Bridge Type Specific Management of Steel Stringer Bridges: Development of Field Calibrated Software Rating Tools and Statistical Bridge Database

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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:


The current 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. A companion 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. *Objective Condition Assessment of Damaged or Deteriorated Bridges in Ohio*, 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 30 specimens considered in this research project.
Chapter 1 Introduction, Background, and Objectives

1.1 INTRODUCTION

1.1.1 Synopsis of the 24 Bridge Project Research

The Ohio Department of Transportation (ODOT), in conjunction with the Federal Highway Administration (FHWA) and other federal and state agencies (e.g., National Science Foundation, Ohio Board of Regents, etc.), has made a very significant investment into bridge research in the past decade. During this period the University of Cincinnati Infrastructure Institute (UCII) was given the opportunity and responsibility to develop and demonstrate new technologies for bridge testing, field-calibrated finite-element modeling, condition evaluation, and rating. Several powerful experimental, nondestructive evaluation techniques, including modal analysis and instrumented truckload testing, were indeed successfully developed and applied to various types of bridges. In the last five years, the focus has been on steel-stringer bridges (reinforced-concrete deck on steel girders) which are the most common bridge type in Ohio and the nation.

This report covers the development of a first of its kind, reliable, and accurate field-calibrated bridge-specific rating tool so that steel-stringer bridges may be accurately rated for any axle configuration of standard as well as non-standard trucks and overloads. The main deliverables from this project are (1) a database of structural test results from a total of 30 steel-stringer bridges in Ohio and (2) an interactive, user-friendly, PC-based, modeling, simulation and rating software package.

The database has been assembled by field testing a statistical sample of 24 steel-stringer bridges and these results of these tests have been merged with the results obtained from 6 bridges previously tested to provide a database of 30 bridges.

The field test information contained in the database has been incorporated into the software package through the use of model calibration studies. In addition, correlation studies
serve to establish the match between the objectively acquired test results contained in the database and visual inspection data already obtained by ODOT on the bridges tested. If test results from impact and/or truckload tests are available, the software permits the incorporation of this objective information into the rating process as well.

1.2 SYNOPSIS OF PREVIOUS RESEARCH EFFORTS

The work covered in this report is a follow-up to previous research efforts that validated and established field testing methods (primarily truckload testing and modal testing) for the purposes of damage detection and condition assessment (Phase I). A decommissioned bridge in Cincinnati at Seymour Avenue (HAM-561-0683) was the original test specimen. The results were presented to ODOT officials and administrators on Wednesday, May 28, 1997. The following list is a brief synopsis of the key findings:

- The basis for a practical, field implementable bridge condition assessment methodology was developed. This methodology utilizes visual inspection, modal analysis, diagnostic truckload testing, structural flexibility, and unit influence line decomposition as core components.

- This methodology is effective in dealing with and deciphering issues that arise in field work as a result of uncertainties brought about by the interaction of bridge structural state/condition, ambient conditions, boundary condition ambiguities, and experimental errors. It has the capability to detect, isolate, and quantify the location and cause of a variation in bridge state.

- Structural flexibility has been revealed as a practical, reliable index which is sensitive to structural condition and damage. Flexibility and influences lines, unlike other aspects of bridge behavior (such as dynamic behavior), are not significantly influenced by environmental/ambient changes and can be easily extracted from identification experiments involving modal analysis and instrumented monitoring.

The results above were acquired through practical implementation of the structural identification concept. Based on these results, it is believed that further advances in bridge
engineering and management may be obtained if the concept of structural identification and its
associated tools (i.e., modal analysis, diagnostic truckload testing, flexibility, etc.) are further
refined and applied to a statistical database of bridges.

In order to accomplish this ultimate goal, the necessary field test, post-processing, and
analytical modeling procedures have been developed, streamlined, and validated through a set of
proof-of-concept studies conducted on an initial sample of 6 bridges. These 6 bridges comprised
an extension to Phase I whose objectives, as well as the scope and implementation strategies, are
discussed below.

1.2.1 Portable Testing Instrumentation Setup

The hardware, software, and instrumentation required for impact testing and instrumented
truckload testing have been incorporated into a self-contained, highly mobile, field operational
package which can be transported and rapidly applied anywhere within the state. All necessary
equipment and supplies (including on-site power requirements) can be transported and all testing
operations staged from within a cargo van.

1.2.2 Streamline On-site Test/Post-Processing Capabilities

Efficient methods for test setup, test execution, and test tear-down were developed for
both impact testing and instrumented truckload testing. Software capabilities have been
developed so that test data can be post-processed at the bridge site. This permits essentially real-
time data interpretation which allows on-the-fly testing modifications, thereby maximizing the
efficiency of the field operations and minimizing the impact on traffic and ODOT. A full set of
field tests (including modal and truckload tests) can be run on a short-span steel-stringer bridge
within 1-2 working days.

1.2.3 Efficient Finite-Element Model Generation/Calibration

Detailed, field-calibrated, 3D FE models have been developed for each of the 6 bridges
tested under this extension. Moreover, 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.). These products will form the core components
of the software package used to generate a generic model that may be used to model any steel-
stringer bridge.
1.2.4 Load Rating

UCII has spent a great deal of time developing rating algorithms based on the various AASHTO-approved rating methods (e.g., load factor design, allowable stress method, inventory rating, operating rating, etc.) to incorporate automated routines for bridge rating into its software package. Methods which permit load rating based on either direct field measurements (through the use of unit-influence line decompositions of truckload testing results) or field calibrated models have been developed.

As an example of the kinds of results obtained during the Phase I Extension, the main results from the testing of PRE-725-0800 and BUT-732-1043 are presented in Figures 1-1 and 1-2. These figures show the results of both modal and truckload tests. A similar set of findings were obtained for the other bridges.

Figure 1-1: The Use of Modal Test Based Flexibility as a Raw/Direct Bridge Condition Indicator
The deflection profiles in Figures 1-1 and 1-2 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. What is noteworthy about the deflection profile within Figure 1-1 are the nonzero deflections at the east abutment, which indicates that the girder is not properly bearing, or supported, at that location. Furthermore, the figure 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 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 (as well as the other bridges in the Phase I extension), Girder 3, the center girder, is the common girder between the two respective traffic lanes on a bridge. Superimposing the Girder 3 deflection profiles developed from the flexibility matrix for each lane essentially reveals that, irregardless 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 1-3 and 1-4 illustrate the results of the crawl speed truckload testing on BUT-732-1043. The instrumentation plan/layout is illustrated at the bottom of each. It shows the placement of 8 strain gages at 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 impact modal test.

Figure 1-3 shows substantial stress felt along the top flange in the middle span compared to the end spans, indicating a significant loss of 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 was visibly evident upon inspection.

Figure 1-4 provides the derived live load moment and corresponding rating factors for the simulated midspan response to an HS20-44 truckload crossing. Post-processing algorithms were
used to decompose the measured two axle truck response 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 1-4). 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. Bridge Analysis and Rating System (BARS) software), because it accounts for the true distribution of load.

Figure 1-4: Post-Processing of Truckload Test Data to Obtain Bridge Ratings

In an attempt to have an apples-to-apples comparison with the current ODOT assessment, the rating factors shown in Figure 1-4 are based upon the girder capacity alone (i.e. non-composite behavior at the limit state) of 1,000 kip-ft (1,356 kN-m) and upon a dead load of 270 kip-ft (366 kN-m) calculated from the measured physical dimensions (e.g. deck thickness of 9.25 in (23.5 cm)).
In a parallel effort, UCII personnel in the laboratory have been working towards the development of software portions of the research. Figure 1-5 provides a flowchart layout of the software package.

Figure 1-5: UCII Field-Test-Based Rating Strategy

The front end of the package consists of a pre-processor module which provides a user-friendly interface to generate the necessary input file for use with SAP2000. This module results in a nominal 3D FE model based on the dimensions and properties obtained from bridge plans. The user can easily define the length, width, number of spans, skew, cross-brace arrangement, girder spacing, sidewalks, parapets, etc. The pre-processor module is also capable of generating the necessary loading conditions to simulate the virtual uniform loading to be used in the correlation to experimental deflected shape under the same loading.

The initial 6 bridge FE models were generated using the pre-processor software. Next, all 6 were manually calibrated using the modal experimental data (mode shapes, natural frequencies, deflected shape).
For the calibration phase, many post-processing programs are needed. As the nominal model is analyzed, the SAP2000 program generates many output files which are each thousands of lines long. Medium stage post-processor programs read and sort through the output files, select the corresponding responses for the experimental results and compare. Manual calibration involves changing structural parameters in the FE model in an effort to minimize the differences between the experimental and analytical results.

The analysis is conducted many times as one or more structural variables are manually changed. This trial and error approach was organized into a logical sequence. The calibration process was further investigated and automated as a part of this research effort.

Load rating is initiated after the analytical results are closely matched with their experimental counterparts. For rating a bridge, HS20-44 truck and lane loads are defined. To find the governing condition, the truck location is moved in 3 ft (0.91 m) increments (~70 loading cases) and the maximum and minimum force developed in each model member is found. The bridge member with the greatest force is found for each span which is called the maximum of maximums. This value is used as a basis for the rating.

1.3 OBJECTIVES OF 24 BRIDGE PROJECT

Based on the successes of Phase I and the Phase I extension, UCII personnel have developed an efficient, reliable, portable field test capability. Both modal and diagnostic truckload testing are conducted in an integrated fashion to optimize efficiency in the field. Total test time, including bridge measurement and marking, sensor mounting, data acquisition, and tear-down is currently running at about 12-14 hours per bridge, i.e. 1-2 working days in the field. This permits time for modal testing of two lanes, as well as crawl speed truckload testing. In addition, key components of an automated bridge analysis, modeling, and rating software package have been developed and debugged.

This work has allowed UCII personnel to undertake the next phase of the research: selection, testing, structural identification, and modeling of a full statistical sample of bridges.

The basic objectives of the research are as follows:
• Identify a statistical sample of 24 steel-stringer bridges and perform field testing (i.e. truckload and modal testing) and structural identification studies on each, resulting in a field-calibrated 3D FE model of each bridge.

• Assemble a comprehensive statistical database of actual structural identification results, rigorously calibrated FE models, and accurate rating factors for 30 bridges (the original 6 bridges from the Phase I extension and the additional 24 bridges tested under this research project) representing a baseline for the general steel-stringer bridge population within Ohio.

• Develop an interactive, user-friendly, PC-based modeling and rating software package that can be used to model and rate any steel-stringer bridge. This package will be unique in that it will recognize the actual 3D structural response mechanisms of the bridge and will incorporate both the generic information contained in the statistical database described above and the as-is condition based on the latest inspection and/or test results available for the bridge.

• Verify and validate the practicality and structural legitimacy of the modeling and rating software and explain to potential users how they can model and interpret the as-is conditions of an inspected steel-stringer bridge from the database that has been generated

Correspondingly, the deliverables that were developed in accomplishing these objectives are:

• a comprehensive database of actual structural identification results, rigorously calibrated FE models, and accurate rating factors for 30 bridges representing the general steel-stringer bridge population within Ohio,

• the incorporation of all testing and database findings/observations into a modeling/simulation and load-rating software package; and

• training sessions for selected ODOT engineers on how to use this software.
The 30 steel-stringer bridges tested represent various ages, design attributes, construction attributes, condition, and records of performance. Based on UCII’s earlier research efforts (i.e. Phase I and Phase I extension, etc.), the field operations have been designed to yield objective, experimental information on the characteristics and behavior of these bridges. As a result, the measurements obtained are suitable for generating a statistical database representative of typical steel-stringer bridge behavior, one which can be used as a baseline of what may be considered normal and customary bridge behavior.

In addition, because of the different attributes that were considered when selecting the test bridges, the information within the database reflects the influence that design, construction, and maintenance decisions have on actual bridge condition and performance. As a result, performing simulation studies using the database measurements will help reveal how different structural and material characteristics, maintenance decisions, etc., relate to service performance and life-cycle cost. This research will thus provide potential insights into improvements in design and maintenance practices and help move ODOT towards a more “type specific” management strategy.

Finally, field testing each of the 30 bridges will provide the objective data needed to develop customized, “field calibrated” models for use in simulation and load-rating. The aforementioned software package will permit an engineer to interactively generate a several levels of model customization for a particular steel-stringer bridge:

- Through software prompts, the engineer will be able to describe the geometry, dimensions, member properties, sub-structure, and support conditions of the respective bridge. This corresponds to the lowest level of customization and will result in a 3D FE model of the structure tuned to the as-designed parameters of the bridge.

- The next level of customization of the model will be accomplished by allowing for the “calibration” of the model against the in-situ behavioral patterns and observed characteristics (natural frequencies, mode shapes, flexibility, deflections, etc.).
• A third and innovative level of customization will be accomplished by allowing for the further calibration against visual inspection results obtained for the particular bridge. For example, if deck concrete is documented as extensively cracked and spalled, the software will permit incorporation of such information into the model.

• Finally, in the event actual field test data is available for the particular bridge under consideration, the software package will allow for calibrations against these field measurements in order to obtain the most accurate, objective model of bridge characteristics and behavior.

In this way the software package was configured to make the maximum use of all information available (i.e., statistical database, visual inspection, and field test measurements) in modeling a given bridge. As a result, each bridge model obtained provided a comprehensive and objective representation of the bridge that can be used to compute the most accurate demands and load-ratings.

1.4 STATISTICAL CONSIDERATIONS

The sample size of 30 bridges for testing is based on standard statistical methods [Aktan et al. 1996]. The argument proceeds as follows: Suppose that a given bridge parameter, X, is to be estimated based on the sample mean of a population of measurements \( (X_1, X_2, X_3, \ldots, X_N) \). Then, it is intuitively obvious that the quality of the estimate is a function of both the population size, \( N \), and the quality of the measurements, \( (X_1, X_2, X_3, \ldots, X_N) \). This intuition can be quantified as follows: The probability that the estimation error (i.e., \( X - X_{\text{Mean}} \)) exceeds some limit \( e \) can be described by the equation

\[
P\left(\left| X - X_{\text{Mean}} \right| \leq M \sigma \right) = e
\] (1-1)

where \( \sigma \) denotes the standard deviation of the measurement distribution (i.e., a measure of the quality of the measurement data), and \( M \) is a function of the sample size and is plotted in Figure 1-6. Using these results, one can see that in order to obtain a parameter estimate with a 99% probability of being within 1 standard deviation of the actual parameter value, the estimate should be based on a population sample size of at least 30.
Chapter 1 - Introduction, Background, and Objectives

Good estimation of parameters (i.e., 99% confidence) requires sample sizes of 30 or more.

<table>
<thead>
<tr>
<th>Std. Dev.</th>
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<td>Number of modes</td>
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<td>Critical mode</td>
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<tr>
<td>First mode</td>
<td>0.72</td>
</tr>
<tr>
<td>Second mode</td>
<td>0.80</td>
</tr>
</tbody>
</table>

Standard Deviations based on spread in 6 bridges tested in Phase I Extension

Figure 1-6: Effects of Sample Size on Estimation Error

1.5 SCOPE OF 24 BRIDGE PROJECT

The research effort can be broken up into several tasks described as follows:

1.5.1 Task 1: Bridge Inventory and Site Selection

As with the 6-bridge Phase I extension project, the steel-stringer bridges serve as test sites that were selected through close collaboration with the ODOT Office of Structural Engineering. The criteria for selecting test sites were:

- **Bridges Selection**: For example, bridges located along an overload route may be deemed desirable test specimens.

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

- **Climate/Environmental Factors**: The influence of such factors as humidity, wind conditions, temperature, use of road salt, etc., was incorporated.

- **Traffic Environment**: The influence of such factors as traffic count, type, speed, etc., was incorporated.
• **Family and Parameter Groupings:** The influence of material characteristics and structure specific design attributes such as composite/non-composite design, skew, girder attributes (number of girders, continuous or simple span, etc.), abutment and foundation type, bearing type, details vulnerable to catastrophic failure, etc., was incorporated.

The selection process was guided by the current Ohio steel-stringer bridge inventory, that is, the current inventory was examined to help establish the number and distribution of steel-stringer bridges with particular structure-specific parameters groupings. This information was used to determine which steel-stringer bridge types should comprise the test bridge population.

1.5.2 **Task 2: Field Operations**

The field testing component of the project has several sub-components:

1.5.2.1 **Initial Database and Analytical Model Development**

An archival database was generated for the selected test bridges. For each bridge, this database contains all available information on design, shop-fabrication, construction, inspections, maintenance, rehabilitation, and existing load-ratings. Regional geography, weather, and local site and environmental conditions were documented as well. All information accrued within the database in this manner was used to develop preliminary analytical models of each respective bridge. These preliminary models helped in the design of field testing operations as indicated below.

1.5.2.2 **Impact Testing**

Impact modal testing methods were also applied at bridge sites. The sensor layout used for such testing requires that numerous accelerometers be positioned on the upper-side of the bridge deck directly above all girder-cross frame intersections and girder-bearing positions associated with the girders under the lanes. Impacts are applied at up to 6-8 of these sensor positions, as indicated in Figure 1-7. To ensure proper excitation and acquisition of pertinent modal characteristics (natural frequencies, mode shapes, etc.), the dynamic characteristics defined by a preliminary finite element model of the bridge were used to specify the exact sensor layout and impact point locations.
For each lane, the average test duration was approximately one hour. Ambient conditions such as temperature and humidity typically do not experience large variations within this short period, thus using this sensor layout minimizes the influence that ambient variations have on bridge behavior during the course of the test and maximizes the quality of test data. In addition, the global and local modal characteristics of the bridge superstructure along two or more girder lines can be measured and acquired when using this layout. A reliable flexibility, one that properly evaluates and assesses the behavior of the tested region, can subsequently be computed from the modal data acquired in this manner.

PC based software has been developed which permits flexibility computation at the bridge site. Hence, impact testing, impact data post-processing, and flexibility computation can all be performed within a single day at the bridge site.
Using this unique impact test method and its associated hardware and software, impact tests were performed at each of the selected bridge sites, and the resulting natural frequencies, mode shapes, and flexibility were used to calibrate its respective FE models as described below.

**1.5.2.3 Diagnostic Truckload Testing**

Crawl speed diagnostic truckload testing methods were applied at 20 of the 30 bridges selected for this project. Such load tests use trucks of known weight to apply a load pattern to a test bridge. As with the impact testing, an analysis involving the preliminary (uncalibrated) 3D FE model of the bridge was conducted to help determine the maximum response measurement locations for both positive and negative moment so that sensors could be optimally located.

While the required access to the girders limits the number of installed gages, UCII has been able to successfully utilize such tests to acquire information about localized bridge response mechanisms, such as the forces/deformations that developed in local regions due to the applied load and how the applied load was distributed along longitudinal and transverse directions. To permit proper simulation of all critical behavior mechanisms, the measured local responses obtained from these tests were used as part of the calibration data for the bridge FE modeling effort.

In addition, crawl-speed tests were conducted for each traffic lane to directly determine the unit influence lines for immediate HS20 and lane load rating of these critical locations. Note that this latter utilization of the truckload tests was developed during the recent 6-bridge extension by integrating several key results from earlier ODOT-funded projects. A reliable influence line, one that properly simulates the member’s behavior in the instrumented region, can subsequently be computed from the truckload data acquired in this manner.

Using the truckload measurements together with a sectional analysis will allow for an estimate of the live load moment for load factor design assessment. PC based software has been developed which permits the computation of influence lines, section properties, and rating factors at the bridge site. Hence, truckload testing (using 15-20 gages), data post-processing, and rating factor computation for the instrumented sections can all be performed within a single day at the bridge site.
1.5.3  Task 3: Development of Modeling and Rating Software

The software development component of the project also has several sub-components.

1.5.3.1  3D Finite Element Calibration

Concurrent with the field test operations, the preliminary finite element model of each test bridge was calibrated to conform to the experimentally determined dynamic characteristics, flexibility, strains, deflections, etc. In order to perform model calibration, certain structural and material properties within the model, such as support spring stiffness, amount of composite action, concrete modulus of elasticity, etc., were selected as variables.

The calibration process itself involved iterations on these variables until the model yields state parameters (e.g. natural frequencies, mode shapes, flexibility, etc.) that match those measured during impact and truckload testing. Calibrations were performed in this manner for the 3D finite element model of each bridge using the SAP2000 analysis software as a basis for the simulation and modeling. At first, manual iteration techniques were employed but ultimately an automated iteration algorithm was developed. This automated model calibration routine is an integral part of the rating software developed through this research.

1.5.3.2  Load-Rating with Calibrated Models

Load ratings for a test bridge were computed after the models of the bridge have been calibrated. Model-based load ratings were performed and compared against the directly computed truckload-test-based ratings and any existing ODOT BARS-based ratings as an additional check to ensure that the models adequately simulate bridge. These ratings were calculated in accordance with the AASHTO Manual for Condition Evaluation of Bridges (AASHTO-CEM) [AASHTO 2000]. Section capacities were defined with the AASHTO Standard Specifications for Highway Bridges (AASHTO-STD) [AASHTO 2002].

1.5.3.3  Rating Software

As indicated earlier, the model based rating software utilizes information about bridge condition that has been acquired through an Ohio DOT mandatory visual inspection. Consequently, the research has established quantitative relationships between visual inspection-
based assessments of condition and objective measures of condition. Such relationships, as were established, were incorporated into the proposed rating software.

This program has a series of input menus that permit the user to enter the geometric and material characteristics of a steel-stringer bridge. This particular information generates a nominal model of the bridge, which is then calibrated using available test (impact, truckload) data and/or the quantitative relationships established between visual inspection-based condition and objective measures of condition. A load-rating of the bridge is performed after the respective grid model has been calibrated.

1.5.4 Task 4: Understanding/Addressing Serviceability Issues

By design of the selection process, the 30 test bridges represent various ages, design and construction attributes, condition, and records of performance. Correspondingly, the objective, experimental information acquired through field testing of these bridges reflects the influence of design, construction, and maintenance decisions on bridge condition and performance. The actual measured service responses (deflections, strains, vibrations, etc.) were evaluated to determine which response characteristics correspond to satisfactory service performance and which are associated with unsatisfactory service performance. This evaluation helps reveal how different bridge management decisions (design, construction, maintenance, etc.) influenced and affected bridge service performance and life-cycle cost.

1.6 FINAL PRODUCT AND BENEFITS

The completion of this project has provided the following products:

1) A model-based load-rating software package that can generate a field calibrated model and compute load-ratings for steel-stringer bridges. Essentially, the calibrated model developed with this software provides a comprehensive and objective representation of the bridge. The software uses this calibrated model to compute demands and load-ratings.

2) An experimental, objective database containing information about the state properties of the selected test bridges. This database provides a statistical representation of
typical steel-stringer bridge behavior. In other words, provides a statistical baseline of what may be considered normal and customary bridge behavior.

3) A report that documents what service level measurements and state parameters are characteristic of either satisfactory or unsatisfactory steel-stringer bridge service performance. This report indicates which bridge management (design, construction, and maintenance) decisions lead to satisfactory or unsatisfactory service performance.

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

In essence, testing 30 steel-stringer bridges (the original 6 bridge extension + additional 24 bridges) permitted the development and implementation of new procedures for inspection, rating, and management. Such procedures will be more objective than existing methods and will be along the spirit of the AASHTO Guide for Strength Evaluation (AASHTO-GSE) [AASHTO 1989].
Chapter 2 Bridge Selection Criteria and Test Specimens

2.1 BRIDGE INVENTORY AND SITE SELECTION

Bridges were selected to permit correlation of the results to the data in ODOT’s database. This correlation would reveal the accuracy and completeness of ODOT’s inventory data and current load-rating approaches and help identify the impact that design attributes and details have on structural serviceability and safety. As with the 6 bridges selected within the Phase I Extension project, the steel-stringer bridges that served as test sites were selected through close collaboration with the ODOT Office of Structural Engineering. As earlier indicated the criteria for selecting test sites was:

- **Bridges Selection**: For example, bridges located along an overload route may be deemed desirable test specimens.

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

- **Climate/Environmental Factors**: The influence of such factors as humidity, wind conditions, temperature, use of road salt, etc. was incorporated.

- **Traffic Environment**: The influence of such factors as traffic count, type, speed, etc. was incorporated.

- **Family/Parameter Groupings**: The influence of material characteristics and structure specific design attributes such as composite/non-composite design, skew, girder attributes (number of girders, continuous or simple span, etc.), abutment/foundation type, bearing type, details vulnerable to catastrophic failure, etc. was incorporated.
The selection process was guided by the current Ohio steel-stringer bridge inventory and is shown in Table 2-1. That is, the current inventory was examined to help establish the number and distribution of steel-stringer bridges with particular structure-specific parameter groupings. This information was used to determine which steel-stringer bridge types should comprise the test bridge population.

### Table 2-1: Bridge Selection Criteria

<table>
<thead>
<tr>
<th>GENERAL BRIDGE TYPE</th>
<th>Continuous steel rolled beams w/ cast-in-place RC deck slab</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>• excludes bridges w/ hinges, curved, and flared beams</td>
</tr>
<tr>
<td></td>
<td>• excludes two-main-member fracture critical structures</td>
</tr>
<tr>
<td>HIGHWAY TYPE</td>
<td>Main Line</td>
</tr>
<tr>
<td></td>
<td>• state or interstate route on bridge</td>
</tr>
<tr>
<td>BRIDGE AGE</td>
<td>Built After 1950</td>
</tr>
<tr>
<td>SKEW ANGLE</td>
<td>High and Low</td>
</tr>
<tr>
<td>OVERALL WIDTH</td>
<td>High and Low</td>
</tr>
<tr>
<td>NUMBER OF SPANS</td>
<td>1 to 5</td>
</tr>
<tr>
<td>OVERALL LENGTH</td>
<td>&lt; 400'</td>
</tr>
<tr>
<td>ADTT</td>
<td>&gt;2500</td>
</tr>
<tr>
<td>GENERAL INSPECTION APPRAISAL</td>
<td>6 or better</td>
</tr>
<tr>
<td></td>
<td>• bridges are open to normal highway traffic</td>
</tr>
<tr>
<td>ODOT DISTRICT AND COUNTIES</td>
<td>Anywhere in Ohio</td>
</tr>
</tbody>
</table>

#### 2.2 ORIGINAL SIX TESTED BRIDGES (PHASE I EXTENSION)

The six bridges selected within the Phase I Extension were all very similar to each other in terms of bridge geometry and design specifications. They were 3-span, 5-girder bridges located within District 8. All six bridges were designed to be non-composite with non-integral abutments and none of them had a span length greater than 100 ft (30.5 m). Five of the six bridges did not have skew; one bridge had a skew angle greater than 10°. The top portion of Table 2-2 has the specs for these six bridges. The goal for this project was to streamline the field testing and post-processing capabilities of UCII and provide objective structural identification of
the tested bridges. Therefore the bridges in this project were intentionally selected because of their similarities to verify that this process was repeatable.

Table 2-2: Bridge Specifications and Parameters

<table>
<thead>
<tr>
<th>Bridge</th>
<th>SFN</th>
<th>Dist.</th>
<th>No. of Spans</th>
<th>No. of Girders</th>
<th>Span</th>
<th>Constructed Type</th>
<th>Abutment Type</th>
<th>Max Span Length</th>
<th>Deck Width</th>
<th>ADTT</th>
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<tbody>
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<td>BUT-73-1043</td>
<td>234154</td>
<td>8</td>
<td>X</td>
<td>X</td>
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<td>X</td>
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<tr>
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<td>1860625</td>
<td>8</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
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<td>X</td>
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24 Bridge Project

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<th>No. of Girders</th>
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<td>8</td>
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</tr>
<tr>
<td>LEO-79-1550L</td>
<td>3101743</td>
<td>8</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
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<td>X</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

2.3 SELECTION OF ADDITIONAL 24 BRIDGES

After successful completion of the Phase I Extension, an additional 24 bridges were selected with the intention of statistically representing the various bridge design specs within ODOT’s inventory. This included, but was not limited to, representing the various districts within Ohio, the overall length of the bridge, number of spans, number of girders, skew angle, age, etc. Bridges with design attributes different than the original six bridges were selected in an incremental fashion; each selected bridge was slightly different than the previous one. Since field-testing methods were developed on six similar test sites the goal was to use this experience and apply it to bridges of varying geometry to statistically represent ODOT’s inventory. At the beginning of this selection process bridges were selected along mainline routes to verify that field testing could be applied to heavily trafficked bridges, while minimizing traffic interruptions. Each bridge is briefly described below.
2.3.1 ATH-33-2306 (0501123)
Located in District 10 on State Route 33 this is the only 2-span bridge tested during this project. Its other main attributes are: 5 girders, non-composite deck, skew angle greater than 10°, a maximum span length less than 100 ft (30.5 m) and a low daily truck count. (No photo, cannot find on ODOT website)

2.3.2 BRO-52-0554 (0800813)
Located in District 9 along State Route 52 this is the only 5-span bridge tested during this project. This bridge has 5 girders, no skew, a maximum span length less than 100 ft (30.5 m) and was not designed for composite action.

2.3.3 BRO-763-0056 (0803774)
Located in District 9 on State Route 763 this is another 1-span bridge. Unlike the 2 previously tested single span bridges, this bridge has 4 girders, was designed for composite action and has a low daily truck count.

2.3.4 CUY-77-0645L & R (1806181 & 1806211)
Located in District 12 on Interstate 77 these bridges have 3-spans, no skew, non-integral abutments, a maximum span length less than 100 ft (30.5 m) and were built for non-composite action. They were selected due to their high daily truck traffic as well as being 7-girder bridges. The extra girders provide a bridge of greater width than previously tested and the selection of these bridges helped establish how wider structures differ in behavior than bridges with less girders.

2.3.5 FAI-33-1309 (2301067)
Located in District 5 on State Route 33 this bridge does not have a high volume of truck traffic. It has a maximum span length greater than 100 ft (30.5 m), a skew angle greater than 10° and was designed for composite action. This specimen was selected to represent the bridge population of 4-span, 5-girder bridges. It should be noted that this bridge was built using high-performance steel and was constructed with integral abutments.
2.3.6 **HAM-126-1279L (3104850)**

Located in District 8 along State Route 126, it is a 3-span, 5-girder bridge with a maximum span length less than 100 ft (30.5 m) and non-integral abutments, which are all common to the six bridges selected during Phase I. Unlike the previous six bridges tested, this bridge was designed for composite action, has a skew angle greater than 10° and has a high daily truck count.

2.3.7 **HAM-126-1317L (3105172)**

Also located in District 8 on State Route 126, this bridge is similar to HAM-126-1279 in terms of number of girders and spans, non-integral abutments, truck traffic, composite action and skew angle. However, the maximum span length is greater than 100 ft (30.5 m) resulting in a larger bridge than had been previously tested. Again, the goal was to select a bridge with slightly different design specs from those already tested.

2.3.8 **HEN-108-0826 (3502279)**

Located in District 2 on State Route 108 this bridge is very similar to HEN-108-0826. It is a 1-span, 5-girder bridge with a skew angle greater than 10° and a maximum span length less than 100 ft (30.5 m).

2.3.9 **HOC-56-1647 (3701506)**

Located in District 10 this bridge is very similar to WIL-34 and HEN-108. This bridge was selected because it does not have a high daily truck count. All other selection criteria are consistent between these 3 bridges. By adding this test specimen, bridge performance can be correlated to truck traffic for single span bridges.

2.3.10 **LAK-90-1641L & R (4304624 & 4304659)**

Located in District 12 on Interstate 90 these two bridges have 3-spans, no skew, non-integral abutments and were designed for non-composite action. They were selected since they have 6 girders, a maximum span length greater than 100 ft (30.5 m) and a high daily truck count. Although a bridge with a span length greater than 100 ft (30.5 m) has been tested it had 5 girders.
2.3.11 MAD-70-1555L & R (4902858 & 4902882)
Located in District 6 on Interstate 70 they are 3-span, 7-girder bridges with no skew, non-integral abutments, a maximum span length less than 100 ft (30.5 m) and were built for non-composite action. Their attributes are very similar to the CUY-77 bridges already tested.

2.3.12 MOT-70-2210 (5706270)
Located in District 7 on Interstate 70 this bridge also has 4 spans, like the RIC-30 bridges, but it only has 4 girders instead of 6 and a skew angle greater than 10°. The other main attributes are consistent with the majority of the previous bridges, i.e. high daily truck count, non-composite action, non-integral abutments and a maximum span length less than 100 ft (30.5 m).

2.3.13 MOT-75-0776 (5706939)
Located in District 7 on Interstate 75 this bridge is very similar to MOT-70-2210. It has 4 spans, 4 girders, non-composite action, etc. The skew angle is the difference between these bridges, with this bridge having a skew of less than 10°.

2.3.14 PER-204-0166 (6402089)
Located in District 5 on State Route 204 this is another single span bridge with a low daily truck count. Unlike HOC-56 this bridge has 7 girders and not 5, all other selection criteria is consistent between these 2 bridges.

2.3.15 RIC-30-1384 (7001320), RIC-30-1438 (7001479), RIC-30-1638 (7001517)
Located in District 3 along State Route 30 these bridges have 6 girders, no skew, non-integral abutments, a high daily truck count and were designed for non-composite action, attributes all similar to the LAK-90 bridges. However, they are the first specimens tested that do not have 3 spans. They all have 4 spans which represent another selection parameter that has been changed to help reflect ODOT’s inventory.

2.3.16 TUS-212-1509 (7904533)
Located in District 11 on State Route 212 this is the first bridge selected that does not have a high daily truck count. It is similar to the 6 original bridges tested, the only difference
being it is located in eastern Ohio, instead of the southwest. Local environmental effects are the distinguishing variable between this bridge and the original 6.

2.3.17 TUS-751-0420 (7906307)

Located in District 11 on State Route 751 this is the second bridge selected that does not have a high daily truck count. It is also the first selection to be a 3-span, 4-girder bridge. 3-span bridges and 4-girder bridges have been included in the database; however no bridge has had both. The other attributes are consistent with previous selections; a skew angle less than 10°, non-composite action, maximum span length less than 100 ft (30.5 m) and non-integral abutments.

2.3.18 WIL-34-2722 (8601747)

Located in District 2 on State Route 34 this is the first bridge tested to have less than three spans. It is a 1-span, 5-girder bridge with a non-composite deck, a skew angle greater than 10° and a maximum span length less than 100 ft (30.5 m). Other than the number of spans, the attributes of this bridge are consistent with those already tested.

2.3.19 WOO-281-1943 (8706336)

Located in District 2 on State Route 281 this bridge does not have a high volume of truck traffic. It has a maximum span length less than 100 ft (30.5 m), was not designed for composite action and has a skew angle greater than 10°. This bridge was selected provide representation of 3-span, 4-girder bridges within the statistical database.

2.4 COMPARISON OF BRIDGE TEST POPULATION VERSUS ODOT INVENTORY

The bottom portion of Table 2-2 shows the specs for the 24 bridges presented above. Figure 2-1 compares the attributes of the 30 bridges tested by UCII versus the ODOT inventory of steel-stringer bridges. As seen in this figure, the population of 30 bridges does a reasonable job of statistically representing the ODOT bridge inventory.

The 30 bridges tested represent various ages, design and construction attributes, condition and records of performance. The data obtained from field testing reflects the influence of design, construction and maintenance on bridge condition and performance. It also provides a statistical
representation of “typical” steel-stringer bridge behavior, a baseline of what may be considered normal bridge behavior.

Figure 2-1: Comparison of UCII Test Database and ODOT Inventory
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.

![Figure 3-1: Drop Hammer Response](image)

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></th>
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</thead>
<tbody>
<tr>
<td>Channels</td>
<td>8 - 48</td>
</tr>
<tr>
<td>Frequency Spans</td>
<td></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>
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<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>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

External Power Supply (PCB Model 584A) used to "power up" sensors

Figure 3-2: HP Network Analyzer System

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

<table>
<thead>
<tr>
<th>DYNAMIC</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Measurement Range</td>
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</tr>
<tr>
<td>Resolution</td>
<td>0.0001 g</td>
</tr>
<tr>
<td>Nominal Sensitivity</td>
<td>1000.0 mV/g</td>
</tr>
<tr>
<td>Frequency Range</td>
<td>0.025 - 800 Hz</td>
</tr>
<tr>
<td>Resonant Frequency</td>
<td>&gt; 3500 Hz</td>
</tr>
</tbody>
</table>

### 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 3-3: PCB 393C Accelerometer
Accelerometers were calibrated using a drop method approach (Figure 3-5) and the hammer utilized a simple ratio technique (Figure 3-6).

Drop Calibration is based on the concept that a body in free-fall experiences a constant 1 g gravitational acceleration

- Sensitivity is determined by measuring the voltage step response of the accelerometer to a 1 g input
  - Accelerometer is mounted to a thin, flexible line which is suspended from a junction with two elastic suspension cords (these cords are "fixed" at their opposite ends)
  - Thin, flexible line is initially taut due to accelerometer weight
  - Striking the junction causes the thin, flexible line to relax - accelerometer free-falls for a brief instant
  - Accelerometer experiences 1 g acceleration during free-fall and voltage output from step response determines sensitivity (=ΔV/1 g)

- Actual drop calibration differs from ideal free-fall step response
  - DC bias voltage, V0, may exist when accelerometer at rest
  - When junction is struck, thin, flexible line does not instantly relax causing a "ramp-up" in response signal
  - Quartz crystal within piezoelectric transducers does not hold constant charge - crystal causes response signal to exponentially decay during free-fall
  - Free-fall ends when line becomes taut again and accelerometer bounces

- Exponential decay is extrapolated to determine the initial voltage amplitude of the step response, Vs
  - ΔV = Vs - V0; thus, sensitivity = ΔV/1 g
CALIBRATION PROCEDURES

Impact Hammer and Reference Load Cell Calibrations

- Impact hammer calibration
  - Hammer, with internal load cell, is impacted upon a reference load cell of known sensitivity (V/lbf)
  - Voltage output is recorded from hammer load cell, force (lbf) output recorded from reference load cell
  - Ratio of peak voltage to peak force (which should occur at same instant in time), defines hammer sensitivity

- Reference load cell calibration
  - Suspended mass, with attached reference load cell and calibrated accelerometer, is modeled as a single degree of freedom system using $F = Ma$
  - $H$, the frequency response of single DOF system, is $1/M$, the inverse of system's total mass - knowing $H$ permits backcalculation of reference load cell sensitivity
    - Impact suspended mass assembly at the load cell
    - Record, in time domain, a voltage output ($V$) from load cell and acceleration output ($g$) from accelerometer
    - Backcalculate reference load cell sensitivity using $H = 1/M = g/lbf = g/[V(M/lbf/V)]$, where $V$ and $g$ are the peak voltage and acceleration, respectively, which occur at the same instant in time
      - $(x lbf/V) = g/[V(1/M)] = Mg/V$; hence load cell sensitivity, in units of V/lbf, is inverse of $(x lbf/V)$

To ensure the response signals, the frequency response functions (FRFs), possess nonzero elements at the end of the time record, an exponential window is applied to the time responses. This prevents any leakage from corrupting the acquired FRFs. The effect of leakage on measured FRFs is an underestimation of the magnitude at the peaks and a distortion of the phase. Such errors corrupt the accuracy of modal parameter estimates and thus compromise modal flexibility. If the system is lightly damped the response may not decay fast enough to be completely observed in the predetermined time record duration.

Utilizing an exponential window will force the signal to be completely observable and will reduce the affect of leakage errors. If the system is heavily damped the response will decay fast enough to make the signal observable. Using an exponential window here will attenuate any noise present after the signal has decayed. This will provide an increased signal-to-noise ratio.
for the output signal. Figure 3-7 shows the affect of applying an exponential window to a heavily damped response signal.

![Figure 3-7: Effect of Exponential Window in Time Domain](image)

After immediate impact, the hammer cannot impart any further excitation onto the system. The data in the force signal following the impulse will consist primarily of noise in the input channel. Due to the short duration of the impulse, the total energy of the noise may be on the same order as the energy of the input force [Fladung 1994]. The input time record is passed through a force and exponential window to attenuate the noise following the impulse. This will improve the signal-to-noise ratio of the input signal (see Figure 3-8).
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\(^9\).

### 3.3 TEST EXECUTION

Figure 3-9 displays an extensive accelerometer layout for a typical bridge 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. Therefore to overcome these effects several impact points are
used.

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 3-10.

---

All sensors measure phenomena and provide output voltage (analog signal). Analog time signals are passed through an amplifier (x1). Signals are passed through analog anti-alias filter (Low Pass Filter) with 12.8 kHz cutoff. Transition band 12.8 kHz-16.384 kHz.* Signals digitized by ADC (Analog to Digital Converter) w/fixed sample rate of 32,768 samples/sec (Hz)* Signals passed through digital filter (digital filter will filter frequency components outside the desired frequency range). The blocksize, N, for each signal is then decimated to ensure that resulting signal (which has frequency components within specified frequency range) is alias-free.* Force/Exponential Window applied to input (impact) signal.* Exponential window applied to all acceleration response signals.* Store averaged results Move to next impact point Repeat for specified number of averages Compute Input Power, Output Power, Cross Power, FRF, Coherence* FFT all signals* If any one signal is not good, reject all data and correct the source of error. If all signals are good, accept data

* Denotes operations performed by HP 3566A Dynamic Signal Analyzer

Figure 3-10: Flowchart for Acquiring FRF Data
3.4 ACQUIRING FRFS

Great care must be exercised when conducting a modal test to ensure the data is of as high quality as possible. Electrical noise (variance) and bias errors (signal processing, calibration, faulty sensor, etc.) are a common problem during a modal test. 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.

For one, 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 of the bridge have dissipated.

The mounting of each sensor and its connection to the data acquisitions (DAQ) system is critical. Any potential for vibration not directly caused by the impact hammer must be eliminated. This situation is generally due to either of two conditions. Improperly mounted accelerometers or a poor connection between a sensor and the cable connecting it to the DAQ can provide unwanted vibration. The sensor is mounted to the deck of the bridge through the use of a base plate mounted in hyrdostone plaster, which forms a bond with the bridge deck. The sensor is secured to the base plate by a setscrew. If the accelerometer is not securely fastened to the base plate, there is potential for a vibratory response that includes the dynamics of the bridge as well as movement of the sensor in relation to the plate it is mounted on. The same unwanted dynamic effect might also exist if the base plate is not firmly bonded to the deck. This will corrupt the frequency response measured, preventing that sensor’s response to be included in post-processing efforts. This may be critical as it common for some modes to be only locally
excited. If this occurs then it could be possible to not detect an important mode of vibration inherent to the structure.

An accelerometer was mounted on the girder directly below its counterpart mounted on the deck of the bridge. By comparing the measured FRFs from these sensors, UCII was able to verify that mounting the sensors directly on the bridge deck measures the vibratory response of the structure, without any additional unwanted dynamics, as long as the accelerometer is properly mounted and secured to the deck. Figure 3-11 shows the FRFs for the two sensors used to provide this confidence study. These responses are in excellent agreement with one another at the peaks as well as at anti-resonant frequencies.

A coherence function is calculated to ensure the measured response was a direct result of the measured input. Accelerometer responses can be quite different on the same bridge based on the point where the measurement is taken. Therefore it is not adequate to deem a response acceptable merely by visual inspection. Although disturbances are often visible in the time history, a high coherence is necessary to ensure the data is not corrupted by noise or some input not related to the impact hammer.
Even with the best of intentions, sometimes the acquired test data is still flawed. Even though the test software visually displays the Input Power Spectrum for the drop hammer as well as the Output Power spectrums, Cross Power spectrums, Coherence function and FRFs for every monitored channel it is possible for a “poor” FRF to be accepted and stored within the modal test data archive. This can mainly be attributed to the significant amount of information that needs to be analyzed in a very short period of time so as to minimize the disruption of traffic access to the bridge. Although these measurements are displayed to the user, there is no automation involved with accepting or rejecting the data. It is the responsibility of the operator to visually inspect each the integrity of each plot for each impact. Therefore, the possibility for human error is introduced at this step of the test process.

After the data has been saved it is possible to remove “poor” FRFs to prevent them from corrupting valid test data. Figure 3-12 shows the Complex Mode Indicator Function (CMIF, discussed further below in Section 3.5) for a data set collected from a UCII modal test and processed in the lab. As seen from the figure, the plot does not provide useful information from 0-12 Hz due to the high frequency oscillation. After further investigation it was determined this corruption of data was from a single sensor having a poor response. The impact test associated with this CMIF plot had six impact locations, resulting in six separate sets of data. The corrupted data was associated with the second impact point and the response of this “bad” FRF is plotted in Figure 3-13.

This sensor does contain a high frequency oscillation coinciding with the high frequency component noticed in the corrupted CMIF plot. This oscillation could be due to a poor connection between the sensor and its cable connecting to the DAQ or perhaps a poor bond between the deck of the bridge and the base plate it sits on. Since the sensor had a bad response for only one of the hit locations its response was most likely due to a poor cable connection. Once the sensor was removed from processing the CMIF was re-plotted resulting in the plot of Figure 3-14. With this poor FRF removed, the CMIF plot was able to indicate the possible location of modes and processing of this test data was able to obtain credible results.
Chapter 3 - Multiple Reference Impact Testing

Figure 3-12: Corrupt CMIF Plot

Figure 3-13: Sensor Response Corrupting CMIF Plot
3.5 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)\textsuperscript{12}. 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 3-12 and Figure 3-14). 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 3-15.
The SV vector (CMIF or singular vector) associated with each peak is a least squares estimate of the eigenvector, or mode shape, for that particular peak. This vector acts as a filter to compute an enhanced Frequency Response Function (eFRF) from the weighted average of the FRF matrix. The eFRF is used to manipulate the FRF to enhance a particular mode of vibration. The left singular vectors, associated with the peaks in the CMIF plot, are used as the filter to accomplish this. This allows modes to be decoupled and represented by a Single-Degree-of-Freedom (SDOF) system [Phillips and Allemang 1998; Phillips et al. 1998]. Once the eFRF matrix has been formed, the poles of the system can be calculated. The FRF matrix is decomposed at each frequency line using SVD methods:

\[
[H(\omega)]_{Nn \times Ni} = \begin{bmatrix}
H_{11} (\omega) & H_{12} (\omega) & \cdots & H_{1N} (\omega) \\
H_{21} (\omega) & H_{22} (\omega) & \cdots & H_{2N} (\omega) \\
\vdots & \vdots & \ddots & \vdots \\
H_{N1} (\omega) & H_{N2} (\omega) & \cdots & H_{NN} (\omega)
\end{bmatrix}
= [U]_{Nn \times Ni} [S]_{Ni \times Ni} [V]^T_{Ni \times Ni}
\]

(3-2)
where:

\[ \omega_i = \text{ith frequency line of the FRF matrix;} \]

\[ N_o = \text{number of measurement points;} \]

\[ N_i = \text{number of inputs;} \]

\[ U = \text{left singular vector matrix;} \]

\[ V^H = \text{Hermitian transpose of right singular vector matrix;} \]

\[ S = \text{singular value matrix (diagonal matrix with Ni real, non-negative elements in descending order).} \]

The significance of decomposing the FRF matrix into three individual matrices can be understood by looking at the FRF model and equate it to the SVD triple matrix product.

\[
\left[ H(\omega_i) \right]_{N_o N_i} = \left[ \psi \right]_{N_o 2N} \begin{bmatrix} 1 \\ \frac{1}{j\omega_i - \lambda_r} \end{bmatrix}_{2N 2N} \left[ L \right]_{2N N_i}^T = \left[ U \right]_{N_o N_i} \left[ S \right]_{N_i N_i} \left[ V \right]_{N_i N_i}^H \tag{3-3}
\]

When comparing the two statements it is straightforward to see how they relate. The singular matrix, \( S \), corresponds to the \([1 / j\omega_i - \lambda_r] \) matrix. As \( \omega_i \) approaches the pole frequency, \( \omega_r \), \([1 / j\omega_i - \lambda_r] \) approaches a maximum value. At \( \omega_i = \omega_r \), the singular value matrix at \( \omega_i \) will be a maximum, which corresponds to the first diagonal element in \( S \). This maximum value indicates the location of a system eigenvalue and is represented by a peak in the CMIF plot. Stating the mathematical FRF definition:

\[
\left[ H(\omega_i) \right]_{N_o N_i} = \sum_{r=1}^{2N} \frac{\left\{ \psi \right\}_r \left\{ \psi \right\}_r^T}{M_{Ar} \left( j\omega_i - \lambda_r \right)} \tag{3-4}
\]

Note that each term of the FRF matrix is represented in terms of a mode shape and pole location. \( M_{Ar} \) is included to properly scale the modal vector coefficients. In the construction of the CMIF all modal vectors are scaled to have unity length, \( \| \psi \|_2 = 1 \). This arbitrary scaling must be accounted for to provide proper calculation of modal flexibility. \( M_{Ar} \) is often referred to as the
modal scale factor (MSF) and is described in Chapter 5 of Hewlett-Packard [McLamore et al. 1971]. The numerators are simply constants and the denominators are a function of frequency. Therefore the numerators can be treated as residues and this is an application of partial fraction expansion. Each term in the FRF matrix can then expressed in terms of poles and residues corresponding to each mode of the system. The $H$ matrix can then be expressed as a summation of these single-degree-of-freedom modes.

$$
\begin{align*}
[H(\omega)] &= \sum_{r=1}^{2N} \frac{[A_r]}{j\omega_i - \lambda_r} \\
&= \sum_{r=1}^{2N} \frac{[A_r]}{j\omega_i - \lambda_r} 
\end{align*}
$$

(3-5)

The left singular values associated with each peak in the CMIF plot are used as a filter to accomplish this (AASHTO 2002). Starting with:

$$
H_{pq}(\omega) = \sum_{r=1}^{2N} \psi_{pr} \frac{Q_r \psi_{qr}}{j\omega - \lambda_r}
$$

(3-6)

where:

$$
Q_r = \frac{1}{M_{Ar}}
$$

The eFRF is defined as:

$$
eFRF_r(\omega) = \frac{Q_r \psi_{qr}}{j\omega - \lambda_r}
$$

(3-7)

$eFRF_r(\omega)$ is the eFRF for mode $r$, and only has to do with the input location and is constant for a given column of the FRF matrix. Obtaining the eFRF from measured data is done in the following manner:

$$
H(\omega) = \sum_{r=1}^{2N} \{\psi\}_r \frac{Q_r \psi_{qr}}{j\omega - \lambda_r}
$$

(3-8)

$$
eFRF_r(\omega) = [\psi]_r^T [M] \{H(\omega)\}
$$

(3-9)
Chapter 3 - Multiple Reference Impact Testing

\[
eFRF_s(\omega) = \{\psi\}_s^T \left[ M \right] \sum_{r=1}^{2N} \{\psi\}_r \frac{Q_r \psi_{qr}}{j\omega - \lambda_r} \tag{3-10}
\]

Due to the principle of modal orthogonality:

\[
\{\psi\}_s^T \left[ M \right] \{\psi\}_s = M_s \quad \{\psi\}_s^T \left[ M \right] \{\psi\}_r = 0 \tag{3-11}
\]

\[
eFRF_s(\omega) = M_s \frac{Q_s \psi_{qs}}{j\omega - \lambda_s} \tag{3-12}
\]

Phillips [Phillips and Alleman 1998; Phillips et al. 1998] provides this argument in detail. The purpose of the eFRF is to yield one degree of freedom for a modal parameter estimation algorithm to estimate the modal frequency and damping. An eFRF is a weighted summation of the measured FRF data, in which the modal vectors obtained from the singular vectors are used as the weighting functions. This enhances the contribution of a single mode while attenuating the effects of the other modes. If other modes are still observable in the eFRF, their contributions can be accounted for with the inclusion of residuals in the estimation algorithm. Figure 3-16 displays a MDOF frequency response function as a superposition of several SDOF response functions.
3.6 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
The first check performed, determining if the system under consideration is time-invariant, is actually performed before estimation of the modal parameters. The data is plotted to ensure it does satisfy the principle of reciprocity. This quality check is subjective since it requires the user to examine two FRFs plotted on one axis to determine their consistency. A plot to examine reciprocity is shown in Figure 3-17. Looking at the figure one can see some discrepancies between the two responses. However the two plots correlate rather well at the peaks, when the signal-to-noise ratio is high, and differ when the two responses are dominated by noise. Since the peaks in an FRF indicate possible eigenvalue locations, the reciprocity plot is satisfactory in the frequency ranges of interest.

![Figure 3-17: Reciprocity (Time-Invariance) Check](image)

Once this has been established, the user is shown the CMIF plot and manually picks the system poles from this screen. This is a tedious task in the case of heavily damped modes. When heavily damped modes exist, they appear as a rather flat “bump” in the CMIF plot, as opposed to lightly damped modes which appear as sharply defined peaks, Figure 3-18. It is the responsibility of the incorporated system checks to determine if these small “bumps” in the CMIF plot correspond to eigenvalues or noise.
The singular vector associated with each selected pole is used as a filter to create the eFRF. This decouples the system into a single-degree-of-freedom representation, which is then used to estimate the system poles, frequency and damping, and modal scaling of the selected modes. The user is asked at the beginning of the process to provide the number of data points to use for curve fitting. Curve-fitting the eFRF data and extrapolating the values from this curve-fit provide the estimates. Therefore the estimates are only as accurate as the synthesized FRF. A curve-fit error is computed and displayed to the user indicating the ability of the algorithm to fit the data. If the error is deemed too high, which can occur from too many data points being included, the process is repeated with fewer points included in the curve-fit. This is done since previous experience has shown that high curve-fit errors generally occur when two closely spaced modes are not decoupled by the eFRF, resulting in two modes being fitted with one polynomial. After all curve-fits have been accepted, the frequency and damping of each pole is displayed to the user. If the damping is high, traditionally over 10% based on previous research, this indicates that the peak in the CMIF plot may not be indicating a system pole but rather just an unimportant bump. The mode is not removed at this time but is marked as questionable and heavily scrutinized by the user during the remaining quality assurance checks.
The next quality check looks at the phase response of the eFRF for each pole and determines if it crosses $90^\circ$ at the location of the selected pole. If it does not cross $90^\circ$ this indicates two possibilities. The first possibility is the eFRF did not completely decouple each mode and yield a single-degree of freedom for that mode. This would allow nearby poles to affect the response of the individual mode under investigation. The other possibility is the pole selected from the CMIF plot is not a system pole but rather an insignificant bump in the plot. If the phase response of the mode in question does not cross $90^\circ$ and is not close in frequency to other modes then it is removed from all processing efforts.

Another quality check performed determines if each mode has been properly decoupled and is truly an independent mode of the system. The Structural Dynamics Research Laboratory (SDRL) has developed a method known as the Modal Assurance Criterion to perform such a task. It is possible for the CMIF plot to have multiple peaks related to one pole and mode shape. MAC determines the correlation between two vectors, in this case the vectors are mode shapes, and assigns a value between 0 and 1 to indicate the level of correlation. From previous efforts it has been established values below 0.9 indicate separate and distinct modal vectors and values greater than 0.9 represent the same mode shape. If a mode fails the MAC check it is removed from processing and is not included in the flexibility calculation. The decision of which mode to remove is based on a collection of factors. Generally when two modes share a high MAC value, the one to eliminate becomes apparent. This may be due to a smaller singular value corresponding to a smaller peak in the CMIF plot, a poor phase response or poor damping.

The last quality check on the data is subjective in nature and is not implemented through the use of code. It is a visual inspection of the mode shape of vibration. This requires an understanding of the sensor layout in regards to the structure being tested. If the mode shape displayed is physically unrealizable then it is obvious the mode is not a global property of the system and should be removed from processing. Typically this is a last resort quality check. If a mode has failed one of the previous steps it generally has a mode shape that is irregular. Figure 3-19 shows two mode shapes.
The first mode shape is considered “good” and the second is considered “poor”. The second mode did not fail any of the preceding quality checks and was not removed from processing efforts until its mode shape was displayed. It is often difficult to ascertain which factor is affecting the response, therefore a decision often times cannot be made until the mode shape is displayed. Deciding which modes to include and which to eliminate often requires several steps. No one check is generally considered sufficient to make a decision, but it is the combined results of these checks that allow the user to have confidence in the results produced.

3.7 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:

\[
\begin{bmatrix}
\{\psi\}_r \{\psi\}_r^T \\
M_{A_r} \left( j\omega_i - \lambda_r \right)
\end{bmatrix}
\]

Any individual FRF, \(H_{pq}(\omega_i)\), may be defined from this expression as:
\[
H_{pq}(\omega_i) = \sum_{r=1}^{N} \left[ \frac{\{\psi\}_{pr} \{\psi\}_{qr}}{M_{Ar}(j\omega_i - \lambda_r)} + \frac{\{\psi\}_{pr}^* \{\psi\}_{qr}^*}{M_{Ar}^*(j\omega_i - \lambda_r^*)} \right]
\]

(3-14)

where:

\(\psi_{pr}\) is the \(p^{th}\) term of \(\{\psi\}_r\) (corresponding to point \(p\) of the \(r^{th}\) eigenvector)

\(\psi_{qr}\) \(qr\) is the \(q^{th}\) term of \(\{\psi\}_r\) (corresponding to point \(q\) of the \(r^{th}\) eigenvector)

Evaluating this equation at \(d_c\) yields:

\[
H_{pq}(\omega_i = 0) = \sum_{r=1}^{N} \left[ \frac{\{\psi\}_{pr} \{\psi\}_{qr}}{M_{Ar}(-\lambda_r)} + \frac{\{\psi\}_{pr}^* \{\psi\}_{qr}^*}{M_{Ar}^*(-\lambda_r^*)} \right] = f_{pq} \tag{3-15}
\]

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, \(\{\delta\}\), are then obtained by multiplying each resulting flexibility matrix with a load vector, \(\{P\}\), comprised of unit loads at each measurement point, \(\{\delta\}=[f]\{P\}\).
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. \( f_{\text{mode 1}} \), \( f_{\text{mode 1+mode 2}} \), \( f_{\text{mode 1+mode 2+mode 3}} \), 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 3-20 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.
3.8 SOFTWARE ENHANCEMENT

The University of Cincinnati Infrastructure Institute programmed the software package utilized to process modal data in MATLAB and titled it MODALCIS5. The software allowed a user to take the measured FRFs and process them until modal flexibility was achieved. A flowchart of this process was presented in Figure 3-15. Preliminary processing of the test data to identify modal parameters (e.g., frequencies, shapes, etc.), verify assumptions (e.g., reciprocity, stationarity), and estimate the flexibility matrix for the test sensor grid used to take several full days of work. Although this analysis would ultimately provide the user with a final result, processing the test data could easily take a few weeks, as the engineer would need to meticulously track down the various subjective criteria in the analysis. There were no quality checks built into the MATLAB software that could provide quantitative, objective feedback to the user regarding the validity of the results. If the final results were questioned the process would start over, and several more days were required to provide results. Automating the selection and estimation of modal parameters as well as providing confidence to the results through use of quality control measures is the contribution of this thesis. The enhancement of the MODALCIS software package used by the University of Cincinnati Infrastructure Institute is presented here.

Many MATLAB functions and subroutines were written to process modal test data. However these subroutines were built as stand alone functions requiring the user to manually run each routine at the appropriate time. The user was also required to sort through several megabytes of data stored in several files to obtain data needed in the calculation of modal parameters and flexibility. This tedious work would allow for the opportunity of mistakes to enter the analysis process, which would not be known until the final result, modal flexibility was established. Due to the length of time and amount of expertise required in processing each test a backlog of tests needing evaluation would accrue.

Automating the procedure would allow processing in a timely manner based upon objective quality assurance measures, not subjective decision making. This does not mean subjectivity has been removed altogether. Once data has been determined to fail a system check, the final result to accept or reject data based on this information rests with the user. A degree of familiarity with the software and expertise of analyzing modal data is required to provide results.
with confidence. This work hopes to reduce the amount of expertise and familiarity needed to a minimum.

The quality control checks that have been implemented in the newest software revision were discussed in Section 3.6 above. Figure 3-21: Updated MODALCIS Analysis Software shows the flowchart of MODALCIS after a series of system checks were implemented.
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.

Linearized condition indices are based on the assumption that a constructed facility may be characterized as incrementally linear at the serviceability limit states. While no soil-foundation-structure system can be strictly linear, the justification in using linearized indices is that most constructed facilities behave linearly in the global sense even when many local nonlinearities, such as due to localized damage in a shake-down state, may exist. In fact, this argument has been the basis of linear analysis for ultimate strength design [ACI 318-95, 1995]. For the general infrastructure management problem, linearized indices offer the advantages that they can be physically conceptualized, can be measured or extracted from measurements during controlled tests at any time, and they can be more easily correlated to facility performance at the serviceability limit states.

In general, a system-or-structural-identification based condition assessment approach to condition assessment is highly recommended. In the system-identification approach, we would
need to generate an a-priori analytical model, design and conduct experiments, conduct
parameter identification to calibrate the analytical model, validate its completeness and reality-
check by evaluating the physical correspondence of the identified parameters. The resulting
field-calibrated analytical model serves as a reality-check on the experiment itself, and for linear
(and as a starting point for non-linear) sensitivity analyses which are required for reliability
evaluation.

For evaluating the sufficiency or relative advantages of different condition or damage
indices, we note the following:

a) Acceptable ranges and stationarity of the indices;

b) Transformation procedures for relating the measured indices to facility performance at design limit states. This should be conducted in conjunction with identifying any defects, deterioration mechanisms or damage; and,

c) Projecting current condition and performance to the future given the expected maintenance, loading environment and service life.

In addition to or in conjunction with the above criteria, we desire condition indices to be:

a) Sensitive to the effects of common deterioration and damage mechanisms that affect the facility;

b) Directly measurable by practical experimentation, and not requiring extensive post-processing. Some indices may require extensive error-prone numerical operations;

c) Robust, i.e. insensitive to experimental errors and uncertainties; and,

d) Conceptual, i.e. directly corresponding to a clear aspect of structural behavior such as deflection, stress, moment, stiffness or flexibility. While some indices, such as the effective flexural rigidity of a member, permit using engineering intuition for damage detection, others with no physical correspondence have been proposed to indicate damage based on an arbitrary scale.
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]. Turer and Aktan (1999) showed that the decomposed unit influence line and its utility to estimate the future effects of proof or “superloads” can provide an accurate and conceptual health index for a structure. 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

While these concepts have been known for many decades, the experimental and analytical tools for a meaningful and practical application to civil structures such as bridges or buildings has only recently emerged. Increased computational capability in desktop and site computing hardware has resulted in significant progress in algorithm development and testing [Ghanem and Shinozuka 1995; Shinozuka and Ghanem 1995; and Aktan et al. 1998b]. Instrumentation and data-acquisition hardware which has been custom-designed for the operating range of civil structures and which can withstand the environmental rigors of the infrastructure has become commercially available [Massicotte and Picard 1994; Alampalli and Fu 1994; Hunt et al. 1994; Levi et al. 1996; Aktan et al. 1998a; Lenett et al. 2000]. Truck and other load testing has been refined to a quick and efficient method of identifying structural condition and capacity rating of highway bridges [Lichtenstein 1998; Schulz et al. 1995; Levi 1997; Levi et al. 1997; Turer and Aktan 1999; Lenett et al. 1999; Lennet et al. 2000b; Hunt 2000].

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,
- Impact factor,
- 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,
- Environmental effects such as thermal stresses.

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.

This research provides a novel approach to condition assessment by determining the below indices for the instrumented sections of the bridge from the measured data without the use of a finite element model. Further details of this condition assessment have been documented and can be provided [Hunt 2000; Helmicki and Hunt 2004; Hunt and Helmicki 2005].

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.

![BDI Full-Bridge Strain Transducer](image)

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.
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. Displacement is perhaps the most fundamental measurable structural response and there are numerous available sensors to measure it. The slide wire potentiometer, or slide wire “pot”, is a displacement transducer which utilizes the process of cable extension to monitor displacement. The slide wire potentiometer used for this project was the Celesco PT101 SWP. The transducer is mounted in a fixed position and a thin steel cable is attached to the specimen. A constant torque spring keeps the cable in tension, and as the specimen moves relative to the fixed position, the cable rotates a precision potentiometer that produces a proportional linear voltage output. Because it is somewhat susceptible to adverse environmental conditions, this transducer was only used intermittently in the field.

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.

![Figure 4-4: Optim MEGADAC with Laptop and Terminal Block](image)

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.
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. Not all of the bridges were tested by truckload as the emphasis was upon the modal test.

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 were needed to install the sensors. ODOT provided 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 either limited the traffic control between morning and evening rush hour (i.e., from 9AM to 4PM) or the test was conducted at night (e.g., from 9PM to 4AM).

Sensor cables were tied off to the crossframes, routed over the side of the bridge, and along the sidewalk and/or railing to the data acquisition system, an Optim Electronics MEGADAC. Tape switches were laid in the traffic lanes to monitor the position and speed of the moving truck. On a typical bridge, this entire process to install a forty sensor plan took about
four hours, the truckload experiment took about one hour, and then it took about two hours to remove everything. Hence, the entire process fit within seven hours or less in order to stay within one shift for the ODOT crew as well as to work between rush hours for the public.

![Figure 4-7: Cable and Data System Installation](image)

ODOT provided a loaded and weighed dump/salt truck for the controlled experiments, which was either a single axle of about 34 kips (151 kN) or a tandem axle of about 57 kips (253 kN). The truck was weighed at a local scale for its gross weight, but also each axle was 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 were placed on the scale. The weight was assumed to be split equally between the back axles. The axle spacings were found manually by measuring tape. The same truck and driver were used during testing in order to minimize the variation in the test. The driver was 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 was tested at least three times per test. Of course, it was not possible to perfectly achieve these goals, but experience has shown that the typical variation in the experiment due to the driver, the vehicle, its actual weights, its path, its speed, etc. was well within reason and the experiment was not especially sensitive to these parameters.
It should be noted that up to 10% error was 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 TRUCKLOAD SPEED

It is desirable for the loading mode frequency \( f_y \) to be separated from the dynamic frequency of the bridge \( f_n \) in order to clearly identify the static influence line for the beam. If the vehicle moves slow (here, \( V \) is the truck speed), the dynamic modes are easily removed by a low-pass filter at \( f_c = 10f_y \) in the frequency domain. One important observation immediately occurs from a conceptualization of a dynamic beam model for a bridge; the magnitude of the dynamic modes for the structure are insignificant (i.e., unity) at frequencies below one-third of its natural frequency. The natural frequency for a highway bridge has been experimentally found to be approximated by \( f_n = 328 / L \), where \( L \) is the shortest span length in feet, although this typically underestimates the parameter [Cantieni 1984]. Hence, the significant loading modes of the influence line would be separable and unaffected from the significant dynamic modes in the frequency domain if the loading bandwidth \( f_{cl} = 10f_y = 5V/L \) was less than the dynamic bandwidth \( f_{cd} = f_n/3 \). Hence, it would be desirable for vehicle speed to be less than 0.067 \( L f_n \) or 15 mph (24 km/h), in general, based upon Cantieni’s results.

For small (and especially short single span) bridges, we lower the truck speed even further. The vehicular modes interact and become coupled with the bridge response; so, again,
we want the truck to move slow enough to minimize this effect and separate the static and dynamic responses in the frequency domain. Let us assume that if the truck’s natural frequency can occur between 2 to 6 Hz, then \( f_c = 10f_y = 5V/L < 2 \) Hz; hence, \( V < 0.4L \) \( \text{ft/sec} \) or 0.27\( L \) mph, where \( L \) is the shortest span length for the bridge in feet (or 0.13\( L \) km/h, where \( L \) is in meters). The speed of the test vehicle is determined accordingly. Of course, this must be checked for each truckload test of each bridge on a case-by-case basis; as the truckload test progresses, the operator must review the measured data to determine if the truck must go slower to remove any undesired dynamic disturbance to the expected bridge response. If the truck must go slow, it has been observed that most trucks have a lower limit of about 5 mph on their speed before it becomes difficult for the driver to maintain a constant smooth speed across the entire length of the structure. At such slow speeds, the driver must use the tachometer instead of the speedometer to maintain a constant speed.

### 4.6 INITIAL TRUCKLOAD TEST OBSERVATIONS

Let us examine the realities of the field truckload test for typical steel-stringer bridges. This objective is accomplished by processing a complete set of strain response data for three-span, medium-length Cross-County Highway bridges over Reading Road (HAM-42-0992) and Hamilton Avenue (HAM-126-0881), during several crawl and high speed experiments with various truck vehicles. The steel superstructure for each bridge was instrumented with strain gage instrumentation on the beams. The concrete decking for HAM-126-0881 was also instrumented with embedded foil strain gages during its construction. The results of these bridge tests will be used to exemplify the results of the stringer bridges tested during this project; however, specific details on each bridge test are provided in the appendices.

The location of maximum stress observed during a set of experiments is certainly a parameter of concern. The response of the instrumented member (e.g. steel flange) can itself be checked against its material capacity for load. If this is tracked over some period of time, this could indicate areas of possible deterioration for the bi-annual inspection. An influence line would provide greater information by normalizing the stress to a unit load and considering the entire length of the load path. However, little assessment of the structure’s condition can be made based upon one sensor reading alone.
For HAM-126-0881, a partially composite design with an abnormally small span ratio, the maximum stress occurs in the middle span on the middle girder at the bottom flange during the diagnostic truckload test (Figure 4-9). For a structure with a more balanced design, we would find the maximum stresses to be more equalized between the various spans and the critical span would be less evident. The corresponding top flange is near zero, indicating a fully composite section as designed. Although only the middle span was designed composite, note the slight but unintended composite action in the endspans and at the piers. This bridge has integral abutments and a significant end restraint may be seen in the measured responses. The response is nearly identical when the truck crosses the bridge in the other (i.e., south) lane due to the symmetry of the bridge. This provides a reality check on the measured responses in situ.

The top flange responses indicate some nonlinear behavior and do not peak at the same location as the bottom flange lines. This may be due to slip-stick performance of the composite interface with the concrete decking; however, it occurs at every instrumented section including the fully composite middle span. Another hypothesis is that some very local transfer of the load occurs at each crossframe or at other secondary members of the bridge structure. A final observation is that the piers indicate significant axial force (almost equal stresses in both top and bottom flange gages) when the point load is directly above them. These unexpected but local responses at the top flange and piers have been observed on many other steel-stringer bridges and will be considered further in future research.

4.7 NEUTRAL AXIS LOCATION AND COMPOSITE ACTION

If there are no other forces acting upon the beam, then the neutral axis coincides with the geometric centroid for the beam. The centroid for the typical steel I-beam used in steel-stringer design lies at the vertical center of the girder. If additional cover plates were added to the top and/or bottom flanges for improved load capacity, the centroid would be slightly off-center accordingly. If composite action, whether designed or unintended, occurs between the steel beam and the concrete decking, then the centroid is raised substantially above the beam center depending upon the bond integrity. If full composite action is designed by welding shear studs to the top of the beam and encasing the top flange within the concrete decking, then the centroid of the composite section is raised near the top flange of the beam. The steel beam is almost
entirely in tension and the concrete decking is completely in compression for a fully composite section in bending under a vertical load.

Figure 4-9: Filtered Truckload Responses for Critical Cross-Sections of HAM-126-0881

The location of the neutral axis for each instrumented section can be determined from the top and bottom flange strain measurements during a diagnostic load test (Figure 4-10). The composite action for the instrumented section can itself be checked against the designed performance. Note that the neutral axis for the middle span remains above fully composite location for most of the truckload test and remains fairly consistent except near supports where the measured values become very small. However, the east span and pier demonstrate varying levels of composite action depending upon the truck position, which indicates slipping at the interface/bond between the concrete deck and steel girder. For example, they demonstrate partial or even full composite action when the load is directly upon them, but noncomposite behavior as
the truck approaches the first/east pier. If this is tracked over some period of time, this could indicate areas of possible deterioration for the bi-annual inspection. An influence line would provide greater information by normalizing the stress to a unit load and considering the change in the neutral axis over the entire length of the load path.

![Neutral Axis Location during Truckload of HAM-126-0881](image)

**Figure 4-10: Neutral Axis Location during Truckload of HAM-126-0881**

### 4.8 POST-PROCESSING OF TRUCKLOAD TESTS FOR DERIVATION OFUILS

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 4-11) [Turer 1997; Turer and Aktan 1999].

The influence line is determined from the measured truckload response with a custom program that performs the following routines:

1) Given tape switch readings at both abutments, determine an average truck speed across the bridge

2) Balance all sensor readings for a zero value when the first truck axle reaches the first tape switch

3) Determine the frequency domain representation for each sensor reading using the FFT function

4) Determine the influence line in the frequency domain by dividing out the phased axle contributions

5) Bandlimit both the sensor reading and influence line in the frequency domain
6) Determine the time domain representation of the bandlimited truck response and influence line

7) Plot and save the results

The assumptions and limitations for the beam flexure equation and the moment-curvature relationship still hold [Beer and Johnston 1992; Rossow 1996]:

1) Small strains and deformations: so that geometry changes are negligible and slopes are very small.

2) Bernoulli Theorem of Bending: plane sections before bending will remain plane after bending.

3) Homogeneity: beam material is consistent throughout the cross section.

4) Hooke’s Law: beam material has a linear stress-strain relationship as defined by the modulus of elasticity, E, and it is the same in both tension and compression.

5) Vertical symmetry: to ensure that the stress distribution is symmetric, that the y axis is the principal axis, the shear center will lie on the y axis, and bending without twisting will occur.

6) Straight beam design: although it can be extended to curved beams if the ratio of depth to radius of undeformed curvature is small.

7) Prismatic beam design: beam is long, slender, and has a constant cross section, although the latter can be relaxed if the change in cross section is gradual and continuous.

8) Stability under bending: cross section is to maintain integrity and shape under bending action (which may not occur for thin-walled elements).

9) One dimension: all deformation is to occur in the x-y plane (i.e., no lateral or torsional behavior).
10) No axial load: only bending is considered.

11) Superposition: the deformation for the simultaneous application of two or more loads is equal to the sum of their respective deformations (Figure 4-11)

Several truckload tests were conducted for HAM-42-0992 at various speeds, temperatures, driving paths, and truckloads. Static tests were also conducted at various bridge positions. Figure 4-12 and Figure 4-13 indicate that a consistent influence line for girder strain was obtained regardless of truckload. This result was also found to hold true for the other instrumented locations at various times and temperatures during the test days. Note that the influence line peaks in the time domain at exactly the position of the gage, as predicted by the theory of an ideal beam.

Truck parameters such as axle spacing and weights were determined at regularly calibrated stations by ODOT. The truck driver was asked to maintain a constant speed of 10 mph (16 km/h) and path across the bridge. Of course, this is not physically possible but the variance in these parameters was not found to dramatically affect the test results. For example, a 10% variance in the truck speed can cause a 3% variance in the peak strain of the influence line [Hunt 2000]. Truckload experiments were monitored for consistent truck speed and occasionally some were disqualified due to large variation; however, results were usually very consistent and repeatable.
Chapter 4 - Truckload Testing

As the bridge includes significant unintended composite action between the deck and girder systems, the top flange strain response is approximately zero. Hence, the bottom flange strain response, scaled by the sectional and elastic moduli, represents the sectional moment at the instrumented location. Further, due to this and other realities (e.g., rotational restraint at the supports), the strain response is not perfectly triangular as predicted by ideal beam theory.
The crawl-speed results are removed from any vehicular or bridge modes in the frequency domain for this medium-length bridge. A lowpass filter with a cut-off frequency of 1.5 Hz will still capture the significant spectral components of the influence line \( f_c > 10f_y = 5V/L = 1.85 \text{ Hz} \), while removing any dynamic effects from the structure \( f_c < f/n/3 = 4.5/3 = 1.5 \text{ Hz} \) or its interaction with the vehicle \( f_c < 2 \text{ Hz} \). Note that, for this structure, we chose to define \( f_c \) by the latter issues.

The influence line can now be used to virtually simulate any truck with a given set of axle spacings and weights and crossing the bridge at crawl speed. The simulated truck response from the influence line is found to be essentially equivalent with the actual crawl-speed and static truck responses (Figure 4-14).

![Figure 4-14: Comparison of Static, Crawl-Speed, and Simulated Truck Responses](image)

### 4.9 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 \( \text{DF} = S/11 \), where \( S \) is the beam spacing in feet or \( \text{DF} = S/3.355 \), 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.

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.

$$DF_{ij} = \frac{M_{ij}}{\sum_{k=1}^{NG} M_{kj}} = \frac{E_S S_y e_{ij}}{\sum_{k=1}^{NG} \sum_{j=1}^{NL} E_S S_{kj} e_{kj}} \quad (4-1)$$

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

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

where: $DF = \text{Max}(DF_{i})$= the max value of the fraction of loaded lanes, $i =$ girder number, $NG =$ total number of girders, $j =$ the number of the loaded lane, $NL =$ total number of lanes, and the other variable follow from the standard nomenclature used by AASHTO.
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).

### 4.10 IMPACT FACTOR

The AASHTO Design Specifications define the impact factor ($IM = 50/(L+125)$, where $L$ is the span length in feet) as the fractional increase in the maximum static response for the rating load due to dynamic effects. Dynamic effects for highway bridges have been considered in terms of its incremental change to the static stress and force for many decades [Fuller et al. 1931]. There are many other mechanisms in addition to span length which contribute to the impact factor, including the dynamic properties (i.e., mass, stiffness, and damping) of the bridge.
and vehicle, the roughness of the road surface, the slope and orientation of the highway, the truck speed, and the presence of other traffic. Hence, this simplified equation for dynamic increment is generally inaccurate [Moses 1979, Ghosn et al. 1986; Bahkt et al. 1989; Goble et al. 1992; Nowak et al. 1994; Deatherage et al. 1995; Kim and Nowak 1998; Sartor et al. 1999] which further compounds the error in the rating process.

There is very little consensus as to the proper calculation of impact factor from measured truckload responses. Bahkt accounts no fewer than eight different methods that researches have utilized in this endeavor [Bahkt and Pinjarkar 1989]. The design truckloads should actually be employed due to the variables noted above; however, it is widely recognized that the code specification (i.e., HS20 truckload) is a fictitious vehicle that is used to mimic the myriad of trucks travelling the highways today. Linear superposition is not possible in considering dynamic increments; hence, it is not possible to readily simulate the dynamic response of the design load from another vehicle’s response. The rating load considers the worst-case scenario of side-by-side trucks in each traffic lane for the bridge; however, this can be especially cumbersome if not dangerous to simulate on the highway system. There is no specification regarding vehicle speed for the impact factor; in general, the posted speed is assumed but larger dynamic increments are achievable at higher speeds. A statistical approach seems more appropriate in characterizing the traffic and its dynamic effect upon the bridge [Nowak et al. 1994].

It has been observed by several researchers that the dynamic increment can be significantly larger if the load is distant from the instrumented section. This can occur due to the ratio of very small signals. Clearly, this has very little merit in defining an impact factor to scale the estimated peak moment for the design load. Cantieni suggested a “zone of direct influence” in order to determine the relevance of a measured impact factor [Cantieni 1984]. In short, the impact factor should only be calculated for a section when the truckload is located practically overhead (i.e., within the beam spacing).

In Figure 4-15, the impact factor for an instrumented section can be determined by considering the ratio of the maxima for the measured and filtered data used to process the influence lines. The AASHTO Manual for Condition Evaluation suggests that field measured
values of the impact factor can be used in lieu of the design specification; however, the code specification for impact factor is used to scale up the liveload moment for rating purposes in this methodology (discussed in Chapter 5).

4.11 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.

We will now examine the utilization of the identified influence lines for condition assessment of steel-stringer bridges.
Chapter 5 Estimation of Structural Capacity

5.1 RATING ALGORITHMS AND METHODS BASED ON TRUCKLOAD TESTS

The AASHTO Manual for Condition Evaluation of Bridges (AASHTO-CEM) [AASHTO 2000] 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.”

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 \( K_a = \frac{\varepsilon_c}{\varepsilon_m} \), where \( \varepsilon_c \) is the maximum strain/stress during the load test and \( \varepsilon_m \) 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 assumptions for the beam flexure equation and the moment-curvature relationship still apply (see Section 4.8). These are the very same assumptions used in the design of the structure. 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 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 noncomposite case (Method 4) in all figures and tables. The deadloads for the other methods are calculated from these.

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.
Chapter 5 - Estimation of Structural Capacity

5.3 METHODS OF SECTION ANALYSIS

To begin the presentation of the various analytical methods, let us first examine the definition of the neutral axis location and its proximity to the geometric centroid for the instrumented section. The location of the neutral axis for each instrumented section can be determined from the top and bottom flange strain measurements during a diagnostic load test.

\[ \phi = \frac{\varepsilon_b - \varepsilon_t}{y_{tg} - y_{bg}} \]  
\[ y_{\text{max}} = \frac{\varepsilon_b}{\phi} + y_{bg} \]

where:
- \( y_{tg} \) is the vertical position of the top gage,
- \( y_{bg} \) is the vertical position of the bottom gage,
- \( \varepsilon_t \) the measured strain from the top gage, and
- \( \varepsilon_b \) is the measured strain from the bottom gage.

As discussed in Section 4.7, 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.

Assessment of the sectional capacity will now reach a theoretical crossroads after determination of the neutral axis location. First, the implications of unintended composite action must be considered in determining sectional inertia, moduli, moment, and capacity rating. Steps may be taken later in the analysis to discount its advantages to the load rating (e.g., capacity can be based upon the steel beam alone, estimated stresses for the rating loads can be increased, etc.); however, if the measured data indicates that the structure exhibited some level of composite action, then its sectional indices must be calculated accordingly. Second, the assumption of no axial force for an ideal beam must be reviewed. While strains due to axial forces can be induced
Chapter 5 - Estimation of Structural Capacity

(e.g., due to rotational restraint at the supports), it is assumed that their magnitude will be insignificant as compared to those due to flexure.

A fully composite section (i.e., designed as per 10.38 and 10.50 of AASHTO-STD) exhibits a linear and continuous strain profile through the entire section:

![Figure 5-1: Ideal Behavior for a Composite and Noncomposite Section of Example Span](image)

It incorporates an effective width, $b_{eff}$, of the concrete decking in its inertia and capacity for load, as defined by 10.38.3 of AASHTO-STD as the smaller of:

1) One-fourth of the span length ($L$),

2) The distance center to center or spacing of the girders ($S_g$), or

3) Twelve times the thickness of the deck. (12 $t_d$)

The deck thickness generally dominates the specification. In accounting for this composite system, the cross-sectional area of the decking is reduced to an equivalent area of steel by the strength ratio of their elastic moduli ($n_{eff} = E_s / E_c$, where $E_s = 29,000$ ksi (200,000 MPa) and $E_c = 57\sqrt{F_c'} \approx 3,600$ ksi (24,800 MPa), although $E_c$ may vary considerably between 2,500 ksi (17,200 MPa) and 4,500 ksi (31,000 MPa) as per AASHTO-STD 10.38.1.3). Thus, one may now calculate the geometrical moment of inertia, $I_t$, and centroid location, $y_t$, of the composite section:
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\[ y_s = \frac{d}{2}, \quad y_c = \frac{t_d}{2} \]

\[ A_c = \frac{b_{eff} \cdot t_d}{n_{eff}} \quad A_i = A_j + A_c \]

\[ I_c = \frac{b_{eff} \cdot t_d^3}{12 \cdot n_{eff}} \]

\[ y_i = \frac{A_j y_j + A_c y_c}{A_j + A_c} \]

\[ I_i = I_j + I_c + A_i (y_j - y_i)^2 + A_c (y_c - y_i)^2 \] (5-3)

If the measured location of the neutral axis \( y_{\text{max}} \) does not coincide with the determined centroid location \( y_i \) for a fully composite section, one has at least two methods that will account for this discrepancy and yet still allow for a linear and continuous strain profile for the section.

5.3.1 Method 1: Fully Composite with Nonzero Axial Force

The assumption of no axial force is removed. An axial force \((P_i)\) is assumed to be acting upon the section such that the centroid \((y_i)\) 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:

![Figure 5-2: First Method of Estimating the Section Moment for Example Span](image)

This method may be especially appropriate for those sections that are actually designed to act composite. The section modulus \((S_i)\) and bending moment \((M_i)\) are then calculated:
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\[ y_1 = y_t, \quad I_1 = I_t, \quad S_1 = \frac{I_t}{y_t} \]

\[ K_1 = \frac{y_t - y_{bg}}{y_t - y_{bg}} \]

\[ P_i = A_i \cdot E_i \cdot \epsilon_i - \epsilon_{yS} K_1 \]

\[ M_i = \frac{(\epsilon_{yS} E_i - P_i / A_i) I_t}{(y_t - y_{bg})} \]  \hspace{1cm} (5-4)

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:

![Diagram](image)

Figure 5-3: Second Method of Estimating the Section Moment for Example Span

The sectional inertia \( (I_2) \), modulus \( (S_2) \) and moment \( (M_2) \) are then recalculated:

\[ y_2 = y_{max}, \quad b_{eff2} = \frac{n_{eff} A_i (y_s - y_2)}{t_d (y_2 - y_c)} \]

\[ A_{c2} = \frac{b_{eff2} \cdot t_d}{n_{eff}} \]

\[ I_{c2} = \frac{b_{eff2} \cdot t_d^3}{12 \cdot n_{eff}} \]  \hspace{1cm} (5-5)

\[ I_2 = I_s + I_{c2} + A_s (y_s - y_2)^2 + A_{c2} (y_c - y_2)^2 \]

\[ S_2 = \frac{I_2}{y_2}, \quad P_2 = 0, \quad M_2 = \frac{\epsilon_{yS} E_s I_2}{(y_2 - y_{bg})} \]
Other researchers have suggested modifying other parameters (e.g., deck thickness, strength ratio) to account for the discrepancy between centroid and neutral axis [Chajes et al. 1997; Elhelbawey et al. 1999]. This method may be appropriate for those sections designed as fully composite if the estimated axial force $P_f$ is nonzero. The larger estimate for the moment would of course be the more conservative choice. This method may be especially appropriate for those sections exhibiting partial composite action (and not designed as fully composite per AASHTO-STD) because the estimated moment acting upon such sections is smaller than that determined by Method 1.

### 5.3.3 Method 3: Partially Composite with No Axial Force

Another method is also introduced that will account for the discrepancy between the centroid of a fully composite section and the measured location of the neutral axis. 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:

![Figure 5-4: Third Method of Estimating the Section Moment for Example Span](image)

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.
Note that the inertia and centroid cannot be defined for a noncontinuous section. The section modulus \((S_{3})\) is therefore defined by the ratio of estimated moment and the stress at the bottom face of the steel beam. 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.

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 \((RCA = P_{\text{Partial}} / P_{\text{Full}})\).

\[
K_s = \frac{y_s - y_{bg}}{y_s - y_{bg}}, \quad P_s = A_s E_s \frac{\varepsilon_c - \varepsilon_s K_s}{1 - K_s}
\]
\[
M_s = \frac{(\varepsilon_c E_s - P_s / A_s) I_c}{(y_s - y_{bg})}, \quad P_d = -P_s
\]
\[
\varepsilon_{id} = \frac{P_d}{A_s E_s} - \frac{t_d \phi}{2}, \quad \varepsilon_{bd} = t_d \phi + \varepsilon_{id}
\]
\[
M_d = \frac{(\varepsilon_{bd} E_s - P_d / A_s) I_c}{(y_c - d)}, \quad P_3 = 0
\]
\[
M_3 = M_s + M_d - P_s y_s - P_d y_c
\]

The section modulus is defined by the ratio of estimated moment and the stress at the bottom face of the steel beam \((\varepsilon_{bs})\).

\[
\varepsilon_{bs} = y_{max} \phi, \quad S_3 = \frac{M_3}{\varepsilon_{bs}} \quad (5-7)
\]

It will be shown below that the estimated moment (and, therefore, the section modulus) is very close to that derived by Method 2.

The remaining composite action \((RCA)\) is defined as the ratio of the axial force in the steel beam as measured relative to that predicted by a fully composite section.
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\[ \varepsilon_{bgc} = (y - y_{bg}) \phi, \quad \varepsilon_{sc} = (y - y_{sc}) \phi \]

\[ P_{sc} = A_s \cdot E_s \left( \frac{\varepsilon_{sc} - \varepsilon_{bgc} K_s}{1 - K_s} \right) \quad RCA = \frac{P_s}{P_{sc}} \cdot 100\% \] (5-8)

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:

![Figure 5-5: Fourth Method of Estimating the Section Moment for Example Span](image)

This generally results in an underestimate of the liveload moment \( M_4 \) and an overestimate of the rating.

\[ y_4 = y_s, \quad I_4 = I_s \]

\[ S_4 = \frac{I_4}{y_4}, \quad P_4 = P_s \]

\[ M_4 = M_s \] (5-9)

5.4 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.

5.4.1 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 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).

5.4.2 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 and upon the transportation department as a whole.

### 5.4.3 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 of performance is meant to represent the “absolute maximum permissible load level to which the structure may be subjected.”

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,
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\( D \), 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.

\[
RF = \frac{C - LF_1 \cdot D}{LF_2 \cdot L \cdot (1 + IM)}
\]  

(5-10)

For the evaluation of rating factor for steel-stringer bridges, the following will be utilized by this research:

For the Allowable Stress Method: \( LF_1 = LF_2 = 1 \)

Inventory Rating: \( C = 0.55 \cdot F_y \) (stress), \( C = 0.55 \cdot F_y \cdot S_{LL} \) (moment, MASinv)

Operating Rating: \( C = 0.75 \cdot F_y \) (stress), \( C = 0.75 \cdot F_y \cdot S_{LL} \) (moment, MASopr)

For the Load Factor Method: \( C = \) 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: \( C = 0.95 \cdot F_y \) (composite), \( C = 0.8 \cdot F_y \) (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.

5.5 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. For compactness of a fully composite design, the following must be satisfied.

\[
D_P = \frac{(A_r f_y - A_r N_r f_{yr})}{0.85 \cdot b_{eff} f_c} < 5D' = 5 \beta \frac{d + t_d + t_h}{7.5}
\]  

(5-11)

where:

- \(A_r\) = area of rebar,
- \(N_r\) = #rebars longitudinal in decking for the section,
- \(f_{yr}\) = yield stress for rebar,
- \(t_h\) = haunch thickness,
- \(D'\) = depth of neutral axis with max strain at 0.003, and
- \(\beta\) = 0.9 for \(F_y = 36 \text{ ksi}\)
- \(\beta\) = 0.7 for \(F_y = 50 \text{ ksi}\) and 70 ksi

If the section is determined as compact, then the ultimate moment is defined as that moment which causes the section to completely shear (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 moment depends upon the actual depth of the stress block \(D_P\).

If \(D_P < D'\), then \(M_{ult,1} = M_p = A_s F_y (d/2 + t_d + t_h - D_P)\).

If \(D_P > 5D'\), then \(M_{ult,1} = M_y = S_1 F_y\). Otherwise,

\[
M_{ult,1} = \frac{1}{4D'} \left[ D' \left( 5M_p - 0.85M_y \right) + D_p \left( 0.85M_y - M_p \right) \right]
\]  

(5-12)

This formulation assumes that the effective deck strength is greater than the steel girder strength such that the neutral axis is located within the decking \((D_P < t_d)\); otherwise, this simple equation does not hold and a proper evaluation of all forces and moment arms about the neutral axis must be conducted. 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 fully composite section with modified effective width $b_{eff2}$ for the decking (i.e., Method 2) is also defined by the compactness of the section. However, the effective width and the sectional modulus $S_2$ is varied depending upon the level of composite action in the section; hence, the ultimate moment also varies accordingly between the two extremes given above.

The ultimate moment for a partially composite section (i.e., Method 3) is also defined by the compactness of the section. The check for compactness will yield the same results as Method 1 because all of the parameters remain unchanged by Method 3. If the section is determined to be noncompact, then the ultimate moment will vary based upon the section modulus $S_3$ that is linearly adjusted based upon the level of composite action for the section. If the section is determined to be compact, then this same linear variation is enforced by adjusting the ultimate moment between the two extremes given by Method 1 with the identified RCA for the section. $M_{ult,3} = RCA \left[ A_s F_y \left( d - D_p - y_s \right) - S_1 F_y \right] + S_1 F_y.$

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 \( M_{ult,4} = S_x F_y \); this is also generally true for negative moment regions, however the limit may be further reduced depending upon its geometry.

5.6 DEADLOAD MOMENT BY THE FOUR ANALYSIS METHODS

Permanenct or dead loads on the superstructure are those loads which always remain and act on the bridge throughout its lifetime. The deadload is the aggregate weight of all superstructure elements acting upon the bridge (i.e., the superstructure must bear its own weight). The capacity of the bridge is reduced accordingly and can be significant in and of themselves. The deadload includes the steel girders, the secondary steel members (e.g., crossframes, splice plates, stiffeners), the concrete decking, its layers of reinforcing rebar, and any stay-in-place formwork. 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., \( M_{DLeff} = \sigma_{DL} S_{LL}, S_{LL} = \{S_1, S_2, S_3, \text{ or } S_4\} \)).

A distinction is made regarding deadload that is constructed or placed on the superstructure after the concrete decking has cured and begun working with the steel framing to resist any load. The superimposed deadload includes the concrete sidewalk and parapet, their reinforcement, any wearing surface for the roadway, utilities and their infrastructure, and transportation signing and lighting. The superimposed deadload moment \( M_{SDL} \) is also determined from first principals and again we use the values determined by the BARS analysis. For a section designed as fully composite, AASHTO-STD allows for the superimposed deadload
to act upon a partial composite section with a section modulus SSDL defined by an effective strength ratio of $3n_{eff}$ (see equations above). Hence, $S_{DL} = I_s / y_s < S_{SDL} < S_I = I_t / y_t$ and the computed stress for superimposed deadload will not be as significant for a fully composite section (i.e., $\sigma_{SDL} = M_{SDL}/S_{SDL}$) as it would be for a noncomposite design (i.e., $\sigma_{SDL} = M_{SDL}/S_{DL}$).

The effective superimposed 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 and the effective superimposed deadload moment is likewise determined (i.e., $M_{SDL_{eff}} = \sigma_{SDL} S_{LL}$, $S_{LL} = \{S_1, S_2, S_3, \text{ or } S_4\}$).

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

In general, the principal loading constraint which highway bridges are designed by is truckloading. Given the variety of trucks in use, it was determined that a standard set of design loading caused by truck traffic needed to be developed. In the early part of the century, designers utilized a train of trucks as design loading for their bridges. As the highway trucking industry grew, and along with it truckloads, bridges began to show evidence of overstressing in structural components. In 1944, a suite of hypothetical truck classes designated as H and HS class trucks were developed by AASHTO. These vehicles were created with two and threes axles, respectively, set at specified spacing. These truck classes still represent the core standard in use today in the United States. For this project, we will specifically examine the HS20-44 truck classification, as is detailed in Figure 5-6.

Replacing the train of trucks in the 1935 design code are laneload configurations which approximate a 40 kip (178.0 kN) truck followed by a train of 30 kip (133.4 kN) trucks. To model this, a uniform distributed laneload is used combined with a concentrated force or point load. For the HS20-44 specification, the laneload is 0.64 kip per linear foot (9.34 kN per linear meter) and the point load is 18 kip (80.1 kN) (Figure 5-6). The laneload is not necessarily continuous over the length of the structure or even within the length of any given span; the laneload is distributed in order to cause the greatest response for the simulated train of vehicles. For simple span (i.e., noncontinuous) bridges and for positive moment considerations, a single point load is used to determine the maximum liveload response; however, for negative moment evaluation of a continuous bridge design, a second point load (equal in weight to the first point
load) is placed in the adjoining span to determine the maximum liveload response. Where truckloads generally govern for short span bridges, the laneload generally governs for long and continuous span bridges.

![Figure 5-6: AASHTO Truck and Lane Load Specifications](image)

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. Note that the rear axle spacing for the design truck is variable from 14 to 30 feet (4.27 m to 9.15 m) to allow for the variability in this parameter in the normal traffic population. For most cases, the smaller axle spacing causes the greater liveload response for the bridge; however, a longer axle spacing may be appropriate for identifying negative moment at the pier.
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 Structure Rating chapter of the ODOT Specifications Manual 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.

5.8 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. Note that the corresponding effective deadload $M_{DL_{eff}}$ and superimposed deadload $M_{SDL_{eff}}$ moments are used with the calculated liveload $M_{LL} = \{M_1, M_2, M_3, \text{ or } M_4\}$ moment by the four analytical methods. 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.

The capacities for the span as a composite section (i.e., identified by Methods 1, 2, and 3) are significantly higher than the capacity for the noncomposite steel beam (i.e., Method 4) as designed. Although partial composite action is clearly present in the measured data from the diagnostic load test, 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 Specifications Manual 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 (i.e., $M_{ASinv} = 0.55 \, F_y \, S_4$, $M_{ASopr} = 0.75 \, F_y \, S_4$, and $M_{ult} = Z_x \, F_y$). Further, the effective deadload and superimposed deadload shall be assigned those moments calculated by first principals for a noncomposite beam (i.e., $M_{DLeff} = M_{DL}$ and $M_{SDLeff} = M_{SDL}$).

Note that the liveload moment must still be defined by the unintended composite action for the section (i.e., $M_{LL} = \{M_1, M_2, \text{ or } M_3\}$) to account for the current condition of the structure. It seems contrary to define the liveload by the current bridge condition of partial composite action and yet define the remaining capacity by the assumed noncomposite action at the allowable stress and limit state. However, note that this would lead to a ridiculously large rating factor for the section (e.g., $M_3 \sim 2M_4$). This modified approach is identical in concept and almost equivalent in practice to that suggested by the NCHRP Manual for Bridge Rating (discussed above). 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 5-7. 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.
5.9 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 lane-load 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.

5.10 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).

A suite of condition indices can be processed directly from the measured and/or modelled sections in order to assess the performance of the structure and compare this with the nominal design values, including:

- Truckload moment, analyzed by four different methods
- Capacity load rating, analyzed by three different concepts and two loading cases
- Working or allowable stress
- Load Factor strength limit state
- Overload serviceability limit state
• HS20 Truckload rating, Inventory and Operating

• HS20 Laneload rating, Inventory and Operating

This research provides a novel approach to condition assessment by using the measured and/or modelled influence lines to virtually simulate the rating loads for immediate field assessment of a highway bridge using the above suite of condition indices.
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 cannot 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.

1) Span Data
2) Section Data
3) Cross Brace Data
4) Meshing Data
5) Cover plate Data
6) Loading Data

The first screen of the UCII Bridge Modeler is a splash screen (see Figure 6-1). It shows the UCII website and provides a direct link to the UCII website (if an internet connection is present). The website can be used to obtain support for the software, if needed.

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.
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Figure 6-1: UCII Bridge Modeler Startup Splash Screen

Figure 6-2: Span Data Entry Screen
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.

The next tab is the most important tab. A lot of information about the critical features of the bridge is input. We will see each of them in detail.

6.2.2 Section Data

This tab, called the “Section Data”, takes input regarding the girders, slab, sidewalk and parapets. There are default values in all of the text fields. The user can change the 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 have been found not to have an effect on the model, therefore they 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. The girder types can either be from a database [Austin et al. 1992] or user defined. 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.

![Section Data Entry Screen](image)

If the bridge has multiple girder sections, the multiple girder sections choice should be “Yes”. This brings up a different grid for choosing number of sections and the girder type in each section. It is assumed that the girder sections are of equal length and type in every girder. Also typically bridges with multiple sections have girder sections that are standard sections. Hence the user can select only pre defined girder types for each section. Figure 6-5 shows the ‘Multiple Girder Section’ selection grid.
By default two sections are defined with equal lengths. If only the number of sections is changed, the lengths are still evenly distributed. But the user can also redistribute the section lengths as per the bridge plans. It must be kept in mind that sum of girder section lengths must be exactly equal to length of the bridge. The actual bridge length, as per the plans, is from the mid point of bearing on one abutment to mid point of bearing on the other abutment and the girders might extend up to a foot beyond the bearings. This discrepancy must be avoided in the model as the length of girder must equal the length of bridge in the model. The girder naming scheme must be kept in mind. Girder 1 is the girder closest to the right edge of the bridge as per the diagram in the Section Data. This translates to the bottom edge of the diagram in the “Span
Chapter 6 - Finite Element Modeling of Bridges

Data” and “X-Brace Data”. The origin in the picture in “Span data” is the right end of the bridge as shown in the sectional view.

6.2.3 Cross Brace Data

The next tab is the “X-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. But nowadays, staggered braces are common too. 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. It is very critical that the “UPDATE” button is checked. Also the distances between the braces must add up to the right length of the bridge with the skew accounted for. The error cannot exceed more than six inches. It is a good idea to write down the lengths in “Notepad” earlier and copy-paste each value into the grid text boxes. The total distance available to distribute between cross braces is equal to length of bridge in zero skew bridges. In positive skew bridges the total distance is from the start on girder one to the end of the last girder. The distance to the first and last cross brace is measured from the respective ends, as shown in the picture. There is also a minimum value for the distance to first and last cross brace. The software does not allow the user to input a value less than that. Since the software checks this compatibility in real time, if number “12” is to be entered, it is best to copy 12 from “notepad” and paste in the text box. In negative skew bridges the total distance available is from start of the last girder to end of first girder as per the naming scheme defined in the “Section Data”. 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 6-6 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 next tab is the “Meshing Data” tab. 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 6-7 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 next tab is the “Cover Plate Data” tab. The lengths of the cover plates and their respective sectional information are entered. The picture on the tab 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. The graphic shown in Figure 6-9 shows the cover plates and the other deck parameters in scale.
6.2.6 Loading Data

The last tab of the software is the “Loading Data” tab. This tab is used to input values to generate the truckloading in the model. 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 truckload simulation has to be done in a couple of steps. First the model is created without the loads and simulated in SAP2000. The SAP2000 program generates an “.eko” file. This file must be put in the Bridge Modeler folder and then the “Create Loads” option in the “Create” menu is selected. The “.eko” file contains the exact X, Y, and Z coordinates of all the nodes in the model. This information is required to generate the load cases for the truckload generation. The load creation process also generates a “.scrap” file with the same file name as the bridge model being created. This file is required for the post processing of the SAP analysis results.
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Figure 6-9: Cover Plate Section View

Figure 6-10: Loading Data Entry Screen
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 step size is calculated with the following formula:

\[
\text{Step size} = \frac{\text{Truck length} + \text{actual bridge length}}{\text{No of steps} - 1}
\]  
(6-1)

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. Assuming that there are two lanes in a 36 ft (10.97 m) wide bridge with each lane at 11 feet (3.35 m) and the truck width being 6.667 ft. (2.03 m), the offset for lane 1, the bottom lane as per the diagram above, is calculated as follows:

\[
\text{Offset} = \left(\frac{36}{2} - 11\right) + \frac{11}{2} - \frac{6.667}{2} = 9.167 \text{ ft. (2.794 m)}
\]  
(6-2)

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 [SAP200 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 be more and 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.

Because the model and test data are matching each other to the different level for different bridges, the initial values of OFs are different to different bridges. If a nominal model simulates the bridge very well, then the initial OF values will be smaller and vice versa. Therefore, the weights of the OFs usually will have to be changed for each bridge.

The Modal Calibration is the combined works of Lei Liu, Li Zhengsheng, Ahmet Turner, Michael Lenett, Xiaoyi Wang, and Nathan Ruth.

### 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
Chapter 7 - Finite Element Model Calibration

(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.

\[
GOF = w_1 \cdot OF_{BGCI} + w_2 \cdot OF_{MAC} + w_3 \cdot OF_{Freq} + w_4 \cdot OF_{UIL} \tag{7-1}
\]

where:

\(OF_{BGCI}\) = the objective function of BGCI
\(OF_{MAC}\) = the objective function of MAC
\(OF_{Freq}\) = the objective function of frequency
\(OF_{UIL}\) = the objective function of UIL
\(w_1\) = the weighting factor of \(OF_{BGCI}\)
\(w_2\) = the weighting factor of \(OF_{MAC}\)
\(w_3\) = the weighting factor of \(OF_{Freq}\)
\(w_4\) = 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:

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

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. The weights used in BUT-732-1043 bridge calibration are:

\[
GOF = 60 \cdot OF_{BGCI} + 2 \cdot OF_{MAC} + 2 \cdot OF_{Freq} + 15 \cdot OF_{UIL} \tag{7-3}
\]

In fact these four weights were not decided in a standard way. Since these values are just for the balances of OF’s, if 60-2-2-15 combination is fine, then if they are all divided by a common number, say “2”, then the GOF can lead to the exactly same calibration results with the new weights 30-1-1-7.5. If GOF value is considered as a whole pie, then the ratios of \(OF_{BGCI}, OF_{MAC}, OF_{Freq}, \) and \(OF_{UIL}\) is displayed in Figure 7-1.

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

In Figure 7-1, the ratio of \(OF_{BGCI}\) is 29%, the ratio of \(OF_{MAC}\) is 27%, the ratio of \(OF_{Freq}\) is 15%, and the ratio of \(OF_{UIL}\) is 29%. These ratios can be adjusted by increasing or decreasing
their weights. The ratio of OFFreq 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 other 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-url)
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 analytical BGCI before the BGCI objective function is applied. After interpolating the analytical BGCI into the location where the accelerometers were placed, the interpolated can be used to calculate the.

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 SAP®2000 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.

In the program, to find a matched mode in $\Psi_{Ana}$ for the $i^{th}$ experimental mode $\Psi(i)_{Exp}$, $MAC_{i,j=1-n}$, is calculated. Usually the maximum value within $MAC_{i,j=1-n}$ indicates the two matching modes share the same mode type. In practice, however this is not always true. Suppose $\Psi(i)_{Exp}$ has a matched mode $\Psi(j)_{Ana}$, sometimes $MAC_{i,j}$ is less than $MAC_{i,k}$ ($j \neq k$) although actually $\Psi(i)_{Exp}$ and $\Psi(i)_{Ana}$ are the same type of mode. This phenomenon occurs because the calibration started from a nominal model, but the nominal model can not represent the true bridge perfectly. It has all types of the modes that the experimental modal data has, but the mode shapes have not been tuned to fit into the exact shapes like those in experimental modes.

To overcome this obstacle, some background about modal frequency has to be introduced. If the two modes in $\Psi_{Exp}$ and $\Psi_{Ana}$ have the same type, they may not have the same index number. This is the case in the example above, $\Psi(i)_{Exp}$ and $\Psi(j)_{Ana}$. They should have a similar frequency. The fact that they share the same mode indicates they are the same eigenvector of the system. Therefore, their corresponding eigen value (frequency) should be the same [Lennet 1998].

For this reason, a “frequency window” is set up when the maximum value is picked up in the vector. Suppose $f_i$ is the frequency for the mode $\Psi(i)_{Exp}$, an interval $[f_i - \delta_i, f_i + \delta_i]$ is set up. When choosing the maximum MAC value within $MAC_{i,j=1-n}$, only those whose frequencies are in the interval $[f_i - \delta_i, f_i + \delta_i]$ are eligible to be chosen. The interval radius $\delta_i$ is a relatively small value to make sure that the right mode is chosen. For different $i$, $\delta_i$ can be different. The value can also be a large value and all the same for different $i$ if there are not any “disturbing” modes in $\Psi_{Ana}$ for any $\Psi(i)_{Exp}$.

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:
\[
OF_{MAC} = \frac{\sum_{i=1}^{q} (1 - MAC_{i,R(i)})}{q}
\]  

(7-6)

Usually \( q \) will be a value around ten. It is because in reality, the first ten experimental modes can be clearly identified as one type of mode. Also, from the previous research by Lenett [1998], 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 [Lenet 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).

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

(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.

![Figure 7-3: Frequency Objective Function Plot](image-url)


7.2.4 Frequency Objective Function

Figure 7-3 can show 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 as well as possible, two factors are considered: the slope of the trend line and the $R^2$ of the trend line.

Let $S$ be the slope of the trend line, from the plot, it is obvious to see that, the closer $S$ is to 1, the better frequencies match. The objective function of slope $OFS$ in the calibration is:

$$OFS = \begin{cases} 1-S & \\ 1 - \frac{1}{S} & \\ \end{cases}$$

(7-8)

The smaller $OFS$ is, the better frequencies match.

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:

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

(7-9)

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

$$OFS_{R^2} = 1 - R^2$$

(7-10)
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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:

$$OF_{Freq} = \frac{OFS + OFR}{2}$$  \hspace{1cm} (7-11)

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).

$$OF_{Freq} = \frac{OF_{Freq,\text{left}} + OF_{Freq,\text{right}}}{2}$$  \hspace{1cm} (7-12)

### 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:

$$OF_{UIL} = \frac{\sum_{i=1}^{n} \left| \text{Max}(UIL_{Exp,i}) - \text{Max}(UIL_{Ana,i}) \right|}{n}$$  \hspace{1cm} (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.

### 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 7-4: Revised Load Condition for Bridge Model Calibration](image)

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 mode appears in the test data of a three-
span bridge. The black curve is the mode F-111-O in the nominal model, the blue one is the same mode (F-111-O) 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.

The reason for the discrepancy between nominal model and test data is FE model parameters. It is the FE parameters that can affect the properties of bridge models. In the field, bridges experience rain, temperature variations, traffic, etc., and therefore there is cracking on deck, rusting under supports, etc. But these features are not modeled in the nominal model. This is because the nominal model is an ideal bridge built according to bridge plans. FE parameters in the model can be adjusted to simulate the effect of those features in the real world bridge. For example, different level of composite actions between steel girders and concrete slab and the different boundary conditions are all able to be simulated by setting some FE parameters to the appropriate values.

This shows the importance of the values of those FE parameters in the model. These parameter values are changed, or calibrated, to affect the model so that the calibrated model can reflect the static and dynamic properties of the real world bridge. 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.
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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 remain unchanged. 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.

In the auto-calibration program, the selected FE parameters have the highest influence to $OF_{BGCI}$, $OF_{MAC}$, $OF_{Freq}$, and $OF_{UIL}$. In other words, they have greatest influence on the GOF. When the value of these parameters are changed, the analytic BGCI, MAC, frequencies, UIL will also be changed. 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:

![Mode: F-111-O](image-url)
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: Cross Section of FE Bridge Model

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.

### 7.3.1 Calibration Order

The order in which the governing parameters are adjusted has been found to be critical to the success of the tuning algorithm. In general, the support link stiffness are tuned first, followed
by the stiffness of the concrete deck, the stiffness of the steel girders, and finally by the level of composite action present in the bridge.

The axial stiffness of the supporting links are selected for tuning first since the level of stiffness affects only the values of BGCI at the support locations. By adjusting the cross sectional area of the supporting links, $A_{SL}$, the deflections calculated in the model can be matched to the deflections determined from the modal analysis very precisely. The relationship between the axial stiffness of the supporting links and the resulting vertical support deflections is almost linear. Furthermore, variations in the remaining critical parameters do not have any appreciable affect on vertical support deflections. In many cases, the axial stiffness of the support links is computed during the initial iterative loop and is often left unchanged during subsequent iterations.

After the axial stiffness of the support links has been determined, the flexural stiffness is adjusted by varying their moment of inertia, $I_{SL}$. Unlike the axial stiffness, $A_{SL}$, the flexural stiffness of the support links has a nonlinear affect on several response indices. The axial stiffness primarily affects the shape of the BGCI but also has a marked affect on the order and shape of the mode shapes of the bridge. Because of the nonlinear relationships and interdependence of parameters, several tuning iterations are typically required to determine appropriate values for the flexural stiffness of the support links.

The next three parameters that are considered relate to the overall stiffness of the structure. The modulus of elasticity of the concrete, $E_C$, thickness of the deck, $t_d$, and modulus of elasticity of the steel, $E_S$, is varied sequentially until the objective function is minimized. The thickness of the deck and modulus of the steel have relative tight constraints as it is thought that the range of variation of these parameters is relatively small. While some would argue that the variability of the steel modulus would be sufficiently small to justify a constant value, other variables such as corrosion and section loss due to riveting in older girders are considered indirectly by allowing some variation in $E_S$. The range of variation for the modulus of the concrete is somewhat larger, though, based on the inherent variability of mechanical properties of concrete. In fact, advanced tuning algorithms actually provide for non-uniform values for the
concrete modulus recognizing possible variations in strength and stiffness from truck-to-truck as
the deck is cast.

The last parameter to be adjusted in the iterative procedure is the level of composite
action of the bridge. This is adjusted by varying the flexural stiffness of the rigid links
connecting the deck to the top flange of the girders. At first, the rigid link stiffness is treated as a
uniform parameter over the length of the bridge. During later stages of the tuning procedure,
however, the parameters are permitted to vary locally to reflect the lower levels of composite
action observed in negative-moment regions of bridges and in high-moment regions of non-
composite structures.

7.3.2 The Area of the Support Links

Both $A_{SL}$ and $I_{SL}$ are the support springs of the bridge. They are functioning as boundary
conditions. Correct boundary conditions are very important in FE modeling to simulate the
bridge static and dynamic performances. After many years of service, the boundary conditions
are not likely to be the same as in a new structure due to aging and deterioration. It is obvious
that the boundary conditions of the bridge need special consideration in order to achieve an
accurate numerical model. There are two types of support springs: vertical spring and horizontal
spring. Vertical springs are mainly related to the vertical deformation at the supports.
Horizontal springs are simulating the changed boundary condition in the horizontal direction.
They represent the degree of restraints based on the actual deterioration. The values of
horizontal spring will affect the deformation, frequency, and mode shapes of the structure.

ASL parameters take effect on the support links between piers and girders, functioning as
the vertical spring supports of the bridge. It is the area property of support links. The support
links are connecting piers and girders vertically (Figure 7-6). By controlling the area of support
links, the axial direction stresses are controlled and used to simulate the magnitude of vertical
spring. The number of this parameter in the model depends on the number of supports on the
bridge. There are two different supports: abutments and piers. The number of this parameter can
be calculated as:

$$N_{ASL} = (N_{Abx} + N_{Pier}) \cdot N_{Girder}$$

(7-14)
where:

\[ N_{ASL} = \text{the number of } A_{SL} \text{ parameters} \]

\[ N_{Abt} = \text{the number of abutments} \]

\[ N_{Pier} = \text{the number of piers} \]

\[ N_{Girder} = \text{the number of girders} \]

Suppose that a bridge has three spans and five girders. Then there should be two abutments, two piers and five girders. \( N_{Abt} = 2, N_{Pier} = 2, N_{Girder} = 5, \) and \( N_{ASL} = (2 + 2) \times 5 = 20 \)

Since the \( A_{SL} \) values are to be calibrated, the location of this parameter in the model is shown in Figure 7-7 for the convenience of understanding, programming, and manual modification if necessary.

In the model, the nominal value to this parameter is 3000 in\(^2\) (1.935 m\(^2\)). During the calibration, those \( A_{SL} \) parameters will be changed to different values automatically to reduce the GOF. The limitation of tuning is set from \( 10^{-2} \) to \( 10^5 \) in\(^2\) (6 \times 10^{-6} \) to 64.52 m\(^2\)).
7.3.3 The Moment of Inertia of the Support Links

Typically, the steel bearings in bridges are designed as pinned or roller bearings. Due to debris, corrosion on the surface of the bearings over years, the girders at the abutments and middle supports can not move or rotate freely. Some degrees of horizontal or rotational restraints are added to the pinned connections by the steel bearings. To simulate the changed boundary condition in the horizontal direction, the horizontal springs are used in the 3D FE model. The coefficients of horizontal springs will represent the degree of restraints based on the actual deterioration. The values of the spring stiffness will affect the deformation, frequency, and mode shapes of the structure. The sensitivity analysis between the BGCI and the coefficients of horizontal springs is shown in Figure 7-8 [Li 2003].

![Figure 7-8: Sensitivity Plot between Coefficients of Horizontal Springs and BGCI for BUT-732-1043 Bridge](image)

In Figure 7-8, the x-axis is the coefficient of horizontal spring value and y-axis is the normalized BGCI deformation summation. It is observed that the summation of BGCI will increase slowly when the coefficient of horizontal spring is increased. “It is obvious that the
horizontal springs have more effects on the deformation in the end span than of the middle span. It is reasonable because the horizontal springs will contribute more moment restrictions at the abutments. “It is noticed a maximum increase of 12% in frequency was obtained when the stiffness of horizontal springs was varied from 0 to 1,500 kip / in (6672 KN/m).”[Li 2003].

The moment of inertia of the support links ($I_{SL}$) is used as horizontal spring functionality. It is also a parameter on the supports links in the bridge models. It is the moment of inertia property of the support links. The number of this parameter in the model is as same as $A_{SL}$ because they are the different properties of the same elements. $N_{ISL} = N_{ASL}$

The location of this parameter in the model is shown in Figure 7-9 for the convenience of understanding, programming, and manual modification if necessary.

![Figure 7-9: The Location of ISL in Models](image)

The nominal value for $I_{SL}$ is 1 in$^4$ (4.16 x $10^{-7}$ m$^4$). During the calibration, those $A_{SL}$ parameters will be changed to different values automatically to reduce the GOF. The limitation of tuning is from $10^{-4}$ to $10^5$ in$^4$ (4.16 x $10^{-11}$ to 0.0416 m$^4$).
In the model calibration, the $I_{SL}$ values on the second pier are usually fixed to 100 in$^4$ ($4.2 \times 10^{-5} \text{ m}^4$) to simulate the fixed pier according to the bridge plan. Therefore in the model, the nominal $I_{SL}$ values on the second pier are set to 100 in$^4$ ($4.2 \times 10^{-5} \text{ m}^4$), and during the calibration, these values on the second pier will always keep 100 in$^4$ ($4.2 \times 10^{-5} \text{ m}^4$). Please check Figure 7-9 to see the fixed value location.

### 7.3.4 The Modulus of Elasticity of Concrete

The modulus of elasticity of concrete is an effective parameter for calibration, too. The modulus of elasticity $E$ is often called Young’s modulus and the value of $E$ shows the degree of elasticity of one material. A larger modulus will increase the stiffness of the structure, which in turn will decrease the deformation of the structure. Compared to the material properties of steel, the properties of concrete are more variable. The modulus of elasticity of concrete is one of the key properties. Due to traffic condition, weather, and other factors, the modulus of elasticity of concrete over years will have some degree of variation. Furthermore, since the strength of the concrete at the time when it was poured is not well known, modulus of elasticity of concrete ($E_C$) is an important parameter in the condition evaluation of the bridge.

The relationship between the BGCI and the modulus of elasticity is shown in Figure 7-10 [Li 2003]. The x-axis is the $E_C$ value and y-axis is the normalized BGCI deformation summation. The BGCI will decrease slowly when the modulus of elasticity of concrete deck increases from 3000 to 4500 ksi (20,685 to 31,028 MPa).

Since concrete is used to build the deck slabs, this parameter defines the modulus of elasticity of the shell elements on the deck in the bridge model. The number of variables for this parameter may vary. A global value of EC for the whole bridge can be used and calibrated. In order to take into account the different levels of deterioration on different locations on a bridge and the different types of concrete, multiple values are used for EC on different locations on the bridge. The shell elements on the deck in a bridge model are grouped into strips along the bridge. There are normally four strips of shell elements defined between any two girders and one strip of shell elements defined at outside of both first and last girder. So the total number of this parameter will be $N_{Ec} = (N_{Girder} - 1) (4) + 2$, where $N_{Ec}$ is the number of $E_C$ parameters and $N_{Girder}$ is the number of girders on this bridge. For example, there will be 18 $E_C$ parameters on a typical five
girder bridge. The location of this parameter in the model is shown in Figure 7-11 for the convenience of understanding, programming, and manual modification if necessary.

The nominal value for $E_C$ is set to 4,750 ksi (32,751MPa). During the calibration, those $A_{sl}$ parameters will be changed to different values automatically to reduce the GOF. The limitation of tuning is from 2,000 ~ 6,000 ksi (13,790 to 41,370 MPa).

### 7.3.5 The Thickness of Deck

The thickness of concrete deck ($t_d$) is one of the best-defined variables in the process of calibration of bridge. However, the actual thickness of the deck may be different from the values when it was constructed. The thickness of the concrete deck may also vary from time to time during the bridge’s life. It may also vary with the location, for example, slope towards to the sidewalk. It is necessary to set the thickness of the deck as one of the variables in the calibration of the bridge. From sensitivity analysis results, it is noticed that the frequencies of the bridge will slowly decrease when the thickness of the deck get larger. A maximum decrease by 4.2% was observed. The increase in the thickness will make the bridge have more stiffness and less flexibility. As a result, the deformation of the bridge will decrease.

![Figure 7-10: Sensitivity Plot Between EC and BGCI for BUT-732-1043 Bridge](image)
The thickness of deck once was considered as a global physical property of a bridge. This consideration is based on the fact that when a bridge was built, the thickness of the deck should be same everywhere. However, in reality, the deck thickness might be different at different locations on a bridge due to the different level of traffic and weather deterioration. After many years of service, the bridge surface is also possibly refurbished. All these facts indicate that there is need to consider the deck thickness locally. Since the shell elements are separated into strips on the deck, it is convenient to adjust the deck thickness by the strips of shell elements along the bridge. \( t_d \) is a property of shell elements. It is related to the number of shell elements on the deck. The number of the \( t_d \) parameter in the model will be, \( N_{td} = N_{Ec} \) where \( N_{td} \) is the number of TD parameters and \( N_{Ec} \) is the number of EC parameters.

The location of this parameter in the model is shown in Figure 7-12 for the convenience of understanding, programming, and manual adjusting if necessary.
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The nominal value of $t_d$ is from the bridge plans. Different bridges have different designed deck thickness values. There is not a fixed initial value for the deck thickness. In Figure 7-12, the TD initial value for this BUT-732-1043 bridge is 8.5 in (0.2159 m). The calibration limitation of $t_d$ tuning is typically set to be from 6 ~ 11 in (0.1524 ~ 0.2794 m), most of the bridges’ $t_d$ values are within this interval.

7.3.6 The Modulus of Elasticity of Steel

The modulus of elasticity of steel ($E_s$) is also a sensitive parameter for the model performance. It describes the degree of elasticity of the girders because steel is used to build the girders. Compared with the modulus of elasticity of concrete, there is less uncertainty on this parameter. Concrete is more vulnerable to be changed by the environment and impact of traffic while steel which is used to build girders was made and used in a more standard way and it is more stable against the influence from environment. However, the modulus of elasticity of steel on a bridge still has some uncertain deviation. When some steels were made under extra care, the $E_s$ value might be higher than it should be and this increases the safety of the steel. In some other cases, during the usage of the bridge, the $E_s$ value of some steels may be less than the design value due to deterioration. The change of $E_s$ value will change the model performances...
so it should be calibrated. The limitation (range) of calibration should be narrow because, as mentioned before, the steel properties are not very flexible to the environment.

The modulus of elasticity of steel is a global value that applies to the whole bridge. Since it is a global value, there is only one $E_S$ value to be adjusted during calibration. The location of this parameter in the model is shown in Figure 7-13 for the convenience of understanding, programming, and manual adjusting if necessary.

![Figure 7-13: The Location of $E_S$ in Models](image)

The nominal value for $E_S$ in the nominal model is 29,000 ksi (199,955 MPa). The calibration limitation of $E_S$ tuning is set to be from 27,000 to 31,000 ksi (186,165 ~ 213,745 MPa).

### 7.3.7 The Moment of Inertia of Rigid Links

Composite action is the response of concrete deck relative to the steel girders below. This composite action is one of the key parameters in calibration. There are three general levels of composite action in the interaction of slab and steel girder. They are non-composite, partially composite, and fully composite action. Bridges are usually designed to have non-composite action between deck and girders. However, they will not act as non-composite in response to moderate loadings. This is because of the adhesion between the concrete deck and steel, also because of frictional forces between them. Due to the different load and environmental
conditions on different bridges, the composite actions between girders and concrete decks is found to be different for each bridge. Even for the same bridge, the composite actions are different at different locations. Thus, the actual composite coefficients need to be identified.

The moment of inertia of rigid links is designed to describe the deck-girder composite relationship in the FE model. In the model, the frame rigid link elements are used to connect shell elements of the deck and frame elements on the top flange of the girders. “A small stiffness will indicate a low level of composite action and a high stiffness indicates a higher level of composite action [Li 2003].” When different values to the rigid link elements are assigned, the GOF will be affected quite efficiently from the sensitivity study of the $I_{RL}$.

In the sensitivity analysis of $I_{RL}$ and BGCI, the $I_{RL}$ value is systematically varied from $10^{-8}$ to 10,000 in$^4$ (0.42 to $4.2 \times 10^{10}$ mm$^4$). The sensitivity of the BGCI to the $I_{RL}$ is then plotted in Figure 7-14. In the figure, the x-axis is the $I_{RL}$ value and the y-axis is the normalized BGCI deformation summation.

![Figure 7-14: Sensitivity Plot Between $I_{RL}$ and BGCI for BUT-732-1043 Bridge](image)

Figure 7-14: Sensitivity Plot Between $I_{RL}$ and BGCI for BUT-732-1043 Bridge
The total number of rigid links variables varies from bridge to bridge. It depends on the number of joints $N_{\text{Joint}}$ on one girder and the number of girders $N_{\text{Girder}}$ on the bridge, the total number of rigid links variables $N_{\text{IrI}}$ will be $N_{\text{IrI}} = N_{\text{Joint}} \times N_{\text{Girder}}$

This means, for every node on the deck, there is a rigid link that connects it with top flange. The location of this parameter in the model is shown in Figure 7-15 for the convenience of understanding, programming, and manual adjusting if necessary.

For example, on Butler-732-1043 Bridge, there are 5 girders, and there are 63 nodes (joints) on each girder, so the total number of moment of inertia of rigid links variables $N_{\text{IrI}}$ is $(63 \times 5) = 315$

The nominal value for $I_{RL}$ is set to 1 in$^4$ (4.162 x 10$^{-7}$ m$^4$). During the calibration, those $I_{RL}$ parameters will be changed to different values automatically to reduce the GOF. The limitation of $I_{RL}$ tuning is set to be from 10$^{-6}$ to 10$^6$ in$^4$ (4.16 x 10$^{-13}$ to 0.416 m$^4$). Although the range is set to be quite wide, the calibrated $I_{RL}$ values are usually in a much narrower limitation.
7.4 CALIBRATION ALGORITHM OVERVIEW

In the last chapter, the parameters that can affect bridge static and dynamic properties were described. Since those parameters in the bridge model have the effect on the BGCI, MAC, frequency and UIL, they are considered as variables in the GOF.

\[
GOF = f(A_{SL}, I_{SL}, E_C, T_D, E_S, I_{RL})
\]  

(7-15)

The function \( f \) connects the relationship between those finite element parameters \( (A_{SL}, I_{SL}, E_C, \text{etc.}) \) and the global objective function. Because of the variety of bridges and field test results, and the complexity of system functions, it is not easy to explicitly define this function \( f \). To find the minimum value of GOF without the knowledge of function \( f \), a gradient-searching based algorithm is 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 7-16 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 7-16.

Suppose the calibration is working on the moment of inertia of rigid links \( (I_{RL}) \) tuning. The objective is to find a set of rigid links values \( I_{Min} \) (\( I \) is a vector representing a set of rigid links’ values in a bridge model, for BUT-732-1043 bridge, \( I \) is a \([315 \times 1]\) vector) so that \( f(I_{min}) = J_{min} \) where \( J_{min} \) is the minimum GOF value. The steepest descent algorithm is used to find the point \( I_{min} \) in the parameter hyper plane.

Figure 7-17 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 a $f(I)$, the steepest descent algorithm is:

1) Select an initial point $I_0$.

2) Find the gradient of the GOF at that point (the red arrow in Figure 7-17).

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

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

![Calibration Hyperplane](image)

Figure 7-16: Illustration of the Relationship Between FE Parameters and the GOF

Finally, the searching in this “rigid links” hyper plane will approach the bowl bottom (the point which has the minimum function value, in Figure 7-16, it is the deepest blue area). How
well the final point is close to the theoretical minimum point (the asterisk in Figure 7-17) depends on the step size.

This is the original idea of the algorithm. However, it can not be directly used in bridge model calibration: it is not efficient enough from the practical point of view. It requires the gradient vector at the current point in every step: going one step along the gradient direction, reach a new point, update the gradient vector again…

Figure 7-17: Illustration of the Steepest Descent Algorithm

Bisection method is used to solve this efficiency problem. Therefore, the original steepest descent algorithm is not used in the calibration program. To see how the program is improved with bisection method, please see Section 7.4.2.

7.4.1 Gradient Approximations

Since the function $f$ is unknown, it is not possible to get the gradient of it at any points by differential. The approximate gradient will be used as alternative. The assumed expression $J = f(x)$ will be expanded with a Taylor series to yield,
Chapter 7 - Finite Element Model Calibration

\[
f(x) = f(x_0) + \nabla f^T(x_0) \cdot \Delta x + \nabla^2 f^T(x_0) \cdot \Delta^2 x + \cdots + \frac{\nabla^k f^T(x_0)}{k!} \Delta^k x \quad (7-16)
\]

In the small neighborhood of \(x_0\), the higher order part of the Taylor series can be neglected and the approximation is:

\[
f(x) \approx f(x_0) + \nabla f^T(x_0) \cdot \Delta x \quad (7-17)
\]

Let \(\Delta f = f(x) - f(x_0)\). Then,

\[
\Delta f \approx \nabla f^T(x_0) \cdot \Delta x \quad (7-18)
\]

where:

- \(\nabla f(x_0)\) is a \([1 \times n]\) vector which is the approximate gradient array at point \(x_0\) in an \(n\)-dimension hyper plane
- \(\Delta x = x - x_0\) is a \([1 \times n]\) vector in the \(n\)-dimension hyper plane
- \(\nabla f = f(x) - f(x_0)\)

With different \(x\), there will be different \(\Delta x\): \(\Delta x_1, \Delta x_2, \Delta x_3, \ldots, \Delta x_n\), then:

\[
\begin{align*}
\Delta f_1 & \approx \nabla f^T(x_0) \cdot \Delta x_1 \\
\Delta f_2 & \approx \nabla f^T(x_0) \cdot \Delta x_2 \\
\vdots & \quad \vdots \\
\Delta f_n & \approx \nabla f^T(x_0) \cdot \Delta x_n
\end{align*} \quad (7-19)
\]

When they are organized into a matrix, they become:

\[
\begin{bmatrix}
\Delta f_1 \\
\Delta f_2 \\
\Delta f_3 \\
\vdots \\
\Delta f_n
\end{bmatrix}
\approx
\nabla f^T(x_0)_{1 \times n}
\begin{bmatrix}
\Delta x_1 \\
\Delta x_2 \\
\Delta x_3 \\
\vdots \\
\Delta x_n
\end{bmatrix}_{n \times n} \quad (7-20)
\]

If \(\Delta F\) and \(\Delta X\) are defined as,
\[ \Delta F = \begin{bmatrix} \Delta f_1 \\ \Delta f_2 \\ \vdots \\ \Delta f_n \end{bmatrix} \quad \Delta X = \begin{bmatrix} \Delta x_1 & \Delta x_2 & \Delta x_3 & \cdots & \Delta x_n \end{bmatrix} \] (7-21)

then Equation (7-20) becomes,

\[ \Delta F_{1 \times n} \approx \nabla f^T (x_0)_{1 \times n} \cdot \Delta X_{n \times n} \] (7-22)

Assuming that \( \Delta X_{n \times n}^{-1} \) exists,

\[ \nabla f(x_0) \approx \left( \Delta F \cdot \Delta X^{-1} \right)^T \] (7-23)

can be used to find the gradient \( \nabla f \) at the point \( x_0 \) on the n-dimension hyper plane. In the program implementation, a set of \( x \) will be chosen so that \( \Delta x_n \) is orthogonal to each other insuring that \( \Delta X^{-1} \) exists. The following routine is used in the program to get an orthogonal \( \Delta x_n \).

Now suppose that,

\[ x_0 = \begin{bmatrix} a_1 \\ a_2 \\ \vdots \\ a_n \end{bmatrix} \] (7-24)

and

\[ \begin{bmatrix} a_1 + \zeta \\ a_2 \\ \vdots \\ a_n \end{bmatrix} \quad x_1 = \begin{bmatrix} a_1 \\ a_2 + \zeta \\ \vdots \\ a_n \end{bmatrix} \quad x_2 = \begin{bmatrix} a_1 \\ a_2 \\ \vdots \\ a_n \end{bmatrix} \quad x_3 = \begin{bmatrix} a_1 \\ a_2 \\ \vdots \\ a_n \end{bmatrix} \quad \cdots \quad x_n = \begin{bmatrix} a_1 \\ a_2 \\ \vdots \\ a_n + \zeta \end{bmatrix} \] (7-25)
This is an easy and programmable way of picking $x_1 - x_n$, which guarantees the existence of $\Delta x^{-1}$.

### 7.4.2 Bisection Method

The steepest descent algorithm described in Section 7.4.1 is the basic idea that is used in the calibration program. But from the practical point of view, it is not efficient enough. It requires the gradient vector at the current point in every step: going one step along the gradient direction, reach a new point, update the gradient vector again...

Since the dimension of the variables is normally quite large (in the case of BUT-732-1043 bridge, the dimension of $I_{RL}$ is 315, the dimension of $E_C$ is 18) and the computation time of the structure analysis software SAP2000, the time spent on getting gradient can be long. It is not wise to spend most of the computing time to get the gradient but precede the minimum point finding just a little on the hyper plane. Bisection method is used as a good aid for this circumstance. Normally, the bisection method is used for root-finding problem.

![Bisection Method Illustration](image.png)

**Figure 7-18: Bisection Method Illustration**
In Figure 7-18, the blue curve shows the curve of a function \( f(x) \), \( p_1 \) and \( p_2 \) are the initial points for root-finding, and \( f(p_1) > 0, f(p_2) < 0 \)

Then

\[
p_3 = \frac{p_1 + p_2}{2}
\]  
(7-26)

After calculation, \( f(p_3) < 0 \). As a result, the root is between \( p_1 \) and \( p_3 \), and

\[
p_4 = \frac{p_1 + p_3}{2}
\]
\[
p_5 = \frac{p_1 + p_4}{2}
\]  
(7-27)
\[
p_6 = \frac{p_4 + p_5}{2}
\]
etc.

This method guarantees the root-finding process.

In the bridge model tuning process, we actually did not know how well the calibration program can achieve before we started. So there is not a reference value (in the traditional bisection method, the reference value is zero). A revised bisection method is developed; the following is the algorithm in detail.

Please see Figure 7-19, at the beginning, a point \( p_1 \) is picked up, find the gradient \( r \) at \( p_1 \) and then go along this direction \( r \) with a step size, pick up another point \( p_2 \) (in this one-dimension chart, \( r \) is toward to minus infinity).

If \( f(p_1) > f(p_1) \), then it means that the function value rises. In this case, it is assumed that there is a minimum point between \( p_1 \) and \( p_2 \), then:

\[
p_3 = \frac{p_1 + p_2}{2}
\]  
(7-28)
\[
p_4 = \frac{p_1 + p_3}{2}
\]  
(7-29)
Select $p_1$ and $p_3$ as the new set for the bisection interval. Keep repeating the steps above for several times, a point which is approximately on the bottom of the curve will be achieved.

![Figure 7-19: Revised Bisection Method](image)

This revised bisection method is combined with the gradient method will save time in the calibration process. A relatively big step size for the gradient method will be applied on the hyper plane to speed up the procedure of finding the next “lower position”. Once the objective function value rises up, from that point, the bisection algorithm will take over to “refine” it. This will find the minimum point along that direction in the hyper plane. Figure 7-20 illustrates for the bisection method application with gradient algorithm.

The algorithm is:

1) Pick up an initial point (point 1).

2) Calculate the gradient array on that point (the red arrow on the point 1).

3) Keep going along with a step size on the hyper plane if the GOF value keeps going down (point 2, 3).
4) Stop at the point that the GOF goes up for the first time (point 4).

5) Apply bisection method between point 3 and point 4. Finally Point 5 is achieved.

6) Repeat step 2 to step 5 from point 5. Finally after bisection application on this direction, point 8 is obtained.

7) Repeat step 2 to step 5 from point 8.

With only several iterations, the algorithm approaches the point on the bottom of the plane. For a more precise result, a smaller step size can be used.

In the calibration program, the first point \( p_1 \) can be decided by two ways. First, the user can specify any initial value to a specific parameter. This means that \( p_1 \) can be set to any preferred location in hyper plane. However, the user might have no idea where is the best starting point. So the second way is mainly used, the program itself goes into the model to pick up the current location as \( p_1 \). Since the parameter values in the nominal model are set to reasonable average values based on civil engineering knowledge, the nominal value in the nominal model is good point to start the calibration.

The step size in the step 3 of the algorithm above was decided by calibration experience. From the calibration results of different bridges, an efficient, stable step size was chosen so that the bridges can be calibrated smoothly. Adaptive step size can be considered in future to further optimize the model calibration.
7.4.3 Calibration Flow Chart

The calibration algorithm, data manipulation and analysis are implemented in an integrated computer program. In this section, the flowchart of the program will be shown.

According to the flowchart in Figure 7-21, 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.
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 BACKGROUND 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

Load rating (described in detail in Chapter 5) is a process of assessing a bridge in its current condition to determine adequacy for a certain vehicle load. Load rating is performed analytically but can be modified based on the results of field testing, FE modeling, or a combination of the two. Figure 1-5 shows a schematic chart of the existing UCII rating strategy. In the existing strategy, the rating factor of the bridge is computed using Equation (5-10) (repeated below for convenience) [AASHTO 2002] where the capacity, $C$, is calculated using the reports of the visual inspection of bridge, the deadload, $DL$, is computed based on simple beam equations, and the liveload, $LL$, is based on a moving load analysis that is adjusted to match field data or FE results when they are available.

$$RF = \left( \frac{C - A_1DL}{A_2(LL + I)} \right)$$ (5-10)

UCII performs truckload tests to arrive at unit influence line diagrams for the girders. Modal tests are performed to obtain dynamic characteristics of the bridges. A Global Objective Function [Wang 2005] depending on the static and dynamic properties is defined and used for calibration of an FE model. A nominal FE model, which is a 3D model of the bridge based on construction plans, is tuned to this objective function by changing experimentally pre-determined parameters to arrive at a tuned FE model. This tuned model is then used for load rating. When field data is limited or not available, the nominal FE model can be used for 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.

Within Equation (5-10), 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.

8.1.2 Basis and History of Lateral Load Distribution Factors

A Distribution Factor (DF) is defined as the ratio of load taken by an individual girder to the total load applied to the bridge section. It is used in an analysis or design to determine the magnitude of load that each girder must be designed for and is inherently a function to the lateral wheel-load distribution of the bridge. Different philosophies of design give different equations for the calculation of distribution factors. Distribution factors have been in use for assessing the live-load distribution on bridges since 1930s. Work done by Newmark [1948] and Westergaard [1930] initiated the concept of wheel load distribution factors.

8.1.2.1 Methods of Computing Distribution Factors:

The most widely accepted methods of computing distribution factors are:

1) Equations in AASHTO specifications

2) Finite element modeling

3) Field tests

The two most widely used design specifications are maintained by the American Association of State Highway and Transportation Officials (AASHTO). These are the AASHTO Standard Specifications (AASHTO-STD) [AASHTO 2002] and AASHTO Load and Resistance Factor Design Specifications (AASHTO-LRFD) [AASHTO 2005].

8.1.2.1.1 Lateral Load Distribution Models – AASHTO-STD:

Equation (8-1) is given in AASHTO-STD for the distribution factor of a steel stringer bridge. The AASHTO-STD formulation was the first to be widely adopted for computing
distribution factors. This model is a result of work by Newmark [1948] and Sanders and Elleby [1970] who idealized the bridges as orthotropic plates. As is apparent in the AASHTO-STD approach, the only factor that affects the load distribution is the spacing of the girders. This equation does not consider variations such as skew, continuity, etc. Until the introduction of AASHTO-LRFD distribution factors, this equation was the predominant equation for estimating lateral load distribution within typical bridges.

\[
DF = \frac{S}{Q} \quad (8-1)
\]

where:

\[S = \text{Spacing of Girders}\]
\[Q = \text{Quantity depending on type of members, 5.5 for steel and prestressed concrete I-girders with concrete deck.}\]

8.1.2.1.2 Lateral Load Distribution Models – AASHTO-LRFD:

The AASHTO-LRFD Specification defines the DF as a function of girder spacing, span length, girder stiffness, deck thickness and skew. Different equations have also been presented for one lane loaded vs. two or more lanes loaded. The AASHTO-LRFD formulas have been developed based on the NCHRP project 12-26 [Zokaie et al. 1991]. While various bridge types were analyzed only the beam and slab type is reported here. A set of 365 beam and slab bridges were randomly chosen from a database and various tools were used for calculating distribution factors. Three levels of sophistication were used in the analyses:

- **Level 1 Methods**: Simplified formulas devised by researchers, during recent years.

- **Level 2 Methods**: Graphical and simple computer-based analyses, nomographs and influence surface methods were used. Also simplified two dimensional methods such as grillage analyses were used.

- **Level 3 Methods**: Various finite element packages were used, with results from each compared. For beam and slab bridges, SAP, FINITE and POWELL programs were used.
The data thus obtained was then statistically processed using “Multi-dimensional Space Interpolation (MSI).” Sensitivity studies were completed on various parameters to determine their effect on load distribution. The parameters that were found to affect the distribution factor were spacing of girders \( S \), span of bridge \( L \), and longitudinal stiffness \( K_s \), and deck thickness \( t_d \).

The AASHTO-LRFD Specification provides a much more refined way of computing distribution factors as consideration is given to more parameters [Mabsout et al. 1997]. Correction factors are given for angle of skew, continuity, etc, which was lacking in the AASHTO-STD Specifications.

### 8.1.2.2 Finite Element Modeling

As was discussed in Chapter 6, a 3D FE model is used for the analysis in this study. The software package used is SAP2000. As opposed to conventional studies in this area, two FE models are used (i.e., nominal and tuned). A nominal model is created based upon the properties and dimensions as read from the plans. A calibration is performed to the nominal model so as to match the results of modal tests and truckload tests. This tuned model can be said to be a closer approximation of the bridge in the field. Thus a real view of the lateral load distribution in the girders is given by this model. In this study where DF for load rating is being devised, the significance of the results obtained form this tuned model is high.

### 8.1.2.3 Field Tests

Truckload tests are the most conventional diagnostic tests used. As was discussed in Chapter 4, a bridge is typically instrumented with strain gages at key locations (mid-span and piers) while trucks of known weight and dimensions (axle spacing, wheel base, etc.) are driven multiple times in each of the lanes. The truckload is basically divided in two parts: a) crawl test, where a truck is slowly driven across the length of the bridge and b) static tests, where trucks are located at strategic positions on the bridge. The first test will yield a unit influence line (UIL) diagram for each location instrumented. It has been demonstrated by Levi [1997] and Turer [1997], that the UIL is a damage sensitive index. Thus by obtaining an accurate estimate of liveload response of the bridge, a refined load rating can be completed.
8.1.3 Need for Refined Distribution Factors

Experimental results show lower distribution factors (and this higher ratings) by truck tests than those obtained by standard accepted equations [Kim and Nowak 1998; Ghosn et al. 1986]. This conservatism involved with distribution factors from the standard equations is not required while load rating a bridge. A much more efficient capacity evaluation will be obtained for a bridge by using the refined distribution factors. This is further corroborated by Barker [2001], who confirms a 20% increase in load carrying capacity by using experimental distribution factors in load rating process.

The refined sources of computing the DFs, such as finite element modeling and experimental results from truckload testing and modal testing, can be applied. These distribution factors, obtained from the refined sources will be then used to develop a “real” DF, which will be the closest approximation of the actual load distribution characteristics, rather than relying upon the standard equations which are a conservative generalization using statistical data for a large set of results.

8.2 COMPARISON OF DISTRIBUTION FACTORS FROM VARIOUS SOURCES

8.2.1 Methodology Employed in This Investigation

The methodology used for this investigation is described here. The various sources for computing a distribution factor have already been introduced in earlier in this chapter. Seventeen bridges from the database of 30 bridge projects were selected. 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.
| Sr. No | Bridge Name  | No. of Spans | No. of Traffic Lanes | Skew (Degs) | Total Width (feet) | Total Length (feet) | Symm (Y/N) | Lane width (feet) | Span 1 Length (feet) | Span 2 Length (feet) | Span 3 Length (feet) | Span 4 Length (feet) | Deck Thickness (in) | No. of Girders | Beam Spacing (ft) | Xframe spacing (ft) | Parapets/Railings |
|-------|--------------|--------------|----------------------|-------------|--------------------|--------------------|------------|------------------|---------------------|---------------------|-------------------|-------------------|-------------------|----------------|-----------------|-------------------|------------------|-----------------|
| 1     | BUT-732-1043 | 3            | 2                    | 0.00        | 36.000             | 195.00             | Y          | 10               | 60.00               | 75.00               | 60.00             | 0.00              | 9.25             | 5                | 8.375           | 10.833           | R                |
| 2     | PRE-725-0880 | 3            | 2                    | 10.00       | 38.000             | 192.00             | Y          | 11               | 56.00               | 80.00               | 56.00             | 0.00              | 9.75             | 5                | 8.375           | 12.000           | R                |
| 3     | CLE-52-0498L | 3            | 2                    | 0.00        | 39.170             | 221.00             | N          | 12               | 68.00               | 85.00               | 68.00             | 0.00              | 8.75             | 6                | 7.500           | 12.250           | R                |
| 4     | HAM-27-1550L | 3            | 2                    | 9.00        | 40.000             | 175.50             | Y          | 12               | 54.00               | 67.50               | 54.00             | 0.00              | 9.00             | 5                | 9.500           | 13.000           | R                |
| 5     | RIC-30-1384  | 4            | 2                    | 5.00        | 46.000             | 254.00             | Y          | 12               | 49.00               | 70.00               | 79.50             | 55.50             | 9.25             | 6                | 8.500           | 13.500           | P*               |
| 6     | RIC-30-1638  | 4            | 2                    | 0.00        | 46.000             | 285.00             | Y          | 12               | 53.50               | 89.00               | 89.00             | 53.50             | 8.50             | 6                | 8.500           | 12.390           | P*               |
| 7     | LIC-158-0164 | 4            | 2                    | 7.00        | 30.833             | 213.00             | Y          | 8                | 44.00               | 62.50               | 62.50             | 44.00             | 7.00             | 4                | 8.667           | 10.333           | P                |
| 8     | CUY-77-0645L | 3            | 3                    | 7.00        | 53.000             | 152.50             | N          | 12               | 44.50               | 63.50               | 44.50             | 0.00              | 9.00             | 7                | 8.420           | 14.000           | P                |
| 9     | CLE-52-0142  | 3            | 2                    | 0.00        | 32.000             | 143.00             | Y          | 12               | 44.00               | 55.00               | 44.00             | 0.00              | 9.50             | 4                | 9.167           | 10.320           | R                |
| 10    | PRE-503-1170 | 3            | 2                    | 35.00       | 36.000             | 156.00             | Y          | 10               | 48.00               | 60.00               | 48.00             | 0.00              | 9.00             | 5                | 7.833           | 11.000           | R                |
| 11    | FAI-33-1309  | 3            | 2                    | 39.30       | 39.500             | 418.51             | Y          | 10               | 82.38               | 130.83              | 126.88            | 78.42             | 8.50             | 5                | 8.000           | 13.000           | P*               |
| 12    | MOT-70-0553  | 4            | 2                    | 6.00        | 30.333             | 238.00             | Y          | 8                | 49.00               | 70.00               | 70.00             | 49.00             | 9.00             | 4                | 8.000           | 17.000           | P                |
| 13    | AUG-75-0201  | 3            | 2                    | 0.00        | 30.333             | 224.00             | Y          | 12               | 42.00               | 70.00               | 70.00             | 42.00             | 9.00             | 4                | 8.000           | 10.667           | P                |
| 14    | MAD-40-0745  | 2            | 2                    | 56.87       | 32.500             | 135.40             | Y          | 12               | 67.70               | 67.70               | 0.00              | 0.00              | 8.75             | 7                | 5.083           | 11.000           | P*               |
| 15    | MOT-75-0776  | 4            | 2                    | 0.00        | 30.833             | 277.00             | Y          | 10               | 57.00               | 81.50               | 81.50             | 57.00             | 8.50             | 4                | 8.000           | 15.000           | P                |
| 16    | RIC-30-1438  | 4            | 2                    | 0.00        | 46.000             | 221.50             | Y          | 12               | 49.00               | 69.50               | 60.50             | 42.50             | 9.25             | 6                | 8.500           | 13.840           | P*               |
| 17    | CLI-132-0083 | 3            | 2                    | 15.00       | 36.333             | 156.00             | Y          | 10               | 48.00               | 60.00               | 48.00             | 0.00              | 9.00             | 5                | 8.000           | 11.167           | P                |

† 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>
<td>X</td>
<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>
<td>MAD-40-0745</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>15</td>
<td>MOT-75-0776</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>-</td>
<td>X</td>
</tr>
<tr>
<td>17</td>
<td>CLI-132-0083</td>
<td>-</td>
<td>X</td>
</tr>
</tbody>
</table>

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. The description of the same is given in this section.

### 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) \( DF_{Field} \), 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.

8.2.2.1 Predictive Methods of Determining Distribution Forces

8.2.2.1.1 $DF_{STD}$ - AASHTO-STD:

AASHTO-STD Specifications have only one parameter in the distribution factor equation, that being the lateral spacing of the girders. Also, the equation is same for one lane loaded vs. two or more lanes loaded and no provision is made for skewed bridges. This equation has been seen to give a conservative estimate of the distribution factors.

For $S \leq 14’$

$$DF_{STD,M1,INT} = DF_{STD,M2,INT} = \frac{S}{11} \quad (8-2)$$

For $S \leq 6’$

$$DF_{STD,M1,EXT} = DF_{STD,M2,EXT} = \frac{S}{11} \quad (8-3)$$

else if $6’ < S \leq 14’$
Chapter 8 – Lateral Load Distribution

\[ DF_{STD,M1,EXT} = DF_{STD,M2,EXT} = \frac{S}{2(4 + 0.25S)} \]  

(8-4)

where:

\[ S = \text{Average Girder Spacing} \]

8.2.2.1.2 \( DF_{LRFD} \) – AASHTO-LRFD

The AASHTO-LRFD specifications provide more comprehensive provisions for lateral load distribution than the AASHTO-STD specifications. Parameters such as stringer spacing \( S \), span length \( L \), and girder stiffness \( K_g \), etc. are included and a correction factor is provided for skewed structures.

\[ DF_{LRFD,M1,INT} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12Lt^3} \right)^{0.1} \]  

(8-5)

\[ DF_{LRFD,M2,INT} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12Lt^3} \right)^{0.1} \]  

(8-6)

\[ K_g = n(I + Ae^2) \]  

(8-7)

\[ DF_{LRFD,M1,EXT} \rightarrow \text{Use Lever Rule} \]  

(8-8)

\[ DF_{LRFD,M2,EXT} = e \cdot DF_{LRFD,M2,INT} \]  

(8-9)

\[ e = 0.77 + \frac{d_e}{9.1} \]  

(8-10)

The distribution factor for an exterior girder should not be less than that shown in Equation (8-11), which is based on a rigid body rotation of the bridge.

\[ DF_{LRFD,EXT,M1} = \frac{N_e}{N_b} + \frac{X_{Ext} \sum_{i=1}^{N_e} e}{\sum_{i=1}^{N_b} x^2} \]  

(8-11)
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For a skewed bridge, a reduction factor may be applied to the distribution factor values.

\[ DF'_{LRFD} = \left(1 - C_1 \left(\tan \theta\right)^{0.5}\right) DF_{LRFD} \quad (8-12) \]

\[ C_1 = 0.25 \left(\frac{K_g}{12Lt_d^2}\right)^{0.25} \left(\frac{S}{L}\right)^{0.5} \quad (8-13) \]

When the skew angle, \( \theta \), in Equations (8-12) and (8-13) is less than 30°, \( C_1 \) shall be taken as zero and when the skew angle is greater than 60°, \( \theta \) shall be taken as 60°.

The parameters in Equations (8-5) through (8-13) are defined as:
- \( S \) = Girder Spacing \( \quad \text{valid over the range of} \quad 3.5' \leq S \leq 16' \)
- \( L \) = Span Length \( \quad 20' \leq L \leq 240' \)
- \( K_g \) = Longitudinal Stiffness Parameter \( \quad 10,000 \text{ in}^4 \leq K_g \leq 7,000,000 \text{ in}^4 \)
- \( n \) = Modular Ration of Steel to Concrete
- \( I \) = Moment of Inertia of the Steel Girder
- \( e_g \) = Distance from the CG of the Steel Girder to the CG of the Concrete Deck
- \( t_d \) = Thickness of the Concrete Deck
- \( \theta \) = Skew Angle (deg)
- \( d_c \) = Distance between the exterior web and the inner face of the curb, taken as positive if the web is inboard of the curb and negative if it is outboard of the curb
- \( N_L \) = Number of loaded lanes under consideration
- \( N_b \) = Number of beams or girders
- \( x \) = Horizontal distance from the center of gravity of the bridge to each girder
- \( X_{ext} \) = Horizontal distance from the center of gravity of the bridge to the exterior girder
- \( e \) = Eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders

8.2.2.1.3 Example Calculations

To illustrate the application of these equations, distribution factors will be computed for a 3 span, 4 girder bridge (CLE 52-0142). A cross section of the bridge is shown in Figure 8-1.
Parameters of the CLE-52-0142 are given here as:

- \( L = 45', 55', \) and \( 45' \)
- Girder type = W30 X 108, \( A = 31.70 \text{ in}^2, I = 4470 \text{ in}^4, d = 29.8" \)
- \( S = 9.167" \)
- \( t_d = 9.5" \)
- \( n = 8 \) (per AASHTO for 4 ksi concrete)

The distribution factors as stipulated in the AASHTO-STD specification are computed as is shown in Equations (8-14) and (8-15).

\[
DF_{STD, M1, INT} = DF_{STD, M2, INT} = \frac{S}{11} = \frac{9.167}{11} = 0.833 \\
DF_{STD, M1, EXT} = DF_{STD, M2, EXT} = \frac{S}{2(4 + 0.25S)} = \frac{9.167}{2[4 + (0.25)(9.167)]} = 0.728
\] (8-14) (8-15)

The distribution factors for interior girders as stipulated in the AASHTO-LRFD specification are computed as is shown in Equations (8-16) through (8-19).

\[
e_g = \frac{d}{2} + \frac{t_d}{2} = \frac{29.8"}{2} + \frac{9.5"}{2} = 19.15"
\] (8-16)

\[
K_g = n\left( I + Ae_g^2 \right) = (8)\left( 4,470 \text{ in}^4 + (31.70 \text{ in}^2)(19.15")^2 \right) = 261,400 \text{ in}^4
\] (8-17)
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\[
DF_{LRFD,M1,INT} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12Lt_y^2} \right)^{0.1}
\]

\[
= 0.06 + \left( \frac{9.167'}{14} \right)^{0.4} \left( \frac{9.167'}{55'} \right)^{0.3} \left( \frac{261,400 \text{ in}^4}{(12)(55')(9.5'')^3} \right)^{0.1} = 0.486
\]  

(8-18)

\[
DF_{LRFD,M2,INT} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12Lt_y^2} \right)^{0.1}
\]

\[
= 0.075 + \left( \frac{9.167'}{9.5} \right)^{0.6} \left( \frac{9.167'}{55'} \right)^{0.2} \left( \frac{261,400 \text{ in}^4}{(12)(55')(9.5'')^3} \right)^{0.1} = 0.665
\]  

(8-19)

The distribution factor for the exterior girders when one lane is loaded is determined using the level rule. Using the lever rule, the reaction in the exterior girder is computed to give the distribution factor by assuming hinges in the deck at all interior girders. For the placement shown in Figure 8-2, \( R_A = 0.70P \). Hence \( DF_{LRFD,M1,EXT} \) is \( 0.70 \times 1.2 = 0.840 \). The number 1.2 used is the multiple-presence factor defined in Chapter 3 of the AASHTO-LRFD Specification.

![Figure 8-2: Distribution Factor DF<sub>M1,EXT</sub> by Using the Lever Rule](image)

The distribution factor for the exterior girders when two or more lanes loaded is computed as is shown in Equations (8-20) and (8-21).

\[ e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{2.5'}{9.1} = 1.04 \]  

(8-20)

\[
DF_{LRFD,M2,EXT} = e \cdot DF_{LRFD,M2,INT} = (1.04)(0.665) = 0.680
\]  

(8-21)
A minimum value for the exterior girder distribution factor is given in Equation (8-11). This is computed for the sample bridge in Equations (8-22) and (8-23).

For the case of one lane loaded,

\[ R = \frac{1}{4} + \frac{13.75(15.33')}{(18.33^2 + 4.58^2 + 4.58^2 + 18.33^2)} = 0.545 \]  

\[ (8-22) \]

Applying the multiple presence factor, \( R = (1.2)(0.545) = 0.654 < 0.840 \), hence the value obtained form the lever rule will govern and \( DFLFD,M1,EXT = 0.840 \).

For the case of two or more lanes loaded,

\[ R = \frac{2}{4} + \frac{13.75(15.33'+3.33')}{(18.33'^2 + 4.58'^2 + 4.58'^2 + 18.33'^2)} = 0.859 \]  

\[ (8-23) \]

Applying the multiple presence factor, \( R = (1.00)(0.859) = 0.859 > 0.680 \), hence \( DFLFD,M2,EXT = 0.859 \).

8.2.2.2 Analytical Methods of Determining Distribution Factors

8.2.2.2.1 \( DF_{\text{Nom}} \) – Nominal FE Model Results:

The nominal three-dimensional finite element model is developed from the information in the construction plans using SAP2000. A typical finite element model is been described in Chapter 6. All the bridges are modeled to represent a partially composite behavior.

Steps for Calculating a Distribution Factor using the 3D FE model:

1) Find the number of lanes from the database. All the bridges in this study are 2 lane bridges, except CUY-77 which is a three lane bridge. The lane location and width is used as marked on the bridge. This is inline with the guidelines given by ODOT to use the lanes as marked for rating/evaluation purposes.

2) The axle load of 1 kip (0.5 kip each wheel) is loaded at the center of each lane at mid span of the longest span. The load is only applied at the center of each lane.
3) The stresses at bottom flange for each girder is noted from the SAP output. The flanges are modeled using a 2 node frame element hence the axial force in the frame element for bottom flange located at the mid span is divided by the area of the frame element representing bottom flange, $A$ to obtain the stress in the bottom flanges.

4) The distribution factor is calculated for one lane loaded and for two or more lanes loaded by using the following equations:

$DF_{\text{Nom,1,}i} = \frac{\sigma_{(i)}}{\left\{ \frac{1}{n} \sum_{i=1}^{g} \sigma_{(i)} \right\}}$ \hspace{1cm} (8-24)

$DF_{\text{Nom,2,}i} = \frac{\sigma_{(i,1)} + \sigma_{(i,2)} + \ldots + \sigma_{(i,n)}}{\left\{ \frac{1}{n} \sum_{i=1,j=1}^{g,n} \sigma_{(i,j)} \right\}}$ \hspace{1cm} (8-25)

5) The maximum DF value from the set of values for all the girders is selected.

All bridges selected in this study had girders with equal depth across the cross section of the bridge except for BUT-732-1043. The equations for the distribution factors of this bridge must include the section moduli of the individual girders as a weighting factor along with the corresponding girder strain. Thus for BUT-732-1043, Equations (8-24) and (8-25) are rewritten as is shown in Equations (8-26) and (8-27), respectively.

$DF_{\text{Nom,1,}i} = \frac{\omega_{(i)} \sigma_{(i)}}{\left\{ \frac{1}{n} \sum_{i=1}^{g} \omega_{(i)} \sigma_{(i)} \right\}}$ \hspace{1cm} (8-26)

$DF_{\text{Nom,2,}i} = \frac{\omega_{(i)} [\sigma_{(i,1)} + \sigma_{(i,2)} + \ldots + \sigma_{(i,n)}]}{\left\{ \frac{1}{n} \sum_{i=1,j=1}^{g,n} \omega_{(i,j)} \sigma_{(i,j)} \right\}}$ \hspace{1cm} (8-27)

where:

$DF_{\text{Nom,1,}i} =$ Distribution factor due to one lane loaded for girder ‘i’ for internal and external girders
\( \text{DF}_{\text{Nom}, 2,i} \) = Distribution factor due to two or more lanes loaded for girder ‘i’ for internal and external girders

\( \sigma_{(i, j)} \) = Stress at girder i resulting from a truck placed in lane j

\( n \) = Number of lanes

\( g \) = Number of girders

To demonstrate the above steps for calculating a distribution factor, sample calculations for the CLE-52-0142 bridge are shown in Table 8-3. Case 1 indicates that lane 1 is loaded, whereas case 2 indicates that lane 2 is loaded. From Table 8-3, it can be seen that \( \text{DF}_{\text{Nom}, M1, \text{INT}} = 0.475 \), \( \text{DF}_{\text{Nom}, M2, \text{INT}} = 0.666 \), \( \text{DF}_{\text{Nom}, M1, \text{EXT}} = 0.298 \), and \( \text{DF}_{\text{Nom}, M2, \text{EXT}} = 0.334 \).

<table>
<thead>
<tr>
<th>Girder</th>
<th>Bottom Flange stress (ksi)</th>
<th>DF</th>
<th>DF</th>
<th>One lane DF</th>
<th>Two lane DF</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case 1</td>
<td>Case 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G1</td>
<td>0.0313</td>
<td>0.0037</td>
<td>0.2985</td>
<td>0.0357</td>
<td><strong>0.298</strong></td>
</tr>
<tr>
<td>G2</td>
<td>0.0498</td>
<td>0.0200</td>
<td>0.4749</td>
<td>0.1909</td>
<td><strong>0.475</strong></td>
</tr>
<tr>
<td>G3</td>
<td>0.0200</td>
<td>0.0498</td>
<td>0.1909</td>
<td>0.4749</td>
<td>0.475</td>
</tr>
<tr>
<td>G4</td>
<td>0.0037</td>
<td>0.0313</td>
<td>0.0357</td>
<td>0.2985</td>
<td>0.298</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.000</td>
<td></td>
</tr>
</tbody>
</table>

Here case 1 is the left lane loaded, while case 2 is the right lane loaded.

### 8.2.2.2.2 \( \text{DF}_{\text{Tuned}} \) – Calibrated FE Model Results

The computation of distribution factor from the tuned model is done by using the process mentioned in Section 8.2.2.2.1. \( \text{DF}_{\text{Tuned}} \) will have the same equations as for \( \text{DF}_{\text{Nom}} \). Sample calculations for the DF of a tuned model of CLE-52-0142 are given in Table 8-4. From the table, it is observed that \( \text{DF}_{\text{Tuned}, M1, \text{INT}} = 0.482 \), \( \text{DF}_{\text{Tuned}, M2, \text{INT}} = 0.676 \), \( \text{DF}_{\text{Tuned}, M1, \text{EXT}} = 0.302 \), and \( \text{DF}_{\text{Tuned}, M2, \text{EXT}} = 0.337 \).
8.2.2.3 Measured Distribution Factors

8.2.2.3.1 $DF_{\text{Field}}$ – Based on Field Test Results:

The distribution factor can also be calculated from the results of a truckload test. In a truckload test, trucks of known weight and axle dimensions are driven over a bridge with sensors are placed in strategic position, typically on the top of bottom flange and bottom of top flange. Ideally, sensors are placed on girders across the cross-section of the bridge at various places along the length.

However, a truckload test can be a lengthy process and a lot of time is taken in the placement of sensors itself. Also, owing to the geographic conditions under a bridge, placement of sensors may be limited. The Data Acquisition System (DAQ) has its limitations in number of channels, hence an upper bound to number of sensors that can be mounted on the bridge is fixed. The limitation of DAQ will be a hindrance in case of a bridge with large number of spans or a bridge with many girders. Hence, there were two conditions of sensor placement in a truckload:

1) The strain gages were mounted laterally across all the girders at the cross section where the DF is critical. Equation (8-28) can be easily employed to compute DF.

$$DF_{\text{Field}} = \frac{\varepsilon \omega_j}{\sum_{j=1}^{k} \varepsilon \omega_j}$$  \hspace{1cm} (8-28)

2) The strain gages are not mounted across all the girders laterally. In this case the DF can be alternatively computed by using a scaling factor. Hence, Equation (8-29) can be used
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\[
DF'_{\text{Field},i} = \left( \frac{1}{(m)(n)} \right) \sum_{i=1, j=1}^{m,n} \left( \frac{\varepsilon_{\text{Field},i,j}}{\varepsilon_{\text{Nom},i,j}} \right) (DF_{\text{Nom},i})
\]  
(8-29)

where:
- \( \omega_i \) = Section modulus of girder \( i \)
- \( DF_{\text{Field},i} \) = Experimental DF for girder \( i \)
- \( DF'_{\text{Field},i} \) = Experimental DF for girder \( i \) using incomplete data
- \( DF_{\text{Nom},i} \) = DF from nominal FE model for girder \( i \)
- \( m \) = Number of instrumented girders
- \( n \) = Number of lanes loaded
- \( \varepsilon_{\text{Field},i,j} \) = strain in girder \( i \) due to load in lane \( j \)
- \( g \) = Number of girders

### 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 8-3 and 8-4, 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.
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The distribution factors calculated using the various methods, are shown in Tables 8-5 through 8-8. 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 8-5: Distribution Factor $DF_{M1,INT}$ from Various Methods

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>DFM$_{M1,INT}$</th>
<th>ASD</th>
<th>LRFD</th>
<th>Field</th>
<th>Nominal</th>
<th>Tuned</th>
</tr>
</thead>
<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>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>0.761</td>
<td>0.431</td>
<td>0.367</td>
<td>0.372</td>
<td>0.350</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>0.682</td>
<td>0.415</td>
<td>0.306</td>
<td>0.319</td>
<td>0.305</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>0.864</td>
<td>0.505</td>
<td>0.408</td>
<td>0.408</td>
<td>0.400</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>0.773</td>
<td>0.445</td>
<td>0.360</td>
<td>0.369</td>
<td>0.366</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>0.773</td>
<td>0.443</td>
<td>0.347</td>
<td>0.348</td>
<td>0.358</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>0.788</td>
<td>0.488</td>
<td>0.481</td>
<td>0.463</td>
<td>0.458</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>0.765</td>
<td>0.473</td>
<td>0.394</td>
<td>0.418</td>
<td>0.365</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>0.833</td>
<td>0.486</td>
<td>0.442</td>
<td>0.475</td>
<td>0.482</td>
</tr>
<tr>
<td>10</td>
<td>PRE-503-1170</td>
<td>0.712</td>
<td>0.431</td>
<td>0.389</td>
<td>0.289</td>
<td>0.300</td>
</tr>
<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>-</td>
<td>-</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>0.773</td>
<td>0.458</td>
<td>-</td>
<td>0.390</td>
<td>0.384</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>

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

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>DFM$_{M2,INT}$</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>-</td>
<td>-</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>0.773</td>
<td>0.636</td>
<td>-</td>
<td>0.570</td>
<td>0.566</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>
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 are only of academic relevance to this study and will not be focused on in the remainder of this chapter. The distribution factor values earlier computed are plotted in Figures 8-5 and 8-6.

![Figure 8-5: Comparison of Distribution Factors $DF_{M1,INT}$](image)

![Figure 8-6: Comparison of Distribution Factors $DF_{M2,INT}$](image)
Tables 8-9 and 8-10 show comparisons of the DF from refined sources. The term “percent variance” (%Var) indicates the percentage difference between the maximum and minimum DF value obtained by the refined sources. Thus for BUT-732-1043, for example, the % variance is,

\[
\% \text{Var} = \frac{(0.385 - 0.369)}{0.385} \times 100 = 4.15\%
\]  

(8-30)

The significance of this term is to indicate the compactness of the band formed by the DF values by refined methods. A bridge with %Var closer to 0 indicates that the bridge has similar DF values using all three refined sources. It is observed that this term is smaller for bridges small skew or no skew.

Table 8-9: $DF_{M_{INT}}$ from Refined Methods

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>DFM$_{INT}$ Field</th>
<th>FE</th>
<th>Tuned</th>
<th>% var.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>0.369</td>
<td>0.385</td>
<td>0.375</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>0.367</td>
<td>0.372</td>
<td>0.350</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>0.306</td>
<td>0.319</td>
<td>0.305</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>0.408</td>
<td>0.408</td>
<td>0.400</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>0.360</td>
<td>0.369</td>
<td>0.366</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>0.347</td>
<td>0.348</td>
<td>0.358</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>0.481</td>
<td>0.463</td>
<td>0.458</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>0.394</td>
<td>0.418</td>
<td>0.365</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>0.442</td>
<td>0.475</td>
<td>0.482</td>
</tr>
<tr>
<td>10</td>
<td>PRE-503-1170</td>
<td>0.389</td>
<td>0.289</td>
<td>0.300</td>
</tr>
<tr>
<td>11</td>
<td>FAI-33-1309</td>
<td>0.320</td>
<td>0.256</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>MOT-70-0553</td>
<td>0.422</td>
<td>0.445</td>
<td>0.405</td>
</tr>
<tr>
<td>13</td>
<td>AUG-75-0201</td>
<td>0.448</td>
<td>0.442</td>
<td>0.448</td>
</tr>
<tr>
<td>14</td>
<td>MAD-40-0745</td>
<td>0.336</td>
<td>0.304</td>
<td>0.340</td>
</tr>
<tr>
<td>15</td>
<td>MOT-75-0776</td>
<td>0.446</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>-</td>
<td>0.390</td>
<td>0.384</td>
</tr>
<tr>
<td>17</td>
<td>CLI-132-0083</td>
<td>-</td>
<td>0.402</td>
<td>0.404</td>
</tr>
</tbody>
</table>
### Table 8-10: $DF_{M2, INT}$ from Refined Methods

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>DFM2, INT</th>
<th>Field</th>
<th>FE</th>
<th>Tuned</th>
<th>% var.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>0.718</td>
<td>0.637</td>
<td>0.742</td>
<td>14%</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>0.525</td>
<td>0.526</td>
<td>0.517</td>
<td>2%</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>0.504</td>
<td>0.474</td>
<td>0.467</td>
<td>7%</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>0.546</td>
<td>0.629</td>
<td>0.604</td>
<td>13%</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>0.573</td>
<td>0.584</td>
<td>0.573</td>
<td>2%</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>0.529</td>
<td>0.528</td>
<td>0.539</td>
<td>2%</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>0.750</td>
<td>0.717</td>
<td>0.733</td>
<td>4%</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>0.659</td>
<td>0.668</td>
<td>0.696</td>
<td>5%</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>0.639</td>
<td>0.666</td>
<td>0.684</td>
<td>7%</td>
</tr>
<tr>
<td>10</td>
<td>PRE-503-1170</td>
<td>0.559</td>
<td>0.492</td>
<td>0.487</td>
<td>13%</td>
</tr>
<tr>
<td>11</td>
<td>FAI-33-1309</td>
<td>0.310</td>
<td>0.493</td>
<td>-</td>
<td>37%</td>
</tr>
<tr>
<td>12</td>
<td>MOT-70-0553</td>
<td>0.699</td>
<td>0.685</td>
<td>0.656</td>
<td>6%</td>
</tr>
<tr>
<td>13</td>
<td>AUG-75-0201</td>
<td>0.643</td>
<td>0.627</td>
<td>0.643</td>
<td>3%</td>
</tr>
<tr>
<td>14</td>
<td>MAD-40-0745</td>
<td>0.412</td>
<td>0.555</td>
<td>0.424</td>
<td>26%</td>
</tr>
<tr>
<td>15</td>
<td>MOT-75-0776</td>
<td>0.465</td>
<td>-</td>
<td>-</td>
<td>0%</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>-</td>
<td>0.570</td>
<td>0.566</td>
<td>1%</td>
</tr>
<tr>
<td>17</td>
<td>CLI-132-0083</td>
<td>-</td>
<td>0.223</td>
<td>0.234</td>
<td>5%</td>
</tr>
</tbody>
</table>

The refined values reported in Tables 8-9 and 8-10 are plotted for each bridge in Figures 8-7 and 8-8, respectively. The numbers 1 to 17 on the x-axis indicate the bridge names as listed in the tables.

![Figure 8-7: $DF_{M1, INT}$ from Refined Methods](image-url)
8.2.4 “Real” Distribution Factor

The term “Real” Distribution Factor ($DF_{REAL}$) 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). From Figures 8-7 and 8-8, 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 8-11 shows the “Real “DF values for interior girders with one lane loaded and with two or more lanes loaded (i.e. $DF_{Real,M1,INT}$ and $DF_{Real,M2,INT}$, respectively).
Table 8-11: “Real” DF Values for Interior Girders

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>DFM \textsubscript{2,INT}</th>
<th>Real DF M\textsubscript{1,INT}</th>
<th>Real DF M\textsubscript{2,INT}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 BUT</td>
<td>732-1043</td>
<td>0.377</td>
<td>0.699</td>
</tr>
<tr>
<td>2 PRE-</td>
<td>725-0880</td>
<td>0.363</td>
<td>0.523</td>
</tr>
<tr>
<td>3 CLE-</td>
<td>52-0498L</td>
<td>0.310</td>
<td>0.482</td>
</tr>
<tr>
<td>4 HAM-</td>
<td>27-1550L</td>
<td>0.405</td>
<td>0.593</td>
</tr>
<tr>
<td>5 RIC-</td>
<td>30-1384</td>
<td>0.365</td>
<td>0.577</td>
</tr>
<tr>
<td>6 RIC-</td>
<td>30-1638</td>
<td>0.351</td>
<td>0.532</td>
</tr>
<tr>
<td>7 LIC-</td>
<td>158-0164</td>
<td>0.467</td>
<td>0.733</td>
</tr>
<tr>
<td>8 CUY-</td>
<td>77-0645L</td>
<td>0.392</td>
<td>0.674</td>
</tr>
<tr>
<td>9 CLE-</td>
<td>52-0142</td>
<td>0.466</td>
<td>0.663</td>
</tr>
<tr>
<td>10 PRE-</td>
<td>503-1170</td>
<td>0.326</td>
<td>0.513</td>
</tr>
<tr>
<td>11 FAI-</td>
<td>33-1309</td>
<td>0.288</td>
<td>0.402</td>
</tr>
<tr>
<td>12 MOT-</td>
<td>70-0553</td>
<td>0.424</td>
<td>0.680</td>
</tr>
<tr>
<td>13 AUG-</td>
<td>75-0201</td>
<td>0.446</td>
<td>0.638</td>
</tr>
<tr>
<td>14 MAD-</td>
<td>40-0745</td>
<td>0.327</td>
<td>0.463</td>
</tr>
<tr>
<td>15 MOT-</td>
<td>75-0776</td>
<td>0.446</td>
<td>0.465</td>
</tr>
<tr>
<td>16 RIC-</td>
<td>30-1438</td>
<td>0.387</td>
<td>0.568</td>
</tr>
<tr>
<td>17 CLI-</td>
<td>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. \( \% \Delta \text{LRFD} \) is defined as the percentage difference between the “real” DF and the DF computed as in the AASHTO-LRFD Specification. Hence \( \% \Delta \text{LRFD} \) can be computed as

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

Further, \( \Delta \text{LRFD,1} \) and \( \Delta \text{LRFD,2} \) are used to indicate the percentage differences on a one lane loaded and multiple lanes loaded case.

\( \% \Delta \text{STD} \) is defined as the percentage difference between the real and DF computed as in the AASHTO-ASD Specification. Hence \( \% \Delta \text{ASD} \) can be computed as

\[
\% \Delta \text{STD} = \left( \frac{DF_{\text{STD}} - DF_{\text{Real}}}{DF_{\text{STD}}} \right)
\]  

Further, \( \Delta \text{STD,1} \) and \( \Delta \text{STD,2} \) are used to indicate the percentage differences on a one lane loaded and multiple lanes loaded. The \( \% \Delta \text{LRFD} \) and \( \% \Delta \text{STD} \) values have been tabulated in Table 8-12.
Table 8-12: \( \Delta_{LRFD} \) and \( \Delta_{STD} \) Values

<table>
<thead>
<tr>
<th>Sr. No</th>
<th>Bridge</th>
<th>( \text{Real DFM}_{1,\text{INT}} )</th>
<th>( % \Delta_{LRFD,1} )</th>
<th>( % \Delta_{ASD,1} )</th>
<th>( \text{Real DFM}_{2,\text{INT}} )</th>
<th>( % \Delta_{LRFD,2} )</th>
<th>( % \Delta_{ASD,2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT-732-1043</td>
<td>0.377</td>
<td>17%</td>
<td>51%</td>
<td>0.699</td>
<td>-11%</td>
<td>8%</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>0.363</td>
<td>16%</td>
<td>52%</td>
<td>0.523</td>
<td>14%</td>
<td>31%</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>0.310</td>
<td>25%</td>
<td>55%</td>
<td>0.482</td>
<td>17%</td>
<td>29%</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>0.405</td>
<td>20%</td>
<td>53%</td>
<td>0.593</td>
<td>16%</td>
<td>31%</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>0.365</td>
<td>18%</td>
<td>53%</td>
<td>0.577</td>
<td>8%</td>
<td>25%</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>0.351</td>
<td>21%</td>
<td>55%</td>
<td>0.532</td>
<td>15%</td>
<td>31%</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>0.467</td>
<td>4%</td>
<td>41%</td>
<td>0.733</td>
<td>-9%</td>
<td>7%</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>0.392</td>
<td>17%</td>
<td>49%</td>
<td>0.674</td>
<td>-4%</td>
<td>12%</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>0.466</td>
<td>4%</td>
<td>44%</td>
<td>0.663</td>
<td>0%</td>
<td>20%</td>
</tr>
<tr>
<td>10</td>
<td>PRE-503-1170</td>
<td>0.326</td>
<td>24%</td>
<td>54%</td>
<td>0.513</td>
<td>8%</td>
<td>28%</td>
</tr>
<tr>
<td>11</td>
<td>FAI-33-1309</td>
<td>0.288</td>
<td>21%</td>
<td>60%</td>
<td>0.402</td>
<td>25%</td>
<td>45%</td>
</tr>
<tr>
<td>12</td>
<td>MOT-70-0553</td>
<td>0.424</td>
<td>4%</td>
<td>42%</td>
<td>0.680</td>
<td>-12%</td>
<td>7%</td>
</tr>
<tr>
<td>13</td>
<td>AUG-75-0201</td>
<td>0.446</td>
<td>-5%</td>
<td>39%</td>
<td>0.638</td>
<td>-9%</td>
<td>12%</td>
</tr>
<tr>
<td>14</td>
<td>MAD-40-0745</td>
<td>0.327</td>
<td>2%</td>
<td>29%</td>
<td>0.463</td>
<td>-19%</td>
<td>0%</td>
</tr>
<tr>
<td>15</td>
<td>MOT-75-0776</td>
<td>0.446</td>
<td>-1%</td>
<td>39%</td>
<td>0.465</td>
<td>24%</td>
<td>36%</td>
</tr>
<tr>
<td>16</td>
<td>RIC-30-1438</td>
<td>0.387</td>
<td>15%</td>
<td>50%</td>
<td>0.568</td>
<td>11%</td>
<td>26%</td>
</tr>
<tr>
<td>17</td>
<td>CLI-132-0083</td>
<td>0.403</td>
<td>8%</td>
<td>45%</td>
<td>0.574</td>
<td>3%</td>
<td>21%</td>
</tr>
</tbody>
</table>

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 Appendix A. 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 \( DF_{STD,M2,INT} \)

The DF for internal girders with one lane loaded for various bridges has been reported in Table 8-5. The \( \Delta_{LRFD,1} \) and \( \Delta_{STD,1} \) was also reported in Table 8-9. These parameters were correlated with several design parameter combinations. The details of correlation and derivation of modification factors are being given here. Skew bridges (bridges with skew greater than 15°)
were not considered in this study. It was observed that with removal of skew bridges, the correlation of $\%\Delta_{LRFD}$ improved dramatically.

It was generally observed that the equations contained within AASHTO-LRFD Specification yielded distribution factors that were much closer to the “real” distribution factors than equations contained within the AASTHO-STD Specification. As a result, use of the DF equations contained within the AASTHO-LRFD Specification along with the modification factors derived in the sections that follow. Modification factors are provided for use with the AASHTO-STD Specification for completeness but use of the AASHTO-LRFD approach is preferred.

### 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 $\%\Delta_{LRFD}$. Figures 8-9 through 8-11 show the regression charts for $\%\Delta_{LRFD}$ with the above-mentioned parameters.

Figure 8-9: Correlation of $S/$ *Width* vs. $\%\Delta_{LRFD}$
The difference $\%\Delta_{LRFD}$ can be correlated to the terms $S/\text{Width}$ and $1/n$ to develop the modification factor. Thus, these terms can be taken in consideration for calculation of a modification factor. First, the correction factor $MF_{LRFD,1,INT}$ for correcting $DF_{LRFD,M1,INT}$ will be derived.
A modification factor for $DF_{LRFD,M1,INT}$ can thus be derived using the equation in Figure 8-10, and the equation for $\%\Delta_{LRFD}$. If $\%\Delta_{LRFD}$ is directly taken as the equation in the regression chart shown in Figure 8-10, the following equation is obtained.

\[
\therefore DF_{LRFD,M1,INT} \left[-4.4659 \left( \frac{S}{(n)(Width)} \right) + 0.3288 \right] = DF_{LRFD,M1,INT} - DF_{Real} \quad (8-33)
\]

\[
\therefore DF_{LRFD,M1,INT} \left[1 - \left(-4.4659 \left( \frac{S}{(n)(Width)} \right) + 0.3288 \right) \right] = DF_{Real} \quad (8-34)
\]

Hence, the modification factor can be written as:

\[
\therefore MF_{LRFD,M1,INT} = \left[1 - \left(-4.4659 \left( \frac{S}{(n)(Width)} \right) + 0.3288 \right) \right] \quad (8-35)
\]

Equation (8-35) can also be written as,

\[
\therefore MF_{LRFD,M1,INT} = \left[1 + 4.4659 \left( \frac{S}{(n)(Width)} \right) - 0.3288 \right] \quad (8-36)
\]

\[
\therefore MF_{LRFD,M1,INT} = \left[(1 - 0.3288) + 4.4659 \left( \frac{S}{(n)(Width)} \right) \right] \quad (8-37)
\]

\[
\therefore MF_{LRFD,M1,INT} = \left[\left(\frac{2}{3}\right) + 4.5 \left( \frac{S}{(n)(Width)} \right) \right] \quad (8-38)
\]
### Table 8-13: Modification Factor $MF_{LRFD,M1,INT}$ for $DF_{LRFD,M1,INT}$

<table>
<thead>
<tr>
<th>Sr. No</th>
<th>Bridge Name S/ (n.Width)</th>
<th>CF</th>
<th>$DF_{LRFD}$</th>
<th>$DF_{Derived, 1}$</th>
<th>$DF_{Real}$</th>
<th>$% \delta_{LRFD}$</th>
<th>$% \Delta_{LRFD}$</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>
</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>
</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>
</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>
</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>
</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>
</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>
</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>
</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>
</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>
</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>
</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>
</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>
</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>
</tr>
</tbody>
</table>

$\% \delta_{LRFD}$ is the difference between the $DF_{Real}$ and $DF_{Derived}$.

As shown in Table 8-13, the difference between the $DF_{LRFD,M1,INT}$ (modified to $DF_{LRFD,Derived}$ by multiplying by $MF_{LRFD,M1,INT}$) 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 in Figure 8-12. The numbers 1 through 14 indicate the corresponding bridge names in Table 8-13. The term $\% \delta_{LRFD}$ is defined as,

\[
\% \delta_{LRFD} = \left( \frac{DF_{Real} - DF_{LRFD,Derived}}{DF_{Real}} \right)
\]  

(8-39)
Figure 8-12: Comparison of $DF_{Derived}$ and $DF_{Real}$ for an Interior Girder with One Lane Loaded

Figure 8-13: Percentage Difference $\Delta_{LRFD}$ and $\delta_{LRFD}$

Figure 8-12 shows the reduction in the $DF_{LRFD}$ after applying the modification factor. Figure 8-13 shows the $\%\Delta_{LRFD}$ and $\%\delta_{LRFD}$ for each of the 14 bridges. The $\%\delta_{LRFD}$ value of each bridge is evidently lowered. Thus the modification factor developed earlier in Equation (8-38) is useful in reducing the conservatism involved in the $DF_{LRFD}$ for an interior girder with one lane loaded. Further, implementing the parameters $(1/n, S/Width$ and $S/(n \times Width))$ individually yields different results than implementing them simultaneously in the modification factor. This is illustrated in Figure 8-14 and Table 8-14.
Table 8-14: Comparison of Values of %d^3 Using Various Parameters

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Bridge Name</th>
<th>%d^2</th>
<th>%δ_{LRFD} (S/Width)</th>
<th>%δ_{LRFD} (1/n)</th>
<th>%δ_{LRFD} S/(Width. n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>17%</td>
<td>7%</td>
<td>5%</td>
<td>5%</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>16%</td>
<td>4%</td>
<td>4%</td>
<td>3%</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>25%</td>
<td>9%</td>
<td>8%</td>
<td>8%</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>20%</td>
<td>11%</td>
<td>8%</td>
<td>9%</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>18%</td>
<td>-1%</td>
<td>-1%</td>
<td>-1%</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>21%</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>4%</td>
<td>2%</td>
<td>2%</td>
<td>3%</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>17%</td>
<td>-8%</td>
<td>-9%</td>
<td>-7%</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>4%</td>
<td>3%</td>
<td>2%</td>
<td>3%</td>
</tr>
<tr>
<td>10</td>
<td>MOT-70-0553</td>
<td>4%</td>
<td>-1%</td>
<td>1%</td>
<td>0%</td>
</tr>
<tr>
<td>11</td>
<td>AUG-75-0201</td>
<td>-5%</td>
<td>-10%</td>
<td>-7%</td>
<td>-8%</td>
</tr>
<tr>
<td>12</td>
<td>MOT-75-0776</td>
<td>-1%</td>
<td>-8%</td>
<td>-4%</td>
<td>-5%</td>
</tr>
<tr>
<td>13</td>
<td>RIC-30-1438</td>
<td>15%</td>
<td>-4%</td>
<td>-5%</td>
<td>-5%</td>
</tr>
<tr>
<td>14</td>
<td>CLI-132-0083</td>
<td>8%</td>
<td>-6%</td>
<td>-5%</td>
<td>-6%</td>
</tr>
</tbody>
</table>

Figure 8-14: %δ_{LRFD} Using Various Sources
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The correlation between the $DF_{\text{Real}}$ and the $DF_{\text{LRFD,Derived}}$ is much better than for $DF_{\text{Real}}$ and $DF_{\text{LRFD}}$.

### 8.3.2 Modification Factor for AASHTO-STD with One Lane Loaded

The $DF_{\text{STD,M1,INT}}$ computed using equations within the AASHTO-STD Specification are shown in Table 8-5. In general, $\%\Delta_{\text{STD}}$ is considerably larger than the $\%\Delta_{\text{LRFD}}$. As a result, use of the AASHTO-LRFD approach in concert with $MF_{\text{LRFD,M1,INT}}$ and $MF_{\text{LRFD,M2,INT}}$ 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.

Figure 8-17: Correlation of $S/\text{Width}$ vs. $\%\Delta_{ASD}$

Figure 8-18: Correlation of $1/n$ vs. $\%\Delta_{STD,i}$
The regression equation from Figure 8-19 is used to calculate the modification factor $MF_{STD,M1,INT}$ for use with $DF_{STD,M1,INT}$. If $\%\Delta_{STD}$ is directly taken as the equation in the regression chart shown in Figure 8-19, the following is obtained,

$$
\therefore \quad DF_{STD,M1,INT} \left( -2.7759 \left( \frac{S}{(n)(Width)} \right) + 0.6073 \right) = DF_{STD,M1,INT} - DF_{Real} \quad (8-40)
$$

$$
\therefore \quad DF_{STD,M1,INT} \left( 1 - \left(-2.7759 \left( \frac{S}{(n)(Width)} \right) + 0.6073 \right) \right) = DF_{Real} \quad (8-41)
$$

Hence, the modification factor can be given as:

$$
\therefore \quad MF_{STD,M1,INT} = \left( 1 - \left(-2.7759 \left( \frac{S}{(n)(Width)} \right) + 0.6073 \right) \right) \quad (8-42)
$$

$$
\therefore \quad MF_{STD,M1,INT} = \left( 0.3927 + 2.7759 \left( \frac{S}{(n)(Width)} \right) \right) \quad (8-43)
$$

$$
\therefore \quad MF_{STD,M1,INT} = \left( 0.39 + 2.78 \left( \frac{S}{(n)(Width)} \right) \right) \quad (8-44)
$$
As shown in Table 8-15 that 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. The numbers 1 through 14 indicate the corresponding bridge names in Table 8-15.

![Figure 8-20: Difference With and Without the Modification Factor](image-url)
Chapter 8 – Lateral Load Distribution

Comparison of $DF_{\text{Derived}}$ and $DF_{\text{Real}}$

Table 8-16: Comparison of $\%\delta_{5}$ from Various Methods

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Bridge Name</th>
<th>$%\Delta_{\text{ASD, 1}}$</th>
<th>$%\delta_{\text{ASD, 1}}$ (S/Width)</th>
<th>$%\delta_{\text{ASD, 1}}$ (1/n)</th>
<th>$%\delta_{\text{ASD, 1}}$ S/(Width x n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BUT 732-1043</td>
<td>51%</td>
<td>7%</td>
<td>5%</td>
<td>5%</td>
</tr>
<tr>
<td>2</td>
<td>PRE-725-0880</td>
<td>52%</td>
<td>8%</td>
<td>8%</td>
<td>7%</td>
</tr>
<tr>
<td>3</td>
<td>CLE-52-0498L</td>
<td>55%</td>
<td>7%</td>
<td>5%</td>
<td>6%</td>
</tr>
<tr>
<td>4</td>
<td>HAM-27-1550L</td>
<td>53%</td>
<td>13%</td>
<td>10%</td>
<td>11%</td>
</tr>
<tr>
<td>5</td>
<td>RIC-30-1384</td>
<td>53%</td>
<td>2%</td>
<td>1%</td>
<td>1%</td>
</tr>
<tr>
<td>6</td>
<td>RIC-30-1638</td>
<td>55%</td>
<td>6%</td>
<td>5%</td>
<td>5%</td>
</tr>
<tr>
<td>7</td>
<td>LIC-158-0164</td>
<td>41%</td>
<td>-2%</td>
<td>-2%</td>
<td>-1%</td>
</tr>
<tr>
<td>8</td>
<td>CUY-77-0645L</td>
<td>49%</td>
<td>-13%</td>
<td>-14%</td>
<td>-12%</td>
</tr>
<tr>
<td>9</td>
<td>CLE-52-0142</td>
<td>44%</td>
<td>5%</td>
<td>4%</td>
<td>5%</td>
</tr>
<tr>
<td>10</td>
<td>MOT-70-0553</td>
<td>42%</td>
<td>-3%</td>
<td>0%</td>
<td>-1%</td>
</tr>
<tr>
<td>11</td>
<td>AUG-75-0201</td>
<td>39%</td>
<td>-9%</td>
<td>-5%</td>
<td>-6%</td>
</tr>
<tr>
<td>12</td>
<td>MOT-75-0776</td>
<td>39%</td>
<td>-10%</td>
<td>-5%</td>
<td>-7%</td>
</tr>
<tr>
<td>13</td>
<td>RIC-30-1438</td>
<td>50%</td>
<td>-4%</td>
<td>-5%</td>
<td>-5%</td>
</tr>
<tr>
<td>14</td>
<td>CLI-132-0083</td>
<td>45%</td>
<td>-7%</td>
<td>-6%</td>
<td>-8%</td>
</tr>
</tbody>
</table>

Figure 8-21: Comparison of $DF_{\text{Derived}}$ and $DF_{\text{Real}}$
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As is evident from Figure 8-22 and Table 8-16, there is very little difference between the \%d5 obtained using various methods. The change between the \%d4 to \%d5 is immense and will lead to a significant reduction in the margin between “real” and equation values for distribution.

Figure 8-23: Correlation of $DF_{Real}$ vs. $DF_{STD, M1, INT}$
Correlation of DF Derived vs. DF Real

\[ y = 0.8252x + 0.0694 \]
\[ R^2 = 0.6835 \]

Figure 8-24: Correlation between \(DF_{Real}\) and \(DF_{Derived}\)

The R-squared value of \(DF_{Real}\) and \(DF_{STD,Derived}\) is improved as compared to \(DF_{STD}\) vs. \(DF_{Real}\). Thus the correction factor introduced here is quite adequately reducing the “band” between the \(DF_{Real}\) and the DF values obtained form the equations.

### 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, \(DF_{M2,INT}\), for the 17 bridges in the database was reported previously in Table 8-6. A comparative chart of \(DF_{M2,INT}\) values from various methods was also presented in Figure 8-7. This comparison shows a more complex traversing of DF values from each individual source for a bridge as compared to the comparison from \(DF_{M1,INT}\). 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. Refer to Figure 5-4.

\[
RCA = \frac{P_{\text{Partial}}}{P_{\text{Full}}} \tag{8-45}
\]

RCA is a linear condition index defining the level of composite action present in a bridge. Thus a bridge with RCA of 100% indicates a bridge representing full 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-
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732-1043, bridge shows a low RCA, which is an indication of a relatively low level of composite action in the bridge. Also, the %Δ 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.

As is seen in Figure 8-25, it is evident that a quadratic trendline provides a reasonable fit to the data set when %ΔLRFD is plotted as a function of the remaining composite action. A modification factor \( MF_{ LRFD, M2, INT} \) for correcting the \( DF_{ LRFD, M2, INT} \) will now be derived based on the regression equation shown in Figure 8-25.

\[
y = 7E-05x^2 - 0.008x + 0.1288
\]

\[R^2 = 0.8883\]

![Figure 8-25: Correlation of %ΔLRFD,2 vs. RCA %](image)

A modification factor for \( DF_{ LRFD, M1, INT} \) can be thus derived using the relationship in Figure 8-25, and the equation for %ΔLRFD. If the % ΔLRFD is directly taken as the equation in the regression chart, we get the following equation

\[
\therefore \quad DF_{ LRFD, M2, INT} \left( 0.00007(RCA)^2 - 0.008(RCA) + 0.1288 \right) = DF_{ LRFD, M2, INT} - DF_{ Real} \quad (8-46)
\]

\[
\therefore \quad DF_{ LRFD, M2, INT} \left( 1 - \left( 0.00007(RCA)^2 - 0.008(RCA) + 0.1288 \right) \right) = DF_{ Real} \quad (8-47)
\]
Equation (8-47) can also be written as,

\[
\therefore \quad DF_{LRFD,M2,INT} \left( 0.8712 + 0.008(RCA) - 0.00007(RCA)^2 \right) = DF_{Real} \tag{8-48}
\]

Hence, the modification factor can be written as:

\[
\therefore \quad MF_{LRFD,M2,INT} = 0.8712 + 0.008(RCA) - 0.00007(RCA)^2 \tag{8-49}
\]

### Table 8-18: Modification Factor $MF_{LRFD,M2}$ for Interior Girders

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Bridges</th>
<th>RCA</th>
<th>$MF_{LRFD,M2}$</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 $DF_{Real}$ and $DF_{Derived}$.

As shown in Table 8-18, 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 8-26. The numbers 1 through 11 indicate the corresponding bridge names in Table 8-17.

Figure 8-26 shows the reduction in the error after applying the correction factors. Figure 8-27 shows the % differences $%\Delta_{LRFD}$ and $%\delta_{LRFD}$ for each of the 11 bridges. The $%\delta_{LRFD}$ values of bridge are evidently lowered. Thus the correction factor developed earlier in Equation (8-49) is useful in reducing the error involved in the $DF_{LRFD,M2,INT}$ for the case of multiple lanes loaded. This improvement is further supported by a comparison of the trendlines correlations in Figures 8-28 and 8-29. Note that the R-squared value is improved from 0.256 to 0.904.
Chapter 8 – Lateral Load Distribution

Figure 8-26: Comparison of $DF_{Derived}$ and $DF_{Real}$

Figure 8-27: Percentage Difference $\%\Delta_{LRFD}$ and $\%\delta_{LRFD}$ for Multiple Lanes Loaded
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 study similar to the one completed in the previous section will be presented. The remaining composite action is again found to be the parameter giving maximum correlation with the $\%\Delta_{STD}$. A correlation of RCA and $\%\Delta_{ASD}$ is shown in Figure 8-30. It is again found that a quadratic trendline shows a good fit with $\%\Delta_{ASD}$ plotted as a function of the remaining composite action.
A modification factor for $DF_{STD,M2,INT}$ can thus be derived using the equation in Figure 8-30 for $\%\Delta_{ASD}$. If the $\%\Delta_{STD}$ is directly taken as the equation in the regression chart shown in Figure 8-30, resulting in the following equation,

\[
\therefore DF_{STD,M2,INT} \left( 5 \times 10^{-5} (RCA)^2 - 0.0051 (RCA) + 0.2165 \right) = DF_{STD,M2,INT} - DF_{Real} \tag{8-50}
\]

\[
\therefore DF_{STD,M2,INT} \left( 1 - \left( 5 \times 10^{-5} (RCA)^2 - 0.0051 (RCA) + 0.2165 \right) \right) = DF_{Real} \tag{8-51}
\]

Hence, the modification factor can be written as:

\[
\therefore MF_{STD,M2,INT} = 0.7835 + 0.0051 (RCA) - 0.00005 (RCA)^2 \tag{8-52}
\]
As shown in Table 8-19 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 8-31. The numbers 1 through 11 indicate the corresponding bridge names in Table 8-19.

<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>% Δ$_{ASD}$</th>
<th>% δ$_{ASD}$</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>

%δ$_{ASD}$ is the difference between the $DF_{Derived}$ and $DF_{Real}$
Figure 8-31: Comparison of $DF_{Derived}$ and $DF_{Real}$

The improvement from $\%\Delta_{ASD}$ to $\%\delta_{ASD}$ is immense and will lead to a significant reduction in the margin between “real” and equation values of DF. The R-squared value of $DF_{Derived}$ vs. $DF_{Real}$ is improved as compared to $DF_{STD,M2,INT}$ vs. $DF_{Real}$. Thus the modification factor introduced here is quite adequately reducing the “band” between the $DF_{Real}$ and the DF values obtained from the equations. The R-squared values from Figures 8-32 and 8-33 are 0.188 and 0.813, respectively.

Figure 8-32: Correlation of $DF_{Real,M2,INT}$ vs. $DF_{ASD}$
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Figure 8-33: Correlation of $DF_{Real,M2,INT}$ vs. $DF_{Derived}$

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. The concept of a “real” distribution factor ($D_{real}$) is introduced which is simply the average of DF values obtained using refined methods. This value of DF for each bridge is the closest estimation of the actual distribution present in each bridge. Thus it is used as the comparator in calculating $\% \Delta_{LRFD}$, $\% \delta_{LRFD}$, $\% \Delta_{STD}$, and $\% \delta_{STD}$ which are used to evaluate the applicability of the proposed 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.

The modification factor for the case of one lane loaded can be presented as is shown in Equation (8-53).

$$MF = Q_1 + Q_2 \left( \frac{S}{(n)(Width)} \right)$$

(8-53)

where:

for AASHTO-LRFD, $Q_1 = 0.39$ and $Q_2 = 2.78$
for AASTH-STD, $Q_1 = 0.33$ and $Q_2 = 4.50$

The modification factors for the case of multiple lanes loaded can be presented as is shown in Equation (8-54)

$$MF = Q_1 + Q_2 \cdot RCA + Q_3 \cdot RCA^2$$

(8-54)
where:

for AASHTO-LRFD, $Q_1 = 0.871$, $Q_2 = 0.0080$, and $Q_3 = -0.00007$

for AASHTO-STD, $Q_1 = 0.784$, $Q_2 = 0.0051$, and $Q_3 = -0.00005$

The modification factors derived in this work are based on the data that are available from the completed experiments. Every effort has been made to include as wide a range of parameters as possible. It should be remembered, however, that since the modification factors proposed herein are empirical in nature, use of these relations with parameters outside the range of the dataset upon which they are based may result in erroneous values.

8.5.1 Additional Discussion

8.5.1.1 Skew Bridges

Modification factors for one lane loaded and multiple lanes loaded have been developed. However, the equations proposed are appropriate for bridges with skew angles no greater than 15°. It was seen that the variation in distribution as computed using the various refined methods was significant for skewed bridges and the correlation is significantly improved with their exclusion. Since there are only 3 of the 17 bridges are skewed, modification factors for skewed bridges were not investigated. A broader dataset is recommended.

8.5.1.2 Exterior Girders

Modification factors have not been derived for moments for exterior girders. This is an acceptable limitation, however, as the exterior girders are rarely the governing components in bridge rating, as most of the traffic load is shared by the interior girders. Because of physical limitations in field testing, strain data from exterior girders was limited. A broader dataset is recommended before this issue is pursued further.
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 and a pool of structures was selected for field test operations. The population was chosen based on several factors including: geographical distribution, climate and traffic factors, various groupings of structural characteristics and parameters, availability of access, traffic control needs, and safety issues. All bridges selected were carried on the Ohio inventory at 150% Ohio legal load. Based on this process, 24 specimens were selected and tested. The data from these tests were merged with data from 6 earlier bridge field tests in order to obtain a 30 bridge data set. The 30 bridge list of test specimens chosen provided a good statistical match with the Ohio steel stringer bridge population. Ten of the twelve districts were represented. The list included bridges from 1-5 spans, 38-359 ft in overall length, 27-55 ft in overall width, 0-52° skew, and 520-94,000 ADT.

9.3 MULTIPLE REFERENCE IMPACT TESTING

This project employed a form of modal testing referred to as multiple reference impact testing as one of its primary experimental field test tools. 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 collection 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 setup 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. The maximum speed for an accurate estimate can be 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.

In addition, the unit influence line formed the basis for a method developed to rate each of the instrumented sections for all of the truckload tests for this project. 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 Appendix A. 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 within 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 of the structure are presented for both the top and bottom flange at these critical locations.

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).
A suite of condition indices can be processed directly from the measured and/or modeled sections in order to assess the performance of the structure and compare this with the nominal design values, including:

- Truckload moment, analyzed by four different methods
- Capacity load rating, analyzed by three different concepts and two loading cases
- Working or allowable stress
- Load Factor strength limit state
- Overload serviceability limit state
- HS20 Truckload rating, Inventory and Operating
- HS20 Laneload rating, Inventory and Operating

This research, thus, provides a novel approach to condition assessment by using the measured and/or modeled influence lines to virtually simulate the rating loads for immediate field assessment of a highway bridge using the above suite of condition indices.

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 SAP input file. The model is then fed as input to SAP 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.
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. The concept of a “real” distribution factor ($DF_{real}$) is introduced, which is simply the average of DF values obtained using refined methods. This value of DF for each bridge is the closest estimation of the actual distribution present in each bridge. Thus it is used as the comparator in calculating $\%\Delta_{LRFD}$, $\%\delta_{LRFD}$, $\%\Delta_{STD}$, and $\%\delta_{STD}$, which are used to evaluate the applicability of the proposed modification factors.

Several parameters, 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.

The modification factors derived in this work are based on the data that are available from the completed experiments. Every effort has been made to include as wide a range of parameters as possible. It should be remembered, however, that since the modification factors proposed herein are empirical in nature, use of these relations with parameters outside the range of the dataset upon which they are based may result in erroneous results.

Modification factors for one lane loaded and multiple lanes loaded have been developed. However, the equations proposed are appropriate for bridges with skew angles no greater than 15°. It was seen that the variation in distribution as computed using the various refined methods was significant for skewed bridges and the correlation is significantly improved with their
exclusion. Since there are only 3 of the 17 bridges are skewed, modification factors for skewed bridges were not investigated. A broader dataset is recommended. Modification factors have not been derived for moments for exterior girders. This is an acceptable limitation, however, as the exterior girders are rarely the governing components in bridge rating, as most of the traffic load is shared by the interior girders. Because of physical limitations in field testing, strain data from exterior girders was limited. A broader dataset is recommended before this issue is pursued further.
STATEMENT OF NEED:

According to the National Bridge Inventory, the most common type of short-to-medium span highway bridge in the US is the reinforced concrete (RC) slab-on-steel girder bridge with RC abutments and piers, comprising approximately half of Ohio’s inventory. For these bridges AASHTO currently offers a number of rating methods each of which is based solely on theoretical and design calculations. 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 on the current approaches.

RESEARCH OBJECTIVES:

- Identify a statistical sample of 24 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 comprehensive statistical database of actual structural identification results, rigorously calibrated FE models, and accurate rating factors for 30 bridges (i.e., 6 bridges from an earlier study plus the 24 bridges tested under this project) representing a baseline for the general steel-stringer bridge population within Ohio.
- Develop interactive, user-friendly, PC-based software packages that can be used to model, simulate, and rate steel-stringer bridges.
- Verify/validate the practicality and structural legitimacy of the modeling and rating processes developed.

RESEARCH TASKS:

- Bridge inventory review and test site selection
• Conduct field testing at bridge sites employing a combination of modal testing and truck-load testing methods.
• Development of software tools for FE modeling, test post processing, calibration, and rating.
• Utilization of database to better understand and address serviceability issues.

RESEARCH DELIVERABLES:

• A series of streamlined standard testing procedures for applying both modal testing and truckload testing practically, reliably, and safely under field conditions to obtain objective, quantitative measurements of key aspects of steel stringer bridge behavior,
• A comprehensive database of actual structural identification results, rigorously calibrated FE models, and accurate rating factors for 30 bridges representing the general steel-stringer bridge population within Ohio, and
• the incorporation of all testing and database findings/observations into a system of software packages to automate FE model development, modal test post processing, truck-load test post processing, model calibration, and rating calculations.

RESEARCH RECOMMENDATIONS:

Partly through conducting this research 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. Follow-on activities and research recommendations based on the findings of this project include:

• 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 examination of a population of 30 bridges has allowed the data collected to 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:

IMPLEMENTATION STEPS & TIME FRAME:

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:

Approved By: (attached additional sheets if necessary)

Office Administrator(s):

Signature: __________________________  Office: _______  Date: ____________

Division Deputy Director(s):

Signature: __________________________  Division: _________  Date: ____________
References


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References


References


Appendix A - Bridge Reports

This appendix is formatted in a pdf file on the enclosed CD-ROM. It contains reports for each of the 30 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 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.