Objective Condition Assessment of Damaged or Deteriorated Bridges in Ohio

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for the Ohio Department of Transportation Office of Research and Development

and the Federal Highway Administration

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Foreword

For more than two decades, the University of Cincinnati Infrastructure Institute (UCII) has been studying a wide variety of issues connected with field instrumentation, structural identification, condition assessment and health monitoring associated with highway bridges. A particular focus has been the development of a nondestructive test and evaluation based strategy based on the use of field test methods (including crawl speed truckload testing and modal testing) combined with finite element modeling and model calibration. The overarching goal for this activity has been to develop methods which provide objective, reliable, accurate, qualitative, and practical instrumentation and field test measurement techniques, coupled with supporting analytical methods, to aid bridge engineers with a wide variety of decisions faced in bridge inventory maintenance and management including rating, rehabilitation, retrofitting, use of new materials, overload permitting and the like. The research project findings discussed in this report are the latest in long line of projects in this area. This work builds mainly on two previous UCII projects which served to lay the foundations of instrumentation and field testing:


A companion project enhanced, extended, and automated these techniques and methods as well as used them to assemble a data set of 30 bridge field tests conducted on “healthy” steel stringer bridges around the state of Ohio. The current project conducted in parallel with this research effort applied these same techniques to 10 low-rated steel stringer structures in Ohio.
Taken together, these two projects enabled both the development and validation of modal testing, truckload testing, and calibrated finite element modeling techniques as well as the assembly of a statistical database of findings for 40 structures. The research findings of this companion project are collected in a separate report:

- Helmicki, A., Hunt, V., Swanson, J. Bridge Type Specific Management of Steel Stringer Bridges: Development of Field Calibrated Software Rating Tools and Statistical Bridge Database, University of Cincinnati Infrastructure Institute Research Report, Cincinnati, OH, 2005.

Other UCII projects ongoing during the timeframe of these companion projects are closely coupled in terms of the types of structural health monitoring and condition assessment issues being addressed. These include:


This report is organized as follows: Chapter 1 provides project introduction and background, including discussions of prior projects and current project objectives. Chapter 2
describes the bridge test specimen selection process, provides a listing of the bridges field tested, and provides summaries of each structure. The multi-reference impact modal testing hardware, operating procedures, post-processing methods and software employed are presented in Chapter 3. The truckload testing hardware, operating procedures, post-processing methods and software employed are presented in Chapter 4. Chapter 5 describes the key theoretical underpinnings of the structural identification strategy developed by which the load capacity rating of a bridge can be derived from either finite element model simulation results or field test measurements. Chapter 6 addresses the methods and software developed for constructing 3D finite element models of steel stringer bridges. Finite element model calibration techniques and software are discussed in Chapter 7. Chapter 8 presents findings from this research that address the conservatism of current AASHTO distribution factor formulas, the impact of this conservatism on the rating process, and adjustment factors which can be applied to AASHTO distribution factors to reduce conservatism providing distribution factor estimates closer to those observed from field testing. Summary and conclusions are given in Chapter 9, and an implementation plan for further studies is presented in Chapter 10. A set of appendices provides succinct summaries of all field tests, finite element models and calibrations, and rating studies conducted for each of the 10 specimens considered in this research project.

Note that much of this work has been documented to a greater extent in the final report for the companion project, *Bridge Type Specific Management of Steel Stringer Bridges: Development of Field Calibrated Software Rating Tools and Statistical Bridge Database*. By definition, the companion project enhanced, extended, and automated techniques and methods as well as used them to assemble a data set of 30 bridge field tests. This project and its final report will focus primarily upon documentation of the test, tuning, and rating results for the 10 selected structures which are either posted or simply low-rated steel stringers in Ohio. Conclusions will be drawn specifically for each structure and its rating, as well as both general and statistical comparisons between these structures and as compared to their design and inspected condition.
Objective Condition Assessment of Damaged or Deteriorated Bridges in Ohio

Problem

The Ohio Department of Transportation (ODOT) currently maintains an inventory of 14,957 bridges across the state. Of these, by far the largest proportion, 6,335, are steel girder/beam bridges with reinforced concrete decks.

In accordance with the National Bridge Inspection Standards (NBIS), the ODOT Office of Structural Engineering is tasked with the responsibility of rating these structures in order to determine their safe live load carrying capacity. ODOT Procedure No. 518-001(P) outlines the standard operating procedures connected with this rating process which is comprised of three main components:

1. Bridge Records and Inventory Data consists of a wide range of information about the bridge and its past history including: location, year built, structural description, skew, spans, length, width, average daily truck traffic, design load, plans and dimensions, etc.

2. Bridge Inspection Data and Report: As per the terms of the Ohio Revised Code, each structure must be visually inspected at routine intervals not to exceed one year in order to determine the physical and functional condition of the bridge. Each inspection examines the overall structural components of the bridge, and any defects found are documented, tracked and investigated to evaluate their causes, extent, and any propagation characteristics.

3. Load Rating Calculations and Report: Using the inputs provided from items 1 and 2 above, standard engineering calculations are used to evaluate the safe load carrying capacity of the structure.
This rating is determined in three ways: Inventory ratings refer to the stresses at the design level and reflect the normal operating conditions/limits of the bridge. Operating rating refer to the maximum possible live load to which the structure may be subjected without loss of safe operation. These 2 sets of ratings are given in terms an AASHTO standard HS20-44 load vehicle configuration. Operating ratings in terms of Ohio legal loads refer to the maximum operating limits stated in terms of standardized Ohio load vehicle configurations (2F1, 3F1, 4F1, and 5C1).

In Ohio, the preferred procedure for determining these ratings for the class of steel beam/girder bridges is through the use of the BARS software package which implements Load Factor Rating (LF) or Allowable Stress Rating (AS) calculations where in the basic rating equation is given by:

\[ RF = \frac{C - A_1 D}{A_2 L(1 + I)} \]

where \( RF \) denotes the rating factor for the live load carrying capacity, \( C \) denotes the capacity of the member, \( D \) denotes the dead load effect on the member, \( L \) denotes the live load effect on the member, and \( I \) denotes the impact factor.

Each of these values is obtained by the rating engineer using the information contained in either the Bridge Record, the AASHTO Standard Specifications, or a combination thereof. Further modification to these parameters is at the discretion of the rating engineer and is subject to the findings contained in the Bridge Inspection Report. The constants \( A_1 \) and \( A_2 \) vary depending on whether LF or AS methods are used and whether Inventory or Operating ratings are being calculated.

When the operating rating of a bridge is determined to be less than 100% of the legal load for which the bridge was designed, the structure must be posted to a lower load limit to ensure safe use by the general traveling public. At this writing, ODOT currently maintains posted ratings (i.e., less than 100% of legal load limits) on 80 bridges in Ohio, of which 21 are steel girder/beam bridges, with the remainder of the inventory being rated between 100% and 150%. In addition, ODOT spends approximately $235M annually on the operation, maintenance, rehab/retrofit, and new construction of the bridges in its inventory.

The difficulty with this situation is that the process described above is known to be uncertain, at best, owing to the fact that the ratings obtained are based on assumed or adjusted physical parameters and/or engineering decisions that may or may not be representative of the actual state of structure in the field. The past decade’s worth of field test effort expended by the University of Cincinnati Infrastructure Institute (UCII), under the auspices of ODOT, FHWA, NSF and others, has provided extensive evidence based on quantitative field measurements and 3D FE modeling capabilities that can be used to calibrate and/or validate some of the assumptions/parameters used in conducting the rating process described above.

Field test-based rating data is able to objectively identify and characterize in-situ structural response mechanisms, thus giving bridge engineers a more thorough description of existing condition and providing the basis for more sound and realistic load ratings. These identified mechanisms include the actual load distribution, unintended composite action, participation of superimposed deadload, material properties, unintended continuity, participation of secondary members, effects of skew, effects of deterioration and damage, unintended bearing restraint, and environmental effects such as thermal stresses.

**Objectives**

The main objective of this research is to non-destructively field test and evaluate ten (10) damaged/deteriorated, low-rated bridges in order to determine load carrying capacity. All test bridges were selected from the Ohio state inventory in conjunction with input from the ODOT Office of Structural Engineering. A detailed field test plan was developed for each test bridge and included modal testing, truckload testing, and calibrated FE modeling in order to derive accurate, objective load ratings.

A secondary objective was to further streamline the field testing and assessment procedures. Specifically, the actual time for a 32-channel modal
impact test and a 24-channel truckload test of a typical 3-span, 2 lane, 250 foot steel-stringer bridge was reduced to less than one workday for each test. This included test setup and teardown.

The benefits of this research to ODOT have been the development of techniques, equipment packages, and a personnel team with the expertise necessary to implement rapid, rigorous, quantitative assessments of critical structures around the state. In addition to these procedures and expertise, deliverables included detailed reports of findings for each of the bridges in the test set.

As a by-product of our previous sponsored research projects, several pieces of PC based software have been developed which permit fast, user-friendly post-processing of field test data, FE modeling, model calibration, and load rating calculation. These allowed impact modal testing, including field operations and data post-processing to be completed with a 2-3 day period, total. Similarly for truckload testing.

Concurrent with the field test operations and as data became available, the preliminary finite element model of each test bridge was calibrated using previously developed software and the SAP 3D finite element model of each bridge.

Load ratings for a test bridge were computed by all three methods outlined above and compared. All resulting values for load-rating were subsequently added to the test bridge database.

When testing and data post-processing of an individual bridge was completed, a report of the findings specific to that bridge was generated. These individual bridge reports summarize:
nondestructive field test methods utilized on that particular bridge, the corresponding test results, analytical model generation and model calibration with field test results, the subsequent field-test-based load-rating, and an analysis of how this field-test-based load-rating compares to the corresponding ODOT documented load-rating.

Conclusions & Recommendations

This project employed both multiple impact reference modal test and crawl speed truckload test methods as its primary experimental field test tools. The research enhanced and extended these methods in several ways: (1) test procedures were standardized across all tests so that similar data sets were obtained for each bridge, (2) test procedures were streamlined so that the data collection portion of the test would not exceed one hour and the total testing time was less than 8 hours per bridge, (3) a series of electronic, mechanical and signal processing techniques were developed to maximize data quality, and (4) test procedures, including pretest logistics, were configured so that they were highly mobile and rapidly deployable.

The research also demonstrated conclusively that one conceptual signature that represents bridge condition and can be determined from either a truckload test or an FE simulation is the unit influence line. The derived influence line was shown to be consistent for various truckloads and axle configurations. From the influence line, several conclusions which directly impact ratings can be immediately drawn regarding the structural condition including the level and consistency of (unintended) composite action, lateral distribution, longitudinal balance of response maxima, impact factor, end restraint, edge stiffening, linearity, and stationarity under load.

Finally, when compared against analytically derived ratings, field test-based ratings yielded a number of interesting observations: (1) for AS truckload ratings governed over lane loads more than 90% of the time, (2) for LF overload ratings governs about 90% of the time, (3) field test based ratings yielded significantly less conservative ratings across the board: on an average there was an improvement of 97.5% in the Rating Factors from BARS to Tuned for AS, 93.04% for LF, 89.75% for OL. On average, the BARS based ratings were 1.01 for AS and 1.17 for LF. The corresponding ratings calculated from field test data tuned FE models were 1.99 for AS and 2.29 for LF. (4) Rating factors obtained by the various methods have been found to follow a pattern: if experimental rating factors obtained by AS for a particular gage location were found to have a minimum at a particular gage location, then experimental rating factors obtained by LF and overload inventory were also lowest at that gage, and (5) all of the non-compositely designed bridges showed some level of remaining composite action despite their deterioration noted on their inspection reports and their posted states.

Implementation Potential

Lessons learned regarding field procedures have led to further streamlining of testing and post-processing methods. These have resulted in a rapidly deployable field test and evaluation capability in which bridges can be modeled, tested, data post-processed, and objective ratings established all within a few days. These results can provide bridge engineers with useful, practical tools for assessment of specific bridges, load rating for overload passages, evaluation of retrofits, etc.

The field methods explored in this research also revealed a number of interesting insights into the behavior of posted and other low rated stringer bridges in the Ohio inventory. In particular, the sample population of bridges tested revealed that factors such as unintended composite action can go unnoticed in visual inspections and yet have a great impact on the structural capacity rating of the structure.

The examination of the data obtained in the testing of these 10 low-rated structures could be combined with the data obtained from other bridge tests (more than 30 other cases exist) to form a database of objective field tests measurements for steel stringer bridge across the state. This database, and the unexplored knowledge it still contains, can be cultivated further to reveal other aspects of steel stringer bridge behavior as well as to help optimize future bridge design and maintenance practices. We expect the data from this research to impact the AASHTO design and evaluation provisions pertaining to stringer bridges.

Finally, the results and methods of this study could be employed to achieve similar gains for other bridge types (such as concrete bridges) which are prevalent in the Ohio bridge inventory.
Chapter 1 Introduction, Background, and Objectives

1.1 INTRODUCTION

1.1.1 Synopsis of the 10 Bridge Project Research

The Ohio Department of Transportation (ODOT) currently maintains an inventory of 14,957 bridges across the state. Of these, by far the largest proportion, 6,335, are steel girder/beam bridges with reinforced concrete decks.

In accordance with the National Bridge Inspection Standards (NBIS), the ODOT Office of Structural Engineering is tasked with the responsibility of rating these structures in order to determine their safe live load carrying capacity. ODOT Procedure No. 518-001(P) outlines the standard operating procedures connected with this rating process which is comprised of three main components [ODOT 1999]:

1. Bridge Records and Inventory Data: consists of a wide range of information about the bridge and its past history including: location, year built, structural description, skew, spans, length, width, average daily truck traffic, design load, plans and dimensions, etc. Additional documentation such as drawings, specifications, photographs, material test data, maintenance and repair history, accident records, posting history, and permit load data may also be available depending on the age and size of the structure.

2. Bridge Inspection Data and Report: As per the terms of the NBIS, each structure must be visually inspected at routine intervals not to exceed two years in order to determine the physical and functional condition of the bridge. More detailed inspections may be necessary to assess structural damage resulting from environmental factors (e.g., flood, earthquake, etc.) or human actions (e.g., traffic accident, etc.) or to identify deficiencies not readily detectable using routine inspection procedures (e.g., deterioration below water lines, etc.). Each inspection examines the overall structural components of the bridge, and defects found are investigated to evaluate their causes, extent, and any propagation characteristics. In addition, field
measurements of defects (i.e., crack size/location, section loss, etc.) are made to provide baseline
data to both track deterioration over time and to conduct load rating calculations.

3. Load Rating Calculations and Report: Using the inputs provided from items 1 and 2, standard engineering calculations [AASHTO 1995-2005; BARS 1995] are used to evaluate the safe load carrying capacity of the structure. This rating is determined in three ways:

- Inventory Rating in terms of AASHTO HS Loading: refer to the stresses at the design level and reflect the normal operating conditions/limits of the bridge. These ratings are given in terms of an AASHTO standard HS20-44 load vehicle configuration.

- Operating Rating in terms of AASHTO HS Loading: refer to the maximum possible live load to which the structure may be subjected without loss of safe operation. These ratings are also given in terms an AASHTO standard HS20-44 load vehicle configuration.

- Operating Rating in terms of Ohio Legal Loads: refers to the maximum operating limits stated in terms of standardized Ohio load vehicle configurations (2F1, 3F1, 4F1, and 5C1).

In Ohio, the preferred procedure for determining these ratings for the class of steel beam/girder bridges is through the use of the BARS software package [BARS 1995] which implements Load Factor Rating (LF) or Allowable Stress Rating (AS) calculations.

In either event, the basic rating equation is given by:

\[
RF = \frac{C - A_L D}{A_L D (1 + I)}
\]  

(1-1)

where

- \( RF \) denotes the rating factor for the live load carrying capacity,
- \( C \) denotes the capacity of the member (depending upon the rating method),
- \( D \) denotes the dead load effect on the member,
- \( L \) denotes the live load effect on the member, and
- \( I \) denotes the impact factor.
Each of these values is obtained by the rating engineer using the information contained in either the Bridge Record or the AASHTO Standard Specifications [AASHTO 2002], or a combination thereof. Further modification to these parameters is at the discretion of the rating engineer and is subject to the findings contained in the Bridge Inspection Report. The constants \( A_1 \) and \( A_2 \) vary depending on whether LF or AS methods are used and whether Inventory or Operating ratings are being calculated.

Using these procedures, ODOT Office of Structural Engineering currently maintains records and ratings on each of the bridges in the Ohio inventory. These records and ratings must be kept up-to-date and must be revised if there is any physical change in the condition of the structure. They are used both to set the safe operating limits of each structure in the inventory and to manage budgetary, personnel, and equipment resources for the purposes of maintenance, rehabilitation, and replacement of these structures.

When the operating rating of a bridge is determined to be less than 100% of the legal load for which the bridge was rated, the structure must be posted to a lower load limit to ensure safe use by the general traveling public. At this writing, ODOT currently maintains posted ratings (i.e., less than 100% of legal load limits) on 80 bridges in Ohio, of which 21 are steel girder/beam bridges, with the remainder of the inventory being rated between 100% and 150% [Moore 2000]. In addition, ODOT spends approximately $235 Million annually on the operation, maintenance, rehab/retrofit, and new construction of the bridges in its inventory [ODOT 1998].

The difficulty with this situation is that the process described above is known to be uncertain, at best, owing to the fact that the ratings obtained are based on assumed or adjusted physical parameters and/or engineering decisions that may or may not be representative of the actual state of structure in the field. The past decade’s worth of field test effort expended by the University of Cincinnati Infrastructure Institute (UCII), under the auspices of ODOT, FHWA, NSF and others, has provided extensive evidence based on quantitative field measurements that call into question some of the assumptions/parameters used in conducting the rating process described above (see Table 1.1). Consequently, such ratings may lead to overly restrictive and costly results in terms of posted traffic load, extensive retrofit/rehab, or the unnecessary complete replacement of structures.
### Table 1-1: Comparison of Analytical and Field Testing Approaches

<table>
<thead>
<tr>
<th></th>
<th><strong>ANALYTICAL ASSESSMENT</strong></th>
<th><strong>FIELD TESTING</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Live and Wind Load</strong></td>
<td>Can only analytically estimate stress levels, magnitude of vibrations, and magnitude of vibration-induced stresses.</td>
<td>Obtain actual stress/strain levels due to respective loads/vibrations can be monitored at mounted sensor locations.</td>
</tr>
<tr>
<td><strong>Induced Vibrations</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Thermally Induced Stresses</strong></td>
<td>Cannot evaluate thermal stress levels nor variations in these levels.</td>
<td>Obtain actual thermal stress/strain levels and variations can be monitored at mounted sensor locations.</td>
</tr>
<tr>
<td><strong>Connection Integrity</strong></td>
<td>Section loss, quantity of rust, bolt integrity, rivet integrity, weld integrity, pin/hinge integrity all subjectively evaluated through visual observations. Cannot evaluate force transfer through the connection.</td>
<td>Sensors mounted on connection plates, pins, and hinges can be used to evaluate how effectively force/load is transferred through the connection as well as what the stress levels are within critical connection components.</td>
</tr>
<tr>
<td><strong>Structural Member Rating</strong></td>
<td>Interpretation of visually observed section loss, rust, cracking, etc. used to evaluate load-carrying ability of structural member.</td>
<td>Field measurements reflect true load-carrying ability of structural member. This information used to evaluate capacity of member as well as live load effects on member.</td>
</tr>
</tbody>
</table>

In short, field test-based rating data, otherwise referred to as objective data, is able to identify and characterize in-situ structural response mechanisms, thus giving bridge engineers a more thorough description of existing condition and providing the basis for more sound and realistic load ratings. These identified mechanisms include the actual load distribution, unintended composite action, participation of superimposed deadload, material properties, unintended continuity, participation of secondary members, effects of skew, effects of deterioration and damage, unintended bearing restraint, and environmental effects such as thermal stresses. More detailed discussions of field test findings and comparisons between analytically (i.e., BARS) generated and field-test based ratings can be found in publications connected with past UCII projects [Levi el al. 1996; Lenett el al. 1999; Helmicki and Hunt, 2004; Hunt and Helmicki, 2005].

Based on past field test efforts, UCII personnel have developed and debugged an efficient, reliable, portable field test capability together with the computer software necessary for finite element (FE) modeling, model calibration and load rating calculation. A schematic of the overall approach is given in Figure 1-1. The primary objective of this research was to bring these tools to bear on the problem of field-testing and load rating of posted bridges.
Both modal and crawl-speed diagnostic truck-load testing are conducted in an integrated fashion to optimize efficiency in the field. Total test time, including bridge measurement and marking, sensor mounting, acquiring all data, and tear-down is generally within one working day per test per bridge, barring any significant delays due to unforeseen events such as a thunderstorm or equipment malfunction. This permits time for modal testing of two lanes, as well as crawl speed truck-load testing of a 2-lane, 3-span, 250 ft (76.2 m) bridge within two working days. A lane-by-lane test method is used to minimize the disturbance to traffic, although momentary bridge closure is required during data acquisition for periods of less than five minutes. Software capabilities have been developed to permit quick post-processing of test data so that data from these tests can be may be post-processed in almost real-time at the bridge site. This permits testing modifications to be made on-the-fly, thereby maximizing the efficiency of the field operations and minimizing the impact on traffic and ODOT. An analysis involving the preliminary (uncalibrated) 3D FE model of the bridge will be conducted so that sensors can be optimally located.
Through previous research efforts on commissioned 3-span, 2-lane steel-stringer bridges, it has been demonstrated that impact tests can be completed in both lanes, using a lane-by-lane test method, within one working day. This involves bridge measurement and marking, sensor mounting, acquiring all data, and teardown, and assumes no significant delays due to unforeseen events such as a thunderstorm or equipment malfunction. Similarly, it has also been demonstrated that crawl-speed truck-load testing can be completed within one working day in a lane-by-lane test method. Consequently, both forms of nondestructive testing can be accomplished on a designated bridge site within 2 working days. The lane-by-lane test method is used to minimize the disturbance to traffic, although occasional bridge closure is required during data acquisition for periods of less than five minutes. Software capabilities have been developed to permit quick post-processing of test data, therefore pertinent test results – natural frequencies, mode shapes, flexibility, influence lines – can be extracted from acquired test data within a short, reasonable time-frame following the tests. This permits testing modifications to be made on-the-fly, thereby maximizing the efficiency of the field operations and minimizing the impact on traffic and ODOT. An analysis involving the preliminary (uncalibrated) 3D FE model of the bridge will be conducted so that sensors can be optimally located.

Under an existing contract with ODOT, UCII has developed a user-friendly software package to automate the development of SAP FE models given user input of key bridge parameters (i.e., geometry dimensions, sectional properties, etc.). As field data becomes available, the preliminary finite element models of each test bridge are calibrated to conform to the experimentally determined dynamic characteristics, flexibility, peak strains, deflections, influence lines, etc. In addition, modeling assumptions such as linearity, elasticity, reciprocity, stationarity, and others are verified by the field tests.

Load ratings for a test bridge are computed after the models of the bridge have been calibrated. PC-based software algorithms which permit rating based on either direct field measurements (through the use of unit-influence line decompositions of truck-load testing results for the instrumented sections) or use of field calibrated FE models to simulate the entire or global bridge response to HS20-44 truck loadings have been developed. Model-based values for capacity, deadload, liveload, and load-ratings are compared against each other as well as with the existing ODOT BARS-based ratings as an additional check to ensure that the models adequately
simulate bridge behavior. All ratings are calculated in accordance with the *AASHTO Manual for Condition Evaluation of Bridges* [AASHTO 2000].

As an example of the kinds of results that can be obtained, several major findings obtained from testing PRE-725-0800 and BUT-732-1043 are discussed below and presented in Figures 1-2 to 1-5. These figures show the results of both modal and truck-load tests. Similar sets of findings have been obtained for nearly forty bridges tested by UCII in Ohio.

The deflection profiles in Figures 1-2 and 1-3 are based on measurements of in-situ flexibility extracted from modal test data and display the deflection profile of girders subjected to simulated uniform loads. Figure 1-3 is a uniform load based deflection profile for Girder 3 (the center girder) of PRE-725-0800. What is noteworthy about the deflection profile within this figure is that there are nonzero deflections at the east abutment, which indicates that this girder is not properly bearing, or supported, at the east abutment. Furthermore, Figure 1-2 reveals that this modal data based observation was corroborated by a visual inspection of this location. Consequently, the measure of in-situ flexibility provided by modal test measurements can be used to assess bridge condition/health and assist the inspection process.

![Figure 1-2: The Use of Modal Test Based Flexibility as a Raw/Direct Bridge Condition Indicator](image-url)
Figure 1-3 displays uniform load based deflection profiles for Girder 3 of both PRE-725-0800 and BUT-732-1043. For each bridge, two profiles are presented. Each profile in the figure for a respective bridge is based on the measurement of flexibility obtained from an impact test performed in one of the traffic lanes of the bridge. For both of these bridges, Girder 3, the center girder, is the common girder between the two respective traffic lanes. Superimposing the Girder 3 deflection profiles developed from the flexibility matrix for each lane essentially reveals that, regardless of which lane test is used, the same measure of flexibility is acquired. Although lane tests on a particular bridge were performed at different times of the day, the measure of flexibility acquired for Girder 3, as demonstrated in Figure 1-2, is consistent.

Figures 1-4 and 1-5 illustrate the results of the crawl speed truck-load testing on BUT-732-1043. The instrumentation plan/layout is illustrated at the bottom of each. It shows the placement of eleven strain gages on the top and bottom flanges at various locations. Displacement gages were installed in each end span in order to corroborate the estimated flexibility matrix obtained from the modal impact test.
Chapter 1 - Introduction, Background, and Objectives

Filtered Truckload Response of Bottom Flange Sensors, West Lane Path

Filtered Truckload Response of Top Flange Sensors, West Lane Path

Field Assessment of Live Load Demand for a Low Rated Bridge

Simulated HS20-44 Truckload Response, Middle Beam, Middle Span

Note: This response is determined by linear superposition of results obtained from actual field crawl-speed tests w/ weighed dump truck

Partial Composite Assessment

RF=1.7, Stress Inventory
RF=2.6, Stress Operating
RF=3.1, LFD Inventory
RF=3.1, LFD Operating

Conclusion: Higher capacity was determined from field tests due to better load distribution.
Figure 1-4 shows substantial stress felt along the top flange in the middle span compared to the end spans, indicating a significant loss of the unintended composite action in this region. This could well be due to the seasonal flooding of the creek over which the bridge is built. Deterioration is visibly evident upon inspection.

Figure 1-5 displays the derived live load moment and corresponding rating factors for the simulated midspan response to an HS20-44 truckload crossing. Post-processing at UCII laboratories was used to decompose the measured two axle truck response (e.g., Figure 1-4) into the unit load or influence response. The unit influence response was then utilized to simulate an HS20-44 truckload crossing in each traffic lane (see top graph of Figure 1-5). Note that several analytical methods were employed, based upon differing assumptions (e.g., axial force, effective deck width, composite action, etc.) in order to determine the appropriate live load moment. This estimation of live load moment (i.e., the existing bridge condition) is the most significant contribution of the field test, as compared to traditional 2D design and rating methods (e.g., BARS software), because it accounts for the existing structural mechanisms such as load distribution, partial composite action, etc. In an attempt at an “apples-to-apples” comparison with the current ODOT assessment, the rating factors shown in Figure 1.5 are based upon a non-composite steel girder capacity of 1,000 kip-ft (1356 kN-m) and a dead load of 270 kip-ft (366 kN-m) calculated from the measured physical dimensions (e.g., deck thickness of 9.25 inches (23.5 cm)).

1.2 OBJECTIVES OF THE 10 BRIDGE PROJECT

The main objective of this research was to non-destructively field test and evaluate ten damaged/deteriorated, low-rated bridges in order to determine the actual load carrying capacity. All test bridges were selected from the Ohio state inventory in conjunction with ODOT Office of Structural Engineering officials. A detailed, customized field test plan was developed for each test bridge and included modal testing, truckload testing, material sampling, calibrated FE modeling as needed in order to derive accurate, objective load ratings. The resulting information will help determine whether the bridge should be rated at the level currently established by ODOT using existing analytically-based rating methods or whether the rating should be increased or decreased.
In addition, the field test data obtained from these tests will be compared against the statistical signatures of “normal” bridges obtained for ODOT by UCII research personnel under a separate contract (i.e., the companion project, *Bridge Type Specific Management of Steel Stringer Bridges: Development of Field Calibrated Software Rating Tools and Statistical Bridge Database*). This comparison will give ODOT additional background as to the differences and variances between existing visual/analytical/numerical rating procedures and field test based evaluations of bridges in the inventory. Finally, the field test data obtained from these tests will provide detailed, localized information about the state of health of the structure. This information will assist ODOT in identifying specific rehabilitation and retrofit modifications as needed.

A secondary objective was to further streamline the testing and assessment procedures. Specifically, the actual time for a 32-channel modal impact test and a 24-channel truckload test of a typical 3-span, 2 lane, 250 ft (76.2 m) steel-stringer bridge was reduced to a half workday for each test. The development of the nominal finite element model (FEM) from the bridge plans was also reduced to a half workday. Preliminary and final assessment post-processing procedures were further automated for next day and next week delivery, respectively.

The benefits of this research to ODOT will be the development of techniques, equipment packages, and a personnel team with the expertise necessary to implement rapid, rigorous, quantitative assessments of critical structures around the state. In addition to these procedures and expertise, deliverables will include detailed reports of findings for each of the bridges in the test set (see Appendices).

It is also important to point out that this research and its companion project represent the integration and streamlining of several objective, quantitative, and efficient bridge assessment techniques recently developed at UCII. Over the course of this period, ODOT, FHWA, NSF, and Ohio Board of Regents (OBR) funds have been used to finance the development of the hardware/software and application methods that serve as the basis of the proposed research. As such, this research represents a capstone project for UCII and ODOT. In addition, this funding will support the education and training of individuals whom will directly perform the research. Furthermore, the proposed research has significant potential for national impact since the rating of damaged, deteriorated and/or low rated structures are problems not unique to Ohio.
1.3 SCOPE OF 10 BRIDGE PROJECT

A task-wise breakdown of the research effort can be given as follows:

1.3.1 Task 1: Bridge Inventory Analysis and Site Selection

Through collaboration with the ODOT Office of Structural Engineering, the Ohio bridge inventory was analyzed to categorize posted bridges (bridges rated below 100% legal load) and non-posted bridges (bridges rated between 100% and 150% legal load). In addition, bridge engineers in each of the state’s twelve districts were interviewed to identify damaged and/or deteriorated bridges around the state. This latter category will include bridges hit by vehicles, exposed to fires, suffering from extensive section loss, deck deterioration, etc.

Using this information, a list of potential test bridges was generated. With continued assistance from the Office of Structural Engineering and the various Ohio districts, this list was narrowed to 10 bridges deemed critical for evaluation. As with previous ODOT-sponsored field testing efforts, UCII based the final test sites on a number of factors including:

- **Bridges ODOT wants investigated:** UCII worked closely with ODOT’s Office of Structural Engineering throughout the selection process, and tailored the final test candidate list to best meet ODOT’s needs. For example, bridges located along an overload route, non-posted bridges with low visual inspection (i.e., condition) ratings, bridges damaged in truck collisions, or bridges scheduled for significant rehab/retrofit activities may be deemed as desirable test specimens.

- **Proximity and Traffic Control Issues:** To make most efficient use of travel funds as well as to evenly spread the effort needed for traffic control and access, test bridges were grouped by proximity within the various ODOT districts.

- **Bridge Type:** The selection process was limited by the current Ohio steel-stringer bridge inventory so that all specimens selected were steel girder/beam bridges with reinforced concrete decks. This ensured that the field test expertise previously developed by UCII on “healthy” steel-stringer bridges could be reliably extrapolated to the class of low rated bridges with a minimum of difficulty and uncertainty.
• **Bridge Size:** Clearly the size of the structure impacts the level of effort necessary in the field to conduct a test and collect data. One of the stated goals of this research effort was to streamline field operations so as to minimize impact on ODOT road crews and the general public. Field operations were streamlined to the point where a generic 3-span, 2 lane, 250 ft (76.2 m) bridge can be tested in 4 hours with either a modal impact or a crawl-speed truckload test or with both tests within an 8 hour workday. This did not limit our tested bridges to such specimens, but it does indicate the expected target specimen upon which the budget and timeline were based.

1.3.2 **Task 2: Initial Database and Analytical Model Development**

An archive/database was generated for each of the ten selected test bridges. Each bridge database contains all available information on design, shop-fabrication, construction, maintenance, rehabilitation, existing load-ratings, and visual inspection reports.

Prior to any field tests, researchers visited and inspected each selected bridge site to verify bridge condition and to plan field operations.

The information accrued in this process was used to develop preliminary analytical FE models of each respective bridge. These preliminary models helped aid in the design of field test operations, sensor layout, etc. as indicated below.

1.3.3 **Task 3: Field Operations Plan Development**

Based on the findings of the site visits, as well as the results of preliminary modeling efforts from Task 2, a field operations plan was developed for each of the ten test bridges. This plan fully documented the proposed field operations for each test bridge including:

• A listing of the NDT field methods described in Section 1.1 (i.e., impact testing, instrumented truckload testing, material sampling/testing) that were employed at the particular site.

• The anticipated test protocol, test parameters, and desired measurements were detailed for each NDT method selected (e.g., sensor suite and layout, impact locations, truck routes, sites for material sampling, laboratory material tests/evaluations, etc.).
A field operations timeline and access requirements list were developed so that appropriate traffic control and snooper/bucket truck availability can be scheduled with ODOT Central Office and District personnel.

A complete listing of the rating calculations and rating methods employed as well as the sources (e.g., field data, modeling/simulation output, engineering assumptions, and engineering design calculations) for all necessary calculation variables/parameters.

The field operation plan optimized the cost/benefit ratio associated with the time spent in the field versus the value of the data collected. The goal was to tailor the field operations at each test bridge site so that information most critical to developing an accurate load rating for that particular bridge was gathered with a minimum of field effort. The target was to spend less than 1 day for field testing of each specimen.

1.3.4 Task 4: Field Operations

For each of the ten test bridges, the respective field operations plan were carried out in coordination with appropriate District personnel. UCII contacted and coordinated with District personnel to schedule all field activities.

UCII personnel were responsible for conducting all testing activities and collecting all necessary data.

ODOT personnel were responsible for all traffic control and provided all necessary access to the bridge to conduct the testing. Depending on the test site and field operation plan, access may include (in addition to traffic control on the bridge deck) providing a snooper truck, manlift/bucket truck(s), or ladders for the test duration so that sensors could be installed on and removed from the underside of the bridge, piers, and/or abutments.

1.3.5 Task 5: Data Post-Processing and Load Rating Development

As a by-product of our previous sponsored research projects, several pieces of PC based software have been developed which permits fast, user-friendly post-processing of field test data, FE modeling, model calibration, and load rating calculation.
Impact testing can be completed within one working day. Impact data post-processing, and computation of natural frequencies, mode shapes, and flexibility can all be performed within 1 day after returning to UCII labs from the bridge site.

Using the truck-load measurements together with a sectional analysis will allow for an estimate of the live load moment for load factor design assessment (e.g., see Figure 1.5). PC based software has been developed which permits the computation of influence line, sectional properties, and rating factors. Hence, truck-load testing (using up to 24 gages), data post-processing, and rating factor computation for the instrumented sections can all be performed within 1 day after returning to UCII labs from the bridge site.

Concurrent with the field test operations and as data became available, the preliminary finite element model of each test bridge was calibrated as discussed above. In order to perform model calibration, certain structural and material properties within the model, such as moments of inertia and concrete modulus of elasticity, etc. were selected as variables. The calibration process itself involved iteration on these variables until the model yields state parameters (e.g., natural frequencies, mode shapes, flexibility, etc.) that match those identified/measured from impact and truck-load test data. Calibration was performed in this manner for the 3D finite element model of each bridge using SAP2000™ analysis software as the basis for simulation.

Load ratings for a test bridge were computed by all three methods outlined in Figure 1.1 and compared. All resulting values for load-rating were subsequently added to the test bridge database.

1.3.6 Task 6: Bridge Rating Report Generation

When testing and data post-processing of an individual bridge has been completed, a report of the findings specific to that bridge was generated. These individual bridge reports summarize:

- the nondestructive field test methods utilized on that particular bridge,
- the corresponding test results,
- analytical model generation and model calibration with field test results,
• the subsequent field-test-based load-rating, and

• an analysis of how this field-test-based load-rating compares to the corresponding ODOT documented load-rating.

1.4 FINAL PRODUCTS AND BENEFITS

At the completion of the project, the following products have become available:

• An experimental, objective database containing information about the structural properties of the selected test bridges. This database provides a representation of low-rated steel-stringer bridge behavior as well as a comparison with the statistical baseline of what may be considered normal and customary bridge behavior.

• A report for each bridge that documents the service level measurements and state parameters for that bridge together with the associated field-test-data-based load rating of the structure.

• Documentation of test-based indices (e.g., objective on-site experimental condition/damage indices that may be used to identify regions of damage/deterioration) and how they may be generated through on-site experimentation.

• In essence, the testing of 10 steel-stringer bridges will prototype the implementation of new procedures for inspection, rating, and management for the class of low rated bridges. Such procedures will be more objective than existing methods and will be along the spirit of the *AASHTO Guide for Strength Evaluation* [AASHTO 1989].
Chapter 2 Bridge Selection Criteria and Test Specimens

2.1 BRIDGE INVENTORY AND SITE SELECTION

Through collaboration with the ODOT Office of Structural Engineering, the Ohio bridge inventory was analyzed to categorize posted bridges (bridges rated below 100% legal load) and non-posted bridges (bridges rated between 100% and 150% legal load). In addition, bridge engineers in each of the state’s twelve districts were interviewed to identify damaged and/or deteriorated bridges around the state. This latter category includes bridges hit by vehicles, exposed to fires, suffering from extensive section loss, deck deterioration, etc.

Using this information, a list of potential test bridges was generated. With continued assistance from the Office of Structural Engineering and the various Ohio districts, this list was narrowed to 10 bridges deemed critical for evaluation.

As with previous ODOT-sponsored field testing efforts, UCII based the final test sites on a number of factors including:

- **Bridges ODOT wants investigated**: UCII worked closely with ODOT’s Office of Structural Engineering throughout the selection process, and tailored the final test candidate list to best meet ODOT’s needs. For example, bridges located along an overload route, non-posted bridges with low visual inspection (i.e., condition) ratings, bridges damaged in truck collisions, or bridges scheduled for significant rehab/retrofit activities may be deemed as desirable test specimens.

- **Proximity and Traffic Control Issues**: To make most efficient use of travel funds as well as to evenly spread the effort needed for traffic control and access, test bridges were grouped by proximity within the various ODOT districts.
• **Bridge Type**: The selection process was limited by the current Ohio steel-stringer bridge inventory so that all specimens selected were steel girder/beam bridges with reinforced concrete decks. This ensured that the field test expertise previously developed by UCII on “healthy” steel-stringer bridges could be reliably extrapolated to the class of low rated bridges with a minimum of difficulty and uncertainty.

• **Bridge Size**: Clearly the size of the structure impacts the level of effort necessary in the field to conduct a test and collect data. One of the stated goals of this research effort was to streamline field operations so as to minimize impact on ODOT road crews and the general public. Field operations were streamlined to the point where a generic 3-span, 2 lane, 250 foot bridge can be tested in 4 hours with either a modal impact or a crawl-speed truckload test or with both tests within an 8 hour workday. This did not limit our tested bridges to such specimens, but it does indicate the expected target specimen upon which the budget and timeline were based.

2.2 **SELECTION OF LOW-RATED BRIDGES**

All of the test bridges were selected from the state inventory in conjunction with ODOT liaisons in the Office of Structural Engineering. The test specimens were selected based on overall length, number of spans, number of girders, skew, etc. Each of the 10 selected bridges is briefly described below.

2.2.1 **ATH-356-0459 (0504203)**

Located in District 10 on State Route 356, this is the only bridge tested by UCII that contains 9 girders, making it an outlier. This bridge had 3 spans, a skew greater than 10°, a low volume of truck traffic and was not designed for composite action. These parameters are consistent with bridges UCII has previously tested.
2.2.2 **LIC-158-0164 (4505379)**

Located in District 5 on State Route 158, this is a 4-span, 4-girder bridge with a skew less than 10° and was not designed for composite action. This bridge also has a low daily truck count. These characteristics are very similar to several bridges tested within the 24 bridge project.

2.2.3 **MAD-40-0745 (4901290)**

Located in District 6 on US Route 40, this is a 2 span, 7-girder bridge. This is the only bridge tested with this configuration. Unlike most bridges included in this project, this bridge has a length-to-width ratio approaching 1. As this ratio approaches 1, the bridge behaves like a plate instead of a beam. This bridge was selected to compare/contrast the differences of bridge behavior as its geometry changes.

2.2.4 **MOT-70-0553 (5704952)**

Located in District 7 on Interstate Route 70, this is another 4-span, 4-girder bridge, much like LIC-158-0164. Unlike LIC-158-0164, this bridge handles a high volume of truck traffic.
2.2.5 AUG-75-0201 (0601926)
Located in District 7 on Interstate Route 75, this is also a 4-span, 4-girder bridge. Like MOT-70-0553 this bridge is also subjected to a large volume of daily truck traffic. The distinguishing characteristic between this specimen and MOT-70-0553 is this bridge was built with integral abutments, while MOT-70 has non-integral abutments.

2.2.6 CLE-52-0142 (1301357)
Located in District 8 on US Route 52, this is a 3-span, 5-girder bridge. This is a very standard configuration within UCII’s test database; however, none of the previously tested bridges of this type were posted. This bridge was selected to directly characterize the influence of damage/deterioration on bridge dynamics.

2.2.7 CLI-132-0083 (1402587)
Located in District 8 on State Route 132, this is a 3-span, 5-girder bridge with a skew greater than 10°. This bridge geometry is similar to bridges within the 24 bridge project, again providing the opportunity to quantify the effects of damage/deterioration on bridge dynamics.
2.2.8 PRE-503-1170 (6803660)

Located in District 8 on State Route 503, this is a 3-span, 5-girder bridge with a skew greater than 10°, which is very similar to CLI-132-0083 above.

2.2.9 LAW-217-0697 (4403002)

Located in District 9 on State Route 217 this is a single span bridge with 11 girders. Much like MAD-40-0745, this bridge was selected to provide information about bridges with a length-to-width ratio approaching 1.

2.2.10 ROS-41-1451 (7102976)

Located in District 9 this is a 3-span, 6-girder bridge with no skew and a high daily truck count. This bridge was selected to help reflect the statistical profile of ODOT’s inventory.
Table 2-1 shows the ten tested bridges and their various attributes. The field test data obtained from these tests can be compared against the statistical signatures of “healthy” bridges obtained for ODOT by UCII research personnel under a separate contract. In addition, the field test data obtained from these tests will provide detailed, localized information about the state of health of the structure. This information will assist ODOT in identifying specific rehab and retrofit modifications as needed.

<table>
<thead>
<tr>
<th>Bridge SFN</th>
<th>Bridge</th>
<th>Dist.</th>
<th>No. of Spans</th>
<th>No. of Girders</th>
<th>Skew</th>
<th>Construct Type</th>
<th>Abutment Type</th>
<th>Max Span Length</th>
<th>Deck Width</th>
<th>ADTT</th>
</tr>
</thead>
<tbody>
<tr>
<td>56345-0254</td>
<td>LIC-158-0164</td>
<td>4505379</td>
<td>5</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>4901290</td>
<td>MAD-40-0745</td>
<td>4901290</td>
<td>6</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>5704652</td>
<td>MOT-70-0553</td>
<td>7</td>
<td>X</td>
<td>X</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
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<td>601926</td>
<td>AUG-75-0325</td>
<td>7</td>
<td>X</td>
<td>X</td>
<td>X</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>1402597</td>
<td>CLI-132-0003</td>
<td>8</td>
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<td>X</td>
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<td>X</td>
<td>X</td>
</tr>
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<td>6803950</td>
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<td>8</td>
<td>X</td>
<td>X</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>4403002</td>
<td>LAW-217-0697</td>
<td>9</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>7102976</td>
<td>ROS-41-1451</td>
<td>9</td>
<td>X</td>
<td>X</td>
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<td>504203</td>
<td>ATH-356-0459</td>
<td>10</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

Table 2-1: Bridge Specifications and Parameters
Chapter 3 Multiple Reference Impact Testing

The modal parameters of a structure may be identified by a variety of testing methods. These include ambient vibration techniques [McLamore et al. 1971], forced excitation [Aktan et al. 1996], pull-back mechanisms [Douglad and Reid 1982] and impact [Aktan et al. 1996]. Impact and forced excitation methods measure both input and output, allowing the system response to be scaled with respect to the input. This is critical since the computation of modal flexibility requires scaled modal vectors [Lennet 1989]. As a result, forced excitation or impact methods should be used to provide the input for modal testing. Impact testing was decided as the method of choice due to its minimal set-up time, equipment requirements and ease of transporting the equipment to and from the work site.

3.1 MULTIPLE REFERENCE IMPACT TESTING

Impact testing typically utilizes an impulse force to excite the structure under evaluation. In theory an impulse force provides a flat response in the frequency domain. Therefore, it is able to provide energy to excite any modes of the test structure that have natural frequencies within the frequency spectrum. Characteristics of the excited modes may be subsequently measured with response sensors such as accelerometers. By providing impact at multiple points, expressed as \( N_i \), significant spatial information may be generated and acquired regarding the modal behavior of the test structure. By measuring the response at these and other points, a response model can be estimated by a least squares method. Such information is needed to uncouple closely spaced modes - modes whose frequencies are close in magnitude - and establish good global estimates of modal parameters, i.e., damping coefficient, damped natural frequency, eigenvector and the modal scaling for each mode. These are the needed parameters for the computation of modal flexibility.
3.2 IMPACT TEST SETUP

Impact testing provides an impulse to the structure being evaluated. Theoretically it provides a constant magnitude across the frequency spectrum. However in practice this is not the case. There is a limited bandwidth for which ample energy is present to excite the modes of the structure. Calibration of the impact device, typically an instrumented hammer, is required to ensure an appropriate bandwidth for the impulse. Typically bridges have modes whose frequencies lay below 200 Hz [Lennet 1989] requiring the hammer to be calibrated to concentrate its energy in the low end of the frequency spectrum. Figure 3-1 shows a typical impact response, in both the time and frequency domain, for the hammer used to provide the input to the bridge.

To acquire the data, a signal analyzer is required that can properly sample the response from the accelerometers. UCII utilizes an HP VXI Mainframe, which can sample an analog signal at a rate of 32,768 Hz. However it is only necessary to obtain data at 200 Hz and below. Any information above 200 Hz will be filtered by the data acquisition system since the modes of interest usually exist below this frequency. A high frequency resolution is required to allow the identification and uncoupling of closely spaced modes. The analyzer permits the selection of 25, 50, 100, 200, 400, 800, 1600 or 3200 spectral lines within the specified frequency range. Thus, for a 200 Hz bandwidth, a frequency resolution of 200Hz/ 3200 lines = 0.0625 Hz is possible.
Since \( T \), the sampling period, is defined as \( 1 / \Delta f \); we have a 16 second time record. Figure 3-2, Figure 3-3, and Figure 3-4 show the HP VXI Mainframe, an accelerometer and the impact hammer used when performing a modal test.

### HP 3566A PC SPECTRUM/NETWORK ANALYZER
HEWLETT PACKARD COMPANY

<table>
<thead>
<tr>
<th>GENERAL DATA ACQUISITION CHARACTERISTICS</th>
<th>8 - 48</th>
</tr>
</thead>
<tbody>
<tr>
<td>Channels</td>
<td></td>
</tr>
<tr>
<td>Frequency Spans</td>
<td>0 - 195 mHz up to 12.8 kHz</td>
</tr>
<tr>
<td>Frequency Resolution</td>
<td>25,50,100,200,400,800,1600,3200</td>
</tr>
<tr>
<td>Sample Rate (per Channel)</td>
<td>((2.56 \times f_{\text{max}})) samples/sec</td>
</tr>
<tr>
<td>Input Range</td>
<td>5 mV peak to 10 V peak</td>
</tr>
<tr>
<td>Dynamic Range</td>
<td>72 dB</td>
</tr>
<tr>
<td>Connections</td>
<td>Grounded or Floating</td>
</tr>
<tr>
<td>Coupling</td>
<td>AC or DC</td>
</tr>
<tr>
<td>AC Coupling roll off</td>
<td>&lt;3 dB at 1 Hz</td>
</tr>
<tr>
<td>Trigger (from Input Channel)</td>
<td></td>
</tr>
<tr>
<td>Internal</td>
<td>Positive or Negative Slope</td>
</tr>
<tr>
<td>Pre-Trigger Delay</td>
<td>0 to a maximum of 8191 samples</td>
</tr>
</tbody>
</table>

### MAINFRAME POWER SUPPLY

| Line Voltage                              | 86 to 127 VAC |
| Line Current                              | \(<6.0\) A |

### ENVIRONMENTAL

| Temperature Range                          | \(+32\) to \(+131\) deg F |
| Relative Humidity                          | 95% at \(+104\) deg F |

### PCB 393C ACCELEROMETER SPECIFICATIONS
PCB PIEZOTRONICS, INC.

| DYNAMIC                                   |       |
| Measurement Range                         | \(\pm 2.5\) g |
| Resolution                                | 0.0001 g |
| Nominal Sensitivity                       | 1000.0 mV/g |
| Frequency Range                           | 0.025 - 800 Hz |
| Resonant Frequency                        | > 3500 Hz |

### ENVIRONMENTAL

| Shock Limit                                | 100 g (peak) |
| Temperature Range                          | -100 to +200 deg F |

### ELECTRICAL

| Excitation (Power req’d)                   |       |
| Constant Current                          | 2 - 20 mA |
| Voltage                                   | 18 - 28 VDC |

Figure 3-2: HP Network Analyzer System

Figure 3-3: PCB 393C Accelerometer
Throughout the testing process, stable averaging is performed on measured FRFs to decrease the effect of variance errors. Stable averaging weights old and new FRFs equally to yield the arithmetic mean for the number of averages selected. A stable average is computed as follows:

\[
A_n = \frac{\sum D_n}{n}
\]  

(3-1)

where:

- \( A_n \) = cumulative average
- \( D_n \) = current quantity
- \( n \) = number of averages

More averages, \( n \), should yield a better statistical average, however this requires additional testing time. Previous work has shown that five averages are sufficient [Kamen and Heck 1997].
3.3 TEST EXECUTION

Figure 3-5 provides an example of an extensive accelerometer layout for MOT-49-1658 (not tested in this project) used in conjunction with several impact points. Utilizing such a test grid will significantly reduce the time duration of an impact test. Reducing the length of a test minimizes possible variations in temperature, humidity and other environmental conditions that may alter bridge behavior, thereby allowing the bridge to be modeled as a time-invariant system. In addition, a compact test interval will minimize the interruption of traffic to the structure.

The sensors are spaced to cover the deck of the bridge and multiple impact points are spread throughout the length of the bridge. Due to the high mass of the bridge relative to the hammer being used, energy is quickly dissipated along the structure. It is quite common for modes to be locally excited. In other words, some modes of vibration can only be observed if a high level of energy is present. To overcome these effects, several impact points are used.

Great care must be exercised when conducting a modal test to ensure the data is of as high quality as possible. Noise can easily affect the output response measured by the accelerometers. This is due to the relatively small signal levels present during data acquisition. Therefore the potential for any outside influences must be eliminated.
The bridge must be shut down to traffic during the testing process. If the bridge is an overpass, measurements must be taken when no traffic is passing under the bridge. This will minimize unwanted responses due to unmeasured traffic input. Also, pedestrians walking on the deck of the bridge can influence the output response of the accelerometers. Although access to the bridge has been halted, it is possible to allow motorists access to the bridge intermittently. If the test is being performed in a high traffic area, the test can be performed in such a way to allow access to one of the traffic lanes. After a measurement has been taken and the data has been accepted/stored, the bridge can be temporarily opened to traffic. Once the traffic has been stopped, the next impact and measurement is conducted when all vibrations have dissipated.

The data used to provide modal flexibility is acquired in the following manner:

1) The impact hammer is connected to channel 1 of the analyzer and the 36 accelerometers are connected to channels 2-37. The hammer is used to provide an impact excitation to the bridge. The impulse force signal serves as the trigger for synchronous data acquisition – data acquisition for 37 channels began when the impulse force signal exceeded 30% of the voltage range for channel 1. (Typically this range is +/- 1V)

2) The digitized time signal for each of the 37 channels is observed in windowed and unwindowed form allowing the observation of each signal. The analyzer also computes the Input Power spectrum for channel 1 and the Output Power spectrums, Cross Power spectrums, Coherence and FRFs for channels 2-37. If all signals and their statistical measurements are of high quality, then the data is accepted. If any channel has some error, all data is rejected, the source of error is determined and eliminated and the impact is reapplied.

3) Impact is reapplied at the same location and steps 2 and 3 are repeated. This is done a total of five times, since n, the number of averages, has been determined to be five. Hence, the parameters calculated in step 3 are based on the average of five impact points.

4) After the storage of data, steps 1 through 4 are repeated at a different grid location.
A flowchart detailing these steps is presented in Figure 3-6.

**Figure 3-6: Flowchart for Acquiring FRF Data**

### 3.4 MODAL PARAMETER ESTIMATION

Developments in the applications of multi-reference impact testing have significantly improved the capability of impact testing for reliable modal data. In the case of condition assessment and damage identification, the reliability of modal parameters (mode shapes, frequency and damping) are of great importance since the damage identification algorithms and methods using dynamic testing rely on the modal parameters. Many different algorithms have been developed to accurately ascertain these values from experimental data. They can be classified into three main parameter estimation groups, Time Domain, Frequency Domain and Spatial Domain. The time and frequency domain algorithms can be further classified as high order or low order. Some of the more popular algorithms are [Allemang 1994]:

- Eigensystem Realization Algorithm, ERA (Time Domain, Low Order)
- Least Squares Complex Exponential, LSCE (Time Domain, High Order)
• Polyreference Time Domain, PTD (Time Domain, High Order)
• Ibrahim Time Domain, ITD (Time Domain, High Order)
• Polyreference Frequency Domain, PFD (Frequency Domain, Low Order)
• Simultaneous Frequency Domain, SFD (Frequency Domain, Low Order)
• Rational Fraction Polynomial, RFP (Frequency Domain, High Order)
• Complex Mode Indicator Function, CMIF (Spatial Domain, Zeroth Order)

The high order model is typically used for cases where the system is undersampled in the spatial domain. The limiting case is when only one measurement is made on the structure. In this situation the linear equation of motion is defined with an order equal or greater than the total number of desired modal frequencies. However utilizing this high order model may introduce significant numerical problems for the frequency domain case.

Low order algorithms are used when the spatial information is complete. This situation is realized when the number of measurement points is greater than the number of modal frequencies desired. The zero order model represents algorithms that only utilize spatial information and ignore the temporal information.

The algorithm implemented by UCII is referred to as the Complex Mode Indicator Function (CMIF) [Fladung 1994; Lenett 1998]. This is a spatial domain method in which estimates of the eigenvalues and eigenvectors are directly obtained from the Singular Value Decomposition (SVD) of the measured FRF matrix. The SVD process is described in detail by Lay [Lay 1996] and can be applied to any matrix. A plot of the Singular Values (SV) of the FRF matrix, known as the CMIF plot, indicates the location and number of eigenvalues present in a data set (e.g., see Figure 3-7). The number of singular values at a spectral line, i.e. the number of curves in the CMIF plot, is dependent on the number of impact points used during the test. Peaks in the CMIF plot are possible locations of eigenvalues. A more detailed explanation of the CMIF algorithm can be found in Catbas [Catbas 1997]. A flowchart for this algorithm is presented in Figure 3-8.
3.5 SELECTION/VALIDATION OF MODEL PARAMETER ESTIMATES

Once the proper parameters have established, a series of system checks is implemented to provide confidence that the results are reliable.
1) Data is time-invariant

2) Acceptable curve-fit to eFRF

3) Damping is within an acceptable range

4) Phase plot for each eFRF crosses -90°

5) Individual modes have been properly decoupled

6) Appropriate mode shape

For further details regarding the quality checks, see the final report for the companion project, *Bridge Type Specific Management of Steel Stringer Bridges: Development of Field Calibrated Software Rating Tools and Statistical Bridge Database.*

### 3.6 MODAL FLEXIBILITY CALCULATION

Once the modal parameters have been identified and determined to be valid, modal flexibility is calculated. The flexibility matrix of the system is the FRF matrix evaluated at zero frequency. With the following mathematical definition of the FRF matrix:

\[
[H(\omega_i)] = \sum_{r=1}^{N} \left[ \begin{array}{cc}
\{\psi\}_r^T & \{\psi\}_r^*T \\
M_{Ap} & M_{Ap}^*
\end{array} \right] \left[ \begin{array}{cc}
j\omega_i - \lambda_r \\
-j\omega_i + \lambda_r^*
\end{array} \right]^{-1} \left[ \begin{array}{cc}
\{\psi\}_r & \{\psi\}_r^*
\end{array} \right]
\]

(3-2)

Evaluating this equation at \(d_c\) yields:

\[
[H_{pq}(\omega_i = 0)] = \sum_{r=1}^{N} \left[ \begin{array}{cc}
\{\psi\}_{pr} & \{\psi\}_{qr}^*
\end{array} \right] \left[ \begin{array}{cc}
M_{Ap} & M_{Ap}^*
\end{array} \right]^{-1} \left[ \begin{array}{cc}
\{\psi\}_{pr}^* & \{\psi\}_{qr}^*
\end{array} \right] = f_{pq}
\]

(3-3)

This last equation indicates that a flexibility coefficient, \(f_{pq}\), the displacement at point \(p\) due to load at point \(q\), can be computed by the summation of the estimated modal parameters. This is referred to as modal flexibility.
As seen in this equation, modal flexibility is computed by summing through $N$ number of modes. Physical structures have an infinite number of modes present in their dynamic behavior. However only $N$ of these modes will be identified and estimated due to experimental limitations such as the frequency bandwidth of the test, as well as the number of impact points and response locations. If an inadequate number of modes are identified from the experimental data the resulting modal flexibility matrix will not properly describe the structure’s flexibility.

The modal flexibility matrix gives an indication of the movement of each point on the bridge deck where a measurement was made. The values contained within the flexibility matrix are not absolute values; instead each row of the matrix indicates the amount of relative displacement for a given point in relation to all other measurement points. A flexibility matrix is computed for each mode of vibration as determined by the CMIF algorithm. Displacement profiles, $\{\mathbf{\delta}\}$, are then obtained by multiplying each resulting flexibility matrix with a load vector, $\{\mathbf{P}\}$, comprised of unit loads at each measurement point, $\{\mathbf{\delta}\}=[\mathbf{f}^\dagger] \{\mathbf{P}\}$.

Determining if a sufficient number of modes are available to provide a reliable displacement profile is achieved by a modal truncation study. This involves computing the flexibility for each mode and summing these individual flexibilities (e.g. $[\mathbf{f}^\dagger]_{\text{mode } 1}$, $[\mathbf{f}^\dagger]_{\text{mode } 1+\text{mode } 2}$, $[\mathbf{f}^\dagger]_{\text{mode } 1+\text{mode } 2+\text{mode } 3}$, etc.). Once the displacement profiles are computed they are superimposed. Modal flexibility “converges” at mode “x” if the deflection profiles do not vary when additional modes are included in the computation of flexibility. If convergence exists then there are a sufficient number of modes estimated to accurately describe the modal flexibility of the structure under test.

Figure 3-9 shows an example of superimposing individual displacement profiles until a consistent flexibility is achieved. In the figure shown flexibility converges once the 13th mode has been included. The inclusion of additional modes did not alter the displacement profile. Previous research has shown creation of the flexibility profile to be a repeatable process that is insensitive to ambient conditions. Therefore the profile can be used as a baseline signature for comparison to future tests. Any deviation from the baseline profile can then be attributed to some change in condition of the bridge itself. This may include damage, such as a truck hitting an overpass, or may be a result of repairs made to the structure.
Figure 3-9: Modal Flexibility Convergence Study
Chapter 4 Truckload Testing

4.1 LOAD TESTING BACKGROUND

One common issue in civil infrastructure systems is the need to monitor, assess, and diagnose structural integrity of the global structure and in its various local components. To avoid hazardous collapse and/or disruption of the service, officials must evaluate the expected useful life of the structure and plan repair or renewal accordingly. Subjective or inaccurate condition assessment has been identified as the most critical technical barrier to effective infrastructure management [Clinton, 1993]. For example, since 1971, conditions of highway bridges are typically expressed in terms of subjective indices which are based on visual inspections alone [AASHTO, 1983 and 1989]. The difficulties of visually inspecting and evaluating an aging constructed facility accurately and completely, even when this may be conducted by experienced engineers, are well-known [FHWA, 1993]. NDE technologies of a rigorous and objective nature are sought to quantitatively identify and evaluate the condition or health of highway structures.

One conceptual signature that represents bridge condition and can be determined from a truckload test is the fundamental structural parameter of the unit influence line, the characteristic response at any instrumented bridge node due to the position of a unit load. Here, the unit load is defined to be a truck axle of one kip total weight and the loading path is considered to be that of a typical tandem or semi truck driven in the marked lane(s). It has been demonstrated that the strain (or stress) influence line is a damage sensitive index by conducting truckload tests on decommissioned concrete and steel highway bridges which were loaded to various damage states [Levi 1997; Levi et al. 1997; Turer 1997]. This research will demonstrate how the influence line can be reliably identified from controlled or service loadings and then used to immediately provide a capacity rating for the instrumented section and/or an estimation of the remaining fatigue life for the instrumented member/connection based upon the relevant AASHTO codes.
4.2 TRUCKLOAD TESTING

The AASHTO Manual for Condition Evaluation of Bridges suggests “load testing as an effective means of evaluating the structural performance of a bridge or its selected components. This applies particularly to those bridges which cannot be accurately modeled by analysis, or to those whose structural response to live load is in question. Load evaluation tests are made to determine the magnitude and variation of loads and load effects such as those due to traffic, temperature changes and wind. Diagnostic load tests are performed to determine the effect on various components of a known load on the structure. Proof load testing is designed to directly determine the maximum live load that the bridge can support safely.”

Truck, proof, or other load testing of highway bridges has usually been reserved as an area of academic research due to the complexities, cost, and disturbance to service of such a field experiment. However, when a structure’s computed capacity is less than the desired level of performance, it is usually beneficial to the owner to objectively identify the actual structural response to controlled loading experiments. The constructed bridge will have many inherent mechanisms to resist the applied load and which are generally not considered in the analysis of its capacity. These identified mechanisms include [Lichtenstein 1998]:

- Load distribution,
- Unintended composite action,
- Participation of superimposed deadload,
- Material properties,
- Participation of secondary members,
- Effects of skew,
- Effects of deterioration and damage, and
- Unintended bearing restraint.
Diagnostic load tests have traditionally been performed in conjunction with significant analytical modeling (e.g., finite element) of the structure. A nominal model is generated based upon the design, site, inspection report, and other information at hand. Virtual load simulations are conducted with the nominal model in order to determine the appropriate locations for the bridge instrumentation. If static loads are to be employed, then the model will dictate the worst-case position of the trucks. In practice, this preliminary effort is quite often bypassed as the predicted locations by the model are generally intuitive, based upon the first principles already incorporated within the linear model. Further, the number of sensor and truck locations during the test are maximized to the limits of all practical constraints in order to minimize the possibility of overlooking some hidden defect or damage. Crawl-speed tests are preferable because continuous measurement of the load response is recorded for the entire traffic lane (as opposed to a finite set of pre-conceived locations). After the load test, the model is then calibrated based upon some cost function. The calibrated model is then used to virtually determine any of the desired indices at any/all locations for condition assessment and damage detection.

4.3 TRUCKLOAD TEST SETUP

In coordination with ODOT, the research team developed a sensor suite for a series of controlled experiments with loaded (and weighed) dump trucks in order to determine the actual condition and load capacity of the structure.

A custom strain transducer, marketed and calibrated by Bridge Diagnostics, Inc., (BDI) has been used extensively for this project. Lab experiments and field tests have shown that these sensors are not only highly sensitive and reliable, but quick and easy to install and remove from steel. These full-wheatstone bridge strain transducers were originally developed in 1970 for use in the driven pile industry. The manufacturer’s calibration is performed with a NIST-traceable system at less than 2% error, verified by a calibrated precision micrometer. For truckload tests, this represents an error of only a few microstrain, which is insignificant in quantifying the behavior of a large structure.
Chapter 4 - Truckload Testing

Figure 4-1: BDI Full-Bridge Strain Transducer

Tapeswitch controflex ribbons were installed on the expansion joints in the marked traffic lanes in order to record the tire crossings of the truck as the vehicle travelled onto and off of the bridge. Given that the location between the switches are known, these measurements provided a reference point spatially and temporally for the truckload test. A tape or ribbon switch is simply two metallic strips or plates encased within an elastomeric material with a thin insulating layer such that a large load applied to the top of the switch/strip would result in electrical continuity between its two sensor leads. By connecting the tape switch between the excitation voltage and ground (with a resistor to limit the current draw), the activation of the switch would result in a clear spike on its data channel.

Figure 4-2: Tapeswitch Controflex Ribbon Switch

Displacement transducers were installed occasionally at a small number of locations, usually in the end span of the bridge, in order to verify the flexibility of the structure under truckload. This allowed a secondary measurement to compare with the estimate obtained by modal test.
A MEGADAC data acquisition system from Optim Electronics was used for this project. Its purpose was to monitor the high-speed sensors installed on the bridge. An Optim MEGADAC is ideally suited to read high-speed sensors for several reasons. One reason is that it uses a GPIB interface that allows high-speed communication between the data acquisition system and a computer. It also has an aggregate 25 kHz sampling speed. Another reason is that this system can read virtually any sensor with minor changes to its operating software. This data system has proven its accuracy and reliability both in the laboratory and over many years of field tests. It would be stationed in the back of the van and run off a Yamaha gas generator for each test. A dedicated computer would interface with the data system in order to run the test software for data collection during a truckload test. All sensor cables would be routed to the van along the bridge railing and/or sidewalk. Simple twist-on military connectors were employed for reliable connection over these many field tests.

To cover the entirety of the structure while minimizing installation time in the field, a general sensor plan was defined in order to utilize our full inventory of available transducers and to minimize cable lengths. This plan evolved over the course of this project. Initial sensor plans were more sparse in order to achieve timely results and allow more time to debug any problems.
that may and did occur in the field; further, we did not immediately invest in a large inventory of any transducer type before we could verify their quick and reliable performance during initial field tests. As the BDI transducers were proven and our team became efficient in their use, sensor plans became larger as we could reliably install a greater number on the test day. You will note that later plans pursued additional details that we could not investigate in earlier tests (e.g., lateral distribution at midspan, effects of girder damage and repairs, abutment restraint) because of the availability of more transducers and the speed at which we could install them.

Figure 4-4: Example Sensor Plan for a Truckload Test

In general, our sensor plan focused upon the critical girders under the traffic load as marked. For a symmetrical bridge, the central girder(s) are usually critical; however, a larger sidewalk or parapet on one side of the bridge will stiffen the bridge near it and move the critical load path off-center and away from it. The critical positive moment is at or near center in each span and can be located by the nominal finite element model. Center spans are gaged first and end spans are gaged as time permits. Sensors are installed in pairs at the inner face of both the bottom and top flange of the girder in order to assess the composite action with the deck. On occasion, the web would be instrumented in lieu of the flange (e.g., to avoid rivets, etc.). In later truck tests, we decidedly pursued instrumentation of all girders at the center span in order to verify the lateral distribution of the load in the section. Exterior girders would only be gaged at the bottom flange, unless these girders were indicated to control the rating for the structure. Crossframes, stiffeners, and other connections were avoided by at least one foot distance.
Pier supports are also gaged to observe negative moment; however, this data comes with inherent concern about local effects. Nonuniform stress concentrations or raisers appear near a point loading or at its resistance by a support, much as they do around changes in geometry. Saint Venant’s Principle dictates that it requires a distance of at least a beam depth away from a support or discontinuity before the stress field can become uniform again; however, these local stresses are reduced rapidly as you begin to move away from the support. Hence, pier sensors are not installed at the support centerline but one foot (.305 m) away in order to reduce these effects yet still record the negative moment response near the pier. Note that we do not use pier sensors to calibrate the finite element model because of this concern; however, modelled and measured responses of the piers have compared favorably (albeit with nonlinear distortion).

Sensor plans for the truckload tests are provided within each rating report in the appendices. All of the bridges were modal tested and most were truckload tests; however, one of the bridges (ROS-41-1451) was not tested by truckload due to access concerns.

### 4.4 TEST EXECUTION

Field execution of the test also evolved over the course of the project and varied depending upon the specifics of each structure, its location, and the available access. Where possible, ladders were employed to reach the underside of the bridge; typically, this was over streams. In many cases, however, the vertical clearance was too high for ladders except in the end spans. Hence, a hydraulic lift, bucket truck, or the snooper would need to be used to install the sensors. ODOT would provide this equipment with an operator, as well as traffic control on and, if necessary, below the bridge on the day of the truckload test. In order to mitigate this effect upon the public, we would limit the traffic control between morning and evening rush hour (i.e., from 9AM to 4PM).

ODOT provides a loaded and weighed dump/salt truck for the controlled experiments. The truck is weighed at a local scale for its gross weight, but also each axle is weighed (i.e., the axle is centered on the scale and the other axle(s) are off the scale). For a tandem axle, both axles must be placed on the scale and the weight is assumed to be split equally between the back axles. The axle spacings were found manually by measuring tape.
The same truck and driver are used during any testing in order to minimize the variation in the test. The driver is instructed to maintain a constant speed (e.g., 15 mph (24 km/h)) depending upon the span lengths of the bridge (see below) and to keep the vehicle centered within the marked traffic lane. Each lane is tested at least three times per test. Of course, it is not possible to perfectly achieve these goals, but past experience has shown that the typical variation in the experiment due to the driver, the vehicle, its actual weights, its path, its speed, etc. is well within reason and the experiment is not especially sensitive to these parameters.

It should be noted that up to 10% error is assumed for the experiment, its measurands, and any ratings. This is a summation of all the uncertainties associated with field instrumentation and experimentation.

4.5 POST-PROCESSING OF TRUCKLOAD TESTS FOR DERIVATION OF UILS

Custom software was developed to calculate the unit influence line of the instrumented sections from the measured truckload data using the concept of linear superposition. The software runs on a laptop almost instantaneously. The theory and coding of this algorithm was developed over several projects supported by ODOT involving many bridges of varying type and dimension [Hunt 2000; Helmicki and Hunt 2004; Hunt and Helmicki 2005].

An influence line is defined as a graphical or formulaic presentation of the variation in magnitude of a force, moment, deflection, or other parameter at a single fixed point in a structure as a function of position of an applied unit load on the structure. The key concept is that the load position is now the functional variable and the structural response is determined for a fixed point on the structure. The unit influence line (UIL) is normalized for a point load of 1 kip or 1,000 lbs (4.45 kN). Due to superposition, an influence line is especially helpful to a bridge engineer to understand the effects of various loads at different positions and/or orientations (e.g., point, uniform, etc.) on the structure and identify the maximum or worst-case loading scenarios for condition assessment. For example, the response to a slowly moving truckload can be determined by adding the weighted sum of influence lines corresponding to each axle weight (Figure 4-5) [Turer 1997; Turer and Aktan 1999].
4.6 DISTRIBUTION FACTOR

The lateral distribution of the load within the positive moment region was examined for all the tested structures (see Chapter 8). Note that initial structures tested for this project did not include gaging to directly measure the distribution, but later tests had sufficient available sensors to instrument the bottom flanges of all the girders at a representative midspan.

The AASHTO Design Specification defines the distribution factor (DF = $S / 11$, where $S$ is the beam spacing in feet or $DF = S / 3.355$, where $S$ is the beam spacing in meters) as the maximum fraction of the rating load transferred to any given member at the location of maximum response for the travel path. The remaining portion of the load is carried or distributed among the other members of the superstructure. This equation for distribution factor was originally considered for orthotropic plates, which are free from edge stiffening and skewness of platform, with a vehicle-to-edge distance of one meter (3.28 feet). However, this latter assumption is quite conservative for most bridge designs; further, many bridge designs incorporate some skew and some edge stiffening in the form of parapets or sidewalks. There are many other mechanisms in addition to beam spacing which contribute to load distribution, including the type and spacing of crossframes and other secondary members, the thickness and strength of the concrete decking, and the stiffness of the girders. Hence, this simplified equation for distribution is generally inaccurate [Ghosn et al. 1986; Goble et al. 1992; Deatherage et al. 1995; Kim and Nowak 1998] which further compounds the error in the rating process.
Chapter 4 - Truckload Testing

As the distribution factor is intended to scale the designed liveload moment for rating purposes, it should be defined from the identified moments for each girder in the given section. The design truckloads should actually be employed; however, any truck with a significant load can be used assuming linear superposition for the structure. The latter assumption should be checked during the diagnostic load test. The rating load considers the worst-case scenario of side-by-side trucks in each traffic lane for the bridge. Hence, the distribution factor must be sum of the fractions for each single lane loading or, alternatively, the fraction for one multiple lane loading multiplied by the number of lanes.

\[
DF_{ij} = \frac{M_i}{\sum_{k=1}^{NG} M_{kj}} = \frac{E_S S_{ij} \varepsilon_{ij}}{\sum_{k=1}^{NG} E_S S_{kj} \varepsilon_{kj}} \quad (4-1)
\]

\[
DF_i = \sum_{j=1}^{NL} DF_{ij} = \sum_{j=1}^{NL} \frac{M_{ij}}{\sum_{k=1}^{NG} M_{kj}} = \sum_{j=1}^{NL} \frac{E_S S_{ij} \varepsilon_{ij}}{\sum_{k=1}^{NG} E_S S_{kj} \varepsilon_{kj}} \quad (4-2)
\]

Note that \( \sum_{i=1}^{NG} DF_i = NL \quad (4-3) \)

where:

- \( DF = \text{Max}(DF_i) \)
- \( i = \) girder number
- \( NG = \) Total number of girders
- \( j = \) The number of the loaded lane
- \( NL = \) Total number of lanes

It is important to again reiterate that the distribution factor is meant to scale the expected design moment for one rating truckload to the worst-case scenario of all lanes loaded; hence, the sum of distribution factors for the girders should equal the total number of lanes [Kim and Nowak 1998].

It has become rather conventional in the above literature regarding diagnostic truckload testing to simplify the above formulation for distribution factor by assuming equivalent section
moduli for all girders and under any lane loading. This avoids the entire process of estimating the moment as there is far from any consensus on this matter. The equation then becomes simply a ratio of measured (or projected) strains at the outer face of the bottom flange for the girder and loaded lane under consideration ($\varepsilon_{ij}$). However, the presence of any parapet or sidewalk can lead to significant edge stiffening and an increase in the sectional moduli for the exterior girders. Further, the exterior girders may be different from the interior girders by design. Hence, any lateral variation in the design of the bridge requires the use of moments in the calculation of the distribution factor.

The AASHTO Manual for Condition Evaluation suggests that field measured values of the distribution factor can be compared to and used in lieu of the design specification. This allows for another immediate evaluation of structural condition directly from the truckload test. The measured distribution factor is typically less than estimated by the design code (see Chapter 8). With regards to rating, however, the measured stresses can be used directly to estimate the liveload stresses and moment for the design loads and we avoid the use of a distribution factor altogether in this employed methodology for liveload capacity rating (discussed in Chapter 5).

4.7 SUMMARY

One conceptual signature that represents bridge condition and can be determined from a truckload test is the fundamental structural parameter of the unit influence line, the characteristic response at any instrumented bridge node due to the position of a unit load. An accurate influence line can be determined by crawl-speed truckload tests given the axle weights, spacings, and speed. The derived influence line is consistent for various truckloads and axle configurations. The maximum speed for an accurate estimate is determined from the shortest span length, the natural frequency of the bridge, and the general bandwidth for vehicle-bridge interaction. From the influence line, several conclusions can be immediately drawn regarding the structural condition including the level and consistency of (unintended) composite action, lateral distribution, longitudinal balance of response maxima, impact factor, end restraint, edge stiffening, linearity, and stationarity under load.
Chapter 5 Estimation of Structural Capacity

5.1 RATING ALGORITHMS AND METHODS BASED ON TRUCKLOAD TESTS

The NCHRP Manual for Bridge Rating Through Load Testing [Lichtenstein 1998] simply suggests that the design rating for the bridge could be scaled by the ratio $K_a = \varepsilon_c / \varepsilon_m$, where $\varepsilon_t$ is the maximum strain/stress during the load test and $\varepsilon_c$ is its corresponding theoretical strain/stress due to the test vehicle and its position which produced $\varepsilon_m$. If the measured stress is less than expected ($\varepsilon_m < \varepsilon_c$, which is quite often the case due to the many inherent but unconsidered mechanisms for load distribution), then the bridge rating is increased. However, if unintended (i.e., not designed) composite action occurs between the steel girder and the concrete decking, then the Manual suggests that the new rating be subsequently reduced by the ratio $S_{nc} / S_{fc}$, where $S_{nc}$ is the sectional modulus for the noncomposite steel beam alone and $S_{fc}$ is the sectional modulus for a fully composite deck-on-girder section. Note that $S_{nc}$ is always less than $S_{fc}$. This reduction is equivalent to scaling up the measured stress to discount any effect of the unintended composite action, which might not be sustainable at the higher levels of load required to yield the steel. However, the Manual makes no allowance for partial composite action (see below) and, hence, may be over-penalizing the measured stress. Further, the measured stress (and any linear interpolation of it based upon the required rating loads) is generally sufficient to represent the bridge condition for the majority of serviced loads, especially when one considers the safety factors required by the code (see below).

In any case, further research is required and, until a consensus is reached, all results should be provided for the discretion of the bridge owner. Hence, this project employed several methods to analyze the sectional properties and rating factors for the instrumented locations of the bridge. These very same methods are also applied to the finite element models.
5.2 OVERVIEW OF RATING ASSUMPTIONS AND POINTS OF REFERENCE

The design assumptions of beam flexure and the linear moment-curvature relationship apply to this methodology as well. Another assumption made in this project was that moment (not shear) would be the controlling factor during the superload. In general, this is the case with most bridges; but, in particular for this project, shear was checked by ODOT with the BARS program [BARS 1995] and found not to be a concern for these specimens.

Deadload forces and moments were provided by ODOT from the BARS analysis for the bridge. These values were checked by the research team using various models. For example, the simple beam model was simulated by UCII in Visual Analysis and similar deadloads were obtained. Instead of using any of these results, it was decided to focus the research results on the improved liveload estimates obtained from the field experiments. The provided deadloads by ODOT are assumed for the appropriate case here; for example, a noncomposite design defines Method 4 in all figures and tables. The deadloads for the other methods are calculated.

This analysis was based upon the 17th edition of the AASHTO Standard Specifications (AASHTO-STD) [AASHTO 2002]. There are no subsequent editions for this specification planned by AASHTO.

Unless stated otherwise, the vertical origin will be arbitrarily considered as the outer face of the bottom flange of the steel girder.

5.3 METHODS OF SECTION ANALYSIS

The location of the neutral axis for each instrumented section can be determined from the top and bottom flange strain measurements. The composite action for the instrumented section can itself be checked against the designed performance. Even when designed as noncomposite (i.e., no shear studs on the top flange), the structure will exhibit some level of unintended composite action. The question remains as to “how much” composite action exists and, further, how to rate the bridge accordingly. If the measured data indicates that the structure exhibited some level of composite action, then its sectional indices must be calculated accordingly.
5.3.1 Method 1: Fully Composite with Nonzero Axial Force

An axial force \( P_1 \) is assumed to be acting upon the section such that the centroid \( (y_1) \) is shifted to the neutral axis location. This can be illustrated as a simple shifting to the left or right of the ideal strain profile of the section along the horizontal axis.

5.3.2 Method 2: Fully Composite with No Axial Force

The assumption of the code specification for effective deck width is removed. The effective deck width is recalculated \( (b_{eff2}) \) to force the geometric centroid \( (y_2) \) to coincide with the neutral axis location. This can be illustrated as both a rotation and translation of the strain profile of the section about the vertical axis.

5.3.3 Method 3: Partially Composite with No Axial Force

The assumption is made that the section exhibits partial composite action and there exists a discontinuous break in the strain profile of the section. The assumption of the code specification for effective deck width is enforced. No axial force is allowed for the net section; however, an axial force is assumed to be acting upon the steel beam such that its centroid is shifted to the measured neutral axis location. This can be illustrated as a simple shifting to the left or right of the ideal strain profile of the section along the horizontal axis. An equal but opposite axial force is assumed to be acting upon the concrete decking such that the net axial force in the section is zero. The curvature is assumed to be constant for the entire section. It will be shown below that the estimated moment \( M_3 \) (and, therefore, the section modulus, \( S_3 \)) are very close to that derived by Method 2.

5.3.4 Method 4: Noncomposite with Axial Force

The final method assumes that the section exhibits noncomposite action and no estimation of the strain profile in the decking is attempted. An axial force \( P_4 \) is assumed to be acting upon the steel beam such that its centroid \( (y_4) \) is shifted to the neutral axis location. This is illustrated as a simple shifting to the left or right of the ideal noncomposite strain profile of the section along the horizontal axis.
5.4 REMAINING COMPOSITE ACTION (RCA)

For Method 3, it is convenient during the rating procedure to define a linear condition index that defines the level of composition action for the deck-on-girder system from none (i.e., zero) to fully composite (one or 100%) based upon the national AASHTO-STD code. Turer suggested the ratio of the axial force in the steel beam as measured relative to that predicted by a fully composite section \( RCA = P_{Partial} / P_{Full} \).

\[
\varepsilon_{bgc} = (y_t - y_{bg})\phi, \quad \varepsilon_{tgc} = (y_t - y_{tg})\phi \\
P_{sc} = A_s \cdot E_S \frac{\varepsilon_{tgc} - \varepsilon_{bgc}K_s}{1-K_s} \quad RCA = \frac{P_s}{P_{sc}} \cdot 100\% \quad (5-1)
\]

5.5 STRUCTURAL CAPACITY FOR DESIGN LOADS

The design of a structure is based upon a set of loading conditions which the component or element must withstand. In order to form a consistent national basis for design, organizations such as AASHTO have developed methods for defining component and element capacities and a standard set of loading conditions to be considered against such capacities. In bridge engineering, there are two principal methods for design in use today: working stress and limit states. The intent of all methods is for the entire structure to operate well within the elastic or linear range of the constructed material. The point where a material ceases to behave elastically is defined as the proportional or yield limit. Once stress and strain are no longer proportional, the material enters the plastic range and a permanent deformation will occur to the member.

5.5.1 Working Stress or Allowable Stress Method

For most of the last century, the working stress approach was the standard by which bridges and other structural engineering projects were designed. An allowable stress is defined for each structural member to allow a safe margin below the controlling criterion under normal working conditions. Typically, the allowable stress is a fraction of some failure stress for the given material (e.g., yield or buckling stress for steel, compressive strength of concrete, etc.). For steel-stringer bridges, the allowable stress is typically the yield stress at the bottom flange of the girder \( F_y \) as per the design plans or per Table 6.6.2.1-1 of the AASHTO-CEM) multiplied by a factor of safety \( FS \), defined below.
5.5.2 Limit States or Load Factor Method

By the 1970’s, the limit states approach began to gain acceptance by the general engineering community and especially in the design of concrete structures. Concrete behaves linearly only over about half of its total compressive strength; hence, concrete elements designed under the working stress approach utilize much less than half of their capacity. The limit state approach makes use of the entire plastic range for the design of structural members and incorporates unique safety or load factors ($LF$, defined below) to account for the inherent variability of each loading configuration. Typically, the ultimate moment is defined to be equal to some failure stress for the given section (e.g., shear or plastic moment) which will vary depending upon the level of composite action between the steel girder and the concrete decking. Other strength limits for safe operation and adequacy of the structure include yielding, fatigue, buckling, overturning, etc.

Load and Resistance Factor Design (LRFD) is an extension of the LFD method that was adopted in 1993 by AASHTO as an alternative method for bridge design. The goal was to develop more comprehensive specifications that would eliminate any gaps and inconsistencies in the Standard Specifications, incorporate the latest in bridge research, and achieve more uniform margins of safety and reliability across a wide variety of structures. LRFD has been mandated for all designs using federal funding by October, 2007. Resistance factors now account for the variability of material properties, dimensions, workmanship and uncertainty in their prediction of resistance. Further, many additional issues and analysis have been added or refined by the AASHTO-LRFD Specifications for Highway Bridges (AASHTO-LRFD) [2005] for future designs. However, this approach has not been required for rating purposes and ODOT has no immediate plans for its implementation; hence, a description of this rating method is not provided in this report.

One of the advantages of the limit states approach is that it also takes into account other aspects of the structure’s usefulness and the variability of the public loading environment. Serviceability is the limit state which defines the performance and behavior of the structure in terms of criteria such as deflection, vibration, drift, etc. which may be observable by the public. Serviceability also concerns the history of the structure in its operation (e.g. past overloads) and defines a limit state in terms of yielding but with load factors reduced from the strength limits.
This autostress approach is derived from the inelastic load redistribution or shakedown for continuous structures which occurs following an overload. The overload has a prestressing effect by inducing stresses over the yield point in the negative moment region and relieves some residual stress. The Standard Specifications refer to this limit state as Overload (10.57), while the LRFD Specifications refer to this limit state as Service II (with further reduced factors). The single structural performance requirement at Overload is control of permanent deformations caused by localized yielding and connection slip to ensure good riding quality.

A belief that is fairly widespread in the transportation community is that the working stress method is overly conservative in the analysis of bridges. The AASHTO Guide for Strength Evaluation (AASHTO-GSE) [AASHTO 1989] states that the load factor method is intended to recognize “the large safety margins present” in the more conventional working stress method. In general, a well maintained bridge with redundant load paths will have a higher rating by the load factor method than with the working stress method. However, the AASHTO-GSE emphasizes the use of site specific data and performance histories in the evaluation of the appropriate load factors for the structure; hence, a deteriorated bridge or a design susceptible to certain failure modes can actually be rated lower by the limit states method as opposed to the rote approach of the allowable stress method. The subjectivity of the limit states approach, therefore, places a great deal of responsibility on the design/test engineer.

### 5.5.3 Inventory and Operating Load Ratings

A quantitative benchmark of the performance of any highway bridge has been standardized by AASHTO. It is not a magic bullet and its assumptions and limits are well known by the transportation field. The load rating, like inspection data, is only a gauge of bridge condition and a component in an overall profile of the structure. Unlike the inspection data, however, the load rating is calculated using analytical rather than subjective methods.

One immediate use for the load rating is in the posting of a bridge (i.e., limiting the type and/or weights of vehicles which may pass over the structure). The load rating may also impact any decisions by the transportation department regarding maintenance, rehabilitation, or replacement of the bridge to meet the local, state, and federal requirements for the highway system.
AASHTO differentiates between lower and upper ranges of bridge performance. The lower or inventory range of performance is meant to imply safe use of the highway bridge on a day-to-day basis “for an indefinite period of time”. There are instances, however, when a vehicle has to carry an abnormally large load over the structure. While a structure can withstand these loads on occasion, it is not desirable to have them repeatedly pass over the structure. An upper or operating range is meant to represent the “absolute maximum permissible load”.

The inventory and operating rating factors ($RF$) can be calculated using either the allowable stress or the load factor method and are represented by the ratio of the remaining capacity-to-liveload for the specified design load. For allowable stress method, the remaining capacity is considered as the factored yield stress reduced by any permanent loads (i.e., deadload, $D$, as defined below) upon the structure. For limit states method, the remaining capacity is considered as the ultimate moment reduced by the factored permanent load upon the structure. In the denominator, the liveload ($L$) is increased by the impact factor and, if appropriate, its load factor. A safe structure necessitates that its rating factor is greater than one.

$$RF = \frac{C - LF_1 \cdot D}{LF_2 \cdot L \cdot (1 + IM)}$$ (5-2)

For the Allowable Stress Method: $LF_1 = LF_2 = 1$

- Inventory Rating: $C = 0.55 F_y$ (stress), $C = 0.55 F_y S_{LL}$ (moment, MASinv)
- Operating Rating: $C = 0.75 F_y$ (stress), $C = 0.75 F_y S_{LL}$ (moment, MASopr)

For the Load Factor Method: $C = \text{ultimate moment, } M_{ult}$

- Inventory Rating: $LF_1 = 1.3, \quad LF_2 = 2.17$
- Operating Rating: $LF_1 = 1.3, \quad LF_2 = 1.3$

For the Overload Method: $C = 0.95 F_y$ (composite), $C = 0.8 F_y$ (noncomposite)

- Inventory Rating: $LF_1 = 1, \quad LF_2 = 1.67$
- Operating Rating: $LF_1 = 1, \quad LF_2 = 1$

where: $S_{LL}$ is the section modulus as defined by the appropriate analytical method.
Chapter 5 - Estimation of Structural Capacity

5.6 ULTIMATE MOMENT BY THE FOUR ANALYSIS METHODS

The ultimate moment for a fully composite section (i.e., Method 1) is based upon the property of compactness. Symmetrical I-shaped beams are defined as compact if they possess a high resistance to local buckling and they provide proper bracing to resist lateral torsional buckling. Compact sections will therefore form a plastic “hinge” at the ultimate moment. The AASHTO-STD Specification for compactness of a fully composite design is met if the depth \((D_P)\) for the compressive stress zone or block in the concrete deck is less than the given limit. If the section is determined as compact, then the ultimate moment is defined as that moment which causes the section to completely yield (i.e., the stress in every fiber of the beam and every layer of the decking for its equivalent area of steel will meet or exceed the plastic limit \(F_y\)).

This formulation is applicable to positive moment regions only. AASHTO-STD Specification 10.50.2 states that concrete in negative moment sections shall not carry tensile stresses; however, longitudinal reinforcement can act compositely with the girder.

If the section is determined as noncompact (i.e., \(D_P > 5D'\) above), then the ultimate moment is defined as that moment which causes the first instance of yielding for the section (i.e., at the outer face of the bottom flange of the steel beam), which is substantially less than that defined for a compact section.

The ultimate moment for a compact and noncomposite beam (i.e., Method 4) is defined as \(M_{ult,4} = Z_x F_y\), where \(Z_x\) is the plastic modulus for the steel girder and any existing cover plates. Compactness checks are also relevant for the steel girder itself (Specification 10.48.1, which covers flange width-to-thickness ratio, web height-to-thickness ratio, and lateral brace spacing), but rolled I-beams are typically fabricated to meet these. The plastic section modulus \(Z_x\) may be calculated directly as that moment which causes the beam to completely shear through the section (i.e., the stress in every fiber of the beam will meet or exceed the plastic limit \(F_y\)). The plastic modulus is well documented in the literature for the common beam types used in bridge construction today; however, \(Z_x\) must be calculated for atypical and built-up girders, obsolete girder designs, sections that include cover plates, etc.
If the section fails the compactness test, then the plastic modulus cannot be used. The girder is then examined for braced noncompact condition (AASHTO-STD Section 10.48.2) and, finally, partially braced condition (AASHTO-STD Section 10.48.4). This latter case was not observed in any of the specimens used in this project. Braced sections in positive moment regions use the yield strength as the ultimate moment \( M_{ult} = S_x F_y \); this is also generally true for negative moment regions, however the limit may be further reduced due to its geometry.

### 5.7 DEADLOAD MOMENT BY THE FOUR ANALYSIS METHODS

By definition, the deadload acts upon the noncomposite section and its moment \( M_{DL} \) is determined from first principals for the steel beam and its supports. Here, we use \( M_{DL} \) as determined by the BARS analysis program in order to provide an apples-to-apples comparison with the BARS rating report; however, this would be modified if the deck thickness was found to be different from that assumed by the BARS analysis. The deadload stress is determined from the sectional modulus for the steel beam alone (i.e., \( \sigma_{DL} = M_{DL} / S_{DL}, S_{DL} = I_s / y_s \)). The effective deadload moment depends upon the section modulus at the event under consideration; during the diagnostic truckload test, the section modulus is determined by one of the four analytical methods (see above) and the effective deadload moment is similarly determined (i.e., \( M_{DLeff} = \sigma_{DL} S_{LL}, S_{LL} = \{S_1, S_2, S_3, \text{ or } S_4\} \)).

### 5.8 LIVELOAD MOMENT AND RATING BY THE FOUR ANALYSIS METHODS

For this project, we will specifically examine the HS20-44 liveload classification. Where truckloads generally govern for short span bridges, the laneload generally governs for long and continuous span bridges. The AASHTO-STD specification requires the evaluation of a worst-case scenario where one design load is present in each lane. Typically, the maximum response occurs where the trucks or point loads are side-by-side in the lanes due to the symmetry of the bridge; however, this may not always be the case (e.g., skewed or curved bridges, damaged structures). A reduction in liveload response is allowed for bridges with three or more lanes (i.e., 10% for three lanes, 25% otherwise) to account for the reduced likelihood that this event will actually occur in the lifetime of the structure.
The AASHTO-STD design loads are meant to cover a lateral width of ten feet (3.05 m) within a traffic lane width of twelve feet (3.66 m). No fractional lane widths are to be considered in the load rating. The traffic lanes are considered to be spaced evenly across the width of the structure; however, this is generally not the case due to sidewalks, parapets, and traffic shoulders. The AASHTO-CEM indicates that “when conditions of traffic movements and volume would warrant it, fewer traffic lanes than specified by AASHTO may be considered”. The ODOT Bridge Design Manual [ODOT 2005] indicates that the “traffic lanes to be used for rating purposes shall be the actual marked travel lanes”. In this paper, the latter specification shall be met and only the marked travel lanes will be considered in the following load ratings.

5.9 RATING METHODOLOGY FOR THIS PROJECT

The HS20-44 design loads are all virtually simulated in each marked traffic lane by linear superposition of the derived influence lines, weighted by the specified axle or point weights. Lane load is calculated by integration over that portion of the influence line that would actually magnify the moment for the section (i.e., bottom flange stress should be the same sign as the moment for the section). The liveload is calculated by each of the four analytical methods and then summarily rated by both the allowable stress, load factor, and overload approaches. Inventory and Operating levels are determined for all rating approaches. However, only the ratings by Method 3 are provided in the appendices because it is considered by the authors to be the most representative method for both partial and full composite action. The rating results for the other methods are available upon request.

Although partial composite action is clearly present, it is not clear whether this additional strength for the section would actually exist at the allowable stress. Most certainly, the unintended composite action would not remain at the ultimate or plastic moment of the limit state approach. The Structure Rating chapter of the ODOT Bridge Design Manual [ODOT 2005] indicates that the “members shall be analyzed as to the intended method of design”. In this project, the ODOT specification shall be met and a reduced load rating will also be determined for any section with unintended composite action. The non-composite capacities shall be utilized. Further, the effective deadload and superimposed deadload shall be assigned those moments calculated by first principals for a noncomposite beam.
This modified approach is identical in concept and almost equivalent in practice to that suggested by the NCHRP Manual for Bridge Rating (discussed above in Section 5.1) [Lichtenstein 1998]. The significant difference between the NCHRP Manual and this method for rating reduction is the acknowledgement of partial composite action. The NCHRP Manual suggests using the liveload moment for full composite action, although this would actually overcompensate for a section with only partial composite action (i.e., $M_1 > M_3$).

An example assessment of the critical span for HAM-126-0881 is presented in Figure 5-1. The liveload moments calculated for the simulated HS20-44 truckload for the middle span by the four analytical methods are presented. The load rating for the middle span is selected as the minimum rating by Method 2, since the section was designed and is identified as fully composite. Note the similarities between Methods 1, 2, and 3. Using the very same deadloads, the field ratings based upon the liveload measurements were found to be twice that of the BARS analysis; this is primarily attributed to its conservative estimate of lateral load distribution.

---

**Figure 5-1: Example of Condition Assessment for Critical Span of HAM-126-0881**
5.10 RATING RESULTS AND DISCUSSION

All of the instrumented sections for all of the truckload tests for this project were analyzed and rated by this methodology. Further, the nominal and tuned finite element models were also analyzed and rated by this same methodology. The models were rated only at locations corresponding to truckload instrumentation. The goal was an apples-to-apples comparison; however, it is also quite possible to use the models to investigate other locations for criticality. If a truckload was not conducted for a specific bridge specimen, then the models were tuned to the modal test results and then analyzed and rated by this methodology for comparison against each other and against the results of the BARS analysis.

The results are presented as reports for each structure in the appendices. Note that the critical instrumented span and pier(s) are reported for both liveload and laneload rating. Further, the section properties for these locations are tabularized for comparison with BARS. Note that the liveload moment and centroid location are often larger than assumed because of the presence of unintended composite action. The assumed properties are also tabularized. Note that the moment per unit deadload is fixed with the BARS program, but discrepancies in the actual and designed geometry (e.g., deck thickness) will lead to accompanying differences in deadload and superimposed deadload moments. Finally, the influence lines and simulated HS20 responses for each of the marked traffic lanes for the structure are presented for both the top and bottom flange at these critical locations.

5.11 SUMMARY

Diagnostic truckload testing and instrumented monitoring have proven to be valuable but objective methods for the condition assessment of highway bridges. The constructed bridge will have many inherent mechanisms to resist the applied load and which are generally not considered in the design or analysis of its capacity. The expected liveload stresses and their distribution can be checked and identified by this methodology. Further, the bridge may have been subjected to unexpected or other forces undetermined by the design whose effect may be measured and used in the load rating of the structure. The truckload test results may be used directly or compared against a finite element model of the structure (nominal and/or tuned).
Chapter 6 Finite Element Modeling of Bridges

6.1 BRIDGE MODELER SOFTWARE OVERVIEW

The University of Cincinnati Infrastructure Institute (UCII) bridge modeler software is designed to quickly and accurately create a 3D FE model for a given concrete deck on steel girder bridge. The bridge modeler program was created in the Visual Basic 6.0 programming language. It is set up to run in Microsoft Windows XP, 2000, NT, or 95. The Bridge Modeler software is the combined work of Ahmet Turner, Divyachapan Padur, and Nathan Ruth.

The bridge modeler software has the ability to output a 3D FE model in SAP2000 format. It can also output a .brg file that contains all of the information about the bridge used to create the 3D FE model. The .brg file can be opened by the modeler program. This allows the user to change a given bridge parameter without having to input all of the bridge data.

The modeler software starts with initial values for each data field. A bridge model can be created without inputting any values after selecting each tab. This allows the user to create a model and become familiar with the software. When a tab is selected the bridge modeler initializes the values on that tab. If a tab is not selected, the values for that tab will not be initialized. The modeling software will produce an error if each tab is not selected. This is a built-in error check to ensure the user has checked each tab and corresponding data field value. Each data field starts out as a solid teal color, when a data field is changed the color changes to white. This allows the user to visually see which values have been changed. For each tab, a different pictorial representation of the bridge during the model creation process is shown. Every tab of the software allows the user to input a certain detail about the bridge and a pictorial representation depicts how the values reflect upon the bridge. Different sectional views of the bridge are provided to the user. This helps the user to understand the process and the options of modeling available and also reduce errors of omission.
The modeler software has built in error checking at every stage of the user input process. All of the text fields are protected against non numerical and negative values. The skew is the only field that allows negative numbers. A right forward skew is inputted as a negative number and a left forward skew as a positive number. Critical parameters are set to be within bounds. The sidewalk for example can not be set to be more than the actual width of the bridge.

6.2 MODEL CREATION PROCESS

The model creation process consists of the following steps, each of which is explained in detail in the sections that follow.

6.2.1 Span Data

The user can either start entering values into the fields directly or can open an existing bridge model file (.brg file). Existing models can be opened using the keyboard shortcut Ctrl+O or from the File menu. After the splash screen the modeler will open with the “Span Data” tab. The screen shot below shows the first tab.

![Figure 6-1: Span Data Entry Screen](image)
The default value in the number of spans text field is one and since it is changed to two, the background of the field is white. The skew angle, total width, and fixed support are the default values and hence retain the teal background. The default value for the span length is 50 ft. (15.24 m). The user can change the values in the respective fields with information from the bridge plans. The picture shows the top down view of the bridge. The bottom left side corner of the bridge from the picture is the origin point of the model and all measurements in the X and Y directions are from that point. The actual bridge is not always oriented the same way as the programs’ picture. The user must make sure the skew angle is appropriately chosen to make the top down view of the picture looks like the same as the actual bridge. A right forward skew is entered as a negative value and a left forward skew as a positive value. The fixed support is the abutment or pier of the bridge to be fixed, while allowing the others to move as if on rollers. If the user chooses to open an exiting file, the program loads the bridge details from the file and also generates the appropriate picture in every tab. It is always a good idea to check all the values before proceeding to create the model.

### 6.2.2 Section Data

This tab takes input regarding the girders, slab, sidewalk and parapets. The user can change the default values as per the bridge plan and can see the sectional view of the bridge update as the values are being inputted. One important aspect is that the diagram in the picture is to scale. The proportions are maintained to present an accurate view. If there are no parapets or sidewalks, the appropriate check boxes can be selected. If there is no sidewalk on top of the bridge, the concrete underneath the deck is simulated as a sidewalk on top of the deck. This way the mass of the concrete can be taken into account in the model. This is usually the case for a bridge that has a railing instead of sidewalks and parapets. Railings are not simulated.

The default option in girders is that they do not have multiple girder sections. Under the no multiple girder section option, the user can define each of the girders individually. To choose girder types from the predefined database, the user needs to right click inside the grid. That brings up a list box from which the predefined girder types can be chosen. In case of more than one girder being of the same type, the user can do a multiple select by holding down the “SHIFT” key and choosing the first column of the required girder number row in consecutive
rows. The screen shot below shows the multiple select with the database list box. Girders 2, 3 and 4 of the bridge are of the type W44 x 224.

![Image of Section Data Entry Screen](image-url)

Figure 6-2: Section Data Entry Screen

### 6.2.3 Cross Brace Data

The number of cross braces, distances between cross braces, type of cross brace, and end cross brace information is entered in this section. Most bridges have evenly distributed cross braces. When the bridge has unevenly spaced braces, the “Even Spacing” button must be unchecked. The individual distances have to be inputted into the grid boxes. Once all the distances have been entered, the “UPDATE” button must be clicked to update the picture and also register into the program. The cross braces can be user defined or can be predefined types [Austin et al. 1992] that can be selected from the drop down box. The same applies to end cross braces too. Through calibration, it has been determined to always simulate end cross braces, even if there are not any on the bridge. Figure 6-3 shows the cross brace data input tab of the UCII Bridge Modeler. The user must make sure that the values are entered in the appropriate units. The picture in the tab is a good way to determine if the numbers are entered correctly.
6.2.4 Meshing Data

The user can determine the size of the model that is to be created. The user can change the number of shell elements between girders on the deck and also the number of shell elements to define the web of the girder. There is constraint in the number of nodes that can be defined in SAP90 models (9999 nodes only). The size of the model can therefore only be varied only in a short interval. For a very long or wide bridge an error may occur then creating the SAP90 model. This error states that the model has too many nodes. If this occurs, the mesh size can be changed in order for the model to use a smaller number of nodes. Figure 6-4 shows the meshing data input tab of the UCII Bridge Modeler. The default value for the number of shells between girders in four and the number of shells to define web of girder is two.
6.2.5 Cover Plate Data

The lengths of the cover plates and their respective sectional information are entered. The picture on the tab in Figure 6-5 provides a visualization of the information. It helps the user get a perspective of how cover plates are modeled. The assumption is that the lengths of cover plates over girders at the piers are the same in every girder. The sectional properties at every pier-girder section can be defined differently.
6.2.6 Loading Data

This tab is used to input values to generate the truckload. The truck parameters and moving information is input into the respective fields. The user has the option to generate uniaxle loads, standard trucks, or custom trucks. A uniaxle load can also be moved in both the directions along the bridge. The default is a custom truck. The user can change the custom details by changing a file called the “Axles.dat” file in the Bridge Modeler folder. The moving information contains the number of steps and the step size. The effective bridge length is the length of the bridge plus the length of the truck.
The offset determines the truck path. It is essential to estimate this distance correctly from the bridge plans. The sidewalk width and lane width is obtained from the bridge plans. The offset distance is the position of the front right side wheel when the truck moves from left to right in the picture. It is assumed that during truck tests the truck is driven in a straight line in the center of each lane. The same setup must be reproduced so that the load simulation in the mode provides results that can be used to calibrate the model against truck test data.

The diagram in the picture box shows a top down view of the bridge with the position of the truck, as per the user inputs, marked on the bridge. The truck is represented as a circle whose diameter is the truck width.
Chapter 7 Finite Element Model Calibration

7.1 CALIBRATION OVERVIEW

A 3D FE model is used to construct the model of a bridge in three dimensions, as in reality. The geometry of the bridge can be well modeled with it, getting closest to the actual bridge structure. Although it takes longer time to do analysis than 1D and 2D models due to the large number of elements, nowadays with good configured computers, the analysis time is acceptable and will not delay the research.

In the modeling strategy, four-node shell elements are used to define the concrete deck in the model. Frame elements are used to define the top and bottom flange of the steel girders. The composite actions between deck and girders are simulated with rigid links connecting deck and girders. [Li 2003]. Shell elements are also used to simulate the piers on bridges.

A model of a bridge is efficiently generated by an intuitive GUI modeler software. A set of bridge physical parameters such as the length of the bridge, the number of girders, sidewalk, and the type of the beams will be provided to the modeler software and a nominal bridge model is created. The model can be analyzed and visualized in SAP2000 software [SAP2000 1995]. This initial model is called the nominal model.

Normally differences exist between the tested dynamic properties of a bridge and the relevant dynamic properties in its nominal model. It is because of some uncertainties about the materials and the dimensions obtained from the bridge plans. Those uncertainties include the changes in the deck thickness, different composite actions between deck and girders, different modulus of elasticity of concrete due to the different concrete conditions, deterioration on the bridge, etc. So even with a well-prepared 3D FE model, those factors above will still affect the simulation and rating results. [Turer 2000]. Therefore, a calibration to the model is very necessary for better prediction and rating of bridge performance.
During the calibration, the nominal model is tuned to match the current condition of the bridge. FE parameters in the model will be modified. The field test data from the same bridge is used for the reference of the model tuning. The field test data includes dynamic modal information for the bridge and the strain responses from the truckloading. From the analysis of the model, the modal and truckload responses of the model are compared to the test data. During calibration, the modal and truckload performance of the model will become more like the field test data; therefore, the model is becoming more and more like the real bridge in this context. Finally, the condition of the bridge can be evaluated and rated using the calibrated model.

7.2 OBJECTIVE FUNCTION OVERVIEW

When it comes to computer-based calibration, an Objective Function (OF) must be specified. This objective function must have the ability to appropriately measure the experimental-analytical differences to correctly represent the calibration status. In the calibration program, any change to the FE parameters to affect static and dynamic performances of the model will be recorded, and the corresponding objective function value will be calculated by the computer. With the information above, a convergence algorithm will be working to optimize the OF therefore minimize the experimental-analytical differences.

7.2.1 Global Objective Function

A Global Objective Function (GOF) is used to evaluate the difference between the test data and the analytical data during the calibration process. The GOF is a summation of four different terms: Bridge Girder Condition Indicator (BGCI, Section 7.2.2), Modal Assurance Criterion (MAC, Section 7.2.3), natural frequencies (Section 7.2.4) and Unit Influence Line (UIL, Section 7.2.5). Each term is evaluated by a specific objective function: \( OF_{BGCI} \), \( OF_{MAC} \), \( OF_{Freq} \), and \( OF_{UIL} \). They are different objective functions with different magnitudes; therefore they can not be directly added together in the GOF. A weighting factor is multiplied to each OF. Otherwise, the OF which has much bigger magnitude value will dominate the GOF and other OF’s will be shadowed.

\[
GOF = w_1 \cdot OF_{BGCI} + w_2 \cdot OF_{MAC} + w_3 \cdot OF_{Freq} + w_4 \cdot OF_{UIL}
\]  

(7-1)
where:

\[ OF_{BGCI} = \text{the objective function of BGCI} \]
\[ OF_{MAC} = \text{the objective function of MAC} \]
\[ OF_{Freq} = \text{the objective function of frequency} \]
\[ OF_{UIL} = \text{the objective function of UIL} \]
\[ w_1 = \text{the weighting factor of } OF_{BGCI} \]
\[ w_2 = \text{the weighting factor of } OF_{MAC} \]
\[ w_3 = \text{the weighting factor of } OF_{Freq} \]
\[ w_4 = \text{the weighting factor of } OF_{UIL} \]

When the calibration is running, the program will calculate the GOF value for each iteration. It shows globally how well the calibration is doing.

UCII has been performing two kinds of bridge test: modal test (to extract the experimental mode shapes, frequencies, flexibility matrix, generate BGCI) and truckload test (to extract experimental UIL). Bridges that have both modal test data and truckload test data available, Equation (7-1) is used for the GOF. But for some other bridges, only a modal test data is available. In this case, only modal test data can be used for calibration reference therefore the GOF equation will be:

\[ GOF = w_1 \cdot OF_{BGCI} + w_2 \cdot OF_{MAC} + w_3 \cdot OF_{Freq} \]  
(7-2)

The weights that associated with those OF’s in Equations (7-1) and (7-2) are important while flexible as well. The objective functions for BGCI, MAC, frequency, and UIL are different. The OF values are not of the same order of magnitude. The weighting factors are used to adjust each OF contribution, or weight, to the GOF. With the weighting factors properly set, the OF’s will have a similar magnitude and have a similar or equal influence on the GOF. This is the principle to set weight: to balance their influence to the GOF. However, sometimes those four terms are really not equally important. The weights can be set so that the one or more OF’s have a greater influence on the total GOF then the other OF’s.
Currently, for the calibration processes over ten bridges, the weights are decided by four integers that can balance their influences. For example, the weights used in BUT-732-1043 bridge calibration are a 60-2-2-15 combination:

\[ GOF = 60 \cdot OF_{BGCI} + 2 \cdot OF_{MAC} + 2 \cdot OF_{Freq} + 15 \cdot OF_{UIL} \]  

(7-3)

The choice of these four weights has not been standardized, other than to define them as integers. The weightings are chosen to arrive at a balanced contribution of each objective function to the GOF. If GOF value is considered as a whole pie, then the ratios of \( OF_{BGCI}, OF_{MAC}, OF_{Freq}, \) and \( OF_{UIL} \) are displayed in Figure 7-1. It should be noted that the weights have meaning relative to each other; hence, a 60-2-2-15 combination can lead to the exactly same calibration results with the new weights 30-1-1-7.5.

![Figure 7-1: The Weighted OF’s Ratios in the GOF at the Beginning of BUT-732-1043 Calibration](image)

These ratios can be adjusted by increasing or decreasing their weights. The ratio of \( OF_{Freq} \) is usually set to be relatively smaller, it is because the frequency match between model data and test data is usually not bad at the beginning of the calibration and the exact values of experimental frequencies are not very accurate from the post processing of test data. For these reasons, a smaller \( OF_{Freq} \) is adequate for the calibration purpose. If the tuning to \( OF_{Freq} \) is over considered, it might influence the tuning of different objective functions.
7.2.2 BGCI Objective Function

To evaluate how well the analytical and experimental BGCI match each other, a function is used:

\[
J = \frac{\sum_{i=1}^{n} |\Delta_{\text{exp}}(i) - \Delta_{\text{ana}}(i)|}{n} \quad (7-4)
\]

where:

\( \Delta_{\text{Exp}}(i) \) = the \( i^{th} \) experimental BGCI node deflection
\( \Delta_{\text{Ana}}(i) \) = the \( i^{th} \) analytical BGCI node deflection
\( n \) = the total number of nodes (accelerometers) in the BGCI

This is the BGCI objective function (\( OF_{BGCI} \)) that is used in the model calibration. It is the average absolute difference between the experimental and analytical BGCI. In Figure 7-2, there are 38 sensors. As a result, there are 38 points, so \( n \) is 38, and \( \Delta_{\text{Exp}} \) is a vector.

![Figure 7-2: Example of Experimental BGCI](image)

Generally the analytical BGCI has more nodes than the experimental BGCI; this causes the original vector to have more elements. An interpolation process is necessary for the analytical BGCI before the BGCI objective function is applied. After interpolating the analytical BGCI into the location where the accelerometers were placed, the interpolated BGCI can be used to calculate the \( J \).
Suppose there are four sets of data from four load conditions:

1) Deflections of girders 2 and 3 when the load is on Girder 2 \((J_1)\)

2) Deflections of girders 2 and 3 when the load is on Girder 3 \((J_2)\)

3) Deflections of girders 3 and 4 when the load is on Girder 3 \((J_3)\)

4) Deflections of girder 3 and 4 when the load is on Girder 4 \((J_4)\)

If all the data above is considered in the calibration (it is the default choice in the calibration program), the BGCI objective function will be:

\[
OF_{BGCI} = \frac{J_1 + J_2 + J_3 + J_4}{4}
\]  

(7-5)

The process to calibrate the BGCI performance of the bridge is the process to minimize its objective function.

### 7.2.3 MAC Objective Function

The modal test data results give a set of mode shapes and their corresponding frequencies. They are placed into a matrix by column. If there are \(m\) mode shapes, and each mode shape vector is a \([p \times 1]\) vector, then the matrix \(\Psi_{Exp}\) is a \([p \times m]\) matrix. The model is simulated using the SAP2000 software package. The analytical mode shapes and frequencies can be obtained from the output files. These analytical mode shapes are placed into another matrix, \(\Psi_{Ana}\). If there are \(n\) mode shapes, then \(\Psi_{Ana}\) is \([p \times n]\).

The objective of MAC calibration is to make the analytical modes as similar as possible to the experimental ones. To do this manually, one can simply look at the experimental mode plot to determine its type, and then go through analytical mode plots to find the matching one very easily. But in the computer program, this matching procedure has to be done automatically.

For every mode in the experimental mode matrix \(\Psi_{Exp}\), there has been found a matched mode in \(\Psi_{Ana}\). Let an array \(R\) be the index array for those matched modes in \(\Psi_{Ana}\), (i.e. \(\Psi(1)_{Exp}\) matches \(\Psi(R(1))_{Ana}\), \(\Psi(2)_{Exp}\) matches \(\Psi(R(2))_{Ana}\), etc.). The MAC objective function will be:
Usually \( q \) will be a value around ten, because the first ten experimental modes can be clearly identified. Also, from the previous research by Lenett, it is showed that those low frequency mode shapes play a more significant role in the bridge flexibility computation. The modes with high frequencies don’t contribute much to the flexibility matrix [Lenett 1998].

Usually there are two sets of test data for one bridge. Each set is corresponding to one half of the bridge test. The final \( OF_{MAC} \) is the arithmetic average of both sides (left side and right side).

\[
OF_{MAC} = \frac{OF_{MAC_{left}} + OF_{MAC_{right}}}{2} \tag{7-7}
\]

From the MAC objective function \( OF_{MAC} \) one can see, since \( MAC_{i,R(i)} \) values should be as high as possible (also remember the maximum MAC value is 1), the smaller \( OF_{MAC} \) is, the better analytical modes are matching up with the experimental modes.

### 7.2.4 Frequency Objective Function

Figure 7-3 illustrates the mathematical solution for this objective. It is a plot of nine points with a linear regression trend line superimposed. The x-axis shows the experimental frequency values, and the y-axis shows the analytical frequency values. Each point uses one experimental frequency as the x value and the corresponding analytic frequency, which shares the same mode type, as the y value. A trend line is plotted for those points. In order to make the analytical frequencies match test frequencies, the slope \( S \) and correlation \( R^2 \) of the trend line must be considered. The objective function of slope \( OF_S \) in the calibration is:

\[
OF_S = \begin{cases} 
1 - S & \text{if } S \leq 1 \\
1 - \frac{1}{S} & \text{if } S > 1 
\end{cases} \tag{7-8}
\]

The smaller \( OF_S \) is, the closer \( S \) is to one, and the better frequencies match.
Chapter 7 - Finite Element Model Calibration

Figure 7-3: Frequency Objective Function Plot

Only the slope can not always guarantee the match of the frequencies. The points can be scattering around the trend line while the slope is perfectly 1 so the $R^2$ value needs to also be considered. It is the square of Pearson product moment correlation coefficient through two data sets. The maximum $R^2$ value is 1. For the value of 1, all the points that form the trend line will be perfectly colinear.

Suppose $F_{Exp}$ is the experimental frequency vector, $F_{Ana}$ is the analytical frequency vector, the size of both vectors is $[n \times 1]$, then the formula for $R$ is:

$$R = \frac{n(F_{Exp} \cdot F_{Ana}) - (\sum F_{Exp})(\sum F_{Ana})}{\sqrt{[n\sum F_{Exp}^2 - (\sum F_{Exp})^2][n\sum F_{Ana}^2 - (\sum F_{Ana})^2]}}$$  \hspace{1cm} (7-9)

The objective function of $R^2$ in the calibration is

$$OFR^2 = 1 - R^2$$  \hspace{1cm} (7-10)

The smaller $OFR$ is, the better the frequency points congregate around the trend line.

For the frequency objective function, both $OFR$ and $OFS$ need to be considered. So the whole frequency objective function $OF_{Freq}$ is:
Again, remember there are two sets of test data for one bridge; each set is corresponding to one half of the bridge test. The final $OF_{MAC}$ is the arithmetic average of both sides (left side and right side).

$$OF_{Freq} = \frac{OF_L + OF_R}{2} \tag{7-11}$$

7.2.5 UIL Objective Function

In Figure 7-4, one can see the difference between those two UIL curves. The most important factor is peak value. If the peak values match, then those two UIL curves will basically match each other. The UIL objective function is designed based on this idea:

$$OF_{UIL} = \frac{\sum_{i=1}^{n} \left[ \text{Max}(UIL_{Exp,i}) - \text{Max}(UIL_{Ana,i}) \right]}{n} \tag{7-13}$$

where:

- $n$ = the number of sensors (Strain gages) on the bridge, each sensor records a set of UIL data
- $UIL_{Exp,i}$ = the experimental UIL data on the $i^{th}$ sensor
- $UIL_{Ana,i}$ = the analytical UIL data on the $i^{th}$ sensor

The more loads are used in the truckload simulation, the more precise the result is. The other factor that needs to be considered is time. Due to the time consumption of the calibration, if too many loads are used, the time to finish a whole calibration will be tremendously extended. Since only the peak values are considered in the objective function, the loads that definitely will not bring the peak stress values can be removed to save the analysis time. The sensors are normally located at the middle of spans, so only the loads around the middle of the spans are used. After removing some loads, the load condition looks like Figure 7-4. Only the loads around the middle of each span are added. This is enough for the $OF_{UIL}$ calculation.
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7.3 CALIBRATION PARAMETERS OVERVIEW

FE modeling is a very powerful tool for bridge evaluation and rating. A model generally is built according to the bridge plans and drawings, thus this nominal model can represent the geometry of the real bridge well. But a model should not only provide the geometric similarity for a real bridge, it should have similar static and dynamic properties as the real bridge. These properties (such as natural frequencies, mode shapes) reveal more information about the performance and health status of the bridge. For this reason, the nominal bridge model does not perfectly represent the real bridge in most cases. In other words, it does show the static and dynamic properties in quality, but in quantity consideration, it can be further refined.

Figure 7-5 is an example that shows the reason of calibration. In the figure, there are three curves of mode F-111-O, which is usually the first or natural mode of a three-span bridge. The black curve is from the nominal model, the blue one is obtained from field test and the pink curve is the same mode from the calibrated model. From this plot, it is clear to see that after calibration, the mode F-111-O in the model is a better match with the same mode in test data.
This is the objective of calibration. By calibrating the model, the model’s static and dynamic properties more closely represent the real world bridge as determined by field tests. Once the parameter refining (calibration) is done, the calibrated model can be used to perform live or dead loading for the bridge rating. The calibrated model reflects the current health status of the real bridge better than the nominal model, so using the calibrated model will give more accurate loading results.

There are thousands of finite element parameters that can affect the behaviors of the model. For the consideration of calibration efficiency, there is no need to calibrate all of the parameters in the model. Some of the parameters are more definite, well-known, they will not be changed. For example, the geometry of the bridge, steel material properties, etc. If these parameters are changed, the bridge is no longer the same bridge as that on the bridge plan. But some other parameters are really variable such as the boundary conditions of the bridge, composite action, and some geometric properties that may change during service. They are not well determined so they need to be calibrated in order for the results for the model to closely match the experimental field test results.

So these parameters can be considered as the variables in the objective function, the GOF tuning process then becomes to the process that adjusts those parameters to the right values which can minimize the objective function.
With the research results about parameter sensitivity study above, the following parameters are identified to be the most sensitive parameters to the GOF therefore they are tuned in the computer calibration program:

1) the Area of the Support Links ($A_{SL}$)

2) the Moment of Inertia of Support Links ($I_{SL}$)

3) the Modulus of Elasticity of Concrete ($E_C$)

4) the Thickness of Deck ($t_d$)

5) the Modulus of Elasticity of Steel ($E_S$)

6) the Moment of Inertia of Rigid Links ($I_{RL}$)

Figure 7-6 is a cross section picture of a bridge. The red color part is the shell elements representing one of the piers beneath the bridge superstructure. $A_{SL}$ and $I_{SL}$ are located on the support locations of the bridge; they are the support springs of the bridge. $E_C$ and $t_d$ are the parameters physically located on the deck. $E_S$ is a parameter about girder property and $I_{RL}$ is the stiffness of rigid links between the deck and top flange.
7.4 CALIBRATION ALGORITHM OVERVIEW

To find the minimum value of the GOF, a gradient-searching based algorithm was developed. In the calibration, every parameter will be searched in the allowed calibration limitation. Eventually a vector of variables for each parameter is found so that this vector can make the GOF value as low as possible.

Figure 7-7 below shows the idea of the steepest descent algorithm in bridge model calibration. Suppose that the X axis and Y axis in the chart represent the FE parameters, and the Z axis is the value of global objective function (GOF). This chart shows how the different FE parameter values will make the objective function value different. The smaller GOF value means the smaller difference between model analysis data and test data. The goal is to find the appropriate FE parameter values that can minimize the GOF. In the real bridge model calibration, usually the FE parameters are multi-dimensional. In this case, the searching for the optimized parameters will be in the multi-dimensional parameter hyper plane instead of in the 3D plane in Figure 7-7.

Figure 7-7: Illustration of the Relationship Between FE Parameters and the GOF
Figure 7-8 shows a parameter hyper plane in Plainview. This is just an illustration to show the idea of steepest descent algorithm so it is only a 2D hyper plane. In reality, like mentioned before, the dimension should be very high (315 dimensional hyper plane for BUT-732-1043 IRL calibration). In the hyper plane, each point stands for a vector of moment of inertia of rigid links $I_{RL}$ in the model and each $I$ corresponds to a $f(I)$. The algorithm is:

1) Select an initial point $I_0$.

2) Find the gradient of the GOF at that point (the red arrow in Figure 7-8).

3) Proceed along the gradient direction with a step size.

4) When it reaches the next point, repeat doing the step 2 and 3.

Finally, the searching in this “rigid links” hyper plane will approach the bowl bottom (the point which has the minimum function value, in Figure 7-7, it is the deepest blue area). How well the final point is close to the theoretical minimum point (the asterisk in Figure 7-8) depends on the step size.
7.4.1 Calibration Flow Chart

The calibration algorithm, data manipulation and analysis are implemented in an integrated computer program. According to the flowchart in Figure 7-9, the calibration for the parameter Area of Support Links ($A_{SL}$) is carried on first. After the tuning for $A_{SL}$ is done, the tuning for Moment of Inertia of Support Links ($I_{SL}$) continues. Then Modulus of Elasticity of Concrete ($E_C$) calibration, Thickness of Deck ($t_d$) calibration, Modulus of Elasticity of Steel ($E_S$) calibration, the last one is Moment of Inertia of Rigid Links ($I_{RL}$) calibration.

Figure 7-9: Overall Calibration Flowchart
It is important to investigate the sequence of the parameter calibration. The decision of this tuning sequence is based on the study for parameter sensitivity [Li, 2003] and the research experience in manual bridge model calibration [Wang, 2005].

After one whole round of calibration is done, i.e. after $I_{RL}$ calibration is done, the calibration will go back to the very beginning ($A_{SL}$ calibration) again and perform the second round. It is because that the parameters’ calibration is actually related to each other so the changes of parameter values will change the calibration initial condition for other parameters. In the second round calibration, only the calibration to highest sectioning resolution of each parameter is performed in order to keep the calibration result from the first round. After the second round of calibration, a further decreased GOF can be seen normally. If a relatively high calibration contribution is showing because of the second round of calibration (i.e. the whole second round calibration makes the GOF decreased more than 10% in the total GOF calibration), a third round calibration can be considered. Although the third round usually only contributes a little (3%, for example) in the GOF calibration.

During this multiple round of calibration process, all the parameters will be interactively adjusted. This strategy can really make the model simulate the real bridge as well as possible. This process not only further calibrates the parameter values but also alleviates the potential impact of calibration sequence to the calibration result.
Chapter 8 Lateral Load Distribution Factors in Steel Stringer Bridges

8.1 BACKGROUNDS AND INTRODUCTION

As of 2003, 27.1% of the nation's bridges (160,570) were classified as structurally deficient or functionally obsolete, an improvement from 28.5% in 2000 [ASCE 2005]. In fact, over the past 12 years, the number of bridge deficiencies has steadily declined from 34.6% in 1992 to 27.1% in 2003. The evaluation of these deficient bridges is performed using various techniques. The most established and largely used technique is visual inspection. Condition ratings are assigned by qualified bridge inspectors to the primary bridge components (deck, superstructure, and substructure) based on National Bridge Inspection Standards. The condition ratings, which range from 0 to 9, describe both the degree of bridge deterioration and the extent to which it is distributed throughout the structure's components. However, it is difficult to comprehensively and accurately evaluate a bridge based on the visual inspections [FHWA 1993]. Another approach that could be used for bridge condition evaluation is through the use of objective, scientific evaluations of condition rather than subjective approaches such as visual inspection. Field tests (e.g. truckload and modal) (Chapter 3 and Chapter 4) and finite element modeling (Chapter 6) form a set of tools to be used in this approach.

The University of Cincinnati Infrastructure Institute (UCII) has achieved much progress in the area of bridge condition assessment based on these testing methods and has already tested numerous bridges to verify these techniques (Chapter 2). UCII has used the tool of field testing and has developed objective functions for calibration of finite element (FE) models (Chapter 7). In the state of Ohio, many bridge structures have been assessed by using the techniques outlined in the earlier parts of this report. The focus of this chapter is to tie together the work completed in the field and in the laboratory to provide recommendations for improved condition assessment and structural capacity estimation.
8.1.1 Load Rating

It has been observed that load ratings based on field testing usually yield higher rating than ratings based performed in the absence of field data. This has been demonstrated by a number of load tests done by UCII and by other researchers [Barker 2001]. A number of factors are known to be responsible for this increase and they have to be studied before accepting the ratings from the field. Some of these are (1) wheel load distribution, (2) support fixity, (3) unintended composite action, and (4) effects of non-structural members. The first of these factors, wheel load distribution, is the focus of this chapter.

The liveload value, \(LL\), is directly related to the lateral load distribution of the bridge. The most widely used method of estimating the amount load distribution present is through the use of simplified equations (discussed in the following section) that yield conservative estimates. For a design type of problem, use of these conservative equations may be justified. This conservatism is uncalled for in a load rating problem, however, where an accurate estimate of actual capacity is required. The objective of this chapter is to develop a means of more accurately evaluating the structural capacity of a bridge in the absence of field data. This objective will be achieved by deriving modification factors to be applied to accepted equations used in design so that computed distribution factors are more appropriate for rating.

8.2 COMPARISON OF DISTRIBUTION FACTORS FROM VARIOUS SOURCES

8.2.1 Methodology Employed in This Investigation

Seventeen bridges from the total database of 40 specimens used in both this research project and its companion project were selected (see the Forward, page xi). Consideration was given to select bridges in a way to represent the trends in the ODOT bridge inventory. For example, of 17 bridges selected, ten are 3 span continuous, six are 4 span continuous, and one is 2 span continuous, which is the generalized trend representing the statistical population of stringer bridges in the state of Ohio. A description of parameters for each of the selected bridges is given in Table 8-1. Table 8-2 shows the status of truckload test and modal test for these bridges.
<table>
<thead>
<tr>
<th>Sr. No</th>
<th>Bridge Name</th>
<th>No. of Spans</th>
<th>No. of Traffic Lanes</th>
<th>Skew (Degr)</th>
<th>Total Width (feet)</th>
<th>Total Length (feet)</th>
<th>Symm † (Y/N)</th>
<th>Lane widths</th>
<th>Span 1 Length (feet)</th>
<th>Span 2 Length (feet)</th>
<th>Span 3 Length (feet)</th>
<th>Span 4 Length (feet)</th>
<th>Deck Thickness (in)</th>
<th>No. of Girders</th>
<th>Beam Spacing (ft)</th>
<th>Xframe Spacing (ft)</th>
<th>Parapets/Railings</th>
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<td>10.833</td>
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<td>2</td>
<td>10.00</td>
<td>38.000</td>
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<td>80.00</td>
<td>56.00</td>
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<td>2</td>
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<td>0.00</td>
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<td>79.50</td>
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<td>13.500</td>
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<td>62.50</td>
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<td>10.333</td>
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<td>7.00</td>
<td>53.000</td>
<td>152.50</td>
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<td>7</td>
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<td>36.000</td>
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<td>10</td>
<td>48.00</td>
<td>60.00</td>
<td>48.00</td>
<td>0.00</td>
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<td>126.88</td>
<td>78.42</td>
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<td>70.00</td>
<td>70.00</td>
<td>49.00</td>
<td>9.00</td>
<td>4</td>
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<td>42.00</td>
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<td>2</td>
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<td>81.50</td>
<td>57.00</td>
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<td>15.000</td>
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<td>2</td>
<td>0.00</td>
<td>46.000</td>
<td>221.50</td>
<td>Y</td>
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<td>49.00</td>
<td>69.50</td>
<td>60.50</td>
<td>42.50</td>
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<td>8.500</td>
<td>13.840</td>
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<td>2</td>
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<td>36.333</td>
<td>156.00</td>
<td>Y</td>
<td>10</td>
<td>48.00</td>
<td>60.00</td>
<td>48.00</td>
<td>0.00</td>
<td>9.00</td>
<td>5</td>
<td>8.000</td>
<td>11.167</td>
<td>P</td>
</tr>
</tbody>
</table>

† Symmetric indicates that the center line of traffic lanes and center line of girder configuration is same.

‘R’ indicates railing only, ‘P’ indicates parapet and sidewalk, ‘P*’ indicates parapet only
Table 8-2: Modal and Truckload Test Status on Specimen Bridges

<table>
<thead>
<tr>
<th>Sr. No</th>
<th>Bridge Name</th>
<th>Truck Load Test</th>
<th>Modal Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>10</td>
<td>PRE-503-1170</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>11</td>
<td>FAI-33-1309</td>
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<td>-</td>
</tr>
<tr>
<td>12</td>
<td>MOT-70-0553</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>13</td>
<td>AUG-75-0201</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>14</td>
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<tr>
<td>17</td>
<td>CLI-132-0083</td>
<td>-</td>
<td>X</td>
</tr>
</tbody>
</table>

Seven of the ten low-rated bridges (# 7, 9, 10, 12-14, and 17 in Table 8-2) tested for this project are examined. Distribution factors were computed for each of the seventeen bridges by using all the refined sources and standard equations and the values thus obtained were compared.

8.2.2 Computation of Distribution Factors for the Investigation

The lateral distribution of moment can be computed in the following five ways:

1) \( DF_{STD} \), as defined by the equation used in AASHTO-STD

2) \( DF_{LRFD} \), as defined by the equation used in AASHTO-LRFD

3) \( DF_{Nom} \), as defined by the nominal FE model for the structure

4) \( DF_{Tuned} \), as defined by the tuned FE model for the structure

5) \( DF_{Fields} \), as defined directly from field tests of the structure
These five methods of determining the lateral load distribution characteristics can be conveniently grouped into three categories—Predicted (AASHTO-STD and LRFD equations for lateral load distribution, $DF_{STD}$ and $DF_{LRFD}$), Analytical (nominal and tuned FE models, $DF_{Nom}$ and $DF_{Tuned}$), and Measured (truckload tests, $DF_{Field}$).

Although provisions are included in both the AASHTO-STD and LRFD specifications for lateral load distribution of shear forces, too, this discussion is limited to the lateral load distribution of bending moment. Hence from each of the above sources, four values can be determined:

1) $DF_{M1,EXT}$ = DF for moment for an exterior girder due to one lane loaded.

2) $DF_{M1,INT}$ = DF for moment for an interior girder due to one lane loaded.

3) $DF_{M2,EXT}$ = DF for moment for an exterior girder due to two or more lanes loaded.

4) $DF_{M2,INT}$ = DF for moment for an interior girder due to two or more lanes loaded.

**8.2.3 Summary of Results**

The distribution factors calculated by the various methods are thus obtained for each girder, and the maximum value is used as the DF representing the bridge. A variation of bottom flange stresses across the girders for CLE-52-0142 is shown below. As seen in Figures 8-1 and 8-2, the maximum response is seen by a girder when the live load is directly or closely placed over it. When lane #1 was loaded, the girders underneath the load showed maximum response. Most bridges in this study have the girder locations symmetrical with respect to the center line of the traffic (refer to Table 8-1). Hence, the numerical values of the stress responses for both lane 1 and lane 2 loaded are similar.
Chapter 8 – Lateral Load Distribution

The distribution factors calculated using the various methods, are shown in Tables 8-3 through 8-6. As evident from these tables, the maximum DF was obtained for all the bridges by using the AASHTO-STD equation for interior girders. The AASHTO-STD approach has been found to be the most conservative approach to calculating distribution factors. The AASHTO-LRFD approach yields the second highest values for the DF in most cases.
### Table 8-3: Distribution Factor $DF_{M1, \text{INT}}$ from Various Methods

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>$DF_{M1, \text{INT}}$</th>
<th>ASD</th>
<th>LRFD</th>
<th>Field</th>
<th>Nominal</th>
<th>Tuned</th>
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<tbody>
<tr>
<td>1</td>
<td>BUT -732-1043</td>
<td>0.761</td>
<td>0.451</td>
<td>0.369</td>
<td>0.385</td>
<td>0.375</td>
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<td>PRE-725-0880</td>
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<td>0.431</td>
<td>0.367</td>
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<td>0.300</td>
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<tr>
<td>11</td>
<td>FAI-33-1309</td>
<td>0.727</td>
<td>0.366</td>
<td>0.320</td>
<td>0.256</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>MOT-70-0553</td>
<td>0.727</td>
<td>0.440</td>
<td>0.422</td>
<td>0.445</td>
<td>0.405</td>
</tr>
<tr>
<td>13</td>
<td>AUG-75-0201</td>
<td>0.727</td>
<td>0.426</td>
<td>0.448</td>
<td>0.442</td>
<td>0.448</td>
</tr>
<tr>
<td>14</td>
<td>MAD-40-0745</td>
<td>0.462</td>
<td>0.335</td>
<td>0.336</td>
<td>0.304</td>
<td>0.340</td>
</tr>
<tr>
<td>15</td>
<td>MOT-75-0776</td>
<td>0.727</td>
<td>0.440</td>
<td>0.446</td>
<td>0.390</td>
<td>0.384</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>0.773</td>
<td>0.458</td>
<td>-</td>
<td>0.407</td>
<td>0.405</td>
</tr>
<tr>
<td>17</td>
<td>CLI-132-0083</td>
<td>0.727</td>
<td>0.437</td>
<td>-</td>
<td>0.402</td>
<td>0.404</td>
</tr>
</tbody>
</table>

Notes: ASD results also apply to LFD, as per AASHTO-STD. M1 refers to one lane loaded, M2 refers to two or more lanes loaded. INT refers to the analysis of an interior girder.

---

### Table 8-4: Distribution Factor $DF_{M2, \text{INT}}$ by Various Methods

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>$DF_{M2, \text{INT}}$</th>
<th>ASD</th>
<th>LRFD</th>
<th>Field</th>
<th>FE</th>
<th>Tuned</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>0.761</td>
<td>0.630</td>
<td>0.718</td>
<td>0.637</td>
<td>0.742</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>0.761</td>
<td>0.605</td>
<td>0.525</td>
<td>0.526</td>
<td>0.517</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>0.682</td>
<td>0.578</td>
<td>0.504</td>
<td>0.474</td>
<td>0.467</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>0.864</td>
<td>0.707</td>
<td>0.546</td>
<td>0.629</td>
<td>0.604</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>0.773</td>
<td>0.625</td>
<td>0.573</td>
<td>0.584</td>
<td>0.573</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>0.773</td>
<td>0.628</td>
<td>0.529</td>
<td>0.528</td>
<td>0.539</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>0.788</td>
<td>0.672</td>
<td>0.750</td>
<td>0.717</td>
<td>0.733</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>0.765</td>
<td>0.651</td>
<td>0.659</td>
<td>0.668</td>
<td>0.696</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>0.833</td>
<td>0.665</td>
<td>0.639</td>
<td>0.666</td>
<td>0.684</td>
</tr>
<tr>
<td>10</td>
<td>PRE-503-1170</td>
<td>0.712</td>
<td>0.558</td>
<td>0.559</td>
<td>0.492</td>
<td>0.487</td>
</tr>
<tr>
<td>11</td>
<td>FAI-33-1309</td>
<td>0.727</td>
<td>0.533</td>
<td>0.310</td>
<td>0.493</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>MOT-70-0553</td>
<td>0.727</td>
<td>0.608</td>
<td>0.699</td>
<td>0.685</td>
<td>0.656</td>
</tr>
<tr>
<td>13</td>
<td>AUG-75-0201</td>
<td>0.727</td>
<td>0.587</td>
<td>0.643</td>
<td>0.627</td>
<td>0.643</td>
</tr>
<tr>
<td>14</td>
<td>MAD-40-0745</td>
<td>0.462</td>
<td>0.390</td>
<td>0.412</td>
<td>0.555</td>
<td>0.424</td>
</tr>
<tr>
<td>15</td>
<td>MOT-75-0776</td>
<td>0.727</td>
<td>0.616</td>
<td>0.465</td>
<td>0.570</td>
<td>0.566</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>0.773</td>
<td>0.636</td>
<td>-</td>
<td>0.223</td>
<td>0.234</td>
</tr>
<tr>
<td>17</td>
<td>CLI-132-0083</td>
<td>0.727</td>
<td>0.595</td>
<td>-</td>
<td>0.223</td>
<td>0.234</td>
</tr>
</tbody>
</table>
### Chapter 8 – Lateral Load Distribution

#### Table 8-5: Distribution Factor $DF_{M1,EXT}$ from Various Methods

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>ASD</td>
<td>0.687</td>
<td>0.687</td>
<td>0.638</td>
<td>0.745</td>
<td>0.694</td>
<td>0.694</td>
<td>0.703</td>
<td>0.690</td>
<td>0.728</td>
<td>0.657</td>
<td>0.667</td>
<td>0.667</td>
<td>0.667</td>
<td>0.482</td>
<td>0.667</td>
<td>0.667</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LRFD</td>
<td>0.662</td>
<td>0.806</td>
<td>0.534</td>
<td>0.694</td>
<td>0.528</td>
<td>0.528</td>
<td>0.652</td>
<td>0.570</td>
<td>0.840</td>
<td>0.781</td>
<td>0.722</td>
<td>0.616</td>
<td>0.631</td>
<td>0.728</td>
<td>0.962</td>
<td>0.528</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Field</td>
<td>0.190</td>
<td>0.257</td>
<td>0.229</td>
<td>0.115</td>
<td>0.096</td>
<td>0.123</td>
<td>0.236</td>
<td>0.218</td>
<td>0.356</td>
<td>0.261</td>
<td>0.390</td>
<td>0.291</td>
<td>0.332</td>
<td>0.168</td>
<td>0.408</td>
<td>0.528</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FE</td>
<td>0.152</td>
<td>0.231</td>
<td>0.211</td>
<td>0.206</td>
<td>0.096</td>
<td>0.114</td>
<td>0.249</td>
<td>0.193</td>
<td>0.299</td>
<td>0.345</td>
<td>0.171</td>
<td>0.254</td>
<td>0.326</td>
<td>0.139</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tuned</td>
<td>0.182</td>
<td>0.260</td>
<td>0.217</td>
<td>0.211</td>
<td>0.116</td>
<td>0.117</td>
<td>0.309</td>
<td>0.217</td>
<td>0.302</td>
<td>0.337</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.170</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: ASD results also apply to LFD, as per AASHTO-STD. M1 refers to one lane loaded, M2 refers to two or more lanes loaded. EXT refers to the analysis of an exterior girder.

#### Table 8-6: Distribution Factor $DF_{M2,EXT}$ from Various Methods

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>ASD</td>
<td>0.687</td>
<td>0.687</td>
<td>0.638</td>
<td>0.745</td>
<td>0.694</td>
<td>0.694</td>
<td>0.703</td>
<td>0.690</td>
<td>0.728</td>
<td>0.657</td>
<td>0.667</td>
<td>0.667</td>
<td>0.667</td>
<td>0.482</td>
<td>0.667</td>
<td>0.667</td>
<td></td>
</tr>
<tr>
<td>LRFD</td>
<td>0.686</td>
<td>0.936</td>
<td>0.780</td>
<td>0.935</td>
<td>0.814</td>
<td>0.814</td>
<td>0.703</td>
<td>0.651</td>
<td>0.992</td>
<td>0.907</td>
<td>0.526</td>
<td>0.616</td>
<td>0.631</td>
<td>0.728</td>
<td>0.962</td>
<td>0.528</td>
<td></td>
</tr>
<tr>
<td>Field</td>
<td>0.212</td>
<td>0.311</td>
<td>0.273</td>
<td>0.153</td>
<td>0.119</td>
<td>0.145</td>
<td>0.242</td>
<td>0.242</td>
<td>0.405</td>
<td>0.297</td>
<td>0.350</td>
<td>0.291</td>
<td>0.332</td>
<td>0.168</td>
<td>0.408</td>
<td>0.528</td>
<td></td>
</tr>
<tr>
<td>FE</td>
<td>0.171</td>
<td>0.285</td>
<td>0.253</td>
<td>0.239</td>
<td>0.119</td>
<td>0.137</td>
<td>0.215</td>
<td>0.215</td>
<td>0.334</td>
<td>0.450</td>
<td>0.375</td>
<td>0.254</td>
<td>0.326</td>
<td>0.139</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Tuned</td>
<td>0.204</td>
<td>0.302</td>
<td>0.263</td>
<td>0.248</td>
<td>0.141</td>
<td>0.139</td>
<td>0.240</td>
<td>0.240</td>
<td>0.339</td>
<td>0.444</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.170</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Notes: ASD results also apply to LFD, as per AASHTO-STD. M1 refers to one lane loaded, M2 refers to two or more lanes loaded. EXT refers to the analysis of an exterior girder.
Even though distribution factors values have been computed for interior and exterior girders, the emphasis is on interior girders. When considering a load rating problem, the most critical case usually is that of the interior girders. Owing to its placement at near the center line of the bridge, the interior girders will bear more liveload (liveload can pass on either side) as compared with the exterior girders which are at a distance from the line of travel of live load. Therefore, the exterior girders will not be focused upon in the remainder of this chapter. The distribution factor values earlier computed are plotted in Figures 8-3 and 8-4.

Figure 8-3: Comparison of Distribution Factors $DF_{M1,\text{INT}}$

Figure 8-4: Comparison of Distribution Factors $DF_{M2,\text{INT}}$
8.2.4 “Real” Distribution Factor

The term “Real” Distribution Factor \(DF_{\text{real}}\) is defined so that one algebraic value can be instituted as the most refined distribution factor for a bridge. This DF value shall be the value or closer to a value obtained after applying a modification factor to the DFs from standard equations (STD and LRFD). It can be seen that the refined methods yield very similar values, thus a definition of “real” DF is possible as the average of DF values from the three refined methods. Table 8-7 shows the “Real “DF values for interior girders with one lane loaded and with two or more lanes loaded (i.e. \(DF_{\text{real,M1,INT}}\) and \(DF_{\text{real,M2,INT}}\), respectively).

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>DFM2, INT</th>
<th>Real DF M1, INT</th>
<th>Real DF M2, INT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>0.377</td>
<td>0.699</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>0.363</td>
<td>0.523</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>0.310</td>
<td>0.482</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>0.405</td>
<td>0.593</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>0.365</td>
<td>0.577</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>0.351</td>
<td>0.532</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>0.467</td>
<td>0.733</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>0.392</td>
<td>0.674</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>0.466</td>
<td>0.663</td>
</tr>
<tr>
<td>10</td>
<td>PRE-503-1170</td>
<td>0.326</td>
<td>0.513</td>
</tr>
<tr>
<td>11</td>
<td>FAI-33-1309</td>
<td>0.288</td>
<td>0.402</td>
</tr>
<tr>
<td>12</td>
<td>MOT-70-0553</td>
<td>0.424</td>
<td>0.680</td>
</tr>
<tr>
<td>13</td>
<td>AUG-75-0201</td>
<td>0.446</td>
<td>0.638</td>
</tr>
<tr>
<td>14</td>
<td>MAD-40-0745</td>
<td>0.327</td>
<td>0.463</td>
</tr>
<tr>
<td>15</td>
<td>MOT-75-0776</td>
<td>0.446</td>
<td>0.465</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>0.387</td>
<td>0.568</td>
</tr>
<tr>
<td>17</td>
<td>CLI-132-0083</td>
<td>0.403</td>
<td>0.228</td>
</tr>
</tbody>
</table>

The following terminology is used in the process of calculating the “Real” DF values. \(\%\Delta_{LRFD}\) is defined as the percentage difference between the “real” DF and the \(DF_{LRFD}\) computed as in the AASHTO-LRFD Specification. Hence \(\%\Delta_{LRFD}\) can be computed as:

\[
\%\Delta_{LRFD} = \left( \frac{DF_{LRFD} - DF_{\text{real}}}{DF_{LRFD}} \right)
\]  

(8-1)
8.3 MODIFICATION FACTORS FOR ONE LANE LOADED

The next step after establishing a “real” value of the distribution factor is to correlate it with the values obtained from the equations. This correlation is then used to derive the modification factors. A set of parameters were studied using regression charts. Charts of the parameters that were studied are being given in the Appendix. Eventually, four modification factors will be established:

1) \( MF_{LRFD,M1,INT} \): Modification Factor for adjusting \( DF_{LRFD,M1,INT} \)

2) \( MF_{LRFD,M2,INT} \): Modification Factor for adjusting \( DF_{LRFD,M2,INT} \)

3) \( MF_{STD,M1,INT} \): Modification Factor for adjusting \( DF_{STD,M1,INT} \)

4) \( MF_{STD,M2,INT} \): Modification Factor for adjusting \( DF_{STD,M2,INT} \)

8.3.1 Modification Factor for AASHTO-LRFD with One Lane Loaded

After observing correlation with various sets of parameters, the parameters, \( S / Width \) (Spacing of Girders / Width of Deck) and \( n \) (No. of Girders) were selected for derivation of the modification factor \( MF_{LRFD,M1,INT} \). These parameters show a good correlation with the \( \%\Delta_{LRFD} \).

\[
\therefore \quad MF_{LRFD,M1,INT} = \left( \frac{2}{3} \right) + 4.5 \left( \frac{S}{(n)(Width)} \right) \tag{8-2}
\]

As shown in Table 8-8, the difference between the \( DF_{LRFD,M1,INT} \) (modified to \( DF_{Derived,1} \) by multiplying by the modification factor \( MF_{LRFD,M1,INT} \) above in Equation 8-2) and \( DF_{Real} \) is considerably lowered by the use of the modification factor. \( \%\delta_{LRFD} \) is defined as the percentage difference between the “real” DF and the derived \( DF_{LRFD,Derived} \) computed using the modification factor above. Hence \( \%\delta_{LRFD} \) can be computed as:

\[
\%\delta_{LRFD} = \left( \frac{DF_{Real} - DF_{LRFD,Derived}}{DF_{Real}} \right) \tag{8-3}
\]
Chapter 8 – Lateral Load Distribution

Table 8-8: Modification Factor $MF_{LRFD,M1,INT}$ for $DF_{LRFD,M1,INT}$

<table>
<thead>
<tr>
<th>Sr. No</th>
<th>Bridge Name</th>
<th>S/ (n.Width)</th>
<th>CF</th>
<th>DF$_{LRFD}$</th>
<th>DF$_{Derived, 1}$</th>
<th>DF$_{Real}$</th>
<th>% δ$_{LRFD}$</th>
<th>%Δ$_{LRFD}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>0.047</td>
<td>0.879</td>
<td>0.451</td>
<td>0.397</td>
<td>0.377</td>
<td>5%</td>
<td>17%</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>0.044</td>
<td>0.868</td>
<td>0.431</td>
<td>0.374</td>
<td>0.363</td>
<td>3%</td>
<td>16%</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>0.032</td>
<td>0.814</td>
<td>0.415</td>
<td>0.337</td>
<td>0.310</td>
<td>8%</td>
<td>25%</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>0.048</td>
<td>0.883</td>
<td>0.505</td>
<td>0.446</td>
<td>0.405</td>
<td>9%</td>
<td>20%</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>0.031</td>
<td>0.809</td>
<td>0.445</td>
<td>0.360</td>
<td>0.365</td>
<td>-1%</td>
<td>18%</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>0.031</td>
<td>0.809</td>
<td>0.443</td>
<td>0.358</td>
<td>0.351</td>
<td>2%</td>
<td>21%</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>0.070</td>
<td>0.985</td>
<td>0.488</td>
<td>0.480</td>
<td>0.467</td>
<td>3%</td>
<td>4%</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>0.023</td>
<td>0.773</td>
<td>0.473</td>
<td>0.365</td>
<td>0.392</td>
<td>-7%</td>
<td>17%</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>0.072</td>
<td>0.991</td>
<td>0.486</td>
<td>0.481</td>
<td>0.466</td>
<td>3%</td>
<td>4%</td>
</tr>
<tr>
<td>10</td>
<td>MOT-70-0553</td>
<td>0.066</td>
<td>0.967</td>
<td>0.440</td>
<td>0.426</td>
<td>0.424</td>
<td>0%</td>
<td>4%</td>
</tr>
<tr>
<td>11</td>
<td>AUG-75-0201</td>
<td>0.066</td>
<td>0.966</td>
<td>0.426</td>
<td>0.411</td>
<td>0.446</td>
<td>-8%</td>
<td>-5%</td>
</tr>
<tr>
<td>12</td>
<td>MOT-75-0776</td>
<td>0.065</td>
<td>0.961</td>
<td>0.440</td>
<td>0.423</td>
<td>0.446</td>
<td>-5%</td>
<td>-1%</td>
</tr>
<tr>
<td>13</td>
<td>RIC-30-1438</td>
<td>0.031</td>
<td>0.809</td>
<td>0.458</td>
<td>0.370</td>
<td>0.387</td>
<td>-5%</td>
<td>15%</td>
</tr>
<tr>
<td>14</td>
<td>CLI-132-0083</td>
<td>0.044</td>
<td>0.868</td>
<td>0.437</td>
<td>0.379</td>
<td>0.403</td>
<td>-6%</td>
<td>8%</td>
</tr>
</tbody>
</table>

%δ$_{LRFD}$ is the difference between the DF$_{Real}$ and DF$_{Derived}$

A chart comparing the difference between the $DF_{Derived}$ and $DF_{Real}$ is shown in Figure 8-5. The numbers 1 through 14 indicate the corresponding bridge names in Table 8-8.

![Figure 8-5: Comparison of $DF_{Derived}$ and $DF_{Real}$ for an Interior Girder with One Lane Loaded](image)

Thus the modification factor developed earlier in Equation 8-2 is useful in reducing the conservatism involved in the $DF_{LRFD}$ for an interior girder with one lane loaded.
8.3.2 Modification Factor for AASHTO-STD with One Lane Loaded

The $DF_{STD,M1,INT}$ computed using equations within the AASHTO-STD Specification are shown in Table 8-3. In general, $\%\Delta_{STD}$ is considerably larger than the $\%\Delta_{LRFD}$. As a result, use of the AASHTO-LRFD approach in concert with $MF_{LRFD,M1,INT}$ and $MF_{LRFD,M2,INT}$ is strongly encouraged. A modification factor will be derived with the aim of reducing $\%\Delta_{STD}$ but this is recommended for use only when the AASHTO-LRFD Specification cannot be implemented. The approach used within this section is quite similar to that used in Section 8.3.1.

After observing correlation with various sets of parameters, the parameters, $S / \text{Width}$ (Spacing of Girders / Width of Deck) and $n$ (No. of Girders) were selected for derivation of the modification factor $MF_{STD,M1,INT}$. These parameters show a good correlation with the $\%\Delta_{STD}$.

$$MF_{STD,M1,INT} = 0.39 + 2.78 \left( \frac{S}{n(\text{Width})} \right) \quad (8-4)$$

Table 8-9: Modification Factor $MF_{STD,M1,INT}$ for $DF_{STD,M1,INT}$

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Bridge Name</th>
<th>S/ (n.Width)</th>
<th>CF 1</th>
<th>DF$_{ASD}$ 2</th>
<th>DF$_{Derived}$ 3</th>
<th>DF$_{Real}$ 4</th>
<th>% diff d5 (4-3)/3 x 100</th>
<th>% diff d4</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>0.047</td>
<td>0.522</td>
<td>0.761</td>
<td>0.397</td>
<td>0.377</td>
<td>5%</td>
<td>51%</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>0.044</td>
<td>0.515</td>
<td>0.761</td>
<td>0.392</td>
<td>0.363</td>
<td>7%</td>
<td>52%</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>0.033</td>
<td>0.481</td>
<td>0.682</td>
<td>0.328</td>
<td>0.310</td>
<td>6%</td>
<td>55%</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>0.045</td>
<td>0.525</td>
<td>0.864</td>
<td>0.453</td>
<td>0.405</td>
<td>11%</td>
<td>53%</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>0.031</td>
<td>0.478</td>
<td>0.773</td>
<td>0.370</td>
<td>0.365</td>
<td>1%</td>
<td>53%</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>0.031</td>
<td>0.478</td>
<td>0.773</td>
<td>0.370</td>
<td>0.351</td>
<td>5%</td>
<td>55%</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>0.070</td>
<td>0.588</td>
<td>0.788</td>
<td>0.463</td>
<td>0.467</td>
<td>-1%</td>
<td>41%</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>0.023</td>
<td>0.456</td>
<td>0.765</td>
<td>0.349</td>
<td>0.392</td>
<td>-12%</td>
<td>49%</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>0.072</td>
<td>0.591</td>
<td>0.833</td>
<td>0.493</td>
<td>0.466</td>
<td>5%</td>
<td>44%</td>
</tr>
<tr>
<td>10</td>
<td>MOT-70-0553</td>
<td>0.066</td>
<td>0.577</td>
<td>0.727</td>
<td>0.420</td>
<td>0.424</td>
<td>-1%</td>
<td>42%</td>
</tr>
<tr>
<td>11</td>
<td>AUG-75-0201</td>
<td>0.066</td>
<td>0.576</td>
<td>0.727</td>
<td>0.419</td>
<td>0.446</td>
<td>-6%</td>
<td>39%</td>
</tr>
<tr>
<td>12</td>
<td>MOT-75-0776</td>
<td>0.065</td>
<td>0.573</td>
<td>0.727</td>
<td>0.417</td>
<td>0.446</td>
<td>-7%</td>
<td>39%</td>
</tr>
<tr>
<td>13</td>
<td>RIC-30-1438</td>
<td>0.031</td>
<td>0.478</td>
<td>0.773</td>
<td>0.370</td>
<td>0.387</td>
<td>-5%</td>
<td>50%</td>
</tr>
<tr>
<td>14</td>
<td>CLI-132-0083</td>
<td>0.044</td>
<td>0.515</td>
<td>0.727</td>
<td>0.375</td>
<td>0.403</td>
<td>-8%</td>
<td>45%</td>
</tr>
</tbody>
</table>

As shown in Table 8-9, the difference between the $DF_{STD,M1,INT}$ (modified to $DF_{Derived}$ by multiplying by $MF_{STD,M1,INT}$) and $DF_{Real}$ is considerably lowered by the use of the modification factor. A chart comparing the difference between the $DF_{Derived}$ and $DF_{Real}$ is shown below in Figure 8-6. The numbers 1 through 14 indicate the corresponding bridge names in Table 8-9.
8.4 MODIFICATION FACTORS FOR MULTIPLE LANES LOADED

After deriving modification factors for the one lane loaded case, the case of two or more lanes loaded will be addressed. A critical load carrying capacity of a bridge is often a function of its response to trucks placed on all lanes of the bridge. The distribution factor, $DF_{M2,\text{INT}}$, for the 17 bridges in the database was reported previously in Table 8-4. A comparison shows a more complex traversing of DF values from each individual source for a bridge as compared to the comparison from $DF_{M1,\text{INT}}$. As was done for case of one lane loaded, the percentage differences ($\%\Delta_{LRFD}$ and $\%\Delta_{STD}$) between equation values and real values of DF were examined while certain parameters were varied. Several design parameters were checked for correlation with the percentage values. The details of the same are being presented here.

8.4.1 Modification Factor for AASHTO-LRFD with Multiple Lanes Loaded

For computation of a modification factor for a two or more lane loaded case (multiple lane loaded case), the bridges with skew angles less than 15° were examined. This resulted in a data set of 14 bridges to be used. Several bridge parameters were investigated for correlation with the $\%\Delta_{STD}$ and $\%\Delta_{LRFD}$. Most design parameters, like the spacing of girders, width of bridge, span length, girder stiffness, etc, failed to show a good correlation with the percentage differences. One of the parameter that did show a strong correlation was the remaining composite action (RCA). The remaining composite action was suggested by Turer (1998) as the ratio of axial force in the steel beam to that predicted by a fully composite section.
RCA is a linear condition index defining the level of composite action present in a bridge. Thus a bridge with an RCA of 100% indicates a section representing full composite action as defined by AASHTO, while an RCA of 0% indicates a section with no composite action. A lesser or greater percentage represents a corresponding linear change in the composite action. Most bridges selected for this study are designed to be non-composite (the only composite bridge is FAI-33-1309). As a result, it is expected that the girder and steel will have a relative slip when load is applied. However, as observed from the strain profiles obtained from the truckload tests, most bridges have an unintended composite action, even in the absence of shear connectors. The RCA values for all the bridges have been represented below.

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Bridge Name</th>
<th>RCA (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>60.43</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>111.23</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>125.78</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>121.34</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>110.69</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>118.79</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>91.89</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>107.02</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>107.43</td>
</tr>
<tr>
<td>10</td>
<td>AUG-75-0201</td>
<td>72.82</td>
</tr>
<tr>
<td>11</td>
<td>MOT-75-0776</td>
<td>136.84</td>
</tr>
</tbody>
</table>

The RCA has shown to have a good correlation with the $\%\Delta_{LRFD}$ and $\%\Delta_{STD}$ for the case of multiple lanes loaded. This relation is an indication of the condition of the bridge having an impact (composite action in this case) on the lateral load distribution of the bridge. The BUT-732-1043, bridge shows a low RCA, which is an indication of a relatively low level of composite action in the bridge. Also, the $\%\Delta$ values for the bridge are significantly lower as compared to those of other bridges in the database. It should be noted that the condition of the bridge has an influence on the load distribution characteristics of the bridge that is observed in the tuned finite element model and the truckload tests (which are the sources that represent the load distribution in the actual bridge). This tends to result in higher-than-expected distribution factors.
Chapter 8 – Lateral Load Distribution

As is seen in Figure 8-7, it is evident that a quadratic trendline provides a reasonable fit to the data set when $\% \Delta_{LRFD}$ is plotted as a function of the remaining composite action. A modification factor $M_{FLRFD, M2, INT}$ for correcting the $DF_{LRFD, M2, INT}$ will now be derived based on the regression equation shown in Figure 8-7.

$$M_{FLRFD, M2, INT} = 0.8712 + 0.008(RCA) - 0.00007(RCA)^2 \quad (8-5)$$

![Image of Figure 8-7: Correlation of $\% \Delta_{LRFD, 2}$ vs. RCA %]

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Bridges</th>
<th>RCA</th>
<th>$M_{FLRFD, 2}$</th>
<th>$DF_{DERIVED}$</th>
<th>$DF_{REAL}$</th>
<th>$DF_{LRFD}$</th>
<th>$% \Delta_{LRFD}$</th>
<th>$% \delta_{LRFD}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>60.43</td>
<td>1.1378</td>
<td>0.717</td>
<td>0.699</td>
<td>0.630</td>
<td>-11%</td>
<td>3%</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>111.23</td>
<td>1.0227</td>
<td>0.618</td>
<td>0.523</td>
<td>0.605</td>
<td>14%</td>
<td>18%</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>125.78</td>
<td>0.8947</td>
<td>0.518</td>
<td>0.482</td>
<td>0.578</td>
<td>17%</td>
<td>7%</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>121.34</td>
<td>0.9382</td>
<td>0.663</td>
<td>0.593</td>
<td>0.707</td>
<td>16%</td>
<td>12%</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>110.69</td>
<td>1.0267</td>
<td>0.642</td>
<td>0.577</td>
<td>0.625</td>
<td>8%</td>
<td>11%</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>118.79</td>
<td>0.9615</td>
<td>0.604</td>
<td>0.532</td>
<td>0.628</td>
<td>15%</td>
<td>14%</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>91.89</td>
<td>1.1274</td>
<td>0.758</td>
<td>0.733</td>
<td>0.672</td>
<td>-9%</td>
<td>3%</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>107.02</td>
<td>1.0519</td>
<td>0.684</td>
<td>0.674</td>
<td>0.651</td>
<td>-4%</td>
<td>2%</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>107.43</td>
<td>1.0492</td>
<td>0.698</td>
<td>0.663</td>
<td>0.665</td>
<td>0%</td>
<td>5%</td>
</tr>
<tr>
<td>10</td>
<td>AUG-75-0201</td>
<td>72.82</td>
<td>1.1573</td>
<td>0.680</td>
<td>0.638</td>
<td>0.587</td>
<td>-9%</td>
<td>7%</td>
</tr>
<tr>
<td>11</td>
<td>MOT-75-0776</td>
<td>136.84</td>
<td>0.7690</td>
<td>0.474</td>
<td>0.465</td>
<td>0.616</td>
<td>24%</td>
<td>2%</td>
</tr>
</tbody>
</table>

$\% \Delta_{LRFD, 2}$ is the difference between the $DF_{\text{Real}}$ and $DF_{\text{Derived}}$.
As shown in Table 8-11, the difference between the $DF_{LRFD,M2,INT}$ (modified to $DF_{Derived}$ by multiplying by $MF_{LRFD,M2,INT}$) and $DF_{Real}$ is considerably lowered by the use of the modification factor. A chart comparing the difference between the $DF_{Derived}$ and $DF_{Real}$ is shown in Figure 8-8. The numbers 1 through 11 indicate the corresponding bridge names in Table 8-10.

![Figure 8-8: Comparison of $DF_{Derived}$ and $DF_{Real}$](image)

**8.4.2 Modification Factor for AASHTO-STD with Multiple Lanes Loaded**

The final step in this study is to develop a modification factor for use with the AASHTO-STD Specification for the case of multiple lanes loaded. A correlation of RCA and $\%\Delta_{ASD}$ is shown in Figure 8-9, and it is again found that a quadratic trendline shows a good fit.

![Figure 8-9: RCA vs. $\%\Delta_{STD}$](image)
A modification factor for $DF_{STD,M2,INT}$ can thus be derived using the equation in Figure 8-9 for $\%\Delta_{ASD}$.

\[
\therefore MF_{STD,M2,INT} = 0.7835 + 0.0051(RCA) - 0.00005(RCA)^2
\]  

(8-6)

As shown in Table 8-12, the difference between $DF_{STD,M2,INT}$ (modified to $DF_{Derived}$ by multiplying by $MF_{STD,M2,INT}$) and the $DF_{Real}$ is considerably lowered by the use of the modification factor. A chart comparing the difference between the $DF_{Derived}$ and $DF_{Real}$ is shown in Figure 8-10. Numbers 1 through 11 indicate the corresponding bridge names in Table 8-12.

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Bridge Name</th>
<th>RCA</th>
<th>CF</th>
<th>$DF_{DERIVED}$</th>
<th>$DF_{REAL}$</th>
<th>$DF_{ASD}$</th>
<th>$%\Delta_{ASD}$</th>
<th>$%\delta_{ASD}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>60.43</td>
<td>0.9091</td>
<td>0.692</td>
<td>0.699</td>
<td>0.761</td>
<td>8%</td>
<td>-1%</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>111.23</td>
<td>0.7322</td>
<td>0.557</td>
<td>0.523</td>
<td>0.761</td>
<td>31%</td>
<td>6%</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>125.78</td>
<td>0.6339</td>
<td>0.432</td>
<td>0.482</td>
<td>0.682</td>
<td>29%</td>
<td>-11%</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>121.34</td>
<td>0.6662</td>
<td>0.575</td>
<td>0.593</td>
<td>0.864</td>
<td>31%</td>
<td>-3%</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>110.69</td>
<td>0.7354</td>
<td>0.568</td>
<td>0.577</td>
<td>0.773</td>
<td>25%</td>
<td>-1%</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>118.79</td>
<td>0.6838</td>
<td>0.528</td>
<td>0.532</td>
<td>0.773</td>
<td>31%</td>
<td>-1%</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>91.89</td>
<td>0.8300</td>
<td>0.654</td>
<td>0.733</td>
<td>0.788</td>
<td>7%</td>
<td>-12%</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>107.02</td>
<td>0.7566</td>
<td>0.579</td>
<td>0.674</td>
<td>0.765</td>
<td>12%</td>
<td>-16%</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>107.43</td>
<td>0.7543</td>
<td>0.629</td>
<td>0.663</td>
<td>0.833</td>
<td>20%</td>
<td>-5%</td>
</tr>
<tr>
<td>10</td>
<td>AUG-75-0201</td>
<td>72.82</td>
<td>0.8897</td>
<td>0.647</td>
<td>0.638</td>
<td>0.727</td>
<td>12%</td>
<td>1%</td>
</tr>
<tr>
<td>11</td>
<td>MOT-75-0776</td>
<td>136.84</td>
<td>0.5451</td>
<td>0.396</td>
<td>0.465</td>
<td>0.727</td>
<td>36%</td>
<td>-17%</td>
</tr>
</tbody>
</table>

$\%\delta_{ASD,2}$ is the difference between the $DF_{Derived}$ and $DF_{Real}$

Figure 8-10: Comparison of $DF_{Derived}$ and $DF_{Real}$
8.5 SUMMARY

Current AASHTO specifications give equations for distribution factors that are shown to be conservative [Hunt 2000; Barker 2001]. As the distribution factor is involved in the computation of rating, this conservatism in DF is reflected in the rating factor. While this inherent conservatism may be acceptable or desirable from a design point of view, it is not desirable for a rating problem. A more refined way of computing distribution factor is needed.

The objective of this study was to develop modification factors for distribution factors resulting from simplified equations contained in the AASHTO-LRFD and AASHTO-STD Specifications. While a significant gain in accuracy can be achieved simply by implementing the approach used in the AASHTO-LRFD Specification as opposed to the AASHTO-STD Specification, the development of modification factors for both approaches is warranted.

Seventeen bridges were selected from the database of bridges and distribution factors were computed for each bridge using predictive, measured, and analytical methods. Although DF values for exterior and interior girders have been computed, only the interior girder DFs for moment were addressed in this study. Furthermore, bridges with skew angles greater than 15° showed little correlation with parameters that were effective for non-skewed bridge. For this reason and because an insufficient data set existed (just three bridges), skewed structures were not addressed. Finally, data of only 14 non-skewed bridges are involved for computation of modification factors.

Several terms, design and conditional, were examined for correlation with the $\%\Delta$ values. The terms that were evaluated include span length, girder spacing, stiffness of girders, deck thickness, width of deck, cross frame spacing, lane width, number of lanes, number of girders, remaining composite action, effective deck width, and general assessment. For the case of one lane loaded, a good correlation of $\%\Delta$ values was observed with the terms (width of deck / spacing of girders) and the number of girders. Similarly, for the case of multiple lanes loaded, a good correlation of $\%\Delta$ values was observed with the remaining composite action of the structure. Thus, the proposed modification factors are based primarily on these parameters.
Chapter 9 Summary and Conclusions

9.1 INTRODUCTION

The results and conclusions from each chapter within this report have been presented within their respective chapters but are summarized in the following sections to provide a brief overview of the project in a central location.

9.2 BRIDGE SELECTION CRITERIA AND TEST SPECIMENS

The field test component of the research was conducted in close collaboration with ODOT personnel at the Central Office and District levels. The steel stringer bridge population in Ohio was surveyed to categorize posted bridges (bridges rated below 100% legal load) and non-posted bridges (bridges rated between 100% and 150% legal load). In addition, bridge engineers in each of the state’s twelve districts were interviewed to identify damaged and/or deteriorated bridges around the state. This latter category includes bridges hit by vehicles, exposed to fires, suffering from extensive section loss, deck deterioration, etc. A pool of test specimens were selected for field test operations, based upon geometry and access concerns.

9.3 MULTIPLE REFERENCE IMPACT TESTING

This project employed a form of modal testing referred to as multiple reference impact testing. It was applied at each bridge site, one lane at a time. The test sensor layout consisted of accelerometers (20-40, depending on bridge size) positioned on the upper-side of the deck directly above girder-cross frame intersections and girder-bearing positions. A load cell equipped drop hammer was then used to apply an impact at 6-8 of these sensor positions. During testing of each lane, the remaining lanes of the structure were left open to traffic at all times except momentarily at the impact times. All load input and acceleration data was recorded for multiple impacts by a PC-based data acquisition system at the site.
The research enhanced and extended modal test methods as previously employed in the field in several ways. First, test procedures were standardized across all tests so that similar data sets were obtained for each bridge. Second, test procedures were developed to streamline the data collect portion of the test so that its duration would not exceed one hour. This was significant in that one hour represents a period of duration so short that inherent environmental and ambient changes which occur throughout the day would not be reflected in the experimental data and affect its quality. Third, a series of electronic, mechanical and signal processing techniques were developed to maximize data quality. Fourth, test procedures were developed so that an entire bridge could be modal tested (including test setup, execution, and teardown) in less than 8 hours and with a minimum impact of the traveling public and minimum support requirements (access and traffic control) on the part of ODOT field personnel. Fifth, the test procedures, including pretest logistics, were configured so that they were highly mobile and rapidly deployable. The test set up hardware and procedures were all self contained including all power, tools, sensors, cabling, and electronics which could be easily packed into a cargo van. Finally, custom software was developed which permitted automatic checking of test data for quality assurance, post processing of test data to obtain modal parameters (frequencies, mode shapes, and modal scalings) and the extraction of bridge flexibility parameters in the form of Bridge Girder Condition Indicators (BGCIs).

9.4 TRUCKLOAD TESTING

This project also employed crawl speed truckload testing as another experimental field test tool. It was applied at approximately half of the bridges selected for this project where the availability of snoopers or use of ladders from the underside of the structure was feasible. The test sensor layout consisted of strain gages (15-20) mounted on the top and bottom flanges of the bridge girders at both positive and negative moment regions. As with the modal tests, crawl-speed tests were conducted for each traffic lane allowing the structure to remain partially open to traffic during testing. Trucks of known axle weights and spacings (provided by ODOT), moving at crawl speeds (< 15 mph), were used to apply a load excitation to the bridge. Tape switches placed at the abutments were used to synchronize truck locations with data readings. All strain data was recorded for multiple truck passes by a PC-based data acquisition system at the site.
Like in the case of modal testing methods, the research enhanced and extended truckload test methods from what was previously employed in the field. First, test procedures were standardized across all tests so that similar data sets were obtained for each bridge. Second, test procedures were developed to streamline the data collection portion of the test so that its duration would not exceed one hour. Third, a series of electronic, mechanical and signal processing techniques were developed to maximize data quality. Fourth, test procedures were developed so that an entire bridge could be truckload tested (including test setup, execution, and teardown) in less than 8 hours and with a minimum impact of the traveling public and minimum support requirements (access and traffic control) on the part of ODOT field personnel. Fifth, the test procedures, including pretest logistics, were configured so that they were highly mobile and rapidly deployable. The test set up hardware and procedures were all self contained including all power, tools, sensors, cabling, and electronics which could be easily packed into a cargo van. Finally, custom software was developed which permitted automatic checking of test data for quality assurance, post processing of test data to obtain truckload parameters (strains, stresses, and unit axle influence lines) and the simulation of HS20, lane load responses, and load capacity ratings.

9.5 ESTIMATION OF STRUCTURAL CAPACITY

The research demonstrated conclusively that one conceptual signature that represents bridge condition and can be determined from either a truckload test or an FE simulation is the unit influence line. The derived influence line has been shown to be consistent for various truckloads and axle configurations. From the influence line, several conclusions can be immediately drawn regarding the structural condition including the level and consistency of (unintended) composite action, lateral distribution, longitudinal balance of response maxima, impact factor, end restraint, edge stiffening, linearity, and stationarity under load.

In addition, the unit influence line formed the basis for a method developed to rate each of the instrumented sections. Further, the nominal and tuned finite element models were also analyzed and rated by this same methodology. The models were rated only at locations corresponding to truckload instrumentation. The goal was an apples-to-apples comparison; however, it is also quite possible to use the models to investigate other locations for criticality.
Note that the critical instrumented span and pier(s) are reported for both truckload and lane load rating. Further, the section properties for these locations are tabularized for comparison with BARS. Note that the liveload moment and centroid location found by the field test are often quite different from their values defined by design or by BARS analysis because of the presence of unintended composite action. Note that the deadload moment is calculated within the BARS program from the design plans, but discrepancies in the actual and designed geometry (e.g., deck thickness) will lead to accompanying differences in deadload and superimposed deadload moments. Finally, the influence lines and simulated HS20 responses for each of the marked traffic lanes of the structure are presented for both the top and bottom flanges.

Diagnostic truckload testing and instrumented monitoring have proven to be valuable but objective methods for the condition assessment of highway bridges. The constructed bridge will have many inherent mechanisms to resist the applied load and which are generally not considered in the design or analysis of its capacity. The expected liveload stresses and their distribution can be checked and identified by this methodology. Further, the bridge may have been subjected to unexpected or other forces undetermined by the design whose effect may be measured and used in the load rating of the structure. The truckload test results may be used directly or compared against a finite element model of the structure (nominal and/or tuned).

9.6 FINITE ELEMENT MODELING OF BRIDGES

3D finite element modeling has been shown to be very useful in condition assessment of highway bridges especially when coupled with actual field data obtained through truck or modal tests. A well calibrated model of an existing bridge can be used as a good tool to simulate various conditions on the bridge which practically do not exist or are too costly to implement right away. A model of a bridge that needs repair can be modeled and the effect of retrofits as a solution can be assessed by using FE techniques. Another significant application of this method would be to study the response of a bridge to a certain special loading condition and to rate the bridge at that loading.

The UCII Bridge Modeler software was developed as part of this research as a comprehensive package that can efficiently create 3D FE models of concrete slab on steel
stringer bridges (including nearly all the common features) from plan data using a simple Graphical User Interface. It forms a preprocessor, written in Visual Basic, which lets users input meaningful data from bridge plans to generate a SAP2000 input file. The model is then fed as input to SAP2000 for analysis, generation of modes, and generation of expected influence lines. Given the bridge plans, a typical FE model can be generated in about 30 minutes. The data required for the model generation is collected through eight tabbed dialog boxes in the program. A diagram of the respective bridge feature being defined is shown to the user in each tab to aid the user and reduce errors. Default values are defined for all required parameters. The user is also able to save and reload model data. What used to take weeks and days to write manually has been automated and greatly reduced in time using this preprocessor.

The major assumptions used in the software include: girders are modeled using shell elements for the web and frame elements for flange; bridge deck, sidewalks, and parapets are modeled using shell elements; the deck and girders are connected using rigid links; piers and abutments are defined using springs; cross frames are defined as frame elements; cover plates, variations in flange thickness, and haunches are incorporated; the preprocessor provides complete flexibility in the generation of truck/lane loads (via AASHTO, FHWA, and state loading conditions); and the program allows user-defined density of the generated mesh as well as locations of outputs.

9.7  **FINITE ELEMENT CALIBRATION**

The calibration of the finite element model to experimental data is achieved by systematically varying parameters in the model input file. Sensitivity analyses are performed in order to identify the critical parameters including: (1) stiffness of vertical springs over supports, (2) stiffness of horizontal springs over piers, (3) stiffness of horizontal springs over abutments, (4) horizontal restraint cases in bridge length direction, (5) moment of inertia of rigid links, (6) thickness of concrete decks, (7) unit weight of concrete deck, (8) modulus of elasticity of concrete deck, and (9) nodal mass over piers. Two general groups are used to categorize these critical parameters. The first group, based on physical properties, includes parameters 1, 2, 3 and 4 (used to simulate boundary conditions), parameter 5 (used to simulate continuity conditions), and parameters 6, 7, 8 and 9 (used to simulate geometry of the critical regions and elements).
The second group, based on influence on the stiffness and mass matrices, include parameters 1, 2, 3, 4 and 5 (stiffness matrix only), parameters 7 and 9 (mass matrix only), and parameters 6 and 8 (stiffness and mass matrices).

On the basis of the sensitivity analysis, the calibration of the finite element model is started. Measured values such as the BGCI, mode shapes (using the MAC), modal frequencies, and truckload strains (represented by UIL’s) are factored into an objective function. This objective function includes the difference between the analytical and experimental quantities mentioned above and importance factors. A systematic, gradient-based optimization procedure for minimizing the objective function has been developed and automated.

Since the output files generated by the finite element analyses are enormous, manual reduction is cumbersome and inefficient. As a result, a post-processing software package was also created for this task. This software uses the input file and scratch files generated by Bridge Modeler and the FE output files to extract the data that are required for creating load response curves and unit influence lines. The automation of the entire process of load simulation and post processing expedites sensitivity studies, tuning, and bridge condition evaluation.

### 9.8 LATERAL LOAD DISTRIBUTION FACTORS

Current AASHTO specifications give equations for distribution factors that are known to be conservative. As the distribution factor is involved in the computation of rating, this conservatism in DF is reflected in the rating factor. While this inherent conservatism may be acceptable or desirable from a design point of view, it is not desirable for a rating problem. A more refined way of computing distribution factor is needed and has been developed as part of this research.

The objective of this study was to develop modification factors for distribution factors resulting from simplified equations contained in the AASHTO-LRFD and AASHTO-STD Specifications. While a significant gain in accuracy can be achieved simply by implementing the approach used in the AASHTO-LRFD Specification as opposed to the AASHTO-STD Specification, the development of modification factors for both approaches is warranted.
9.9 STATISTICAL OBSERVATIONS DERIVED FROM FIELD RATINGS

For each of the ten test specimens listed in Chapter 2, field tests (as discussed in Chapters 3 and 4) were conducted. In addition, an FE model was developed (as discussed in Chapter 6). Using these pieces of information, together with the original ODOT BARS reports on these structures, 4 sets of load ratings were calculated and compared: BARS, ratings directly from the experimental data, ratings based on the nominal FE model and ratings based on the tuned FE model. In each case, operating and inventory ratings were computed using AASHTO allowable stress, load factor, and overload methods were employed. In addition, for the case of experimental results, section properties were calculated in 4 ways. Finally the concept of Remaining Composite Action (RCA) was developed. The master table of ratings is given below. A number of conclusions are readily apparent from these results.

1. Inventory Rating is less than Operational Rating. Hence, all statistical findings are based on Inventory Ratings.

2. Truck Load Rating generally governed over Lane Load Rating for the sample space.

3. Rating factors obtained by various methods: Allowable stress method (AS), Load Factor Method (LF), Overload Inventory (OL) been found to follow a pattern. For example, if experimental rating factors obtained by AS for a particular gage are lowest of all gages instrumented on the bridge then experimental rating factors obtained by LF and overload inventory are also the lowest for that gage.

4. In all cases, AS ratings tended to be lower than either LF or OL.

5. Overload controls the rating over LF most of the time. Consider the following chart:

<table>
<thead>
<tr>
<th>METHOD</th>
<th>%</th>
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<tbody>
<tr>
<td>BARS</td>
<td>87.5</td>
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<tr>
<td>NOMINAL</td>
<td>96.875</td>
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<tr>
<td>TUNED</td>
<td>96.875</td>
</tr>
<tr>
<td>EXPERIMENT</td>
<td>94.117</td>
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</table>
6. On an average there is an improvement of 97.5% in the AS Rating Factors from BARS to Tuned; 93.04% for LF Rating Factors; 89.75% for OL Rating Factors.

7. The Mean, Variance, Standard Deviation of the Ratings obtained are as follows:

<table>
<thead>
<tr>
<th>Table 9-2: Statistical Results of AS Rating for All 10 Bridges</th>
</tr>
</thead>
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<tr>
<td><strong>STATISTIC</strong></td>
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<td>STD. DEV.</td>
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<th>Table 9-3: Statistical Results of LF Rating for All 10 Bridges</th>
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<td><strong>STATISTIC</strong></td>
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<th>Table 9-4: Statistical Results of OL Rating for All 10 Bridges</th>
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</tr>
<tr>
<td>VARIANCE</td>
</tr>
<tr>
<td>STD. DEV.</td>
</tr>
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</table>
8. All of the non-compositely designed bridges showed some composite action measured by the theoretical index Remaining Composite Action (RCA).

9. In the moment plots, usually method 1 and method 4 moments bound the method 2 and method 3 (see Chapter 5 regarding the various rating methods). But when RCA is above 100%, moments by methods 2 and 3 have been generally found to be greater than method 1. Section method 2 and method 3 moment values are found to be close.

The following tabularize the results by rating method for all ten bridges considered in this project. For each bridge, the instrumented location with the worst HS20 truckload ratings and the worst laneload (LL) ratings are provided, as specified in the fifth column of the table.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>SFN</th>
<th>Response</th>
<th>Worst gage</th>
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<th>NOMINAL</th>
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Table 9-6: Critical LF Rating for Truckload and Laneload per Bridge

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<th>Worst gage</th>
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Table 9-7: Critical OL Rating for Truckload and Laneload per Bridge

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<th>Response</th>
<th>Worst gage</th>
<th>BARS</th>
<th>NOMINAL</th>
<th>TUNED</th>
<th>EXP.</th>
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STATEMENT OF NEED:

The most common type of short-to-medium span highway bridge in the ODOT inventory is the reinforced concrete (RC) slab-on-steel girder bridge with RC abutments and piers. About 6,335 of the 14,957 bridges in the ODOT inventory are of this type. As per the requirements of the National Bridge Inspection Standards (NBIS), the ODOT Office of Structural Engineering is tasked with the responsibility of rating these structures in order to determine their safe live load carrying capacity. ODOT Procedure No. 518-001(P) outlines the standard operating procedures connected with this rating process which is comprised of three main components: 1) Bridge Records and Inventory Data, 2) Bridge Inspection Data and Report, and 3) Load Rating Calculations and Report. The current procedure for determining these ratings is through the use of the BARS software package which implements Load Factor Rating (LF) or Allowable Stress Method (AS) calculations. The parameters used in these calculations can be modified at the discretion of the rating engineer subject to the findings contained in the Bridge Inspection Report. When the operating rating of a bridge is determined to be less than 100% of the legal load for which the bridge was designed, the structure must be posted to a lower load limit to ensure safe use by the general traveling public. At this writing, ODOT currently maintains posted ratings (i.e., less than 100% of legal load limits) on 80 bridges in Ohio, of which 21 are steel girder/beam bridges, with the remainder of the inventory being rated between 100% and 150%. In addition, ODOT depends, at least in part, on the these rating to distribute the approximately $235M it spends annually on the operation, maintenance, rehab/retrofit, and new construction of the bridges in its inventory. However, these methods are known to result in subjective/conservative ratings, not calibrated against field measurements, and hence possibly not objectively reflecting the realities and impacts of individual bridge condition as well as design details. Visual inspections provide at best qualitative and subjective information on bridge condition which is hard to incorporate into the rating process. One result of this approach is the potentially inefficient management and maintenance practices based on conservative/subjective ratings and inspection reports. In addition, there is the need to understand the true impact design and construction practices have on serviceability and safety of the bridge inventory, something which cannot be explored based solely on the current approaches.
RESEARCH OBJECTIVES:

• Identify a sample of 10 low rated/posted steel-stringer bridges and perform field testing and structural identification studies on each, resulting in a field-calibrated 3D finite element (FE) model of each bridge.
• Assemble a database of actual structural testing results, calibrated FE models, and field test based rating factors for these 10 bridges and compare them against their BARS counterparts.
• Further streamline the field testing procedures of truckload testing and multiple reference impact testing in order to increase the practicality and structural legitimacy of these methods as tools for use in rating process.

RESEARCH TASKS:

• Bridge inventory review and test site selection
• Conduct field testing at bridge sites employing a combination of standardized approaches to modal testing and truck-load testing methods.
• Development calibrated FE models and field test based ratings.
• Compare field test based ratings against analytical BARS counterparts.

RESEARCH DELIVERABLES:

• An experimental, objective database containing information about the structural properties of the selected test bridges. This database will provide a representation of low-rated steel-stringer bridge behavior.
• A report for each bridge that documents the service level measurements and state parameters for that bridge together with the associated field-test-data-based load rating of the structure.
• Documentation of test-based indices (e.g., objective on-site experimental condition/damage indices that may be used to identify regions of damage/deterioration) and how they may be generated through on-site experimentation.
• Modal testing, truckload testing, and calibrated FE modeling of steel stringer bridges has become a well developed methodology to produce objective, reliable, and accurate predictions of the behavior and capacity of steel stringer bridges.
• The field testing components of the research have been standardized and streamlined so that they can be deployed quickly, practically, and safely with minimum impact to traveling public and ODOT.
• Filed testing of even low rated bridges has revealed that there are a number of factors, not readily apparent through visual inspection, that lead most steel stringer bridges to have higher ratings than would be obtained via analytical/BARS based methods.

RESEARCH RECOMMENDATIONS:

• Field procedures and methods proven in this research could be incorporated into ODOT’s procedures for Rating and Analyses as well as Bridge Operations and Maintenance as an information gathering tool in cases where field data would provide useful data in the inventory management decision making process. Examples include site specific assessment of specific bridges, load rating for overload passages, evaluation of retrofits, etc.
• The testing of these 10 low rated steel-stringer bridges can act as a prototype the implementation of new procedures for inspection, rating, and management for the class of low rated bridges. Such procedures will be more objective than existing methods and will be along the spirit of the 1989 AASHTO Guide for Strength Evaluation.
• These same procedures could to adapted for the testing of bridges which have recently experienced damage or are in a state of deterioration to help bridge engineers better assess load ratings. In addition, testing could be used pre- and post- repair in order to characterize and assess the success of repair measures.
• The examination of a population of 10 low rated structures, together with the previous study of 30 bridges can be used to document a number of previously unknown causative effects on bridge response and behavior. This database, and the unexplored knowledge it still contains, can be cultivated further to reveal other aspects of steel stringer bridge behavior as well as to help optimize future bridge design and maintenance practices. Such an investigation should culminate in recommended changes to AASHTO’s design and evaluation provisions pertaining to stringer bridges (e.g., distribution factors).

• The results and methods of this study could be employed to achieve similar gains for other bridge types (such as concrete bridges) which are prevalent in the Ohio bridge inventory. Application of these techniques could help address open issues in the management, maintenance, and evaluation for these bridge types too as well as identify key factors effecting bridge performance.

PROJECT PANEL COMMENTS:
• Available at a later date

IMPLEMENTATION STEPS & TIME FRAME:
• To be determined

EXPECTED BENEFITS:
• Provide ODOT with a toolbox of proven field test methods which can be rapidly deployed to provide accurate field measurement to assist in bridge management, maintenance, and evaluation decisions for a wide variety of bridges and under a wide variety of circumstances;
• Provide ODOT with a tool that can perform accurate load-ratings for both steel-stringer and concrete bridges under a wide variety of situations;
• Permit correlation of field test results with the data in ODOT’s bridge database. Such correlation will augment and improve ODOT’s inventory data and current load-rating approaches; and,
• Help evaluate/identify the impact that design attributes and details have on structural serviceability and safety.

EXPECTED RISKS, OBSTACLES, & STRATEGIES TO OVERCOME THEM: None

OTHER ODOT OFFICES AFFECTED BY THE CHANGE: None

PROGRESS REPORTING & TIME FRAME:

TECHNOLOGY TRANSFER METHODS TO BE USED:
The Office of Research and Development will make copies of the research report available as well have it posted to the ODOT website. A short course could be developed to train ODOT personnel in the use of field test techniques and/or use of software for FE modeling and rating.

IMPLEMENTATION COST & SOURCE OF FUNDING:
References


ACI 318-95 (1995). *Building Code Requirements for Structural Concrete*, American Concrete Institute, Committee 318, Farmington, MI.


References

Allemang, R. J. (1994). Vibrations: Analytical and Experimental Modal Analysis, UC Structural Dynamics Laboratory, UC- SDRL CN 20-263-662/663/664, University of Cincinnati, Cincinnati, OH.


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*Project Report*, Report UC-CII-00, University of Cincinnati Infrastructure Institute, University of Cincinnati, Cincinnati, OH.


References


References


Appendix A - Bridge Reports

This appendix contains reports for each of the 10 bridges that make up the data set upon which the conclusions are based. Depending on the availability of experimental data, each report consists of up to three parts. The first is a general description of the structure and general data regarding the experiments that were conducted (date, temperature, etc.). The second is a description of the FE model that was generated and a report about the results of the model calibration. The third is a report detailing the rating process for the structure. These three parts is described in detail in the following paragraphs.

The first section of each bridge report provides details about the modal experiments that were conducted on the test bridge and details about the post processing methods that were applied to the modal data. In most cases, this consists of figures illustrating the accelerometer locations and impact points for each of two tests per bridge (in most cases). A Complex Mode Indicator Function (see Chapter 3 for details) is shown to illustrate how the modes were selected. Next, the frequencies and mode shapes are presented for the first 8 modes are presented. Finally, the results of a truncation study are presented to determine how many modes are required in a flexibility analysis to adequately characterize the dynamic behavior of the bridge.

The second part of each bridge report focuses on the FE modeling and model calibration. The first thing that is presented in this section is the weighting factors that are incorporated in the global objective function (GOF) (see Chapter 7 for details). Next, the magnitudes of the GOF and individual objective functions are plotted so to graphically illustrate the effectiveness of the tuning through several rounds. Finally, the response indices (modal frequencies, mode shapes, MAC values, UIL correlation, BGCIs, etc.) are presented so that the nominal and tuned FE results can be compared and contrasted with the experimental results.

The third part of each bridge report includes the details of the truckload test and the approach used in load rating the structure. When truckload data is available, ratings are provided for based on both the field data and the calibrated FE model. When truckload data is not available for a structure, the load rating is based only on the calibrated FE model.