CONDITION ASSESSMENT OF EXISTING BRIDGE STRUCTURES

GTRC Project No. E - 20 - K90
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Report of Task 4 – Development of guidelines for condition assessment, evaluation and rating of bridges in Georgia

FINAL REPORT

Prepared for

GEORGIA DEPARTMENT OF TRANSPORTATION

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ABSTRACT

Condition assessment and safety verification of existing bridges and decisions as to whether posting is required currently are addressed through analysis, load testing, or a combination of these methods. Structural analysis-based rating is by far the most common method for rating existing bridges. Load testing may be indicated when the analysis produces an unsatisfactory result or cannot be completed due to a lack of design documentation, information, or the presence of deterioration. The current rating process is described in the American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation (MBE), First Edition (2008). This recently published Manual permits ratings to be determined by either allowable stress (ASR) or load factor (LFR) methods (Section 6B), or by the load and resistance factor (LRFR) method (Section 6A). The LRFR method is keyed to the AASHTO LRFD Bridge Design Specifications, Fourth Edition (2007) which has been required for the design of new bridges since October, 2007. The State of Georgia currently utilizes the LFR method, which was permitted under the Manual for Condition Evaluation of Bridges, Second Edition. These three rating methods which continue to be commonly used – ASR, LRF, LRFR - may lead to different rated capacities and posted limits for the same bridge, a situation that has serious implications with regard to public safety and the economic well-being of communities that may be affected by bridge postings or closures. To address this issue, the Georgia Institute of Technology has conducted a research program, sponsored by the Georgia Department of Transportation, to develop improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed.

The product of this research program is the Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia. These guidelines address condition assessment and capacity evaluation by analysis, load test, or a combination of the two methods, depending on the circumstances and preferences of the GDOT. Part I of this report summarizes the technical approach taken to develop the Recommended Guidelines. Part II presents the Recommended Guidelines. An Appendix to Part II illustrates their use in typical rating situations.

KEY WORDS:

Bridges; concrete (reinforced); concrete (pre-stressed); condition assessment; loads (forces); probability; reliability; steel; structural engineering.
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EXECUTIVE SUMMARY

Bridge structures in the United States are at risk from aging, leading to structural deterioration from aggressive environmental attack and other physical mechanisms, service demands from increased traffic and heavier loads, and deferred maintenance. Condition assessments of an existing bridge may be conducted to develop a bridge load rating, confirm an existing load rating, change a rating for future traffic, or to determine whether the bridge must be posted in the interest of public safety. Changes in traffic patterns; concern about faulty building materials or construction methods; discovery of a design/construction error after the structure is in service; concern about deterioration discovered during routine inspection; and damage following extreme load events may prompt such evaluations. In the State of Georgia, rating calculations have yet to be performed on 1,587 of the bridges that the Georgia Department of Transportation (GDOT) monitors. Moreover, approximately 1,982 of the 8,988 bridges monitored by the GDOT have been determined to require posting. Posting or other restrictions may have a severe economic impact on the State economy, which depends on the trucking industry for distribution of resources and manufactured goods. The economics of upgrading or posting a bridge makes it imperative that condition assessment criteria and methods (either by analysis or by testing) be tied in a rational and quantitative fashion of public safety, functional requirements, and economics.

Condition assessment and safety verification of existing bridges, and decisions as to whether posting is required, are addressed through analysis, load testing, or a combination of methods. Bridge rating by structural analysis is by far the most common (and most economical) procedure for rating existing bridges. Load testing may be indicated when analysis produces an unsatisfactory result of when the analysis cannot be completed due to lack of design documentation, information, or the presence of deterioration. Until recently, the bridge rating process was described in the American Association of State Highway and Transportation Officials (AASHTO) Manual for Condition Evaluation for Bridges, Second Edition,\textsuperscript{1} which permitted ratings to be determined through either the Allowable Stress Rating (ASR) or Load Factor Rating (LFR). The State of Georgia traditionally has utilized the load factor (LFR) method for those bridges that have been rated. A third and more recent method, found in the Manual for Condition Evaluation and Load and Resistance Factor Rating,\textsuperscript{2} was keyed to the new AASHTO LRFD Bridge Design Specifications, Fourth Edition.\textsuperscript{3} The recently issued Manual for Bridge Evaluation, First Edition (MBE, 2008),\textsuperscript{4} consolidates the existing rating methods and permits ratings to be determined by either ASR or LFR methods (Section 6B) or the load and resistance factor (LRFR) method (Section 6A). The ASR, LFR and LRFR rating methods may lead to different rated capacities and posted limits for the same bridge,\textsuperscript{5} a situation that cannot be justified from a professional engineering viewpoint and carries


serious implications with regard to the safety of the public and the economic well-being of businesses and individuals who may be affected by bridge postings or closures.

The Georgia Department of Transportation has an urgent need for condition assessment tools that can be used with confidence to determine whether or not to post certain bridge structures. To address this need, the Georgia Institute of Technology has recently completed a multi-year research program, sponsored by the GDOT, aimed at making improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed. The end product of this research program is a Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia. The Recommended Guidelines address condition assessment and evaluation by analysis, load test, or a combination of the two methods, depending on the circumstances. Consistent with the AASHTO LRFD Bridge Specifications, they have a sound basis in structural engineering and structural reliability principles, allowing them to be updated as changing circumstances (traffic demands, additional data, material deterioration, and other factors) warrant. The Recommended Guidelines are presented in a relatively simple, practical and familiar form that is suitable for implementation in routine bridge rating assessments.

The research program undertaken by Georgia Tech for the State of Georgia consists of four tasks:

Task 1: Review and critically appraisal of the state-of-the-art of bridge condition assessment
Task 2: Bridge evaluation by load testing
Task 3: Bridge evaluation by advanced analysis
Task 4: Development of Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia

The results of Tasks 1, 2 and 3 have been reported previously.\textsuperscript{5, 6} The review in Task 1 revealed common rating practices and difficulties that States encounter in applying the alternative AASHTO bridge rating procedures consistently. On the international scene, it was found that modern bridge rating procedures worldwide have adopted reliability principles as their basis, utilize the limit state (as opposed to allowable stress) philosophy in strength calculations, and allow the ratings to be performed using a deterministic format with an underlying reliability basis. These approaches are consistent philosophically with the LRFR method, although the load and resistance factors may differ. The ultimate limit states are typically required as the governing limit states for safety checking for majority of the bridge types. Tasks 2 and 3 were performed concurrently. A key ingredient of the research program was development of advanced finite element models of bridges and the validation of such models by means of load tests, with the objective of using similar finite element modeling techniques to extend the scope of the investigation to a broad selection of bridges, to conduct “virtual load tests” of bridges in that extended group, and to use those evaluations as a basis for critically appraising and revising, as appropriate, the current bridge rating process. To this end, the GDOT bridge database was screened to identify candidate bridges for load testing, with the assistance of GDOT bridge engineering staff. Four bridges representing the type of structures that currently are of most concern to GDOT staff were identified, based on a series of primary


and secondary criteria such as structural and material types, age (design load), condition ratings and span lengths, and FE models of these four bridges were developed. Concurrently, the four bridges were load-tested with the assistance of GDOT maintenance staff. These bridges include reinforced concrete T-beam, pre-stressed girder, and steel girder bridges. The predicted and observed deflections agreed closely for all four bridges tested; discrepancies can be attributed to various uncertainties associated with experimental data collection under field conditions and the many assumptions that were required in the FE analyses, including homogeneity and magnitude of in situ material properties, and idealized boundary conditions. For all four bridges tested and analyzed, it was found that the current load rating procedures lead to unnecessarily conservative bridge ratings.

This Task 4 report is presented in two parts. Part I summarizes the research accomplishments described in detail in the previous task reports and provides the archival technical basis for the Recommended Guidelines. Part II presents the Recommended Guidelines and commentary. The Recommended Guidelines are keyed to the LRFD option in the Manual for Bridge Evaluation, First Edition (2008); they modify selected portions of the Manual for Bridge Evaluation to make it specifically applicable to condition assessment and rating of reinforced concrete, pre-stressed concrete, and steel girder bridges in the State of Georgia. In addition, a new Section permits a direct reliability-based approach to bridge rating where circumstances warrant; the provisions in this section are somewhat more complex than those in the standard formula-driven rating process, but are likely to result in a less conservative rating if used, thus justifying the additional effort. An Appendix to Part II of the report illustrates the use of the Recommended Guidelines in specific rating situations, and compares the ratings obtained to those that would be obtained using the existing AS, LF and LRFR methods. Implementation of the Recommended Guidelines in Georgia is likely to result in less conservative bridge ratings and posting requirements for most bridges in the State; for the four bridges studied in detail, the current ratings are 20 to 40% more conservative than the Recommended Guidelines would suggest is necessary. The main reasons for the less conservative ratings are: more realistic girder distribution factors; an improved procedure for permitting the use of in situ material properties through an enhanced statistically-based sampling plan; a newly derived condition factor, $\phi_c$, which is keyed to the latest bridge inspection; and the use of structural evaluation methods (e.g., strut-and-tie analysis, finite element analysis) that capture the mechanics of structural behavior more accurately in limit states that govern the rating process (e.g., pier cap shear capacity, bridge system level capacity).

During the period in which the research reported herein was conducted (August 2005 – May, 2009), the two available AASHTO rating manuals were the Manual for Condition Evaluation for Bridges, Second Edition and the Manual for Condition Evaluation and Load and Resistance Factor Rating. The Manual for Bridge Evaluation, First Edition (2008) (MBE) was adopted by the AASHTO Highways Subcommittee on Bridges and Structures in 2005, but was disseminated to the state bridge offices in mid-2009 and was unavailable to the research team until the project was in its final stages. Accordingly, Part I of this final report is based on the AASHTO documents that were available at the time that the research was performed. A close scrutiny of the provisions in the new MBE has revealed that none of the findings and recommendations in Part I are affected by the new document. In contrast, the Recommended Guidelines in Part II are keyed to the organization and provisions in the MBE (2008), in recognition that they are likely to be used with this more recent AASHTO document and to facilitate their adoption by bridge engineering staff.
PART I

RESEARCH SUMMARY
1. INTRODUCTION

1.1 Background to Research Program

Bridge structures in the State of Georgia are at risk from aging, leading to structural deterioration from service demands from increasing traffic and heavier loads, from aggressive environmental attack and other physical mechanisms, and from deferred maintenance. Condition assessment and evaluation of existing bridges may be prompted by changes in traffic patterns; concern about faulty building materials or construction methods; discovery of a design/construction error after the structure is in service; concern about deterioration discovered during routine inspection; and damage following extreme load events. A condition assessment may be conducted to develop a bridge load rating, confirm an existing load rating, increase a load rating for future traffic, or determine whether the bridge must be posted in the interest of public safety. The Bridge Inventory Management System in the State of Georgia lists 8,988 bridges, which are monitored by the Georgia Department of Transportation (GDOT). While rating calculations have yet to be performed on 1,587 of these bridges, it has been determined that approximately 1,982 of them require posting. Posting or other restrictive actions may have a severe economic impact on the state economy, which depends on the trucking industry for distribution of resources and manufactured goods. The economics of upgrading or posting a bridge makes it imperative that condition assessment criteria and methods (either by analysis or by testing) be tied in a rational and quantitative fashion to public safety, functional requirements and economics.

Condition assessment and safety verification of existing bridges, and decisions as to whether posting is required are addressed through analysis, load testing, or a combination of methods. Of these, bridge rating by structural analysis is by far the most common (and most economical) method. Load testing may be indicated when analysis produces an unsatisfactory result or when the analysis cannot be completed due to lack of design documentation, information, or the presence of deterioration. Until recently, the customary rating process used in most states has been described in the American Association of State Highway and Transportation Officials (AASHTO) Manual for Condition Evaluation of Bridges, Second Edition, which allows ratings to be determined through either allowable stress methods (ASR) or load factor methods (LFR). The State of Georgia currently utilizes the LF method for those bridges in the state that have been rated. A third (and more recent) rating procedure found in the Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges is keyed to the AASHTO Load and Resistance Factor Design (LRFD) method, which is defined in the AASHTO LRFD Bridge Design Specification, Fourth Edition. The LRFR method is being introduced in bridge maintenance, and some states are beginning to use it to determine bridge ratings. These three competing

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rating methods may lead to different rated capacities and posting limits for the same bridge\(^5\), a situation that cannot be justified from a professional engineering viewpoint and has potentially serious implications with regard to the safety of the public and the economic well-being of businesses and individuals who may be affected by bridge postings or closures. The cost and social impact of failing to meet a performance objective when evaluating an existing bridge can be very large, particularly if posting is indicated. Accordingly, the economics of upgrading or posting makes it imperative to determine condition assessment criteria and methods (either by analysis or by testing) that are tied in a rational and quantitative fashion to public safety and functional objectives.

The Georgia Department of Transportation has an urgent need for condition assessment tools that can be used with confidence to determine whether or not to post certain existing bridge structures. To address this need, the Georgia Institute of Technology has conducted a multi-year research program, sponsored by the GDOT, aimed at making improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed.

### 1.2 Project Objectives and Scope

The objective of this research program was to develop a *Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia*, for practical use by the GDOT in rating bridges. These *Recommended Guidelines* have been developed with the aid of advanced finite element modeling techniques and structural reliability principles that are consistent with those used to develop the AASHTO LRFD Bridge Design Specifications, Fourth Edition, and the LRFR option in the AASHTO Manual for Bridge Evaluation, First Edition (2008). The *Recommended Guidelines* address condition assessment and evaluation by analysis, load test, or a combination of the two methods, depending on the circumstances and preferences of the GDOT. They have a sound basis in structural engineering and structural reliability principles, allowing them to be updated as changing circumstances (traffic demands, supporting databases on materials and traffic loads, material deterioration, etc) warrant. At the same time, they are presented in a simple and familiar form that is suitable for implementation in routine rating assessments.

This report contains two parts. Part I summarizes the technical bases for the *Recommended Guidelines*. Part II presents the *Recommended Guidelines* in a form that is suitable for implementation in GDOT engineering manuals and policy statements.

During the period in which the research reported herein was conducted (August 2005 – May, 2009), the two AASHTO rating manuals available to the research team were the *Manual for Condition Evaluation of Bridges, Second Edition* and the *Manual for Condition Evaluation and Load and Resistance Factor Rating, First Edition*. The *Manual for Bridge Evaluation, First Edition* (MBE)\(^6\) became available to the research team in June, 2009. Accordingly, Part I of this final report is based on the

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AASHTO documents that were available at the time that the research was performed. A close scrutiny of the provisions in the new MBE has revealed that none of the findings and recommendations in Part I are affected by the new document. The Recommended Guidelines in Part II are keyed to the organization and provisions in the MBE (2008), in recognition that they are likely to be used with this more recent AASHTO document and to facilitate their adoption by bridge engineering staff.
2. TECHNICAL APPROACH

The research program leading to the Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia consisted of four tasks:

- **Task 1**: Review and appraisal of the state-of-the-art of bridge condition assessment
- **Task 2**: Bridge evaluation by load testing
- **Task 3**: Bridge evaluation using advanced analysis techniques
- **Task 4**: Development of Recommended Guidelines for Bridge Condition Assessment in Georgia

Task 1 established the frame of reference for the study. In Task 2, four bridges selected with the assistance of GDOT Staff were load-tested, while in Task 3, advanced finite element models of these bridges were developed. Tasks 2 and 3 were conducted concurrently. The load test results were used to validate the finite element modeling process to the extent that the FE modeling could be used with confidence to conduct virtual load tests of any bridge in the State of Georgia to provide technical support for the Guidelines development. Task 4 focused on the development of the Recommended Guidelines, based on the findings from Tasks 1 through 3, which have been presented in previous reports.

2.1 Summary of Task 1: Review and Appraisal of State-of-the-art of Bridge Condition Assessment

A comprehensive review was undertaken of current procedures for performing condition assessments of existing civil infrastructure, aimed at achieving a general perspective on technical issues associated with condition assessment methodologies used for bridges. The review emphasized current practices in the United States, but practices in several other industrialized countries were also summarized to provide additional context. Reliability-based condition assessment tools and the existence of databases to model uncertainty that would support bridge assessment by the GDOT also were examined. The results of this review established the context for the conduct of Tasks 2 and 3.

Ongoing activities in the Departments of Transportation in selected states were scrutinized. Current practices with regard to bridge inspection, including underwater inspection of load-bearing components, analysis and load testing, and posting of bridges were reviewed. A questionnaire was mailed to state bridge maintenance engineers in all 50 states seeking additional information on a subset of the queries in a May, 1997 survey conducted as part of NCHRP Project 12-46, specifically with regard to the adoption of the AASHTO LRFR Guidelines. 41 states replied to the survey and their responses were fully reviewed in detail in the Task 1 report. To summarize the findings of this survey:

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• All states either perform load rating analysis or make judgment calls as to the load capacity of all of their bridges. Most states are working toward 100% load rating. It is the current policy of most states to rate all new structures when they are designed or constructed. Existing unrated bridge structures are being rated as time and resources permit.

• Twenty six of the 41 responding states (63%) reported that the primary rating method is the LFR method. The AS (Allowable Stress) method is used occasionally in cases where LFR does not appear applicable. Six states (15%) reported using a combination of AS, LFR, and LRFR methods, depending on the specification governing the design of the bridge. Few states are using the new LRFR procedures. Several states expressed concerns about the apparent differences in ratings achieved by LRFR and previously by LFR and ASR.\(^9\)

• Most states use the HS20-44 vehicle as a basis for rating for both inventory and operating loads, with some using the AASHTO legal vehicles as well.

• The decision as to when to post a bridge is highly variable from state to state. There is no consensus on whether to post at operating, inventory, or legal loads. Most states use one of these three load cases, coupled with engineering judgment. States participating in the survey reported that the percentage of posted bridges on local roads is typically anywhere from 10 to 100 times the percentage of posted bridges on state roads.

• Fifteen of the responding states (37%) perform some form of diagnostic testing. Two states reported that they use load testing extensively to avoid having to perform costly repairs, replace or post due to unreliable or unfavorable AASHTO rating factors.

• All states perform some sort of scour inspection in conformance with FHWA requirements; beyond that, the level of inspection varies from state to state.

• Proof load tests are seldom performed for bridge rating purposes.

Bridge rating practices in the United States also were benchmarked against international bridge evaluation guidelines. Documents used for bridge rating in the United Kingdom,\(^10\) Australia,\(^11\) and Canada\(^12\) were secured. It was found that, in general, modern bridge rating procedures worldwide have adopted reliability principles as their basis. All utilize the limit state (as opposed to allowable stress) philosophy to allow the safety checking to be performed in a deterministic manner without explicit reference of the reliability indices. Thus, the approach is consistent philosophically with the LRFR method, although the load and resistance factors used in bridge rating may differ. The ultimate strength limit states are typically required as the governing limit states for safety checking for majority of the

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\(^9\) This issue in current bridge rating procedures provides strong motivation to resolve the discrepancies between the ASD, LFR and LRFR using quantitative reliability-based methods. This approach is introduced in Task 4 below.

\(^10\) BD21/01, 2001, Assessment of Highway Bridges and Structures; BA54/94, 1994, Load Testing for Bridge Assessment

\(^11\) Standards Australia International Australian Standard Bridge Design, Sydney, NSW 2001 Australia

Chapter 2
Technical Approach

Chapter 2
Report of Task 4
Technical Approach

bridge types; serviceability and fatigue are not regularly mandated unless signs of distress or fatigue related defects are observed. The view towards load testing is somewhat different from country to country, which leads to different treatment of the provisions on this subject in different guidelines.

2.2 Summary of Task 2: Bridge Evaluation by Load Testing

A key ingredient of the research program was the validation of finite element modeling of bridges by means of load tests conducted on four bridges representing the type of structures that currently are of most concern to GDOT staff. Once completed successfully, similar finite element modeling techniques then could be used to extend the scope of the investigation to a broad selection of bridges, to conduct “virtual load tests” of bridges in that extended group, and to use those evaluations as a basis for critically appraising and revising, as appropriate, the current bridge rating process in the State of Georgia.

The Georgia Department of Transportation bridge database was screened to identify candidate bridges for load testing, with the assistance of GDOT bridge maintenance staff. Of these, 20 that were located within approximately 50 miles of the Atlanta metropolitan area to facilitate easy access during the project were identified as being representative of the majority of the bridge population in the state that might be considered problematic from a rating standpoint, based on a series of primary and secondary criteria such as structural and material types, age, condition ratings and span lengths. Following preliminary site visits and a final review of these bridges for suitability and testing feasibility with State bridge maintenance engineering staff, four bridges were selected for load testing and subsequent detailed analysis. Since these bridges play a central role in establishing the technical basis for the Recommended Guidelines, their characteristics are summarized below.

2.2.1. Reinforced concrete T-beam bridge – straight approach (Bridge ID 129-0045)

Figure 2.2.1 Straight RC T-Beam Bridge (ID: 129-0045, Gordon County)

This bridge carries SR 156 (two lanes) over Oothkalooga Creek approximately one mile west of Calhoun in Gordon County, GA. It was designed according to the AASHTO 1953 Design Specification for H-15 loading, and was constructed in 1957. At the time it was selected for load testing, it had been scheduled for replacement in 2008 for functional rather than safety reasons. The bridge had eight simple spans, seven of which are 40 ft (12.2 m) and one (over the channel) of which is 45 ft (13.7 m).
girders were 18 ½ in x 24 ¾ in (470 mm x 629 mm), except for the long span which was 31 ¾ in (807 mm) in depth, and were spaced 7.3 ft (2.2 m) apart. The roadway had a 6-in (152 mm) thick deck with a deck width of 32.3 ft (9.8 m) and a roadway width of 25.7 ft (7.8 m). The design documentation indicated that the specified 28-day compression strength of the concrete was 2,500 psi (17.2 MPa), while the yield strength of the reinforcement was 40 ksi (276 MPa). The bridge carried average daily truck traffic (ADTT) of 458. The concrete deck had a condition rating of 5, the supporting reinforced concrete T-beam superstructure was rated at 7, and the concrete bent and pier substructure were rated at 6. The latest inspection report indicated that all caps had minor hairline cracking, and that several areas of exposed cap reinforcement were present. All beams were reported to show signs of typical flexural cracking. The entire deck had moderate surface deterioration, scaling, and cracking, and had been repaired in several notably bad sections. Although the latest GDOT load rating performed using the LF rating method determined the girder shear capacity to 25 tons to be the limiting factor, the bridge was not posted. (Subsequent to the load test, this bridge was demolished in May 2008, providing an opportunity for in situ strength testing of the concrete; these tests are discussed in Section 3.1 of this report.)

2.2.2 Reinforced concrete bridge – skewed approach (Bridge ID: 015-0108)

This 12-span structure carries Old Alabama Rd. over an extensive flood plain and Pumpkinvine Creek 3.7 miles south of Cartersville, GA in Bartow County. The ADTT is 709. The centerline of this two-lane bridge is skewed 30 degrees with respect to the girder supports. The bridge was designed using the 1977 AASHTO Specifications and was constructed in 1979. The bridge was selected, in part, because the GDOT BIMS database reported it as being designed for H-15 loading; it was discovered later that the bridge actually had been designed for H-20 loading. The eleven spans over the flood plain are carried by 40-ft (12.2 m) reinforced concrete T-beams, cast monolithically with the 7.75 in (197 mm) thick deck slab. The other span (over the channel) is a 70-ft (21.3 m) pre-stressed concrete I-beam span. The deck width is 40.25 ft (12.3 m) and the roadway width is 40 ft (12.2 m). The specified 28-day compression strength of the concrete was 2,500 psi (17.2 MPa), while the yield strength of the reinforcement was 40 ksi (276 MPa). The current bridge condition ratings for substructure, superstructure, and deck are 6, 6, and 7 respectively; bridge capacity rating is governed by the flexural limit state and the bridge is posted for three truck loads: H (21 tons), Tandem (19 tons), and Log (24 tons). There is minor cracking and
spalling in a number of the bents and abutments, as well as in the T-beams, but no need of immediate repair.

2.2.3 **Pre-stressed concrete girder bridge (Bridge ID: 223-0034)**

![Pre-Stress Bridges (ID: 223-0034, Paulding County)](image)

This bridge carries State Route 120 over Little Pumpkinvine Creek approximately 5 miles south of Dallas in Paulding County GA. The ADTT is 980. The bridge was designed for HS-20 loading using the *AASHTO 1989 Specifications* and was constructed in 1992. The main structural system consists of pre-stressed concrete I-Beams arranged in four simply supported spans. The bridge is 216 ft (65.8 m) long and consists of two 40-ft (12.2-m) Type III pre-stressed I-girder spans and two 68-ft (20.7-m) Type III prestressed I-girder spans. The centerline of the bridge is essentially perpendicular to the girder supports. The bridge has a deck width of 43⅛ ft (13.2 m) and a roadway width of 40 ft (12.2 m). The 68-ft (20.7-m) spans are comprised of five type III I girders that are composite with the 9⅛ in (232 mm) thick slab (Figure 2.2.3). The girders are constructed with 6,000 (41 MPa) concrete and prestressing strand with an ultimate tensile strength of 270 ksi (1,862 MPa). The bridge is in good condition, with substructure, superstructure and deck condition numbers of 7, 8 and 7, respectively, and is not posted.

2.2.4 **Steel girder bridge (Bridge ID: 085-0018)**

This bridge carries SR 136 over the Etowah River 5.7 miles east of Dawsonville, GA in Dawson County. The centerline of the bridge is perpendicular to the girder supports. The bridge was designed using the *AASHTO 1961 Specifications*, with interim revisions through 1963, for H-15 loading, and was constructed in 1965. The bridge is 196 ft (59.7 m) long and its four 49 ft (12.2 m) spans are supported by four steel girders spaced at 8 ft (2.4 m) on centers; the two facia girders are W33x118, while the two interior girders are W33x130, with a full-depth diaphragm located at mid-span (Figure 2.2.4). The two-lane bridge has a (non-composite) concrete deck 6.5 in (165 mm) thick, with overall width of 32 ft (9.8 m) and a roadway width of 26 ft (7.9 m). The specified 28-day compression strength of the concrete in the deck slab and the pier caps was 3,000 psi (20.7 MPa), while the yield strength of the reinforcement was 40 ksi (276 MPa). The girders were A36 steel, with nominal yield strength 36 ksi (248 MPa). At the time of selection, the bridge had been last inspected on June 30, 2005, and at that time the deck and
substructure both were assigned a condition assessment rating of 6. At that time, the inspection report indicated that there was spalling, aggregate exposure, and transverse cracking in the deck in all spans. The bridge has been determined to require posting because of a perceived deficiency in the shear capacity of the reinforced concrete pier cap (see Figure 2.2.4), and has been posted for a 21-ton H load, 25-ton HS load, 23-ton Tandem load, 32-ton 3-S-2 load, and 27-ton Log load. The piles have minor pitting and the beams have minor deflections. The bridge carries an ADTT of 280.

Figure 2.2.4 Steel Girder Bridge (ID: 085-0018, Dawson County)

2.2.5 Bridge load tests

All load tests and their instrumentation were designed with the assistance of preliminary finite element models of the bridges (the finite element modeling procedure is described in more detail in Section 2.3) and estimated test truck loads. The load tests were performed by GDOT employees, following the test protocols instituted by the principal investigators and under their general supervision. A consultant was also engaged to assist in the conduct of the tests and to provide independent verification of the bridge responses measured in the load tests.

Each bridge was loaded by four fully loaded GDOT dump trucks weighing approximately 60,000 lb each (details are presented in Chapter 3). Thus, the maximum test load on each bridge was on the order of 125% to 150% of the design live load. Prior to the conduct of the load testing of each bridge, the actual test vehicles were weighed. During each test, the truck placement on the bridge spans and wheel locations were measured. Each load test involved temporary closure of the bridge while the trucks were
moved onto the bridge, one at a time, with measurements taken at each stage of the test. Load tests were repeated three times to identify possible instrumentation problems and to ascertain the reproducibility of the test procedure.

Load tests of these four bridges were conducted during the period September 26, 2006 – May 10, 2007. A summary of the load test results, comparing the predicted and observed girder displacements for the RC T-beam Bridge (ID 129-0045) is presented in Section 2.3, which follows. Details of the testing program, including the bridge selection, instrumentation, testing process and the post-test assessment of the measurements for all four bridges, can be found in the Task 2 report.7

2.3 Summary of Task 3: Bridge Evaluation Using Advanced Analysis Techniques

Finite element (FE) models of the four bridges identified in Task 2 were developed concurrent to the conduct of Task 2 for several purposes: to benchmark current bridge rating procedures followed by the Georgia DOT independently and to identify issues that might not be apparent with simplified rating methods; to assist in the design of the instrumentation used to monitor the Task 2 load tests; and to provide technical support for the recommended condition assessment and evaluations in the Recommended Guidelines to be developed in Task 4. Improvements to current bridge rating guidelines require an understanding of bridge system behavior subjected to extreme load events which may well exceed the load level applied in the tests conducted in Task 2. In addition, finite element models, once validated through field inspection, material testing and load tests, can then be applied to conduct “virtual” proof load tests to predict the load capacity of bridges of interest in the Georgia Bridge Inventory and to support the technical development of the Recommended Guidelines using structural reliability principles in Task 4.

Three-dimensional (3D) nonlinear finite element models of the superstructure of each sample bridge were developed in Task 3. All FE models of the sample bridges initially were developed using the ABAQUS13 commercial FE package according to bridge design and construction documents secured from the GDOT (FE modeling details can be found in Appendix A of the Task 2 Report7). Before conducting the load tests, FE analyses were performed using estimated bridge material strengths and anticipated vehicle weights and arrangements to assist in designing test instrumentation, to identify test vehicle locations, and to anticipate and guard against potential bridge vulnerabilities that might become apparent during the load tests. Following the load tests, FE analyses were revised to reflect the actual test vehicle weights and wheel locations in the tests, and predicted responses were compared with test measurements to determine the accuracy with which FEA can predict bridge behavior.

The comparison of predicted and measured response for the RC T-beam Bridge ID 129-0045, which is presented in Figures 2.3.1 and 2.3.2, is typical of the results for all four bridges. Figure 2.3.1 shows the locations of the trucks on the bridge. Figure 2.3.2 compares girder displacements at mid-span predicted by the FE analysis and the displacements measured by potentiometers at the same location. The predicted and observed girder deflections showed good agreement for all four bridges tested. The discrepancies are invariably within 20% and, in the majority of cases, are substantially less. Such differences can be attributed to various uncertainties associated with experimental data collection under field conditions and the many assumptions made in the FE analyses, including homogeneity and magnitude of in situ material properties, and idealized boundary conditions. Taking these factors into account...
consideration, the FE models of the four test bridges are deemed to be sufficient to describe and quantify the load-bearing mechanisms that affect the bridge capacity and its load ratings.

![Diagram of Reinforced Concrete T-beam Bridge ID 129-0045: arrangement of trucks on bridge during test](image1)

**Figure 2.3.1:** Reinforced concrete T-beam Bridge ID 129-0045: arrangement of trucks on bridge during test

![Graphs showing Girdler displacements](image2)

**Figure 2.3.2:** Reinforced concrete T-beam Bridge ID 129-0045: Girder displacements due to truck loadings
2.4 Summary of Task 4: Development of Recommended Guidelines for Bridge Condition Assessment

The *Recommended Guidelines for Condition Assessment of Bridges in Georgia* developed in this task builds upon experience with the *AASHTO LRFR Manual* and is intended to be used in conjunction with the LRFR option in the recently issued *Manual for Bridge Evaluation, First Edition* (2008). The *Recommended Guidelines* are presented in their entirety in Part II of this report. They are based on structural reliability principles and permit bridge condition assessment and rating evaluation to be performed at two levels.

The first level is similar to the procedures in the *MBE* for ease of implementation. Modifications to the existing provisions in Sections 1 through 8 of the *MBE* that are suggested by the current research (discussed in Chapter 3 of this report) are significant for purposes of condition assessment, but are sufficiently few that the organization of the *Recommended Guidelines* in Part II of this report (written in specification language) parallels the parent document and simply points out the provisions in the *MBE* that should be changed based on the current research.

The second level couples bridge condition evaluation quantitatively to the results of bridge inspections, material tests and load tests through a process that utilizes a direct reliability assessment. In contrast to the deterministic rating format in the first level, the second level presents explicit reliability-based rules for assessment purposes. Engineering judgments and indirect information can be incorporated systematically into the reliability-based assessment framework together with the observed field data to obtain improved estimates regarding the bridge capacity, rating, and need (if any) for posting. This assessment level highlights the learning process through field inspections or load tests of a given bridge. Section 9 in the *Recommended Guidelines* in Part II furnishes these provisions.
3. SUMMARY OF MAJOR RESEARCH FINDINGS

The research program leading to the Recommended Guidelines in Part II consisted of both experimental and analytical components, and was designed to achieve an understanding of the behavior of typical existing bridges through a spectrum of loadings ranging from service to extreme conditions. This research revealed that bridge rating practices that are currently utilized by the State of Georgia may lead to estimated bridge capacities that are excessively conservative and result in unnecessary bridge posting. These sources of conservatism arise from several factors. While not all of these factors may be present in every bridge that is a candidate for evaluation and rating, they occur with sufficient frequency that bridge maintenance engineers should be aware of their consequences in bridge evaluation. This chapter summarizes findings from the research program that underlie the Recommended Guidelines.

3.1 In situ Testing to Determine Strength of Structural Materials

Section 2.2.1 summarized the load test of Bridge ID 129-0045, a reinforced concrete T-beam bridge that was designed according to the AASHTO 1953 Design Specification for H-15 loading, and was constructed in 1957. The specified 28-day compression strength of the concrete was 2,500 psi (17.2 MPa), while the yield strength of the reinforcement was 40 ksi (276 MPa). This bridge was load-tested in September, 2006. The bridge was scheduled for demolition in May, 2008, providing an opportunity to secure drilled cores to determine the statistical properties of the in situ strength of the 51-year old concrete in the bridge.

Four-inch diameter drilled cores were taken from the slab of the bridge in May, 2008, prior to its demolition. The reinforcement within the slab was located prior to coring using a magnetometer. Seven (7) cores were taken from the slab at seven different locations along both the length and width of the bridge. The bridge then was demolished, but the contractor was instructed to leave the beams intact and pull them up onto the roadway. Girder No. 1 (see Figure 2.3.1) did not survive demolition and yielded no cores. Three cores were then taken from the other 3 girders. Due to the damaged state of the girders, the cores ranged from the full width of the girder [18 in (457 mm)] to 9 in (229 mm). The girder cores were then cut into 8-in (203 mm) lengths and the jagged ends were smoothed and capped, resulting in a total of 14 girder test cylinders. Tests of the 4 x 8 in (102 x 203 mm) cylinders were conducted at Georgia Tech during June - July, 2008. Testing and analysis conformed to ASTM Standard C42.14

The results of these core sample tests are presented in Table 3.3.1. An analysis of these data indicated no statistically significant difference in the concrete compression strength in the girders and slab, and the data were combined for further analysis. The mean (average) compression strength of the concrete is 4,820 psi (33 MPa) and the coefficient of variation is 12%, which is representative of good-quality concrete.15 The mean strength is 1.93 times the specified compression strength of the concrete.

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This increase in compression strength over a period of more than 50 years is typical of the increases found for good-quality concrete by other investigators.\textsuperscript{16}

If these results are typical of well-maintained concrete bridges, there is likely to be substantial strength gain in concrete over the 28-day strength that is customarily specified for design and is used for condition evaluation of a bridge. Accordingly, the Bridge Engineer should be encouraged to rate a bridge using the best possible information from \textit{in situ} material strength testing through incentives to acquire and utilize such test data in the rating process. The \textit{Recommended Guidelines} in Section 5 – Material Testing, Subsection 5.3 – Material Sampling and Subsection 5.5 – Interpretation and Evaluation of Test Results - provide such an incentive. It is customary to base the specified compression strength of concrete on the 10-percent exclusion limit of a normal distribution of cylinder strengths (ACI Standard 318-05). Using Bridge ID 129-0045 as an example, the 75\% lower confidence interval on the 10-percent exclusion limit of compression strength for a sample of 21 tests, $f_c$, would be \textit{(Recommended Guidelines, section 5.5)},

\begin{equation}
  f_c = (1 - k V) \bar{X}
\end{equation}

in which $\bar{X}$ = sample mean, $V$ = sample coefficient of variation, and $k$ equals 1.520 to obtain the 75\% lower confidence interval on the 10\%-ile value of the population distribution of 21 samples. Substituting the statistics in Table 3.3.1, one would obtain $f_c = (1-1.520 \times 0.12) \times 4,820 = 3941$ psi (27.17 MPa), a value that is 58\% higher than the 2,500 psi (17.2 MPa) that otherwise would be used in the rating calculations. The Recommended Guidelines adopt this approach in Section 5.5.

\begin{table}[h]
\centering
\begin{tabular}{|l|c|c|c|c|}
\hline
Source & Number & Average (psi) & Standard deviation (psi) & Coefficient of variation \\
\hline
Girder & 14 & 4,880 & 603 & 0.12 \\
Slab & 7 & 4,698 & 573 & 0.12 \\
Overall & 21 & 4,820 & 586 & 0.12 \\
\hline
\end{tabular}
\caption{Compression tests of 4 x 8 cores drilled from Bridge ID 129-0045 (Gordon County, GA)}
\end{table}

In the FE modeling of this bridge that preceded these strength tests, the compression strength was set at 2,500 psi, which was the only information available before the material test. In order to determine the impact of using the actual concrete strength in an older bridge on the rating process, the finite element model was revised to take the increased concrete compression strength (and the corresponding increase in stiffness) on bridge behavior. Results indicate only a modest enhancement in estimated bridge performance because the structural capacity of this particular bridge is dependent on other factors that are not impacted by concrete aging.

3.2 Finite Element-based Load Distributions among Girders

Girder distribution factors (GDF) are an important ingredient of bridge capacity evaluation. The GDF are different in the AASHTO ASD and LF rating methods from those in the LRFR rating method; these differences are one reason for the complaints received in the survey (Task 1) regarding the

inconsistency in the bridge ratings obtained from these methods. These differences were examined using finite element models developed in Task 3 of the four typical Georgia bridges (summarized Chapter 2).

Three-dimensional (3D) nonlinear finite element models of the superstructure of each bridge were developed. All FE models of the sample bridges initially were developed using the ABAQUS commercial FE package according to bridge design and construction documents secured from the GDOT (modeling details can be found in Appendix A of the Tasks 2 and 3 Report1). Subsequently, FE models were also developed using OpenSees, an open-source platform available from the University of California at Berkeley which computationally is highly efficient in its execution. [Computational efficiency was essential for the successful completion of the system reliability analysis, to be described subsequently, but the bridge responses in the elastic range were used to assess the GDFs as well.] Both FE platforms gave essentially the same distributions of forces to the bridge girders. These FE models were validated by the load tests of the four bridges, as noted earlier.

The load ratings calculated by the FE models substantially exceed the load ratings that are obtained by the Load Factor Rating (LFR) method2, which currently is used by the State of Georgia for bridge rating. Considering the straight-approach reinforced concrete Bridge ID 129-0045 (Gordon County) as an example, the LFR procedure results in a rating factor of 0.75 for the HS-20 design load at inventory level, while the rating by using load distribution factor obtained from FE analysis of this bridge for the same vehicle is 1.10, representing an increase of 47%. The actual distribution of load to the supporting girders is among the factors that contribute to this difference. The FE analysis of this bridge indicates that only about 41% of applied load actually went into the interior girder, while the girder distribution factor in the current LFR method would require up to 60% of live load to be apportioned to that girder; with LRFR, the percentage would be 69%. Other factors which are revealed by the FE analysis but have been ignored by existing rating guidelines, such as additional stiffness from transverse diaphragms and actual support conditions, can also contribute significantly to the difference between load ratings by FE methods and those calculated according to the LFR guidelines. It is apparent that if the customary rating practice (LFR, ASR or LRFR) suggests that a bridge is a candidate for posting, a more accurate structural analysis model should be employed to verify whether more accurate GDFs might change that decision, in view of its potential economic impact. Furthermore, it was surprising to note that, at least for the four bridges analyzed in detail in this study, the older GDFs used in the ASR and LFR rating methods yielded girder moments and shears that compared more favorably with the results of the FE models than the more recent GDFs used in the LRFD/LRFR procedures.

Table 3.2.1: Comparison of the moment distribution factors for interior girders

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>LFR/ASR</th>
<th>LRFR</th>
<th>FEM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete T</td>
<td>0.597</td>
<td>0.69</td>
<td>0.407</td>
</tr>
<tr>
<td>Concrete Skew T</td>
<td>0.757</td>
<td>0.73</td>
<td>0.482</td>
</tr>
<tr>
<td>Prestressed</td>
<td>0.818</td>
<td>0.85</td>
<td>0.521</td>
</tr>
<tr>
<td>Steel Girder</td>
<td>0.725</td>
<td>0.72</td>
<td>0.513</td>
</tr>
</tbody>
</table>

3.3 Shear Capacity Rating of Deep Reinforced Concrete Beams

The posting of the steel girder bridge in Dawson County (Bridge ID 085-0018) is based on the assessed shear capacity of its center pier cap (Figure 2.2.4). Prior to conducting the load tests, then, a series of independent analyses of the reinforced concrete pier cap for this bridge was undertaken. The pier cap behaves as a “deep beam,” in that its shear span is relatively short (its shear span/effective depth ratio, $a/d$, is approximately 1.0) in comparison to that of beams used to develop the shear capacity equations used in the current rating procedure (the approach to shear strength in ACI Standard 318 presumes that $a/d$ is greater than 3). These shear capacity equations are known to underestimate the actual shear capacity of deep reinforced concrete beams, in some cases significantly (Hawkins et al, 2005).

Recent years have seen the development of alternative models (e.g., Modified Compression Field Theory; Strut-and-Tie) to estimate shear capacity of reinforced concrete beams that address the issues above. The use of such models might enhance the rating of Bridge ID 085-0018 significantly. As a starting point to examine whether this might be the case, an independent analysis was first conducted using the current GDOT rating method, which confirmed the current posted limits. It was found that in all cases, the operating level load governed the ratings. Next, the capacity in shear was assessed using two separate procedures: a strut-and-tie (S&T) model (Tang and Tan, 2004) and finite element analysis. In the FE model, the shear capacity was assumed to be reached when steel reinforcement over the support acting as the tie of the S&T mechanism starts yielding. Figure 3.3.1, obtained from the FE analysis of the pier cap, clearly shows the development of arch action, which the S & T model captures. Table 3.3.1 summarizes the analysis of the posting limits for five GA legal loads. The S & T shear strength model leads to ratings that range from 24% (HS-20) to 30% (Tandem) higher than the current method. In turn, the S & T shear strength estimates are substantially less than the FE results. Either way, modern analysis reveals that the current posting limits for this bridge based on shear capacity appear to be conservative. Had the S & T model been used to determine the shear strength of the pier cap, the posted limits would have increased to 34 tons for HS20, 33 tons for Tandem and 44 tons for 3S2, as tabulated in Table 3.3.1.

![Figure 3.3.1 Development of arch action in deep beam](image)

To determine whether or not this observation is unique to this particular bridge, further analyses were performed of pile caps at two additional bridges identified through the Georgia DOT database – Bridge ID 083-0016 and ID 097-0032 – where the shear capacity of the pile cap also governs posting.
Table 3.3.2, developed using an HS-20 rating vehicle, reveals that the current rating procedure appears to result in excessively conservative posted loads for these bridges as well. Furthermore, the ratings are particularly conservative when the loads transmitted from the girders to the pile or pier cap are close to the support of the pile cap, leading to a short shear span. Accordingly, in the Section 6A.5.9 of the Recommended Guidelines, the strut-and-tie model is recommended for assessing the capacity in shear in rating existing bridge sub-structures.

Table 3.3.1 Analysis of current posted limits (tons) for various legal loads for Bridge ID 085-0018

<table>
<thead>
<tr>
<th>Method</th>
<th>H20</th>
<th>HS20</th>
<th>Tandem</th>
<th>3S2</th>
<th>Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>GDOT Method</td>
<td>22</td>
<td>25</td>
<td>24</td>
<td>32</td>
<td>28</td>
</tr>
<tr>
<td>Strut and tie</td>
<td>30</td>
<td>34</td>
<td>33</td>
<td>44</td>
<td>38</td>
</tr>
<tr>
<td>Finite element</td>
<td>40</td>
<td>45</td>
<td>43</td>
<td>58</td>
<td>49</td>
</tr>
</tbody>
</table>

Table 3.3.2 Analysis of current posted loads (tons) using HS20 vehicle

<table>
<thead>
<tr>
<th>Method</th>
<th>Bridge ID</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>085-0018</td>
</tr>
<tr>
<td>GDOT Method</td>
<td>25</td>
</tr>
<tr>
<td>Strut and Tie</td>
<td>34</td>
</tr>
<tr>
<td>Finite Element</td>
<td>45</td>
</tr>
</tbody>
</table>

3.4 Condition Factor, $\phi_c$

The physical condition of the bridge is reflected in the capacity rating equation (eq. 6-1 in the AASHTO LRFR Manual; eq 6A.4.2.1-1 in the Manual of Bridge Evaluation) through the condition factor, $\phi_c$. The current condition factors are related to qualitative descriptions of bridge condition – good, fair and poor. A revised set of values of $\phi_c$ that are tied to the rating factors have been developed to be consistent with the structural reliability-based evaluation philosophy embodied in the MBE and to incorporate recent developments in bridge resistance degradation modeling and comprehensive databases of bridge condition rating history.

The relation between condition rating and statistical characteristics of bridge resistance is developed by mapping the average condition rating history of non-interstate bridges in the National Bridge Inventory (Bolukbasi, et al, 2004) onto the 75-year stochastic bridge resistance model with medium degradation rate (Enright and Frangopol, 1998). The relation between bridge condition rating and age (in an average sense) is illustrated in Figure 3.4.1, in which the coefficients are obtained from regression analysis of data available in the NBI.

The bridge resistance is modeled as:

$$R(t) = R_0 g(t)$$  \hspace{1cm} (3.4.1)

\hspace{1cm} 18 The posted loads reported in this line of the table are taken from the GDOT database.
in which, $R_0$ is a random variable describing the resistance of a newly-constructed bridge and $g(t)$ is the degradation function. The mean and COV of $g(t)$ as a function of time are presented in Figure 3.4.2. By combining the information in Figures 3.4.1 and 3.4.2, one obtains a relation between condition rating and the statistics of $g(t)$. If flexural resistance is considered, $R_0$ is a lognormal variable with mean of $1.14R_n$ and COV of 13% (Nowak, 1999), where $R_n$ is the nominal flexural strength of a bridge component. Figure 3.4.3 shows the mean, $E[R/R_n]$, and COV, $V[R/R_n]$, of the normalized resistance modal as a function of condition rating.

Using these statistics, along with the load statistics for girder spans of 40 to 140 feet used for calibrating the AASHTO LRFD load factors (Nowak, 1999), one determines the values of $\varphi_c$ necessary to achieve a target reliability of 3.5 through an optimization process illustrated in Figure 3.4.4. The proposed condition factors are presented in Table 3.4.1, and in the Recommended Guidelines in Section 6A.4.2.3.

![Figure 3.4.1. Condition Rating vs Age (Bolukbasi, et al., 2004)](image)

Table 3.4.1 Proposed condition factors

<table>
<thead>
<tr>
<th>Structural Condition Rating (SI&amp;A Item 59)</th>
<th>$\varphi_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\geq 8$</td>
<td>1.00</td>
</tr>
<tr>
<td>7</td>
<td>0.95</td>
</tr>
<tr>
<td>6</td>
<td>0.85</td>
</tr>
<tr>
<td>5</td>
<td>0.75</td>
</tr>
<tr>
<td>$\leq 4$</td>
<td>0.70</td>
</tr>
</tbody>
</table>
Figure 3.4.2. Time-dependent mean and coefficient of variation of bridge capacity

Figure 3.4.3. Time-dependent statistics mean and COV of bridge flexural capacity
3.5 Direct Reliability Assessment

Bridge condition evaluation that utilizes a direct reliability assessment couples the evaluation process closely and quantitatively to the results of bridge inspections, in situ material tests and load tests. Engineering judgments and indirect information can be incorporated systematically into the reliability-based assessment framework together with the observed field data to obtain improved estimates of bridge capacity and ratings, and to identify the need (if any) for posting. This assessment level highlights the learning process through field inspections or load tests of a given bridge. Section 9 in the Recommended Guidelines furnishes explicit reliability-based rules for bridge evaluation purposes.

The current design requirements in LRFD provide the starting point for assessment through evaluation. For bridge structures, the AASHTO LRFD Bridge Specifications (2007) stipulate that,

$$1.25D + 1.5D_A + 1.75(L + I) < \phi R_n$$

in which \(D = \) dead load excluding weight of asphalt surface, \(D_A; \) \((L + I)\) represents live load including impact; \(\phi R_n\) is the design strength, where \(R_n\) is the nominal resistance; and \(\phi\) is the resistance factor which depend on the particular limit state of interest. Structural reliability methods can be used to update the load and the resistance factors for the purpose of rating for different service scenarios, leaving the bridge engineers with a safety checking procedure with which they are familiar and is consistent with bridge engineer software. The reliability-based rating method is procedurally consistent with current GDOT procedures.
Reliability-based condition assessment of existing bridge structures requires (1) structural models of the performance limits of the bridge as a structural system; (2) statistical databases to describe the important loads acting on the bridge during its service life and structural resistances, ideally in the \textit{in situ} condition; (3) a quantitative reliability assessment method to properly integrate the numerous uncertain factors that might affect bridge performance into a consistent risk-informed decision framework; and (4) a framework for revising and updating the risk-informed performance metrics during the service life of the bridge as information from periodic inspections is added to the BIM. These essential elements are discussed below.

3.5.1 Bridge system capacity

One of the key conclusions from Tasks 2 and 3, in which bridge response measurements obtained from the load tests of the four bridges were compared with the results of the pre- and post-test finite element analyses of those bridges using ABAQUS, was that the finite element modeling procedure is sufficiently accurate that it can be used to conduct “virtual” load tests of any bridge in the State of Georgia bridge inventory. These virtual tests can provide the analytical basis for developing recommendations for improving guidelines for bridge ratings using structural reliability principles. As noted in the introductory section above, such guidelines require the bridge to be modeled as a structural system in order to properly identify the performance limit states on which such guidelines are to be based.

To identify such performance limit states and to gain a realistic appraisal of the conservatism inherent in current bridge design and condition rating procedures, a series of static “pushdown” analyses of the four bridges were performed. These analyses are aimed at determining the actual structural behavior of typical bridges when loaded well beyond their design limit; as a sidelight, they provide additional information to support rational evaluation of permit load applications (section 6A.4.5 in the Manual of Bridge Evaluation). In a pushdown analysis, two rating vehicles are placed side by side on the bridge in a position that maximizes the response quantity of interest in the evaluation (maximum moment, shear, deflection, etc.). The loads are then scaled upward statically, and the performance of the bridge system is monitored. The dead weight of the bridge structure is included in the analysis. The response initially is (essentially) elastic. As the static load increases, however, elements of the bridge structure begin to yield, crack or buckle, and the generalized load-deflection behavior becomes nonlinear. If the bridge structure is redundant and the structural element behaviors are ductile, substantial load redistribution may occur. At some point, however, a small increment in static load leads to a large increment in displacement; at that point, the bridge has reached its practical load-carrying limit, and is at a state of incipient collapse.

A static pushdown analysis was performed on the four Georgia bridges identified in Chapter 2. In all four cases, two HS20 vehicles were placed side by side on two adjacent lanes at a point so as to maximize the elastic moment in the girders. These vehicle loads were scaled upward by a load factor until the point of first yield in the bridge structural system was observed. The dead load was held constant throughout the pushdown analyses. An illustration of the static pushdown analysis of the reinforced concrete T-beam Bridge ID 129-0045 is presented in Figure 3.5.1. The point of initial yielding occurs at approximately 4.31 times the HS20 design load, at a deflection of approximately 1.4 in (36 mm), which is equal to approximately 1/345 times the span. The ultimate live load capacity of the bridge is approximately 4.8 times the applied HS20 loads. It is interesting to note from Figure 3.5.1 that this 51-yr old bridge shows a considerable degree of ductility in behavior. The level of load imposed by the four fully loaded trucks during the load test is also shown in the figure; the test load was approximately 1.3 times the two side by side HS20 loads (in terms of maximum moment). Clearly, the capacity of this bridge system is substantially in excess of what a girder-based calculation would indicate.
To accelerate the FE-based reliability analyses that form the basis for the proposed load rating criteria in the *Recommended Guidelines*, efficient FE models of the four bridges also were developed using the open-source platform, OpenSees. These OpenSees models were validated using the more detailed (and costly) ABAQUS models, which had been validated in the load tests, as the system was loaded to beyond its design limit. Figure 3.5.1 shows that the behavior predicted by the two FE models is reasonably consistent. The use of the OpenSees models in bridge reliability assessment will be described in the following section.

The results of the static pushdown analyses of all four bridges are summarized in Table 3.5.1. The elastic ranges of all four bridges are in excess of 4 times the design load level, indicating the level of conservatism associated with traditional design and rating procedures.

![Figure 3.5.1 Push-down analysis of RC T-beam bridge ID 129-0045](image)

**Table 3.5.1 Analysis of bridge capacity, determined as the point of first yield**

<table>
<thead>
<tr>
<th>Bridge ID</th>
<th>County</th>
<th>Type</th>
<th>Design load</th>
<th>Load factor on design load</th>
<th>Load factor on HS20</th>
</tr>
</thead>
<tbody>
<tr>
<td>129-0045-0</td>
<td>Gordon</td>
<td>RC – T – straight – not posted</td>
<td>H-15</td>
<td>7.46</td>
<td>4.31</td>
</tr>
<tr>
<td>015-0108-0</td>
<td>Bartow</td>
<td>RC – T – skewed - posted</td>
<td>HS-15</td>
<td>6.00</td>
<td>4.50</td>
</tr>
<tr>
<td>223-0034-0</td>
<td>Paulding</td>
<td>Prestressed – straight – not posted</td>
<td>HS-20</td>
<td>5.94</td>
<td>5.94</td>
</tr>
<tr>
<td>085-0018-0</td>
<td>Dawson</td>
<td>Steel girder – straight - posted</td>
<td>H-15</td>
<td>9.93</td>
<td>5.37</td>
</tr>
</tbody>
</table>
3.5.2 Supporting databases on structural loads and capacities

As part of the effort to develop the AASHTO LRFD Bridge Specifications, extensive databases were developed to describe the in situ strength of individual bridge girders and vehicle live loads probabilistically (Nowak, 1999; Moses, 2001). [The HL-93 live load model is an outgrowth of this prior research.] That earlier research focused on the capacity of individual bridge girders; system effects were included indirectly and approximately through new girder distribution factors that were developed in the course of the project. The capacity of a bridge structural system is likely to be different from what the capacity predicted from an analysis of individual girders. In order to determine the additional level of conservatism (if any) that arises from system behavior, a finite element-based system reliability analysis of all four study bridges were conducted. Such an analysis provides additional perspective on the (unknown) level of conservatism furnished by the current generation of reliability-based condition evaluation and rating procedures embodied in the Manual for Bridge Evaluation, and has implications for the use of such methods in permit ratings for extreme vehicle loads.

![Figure 3.5.2 Lognormal fit of the bridge system resistance of the RC Bridge (ID: 129-0045)](image)

The capacity of each bridge system was determined by FE-based Monte Carlo simulation using a Latin Hypercube stratified sampling technique\(^\text{19}\) to achieve efficient coverage of the sample space with a relatively few FE analyses. The capacities of the individual girders are modeled by random variables, with statistical parameters as defined in the LRFD databases mentioned previously. The limit state of performance was taken as the point at which the bridge system exits the elastic range, as identified from its load-deflection curve (cf Figure 3.5.1). The results of the flexural capacities so determined from the

FE analysis were rank-ordered and plotted on lognormal probability paper, as illustrated in Figure 3.5.2 for the straight approach RC bridge (ID 129-0045). The lognormal distribution provides a good fit to these data. The mean and coefficient of variation in the capacity of this bridge (against first yield) are 4.31 times the HS-20 design load and 15%, respectively. The variability is of the same order as the individual girder capacities (Nowak, 1999), but the larger mean is characteristic of the beneficial system effects in a system reliability assessment. It may be appropriate to factor in this additional conservatism in bridge evaluation on a case-by-case basis.

3.5.3 Bridge system reliability assessment

The metric for acceptable performance in a reliability-based condition evaluation of an existing structural component or system is described by the limit state probability, defined as:

\[ P_f = P[R < S | H] < P_{\text{Target}} \]  

(3.5.2)

in which, \( R \) = structural capacity and \( S \) = total load effect, dimensionally consistent with \( R \). The limit state probability is often replaced by the reliability index, \( \beta \), which is related (approximately) to \( P_f \) by

\[ \beta = \Phi^{-1}(1 - P_f) > \beta_{\text{Target}} \]  

(3.5.3)

The design requirements (load combinations and resistance criteria) in the AASHTO LRFD Bridge Design Specifications (2007) were developed so as to achieve a \( \beta = 3.5 \) (based on a 75-yr service life of a bridge). This same reliability-based approach carried over to the AASHTO LRFR Manual (2003) and the Bridge Evaluation Manual (LRFR option) (2008), where the target \( \beta \) for design-level reliability (Inventory Level) is 3.5, while the target \( \beta \) for Operating Level, Legal Loads, and Routine Permit loads is 2.5.

As a matter of current interest and to provide additional context for the reliability-based recommendations in the Recommended Guidelines, a reliability analysis of the bridge ID 129-0045 structural system evaluated in the previous section was conducted. The probability distribution of the system resistance is described by the lognormal distribution in Figure 3.5.2, while the probability distribution of the live load is taken from Nowak’s weigh-in-motion study (Nowak, 1999) that supports the LRFD Bridge Design Specifications (2007). The probability of failure is obtained by convolving these distributions, leading to \( P_f = 5.4 \times 10^{-5} \). The corresponding reliability index is \( \beta = 3.87 \) (on a 75-year basis). It is interesting to note that the reliability for this 52-year old bridge is comparable to the current target for new bridges stipulated in the AASHTO LRFD Bridge Specifications (2007), suggesting that many well-maintained older bridges in the State of Georgia inventory that have been designed for lower loads nevertheless may be entirely adequate to resist modern traffic loads safely without further maintenance or administrative action.

A major difference between the reliability basis for the AASHTO LRFR Manual and Manual of Bridge Evaluation (LRFR option) and Section 9 of the Recommended Guidelines (Part II) arises from the presence of term \( H \) in eq (3.5.2), which represents what has been learned from knowledge regarding prior (successful) performance, from the in-service inspection, and from the results of whatever in situ testing has been conducted. For example, suppose that the bridge has been proof tested to a level, \( s_p \). If the bridge sustained the proof test successfully with no damage, then the bridge capacity certainly is greater than \( s_p \), even if the actual capacity remains unknown. Accordingly, eq (3.5.2) becomes,
Chapter 3  
Summary of Major Research Findings

Chapter 3  
Report of Task 4  
Part I

\[ P_f = P[R < S \mid R > s_p] < P_{\text{Target}} \]  (3.5.4)

This probability is smaller than the unconditional limit state probability on which the current bridge ratings are based because the lower tail of the resistance distribution has been removed by the bridge passing the proof load test successfully. Conversely, the load factors used in condition evaluation that are necessary to achieve a stipulated target reliability index (say, 2.5) should be less than those obtained from an unconditional reliability assessment, such as that used to develop the AASHTO Manual’s Generalized Live Load Factors for Legal Loads. As a result, modifications to that manual’s Table 6A.4.4.2.3a-1 for a specific bridge might be warranted if this successful prior behavior is to be taken into account, as discussed later in 3.5.4.2. Alternatively and preferably, Section 9 provides a straightforward method for incorporating such prior information in the rating process, provided that such information only alters the statistical parameters but not the distribution types of the load or the resistance.

In life-cycle cost/benefit analysis of maintenance, rehabilitation and repair strategies, the target probability, \( P_{\text{Target}} \), should depend on the economics of rehabilitation/repair, consequences of future outages, and the bridge rating sought. However, bridges usually are rated and posted because of concerns regarding public safety. In the circumstances, the load requirements for bridge evaluation in the Recommended Guidelines are based on the reliability targets above that have been established previously in the AASHTO LRFR Manual (2003) and the Manual for Bridge Evaluation (2008).

3.5.4 Role of in-service inspection and testing on bridge reliability

When a bridge is inspected periodically, the knowledge gained, if properly quantified, can be used to revise estimates of \textit{in situ} strength and load carrying capacity or to forecast the remaining service life of a bridge. If the condition evaluation process allows such information to be reflected in the bridge rating, there is a clear incentive to obtain quantitative \textit{in situ} measurements through modern non-destructive evaluation (NDE) techniques, such as Carpenter hammer sounding, Schmidt rebound hammer and ultrasonic pulse velocity through the term \( H \) in eq (3.5.2). The beneficial effect of aging in a well-maintained concrete bridge has already been noted. Knowledge that the structural system has withstood challenges, some of which may have exceed the design basis, during a prior service period provides additional information that should be reflected in its condition assessment (Bartlett and Sexsmith, 1991; Fu and Tang, 1995). However, NDE also contributes several sources of uncertainty to condition assessment. First, performance-degrading defects must be detected. Second, the defects must be located and measured accurately in order to determine whether they might impact future performance, which might be difficult for certain field conditions. Third, one must quantify the impact of deterioration or maintenance on strength \textit{in situ}.

A properly conducted proof load test can be an effective way to update the bridge load capacity assessment in situations where the analytical approach produces low ratings, or structural analysis is difficult to perform due to deterioration or lack of documentation (Moses and Lebet, 1994). A proof load test indicates a minimum load capacity, and thus might be used if a decision as to whether to post a bridge must be made quickly. However, it does not reveal the actual bridge capacity; nor does it provide a meaningful measure of the safety of the bridge over a projected service life. Identifying an appropriate proof test level is problematic. The test is non-informative insofar as public safety is concerned unless it

is of the same order of magnitude as the design live load (Ellingwood, 1996); conversely, too high a proof load runs the risk of damaging the bridge. The tradeoff between the information gain and the cost of damaging the bridge during the test must be factored into the decision to load test.

Probabilistic information on structural resistance is incorporated in the Level 1 (Part II, Section 6) assessment procedure through the resistance factors, but the assessment procedure itself is deterministic. The Level 2 direct reliability assessment procedure (Part II, Section 9) allows the evaluator to change the characteristic values of the variables used in the condition evaluation on the basis of the outcomes of material sampling and testing and in situ measurements and observations. The prior assumed statistics of load or/and resistance distributions are updated using in situ knowledge gathered through field inspections. Subjective judgments based on intuition, experience, or indirect information also can be incorporated systematically with the observed data to obtain balanced estimation. The updated information takes all sources of uncertainty into account, giving greater weight to the empirical (data-based) observations as their sample size grows. These updated statistics can be used directly in revising previously calculated load rating factors and bridge reliability indices. Some examples follow:

3.5.4.1 Proof load test

Proof load testing with prescribed proof load $q^*$ can be applied to structural components to screen out the lower tail of the strength distribution. Since the reliability is very sensitive to the lower tail of the strength distribution, this screening can greatly improve the revised reliability. The posterior strength distribution after successful proof load testing with proof load $q^*$, $f^*_R(r)$, can be written as:

$$f^*_R(r) = \frac{f_R(r)}{1 - F_R(q^*)}; r \geq q^*$$  \hspace{1cm} (3.5.9)

$$f^*_R(r) = 0; \hspace{1cm} r \geq q^*$$

where $f_R(r)$ and $F_R(r)$ are the prior probability density function and cumulative function of resistance respectively. If used in a reliability-based condition assessment, this strength distribution usually will result in a higher reliability index for a given rating vehicle.

3.5.4.2 Service-proven strength

Service loads on a bridge provide a proof test, of sorts, and the fact that a bridge has survived its load spectrum for a period of time indicates that it possesses a minimum level of integrity. In such a proof test, the magnitude of the proof load is unknown, but can be determined statistically. Accordingly, the lower tail of the resistance distribution is not truncated, as in eq (3.5.9), but is “thinned,” the impact of this thinning on system reliability is similar to that evidenced by a traditional proof test, although not to the same degree. For a surviving structure subjected to a sequence of random vehicle loads, the magnitude of which is described by probability distribution function $F_Q(r)$ determined using weigh-in-motion data (e.g., Nowak, 1999), the revised strength $f^*_R(r)$ can be written as (Ellingwood, 1994):
where \( f_R(r) \) and \( F_Q(r) \) are the prior probability density function of resistance and the cumulative load distribution function, respectively.

To illustrate the benefit of prior successful bridge performance on rating, consider the concrete T beam bridge (ID: 129-0045), which gave 52 years of serviceable performance. Prior to considering the benefit of successful bridge performance, the mean and COV of bridge capacity can be determined from Figure 3.4.3; the use of these statistics leads to prior reliability index \( \beta = 2.54 \). The updated distribution of resistance, as determined from Eq 3.5.10, is illustrated in Figure 3.5.3. The time-dependent influence of service load on resistance causes the lower tail of the resistance distribution to be “thinned” even after only ten years of service life. The decrease in the updated failure probability and increase in reliability index with time is illustrated in Figure 3.5.4. This increase in reliability for a service-proven structure translates to an increase in bridge capacity rating factors, as indicated in Figure 3.5.5. Rating factors with respect to HL-93 loading (inventory level) prior to and after considering the successful service of this bridge are summarized in Table 3.5.2. These results indicate a 16% increase in flexural ratings and a 40% increase in shear ratings when successful performance during the 52-year service load history is considered. The fact that the rating in shear increases more that in flexure is a result of the larger COV of the prior shear resistance distribution of the bridge.

![Figure 3.5.3 Influence of service load on updated distribution of structural resistance for the RC Bridge (ID: 129-0045)](image-url)
Figure 3.5.4 Updated failure probabilities and reliability indices for the RC Bridge (ID: 129-0045)

Table 3.5.2 Comparison of rating factors computed before and after considering service load history for the RC Bridge (ID: 129-0045)

<table>
<thead>
<tr>
<th>Rating Factor</th>
<th>Flexure</th>
<th>Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior girder</td>
<td>Exterior girder</td>
</tr>
<tr>
<td>Before updated by service load history</td>
<td>0.75</td>
<td>0.65</td>
</tr>
<tr>
<td>After updated by service load history</td>
<td>0.87</td>
<td>0.81</td>
</tr>
</tbody>
</table>
Figure 3.5.5 Updated rating factors respect to HL-93 at inventory level for the RC Bridge (ID: 129-0045)
4. SUMMARY AND CONCLUSIONS

4.1 Summary

This report summarizes a multi-year research program, conducted by a project team at the Georgia Institute of Technology under the sponsorship of the Georgia Department of Transportation, aimed at making improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed. The end product of this research program is a Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia. The Recommended Guidelines address condition assessment and evaluation by analysis, load test, or a combination of the two methods, depending on the circumstances. Consistent with the AASHTO LRFD Bridge Specifications, they have a sound basis in structural engineering and structural reliability principles, allowing them to be updated as changing circumstances (traffic demands, additional data, material deterioration, and other factors) warrant. The Recommended Guidelines are presented in a relatively simple, practical and familiar form that is suitable for implementation in routine bridge rating assessments. The research included the following four major elements: review and critical appraisal of existing bridge rating procedures; bridge load testing; advanced bridge performance analysis using finite element modeling coupled to structural reliability analysis; and development of Recommended Guidelines based on the research products of the first three elements.

This report is in two parts. Part I summarizes the project’s research accomplishments and provides an archival technical basis for the Recommended Guidelines. Part II, which follows, presents the Recommended Guidelines and commentary. The Recommended Guidelines are keyed to the LRFD option in the Manual for Bridge Evaluation, First Edition (2008); they modify selected portions of the Manual for Bridge Evaluation to make it specifically applicable to condition assessment and rating of reinforced concrete, pre-stressed concrete, and steel girder bridges in the State of Georgia. A new Section permits a direct reliability-based approach to bridge rating where circumstances warrant; the provisions in this section are somewhat more complex than those in the standard formula-driven rating process, but are likely to result in a less conservative rating if used, thus justifying the additional effort. An Appendix to Part II of the report illustrates the use of the Recommended Guidelines in specific rating situations, and compares the ratings thus obtained to those that would be obtained using the existing AS, LF and LRFR methods.

4.2 Conclusions

The AASHTO Bridge Evaluation Manual (LRFR option) extends the limit state design philosophy to the bridge evaluation process and introduces a rational method for achieving a uniform target level of safety for the bridge systems that is based on modern structural reliability principles. However, the rating calculations are based on the performance of individual structural components, with system effects taken into account through force distribution factors that may be excessively conservative. The bridge is not considered as an integrated structural system. Moreover, bridge ratings often rely on design documentation. Since each existing bridge has sustained a unique spectrum of service loads in its as-built condition, a generalized analytical rating procedure which does not incorporate in situ data may result in inaccurate bridge ratings. Usually the ratings will be more conservative than necessary; occasionally, they may not be sufficiently conservative for public safety.
Advancing current bridge evaluation and rating practices requires a better understanding of bridge system behavior, better utilization of available in situ data, and better modeling of the load process and other stochastic processes such as fatigue, corrosion and concrete aging. Implementation of the Recommended Guidelines presented in Part II in Georgia is likely to result in less conservative bridge ratings and posting requirements for most bridges in the State. The main reasons for the less conservative ratings are: more realistic girder distribution factors; improved procedure for permitting the use of in situ material properties through an enhanced statistically-based sampling plan; a newly derived condition factor, $\phi_c$, which is keyed to the latest bridge inspection; and the use of structural evaluation methods (e.g., strut-and-tie analysis; finite element analysis) that capture the mechanics of structural behavior more accurately in limit states that govern the rating process (e.g., pier cap shear capacity; bridge system level capacity).

ACKNOWLEDGEMENTS

The research reported herein was supported by the Georgia Department of Transportation under Award No.RP-05-01, with Mr. Rick Deaver and Dr. Stanley Kim as program managers. This support is gratefully acknowledged. The authors would like to acknowledge the contributions of Messrs. Benjamin Rabun, Paul Liles, and Kevin Schwartz for their assistance in working with the bridge maintenance records in the Georgia Bridge Inventory, and in identifying the four bridges selected for testing, analysis and proof of concept. However, the views expressed in this report are solely those of the authors, and may not represent the positions of the Georgia DOT.
BIBLIOGRAPHY


PART II

RECOMMENDED GUIDELINES FOR CONDITION ASSESSMENT, EVALUATION AND RATING OF BRIDGES IN GEORGIA
PREFACE

These recommended *Guidelines for the Condition Assessment and Evaluation of Existing Bridges in Georgia*, hereinafter referred to as the *Guidelines*, are keyed to the LRFD option in the *Manual for Bridge Evaluation, First Edition* (2008), hereinafter referred to as the *Manual*. The *Guidelines* modify selected portions of the *Manual* to make it specifically applicable to condition assessment and rating of reinforced concrete, pre-stressed concrete and steel girder bridges in the State of Georgia. A non-mandatory *Commentary* furnishes background information and references for the benefit of the engineer seeking further understanding of the derivation and limitations associated with the provisions in the *Guidelines*.

The reader is cautioned that professional judgment must be exercised when data or recommendations in the *Guidelines* are applied. Condition assessment and evaluation of existing structural components and systems is within the scope of expertise of a licensed engineer with demonstrated competence for the application of engineering principles in the evaluation of a particular bridge structure or substructure.

For ease of interpretation and use by the Bridge Engineer, these *Recommended Guidelines* have been prepared using the strikeout/underline format that is customary in specifications and standards development. Provisions that represent *additions* to the provisions in the *Manual* are underlined. Provisions in the *Manual* that are replaced by provisions in the *Guidelines* are shown as *struck-out*. 
SECTION 1
INTRODUCTION

Add the following to Section 1 of the Manual for Bridge Evaluation immediately before 1.1:

Section 1 defines the purpose, scope and applicability of the Guidelines, presents definitions and terminology used herein, and identifies necessary supporting codes, standards and specifications, and other documents. The section is organized as follows:

1.1 PURPOSE
1.2 SCOPE
1.3 APPLICABILITY
1.4 QUALITY MEASURES
1.5 DEFINITIONS AND TERMINOLOGY
1.6 REFERENCES

1.1 PURPOSE

Replace 1.1 with the following:

These Guidelines provide uniformity in methods used for evaluating the physical condition and capacity of girder bridges, for determining bridge ratings for inventory, operating and legal loads using the Load and Resistance Factor Rating method, and for identifying the need for posting existing bridges in the State of Georgia, when used in conjunction with the American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation, First Edition (2008).

1.3 APPLICABILITY

Replace 1.3 with the following:

The Guidelines are intended for evaluating highway girder bridges commonly used in the State of Georgia, including reinforced concrete tee, prestressed and steel girder bridges, which are subjected primarily to permanent gravity loads and vehicular loads and in which the spans are 120 ft or less. The Guidelines do not apply to wood bridges. Nor do the Guidelines apply to the evaluation of existing bridges for extreme events such as earthquake, vessel collision, wind, flood, ice, or fire effects. Load capacity rating of movable bridges, suspension spans, cable-stayed bridges, prestressed concrete segmental bridges, curved girder bridges truss bridges and other complex bridges may involve additional considerations and loads not specifically addressed in these Guidelines and the rating procedures should be augmented with additional evaluation criteria where required.
1.6 REFERENCES

Add the following references:


Add the following to Section 2 of the Manual for Bridge Evaluation just before 2.1:

Section 2 provides guidance on building bridge records and reports which make up a complete bridge file. The section is organized as follows:

2.1 GENERAL
2.2 COMPONENTS OF BRIDGE RECORDS
2.3 INVENTORY DATA
2.4 INSPECTION DATA
2.5 CONDITION AND LOAD RATING DATA
2.6 LOCAL REQUIREMENTS

2.4 INSPECTION DATA

2.4.1 General

Add the following item to 2.4.1:

(7) Deterioration Condition. Quantitative condition assessment of bridges should establish whether deterioration is happening and, if so, the impact of that deterioration on the bridge load carrying capacity with respect to its original condition. This information should be provided in a qualitative manner, including a description of the locations and types of deterioration and levels of damage, and should be supported with photographs and in situ testing whenever possible.

Modify 2.5.2 as follows:

2.5.2 Revised Condition and Load Rating Data

When maintenance or improvement work or change in strength of members or dead load has altered the condition or capacity of the structure, the safe load capacity should be recalculated and should be added to the bridge information database.
SECTION 3
BRIDGE MANAGEMENT SYSTEMS

Add the following to Section 3 of the Manual for Bridge Evaluation just before 3.1:

Section 3 addresses the objectives and components of bridge management systems, and introduces common ingredients of effective bridge management. The section is organized as follows:

3.1 INTRODUCTION
3.2 OBJECTIVES OF BRIDGE MANAGEMENT SYSTEMS
3.3 COMPONENTS OF A BRIDGE MANAGEMENT SYSTEM
3.4 NATIONAL BRIDGE MANAGEMENT SYSTEMS
3.5 REFERENCES

3.3 COMPONENTS OF A BRIDGE MANAGEMENT SYSTEM

3.3.2 Data Analysis

3.3.2.1 Condition data analysis

Add the following as the fifth paragraph of 3.3.2.1:

In addition to material deterioration, existing bridges may be at risk from factors such as increases in traffic and deficiencies in the original design. Some bridges may have been designed for loads that are no longer adequate due to traffic changes, but may not necessarily have developed any significant or noticeable signs of distress because extreme load conditions and other unfavorable circumstances or service conditions may not yet have occurred. For this reason, bridge capacity ratings (Section 6) are an additional source of data that should be incorporated in bridge management systems. Such data can guide the selection of optimal strategies for repair, rehabilitation and replacement that maximize the net benefit to the public.
Add the following to Section 4 of the Manual for Bridge Evaluation just before 4.1:

Section 4 covers methods and frequencies of bridge inspection, addresses safety issues for both inspection personnel and the public, and provides guidelines and procedures for conducting inspections and making field measurements for bridge components and systems. The section is organized as follows:

4.1 GENERAL
4.2 TYPES
4.3 FREQUENCY
4.4 QUALIFICATION AND RESPONSIBILITY OF INSPECTION PERSONNEL
4.5 SAFETY
4.6 PLANNING, SCHEDULING, AND EQUIPMENT
4.7 INSPECTION FORMS AND REPORTS
4.8 PROCEDURES
4.9 SPECIAL STRUCTURES
4.10 FATIGUE-PRONE DETAILS
4.11 FRACTURE-CRITICAL MEMBERS
4.12 DATA COLLECTION FOR LOAD RATING
4.13 REFERENCES

4.3 FREQUENCY

Add the following as the third paragraph in 4.3:

For bridges that have been proof-load-tested, minor structural damage may have occurred during the test that may not be immediately apparent. Under unfavorable conditions, such damage subsequently may cause accelerated deterioration of the bridge structure. Therefore, bridges that have been subjected to proof testing procedures should be thoroughly inspected at 24 hours, at 6 months and at one year following the test, before resuming inspections at normal inspection intervals.

4.8 PROCEDURES

4.8.2 Substructure

4.8.2.4 Pile Bents

Revise paragraph 6 of 4.8.2.4 as follows:
Examine Steel and concrete piles shall be examined both in the splash zone and below the water surface for corrosion and deterioration. Steel piles shall be examined for evidence of general or flange-local buckling.

Add the following paragraph at the end of 4.8.2.4:

Article 6A.5.9 stipulates that shear need not be considered when rating for design or legal loads unless visible signs of shear distress are observed. Accordingly, careful attention should be given during inspection to shear-critical members, such as reinforced concrete pier or pile caps or corbels, for possible indications of deficiencies in shear capacity, such as diagonal cracks, relative displacement across a crack or spalled concrete along diagonal cracks. Special attention should be given to pier caps in which the loads from the girders are not lined up with the supporting piers or piles.

4.8.3 Superstructure

4.8.3.1 Steel Beams, Girders and Box Sections

Add the following to 4.8.3.1:

The inspector shall check that the reinforced concrete deck is acting composite with the steel girder, if the design and construction documents show that composite action was intended by the designer. Movement between the bottom of the deck and the top flange of supporting girders or loss of camber may be indicative of a breakdown in the composite action.

4.12 DATA COLLECTED FOR LOAD RATING

4.12.4 Inspection for Resistance

Add the following as commentary to section 4.12.4:

C 4.12.4

In order to reflect the in situ physical condition of a bridge in estimating its structural resistance, modern non-destructive evaluation (NDE) techniques may be used to collect usable data for capacity rating. Some of the NDE methods available, such as rebound hammer and ultrasonic pulse velocity, will be discussed in more detail in the Section 5, Material Testing. The benefit and the cost of introducing these technologies should be considered in designing in-service inspection programs and in making decisions regarding maintenance.
SECTION 5
MATERIAL TESTING

Add the following to Section 5 of the Manual for Bridge Evaluation just before 5.1:

Section 5 identifies in situ test methods for identifying defects in reinforced concrete, steel and timber bridge structures and estimating their strength, provides guidelines for sampling materials from structures in situ, and provides guidance for the interpretation of test results. The section is organized as follows:

5.1 GENERAL
5.2 FIELD TESTS
5.3 MATERIAL SAMPLING
5.4 LABORATORY TESTS
5.5 INTERPRETATION AND EVALUATION OF TEST RESULTS
5.6 TESTING REPORTS
5.7 REFERENCES

5.3 MATERIAL SAMPLING

Add the following provisions to 5.3:

5.3.1 General Requirements

Samples for defining engineering properties to be used for load rating purposes shall be representative of the material property population for which the nominal values are to be calculated (AASHTO R 9, 2005). These samples shall be taken from a low-stress area of the bridge structure. A minimum of five (5) samples, free of outliers in accordance with Article 5.3.2, shall be retained for interpreting and evaluating test results in accordance Article 5.5. The results of this analysis shall become part of the Bridge Management System (BMS).

5.3.2 Treatment of Outliers

The data being analyzed shall be screened for outliers using the Maximum Normed Residual (MNR) method as described in Article 5.3.2.1. A sample is identified as an outlier by this method if its value has an absolute deviation from the sample mean which, when compared to the sample standard deviation, is too large to be due to chance. This method detects one outlier at a time; hence the significance level pertains to a single decision.

5.3.2.1 For a sample of size \( n \), arrange the data values \( \{x_1, x_2, x_3, ..., x_n\} \) in order of increasing magnitude with \( x_n \) being the largest value. Calculate the MNR statistic
as the maximum absolute deviation from the sample mean ($\bar{x}$) divided by the sample standard deviation ($s_{n-1}$):

$$MNR = \max \left| \frac{x_i - \bar{x}}{s_{n-1}} \right|$$  \hspace{1cm} (5.3.2.1-1)

5.3.2.2 Calculate the critical $MNR$ value, $CV$, based on a 5% significance level using the following approximation:

$$CV \approx \left( 2 - \frac{8}{5\sqrt{n}} \right)^2$$  \hspace{1cm} (5.3.2.2-1)

5.3.2.3 There are no outliers in the sample of observations if $MNR \leq CV$. If the $MNR$ statistic is found to be greater than $CV$, then the MNR shall be denoted a possible outlier. The possible outlier shall be investigated to determine whether there is an assignable cause for removing it from the data set. If no cause can be found, it shall be retained in the data set. If an outlier is clearly erroneous, it can be removed after careful consideration, provided that the decision to remove the value is documented as part of the data analysis report. If an outlier is removed from the dataset, the sample mean and standard deviation shall be recalculated. This process shall be repeated until the sample of observations becomes outlier-free.

5.5 INTERPRETATION AND EVALUATION OF TEST RESULTS

Add the following to 5.5:

Material property values derived from test results to be used in computing the bridge component nominal capacity shall be statistically based and shall represent the 75 percent lower confidence on the 10th-percentile value of a specified population. This value can be computed as follows:

$$x_k = \Omega \bar{x}$$  \hspace{1cm} (5.5-1)

where

$$\Omega = 1 - kV_x$$  \hspace{1cm} (5.5-2)

In the above equations,
\(x_R\) = reference property (characteristic value);
\(x\) = sample mean (average);
\(V_s\) = sample coefficient of variation;
\(\Omega\) = data confidence factor that accounts for the uncertainty associated with a finite sample size; and
\(k\) = factor that depends on the confidence level and sample size. Table 5.5-1 provides factors \(k\) appropriate for estimating the 10th percentile of the distribution.

Table 5.5-1 Factors \(k\) for the lower 10th percentile estimates

<table>
<thead>
<tr>
<th>Sample Size, (n)</th>
<th>(k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1.961</td>
</tr>
<tr>
<td>6</td>
<td>1.860</td>
</tr>
<tr>
<td>7</td>
<td>1.791</td>
</tr>
<tr>
<td>8</td>
<td>1.740</td>
</tr>
<tr>
<td>9</td>
<td>1.702</td>
</tr>
<tr>
<td>10</td>
<td>1.671</td>
</tr>
<tr>
<td>11</td>
<td>1.646</td>
</tr>
<tr>
<td>12</td>
<td>1.624</td>
</tr>
<tr>
<td>13</td>
<td>1.606</td>
</tr>
<tr>
<td>14</td>
<td>1.591</td>
</tr>
<tr>
<td>15</td>
<td>1.577</td>
</tr>
<tr>
<td>16</td>
<td>1.566</td>
</tr>
<tr>
<td>17</td>
<td>1.554</td>
</tr>
<tr>
<td>18</td>
<td>1.544</td>
</tr>
<tr>
<td>19</td>
<td>1.536</td>
</tr>
<tr>
<td>20</td>
<td>1.528</td>
</tr>
<tr>
<td>21</td>
<td>1.520</td>
</tr>
<tr>
<td>22</td>
<td>1.514</td>
</tr>
<tr>
<td>23</td>
<td>1.508</td>
</tr>
<tr>
<td>24</td>
<td>1.502</td>
</tr>
<tr>
<td>25</td>
<td>1.496</td>
</tr>
</tbody>
</table>

5.6 REFERENCES

Add the following references:

SECTION 6
LOAD AND RESISTANCE FACTOR RATING

Add the following to Section 6 of the Manual for Bridge Evaluation just before 6.1:

This section provides a methodology for load rating a bridge that is consistent with the load and resistance factor design philosophy of the AASHTO LRFD Bridge Design Specifications. The Guidelines herein shall be used with Section 6A of the Manual – Load and Resistance Factor Rating – and provide essentially uniform reliability in bridge ratings. Detailed guidelines that apply to concrete, steel and wood bridges within the Scope of the Guidelines are presented.

This section is organized as fellows:

6A.1 INTRODUCTION
6A.2 LOADS FOR EVALUATION
6A.3 STRUCTURAL ANALYSIS
6A.4 LOAD-RATING PROCEDURES
6A.5 CONCRETE STRUCTURES
6A.6 STEEL STRUCTURES
6A.7 WOOD STRUCTURES
6A.8 POSTING OF BRIDGES
6A.9 SPECIAL TOPICS MASONRY ARCHES AND HISTORICAL BRIDGES
6A.10 REFERENCES

6A.1 INTRODUCTION

6A.1.6 Evaluation methods

Replace the last two sentences in the section of 6A.1.6 as follows:

Only load and resistance factor rating of bridges by analysis is presented in this section. Load testing and safety evaluation for special cases are discussed in Sections 8 and 9, respectively.

6A.3 STRUCTURAL ANALYSIS

6A.3.4 Analysis by Field Testing

Add the following as the second paragraph of 6A.3.4:

Measurements from load tests need to be carefully interpreted before being utilized in rating calculations. Since loading applied in a diagnostic load test generally is kept well below the extreme load level of concern in the design limit state, possible causes of beneficial structural behavior indicated by the test
measurements, such as end fixity, load distribution and composite action, should be carefully examined before incorporating them into the load rating analysis. However, load tests involve large cost, which need to be systematically considered beforehand.

6A.4 LOAD-RATING PROCEDURES

6A.4.2 General Load-Rating Equation

6A.4.2.1 General

*Revise the lower limit on the product \( \varphi_s \varphi_c \), as follows:*

Where the following lower limit shall apply:

\[
\varphi_s \varphi_c \geq 0.85 \text{ or } 0.70
\]

(6A.4.2.1-3)

6A.4.2.3 Condition Factor: \( \varphi_c \)

*Modify 6A.4.2.3 and its commentary as follows:*

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles. The condition factor shall be determined in accordance with Table 6A.4.2.3-1.

Table 6A.4.2.3-1  Condition Factor: \( \varphi_c \)

<table>
<thead>
<tr>
<th>Structural Condition of Member</th>
<th>( \varphi_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good or satisfactory</td>
<td>1.00</td>
</tr>
<tr>
<td>Fair</td>
<td>0.95</td>
</tr>
<tr>
<td>Poor</td>
<td>0.85</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Structural Condition Rating (SI&amp;A Item 59)</th>
<th>( \varphi_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \geq 8 )</td>
<td>1.00</td>
</tr>
<tr>
<td>7</td>
<td>0.95</td>
</tr>
<tr>
<td>6</td>
<td>0.85</td>
</tr>
<tr>
<td>5</td>
<td>0.75</td>
</tr>
<tr>
<td>( \leq 4 )</td>
<td>0.70</td>
</tr>
</tbody>
</table>

C6A.4.2.3

The physical condition of the bridge is reflected in the capacity rating equation (eq. 6A.4.2.1-1) through the condition factor, \( \varphi_c \). The revised
values of $\varphi_c$ have been developed to be consistent with the structural reliability-based evaluation philosophy embodied in the Manual and to incorporate recent advances in modeling degradation of bridge resistance and comprehensive databases of bridge condition rating history.

6A.4.4 Legal Load Rating

6A.4.4.2 Live Loads and Load Factors

6A.4.4.2.3 Generalized Live-Load Factors: $\gamma_L$

6A.4.4.2.3a Generalized Live-Load Factors for Routine Commercial Traffic: $\gamma_L$

Modify 6A.4.4.2.3a and its commentary as follows:

Table 6A.4.4.2.3a-1 Generalized Live-Load Factors for Routine Commercial Traffic: $\gamma_L$

<table>
<thead>
<tr>
<th>Traffic Volume (one direction)</th>
<th>Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unknown</td>
<td>1.80</td>
</tr>
<tr>
<td>ADTT $\geq$ 5000</td>
<td>1.80</td>
</tr>
<tr>
<td>ADTT = 1000</td>
<td>1.65</td>
</tr>
<tr>
<td>ADTT $\leq$ 100</td>
<td>1.40</td>
</tr>
</tbody>
</table>

Exceptions: For HS20 or Tandem vehicles, the live load factor shall be $\gamma_L$ = 1.50, when ADTT $\geq$ 5000; $\gamma_L$ = 1.35, when ADTT = 1000; and $\gamma_L$ = 1.15, when ADTT $\leq$ 100.

C 6A.4.4.2.3a

Add the following to C 6A.4.4.2.3a:

The live load factors in Table 6A.4.4.2.3a-1 were calibrated for operating level reliability based on a 5-year exposure using AASHTO legal loads (Type3, 3S2, and 3-3) as reference vehicles. These factors, therefore, apply to AASHTO legal load ratings and to ratings for state legal loads that have only minor variations from the AASHTO legal loads. Using these factors with rating vehicles that are more severe than AASHTO legal loads may lead to unduly conservative bridge ratings. Load effects from HS20 and Tandem vehicles used as Georgia legal loads are, on average, 20% higher than those of AASHTO legal loads for bridges similar to those considered in these Guidelines; therefore it is permitted to reduce live load factors applied to HS20 and Tandem Georgia legal loads to eliminate undue conservatism.
6A.5 CONCRETE STRUCTURES

6A.5.2 Materials

6A.5.2.1 Concrete

*Revise Table 6A.5.2.1-1 as follows:*

Table 6A.5.2.1-1 Minimum Compressive Strength of Concrete by Year of Construction.

<table>
<thead>
<tr>
<th>Year of Construction</th>
<th>Compressive Strength, $f'_c$, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prior to 1959</td>
<td>2.5</td>
</tr>
<tr>
<td>1959 and later - 1970</td>
<td>3.0</td>
</tr>
<tr>
<td>1971 and later</td>
<td>3.5</td>
</tr>
</tbody>
</table>

*Revise the last paragraph of 6A.5.2.1 as follows:*

Where the quality of the concrete is uncertain, cores should be taken for mechanical property testing. Where mechanical properties have been established by testing, the nominal value for strength is typically taken as the mean of the test values minus 1.65 standard deviations to provide 95 percent confident limit. Average test values should not be used for evaluation. Guidance on material sampling for bridge evaluation is provided in Section 5 of these Guidelines.

6A.5.2.2 Reinforcing Steel

*Revise Table 6A.5.2.2-1 as follows:*

Table 6A.5.2.2-1 Yield Strength of Reinforcing Steel.

<table>
<thead>
<tr>
<th>Year of Construction</th>
<th>Yield Strength, $f_y$, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unknown steel constructed prior to 1954</td>
<td>33.0</td>
</tr>
<tr>
<td>Structural Grade</td>
<td>36.0</td>
</tr>
<tr>
<td>Billet or intermediate grade, Grade 40, and unknown steel constructed during or after between 1954 and 1970</td>
<td>40.0</td>
</tr>
<tr>
<td>Unknown steel constructed 1971 and later</td>
<td>60.0</td>
</tr>
<tr>
<td>Rail and hard grade, Grade 50</td>
<td>50.0</td>
</tr>
<tr>
<td>Grade 60</td>
<td>60.0</td>
</tr>
</tbody>
</table>
6A.5.2.3 Prestressing steel

*Revise 6A.5.2.3 as follows:*

Where the tensile strength of the prestressing strand is unknown, the values specified in Table 6A.5.2.3-1 based on the date of construction may be used.

6A.5.9 Evaluation for Shear

*Revise the first paragraph of 6A.5.9 as follows:*

Reinforced girders, pier caps and other structural components where shear capacity may be of concern in bridges designed prior to 1970, or in which the H-15 truck was the basis for structural design, shall be checked for shear under design and legal loads. It is permitted to use a strut-and-tie analysis when the shear span is less than or equal to twice the effective depth of the component. The shear capacity of other existing reinforced and prestressed concrete bridge members should need only be evaluated for permit loads, provided that they show no visible signs of shear distress.

*Add the following paragraph as the first paragraph in C6A.5.9:*

The strength evaluation of RC pier caps and prestressed concrete members in shear may benefit from modern methods for evaluating shear capacity, such as the strut-and-tie method or modified compression field theory (MCFT). Typical reinforced concrete pier caps having relatively short shear spans behave structurally as deep beams; in such cases, the assumptions used in developing traditional shear equations are no longer valid, and the shear transfer mechanism is better modeled by a tied arch. Traditional shear equations also do not capture the positive effects of pre-stressing on the shear capacity of prestressed concrete members.

*Add the following new section 6A.5.14 and commentary:*

6A.5.14 Effects of Deterioration on Load Rating

Structural deterioration in the form of corrosion of steel or steel reinforcement and cracking of concrete shall be considered in developing load ratings for bridges.

C 6A.5.14

A deteriorated structure may behave differently from the structure as originally designed and different failure modes may govern its load capacity. Corrosion of the embedded reinforcing steel is the major type of deterioration that causes loss of moment capacity of RC members. Effective field inspection data should be used to quantify this type of loss. Furthermore, reinforcement at a section should be
considered effective only to the extent that stress in the steel can be developed due to bar anchorage. Otherwise, the effective steel area used to calculate the flexural capacity must be reduced in proportion to the reduction of steel stress that can be developed.

When there is evidence of shear distress, an evaluation of the remaining shear capacity of the member requires professional engineering judgment. If inspection reveals vertical displacement across a diagonal crack or spalled concrete along a diagonal crack, the contribution of the concrete to the shear capacity should be neglected.

6A.6.2 Materials

6A.6.2.1 Structural Steels

Revise Table 6A.6.2.1-1 as follows:

<table>
<thead>
<tr>
<th>Year of Construction</th>
<th>Minimum Yield Point or Minimum Yield Strength, $F'_y$, ksi</th>
<th>Minimum Tensile Strength, $F'_t$, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prior to 1905</td>
<td>26</td>
<td>52</td>
</tr>
<tr>
<td>1905 through 1936</td>
<td>30</td>
<td>60</td>
</tr>
<tr>
<td>After 1936 through 1970</td>
<td>33</td>
<td>66</td>
</tr>
<tr>
<td>1971 and after</td>
<td>36</td>
<td>58</td>
</tr>
</tbody>
</table>

Revise the third paragraph of 6A.6.2.1 as follows:

In case where initial evaluation suggests load capacity inadequacies, or there is doubt about the nature and quality of a particular steel, material, the mechanical properties should be verified by testing. Mechanical properties of the material steel should be determined based on coupon tests. Average test values should not be used for evaluation. Guidance on material sampling for bridge evaluation is provided in Section 5 of this Manual.

6A.8 POSTING OF BRIDGES

6A.8.3 General Load-Rating Equation

Revise paragraphs 3 and 4 of 6A.8.3 as follows:

When the RF is between 0.3 and 1.0 for any legal vehicle, then the following equation should be used to establish the safe posting load for that vehicle type:

$$\text{Safe Posting Load} = W \times RF$$  \hspace{1cm} (6A.8.3-1)
where:

\[ RF = \text{Legal load rating factor} \]
\[ W = \text{Weight of rating vehicle} \]

When the RF for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the span. When the RF falls below 0.3 for all three AASHTO legal trucks, then span should be closed for traffic.

Delete Fig. 6A.8.3-1

6A.9 SPECIAL TOPICS MASONRY ARCHES AND HISTORIC BRIDGES

Add the following to Section 6A.9 of the Manual for Bridge Evaluation just before 6A.9.1:

This section permits the Allowable Stress load rating method to be used for unreinforced masonry arch bridges and also addresses safety and maintenance-related issues pertaining to historic bridges.

Sections A6.9.1 and A6.9.2 in the Manual are deemed sufficient, and no additions or deletions are proposed.

6A.10 REFERENCES

Add the following references:


SECTION 7

FATIGUE EVALUATION OF STEEL BRIDGES

This section is organized as fellows:

7.1 LOAD-INDUCED VERSUS DISTORSION-INDUCED FATIGUE
7.2 LOAD-INDUCED FATIGUE-DAMADGE EVALUATION
7.3 DISTORSION-INDUCED FATIGURE EVALUATION
7.4 FATIGUE CONTROL FOR OLDER BRIDGES
7.5 REFERENCES

The provisions in the Manual for Bridge Evaluation are deemed sufficient, and no additions or deletions are proposed.
Bridge rating by analysis usually underestimates the actual load capacity of a bridge due to factors that are not normally considered in routine design and evaluation by analysis. Nondestructive load testing provides an improved estimate of capacity and may avoid unnecessary or ill-advised posting. Section 8 describes the role of non-destructive load testing in bridge evaluation, distinguishing between diagnostic tests, which are performed to determine response characteristics of the bridge, and proof testing, which is used to establish its maximum safe load capacity. The section also provides general procedures for performing load tests and for using the test results to calculate bridge ratings.

Section 8 is organized as follows:

8.1 INTRODUCTION
8.2 FACTORS WHICH INFLUENCE THE LOAD-CARRYING CAPACITY OF BRIDGES
8.3 BENEFIT OF NON-DESTRUCTIVE LOAD TESTS
8.4 TYPES OF NON-DESTRUCTIVE LOAD TESTS
8.5 LOAD TEST MEASUREMENTS
8.6 CIRCUMSTANCES IN WHICH WHEN NOT TO LOAD TESTING MAY BE WARRANTED
8.7 BRIDGE SAFETY DURING LOAD TESTS
8.8 LOAD RATING THROUGH LOAD TESTING
8.9 USE OF LOAD TEST IN PERMIT DECISION
8.10 SERVICEABILITY CONSIDERATIONS
8.11 REFERENCES
APP. 8 GENERAL LOAD TESTING PROCEDURES

8.2 FACTORS WHICH INFLUENCE THE LOAD-CARRYING CAPACITY OF BRIDGES

Add the following new section to 8.2:

8.2.7 Effect of Skew

If the bridge is skewed, the maximum moments in the girders for an applied load are reduced from the maximum girder moments in a bridge with a straight road approach. Moreover, the deflections of a skewed bridge are generally smaller than those of a bridge with a straight approach. However, the effect of skew is relatively small on girder bridges until the angle of skew is greater than 45 degrees.

8.5 LOAD TEST MEASUREMENTS

Add the following paragraph as the third paragraph in 8.5:
Strain measurements depend strongly on the section properties at the location where the gage is mounted. Displacement measurements reflect the overall (global) behavior of the bridge. Therefore, for determining the overall flexural behavior of reinforced concrete girders, displacement measurements should always be taken, in addition to strain measurements, because local cracking and the non-homogeneity of concrete material may confound the measured strains.

Add the following Commentary to 8.5

C 8.5 Strain Measurements

Measurements in situ of strain in reinforced or prestressed concrete members can be unreliable due to the formation of cracks under or in proximity to the gage and non-homogeneity of the concrete. Measurement uncertainties can be mitigated, to a degree, by increasing the gage length or using supplementary independent measurement technologies. The Engineer is cautioned to give this measurement uncertainty due consideration in specifying load test instrumentation.

8.6 CIRCUMSTANCES IN WHICH WHEN NOT TO LOAD TESTING MAY BE WARRANTED

Replace Article 8.6 in the Manual with Articles 8.6.1 and 8.6.1 and their commentaries that follow:

8.6.1 When to Load Test

The following is a list of situations for which load testing may be beneficially for bridge rating:

• Small span bridges where required traffic effects (i.e. the test loading) are easily simulated;

• Older bridges of archaic construction, for which design documentation is unavailable and structural modeling is difficult;

• Bridges with internal structural complexities such as transverse girders;

• Bridges in good physical condition and having analytical rating factor between 0.6 - 0.8.

Add the following Commentary to 8.6.1

C 8.6.1

If the results of an inspection or evaluation, e.g. Condition Rating less or equal to 4 or Load Rating Factor less or equal to 0.3, lead to the conclusion that a bridge is deficient and that posting, strengthening, or replacement may be appropriate, a more detailed assessment of the capacity of the bridge through a non-destructive load test may be warranted.
Posting/strengthening/replacement of any bridge is expensive, and if the bridge is on a major traffic artery, the expense is magnified by traffic restriction due to the posting limit or the inconvenience created by the construction process of replacement or strengthening. Further testing and analysis often is justified on cost/benefit grounds. The question of whether the potential benefits from the results of such testing and analysis will justify the time, effort, and expense involved must be answered for each particular condition assessment situation.

8.6.2 When Not to Load Test

The following conditions could render a bridge an unsuitable candidate for load testing:

- The cost of testing reaches or exceeds the cost of bridge strengthening or the estimated cost of posting the bridge according to its analytical rating;
- Pretest evaluation shows that the load test is unlikely to show the prospect of improvement in estimated load-carrying capacity;
- According to calculations, the bridge cannot sustain even the lowest level of load;
- There is a possibility of sudden failure (shear or fracture);
- Load test may be impractical because of access difficulties or site traffic conditions.

8.8 LOAD RATING THROUGH LOAD TESTING

8.8.3 Proof Load Tests

Add the new section 8.8.3.4 and commentary to 8.8.3:

8.8.3.4 Bridge Evaluation Following Proof Test

Special Inspections as defined in Section 4.3 shall be performed 24 hr, 6 months and 1 yr following the proof test for bridges. If such inspections indicate no apparent deficiencies, it is permitted to resume Routine Inspections at the regular scheduled intervals.

C 8.8.3.4

A successful proof test provides a lower bound on the true capacity of the bridge and hence on its rated capacity. Structural damage during a properly conducted proof test is unlikely, and the Manual requires that a proof test be halted if distress becomes apparent prior to reaching the target proof load. The likelihood of damage increases if the proof test is used as a basis for incorporating ratings for heavy permit vehicles. Should minor structural damage occur, it initially may be concealed but subsequently may lead to accelerated deterioration of the bridge structure. Accordingly, Special Inspections should be performed at more frequent intervals immediately following completion of the proof
test until it can be established that no concealed damage occurred. If such inspections indicate no apparent deficiencies within a 12-month period following the proof test, Routine Inspections can be resumed at the regular scheduled intervals.

8.11 REFERENCES

*Add the following references:*


SECTION 9
DIRECT RELIABILITY-BASED SAFETY ASSESSMENT OF BRIDGES


Because the entire section is new, the strikeout/underline format followed in Sections 1 through 8 is not used in Section 9.

Section 9 provides an alternative bridge load rating technique that permits direct use of reliability indices in the rating process and appreciably facilitates the systematic inclusion of in situ data into a reliability based evaluation framework. The section is organized as follows:

9.1 GENERAL
9.2 DIRECT RELIABILITY ASSESSMENT
9.3 REFERENCES

9.1 GENERAL

Subject to the approval of the Bridge Owner, it is permitted to develop the bridge rating using an alternative evaluation procedure that allows a direct use of reliability indices ($\beta$) in the bridge rating process. This section presents a standardized method for developing such ratings.

The following circumstances may lead a Bridge Owner to consider or adopt the direct reliability analysis method in rating a bridge:

a) Bridges having live loading characteristics that differ markedly from the descriptions contained in the Manual;

b) Well-maintained bridges that show no visible distress after many years in service;

c) Bridges with material properties or levels of deterioration that differ from those considered herein;

d) Bridges where failure may have a more significant consequence on the community served than typical bridges considered in the Manual;
e) Bridges for which a variety of hazards, such as wind, ice, earthquake, etc., must be considered;

f) Bridges in which strengths have been determined by in situ material testing, by load testing, or other means such that more accurate knowledge of likely behavior is available.

g) Bridges in which the Bridge Owner is using calculated risks as part of an overall Bridge Safety Management System.

The Engineer using the direct reliability analysis method must be familiar with basic structural reliability methods.

C 9.1 General

The provisions in Section 9 provide guidelines for risk-informed bridge assessment. The direct safety assessment method is intended for use in special bridge evaluation circumstances, where approved by the Bridge Owner. There are a number of circumstances where the direct safety assessment method is likely to yield benefits in bridge rating. Such circumstances include bridges with design characteristics that are not reflected in the extensive database on which the provisions in the Manual was developed, bridges in which loads other than traffic loads may be a significant safety consideration, and bridges for which in-service inspection and maintenance have provided extensive supplementary information on structural behavior that might be utilized effectively in bridge rating. On the other hand, there is little benefit to be gained in using the direct assessment method in lieu of the conventional load and resistance factor method when the target reliability index and statistical parameters are the same as those assumed in the calibration of evaluation factors presented in the Manual and these Guidelines.

It is important to note that the structural reliability measures (reliability indices, $\beta$) reflected in the AASHTO LRFD Bridge Design Specifications, 4th Edition (2007) and the Manual for Bridge Evaluation (2008), LRFR option, are notional values that do not include risks due to human error, gross negligence, and failure modes that are either ignored by the evaluator or poorly understood.

9.2 DIRECT RELIABILITY ASSESSMENT

9.2.1 General

Subject to the approval of the Bridge Owner, it is permitted to determine the bridge rating from Eqs (9.2.1-1) which provides a direct solution for the rating factor given the mean values of resistance, dead load, live load, respective coefficients of variations, and target safety index.

$$RF = \frac{\bar{R} \exp[-\beta_f \left\{ V_R R_1 (1 - kV_R) \right\}^2 + \sigma_s^2 \left\{ V_R R_1 (1 - kV_R) \right\}^2]}{L} - D$$  \hspace{1cm} (9.2.1-1)
Alternatively, given statistical data on loads and structural resistance, it is permitted to calculate the reliability index, $\beta$, for a component directly from Eq (9.2.1-2).

\[
\beta = \frac{R(1-kV_R)[1-\ln(1-kV_R)]-\bar{S}}{\sqrt{\left(V_R\bar{R}(1-kV_R)\right)^2 + \sigma_S^2}}^{1/2} \tag{9.2.1-2}
\]

If $\beta$ calculated from eq (9.2.1-2) is greater than or equal to 2.5, the operating (or permit) rating is satisfied; if $\beta$ is greater than or equal to 3.5, reliability of the bridge is comparable to that of a newly-constructed bridge that is designed satisfying the AASHTO LRFD Bridge Design Specifications, 4th Edition (2007).

In both Eqs (9.2.1-1) and (9.2.1-2), the parameters are defined as follows:

\[
\begin{align*}
\bar{L} &= \text{Mean live-load effect} \\
\bar{D} &= \text{Mean dead-load effect} \\
\bar{S} &= \text{Mean total-load effect} \\
\bar{R} &= \text{Mean resistance} \\
V_S &= \text{Coefficient of variation of load effect} \\
V_R &= \text{Coefficient of variation of resistance} \\
\beta_T &= \text{Target safety index:} \\
&\quad 3.5 \text{ for inventory rating;} \\
&\quad 2.5 \text{ for operating rating} \\
\beta &= \text{Actual safety index of a component} \\
k &= \text{constant, which may be taken as equal to 2.}
\end{align*}
\]

9.2.2 Structural Resistance: $R$

The resistance of a structural element or component, $R$, shall be modeled as the product of nominal resistance, $R_n$, and three statistically independent random parameters:

\[
R = MFPR_n \tag{9.2.2-1}
\]

where:

$M$: Material factor representing uncertain material strength or modulus of elasticity;

$F$: Fabrication factor representing uncertain structural dimensions; and
$P$: Professional factor representing uncertainties due to the method of calculating component strength, and describing the accuracy with which experimental strength data can be modeled by structural analysis.

Resistance, $R$, shall be described by a lognormal distribution, with mean ($\bar{R}$) and COV ($V_R$) computed as follows:

$$\bar{R} = \lambda_R R_n$$

where

$$\lambda_R = \lambda_{FM} \lambda_P$$

$$V_R = (V_{FM}^2 + V_P^2)^{1/2}$$

where $\lambda$ is a bias factor (ratio of mean to nominal values) of the random variable and $V$ is its COV, both indicated by its subscript. The statistical parameters of resistance shall be determined by rational analysis. It is permitted to use the values presented in Table 9.2.2-1 (Nowak, 1999).

Table 9.2.2-1 Statistical Parameters of Resistance

<table>
<thead>
<tr>
<th>Type of Structure</th>
<th>$\lambda_{FM}$</th>
<th>$V_{FM}$</th>
<th>$\lambda_P$</th>
<th>$V_P$</th>
<th>$\lambda_R$</th>
<th>$V_R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-composite steel girders</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment (compact)</td>
<td>1.10</td>
<td>0.08</td>
<td>1.02</td>
<td>0.06</td>
<td>1.12</td>
<td>0.10</td>
</tr>
<tr>
<td>Moment (non-com.)</td>
<td>1.09</td>
<td>0.08</td>
<td>1.03</td>
<td>0.06</td>
<td>1.12</td>
<td>0.10</td>
</tr>
<tr>
<td>Shear</td>
<td>1.12</td>
<td>0.08</td>
<td>1.02</td>
<td>0.07</td>
<td>1.14</td>
<td>0.11</td>
</tr>
<tr>
<td>Composite steel girders</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment</td>
<td>1.07</td>
<td>0.08</td>
<td>1.05</td>
<td>0.06</td>
<td>1.12</td>
<td>0.10</td>
</tr>
<tr>
<td>Shear</td>
<td>1.12</td>
<td>0.08</td>
<td>1.02</td>
<td>0.07</td>
<td>1.14</td>
<td>0.11</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment</td>
<td>1.12</td>
<td>0.12</td>
<td>1.02</td>
<td>0.06</td>
<td>1.14</td>
<td>0.13</td>
</tr>
<tr>
<td>Shear w/ steel</td>
<td>1.13</td>
<td>0.12</td>
<td>1.08</td>
<td>0.10</td>
<td>1.20</td>
<td>0.16</td>
</tr>
<tr>
<td>Shear w/o steel</td>
<td>1.17</td>
<td>0.14</td>
<td>1.20</td>
<td>0.10</td>
<td>1.40</td>
<td>0.17</td>
</tr>
<tr>
<td>Prestressed concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment</td>
<td>1.04</td>
<td>0.05</td>
<td>1.01</td>
<td>0.06</td>
<td>1.05</td>
<td>0.08</td>
</tr>
<tr>
<td>Shear w/ steel</td>
<td>1.07</td>
<td>0.10</td>
<td>1.08</td>
<td>0.10</td>
<td>1.15</td>
<td>0.14</td>
</tr>
</tbody>
</table>

9.2.3 Load Effects

Bridge rating conducted in accordance with Section 9 shall consider dead load and live load (including static and dynamic effects) unless otherwise stipulated by the Bridge Owner. The statistical models of dead and live load shall be determined by rational analysis. It is permitted to treat dead and live loads as normal random variables, with
parameters defined in the following (Nowak, 1993, 1999). Statistical parameters for dead load suitable for use in direct safety assessment are tabulated in Table 9.2.3-1. The bias and COV in static live load (excluding dynamic impact) are presented in Table 9.2.3-2. Dynamic Impact factor \((I)\) has a mean of 0.1 and COV of 80%. The overall COV of the live load effects including dynamic impact is 18%.

### Table 9.2.3-1 Statistical Parameters of Dead Load

<table>
<thead>
<tr>
<th>Component</th>
<th>Bias Factor</th>
<th>C.O.V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factory-made members</td>
<td>1.03</td>
<td>0.08</td>
</tr>
<tr>
<td>Cast-in-place members</td>
<td>1.05</td>
<td>0.10</td>
</tr>
<tr>
<td>Asphalt</td>
<td>3.5 inch*</td>
<td>0.25</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>1.05</td>
<td>0.10</td>
</tr>
</tbody>
</table>

* mean thickness

### Table 9.2.3-2 Statistical Parameters of Static Live Load Effect

<table>
<thead>
<tr>
<th>Span(ft)</th>
<th>Moment</th>
<th>Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bias Factor</td>
<td>C.O.V</td>
</tr>
<tr>
<td>Less than 20</td>
<td>1.50</td>
<td>0.12</td>
</tr>
<tr>
<td>Greater than or equal to 20</td>
<td>1.35</td>
<td>0.12</td>
</tr>
</tbody>
</table>

### C 9.2 Direct Reliability Analysis

Equations (9.2.1-1) and (9.2.1-2) are based on a first-order reliability analysis, in which the resistance is modeled by a lognormal distribution and the combined load effect is modeled by a normal distribution (Nowak, 1999). Since the solution cannot be obtained in closed-form, both equations represent an approximation (\(k\) is close to 2 in most practical situations). The maximum error in these equations over the range of resistance and load statistics that is typical in bridge rating is less than 5%.

The resistance, \(R\), must include inherent variability of material strength, fabrication uncertainties and the ability of the strength equations to represent the structural mechanics of behavior correctly. The material factor \((M)\) represents material properties such as strength and modulus of elasticity. The fabrication factor \((F)\) includes geometry, dimensions, and section area and modulus. The professional factor \((P)\) pertains to the method of analysis and calculation of components strength and how tests and advanced analysis procedures compare with such calculated predictions. These three variables must be reflected in the mean and COV of the
resistance, R. The statistics of each random variable must be considered carefully to determine their bias and COV.

The statistics of the dead load must reflect both the uncertainty of the weight of the structure, the wearing surface and the uncertainty in the calculations of the dead load effects on the component being checked. The live load statistics must reflect factors such as the truck weight distribution, dimensions and the load distribution to the axles, dynamic load uncertainties, and load distribution to the supporting girders. A model for multiple presences of heavy vehicles on the bridge span is also needed to obtain superposition influence.

A limited database of statistical parameters appropriate for use in direct safety methods (Tables 9.2.2-1 and 9.2.3-1) is given in NCHRP Report 368, *Calibration of the LRFD Bridge Design Code* (Nowak, 1999) and in other references in 9.3. The statistics of P in columns 4 and 5 of Table 9.2.2-1 are based on member strength calculations at the level of sophistication found in typical textbooks on structural behavior and design. At a more advanced level of analysis, if the strength calculations are based on nonlinear finite element analysis, the mean bias typically would be on the order of 1.0 – 1.05, and the coefficient of variation would be approximately 50% of the values in Table 9.2.2-1. For applications not covered in these Guidelines, evaluators must develop statistical parameters based on test or analysis. The references in Article 9.3 provide a resource for this development.

9.3 REFERENCES


APPENDICES

EXAMPLES OF BRIDGE RATING USING RECOMMENDED GUIDELINES

Appendix A provides examples to illustrate the practical use of the Recommended Guideline.

This section is organized as fellows:

A. RATING OF A STRAIGHT CAST-IN-PLACE REINFORCED CONCRETE T-BEAM BRIDGE
B. RATING OF A STEEL GIRDER BRIDGE
C. RATING OF A PRE-STRESSED CONCRETE GIRDER BRIDGE
APPENDIX A

RATING OF A STRAIGHT CAST-IN-PLACE REINFORCED CONCRETE T-BEAM BRIDGE
(GDOT BRIDGE ID # 129-0045)

Rating by the Load and Resistance Factor Method (LRFR) Using Recommended Guidelines
A-1 Update Conditional Factor $\phi_c$

The bridge superstructure has a Condition Rating of 7; thus, the conditional factor $\phi_c = 0.95$ from Table 6-3 of the Recommended Guidelines.

A-2 Update Material Properties and Nominal Resistance

The results of the core sample tests on this bridge are presented in Table 3.3.1 of Part I and are reproduced in Table A.6.1 below. Following Sections 5.3 and 5.5 of the Recommended Guidelines, for 21 samples, the updated concrete strength is:

$$f_c = (1 - kV) \bar{x} = (1 - 1.520 \times 0.12) \times 4,820 = 3,900 \text{ psi}$$

Table A-2.1 Compression tests of 4 in x 8 in cores drilled from Bridge ID 129-0045

<table>
<thead>
<tr>
<th>Source</th>
<th>Number</th>
<th>Average (psi)</th>
<th>Standard deviation (psi)</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>14</td>
<td>4,880</td>
<td>603</td>
<td>0.12</td>
</tr>
<tr>
<td>Slab</td>
<td>7</td>
<td>4,698</td>
<td>573</td>
<td>0.12</td>
</tr>
<tr>
<td>Overall</td>
<td>21</td>
<td>4,820</td>
<td>586</td>
<td>0.12</td>
</tr>
</tbody>
</table>

A-2.1 Interior Beam Flexural Capacity:

Steel reinforcement yield strength: $f_y = 40$ ksi
Concrete compressive strength: $f'_c = 3900$ psi

$$\beta = 0.85 \text{ for } f'_c \leq 4 \text{ ksi}$$

$$c = \frac{A_sf_y}{0.85f'_c \beta b_c} = \frac{12.48 \times 40}{0.85 \times 3.9 \times 0.85 \times 86} = 2.06 \text{ in } < t_s$$

$$a = c\beta = 0.85 \times 2.06 = 1.75 \text{ in}$$

$$M_a = A_sf_y \left( d - \frac{a}{2} \right) \frac{1}{12} = 12.48 \times 40 \times (26.59 - \frac{1.75}{2}) \div 12 = 1,070 \text{ kip-ft}$$

A-2.2 Interior Beam Shear Capacity:

$$V_c = 0.0316\beta \sqrt{f'_c b_v d} = 0.0316 \times 2 \times \sqrt{3.9 \times 18 \times 26.59} = 60 \text{ kips}$$

$$V_s = \frac{A_v f_y d \cot(\theta)}{S} = \frac{0.4 \times 40 \times 26.59 \times \cot\left(\frac{\pi}{4}\right)}{12} = 35.5 \text{ kips}$$

$$V_n = V_c + V_s = 60 + 35.5 = 95.5 \text{ kips}$$
A-2.3 Exterior Beam Flexural Capacity:

Steel reinforcement yield strength: $f_y = 40$ ksi
Concrete compressive strength: $f'_c = 3900$ psi

$\beta = 0.85$ for $f'_c \leq 4$ksi

$c = \frac{A_s f_y}{0.85 f'_c \beta b_c} = \frac{11.9(40)}{0.85(3.9)(0.85)(76)} = 2.22$ in $< t_c = 6$ in

$a = \beta c = 0.85(2.20) = 1.89$ in

$M_n = A_s f_y \left( d - \frac{a}{2} \right) \frac{1}{12} = 11.9(40) \left( 25.7 - \frac{1.89}{2} \right) \frac{1}{12} = 982$ k-ft

A-2.4 Exterior Beam Shear Capacity:

$V_c = 0.0316\beta \sqrt{f'_c b_c d} = 0.0316 \times 2 \times \sqrt{3.9 \times 18 \times 25.7} = 58$ kips

$V_s = \frac{A_s f_y d \cot(\theta)}{S} = \frac{0.4 \times 40 \times 25.7 \times \cot(\frac{\pi}{4})}{12} = 34.3$ kips

$V_n = V_c + V_s = 58 + 34.3 = 92.3$ kips

A-3 Analysis

Dead loads are as computed in Task 1 Report Appendix C.

A-3.1 Maximum Bending Moment

A-3.1.1 Maximum live load moment for an interior beam

With $P = 32$ kips, $g_m = 0.407$ (obtained from finite element analysis), and IM =1.33:

$M_{LL+IM\ (HS\ 20)} = 234$ k-ft

$M_{LL+IM\ (HL\ 93)} = 286$ k-ft

A-3.1.2 Maximum live load moment for an exterior beam

With $P = 32$ kips, $g_{m\ (exterior)} = 0.395$ (obtained from finite element analysis), and IM =1.33:

$M_{LL+IM\ (HS\ 20)} = 227.0$ k-ft

$M_{LL+IM\ (HL\ 93)} = 278$ k-ft
A-3.2 Maximum Shear Force

A-3.2.1 Interior beam maximum live load shear force at $d_v = 2.1\, ft$

Interior beam shear distribution factor ($g_v = 0.512$) computed using FE Analysis validated by diagnostic load test:

\[ V_{HS20}(d_v) = 34.65\, kips \]
\[ V_{HL93}(d_v) = 40.35 \, kips \]

A-3.2.2 Exterior beam maximum live load shear force at $d_v = 2.0\, ft$

Exterior beam shear distribution factor ($g_v = 0.437$) computed using FE Analysis validated by diagnostic load test:

\[ V_{HS20}(d_v) = 29.70\, kips \]
\[ V_{HL93}(d_v) = 34.59 \, k - ft \]

A-4 Updated Rating Calculation

Resistance Factor (for shear and flexure) $\phi = 0.9$
Condition Factor (related to NBI Item 59) $\phi_c = 0.95$
System Factor (related to structural redundancy) $\phi_s = 1$

Figure A-4.1 Load and Resistance Factor Rating (LRFR) Calculation for HL93 load at Inventory and Operating Levels and HS20 load at the Legal Level (Using the Recommended Guidelines)

| Flexure (Interior girder) | Inventory Level | Operating Level | $RF$ | $\phi$ | $\phi_c$ | $\phi_s$ | $M_n$ | $M_D$ | $M_{HL93}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ $\gamma_{LL}$ | $\gamma_{DC}$ | $\gamma_{LL}$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ | $\gamma_{LL}$M$H_L93$ | $\gamma_{DC}$M$D$ |
### Appendix A  |  Report of Task 4  
**Rating of A Straight Cast-in-place Reinforce Concrete T-Beam Bridge**  
**Part II**

| Flexure (Exterior girder) | Legal Level  
|---------------------------|-------------------------------|-----------------------------|-------------------|
|                           | $\gamma_{DC} = 1.25$  
|                           | $\gamma_{LL} = 1.5$  
| $RF = \frac{\phi \phi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}}$  
| $= \frac{0.9(0.95)(1)(1070)-1.25(241.9)}{1.5(234)}$  
| $= 1.74$ tons |  
| Inventory Level  
| $\gamma_{DC} = 1.25$  
| $\gamma_{LL} = 1.75$  
| $RF = \frac{\phi \phi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}}$  
| $= \frac{0.9(0.95)(1)(982)-1.25(230.6)}{1.75(278)}$  
| $= 1.13$ tons |  
| Operating Level  
| $\gamma_{DC} = 1.25$  
| $\gamma_{LL} = 1.35$  
| $RF = \frac{\phi \phi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}}$  
| $= \frac{0.9(0.95)(1)(982)-1.25(230.6)}{1.35(278)}$  
| $= 1.47$ tons |  
| Shear (Interior Girder) | Legal Level  
|---------------------------|-------------------------------|-----------------------------|-------------------|
|                           | $\gamma_{DC} = 1.25$  
|                           | $\gamma_{LL} = 1.5$  
| $RF = \frac{\phi \phi_c \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$  
| $= \frac{0.9(0.95)(1)(95.5)-1.25(22.45)}{1.75(40.35)}$  
| $= 0.76$ tons |  
| Inventory Level  
| $\gamma_{DC} = 1.25$  
| $\gamma_{LL} = 1.75$  
| $RF = \frac{\phi \phi_c \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$  
| $= \frac{0.9(0.95)(1)(95.5)-1.25(22.45)}{1.35(40.35)}$  
| $= 0.98$ tons |  
| Operating Level  
| $\gamma_{DC} = 1.25$  
| $\gamma_{LL} = 1.35$  
| $RF = \frac{\phi \phi_c \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$  
| $= \frac{0.9(0.95)(1)(95.5)-1.25(22.45)}{1.35(40.35)}$  
| $= 1.03$ tons |  
| Shear (Exterior Girder) |  
|---------------------------|-------------------------------|-----------------------------|-------------------|
|                           | $\gamma_{DC} = 1.25$  
|                           | $\gamma_{LL} = 1.75$  
| $RF = \frac{\phi V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$  
| $= \frac{0.9(0.95)(1)(92.3)-1.25(21.53)}{1.75(34.59)}$  
| $= 0.86$ tons |  

---

A-6
A-7

A-5 Tabulated Rating Results

Table A-5.1 Rating Results Comparison for HL93 Load

<table>
<thead>
<tr>
<th>Rating Method</th>
<th>Structural Action</th>
<th>Inventory (HL93)</th>
<th>Operating (HL93)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO LRFR (2007)</td>
<td>Flexure</td>
<td>0.65</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>0.45</td>
<td>0.58</td>
</tr>
<tr>
<td>Recommended Guidelines</td>
<td>Flexure</td>
<td>1.13</td>
<td>1.47</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>0.76</td>
<td>0.98</td>
</tr>
</tbody>
</table>

Table A-5.2 Rating Results Comparison for HS20 Vehicle

<table>
<thead>
<tr>
<th>Rating Method</th>
<th>Structural Action</th>
<th>Inventory (HS20)</th>
<th>Operating (HS20)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASR AASHTO (2002)</td>
<td>Flexure</td>
<td>0.70 (25.2 tons)</td>
<td>1.25 (45.0 tons)</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>0.41 (14.8 tons)</td>
<td>0.75 (27.0 tons)</td>
</tr>
<tr>
<td>LFR AASHTO (2002)</td>
<td>Flexure</td>
<td>0.75 (27.0 tons)</td>
<td>1.25 (45.0 tons)</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>0.43 (15.5 tons)</td>
<td>0.72 (25.9 tons)</td>
</tr>
<tr>
<td>AASHTO LRFR (2007)</td>
<td>Flexure</td>
<td>N.A.</td>
<td>0.93 (33.5 tons)</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>N.A.</td>
<td>0.61 (22.0 tons)</td>
</tr>
<tr>
<td>Recommended Guidelines</td>
<td>Flexure</td>
<td>N.A.</td>
<td>1.62 (58.3 tons)</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>N.A.</td>
<td>1.03 (37.1 tons)</td>
</tr>
</tbody>
</table>
Appendix B

RATING OF A STEEL GIRDER BRIDGE
(GDOT BRIDGE ID # 085-0018)

Example B1:
Superstructure Rating by the Load and Resistance Factor Method (LRFR) Using Recommended Guidelines
**B1-1 Proposed Rating Method**

Update Conditional Factor $\varphi_c$

The bridge superstructure has a Condition Rating of 8; thus, the conditional factor $\varphi_c = 1.0$ from Table 6-3 of the *Recommended Guidelines*.

Dead loads and member capacity are taken from Task 1 Report, Appendix F.

**B1-2 Analysis**

**B1-2.1 Maximum live load moment for an interior beam**

With $P = 32$ kips, $g_m = 0.513$ (obtained from finite element analysis), and $IM = 1.33$:

$$M_{LL+IM(HS20)} = 398k - ft$$

$$M_{LL+IM(HL93)} = 489k - ft$$

**B1-2.2 Maximum live load moment for an exterior beam**

With $P = 32$ kips, $g_{m(\text{exterior})} = 0.434$ (obtained from finite element analysis), and $IM = 1.33$:

$$M_{LL+IM(HS20)} = 337k - ft$$

$$M_{LL+IM(HL93)} = 415k - ft$$
B1-3 Updated Rating Calculation

Resistance Factor (for shear and flexure) \( \phi = 0.9 \)
Condition Factor (related to NBI Item 59) \( \varphi_c = 1.0 \)
System Factor (related to structural redundancy) \( \phi_s = 1 \)

Figure B1-3.1 Load and Resistance Factor Rating (LRFR) Calculation for HL93 load at Inventory and Operating Levels and HS20 load at the Legal Level (Using the Recommended Guidelines)

<table>
<thead>
<tr>
<th>Flexure (Interior girder)</th>
<th>Inventory Level</th>
<th>Operating Level</th>
<th>Legal Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_{DC} = 1.25 )</td>
<td>( \gamma_{LL} = 1.75 )</td>
<td>( \gamma_{DC} = 1.25 )</td>
<td>( \gamma_{LL} = 1.75 )</td>
</tr>
<tr>
<td>Flexure (Exterior girder)</td>
<td>( \gamma_{DC} = 1.25 )</td>
<td>( \gamma_{LL} = 1.75 )</td>
<td>( \gamma_{DC} = 1.25 )</td>
</tr>
</tbody>
</table>

\[
RF = \frac{\phi \varphi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1,401) - 1.25(316.7)}{1.75(489)} = 1.01
\]

\[
RF = \frac{\phi \varphi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1,401) - 1.25(316.7)}{1.35(489)} = 1.31
\]

\[
RF = \frac{\phi \varphi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(0.95)(1)(1,401) - 1.25(316.7)}{1.43(398)} = 1.52
\]

\[
RF = \frac{\phi \varphi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1,245) - 1.25(259.6)}{1.75(415)} = 1.10
\]

\[
RF = \frac{\phi \varphi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1,245) - 1.25(259.6)}{1.35(415)} = 1.42
\]

\[
RF = \frac{\phi \varphi_c \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HL93}} = \frac{0.9(1)(1,245) - 1.25(259.6)}{1.43(337)} = 1.65
\]
### B1-4 Tabulated Rating Results

**Table B1-4.1 Rating Results Comparison for HL93 Load**

<table>
<thead>
<tr>
<th>Rating Method</th>
<th>Structural Action</th>
<th>Inventory (HL93)</th>
<th>Operating (HL93)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO LRFR (2007)</td>
<td>Flexure</td>
<td>0.72</td>
<td>0.93</td>
</tr>
<tr>
<td><strong>Recommended Guidelines</strong></td>
<td>Flexure</td>
<td>1.01</td>
<td>1.31</td>
</tr>
</tbody>
</table>

**Table B1-4.2 Rating Results Comparison for HS20 Vehicle**

<table>
<thead>
<tr>
<th>Rating Method</th>
<th>Structural Action</th>
<th>Inventory (HS20)</th>
<th>Operating (HS20)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASR AASHTO (2002)</td>
<td>Flexure</td>
<td>0.82 (29.5 tons)</td>
<td>1.33 (47.9 tons)</td>
</tr>
<tr>
<td>LFR AASHTO (2002)</td>
<td>Flexure</td>
<td>0.71 (25.6 tons)</td>
<td>1.18 (42.5 tons)</td>
</tr>
<tr>
<td>AASHTO LRFR (2007)</td>
<td>Flexure</td>
<td>N.A.</td>
<td>1.08 (38.9 tons)</td>
</tr>
<tr>
<td><strong>Recommended Guidelines</strong></td>
<td>Flexure</td>
<td>N.A.</td>
<td>1.52 (54.7 tons)</td>
</tr>
</tbody>
</table>
Example B2:

Pier Cap Rating by the Load and Resistance Factor Method (LRFR) Using Recommended Guidelines
**B2-1 Proposed Rating Method**

Update Conditional Factor $\phi_c$

The bridge superstructure has a Condition Rating of 7; thus, the conditional factor $\phi_c = 0.95$ from Table 6-3 of the *Recommended Guidelines*.

Dead load is taken from the Task 1 Report, Appendix F.

**B2-2 Strut and Tie Model (Article 5.6.3 AASHTO 2007)**

As per Article 5.8.1.1 of AASHTO 2007, “Components in which the distance from the point of 0.0 shear to the face of the support is less than 2d, or components in which a load causing more than ½ of the shear at a support is closer than 2d from the face of the support, may be considered to be deep components for which the provisions of Article 5.6.3 and the detailing requirements of Article 5.13.2.3 apply.”

Article 5.8.1.2 of AASHTO 2007 states that regions such as this pier cap are to be designed by the strut-and-tie method, as defined in Article 5.6.3. However, this pier cap does not meet the minimum requirement of Article 5.13.2; therefore Article 5.6.3 does not apply.

**B2-2.1 Strength of Tension Ties (Article 5.6.3.4.1 AASHTO 2007)**

$$ P_n = f_y A_{st} + A_{ps} \left[ f_{pe} + f_y \right] $$

Where:

- $A_{st}$ = Total area of longitudinal mild steel reinforcement in the tie (6 #10 bars = 7.62 in$^2$)
- $A_{ps}$ = Area of prestressing steel (0.0 in$^2$); (pier cap is not pre-stressed).
- $f_y$ = Yield strength of mild steel longitudinal reinforcement (40 ksi)
- $f_{pe}$ = Stress in prestressing steel due to prestress after losses (ksi)

Yield strength of tension tie:

$$ P_n = f_y A_{st} = 40(7.62) = 305 \text{kips} $$

The capacity of the tension tie will be used to check bearing in the nodal regions and strength of the compression ties.

**B2-2.2 Proportioning of strut and node at bearing of superstructure**

Effective sectional area of strut (Article 5.6.3.3.2 AASHTO 2007)

$$ A_{st} = \text{Effective sectional area of strut} $$

Two types of nodes are present in this pier cap:

- Struts anchored by bearing and reinforcement (at the location of bearing of the superstructure)
• Struts anchored by bearing and strut (at the location of pier cap column joint)

By inspection, bearing at the superstructure is likely to govern the width of the strut due to the fact that the bearing area is approximately one quarter of that at the location of the pier cap column joint. Therefore the effective sectional area of the strut will be computed at the location of superstructure bearing, and then be checked to ensure that that location is correct.

\[ f_c' = 2.5 ksi \]

\[ d_b = \text{bar diameter (1.27” for #10 bars)} \]

\[ w_s = 36” \text{ width of strut (out of plane width of pier cap)} \]

\[ w_b = \text{width of bearing plate (out of plane)} \]

\[ l_b = 13” \text{ length of bearing plate (in plane)} \]

\[ h_a = \text{height of the reinforcement bearing area, equal to the distance between the two extreme layers of reinforcement in the tension tie, plus } 6d_b \text{ above and below the extreme tension layer} \]

\[ \theta_s = \text{least angle between the tension tie reinforcement and compression strut} \]

B2-2.2.1 Effective sectional area of strut at bearing of superstructure

\[ l_b \sin(\theta_s) + h_a \cos(\theta_s) \] (Article 4.6.3.3.2 AASHTO 2007)

For this pier cap \( w_s \) is equal to the width of the bearing plates under the steel stringers of the superstructure. Each plate is 13” along the plane of the pier cap and 8” out of the plane of the pier cap. Two stringers frame into the pier cap at each location so the total out-of-plane width of the bearing plates is 16” which is less than the 36” out-of-plane width of the pier cap. Therefore:

\[ w_s = 16” \]

\[ h_a \] is equal to the distance from the surface of the pier cap to the centroid of the outer layer of tension tie reinforcement (3.14”) (in this case there is only one layer of tension tie reinforcement) plus \( 6d_b = 7.62” \) thus:

\[ h_a = 10.76” \]

\[ \theta_s \] is determined by the geometry of the struts so a 1st attempt is taken:

\[ \theta_s = 30° \]

The in plane width of the strut is:

\[ (13 \sin(30°) + 10.76 \cos(30°)) = 15.82 in \]

B2-2.2.2 Proportioning of node region at bearing of superstructure (Article 5.6.3.5 AASHTO 2007)
0.75φf'_c when nodal region anchors a tension tie in one direction

φ = 0.7 for bearing on concrete (Article 5.5.4.2 AASHTO 2007)

Limit of vertical force at bearing:

0.75φf'_cbw = 0.75(0.7)(2.5)(13)(16) = 273 kips

273 kips is greater than 305 tan(30) = 176 kips therefore strength of the tension tie still governs

Limit of horizontal force at bearing:

0.75φf'_chn = 0.75(0.7)(2.5)(10.76)(36) = 508 kips

508 kips is greater than 305 kips therefore strength of the tension tie still governs

B2-2.3 Limit of Compressive Stress in Strut (Article 5.6.3.3.3 AASHTO 2007)

\[ f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \leq 0.85f'_c \]

\[ \varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2\alpha_s \]

where:

\( \alpha_s = \) The smallest angle between compression strut and adjoining tension ties

\( \varepsilon_s = \) The tensile strain in the concrete in the direction of the tension tie

\[ \varepsilon_s = \frac{f_{tie}}{E_s} = \frac{40}{29,000} = 0.00138 \text{in/in} \]

\[ \varepsilon_1 = 0.00138 + (0.00138 + 0.002)\cot^2(30) = 0.01152 \text{in/in} \]

\[ f_{cu} = \frac{2.5}{0.8 + 170(0.01152)} = 0.906ksi \leq 0.85(2.5) = 2.125ksi \]

\[ P_n = f_{cu}A_s = (0.906)(15.82)(36) = 516 kips \]

516 kips is greater than \( \frac{305}{\cos(30)} = 352 kips \) therefore the strength of the tension tie still governs

B2-2.4 Proportioning of strut and node at pier cap/column joint

\[ w_s = 36" \]
$w_b = 36''$

$l_b = \frac{36}{2} = 18''$

$h_i = \text{height of horizontal compression strut}$

$\theta_i = 30''$

**B2-2.4.1 Effective sectional area of strut at bearing of superstructure**

\[
l_b \sin(\theta_i) + h_i \cos(\theta_i) \quad \text{(Article 4.6.3.3.2 AASHTO 2007)}
\]

0.85$\phi f'_c$ when nodal region is bounded by compression struts and bearing regions exclusively

$\phi = 0.7$ for bearing on concrete (Article 5.5.4.2 AASHTO 2007)

\[
h_s = \frac{305}{(0.85)(0.7)(2.5)(36)} = 5.7\text{in}
\]

In-plane width of strut:

\[(18 \sin(30) + 5.7 \cos(30)) = 14\text{in}\]

**B2-2.4.2 Proportioning of node region at bearing of superstructure (Article 5.6.3.5 AASHTO 2007)**

Limit of vertical force at bearing:

\[
0.85\phi f'_c l_b w_b = 0.85 (0.7)(2.5)(18)(36) = 850\text{kips}
\]

960 kips is greater than $305 \tan(35) = 214\text{kips}$ therefore strength of the tension tie still governs

Limit of Horizontal force at bearing:

\[
0.85\phi f'_c h_i w_i = 0.85 (0.7)(2.5)(5.7)(36) = 305\text{kips}
\]

308 kips is equal to 305 kips therefore strength of the tension tie still governs

**B2-2.5 Factored Resistance of (Article 5.6.3.2)**

\[
P_r = \phi P_n
\]

Where:

$P_n = \text{Nominal resistance of strut or tie}$

$\phi = \text{Resistance factor for tension or compression (Article 5.5.4.2 AASHTO 2007)}$
Since the strength of the tension tie governs the strut and tie model:

\[ P_n = 305 \tan(30) = 176 \text{kips} \]
\[ P_r = (1)176 = 176 \text{kips} \]

**B2-2.6 Detailing requirements for Deep Beams (Article 5.13.2.3 AASHTO 2007)**

\[ N_R = \phi f_y A_s \geq 0.12 b_v s \]

where:
- \( A_s \) = Area of steel stirrups in spacing \( s \)
- \( \phi \) = Resistance factor specified in (Article 5.5.4.2 AASHTO 2007)
- \( b_v \) = Width of web
- \( s \) = Spacing of reinforcement

Additionally the spacing of transverse reinforcement shall not exceed \( d/4 \) or 12 in.

The spacing of longitudinal reinforcement shall not exceed \( d/3 \) or 12 in.

Since the pier cap in question was designed prior to strut and tie provisions it meets none of these detailing requirements.

**B2-2.7 Live and Dead Loads**

All loads, distribution and impact factors are the same as those computed in Appendix B1 and thus are not presented here.

**B2-2.8 Rating Calculation (LRFR)**

Inventory \( \gamma_{DC} = 1.25 \quad \gamma_{LL} = 1.75 \)

Operating \( \gamma_{DC} = 1.3 \quad \gamma_{LL} = 1.35 \)

Live load factor for legal load level \( \gamma_{LL} = 1.43 \) (LRFR Table 6-5 based on ADTT of 210)

Resistance Factor (for tension in steel in anchorage zones) \( \phi = 1 \) (Article 5.5.4.2.1 AASHTO 2007)

Condition Factor (related to NBI Item 59) \( \phi_c = 1 \)

System Factor (related to structural redundancy) \( \phi_s = 1 \)
Table B2-2.1 Load and Resistance Factor Rating (LRFR) Calculation for HL93 load at Inventory and Operating Levels and HS20 load at the Legal Level (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO LRFD Specifications (2007))

<table>
<thead>
<tr>
<th>Shear</th>
<th>Inventory Level</th>
<th>Operating Level</th>
<th>Legal Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$RF = \frac{\phi \phi \phi \phi P - \gamma_{DL}{}^{V}V_{HL93}}{}$</td>
<td>$RF = \frac{\phi \phi \phi \phi P - \gamma_{DL}{}^{V}V_{HL93}}{}$</td>
<td>$RF = \frac{\phi \phi \phi \phi P - \gamma_{DL}{}^{V}V_{HS20}}{}$</td>
</tr>
<tr>
<td></td>
<td>$= \frac{(1)(1)(176) - 1.25(62)}{1.75(104)} = 0.54$</td>
<td>$= \frac{(1)(1)(176) - 1.25(62)}{1.35(104)} = 0.70$</td>
<td>$= \frac{(1)(1)(176) - 1.25(62)}{1.43(73.8)} = 0.93$</td>
</tr>
<tr>
<td></td>
<td>19.4 tons</td>
<td>25.2 tons</td>
<td>33.5 tons</td>
</tr>
</tbody>
</table>

B2-3 Tabulated Rating Results

Table B2-3.1 Rating Results Comparison for HL93 Load

<table>
<thead>
<tr>
<th>Rating Method</th>
<th>Structural Action</th>
<th>Inventory (HL93)</th>
<th>Operating (HL93)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO LRFR (2007)</td>
<td>Shear</td>
<td>0.43</td>
<td>0.56</td>
</tr>
<tr>
<td>S&amp;T</td>
<td>Shear</td>
<td>0.54</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Table B2-3.2 Rating Results Comparison for HS20 Vehicle

<table>
<thead>
<tr>
<th>Rating Method</th>
<th>Structural Action</th>
<th>Inventory (HS20)</th>
<th>Operating (HS20)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASR AASHTO (2002)</td>
<td>Shear</td>
<td>0.26 (9.36 tons)</td>
<td>0.75 (27.0 tons)</td>
</tr>
<tr>
<td>LFR AASHTO (2002)</td>
<td>Shear</td>
<td>0.43 (15.5 tons)</td>
<td>0.72 (25.9 tons)</td>
</tr>
<tr>
<td>AASHTO LRFR (2007)</td>
<td>Shear</td>
<td>N.A.</td>
<td>0.75 (27.0 tons)</td>
</tr>
<tr>
<td>S&amp;T</td>
<td>Shear</td>
<td>N.A</td>
<td>0.93 (33.5 tons)</td>
</tr>
</tbody>
</table>
APPENDIX C

RATING OF A PRE-STRESSED CONCRETE BRIDGE
(GDOT BRIDGE ID # 223-0034)

Rating by the Load and Resistance Factor Method (LRFR)
Using Recommended Guidelines
C-1 Proposed Rating Method

C-1.1 Update Conditional Factor $\varphi_c$

The bridge superstructure has a Condition Rating of 7; thus, the conditional factor $\varphi_c = 0.95$ from Table 6-3 of the Recommended Guidelines.

C-1.3 Interior Beam Flexural Capacity: $M_n = 4,542 \text{ kip-ft (Task 1 Report, Section E1-8)}$

C-1.4 Interior Beam Shear Capacity: $V_n = 287.9 \text{ kips (Task 1 Report, Section E1-8)}$

C-1.5 Exterior Beam Flexural Capacity: $M_n = 4,510 \text{ k} - \text{ft (Task 1 Report, Section E1-9)}$

C-1.6 Exterior Beam Shear Capacity: $V_n = 267.9 \text{ kips (Task 1 Report, Section E1-9)}$

C-2 Analysis

C-2.1 Maximum Bending Moment

C-2.1.1 Maximum live load moment for an interior beam (31.07 ft)

With $P = 32 \text{ kips}$, $g_m = 0.421$ (measured during diagnostic load test), and $IM = 1.33$:

\[
M_{LL+IM(HS20)} = 520 \text{ k} - \text{ft} \\
M_{LL+IM(HL93)} = 689 \text{ k} - \text{ft}
\]

C-2.1.2 Maximum live load moment for an exterior beam (31.07 ft)

With $P = 32 \text{ kips}$, $g_{m(\text{exterior})} = 0.408$ (obtained from finite element analysis), and $IM = 1.33$:

\[
M_{LL+IM(HS20)} = 446 \text{ k} - \text{ft} \\
M_{LL+IM(HL93)} = 574 \text{ k} - \text{ft}
\]

C-2.2 Maximum Shear Force

C-2.2.1 Interior beam maximum live load shear force at (3.341 ft)

Interior beam shear distribution factor ($g_v = 0.493$) computed using FE analysis and validated by diagnostic load test:

\[
V_{HS20} = 38.3 \text{ kips} \\
V_{HL93} = 48.1 \text{ kips}
\]
C-2.2.2 Exterior beam maximum live load shear force at (3.341 ft)

Exterior beam shear distribution factor (\(g_v = 0.432\)) computed using FE Analysis validated by diagnostic load test:

\[ V_{HS20} = 33.5 \text{kips} \]
\[ V_{HL93} = 41.8 \text{ k} - \text{ft} \]

C-2.3 Live Load Concrete Stresses

C-2.3.1 Interior beam stress at top of beam from live load (including impact and distribution factors)

\[
S_{lt} = \frac{M_{ll}}{S_{le}} = \frac{8,268}{49,990} = 0.165 \text{ksi}
\]

C-2.3.2 Interior beam stress at bottom of beam from live load (including impact and distribution factors)

\[
S_{lb} = \frac{M_{lb}}{S_{lb}} = \frac{8,268}{10,907} = 0.758 \text{ksi}
\]

C-2.3.3 Exterior beam stress at top of beam from live load (including impact and distribution factors)

\[
S_{lt} = \frac{M_{lt}}{S_{le}} = \frac{6,888}{44,374} = 0.155 \text{ksi}
\]

C-2.3.4 Exterior beam stress at bottom of beam from live load (including impact and distribution factors)

\[
S_{lb} = \frac{M_{lb}}{S_{lb}} = \frac{6,888}{10,784} = 0.639 \text{ksi}
\]

C-3 Rating Calculation

<table>
<thead>
<tr>
<th>Inventory Level</th>
<th>Operating Level</th>
<th>Legal Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\gamma_{DC} = 1.25)</td>
<td>(\gamma_{DC} = 1.25)</td>
<td>(\gamma_{DC} = 1.25)</td>
</tr>
<tr>
<td>(\gamma_{LL} = 1.75)</td>
<td>(\gamma_{LL} = 1.35)</td>
<td>(\gamma_{LL} = 1.55 (ADTT = 655))</td>
</tr>
</tbody>
</table>

Table C-3.1 Load and Resistance Factor Rating (LFR) Calculation for HS20 (Using the dynamic load allowance and load distribution factors stipulated in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007))
<table>
<thead>
<tr>
<th></th>
<th></th>
<th>Legal Level</th>
<th>$RF = \frac{\phi \phi_s M_n - \gamma_{DC} M_D}{\gamma_{LL} M_{HS20}}$</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi = 1$</td>
<td>$\phi = 0.95$</td>
<td>$\phi_s = 1$</td>
<td>$135.0$ tons</td>
</tr>
<tr>
<td>Flexure (Exterior girder)</td>
<td>$\phi = 1$</td>
<td>$\phi = 0.95$</td>
<td>$\phi_s = 1$</td>
<td>$109.8$ tons</td>
</tr>
<tr>
<td>Inventory Level</td>
<td>$RF = \frac{\phi \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$</td>
<td>$\phi = 0.9$</td>
<td>$\phi_s = 1$</td>
<td>$98.3$ tons</td>
</tr>
<tr>
<td>Operating Level</td>
<td>$RF = \frac{\phi \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$</td>
<td>$\phi = 0.9$</td>
<td>$\phi_s = 1$</td>
<td>$107.6$ tons</td>
</tr>
<tr>
<td>Legal Level</td>
<td>$RF = \frac{\phi \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$</td>
<td>$\phi = 0.9$</td>
<td>$\phi_s = 1$</td>
<td>$114.1$ tons</td>
</tr>
<tr>
<td>Shear (Interior Girder at 3.341 ft)</td>
<td>$\phi = 0.9$</td>
<td>$\phi_s = 1$</td>
<td>$140.0$ tons</td>
<td></td>
</tr>
<tr>
<td>Inventory Level</td>
<td>$RF = \frac{\phi \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$</td>
<td>$\phi = 0.9$</td>
<td>$\phi_s = 1$</td>
<td>$178.6$ tons</td>
</tr>
<tr>
<td>Operating Level</td>
<td>$RF = \frac{\phi \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$</td>
<td>$\phi = 0.9$</td>
<td>$\phi_s = 1$</td>
<td>$81.4$ tons</td>
</tr>
<tr>
<td>Legal Level</td>
<td>$RF = \frac{\phi \phi_s V_n - \gamma_{DC} V_{DL}}{\gamma_{LL} V_{HL93}}$</td>
<td>$\phi = 0.9$</td>
<td>$\phi_s = 1$</td>
<td>$63.4$ tons</td>
</tr>
<tr>
<td>Top of Beam Inventory Level</td>
<td>$RF_i = \frac{S_i - S_{all}}{S_{all}}$</td>
<td>$2.4 - 1.758$</td>
<td>$0.165$</td>
<td>$3.89$</td>
</tr>
<tr>
<td>Bottom of Beam Inventory Level</td>
<td>$RF_i = \frac{S_i - S_{all}}{S_{all}}$</td>
<td>$2.4 - 1.631$</td>
<td>$0.155$</td>
<td>$4.96$</td>
</tr>
<tr>
<td>Exterior Beam Serviceability (at midspan)</td>
<td></td>
<td></td>
<td></td>
<td>178.6 tons</td>
</tr>
<tr>
<td>Top of Beam Inventory Level</td>
<td>$RF_i = \frac{S_i - S_{all}}{S_{all}}$</td>
<td>$2.4 - 1.758$</td>
<td>$0.165$</td>
<td>$63.4$ tons</td>
</tr>
<tr>
<td>Bottom of Beam Inventory Level</td>
<td>$RF_i = \frac{S_i - S_{all}}{S_{all}}$</td>
<td>$2.4 - 1.631$</td>
<td>$0.155$</td>
<td>$2.26$</td>
</tr>
</tbody>
</table>
## C-4 Tabulated Rating Results

### Table C-4.1 Rating Results Comparison for HL93 Load

<table>
<thead>
<tr>
<th>Rating Method</th>
<th>Structural Action</th>
<th>Inventory (HL93)</th>
<th>Operating (HL93)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>AASHTO LRFR (2007)</strong></td>
<td>Flexure</td>
<td>1.18</td>
<td>1.53</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>1.05</td>
<td><strong>1.36</strong></td>
</tr>
<tr>
<td></td>
<td>Stress</td>
<td><strong>0.82</strong></td>
<td>N.A.</td>
</tr>
<tr>
<td><strong>Recommended Guidelines</strong></td>
<td>Flexure</td>
<td>2.51</td>
<td>3.25</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>2.11</td>
<td><strong>2.73</strong></td>
</tr>
<tr>
<td></td>
<td>Stress</td>
<td><strong>1.76</strong></td>
<td>N.A.</td>
</tr>
</tbody>
</table>

### Table C-4.2 Rating Results Comparison for HS20 Vehicle

<table>
<thead>
<tr>
<th>Rating Method</th>
<th>Structural Action</th>
<th>Inventory (HS20)</th>
<th>Operating (HS20)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LFR AASHTO (2002)</strong></td>
<td>Flexure</td>
<td>1.54 (55.4 tons)</td>
<td>2.57 (92.5 tons)</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>1.43 (51.5 tons)</td>
<td><strong>2.39 (86.0 tons)</strong></td>
</tr>
<tr>
<td></td>
<td>Stress</td>
<td><strong>1.28 (46.1 tons)</strong></td>
<td>N.A.</td>
</tr>
<tr>
<td><strong>AASHTO LRFR (2007)</strong></td>
<td>Flexure</td>
<td>N.A.</td>
<td>1.72 (61.9 tons)</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>N.A.</td>
<td><strong>1.47 (52.9 tons)</strong></td>
</tr>
<tr>
<td></td>
<td>Stress</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td><strong>Recommended Guidelines</strong></td>
<td>Flexure</td>
<td>N.A.</td>
<td>3.75 (135.0 tons)</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>N.A.</td>
<td><strong>2.99 (107.6 tons)</strong></td>
</tr>
<tr>
<td></td>
<td>Stress</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
</tbody>
</table>