roadway beneath. As that point load is transferred from the roadway through the fill and ultimately to the arch, it is distributed over a certain area. Once the load reaches the arch, it cannot be reasonably approximated as a single-point load, unless the depth of fill is very small. For the mesh of STIF42 elements used to model the arch ring, a series of vertical nodal forces is required to represent the axle load.

The Boussinesq stress distribution is an equation for finding the stress in a soil mass due to surface loading. The form of the equation is given in Equation 4.1.

\[
\Delta \sigma_v = \frac{3Q}{2\pi} \frac{Z^3}{(r^2 + Z^2)^{3/2}} = \frac{Q}{Z^2} \frac{3}{2\pi \left[ 1 + \left( \frac{r}{Z} \right)^2 \right]^{1/2}}
\]  

(4.1)

where \( \Delta \sigma_v \) = change in vertical stress
\( Q \) = point load
\( Z \) = depth of fill
\( r \) = horizontal distance from load \( Q \) to point of interest

If the coordinates of the arch nodes are known, along with the magnitude and coordinates of the axle load, the stress at a point on the arch extrados can be found. Since ANSYS needs vertical nodal forces, the stress \( \Delta \sigma_v \), is multiplied by the 12-inch width and the specific length of the block to convert stresses to forces. All calculations can be easily performed on a spreadsheet, such as Microsoft Excel\textsuperscript{TM}, to find the complete nodal force breakdown. All that are required are the nodal coordinate of the extrados, the fill depth at the crown, the length of one voussoir, and the magnitude of the axle load.
4.3 Procedure For Using ANSYS Finite Element Model

The following steps are taken to determine the deflections and/or thrust line in a masonry arch under fill using the previously described models:

- The Boussinesq spreadsheet is generated using the coordinates of the arch ring extrados. For any axle load magnitude or location, a load distribution to the arch ring can then be determined. These loads will be applied to the arch ring as live loads.

- The arch ring mesh is generated according to the dimensions of the actual arch bridge. A thickness of 12-in is input again to give self-weight to the arch. The fill dead loads are applied to the arch model, and the displacements are output by ANSYS.

- A second run is performed with both fill dead loads and axle live loads applied. The difference between the displacements for this run and the dead load-only run are the displacements that can be compared to the field test data.

- ANSYS can also register the status (i.e., open or closed) of all gap elements, and the values of the normal and tangential forces at each gap. With this data, the location of the thrust line in the arch can be found. The procedure is elementary at any joint where one gap is open and the other is closed. Since such a joint is hinging about the closed gap, the thrust line must pass through the closed gap.

4.4 Results

4.4.1 Comparing Field Tests and Finite Element Models

The field tests resulted in the accumulation of a great deal of displacement data. For a given axle configuration, the deformed shape of the arch ring was found. To verify the general applicability of the finite element model, the field displacements were compared to ANSYS-generated displacement output. The criteria for comparing different meshes, labeled mesh A and mesh B, were their ability to accurately duplicate the displacements found in the field tests. The best model produced displacements that most closely agreed with the test data.

The axle loads were then applied as nodal forces. These forces were found using the Boussinesq spreadsheet procedure. Subsequent ANSYS runs were conducted with the fill loads and superimposed live loads both present. The displacements from the fill load-only runs were
subtracted from the displacements from the combined load runs to determine the displacements caused by live loads only. These values were compared to the test data. The three mesh characteristics of voussoir modulus of elasticity, cable stiffness, and abutment spring stiffness were varied until the field data and computer displacement output were in relatively close agreement.

Four bridges were studied using the ANSYS finite element model procedure: The Jones Road bridge and the Mt. Vernon bridge, in Ohio, and bridges at Bridgemill and Strathmashie, in the United Kingdom in 1984 and 1989, respectively, as part of a Transport and Road Research Laboratory study (Hendry et al., 1985; Page, 1989). The particulars on the two UK bridges are given in Table 4.1.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Bridgemill</th>
<th>Strathmashie</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (in)</td>
<td>720</td>
<td>371</td>
</tr>
<tr>
<td>Height (in)</td>
<td>112</td>
<td>118</td>
</tr>
<tr>
<td>Span/Height</td>
<td>6.44</td>
<td>3.14</td>
</tr>
<tr>
<td>Thickness (in)</td>
<td>28</td>
<td>23.6</td>
</tr>
<tr>
<td>Roadway Width</td>
<td>25'-1&quot;</td>
<td>16'-1&quot;</td>
</tr>
<tr>
<td>Arch Barrel Width</td>
<td>27'-3&quot;</td>
<td>19'-1&quot;</td>
</tr>
<tr>
<td>Fill Over Crown (in)</td>
<td>28</td>
<td>16</td>
</tr>
<tr>
<td>Angle of Embrace</td>
<td>69°</td>
<td>130°</td>
</tr>
<tr>
<td>Number of Voussoirs</td>
<td>≈60</td>
<td>≈90</td>
</tr>
</tbody>
</table>

Neither UK bridge was still in service. The Bridgemill bridge was still in reasonably good condition, with minor cracks and little distortion. The Strathmashie bridge was in poor condition. A major longitudinal crack was present in the arch barrel, and much of the mortar was missing from the rubble masonry arch.

The UK tests involved a gradually increasing line load, placed at the quarter-span point. The load was gradually increased in magnitude until failure of the bridge. The data for the UK study were presented in graphs of load magnitude vs. displacement for key points along the arch ring, unlike the truck location vs. displacement data gathered in the tests of this study. To model the two UK tests using ANSYS, the same procedure was followed as for the tests of this study, with the following exception: the arch live loads found by the Boussinesq spreadsheet were
increased in magnitude for three consecutive tests. These three runs were compared to the UK data.

4.4.1.1 Jones Road Bridge

The Jones Road finite element model used one STIF42 element for each of the 23 voussoirs. Because the blocks were modeled one-to-one by STIF42 elements, the gap elements were located in the same position as the joints in the bridge itself. Four truck axle locations were chosen for the ANSYS runs: above the crown, and three additional locations between the crown and the springing. The half-load, half-span test data were used first to verify the ANSYS displacement results.

Figure 4.2 shows the displacements at the crown from the field test, and compares them to the ANSYS output. Based on better agreement, only mesh B is used in the subsequent description. Figure 4.3 compares the field test data to the results for the far abutment. Although a reasonable duplication of the field results is achieved, the finite element model consistently underestimated the displacements for loads outside the middle third to middle half of the span. Although agreement was quite close for axle loads at or near the crown, a gradual underestimation of the loads was noticed as the axle loads moved away from the crown. This effect was present for crown, haunch, and abutment displacements.
**Figure 4.2** Comparison of field results and finite element results: Jones Road bridge.

*This figure illustrates the effect of two alternative finite element meshes in the half-load, half-span test of the Jones Road bridge.*

To further verify the applicability of this analysis to the Jones Road bridge, another series of computer runs were made to predict the full-load, half-span test. The same mesh was used, but the axle loads were taken from the fully loaded test truck. The results for vertical displacement at the crown and horizontal displacement at the far abutment are shown in Figure 4.4.
Figure 4.3 Comparison of field results and finite element results: Jones Road bridge.

This figure shows the horizontal displacement of the abutment for the half-load, half-span test, using mesh B.

The maximum ANSYS displacements are usually close to the displacements measured in the field. The same errors occur as in the half-load test; that is, underestimating the loads for axles away from the crown.
oad distribution analysis may be needed to overcome this situation, but is not simple analysis procedure.

results are reasonably accurate given the random nature of masonry and the problem. To further test the limits of the finite element procedure, it was used to compare the results from two tests conducted in the United Kingdom.

bridgemill Bridge

bridgemill bridge had approximately 60 voussoirs, and thus could not be modeled per voussoir. The field test consisted of applying a gradually increasing line sprout-span point until total failure of the bridge. Displacements were monitored at several points along the haunches, and at the crown by means of LVDT’s and eying equipment. Graphs of load versus displacement were created for each arch ring.

4.6 shows a comparison between the field displacements and the ANSYS for the load point at the 1/4 point of the arch ring. The ultimate load carried by the approximately 25,000 lbs/ft of width, which was the limit of the hydraulic jacks used. Although the bridge did not collapse, it had severe local crushing and spalling, to be close to its capacity. From Figure 4.6 it can be seen that the ANSYS model the displacements in the arch ring up to a maximum load of about 8,000 - 12,000 load increases past that point (not shown in figures), the finite element model estimates the displacements. ANSYS provides a linear load-deflection response, appropriate for the first third or first half of the loading history. Once the bridge begins plastic response, however, the ANSYS results are no longer useful. The load where displacements and ANSYS displacements diverge falls somewhere near one-third to the ultimate load of the bridge.
The testing procedure was basically identical to that used in the Bridgemill test. A line load at the quarter-span was gradually increased to failure, which occurred as a total collapse at a load of approximately 14,500 lbs/ft. Displacement gages and surveying were used to monitor the displacements in the bridge at the abutments, haunches, and crown during the test.

![Graph showing load vs. displacement for Strathmashie bridge](image)

**Figure 4.7:** Vertical displacement results: Strathmashie bridge.

The ANSYS runs consisted of three load increments applied to a particular mesh. The displacements for a variety of meshes were examined to see which mesh best duplicated the test data. In Figure 4.7, the ANSYS displacements are compared to the test data for vertical displacements at the 1/4 point. Reasonable agreement is reached for all locations along the arch ring for loads less than one-half of the failure load. For loads above that, ANSYS begins to underestimate the displacements. The ability of the finite element model to predict displacements in the linear range is verified.

The small value for voussoir modulus of elasticity, 150,000 psi, is an interesting aspect of this mesh. Because the Strathmashie arch consists of small randomly-shaped voussoirs, a large proportion of the arch ring is made up of masonry joints. Of those joints, many had little or no masonry present. The poor condition of the arch ring caused it to lose a substantial amount of
strength and stiffness. The small value for modulus of elasticity was required to achieve a level of agreement between the field displacements and the ANSYS displacements. The modulus of elasticity of the overall stonework must be used in the finite element mesh, not simply the assumed modulus of elasticity of the stone itself. This was the basis for the two-thirds reduction in the value for the Bridgemill bridge. A larger reduction factor would undoubtedly be required for the deteriorated Strathmashie bridge.

4.4.2 Thrust Line Calculations

The ability to predict the location of hinge formation and probable mechanisms in an arch are valuable applications of the thrust line concept. As the thrust line approaches the extrados or intrados of the arch, the likelihood of crack formation in the joint increases. A joint with the thrust line touching the extrados or intrados of the arch has a tendency to form a hinge. Four such hinges in an arch would lead to its failure as a mechanism, as discussed in section 2.2.

The ANSYS output includes normal forces at the intrados and extrados at each joint, which can be used to determine the thrust line position at the joint. For the Jones Road and Mt. Vernon meshes, the thrust lines were found when the truck axle loads were imposed. The gap forces were placed on a spreadsheet, which was used to calculate the eccentricity of the thrust along the arch ring.

4.4.2.1 Jones Road Bridge

Figure 4.8 shows two thrust lines in the Jones Road bridge for the half-loaded truck test. The thrust line for an axle above the crown is given, as well as the thrust line for an axle load above joint 9, approximately the quarter-span point. Because the Jones Road mesh placed the gap elements in the same location as the joints in the actual bridge, the x-axis of the thrust line plot was the joint number. Joint 1 is at the near abutment, and joint 24 is at the far abutment.

The thrust line for the axle load at the crown case shows the formation of a hinge at the crown. A positive eccentricity of 9 in is present at joints 12 and 13, the two joints on each side of the keystone. This can be considered a single hinge, because two adjacent hinges forming on the same side of the arch ring will act as one hinge. Such a hinge is a sign that a potential crack
would form in the intrados of the arch ring at the crown under these loading conditions. The thrust line does not touch the extremes of the arch ring at any other location, and shows the arch is in no danger of a mechanism failure under this load case.

When the axle load is above the quarter-span point of the arch, a very different shape is found for the thrust line. In this case, the thrust line does not touch the extrados or intrados of the arch. When the half-loaded truck rear axle is above the quarter-span, no hinge formation is expected. A mechanism failure is less likely to occur under quarter-span loading than crown loading for a given axle load. This contradicts much of the prevailing opinion on masonry arch bridges, which claims the quarter-span point of the arch is the critical load location. For an in-service bridge under fill, the critical load location appears to be at or near the crown. The two thrust lines in Figure 4.9 show this graphically. Higher eccentricities exist in the thrust line for crown loading than for quarter-span loading. The formation of a mechanism is more likely when the load is near the crown. One reason is the smaller depth of fill. The axle load is distributed over a smaller area of arch barrel when the depth of fill is relatively small. The deeper fill above the quarter-span point allows the load to applied to a larger area of the arch barrel, reducing the load on a unit width of arch.
Figure 4.8: Thrust lines: Jones road bridge.

*These figures show the thrust line determined by the ANSYS model half loaded truck. The horizontal axis shows the joint location around the arch ring.*
4.4.2.2 Mt. Vernon Bridge

The same procedure was followed to find the family of thrust lines for the one-truck, edge of road test runs at the Mt. Vernon bridge. The thrust line is shown in Figure 4.9 for a loading condition with the rear tandem just beyond the crown. In this case, the entire thrust line is confined within the arch ring.

![Graph showing the thrust line for one truck condition: Mt. Vernon bridge.](image)

**Figure 4.9:** Thrust line for one truck condition: Mt. Vernon bridge.

*This figure shows the thrust line for one truck located at the crown. The horizontal axis shows the angle from the horizontal of the joint where the thrust is calculated. The abutments are at 40° and 140°, and the crown is at 90°.*

4.5 Evaluation of Finite Element Procedure

The displacements from the Jones, Mt. Vernon, Bridgemill, and Strathmashie field tests have helped develop a finite element procedure that can accurately predict the displacement behavior of a masonry arch bridge. ANSYS can also provide information on the forces transferred through the joints in the bridge, leading to a simple method for finding the location of the thrust line for a given loading case. Although the general applicability of the model has been
established, better results will be obtained when the known sources of error are identified and corrected as much as possible.

4.5.1 Possible Sources of Error in Finite Element Results

Discrepancies between field results and the model results can occur when the axle load passes through a region of deeper fill. This lessens the predictive capability of the model for loads away from the middle half of the span. The displacements for such cases are more dependent on the transverse distribution of the load through the fill. These effects cannot be accounted for in a two-dimensional model. The critical load location for a typical mechanism failure, however, will nearly always be within the middle half of the arch span. Loads between the quarter-span point and the crown are of primary interest to the analyst. Loads away from the crown do not influence the load carrying capacity of the arch.

The presence of haunching in a given bridge can also cause a lack of agreement between the finite element displacements and the actual displacements. If the user of the finite element procedure wishes to account for this factor in the model, additional elements can be added, or the existing elements modified, to correct the error. If this is not done, more accurate results for crown and abutment displacements can still be found.

4.5.2 Guidelines for Application of Finite Element Procedure

The best values for the assumed material properties for each of the four bridges used in the ANSYS tests are summarized in Table 4.2.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>E of Voussoirs (psi)</th>
<th>E of Cables (psi)</th>
<th>Spring Stiffness (lb/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jones Road</td>
<td>3,000,000</td>
<td>500,000</td>
<td>10,000,000</td>
</tr>
<tr>
<td>Mt. Vernon</td>
<td>5,000,000</td>
<td>30,000,000</td>
<td>$2 \times 10^8$ (far abutment)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$2 \times 10^9$ (near abutment)</td>
</tr>
<tr>
<td>Bridgemill</td>
<td>750,000</td>
<td>600,000</td>
<td>500,000</td>
</tr>
<tr>
<td>Strathmashie</td>
<td>150,000</td>
<td>500,000</td>
<td>500,000</td>
</tr>
</tbody>
</table>
The following guidelines identify some of the factors to consider when choosing which values to use in a load rating analysis.

4.5.3 Modulus of Elasticity of Vousoirs

The modulus of elasticity of the vousoirs in the model must represent the overall material property of the stonework, including any mortar present, to achieve good results. For ashlar masonry with very tightly-fitted joints, the effective elastic modulus of the overall stonework may be close to the actual elastic modulus of the vousoir material. Several aspects of a bridge’s construction and condition, however, can reduce the overall elastic modulus. Some of these include:

**Thick mortar joints:** Because the elastic modulus of mortar is far less than that of stone, thick mortar joints may reduce the effective modulus of the arch ring.

**Arches with many small vousoirs:** An arch with a number of small vousoirs, such as Strathmashie, will have a larger proportion of mortar joints than an arch with fewer but larger vousoirs.

**Rubble masonry construction:** Unless extremely well-fitted, rubble masonry tends to have a more irregular contact surface between stones. This can cause gaps in the joints, and usually indicates thicker mortar joints. Both of these effects tend to reduce the effective modulus of elasticity. This was the case at Strathmashie.

**Missing mortar in joints:** Even well-constructed joints can lose mortar over time. Joints with missing mortar can reduce the stiffness and strength of the arch, and cause local weak points that can bring about an early failure.

**General deterioration:** If the overall condition of the bridge is poor, it will tend to be less stiff than a well-maintained bridge. Because most masonry arch bridges are very old, many are not in good condition and the effective stiffness of the bridge is lessened. Major cracks, broken or missing stonework, and permanent deformations in a bridge are signs of overall deterioration in a bridge.
All of the above factors contribute to some reduction in the modulus of elasticity for the finite element model. None of these applied to the Jones Road and Mt. Vernon bridges, and their elastic moduli were correspondingly high. Typical values for sandstone specimens are between 1,900,000 and 7,700,000 psi, and the values used for these two bridges, 3,000,000 psi and 5,000,000 psi, respectively, were well inside this range (Merritt and Ricketts, 1994). The Bridgemill bridge was made from a relatively soft sandstone and was slightly below this range. All of the reduction factors listed applied to the bridge at Strathmashie, and its effective modulus of elasticity of 150,000 psi reflected this fact.

4.5.4 Cable Stiffness

The cable stiffness was relatively low for the Jones Road, Bridgemill, and Strathmashie meshes. For the Mt. Vernon mesh, a much higher value for the cable modulus of elasticity was needed to reach agreement between the field data and the ANSYS data. The soil properties of the fill surrounding the arch are the key factor in determining this mesh property. The Jones, Bridgemill, and Strathmashie bridges all had saturated clayey silts as fill. Such soils are not particularly firm and provide relatively little restraint against outward movements of the arch ring. The Mt. Vernon arch was surrounded by a dense granular fill. This type of soil provides a relatively high degree of restraint against the arch. When determining the stiffness of the cables in the mesh, the nature of the surrounding soil must be considered. As seen in the meshes used in this study, a bridge with a silty clay fill must be given a lower cable stiffness than a bridge with a dense, granular fill.

4.5.5 Abutment Spring Stiffness

The ability of the abutments to resist horizontal thrusts is a key variable in the overall behavior of the arch. If the abutments are known to rest against rock formations or other firm foundations, they may be relatively immovable under service loads. The abutment spread at the Mt. Vernon bridge was nearly zero, due to the fact that the abutments rested on pile foundations. Such foundations resist horizontal movements much more than spread footings. The Jones Road bridge rested on spread footings, and was seen to have much larger abutment movements than the Mt. Vernon bridge. Knowledge of the foundations of a given bridge is critical in deciding on a
reasonable spring stiffness. Meshes for bridges resting on pile foundations must be given higher spring stiffnesses than those for bridges with spread footings.
5. ELASTIC FRAME ANALYSIS

5.1 Introduction

Although the results of the finite element analysis described in the previous chapter are revealing, a simpler method of analysis is proposed for use as the basis of a load rating procedure for masonry arch bridges. The findings of the above finite element model can be applied to a quicker and more widely available means of analyzing structures. Although a hand method might be preferred by the bridge community, methods such as moment distribution or moment area are inapplicable to an arch structure because of interaction between axial forces and moments. Elastic frame analysis programs are widely distributed, inexpensive, understandable by any engineer, and easily implemented on a desktop or laptop personal computer. The results of the testing program and the finite element analysis show a global linearly elastic behavior for masonry arch bridges in the range of loadings of interest to bridge owners. Although the thrust line in an arch under service loading often leaves the middle third of the arch ring, violating an assumption of elastic arch analysis, this has been shown in the previous chapters to have a minimal influence on the overall behavior of the structure.

In this section, a model of some of the bridges in the later testing program is developed based on the use of elastic frame analysis, implemented through the widely available and inexpensive RISA-2D program (RISA Technologies, 1994). The analysis is completely independent of the software used, that is any plane frame analysis program, such as SAP-90 (Wilson and Habibullah, 1989) or STAAD-III (Research Engineers, 1982), etc. would work equally well for this task. The results of the frame analysis are compared to the results of the field testing. The output of the frame analysis is used to develop data explaining the axial force and bending moment encountered at any location along the arch ring. The thrust and moment results will be used in the subsequent chapter as a basis for establishing load ratings of the structures. Because parameters such as modulus of elasticity of the masonry and support stiffness are never well known, the sensitivity of the frame analysis results to variations in these parameters is studied. The chapter concludes with recommendations for the application of plane frame analysis to the analysis of masonry arch bridges.
5.2 Elastic 2-D Frame Analysis Programs

Elastic frame analysis programs are widely distributed within the structural engineering community for use with personal computers. They are used for designing of steel, concrete, timber, and masonry structures, and for general analysis of framed structures. The geometry of a framed structure is defined by the plane coordinates of nodal points, and by the elements that connect nodal points. The inputs consist of the geometry of node points, node connectivity, element properties, support releases or springs, nodal or element loads and load combinations. The solution is generally based on the stiffness method. The output consists of nodal displacements in plane coordinates, and member forces. Although the specific program RISA-2D was used in this study, identical results can be expected from the application of any other plane frame analysis program.

5.3 Arch Bridge Model

5.3.1 Frame

To model an elastic arch as a frame structure, it is necessary to divide the circumference into segments, and to identify nodal coordinates at the end of each of the segments. The nodes are joined by straight segments and fixed at the abutments. Spring supports allowing horizontal translation, but not vertical displacement or rotation, are added at each of the supports, in view of the observed horizontal displacements in the field testing program. Although RISA-2D allows the members between nodes to be curved, no significant difference in the response between straight and curved elements was found and, as not all frame analysis programs allow curved elements, this feature was not used. It was found that ten segments around the circumference of the arch, as in BA 16/93 (Department of Transport, 1993b), are sufficient to capture the behavior of the arch ring.

5.3.2 Properties

Appropriate member properties for entry into the frame analysis program were determined by a analysis of the field data, finite element results, and consultation with the literature on stone
masonry construction. The geometric properties, cross sectional area and moment of inertia, are based on the geometry of a unit (1-ft) width of the arch ring, discounting the effects of spandrel walls, fill, and haunching. Neglecting the strengthening and stiffening effects of these features is conservative from the point of view of load rating, and allows for these features to be replaced in the future. The fundamental material stiffness property used in the analysis is an effective modulus of elasticity representing the combined effect of masonry units (voussoirs), mortar, and joints. As is shown below, the resulting modulus of elasticity is significantly lower than the modulus of elasticity of the units alone, reflecting the general use of soft mortars in the construction of arch bridges. The stiffness resulting from data matching of frame analysis results is also slightly lower than that determined for the finite element modeling, reflecting general differences in the procedures. Elastic spring stiffnesses are also input for the horizontal supports at the abutments and interior piers. The stiffness constants used are based on matching the field data and finite element results. Abutment spring stiffnesses are generally greater than pier spring stiffnesses, due to the compliance of an adjacent span relative to the abutment. This effect was observed in the test of the Mt. Vernon bridge, described previously.

5.3.3 Loads

The analysis of the arch ring uses the geometry and properties of a model of a unit width of the arch ring. The loads applied to this arch segment must be determined with reasonable accuracy to complete the analysis. The self weight of the arch ring is computed accurately by the program, given the cross sectional area and density of the material of the arch ring. Dense stone weighs 160 lbs/ft³, so this value was used consistently for the self weight of the arch ring. Superimposed dead loads include the weight of the fill and the weight of the paving material. The superimposed load on an arch segment was taken to be the total weight of the segment of fill and paving lying above the element, uniformly distributed over the element. The geometry of this segment is illustrated in Figure 5.1. The weight of the segment, given a unit width of arch ring, is the area of the segment times the unit density. The unit density is taken as 110 lbs/ft³—a reasonable value for soil materials. The area can be calculated exactly using a formula from Boothby (1995) or using a computerized drafting program, such as AutoCAD. A good
approximation can be obtained by taking the area of a trapezoid having the same corner points as the area shown in the figure.

![Figure 5.1: Geometry of arch ring and fill segment.](image)

The live load is taken as a linearly varying vertical pressure on the back of the arch ring resulting from a truck axle load. The pressure due to an axle load is calculated at the elevation of the two
nodal points, based on a trapezoidal pressure distribution, as illustrated in Figure 5.2. The total axle load is assumed to be applied to a length of 1 ft. The width of the influence of the axle is assumed to be one traffic lane, or 10 ft. The load is assumed to be dispersed through the fill at a slope of 2 vertical to 1 horizontal. At a depth of 2 ft below the road surface, the total axle load will be applied over a 3-ft-by-10-ft area, and the pressure will be 1/30 the axle load/ft². This procedure is similar to that given in BA 16/93 (United Kingdom Department of Transport, 1993b). The calculation of the live load, then, involves the determination of the depth of the fill above each of the two nodes of the arch ring segment, the determination of the wheel load pressure at each of the two nodes, and the assumption of a linearly varying pressure from one node to the other.

5.3.4 Comparison of results to Field Data

Four of the bridges in the testing program were modeled under the actual loading of the test program for comparison of results. The results are compared on the basis of the profile of an entire span of the arch (or two spans, in the case of the Vincent Road bridge) under a single load placement, rather than the deflections of a single point under multiple placements of the load, as in the finite element study in Chapter 3. The bridges for which this comparison was undertaken are the Jones Road bridge, representative of the three similar bridges in the Oberlin area, the Mt. Vernon bridge, the Vincent Road bridge, representative of a multiple span structure, and the Mussey Avenue bridge. The best matches are illustrated in Figures 5.3 through 5.6 for crown loading. Table 5.1 below indicates the values of modulus of elasticity and abutment spring stiffness used to obtain the match. The table further indicates the values of these constants used in the finite element study.

<table>
<thead>
<tr>
<th></th>
<th>Frame Analysis</th>
<th>Finite Element Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>E (ksi)</td>
<td>E (ksi)</td>
<td>K (k/in)</td>
</tr>
<tr>
<td>Jones</td>
<td>2500</td>
<td>1500</td>
</tr>
<tr>
<td>Vernon</td>
<td>2500</td>
<td>10000</td>
</tr>
<tr>
<td>Vincent</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>Elyria</td>
<td>5000</td>
<td>10000</td>
</tr>
</tbody>
</table>

Table 5.1: Elastic constants for frame analysis.
Figure 5.3: Frame analysis to field data matching: Jones Road bridge.

This figure shows the measured and calculated displacements for the bridge with the load placed at the crown. The horizontal axis represents the location at which the displacement is measured. The units are inches along the intrados, as shown in the inset diagram.
Figure 5.4: Frame analysis to field data matching: Mt. Vernon bridge.

This figure shows the measured and calculated displacements for the bridge with the load placed at the crown. The horizontal axis represents the location at which the displacement is measured. The units are inches along the intrados, as shown in the inset diagram.
Figure 5.5: Frame analysis to field data matching: Vincent Road bridge.

This figure shows the measured and calculated displacements for the bridge with the load placed at the crown. The horizontal axis represents the location at which the displacement is measured. The units are inches along the intrados, as shown in the inset diagram.
Figure 5.6: Frame analysis to field data matching: Elyria bridge.

_This figure shows the measured and calculated displacements for the bridge with the load placed at the crown. The horizontal axis represents the location at which the displacement is measured. The units are inches along the intrados, as shown in the inset diagram._

The following observations can be made on the basis of this effort.

- It is possible to reasonably match the field observations of arch bridges using a simple frame analysis model.

- The finite element model employed in Chapter 4, which uses hinges and gaps to replicate the behavior of a mortar joint between masonry units, is more sophisticated than the frame analysis model in this Chapter. It can be observed that the frame analysis model is equally successful in capturing the behavior of the structure, however, as represented by the profile of the deflected shape under load.
Relatively small values of the modulus of elasticity for stone masonry are appropriate for the modeling of stone arch bridges. The correct value to use is significantly less than the modulus of elasticity of the stone and of the value obtaining a match using the finite element method.

5.3.5 Parametric Studies

It can be noted from the study above that the modeling of a stone arch bridge by the frame analysis method, or any method, involves estimation of some of the parameters used in the model. It is important to determine which of the parameters influences the outcome of the load rating analysis, and what sort of estimates should be made to result in an appropriately conservative load rating. To address these questions, a parametric study was undertaken on the results of the analysis of the first span of the Mt. Vernon bridge. The load rating of a bridge is influenced only by the position and magnitude of the thrust line, which determines the stresses that are developed in the masonry to resist the applied live load. The magnitude of the thrust is fixed by the geometry of the structure and the loads applied to the structure. The deflections measured in the field are not important in themselves, but have been used only as measures of the quality of the analytical models developed for analysis of the structure. Hence, a choice of the modulus of elasticity or support spring stiffness that overestimates or underestimates the deflection will not influence the load rating, unless it also influences the location of the thrust line. The AASHTO deflection criterion for highway bridges is not significant in the load rating of masonry arch bridges. The maximum deflection observed in the testing program was .06 in. Based on the deflection criterion of l/1000, this deflection would not be excessive for any structure over a 5-ft span.

For the Mt. Vernon bridge, the position of the thrust line is shown for five different choices of the modulus of elasticity in Figure 5.7. As the five thrust lines are superimposed on the figure, it is clear that the thrust line position and, hence, the stresses in the masonry, are insensitive to the modulus of elasticity of the masonry. The thrust line position is shown for the same structure for seven choices of pier and abutment spring stiffness. The abutment and pier spring stiffnesses are increased or reduced in the same proportion from an abutment stiffness of 100 k/in/foot of width at the abutment and 10 k/in/foot of width at the pier to 200 times these
values. It is apparent from the summary plot of these results in Figure 5.8 that the thrust line position is very sensitive to the selection of this parameter. A reduced horizontal stiffness at the abutment or pier results in significantly greater moments within the arch, hence in significantly greater eccentricities of the thrust. A greater eccentricity of the thrust is more likely to result in a local failure of the material of the arch ring.

![Diagram](image.png)

**Figure 5.7:** Effect of changes in $E$ on thrust line location.

*This figure shows that the thrust line position is insensitive to the choice of $E$. The horizontal axis represents the joint at which the thrust line location is calculated.*
Figure 5.8: Effect of changes in support spring constant on thrust line location.  
This figure shows that the thrust line position is extremely sensitive to the abutment stiffness.

5.4 Proposed Modeling Procedure

The arch ring is to be modeled as a series of at least 10 linearly elastic segments, with node points at the centerline of the arch ring. The cross sectional properties of a 1-ft width of arch ring shall be used in the analysis, based on the depth of the arch ring at the exposed faces. In the absence of testing, the modulus of elasticity of the material of the arch ring shall be assumed to be 500 ksi. The arch shall be assumed to be fixed against rotation at the abutment and interior piers, but with elastic restraint against horizontal sliding. The value of the spring constant, in the absence of testing, shall be taken from the table below.

Table 5.2: Recommended elastic constants for rating analysis.

<table>
<thead>
<tr>
<th></th>
<th>Abutment</th>
<th>Interior Pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good quality, sound masonry</td>
<td>5000 k/in</td>
<td>2500 k/in</td>
</tr>
<tr>
<td>Weak masonry</td>
<td>1000 k/in</td>
<td>500 k/in</td>
</tr>
</tbody>
</table>

Field testing can also be used to determine the appropriate values for use in testing. A deflected profile of the bridge can be determined in the field under a specific loading, and the shape can be matched to the shape determined by the following analysis method.
The dead load of the arch ring shall be computed, using a unit density of 160 lbs/ft³. The dead load of the fill and road surface shall be computed, using a unit density of 110 lbs/ft³. The rating axle load shall be applied to an area of 10 feet wide and one foot long of the deck of the structure, and distributed lengthwise through the fill at a slope of 1 horizontal to 2 vertical. On the basis of this load distribution, the pressure at each end of each element of the arch ring at the extrados shall be computed, and a linearly varying distribution of pressure shall be assumed.
6. LOAD RATING PROCEDURE

6.1 Introduction

The preceding chapters have furnished an experimental and analytical basis for the development of load-rating procedures for masonry arch bridges. In the following chapter, the load rating procedure proposed in this project is described in detail. This chapter includes the application of the frame analysis procedure to the modeling of an arch bridge, the development of a resistance model for stone masonry, the estimation of material properties, and the determination of inventory and operating ratings. An extended example is furnished in section 6.6, and the proposed procedure is summarized in section 6.7.

6.2 Analysis

The analysis shall be conducted in accordance with the modeling guidelines of the previous chapter. In accordance with the AASHTO Manual for the Maintenance Inspection of Bridges (AASHTO, 1983), a load factor of 1.3 shall be used for the dead loads, a factor of 1.67 shall be used on live load + impact for the inventory rating and 1.00 for the operating rating. For all but the longest span arches, a single wheel load from the heaviest axle applied at the crown will produce the most critical condition. Although a concentrated load applied at the 1/4 or 1/3 point is more critical, the distribution of wheel loads through the fill and the additional dead load of the fill at the haunches makes 1/4-or 1/3-point loading less critical. The applications of the procedure to some of the bridges tested in this program will demonstrate this.

6.3 Strength of Jointed Stone Masonry

Although the usual development of the plastic collapse of masonry arches reflects the assumption of unlimited strength in the masonry, this assumption is unrealistic. The strength of a masonry joint can be investigated analytically, as in Taylor and Mallinder (1993), by constructing a moment/axial force interaction diagram for the arch ring. The simplest interaction diagram, based on limit states analysis, incorporates the assumption that the masonry reaches a peak compressive stress, followed by a limited softening behavior under the combined action of axial force and bending. The envelope of axial force and moment can be drawn for each arch under an
appropriate set of assumptions. The envelopes developed by Taylor and Mallinder differ very little from the simple proposed failure envelope sketched in Figure 6.1. This failure envelope is defined by the equation

\[
\frac{1}{4} M - \left( \frac{P}{P_0} \right) + \left( \frac{P}{P_0} \right)^2 \leq 0
\]

where \( P_0 = bh\sigma_0 \) is the maximum concentric axial force, \( M_0 = 0.125bh^2\sigma_0 \) is the maximum moment (at an eccentricity of \( h/4 \)), and \( \sigma_0 \) is the peak stress in the masonry. Figure 6.1 represents a non-dimensionalized curve for this inequality, with the interior representing the “safe” zone: combinations of axial force and moment which can be plotted within the shaded region are insufficient to cause the formation of a hinge. The envelope representing strictly elastic behavior and the dashed line representing the rigid block of infinite strength are also plotted on this figure. Note that figure 6.1 is comparable to a column interaction diagram in reinforced concrete design.

---

**Figure 6.1:** Non-dimensionalized mortar joint strength.

*This figure can be used as an envelope of joint strength for any joint by substituting \( P_0 = bh\sigma_0 \) and \( M_0 = \frac{1}{8} bh^2\sigma_0 \).*
6.4 Determination of Load Rating

The load rating of a structure is determined by comparing the axial thrust and moment at each node of a structure, as calculated in the elastic frame analysis model of Chapter 5, to the axial thrust moment envelope developed in the previous section. The maximum stress to be used in Equation 6.1 can be determined by testing material removed from the structure or, in the absence of such tests, empirical values can be developed from the results of the present testing program. The moments and axial thrusts used in the analysis result from the application of a rating vehicle wheel load to a critical point in the structure, usually the crown. A rating factor of 1 is applied to the load factor formulas in the Manual for the Maintenance Inspection for Bridges (AASHTO, 1983). The load factors used are given in Table 6.1.

<table>
<thead>
<tr>
<th></th>
<th>Dead</th>
<th>Live</th>
<th>Impact</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inventory</td>
<td>1.3</td>
<td>1.3×1.667</td>
<td>0.3</td>
</tr>
<tr>
<td>Operating</td>
<td>1.3</td>
<td>1.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 6.1: Load factors for bridge rating.

Figures 6.2 through 6.5 show such plots for an H20 rating vehicle for the Mt. Vernon bridge, the Jones Road bridge, the Vincent Road bridge, and the Mussey Avenue bridge. In these trial plots, rather than using a fixed maximum compressive stress, contours for several different stress levels: 100 psi, 250 psi, 500 psi, and 1000 psi, are shown. The results of this analysis are briefly discussed below for each of the four structures.

6.4.1 Jones Road Bridge

This structure was observed to be in moderate-to-good condition. No real evidence of structural distress was noted in the field investigation of the structure, and the response of the structure is quite stiff. The ring thickness-to-span ratio is moderate at .066, and the fill appears to consist of very poor quality wet native silts and clays. No bulging of the spandrel walls, longitudinal cracks, or other commonly observable defects were apparent. The plot of a factored H20 wheel load at the 1/3 point shows all nodes within the safe zone for a peak stress of 250 psi.
The eccentricities are relatively high, as the cluster of points appears to be closer to the horizontal (moment) axis.

6.4.2 Mt. Vernon Bridge

This structure was observed to be in excellent condition, with no evidence of any structural problems, apart from the cracks at the abutment discussed in Chapter 4. The ring is relatively slender at a ring thickness/span ratio of .051. The fill is compacted gravel, and there is evidence that haunching was applied during the original construction. The bridge has just undergone a rehabilitation of the pile caps and cutwaters. The plot of a factored H20 wheel load at the 1/3 point shows. The calculated eccentricity at each joint is quite low and the thrust line is contained in the interior of the intersection diagram. The points are almost all contained within the envelope of safe points for a peak stress of 100 psi.

6.4.3 Vincent Road Bridge

This bridge is in poor condition, based on the observation of shifted stones in the spandrel walls and abutments, and loss of mortar and shifting of voussoirs inside the arch barrel. The Vincent Road bridge is a two-span structure, with a moderately thick arch ring (thickness/span ratio 0.067). The fill is a very poor native clayey silt. In spite of the relatively thick arch ring, an AASHTO H20 wheel load produces eccentricities greater than half the ring thickness, and over half the joints have similarly high eccentricities. Moreover, the stress level is relatively high for the weathered sandstone material of the structure, being well above the 250 psi contour at several joints. It is also noteworthy that this bridge underwent the largest crown displacements in the testing program. The peak displacement of 0.06 inches was over four times the peak displacement encountered in the Mussey Avenue bridge, described below, a two-span bridge with a span of 50 ft—over three times that of the Vincent Road bridge.

6.4.4 Mussey Ave. Bridge

The Mussey Avenue bridge consists of a double 50-ft span, with a 3-ft-thick arch ring, moderately thick at a thickness/span ratio of .06. The structure is in excellent condition.
The results of the frame analysis model show low eccentricities, and almost all of the moment/axial thrust coordinates contained within the safe envelope for a peak stress of 250 psi.

6.5 Observations and Conclusions

The foregoing results indicate that the elastic frame analysis of the structure, coupled with the construction of axial force/moment interaction curves and the plotting of frame analysis results on this interaction diagram, leads to useful information; this correlates very well to field observations. Three bridges in good condition were examined and were found to have reasonable eccentricities throughout the structure under an H20 rating vehicle, while the structure in poor condition was found to be deficient. It is recommended that the inventory and operating rating for ashlar masonry structures be established on the basis of the 500 psi contour, unless particularly low quality, weathered, or rubble masonry are present in the arch ring. Based on the rating engineer’s judgment, these values could be lowered for masonry appearing particularly weak, or raised, if justified by testing of samples of masonry from the structure. The joints where axial thrust moment coordinates are not contained in the 250 psi envelope, especially those with high eccentricities, indicate where weaknesses in the structure may exist, and where special inspection attention may be warranted. For instance, in the Mt. Vernon bridge, the abutment, where a crack has already been described, appears to be undergoing excessive moments under 1/3-point loading.
Figure 6.2: H20 inventory rating Jones Road bridge (rating factor=1.0).

Figures 6.2 through 6.5 illustrate the calculated axial forces and moments for various locations of an H20 axle compared to the interaction diagram for the given bridge.
Mount Vernon Bridge: Inventory Rating

Figure 6.3: H20 inventory rating. Mt. Vernon bridge (rating factor=1.0)
Figure 6.4: H20 operating rating: Vincent Road bridge (rating factor=1.0)
Figure 6.5 H20 inventory rating: Mussey Avenue bridge, Elyria (rating factor=1.0).
6.6 Example Load Rating: Plum Creek Bridge, Oberlin, OH (Lorain Co. File #4703278)

The proposed load rating procedure will be applied to the Oberlin bridge, a structure in fair condition located on a US highway, for which test results are available. The proposed load rating procedure is broken into the following steps.

- Determination of geometry of structure
- Assignment of section and material properties
- Determination of dead loads
- Placement of live loads; determination of member live loads
- Analysis of structure; determination of axial thrusts and bending moments
- Comparison of axial thrust and joint capacity
- Determination of inventory and operating rating

6.6.1 Determination of geometry and support conditions of structure

As shown in Table 3.1, the structure has a span of 20'-0", a rise of 8'-6", an angle of embrace of 161°, an intrados radius of 10'-2", and a ring thickness of 16". The arch is modeled as an elastic bar located at the midplane of the arch barrel. This bar has an angle of embrace of 161°, and a radius of 10'-10". The bar is divided into 10 equal straight segments, each with an included angle of 16.1°. The nodes will be assigned x-y coordinates, with the origin placed at the center of curvature of the arch barrel. The coordinates of the 11 nodes representing the arch ring are summarized in Table 6.2, and the procedure is illustrated in Figure 5.1.
Table 6.2: Nodal coordinates: Oberlin bridge.

<table>
<thead>
<tr>
<th>Node</th>
<th>x coordinate (inches)</th>
<th>y coordinate (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>128.2</td>
<td>21.4</td>
</tr>
<tr>
<td>2</td>
<td>117.2</td>
<td>56.2</td>
</tr>
<tr>
<td>3</td>
<td>97.1</td>
<td>86.5</td>
</tr>
<tr>
<td>4</td>
<td>69.3</td>
<td>110.0</td>
</tr>
<tr>
<td>5</td>
<td>36.0</td>
<td>124.9</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>130.0</td>
</tr>
<tr>
<td>7</td>
<td>-36.0</td>
<td>124.9</td>
</tr>
<tr>
<td>8</td>
<td>-69.3</td>
<td>110.0</td>
</tr>
<tr>
<td>9</td>
<td>-97.1</td>
<td>86.5</td>
</tr>
<tr>
<td>10</td>
<td>-117.2</td>
<td>56.2</td>
</tr>
<tr>
<td>11</td>
<td>-128.2</td>
<td>21.4</td>
</tr>
</tbody>
</table>

Members numbered 1 through 10 will be specified between the nodes: member 1 spans from node 1 to node 2, member 2 from node 2 to node 3 and so forth.

6.6.2 Assignment of section and material properties

For a 1-ft width of the structure, a cross sectional area of 192 in$^2$ and a moment of inertia of 4,096 in$^4$ are assigned. As a single-span bridge without measurements of the horizontal restraint at the supports, the supports will be modeled as fixed against rotation, with a horizontal spring support having a spring constant of 2,000 kips/in. Although the modulus of elasticity has been shown to have no influence on the rating of the bridge, a careful choice of $E$ will influence the comparison between the analysis and field testing results. This is a fairly loose-jointed limestone bridge. A value of E of 2,500 ksi will be selected.

6.6.3 Determination of dead loads

Dead loads of the arch ring are calculated by determining the area of the arch ring represented by each member and multiplying by a unit density of 160 lbs/ft$^3$. The area of a circular segment is given by the formula
\[ A = \alpha \left( R \pi + \frac{r^2}{2} \right) \]  

(6.2)

where \( \alpha \) is the angle of embrace of the segment in radians, \( R \) is the radius of the intrados, and \( r \) is the thickness of the segment. In this case, the angle of embrace of each of the segments is 16.1°, or 0.281 radians, hence the area is 4.06 \( \text{ft}^2 \) and the dead load of the ring segment is 0.65 kips.

The area of fill lying above each of the members and, consequently, the dead load of the fill, is given in the following table. Included in the dead load is an additional allowance of 20 psf for a future wearing surface.

<table>
<thead>
<tr>
<th>Element</th>
<th>Fill Area (( \text{ft}^2 ))</th>
<th>Dead Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,10</td>
<td>8.72</td>
<td>0.96</td>
</tr>
<tr>
<td>2,9</td>
<td>11.10</td>
<td>1.22</td>
</tr>
<tr>
<td>3,8</td>
<td>9.53</td>
<td>1.05</td>
</tr>
<tr>
<td>4,6</td>
<td>6.44</td>
<td>0.71</td>
</tr>
<tr>
<td>5,6</td>
<td>4.19</td>
<td>0.46</td>
</tr>
</tbody>
</table>

It is sufficiently accurate to take each the dead load as nodal loads at the end of each element. Thus, the following table shows the member dead loads for each of the members.

<table>
<thead>
<tr>
<th>Node</th>
<th>Dead Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,11</td>
<td>.47</td>
</tr>
<tr>
<td>2,10</td>
<td>1.06</td>
</tr>
<tr>
<td>3,9</td>
<td>1.1</td>
</tr>
<tr>
<td>4,8</td>
<td>0.83</td>
</tr>
<tr>
<td>5,7</td>
<td>0.54</td>
</tr>
<tr>
<td>6</td>
<td>0.41</td>
</tr>
</tbody>
</table>

6.6.4 Placement of live loads; determination of member live loads

An H20 wheel load of 16 kips must be placed on the structure at key points. The wheel load is converted to a pressure acting on the member, based on the depth of the point on the
member at which the wheel load acts. As an example, in table 6.5, a wheel load of 16 kips is
applied to an area 1-ft square located at x=-5.0 ft; that is, the quarter point of the span, 5.0 ft to
the left of the crown. Distributing the pressure through the fill at an angle of two vertical to one
horizontal results in a linearly varying pressure, partially on element 6 between nodes 6 and 7,
entirely on element 7, and partially on element 8. The pressure at any point is in relation to the
depth of the element

\[ p = \frac{W}{10(d + 1)^2} \]  

(6.3)

where \( p \) is the pressure, \( W \) is the axle load, and \( d \) is the depth of the fill at the point in question. It
is sufficiently accurate to calculate the nodal pressures at each node under the influence of an axle
and to vary the pressure linearly from node to node. The calculation of the partially distributed,
uniformly varying loads to be placed on the elements of the arch is displayed in Table 6.5.

6.6.5 Analysis of structure; determination of axial thrusts and bending moments

The application of the Inventory Rating load factors shown in Table 6.1 to the dead loads
and live loads calculated above results in the nodal axial thrusts and bending moments shown in
Table 6.6.

| Table 6.5: Live load pressures \( x=-5 \) (H20 axle). |
|-----------------|---------------|---------------|
| Node | Depth | Pressure |                |               |
| 1 | 10.7 | \( x=-5 \) | 0 | 0 | 0 |
| 2 | 7.8 | \( x=-3 \) | 0 | 0 | 0 |
| 3 | 5.3 | \( x=0 \) | 0 | 0 | 0 |
| 4 | 3.4 | \( x=-5 \) | .51 | 0 | 0 |
| 5 | 2.1 | \( x=-3 \) | .73 | 1.03 | 0 |
| 6 | 1.7 | \( x=0 \) | 0 | 1.19 | 0 |
Table 6.6: Axial thrusts and moments: Oberlin bridge H20 inventory rating.

<table>
<thead>
<tr>
<th>Element</th>
<th>x=-5</th>
<th>x=-3</th>
<th>x=0</th>
<th>x=-0</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Thrust</td>
<td>Moment</td>
<td>Thrust</td>
<td>Moment</td>
</tr>
<tr>
<td>1</td>
<td>15.67</td>
<td>1.59</td>
<td>15.86</td>
<td>-5.2</td>
</tr>
<tr>
<td>2</td>
<td>13.49</td>
<td>5.2</td>
<td>14.31</td>
<td>5.24</td>
</tr>
<tr>
<td>3</td>
<td>10.07</td>
<td>0.17</td>
<td>12.28</td>
<td>5.84</td>
</tr>
<tr>
<td>4</td>
<td>6.73</td>
<td>-4.58</td>
<td>10.37</td>
<td>0.09</td>
</tr>
<tr>
<td>5</td>
<td>5.69</td>
<td>-3.23</td>
<td>8.32</td>
<td>-7.98</td>
</tr>
<tr>
<td>6</td>
<td>5.8</td>
<td>-0.5</td>
<td>8.26</td>
<td>-6.1</td>
</tr>
<tr>
<td>7</td>
<td>6.27</td>
<td>1.56</td>
<td>9.12</td>
<td>1.16</td>
</tr>
<tr>
<td>8</td>
<td>7.28</td>
<td>2.91</td>
<td>10.28</td>
<td>5.46</td>
</tr>
<tr>
<td>9</td>
<td>8.82</td>
<td>3.1</td>
<td>11.75</td>
<td>6.04</td>
</tr>
<tr>
<td>10</td>
<td>10.31</td>
<td>0.86</td>
<td>12.92</td>
<td>1.68</td>
</tr>
</tbody>
</table>

Figure 6.6 shows the values of thrust and moment shown in Table 6.6 plotted on a joint resistance diagram for H20 Inventory Rating, with a rating factor of 1. The resistance envelope is exceeded at the back of the arch ring, especially for crown loading, but the positive moment resistance envelope is not exceeded. Because of the additional resistance of the fill and spandrels, and because the eccentricity does not exceed the combined depth of the fill at the crown plus half of the arch ring, the structure qualifies for at least an H20 rating. On the other hand, for the crown loading, the point corresponding to one of the members is on the 500 psi envelope. Any increase in the live load would result in an increase in the bending moment and in the positive moment envelope being exceeded. Thus, this structure has an inventory rating of H20. Since the H20 live loads for an inventory rating factor of 1.0 and an operating rating factor of 1.67 are identical, the operating rating of this structure is H33.
Figure 6.6: H20 inventory rating, Oberlin bridge (rating factor=1.0).
6.7 Summary of Proposed Load Rating Procedure for Masonry Arch Bridges

The load rating procedure is developed for a unit (one foot) width of the bridge structure. The geometric properties of a one foot width must be determined by field measurements, or examination of plans, if available. The depth of the arch ring is used to find the area \( A \) and moment of inertia \( I \) of a one foot width of the arch barrel by the formulas

\[
A = bh \\
I = \frac{bh^3}{12}
\]  \hspace{1cm} (6.4)

where \( b \) is the width, taken as 12” and \( h \) is the depth of the arch ring.

In the absence of testing, the modulus of elasticity \( E \) and the ultimate strength of the masonry \( \sigma_0 \) shall be assumed on the basis of the following table

**Table 6.7 Modulus of elasticity and ultimate strength of masonry**

<table>
<thead>
<tr>
<th></th>
<th>Modulus of Elasticity ( E ) (ksi)</th>
<th>Ultimate strength ( \sigma_0 ) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sound, cut sandstone/limestone Joints less than 1/2”</td>
<td>2500</td>
<td>500</td>
</tr>
<tr>
<td>Cut stone: deteriorated or with joints greater than 1/2”</td>
<td>1000</td>
<td>250</td>
</tr>
<tr>
<td>Rubble masonry or severely deteriorated cut stone</td>
<td>500</td>
<td>100</td>
</tr>
</tbody>
</table>

The strength interaction diagram between axial thrust and moment shown in Figure 6.1 will be used for the rating of the structure, based on the stresses given in the table above. The interaction diagram is non-dimensionalized, and can be adapted to any given structure by multiplying the horizontal axis by the ultimate axial force \( P_0 \) and the vertical axis by the ultimate moment \( M_0 \). where

\[
P_0 = bh\sigma_0 \\
M_0 = 0.125bh^2\sigma_0
\]  \hspace{1cm} (6.6)
The moment of inertia and cross sectional area, determined above, shall be used as section properties of an equivalent system of elastic bars, described in Section 5.3, with an example given in Section 6.5.1. The centerline of the arch ring is to be divided into at least 10 segments. Each segment shall be represented by a straight bar with \( E, A, \) and \( I \) determined on the basis of a unit width of the arch ring. The resulting plane frame system shall be fixed at the supports in the vertical direction and shall be considered fixed for bending moments. However, in the horizontal direction, elastic restraint, representing the elastic restraint of the abutments, shall be provided. The spring constant for the elastic restraint shall be determined on the basis of Table 6.8 (Identical to Table 5.1).

**Table 6.8: Elastic Constants for Frame Analysis**

<table>
<thead>
<tr>
<th>Condition</th>
<th>Abutment</th>
<th>Interior Pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good quality, sound masonry</td>
<td>5000 k/in</td>
<td>2500 k/in</td>
</tr>
<tr>
<td>Weak masonry</td>
<td>1000 k/in</td>
<td>500 k/in</td>
</tr>
</tbody>
</table>

The dead load of the arch ring shall be computed, using a unit density of 160 lbs/ft\(^3\). The dead load of the fill and road surface shall be computed, using a unit density of 110 lbs/ft\(^3\). The rating axle load shall be applied to an area of 10 feet wide and one foot long of the deck of the structure, and distributed lengthwise through the fill at a slope of 1 horizontal to 2 vertical as in Figure 5.2. Several positions of the main axle of the rating vehicle shall be investigated, including at least the crown and the 1/4 point of the span.

The load factors to be investigated are given in Table 6.1. In this procedure, a separate investigation must be conducted for each combination of load factors and rating factors. Initially, the inventory and operating ratings should be checked separately with a rating factor of 1.0. If the structure fails to meet the desired rating by the criteria given below, then it should be re-checked with a lower rating factor, until the strength criteria are satisfied. For a given set of load factors and rating factor, and a given position of the rating vehicle, the output from the frame analysis program will give a compressive axial force and bending moment at each of the nodes of the frame representing the arch ring. The combinations of axial force and bending moment are

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plotted on the interaction diagram representing the strength of the arch ring, whose construction is described above. The "safe" zone in the interaction diagram is the interior.

The strength of the structure is acceptable for a given combination of load factors and rating factors if all of the points corresponding to axial thrust and moment are contained within the interior of the interaction diagram. Because of the additional strength furnished to the structure by the spandrel walls and fill, the bending moment capacity of the arch ring may also be safely exceeded if the moments cause compression on the back of the arch. In such a case, the points representing the calculated axial thrust and moment may safely lie outside the interaction envelope for negative bending moments. This can be seen in Figure 6.6, for example, where four of the points for the x=-5 axle position lie outside the envelope on the left side of the vertical axis, and none on the right side. This condition may be acceptable, provided the eccentricity is checked at each point where this occurs. The eccentricity is calculated from the moment $M$ and the axial force $P$ by the following formula

$$e = \frac{M}{P}$$

(6.7)

For the structure to satisfy the strength criterion, any eccentricities that exceed 1/2 the depth of the arch ring must be

a) negative, that is towards the outside of the arch ring
b) less than the combined depth of 1/2 the arch ring and the fill over the crown
c) less than the combined depth of 1/2 the arch ring and one foot.

For example, for the Oberlin bridge, for which the interaction diagram is shown in Figure 6.6, the four points outside the interaction diagram all have negative eccentricity, satisfying (a) above. The maximum eccentricity is 12 ft-k/10 ft=1.2 ft. For this structure, the combined depth of 1/2 the arch ring and the fill over the crown is 1'8", or 1.67 ft, so both (b) and (c) are satisfied. On the other hand, interaction diagrams for the Vincent Rd. bridge are shown in Figure 6.4. Four this structure, five interaction points lie outside the interaction diagram, including one with eccentricity towards the inside of the arch ring. Moreover, the eccentricity of the other points exceeds the combined depth of 1/2 the arch ring and the fill over the crown of 0.5 feet.
The inventory and operating ratings of the structure are based on the largest rating factor, for which the axial thrusts and moments calculated by the frame analysis procedure satisfy all the above criteria for the estimated strength. Structures that do not meet the criteria for the desired inventory and operating rating may be given a lower rating, or testing may be undertaken to determine material properties, such as ultimate strength of the masonry or horizontal stiffness at the abutment more accurately.
7. ACKNOWLEDGMENTS

The authors would like to acknowledge the excellent cooperation they received from personnel at all levels of the Ohio Department of Transportation. In particular, they would like to recognize the assistance of Vik Dalal, Karen Young, Cindy Wengerter, and Mark Rafeld.
8. BIBLIOGRAPHY


APPENDIX A

CEMETERY ROAD BRIDGE
TEST RESULTS

The horizontal axis in these graphs represents the truck position in inches. For the half-span tests, the 0 position is the crown of the arch. Negative values represent the distance of the rear axle (or the centerline of the rear tandem) from the crown for the truck coming on to the bridge, while positive values represent the same distance for the truck leaving the bridge. For the full-span tests, the 0 position is off the bridge. Negative values represent the distance to the zero position for the truck crossing the bridge and approaching the 0 position, and positive values represent the truck going in the other direction.

The vertical displacement represents the displacement in inches at the named transducer. The locations of the transducers are shown in Table 3.2 and Figure 3.2.
Figure A.1  Half Load, Left Edge, Half Span
Figure A.2  Half Load, Left Edge, Full Span
Figure A.3  Half Load, Right Edge, Half Span
Figure A.4  Half Load, Right Edge, Full Span
Figure A.5  Full Load, Left Edge, Half Span
Figure A.6 Full Load, Left Edge, Full Span
Figure A7  Full Load, Right Edge, Half Span
APPENDIX B

JONES ROAD BRIDGE
TEST RESULTS

The horizontal axis in these graphs represents the truck position in inches. For the half-span tests, the 0 position is the crown of the arch. Negative values represent the distance of the rear axle (or the centerline of the rear tandem) from the crown for the truck coming on to the bridge, while positive values represent the same distance for the truck leaving the bridge. For the full-span tests, the 0 position is off the bridge. Negative values represent the distance to the zero position for the truck crossing the bridge and approaching the 0 position, and positive values represent the truck going in the other direction.

The vertical displacement represents the displacement in inches at the named transducer. The locations of the transducers are shown in Table 3.2 and Figure 3.2.
Figure B.1  Half Load, Half Span
Figure B.2  Half Load, Full Span
Figure B.3  Full Load, Half Span
Figure B.4 Full Load, Full Span
APPENDIX C

OBERLIN BRIDGE
TEST RESULTS

The horizontal axis in these graphs represents the truck position in inches. For the half-span tests, the 0 position is the crown of the arch. Negative values represent the distance of the rear axle (or the centerline of the rear tandem) from the crown for the truck coming on to the bridge, while positive values represent the same distance for the truck leaving the bridge. For the full-span tests, the 0 position is off the bridge. Negative values represent the distance to the zero position for the truck crossing the bridge and approaching the 0 position, and positive values represent the truck going in the other direction.

The vertical displacement represents the displacement in inches at the named transducer. The locations of the transducers are shown in Table 3.2 and Figure 3.2
Figure C.1  Half Load, Center of Road, Half Span
Figure C.2  Half Load, Center of Road, Full Span
Figure C.3  Half Load, Left Edge, Half Span
Figure C.4  Half Load, Left Edge, Full Span
Figure C.5  Full Load, Center of Road, Half Span
Figure C.6 Full Load, Center of Road, Full Span
Figure C.7 Full Load, Left Edge, Half Span
Figure C.8  Full Load, Left Edge, Full Span
APPENDIX D

MT. VERNON BRIDGE TEST RESULTS

The horizontal axis in these graphs represents the truck position in inches, measured to the centerline of the tandem rear axle. All tests were done on a single span with the truck crossing the span onto the bridge. The 0 position represents the crown of the arch of the first span of the bridge. Negative values represent the truck on the abutment side of the crown, while positive values represent the truck approaching the first interior pier.

The vertical displacement represents the displacement in inches at the named transducer. The locations of the transducers are shown in Tables 3.2 and 3.3 and Figure 3.2.
Figure D.1  One Truck, Right Lane (cont. on next page)
Figure D.1 (cont.)
Figure D.2 One Truck, Straddling Line (cont. on next page)
Figure D.2 (cont.)
Figure D.3  One Truck, Second Lane (cont. on next page)
Figure D.3 (cont.)
APPENDIX E

VINCENT ROAD BRIDGE TEST RESULTS

These graphs represent the deflected profile of the arches of the two spans of the Vincent Road Bridge for a series of truck locations. The horizontal axis represents the position (in inches) of the transducer measured along the intrados of the arch, with the 0 position being the centerline of the interior pier. The vertical axis represents the displacement at each transducer for the truck position noted in the legend. The truck position is also measured in inches from the centerline of the pier.
Vincent Road Bridge Profile
Half Loaded Truck (x=-114)

Displacement (in)

LVDT Location (in)
Vincent Road Bridge Profile
Half Loaded Truck (x=0)
Vincent Road Bridge Profile
Half Loaded Truck (x=+114)
Vincent Road Bridge Profile
Half Loaded Truck (x=+159)
Vincent Road Bridge Profile
Fully Loaded Truck (x=-114)

Displacement (in)

LVDT Location (in)
Vincent Road Bridge Profile
Fully Loaded Truck (x=+69)
Vincent Road Bridge Profile
Fully Loaded Truck (x=+114)

LVDT Location (in)
Displacement (in)
Vincent Road Bridge Profile
Fully Loaded Truck (x=+159)

Displacement (in)

LVDT Location (in)

df

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APPENDIX F

ELYRIA BRIDGE (MUSSEY AVE.)
TEST RESULTS

These graphs represent the deflected profile of the arches of the two spans of the Mussey Ave. Bridge in Elyria for a series of truck locations. The horizontal axis represents the position (in inches) of the transducer measure along the intrados of the arch, with the 0 position being the centerline of the interior pier. The vertical axis represents the displacement at each transducer for the truck position noted in the legend. The truck position is also measured in inches from the centerline of the pier.
Mussey Ave. Bridge Profile
Left Lane Truck (All Positions)
Mussey Ave. Bridge Profile
Left Lane Truck ($x=-300$)
Mussey Ave. Bridge Profile
Left Lane Truck (x=0)
Mussey Ave. Bridge Profile
Left Lane Truck (x=+546)
Mussey Ave. Bridge Profile
Right Lane Truck (x=-150)
Mussey Ave. Bridge Profile
Right Lane Truck (x=+300)

Displacement (in)

LVDT Location (in)
Mussey Ave. Bridge Profile
Right Lane Truck (x=+546)
Mussey Ave. Bridge Profile
Right Lane Truck (x=+588)
Mussey Ave. Bridge Profile
Two Trucks (All Positions)
Mussey Ave. Bridge Profile
Two Trucks (x=+150)
Mussey Ave. Bridge Profile
Two Trucks (x=±300)

Displacement (in)
LVDT Location (in)
Mussey Ave. Bridge Profile
Two Trucks (x=+348)
Mussey Ave. Bridge Profile
Two Trucks (x=+546)
Mussey Ave. Bridge Profile
Two Trucks (x=+696)
Mussey Ave. Bridge Profile
Two Trucks (x=+996)