National Cooperative Highway Research Program

NCHRP Report 352

Inelastic Rating Procedures for Steel Beam and Girder Bridges

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National Cooperative Highway Research Program

Report 352

Inelastic Rating Procedures for Steel Beam and Girder Bridges

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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FOREWORD

By Staff
Transportation Research Board

This report contains the results of a study that developed a bridge rating method, which takes advantage of the inelastic reserve capacity of steel beam and girder bridges. The findings of this study will be of immediate interest to engineers responsible for bridge ratings, bridge maintenance, bridge design, bridge specifications, and bridge management at the federal, state, and local levels.

Rating methods using elastic analysis assumptions and contemporary design procedures have classified numerous older steel beam and girder bridges in the U.S. as structurally deficient. It is likely that many of these bridges would be rated as structurally acceptable if a steel structure's inelastic reserve capacity and three-dimensional action were considered in the strength analysis. Conservative design procedures make it probable that many steel beam and girder bridges would be rated as adequate, even without exceeding the elastic limit, if more sophisticated analytical techniques were used. In other cases, more comprehensive ratings would permit relatively simple and economical measures such as adding lateral bracing or providing composite action to be evaluated for the considerable additional strength they provide.

Under NCHRP Project 12-28(12), "Inelastic Rating Procedures for Steel Beam and Girder Bridges," the University of Minnesota surveyed different bridge rating methods and critically evaluated and studied them through rating studies of typical bridges. The various analytical tools and the limit states used to rate steel beam and girder bridges were ranked, and the analytical techniques appropriate for developing inelastic bridge rating procedures were identified. The research showed that current rating procedures for the subject bridges are very conservative because they ignore reserve capacity due to (1) the load distribution both longitudinally and laterally and (2) the ductility and rotational capacity of the main members.

Using the concept of shakedown (i.e., composite and noncomposite multigirder structures will adapt and respond in the elastic range of working loads after some initial deformations in the plastic range occur due to overloading), two bridge rating procedures—based on inelastic reserve capacity—were developed. The first is a *grid analysis,* which is applicable to complete beam-and-slab bridges with compact beams. It provides the rating factor at the shakedown limit. The second, called the *residual damage analysis* (RDA), is applicable to individual bridge beams of compact or noncompact composite or noncomposite sections. This analysis provides the residual damage for a given rating factor. A PC-based prototype computer program titled Inelastic Bridge Rating (IBR) was developed to incorporate both of these inelastic rating methods. The University of Minnesota researchers evaluated and rated five actual bridges using the RDA procedure and tested a one-third scale model bridge to destruction to experimentally verify the theoretically developed IBR method. The prototype software is available in the $3\frac{1}{2}$-in. high-density IBM-PC floppy diskette format by sending a $5 check payable to TRB to: CRP-Software, 12-28(12), c/o Transportation Research Board, 2101 Constitution Avenue, NW, Washington, D.C. 20418.

The report's Appendix A contains a proposed addendum to the AASHTO *Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges,* which would expand that bridge rating document to include the Inelastic Bridge Rating (IBR) method. This addendum is an instruction manual for IBR, containing formulae to apply IBR by manual calculation, as well as the solution to an example problem.

The report's conclusions indicate that: "Although the concept of shakedown is not familiar to most rating engineers, it is easy to grasp from the conceptual point of view. In addition, the calculation of the shakedown loads follows directly from the elastic calculations for the analysis of the bridge, and represents very little, if any, additional effort on the part of the rating engineer."

CONTENTS

ACKNOWLEDGMENTS

The research reported herein was performed at the Department of Civil and Mineral Engineering, University of Minnesota, Twin Cities.

Theodore V. Galambos, Record Professor of Engineering, was the Principal Investigator. The other authors of this report are Roberto T. Leon, Associate Professor of Civil Engineering; Catherine W. French, Associate Professor of Civil Engineering; and Michael Barker and Burl Dishongh, doctoral students.

The work was done under the general supervision of Professor Galambos, with the assistance of Graduate Research Assistants Paul Bergson, Siu-Yue Tam, and Phil Sauser. The experimental work carried out in this project was conducted at the Structural Engineering Laboratory at Iowa State University with the cooperation of F. Wayne Klaiber, Professor of Civil Engineering.

INELASTIC RATING PROCEDURES FOR STEEL BEAM AND GIRDER BRIDGES

SUMMARY

This project developed a rating methodology for existing bridges that incorporates some of the inelastic capacity present in most multigirder bridges. Composite and noncomposite multigirder bridges possess substantial load redistribution capacity because of the stiffness provided by the concrete deck, diaphragms, and cross-bracing. Current rating procedures differ little from design provisions for new bridges, and do not recognize the structure's ability to redistribute loads once local yielding has begun.

In selecting the limit states for this new rating procedure, care was taken to consider both analytical and practical issues. Because of the cyclic nature of loads applied to bridges, an ultimate strength limit state is unconservative and should not be used to rate them. Rather *shakedown,* or that load causing a set of residual moments throughout the structure such that the bridge responds to subsequent loads of the same magnitude or smaller in an elastic fashion, is the recommended limit state to be used when cyclic loads are present. Although the concept of shakedown is not familiar to most rating engineers, it is easy to grasp from the conceptual point of view. In addition, the calculation of the shakedown loads follows directly from the elastic calculations made for the analysis of the bridge, and represents very little, if any, additional effort on the part of the rating engineer.

Research conducted under NCHRP Project 12-28(12) clearly indicates that the use of inelastic action in rating straight, composite, and noncomposite multigirder bridges is justified. The permanent deformations expected under the factored rating vehicles are very small, less than what is visually evident, and the members possess, in general, more than adequate ductility to allow for required rotations. Rating factors using inelastic action not only yield a more realistic assessment of the structure's capacity but also provide the means to determine economical and reliable strengthening and repair methods.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 RESEARCH PROBLEM STATEMENT

Recent surveys of all highway bridges in the United States indicated that approximately 40 percent were either structurally deficient or functionally obsolete and in need of rehabilitation or replacement (*1*). According to the data in the National Bridge Inventory, over 36 percent of existing bridges in the U.S. can be classified as steel beam and girder bridges, and constitute by far the most common bridge type in service today (2). For inventory purposes, they are classified as steel stringer (27.2%), continuous steel stringer (7.6%), or steel girder-floor beam (1.9%). Not only are these the most common types of bridge, but they are also the ones that will need the most attention in the coming years because many of them are approaching the end of their estimated useful life.

Many older steel beam and girder bridges have been judged to be inadequate for current traffic based on rating methods using elastic analysis assumptions and contemporary design procedures. The two primary reasons why these bridges are found to be inadequate are that (1) truck loads and number of trucks have increased since the bridges were designed, and (2) most of the bridges have undergone at least slight deterioration over the years.

It is likely that many older bridges could be rated as acceptable if the inelastic reserve capacity of the steel structure and three-dimensional action were considered in the strength analysis. Given the conservatism of contemporary design procedures, it is likely that many bridges will be rated as adequate even without exceeding the elastic limit if more sophisticated analytical techniques are used. In other cases, relatively simple and economic measures could be taken to increase the strength considerably (e.g., added lateral bracing, providing continuity or composite action) without resorting to extensive rehabilitation or replacement.

1.2 RESEARCH OBJECTIVES AND APPROACH

The objective of NCHRP Project 12-28(12) was to develop a rating method that incorporates both (1) the inelastic reserve capacity of the steel members and (2) the redistribution capacity due to the slab and composite action into a realistic assessment of the structural capacity. The techniques developed are applicable to simple and continuous, composite and noncomposite, straight bridges made of rolled steel beams or plate girders. The project focused on conditions of overload and ultimate load only, because the service load range is basically a fatigue requirement. Thus, fatigue was explicitly excluded from consideration in this project. This report addresses the completion of all the tasks of NCHRP Project 12-28(12) "Inelastic Rating Procedures for Steel Beam and Girder Bridges."

The principal aim of this research was the development of recommendations for rating existing multigirder steel bridges in which the inherent ductility and inelastic reserve of this particular bridge type can be utilized. An examination of the prevailing rating methods around the world revealed that this inelastic reserve is not being explicitly used. A thorough review of existing research literature showed that it is feasible to count on the availability of the inelastic reserve, even for noncompact sections and especially for composite bridges.

Based on the research findings, a rating philosophy was developed, which has as its basis the limit state of shakedown under repeated applications of the factored rating vehicle. This means that some inelastic damage is permitted under the factored load, but after initially sustaining some residual deformation, all further applications of this load are to be resisted elastically because of the beneficial effects of the induced residual moments.

On the basis of this research philosophy, a set of computational procedures, incorporated in the scheme called Inelastic Bridge Rating (IBR), was developed and tested against the results of a model bridge experiment. Application was made to the rating and modification of existing bridges, and an addendum was proposed to a contemporary bridge rating specification.

1.3 SCOPE OF THE INVESTIGATION

This project encompassed the development of inelastic bridge rating guidelines for steel plate girder and composite bridges. The rating process has a number of substantial differences over the design process: member sizes and dimensions are known, actual material strengths are available or can be determined, and the dead load can be determined from field data. Consequently, it is possible to reduce the dead load factor and increase the resistance factors because of a reliable knowledge of the geometry and material properties of the existing bridge.

Explicitly stated, the objective of this project was to develop bridge rating guidelines that follow the framework presented in *NCHRP Report 301* (*3*). The primary difference is that the guidelines developed in this investigation will account for the inelastic capacity of the steel structure.

The inelastic reserve strength that will be considered in the rating method may be attributed to several sources including: (1) two-dimensional interaction between bridge elements; (2) plastic force redistribution; (3) unaccounted-for restraints and redundancies; (4) induced automoments existing from prior inadvertent overloading (automoments are residual moments that remain in the statically indeterminate structure after the loads causing inelastic behavior are removed); and (5) post-buckling strength of plate elements.

Rating methods that account for these factors were developed in this project. These methods will permit more precise rating of older bridges and may also be used to plan more effective strengthening schemes.

The remainder of this report highlights the research in the main body of the work. Details are found in the eight Appendixes (Appendixes A, B, F, and H are published herein; C, D, E, and G are available for loan) and five academic dissertations prepared by the five graduate students who worked on this project (*4–8*).

CHAPTER 2

FINDINGS

2.1 INTRODUCTION

The main objective of this research project was to develop a method of rating existing straight steel beam and girder bridges that makes use of the inelastic reserve capacity inherent in steel. All highway bridges are evaluated periodically to assess their current condition in relation to their status when they were last inspected. This exercise is performed in the United States every 2 years as mandated by law. The process of bridge evaluation consists of two important operations: *Inspection* and *Evaluation and Rating.*

Bridge inspection determines the actual condition of the bridge based on field inspection, field measurements, and possibly load testing. Careful records are taken and the results of the current inspection are compared to previous records to determine if there are any changes in the bridge condition. If there are substantive changes, or trends of deterioration are verified, then the bridge is rated for the new conditions.

Bridge evaluation and rating is a mathematical exercise by which the strength of the bridge is determined. The specific outcome of the analysis is the rating factor (*RF*). The rating factor is the ratio of the calculated capacity of the bridge to the weight of the rating vehicle times an appropriate load factor. If *RF* becomes less than unity, then the bridge is judged to be deficient, and some type of action is called for, such as:

1. Posting (reduce live load and/or speed),
2. Repairing the bridge,
3. Replacing the bridge, or
4. Closing bridge to traffic.

One very important feature of the evaluation and rating process is to subject the mathematical conclusions to the judgment and experience of professional bridge engineers.

2.2 BRIDGE EVALUATION AND RATING METHODS

2.2.1 General Discussion of Bridge Evaluation and Rating

Bridge evaluation and rating is concerned with two major issues: (1) What vehicle, or group of vehicles, should the bridge be rated for? (2) How should the capacity of the bridge be evaluated?

The first issue depends on the authority having jurisdiction over the bridge—an AASHTO design vehicle, an AASHTO rating vehicle (see Figure 2.1), a State-specified vehicle, or a Special Permit vehicle may be the basis of rating. The selection of the rating vehicle is not within the scope of this work but is mentioned to emphasize the importance of this issue to the general topic of bridge rating. The second issue, or the question of how the capacity of an existing bridge should be evaluated, is addressed in this research.

Bridge evaluation and rating is generally performed in the U.S. according to the requirements of the AASHTO *Manual for the Maintenance Inspection of Bridges* (*9*). An alternate method has also been adopted by AASHTO (*10*) based on the research by Verma and Moses (*3*). The present report proposes an extension of this AASHTO alternate method to include inelastic behavior.

The components of the capacity evaluation during bridge rating are the following:

- Load factors (or safety factors)
- Resistance factors
- Methods of structural analysis
- Limit states

Load factors are factors by which the nominal load effects are multiplied to account for the uncertainties inherent in the load and load effect determination. Load factors are typically larger than unity. *Resistance factors* are factors by which the nominal resistances are multiplied to account for the uncertainties inherent in the determination of the resistances. Resistance factors are typically less than unity. Nominal loads and resistances are the specified loads and strengths based on the legislated rating vehicle(s) and on specified material properties, respectively. *Methods of structural analysis* deal with the formal determination of the load effects from the given loads for the given geometric, cross-sectional, and material properties. *Limit states* define limits of structural usefulness.

With regard to structural analysis and the limit states, there are presently three AASHTO methods for designing steel beam and girder bridges against the ultimate load, which is defined as the weight of the design vehicle multiplied by its load factor:

1. *Service Load Design Method* (also called *Allowable Stress Design,* to be abbreviated herein as ASD);
2. *Strength Design Method* (also called *Load Factor Design,* to be abbreviated herein by LFD);
3. *Alternate Load Factor Design Method* (also called *Autostress Design,* to be abbreviated herein as ALFD).

The structural analysis required for the ultimate force determination of bridges with compact members, and the corresponding limit states, are as follows:

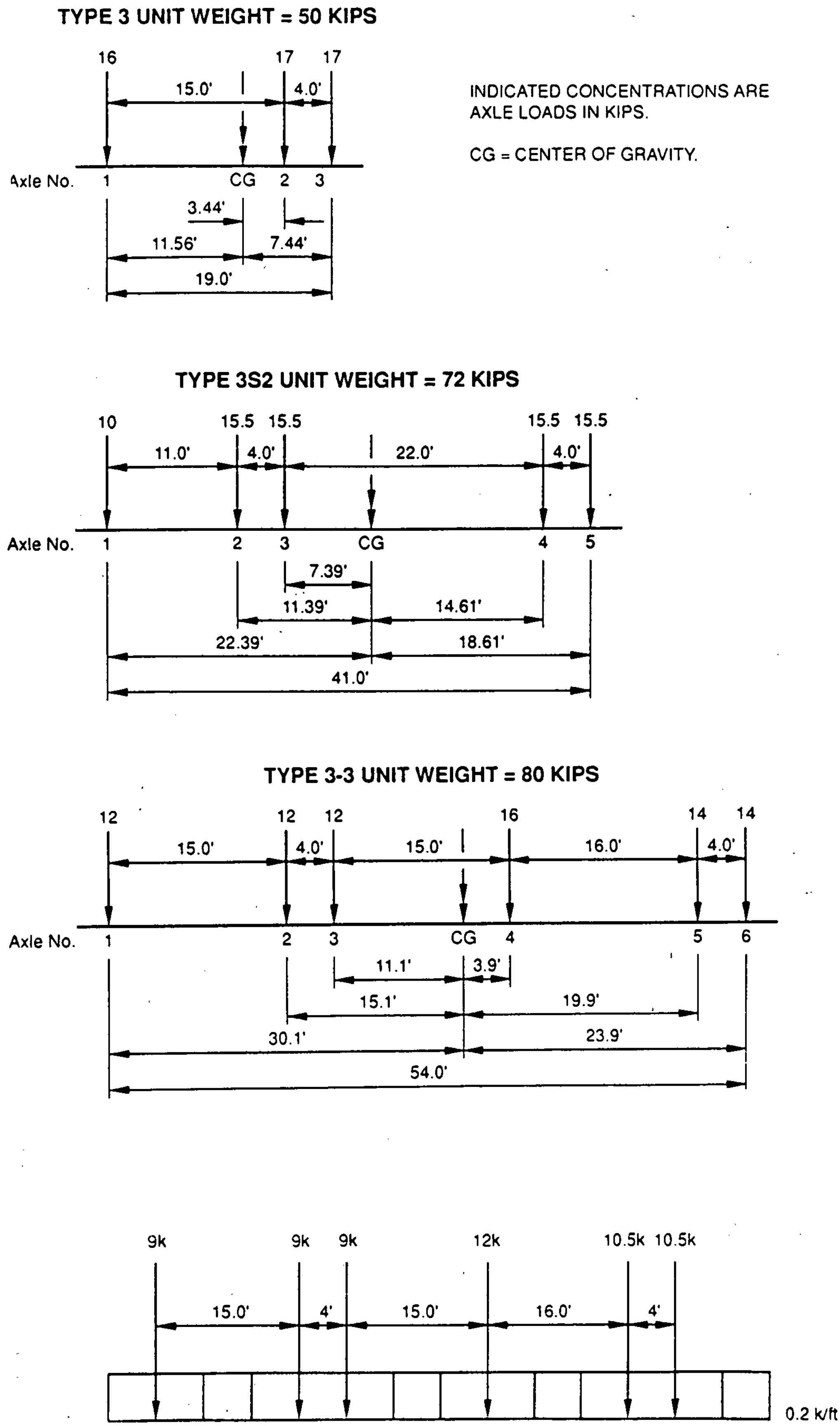


Figure 2.1. AASHTO rating vehicles (9).

Method of Design	Method of Analysis	Limit State
ASD	linear elastic	initiation of yield
LFD	linear elastic	formation of plastic hinge
ALFD	plastic	formation of plastic mechanism

While the first method appears to strictly enforce elastic behavior under the factored (ultimate) load, a 20 percent increase of the allowable stress is permitted at the interior supports of continuous beams. The second method also permits an inelastic moment redistribution by allowing a reduction of 10 percent of the moment at the interior support. The first two methods allow implicit plastification of an unknown extent, whereas the third method uses explicit plastic action. Thus it is important to recognize that the traditional bridge design methods have built into them an implicit expectation of inelastic action at ultimate load.

2.2.2 Selection of Methods to Be Examined

The following four rating documents were considered in this study:

- AASHTO *Manual for Maintenance Inspection of Bridges* (*9*)
- The *Ontario Highway Bridge Design Code* (*11*)
- The British *Assessment of Highway Bridges and Structures* (*12*)
- AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* (*10*)

The decision to examine these four documents stems from discussions with rating engineers from different states in the U.S. and with researchers from around the world. In the U.S., representatives of transportation agencies in Nebraska, South Dakota, Florida, Wisconsin, New Mexico, Colorado, North Carolina, Minnesota, Illinois, Pennsylvania, Missouri, and Tennessee were contacted through letters and telephone calls. From these informal discussions and contacts, it was clear that the AASHTO "Manual for Maintenance and Inspection" is used almost exclusively in the U.S. Although some states use slightly different vehicles for rating (*13*), the analytical tools and the rating philosophy are basically the same.

Insofar as bridge rating in other countries is concerned, the general approach is very much the same (*14*). Researchers from Canada, Great Britain, Germany, Italy, France, Spain, Australia, and Japan were contacted. From their comments, it appears that there is little difference made between design and rating approaches for the individual countries. The British and Ontario rating specifications were chosen for further examination.

2.2.3 Description of Bridge Rating Methods

The following questions were posed for each of the three rating methods:

1. How is the current state of the structure considered in the rating?
2. What are the rating vehicles?
3. What are the limit states?
4. What are the underlying reliability assumptions?
5. What are the structural models to be used in rating the bridges?

2.2.3.1 AASHTO Manual for Maintenance Inspection of Bridges (9)

This Manual begins with requirements of the key operation of any rational bridge evaluation—the process of inspection. Questions of who is qualified to inspect, how often inspection should take place, how and what to inspect, and what to report are laid out in detail in the manual. For the purposes of this discussion, it is important to note that the frequency of inspection is at least once every 2 years. All portions of the bridge are to be thoroughly examined. The criteria for steel girders state that the inspector must record all signs of cracking, buckling, corrosion, or misalignment. The inspection report must reflect the actual state of the bridge at the time of inspection, as well as the changes in this condition since the previous inspection. The rating engineers will thus have at their disposal the data necessary to perform the capacity evaluation. In this evaluation, they must use the actual dimensions of the members, reduced as necessary for corrosion or other damage based on field measurements. Dead loads must be computed from the actual mass of the bridge.

Two rating levels are defined: "Inventory Rating" and "Operating Rating." Inventory rating is for live loads that can be safely supported for an indefinite period of time. Operating rating is the load level corresponding to the capacity of the structure. The individual state transportation departments may specify their own standard vehicle for the inventory rating. The AASHTO Manual recommends the use of either the standard H or HS vehicle, or the more critical vehicle of three "Typical Legal Load Types" shown in Plate 11 of the Manual (see also Figure 2.1). These three trucks were selected from actual maximum legal loads conforming closely to regulations of a major number of states. The difference between the three rating trucks is essentially the axle spacing and the number of axles (i.e., the length of the vehicle). As far as the live loading on the bridge is concerned, the vehicles used in rating represent "typical" legal conditions to be expected on the bridge. If a specific overweight vehicle exceeds the legal limit, a special permit must be issued. In no instance may this vehicle exceed the operating rating. The operating rating is the absolute maximum permissible load level to which the structure may be subjected.

The AASHTO Manual contains general guidelines as to the number of loaded lanes and the number of trucks per lane. For bridges of less than a 200-ft span, the usual load is one truck per lane. Wind loads, longitudinal loads, thermal forces, and deflection limits need not be considered in rating. The AASHTO (*15*) design values are to be used for impact, but judgmental reductions are allowed when it is not possible, due to particular conditions of alignment, enforced speed posting, and so on, to attain the maximum legal speed limit. The distribution of axle loads to stringers, beams, and girders is in accordance with the AASHTO design specifications.

The limit states implied in the AASHTO Manual are those of the current (1989) AASHTO design specification. The bridge may be rated either by the Allowable Stress Design (ASD) or by the Load Factor Design (LFD) method. In the case of inventory rating with ASD, the basic flexural stress for a compact shape is $0.55F_y$, where F_y is the applicable yield stress of the steel. When LFD is used in rating, then the ultimate member capacities and the load and resistance factors of the AASHTO design specification apply.

The limit states in the AASHTO rating manual are thus the same as those used in design. However, actual dimensions and properties are used in the capacity checking process. The underlying assumptions about reliability are in fact the same as those for design. However, in Section 5.1.3 of the AASHTO Manual, recognition is given to the fact that rating and design are not the same as far as reliability is concerned. This section states:

> For all matters not definitely covered by these specifications, the current standard specifications used for the design of new bridges shall be used as a guide. However, there may be instances in which an Engineer, based on his knowledge of the condition and performance characteristics of a bridge under traffic, may make a judgment that the action of a member within the structure is not consistent with the design concept of the controlling specifications. In this situation, he may modify the design criteria within safe limitations and, following sound principles of engineering mechanics, base his capacity analysis for the member on its known action under load. Deviations from controlling specifications shall be fully documented.
>
> As a guide to where modifications of design practice may be considered, the following facts should be kept in mind:
>
> 1. The factors of safety used in designing new bridges may provide for an increase in traffic volume, a variable amount of deterioration, and extreme conditions of long continued loading. Use of the Operating Rating as the load limit of existing bridges applies only to frequently inspected bridges. Bridges which have weight limits or have members stressed to near the operating rating stress are inspected more frequently than other structures; hence, the rating in reality is being reevaluated by the Engineer at each inspection through determination if any deterioration or distress has occurred which will materially affect its load carrying capacity.
> 2. The factors of safety used in rating existing structures must provide for unbalanced loads, reasonably possible overloads and illegal or careless handling of vehicles. For both design and rating, factors of safety must provide for lack of knowledge as to the distribution of stresses, possible minimum strengths of individual pieces of the materials used as compared to quoted average values, possible differences between the strength of laboratory test pieces and the material under actual conditions in the structure, and normal defects occurring in manufacture or fabrication.

While the differences are thus recognized, no specific means are provided to quantify the distinctions in the rating process. The limit states are those of members rather than of the system, and no cognizance is given to redundancy and to past successful performance.

The modeling of the bridge for analysis is based on the same assumptions as are made for design, taking into account the relevant actual conditions of the bridge. This means that for beam and girder bridges the forces are determined by elastic analysis for the individual members utilizing the AASHTO distribution procedures.

While one can recognize obvious shortcomings of the AASHTO method of rating existing bridges, a careful reading reveals that the *Manual for Maintenance Inspection* is a comprehensive and sophisticated document. It places high emphasis on judgment based on experience, and its shortcomings err on the side of conservatism and safety. This method of rating is essentially an extension of the design process for an existing bridge; it does not recognize the benefits of having survived many of the uncertain events for which load factors are needed in design.

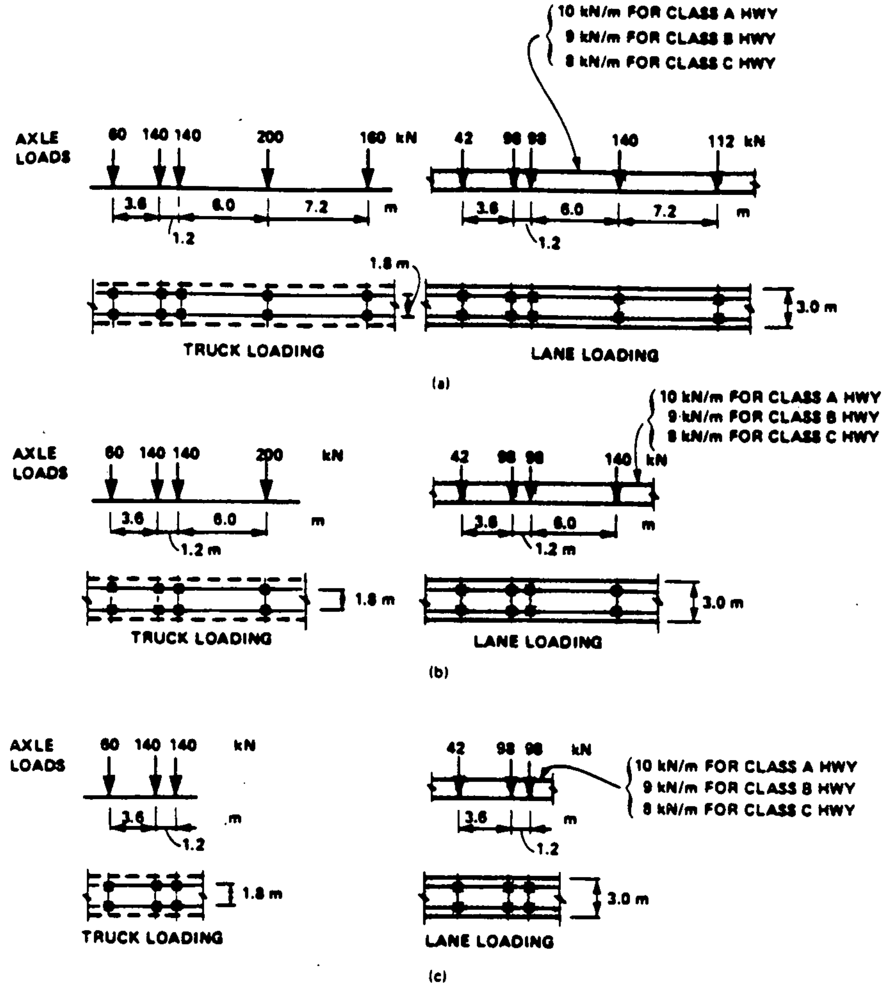
2.2.3.2 The Ontario Highway Bridge Design Code (11)

The bridge rating criteria are contained in Chapter 14, "Evaluation of Existing Bridges," of the *Ontario Highway Bridge Design Code*. The Ontario criteria are in many ways very similar to the AASHTO criteria; however, there are some notable differences, which will be highlighted in the following discussion.

The key element of the Ontario method is also the inspection prior to the evaluation. In the Ontario code this operation is called "Condition Survey." The main goal is "to determine if the structural assemblage and its individual components are responding to the past and present loads in an acceptable manner and to extrapolate these results for a reasonable period of time into the future" (*11*). The condition of the bridge, its material properties and dimensions, as well as any damage is to be recorded. The condition survey thus presents the rating engineers with the data necessary to make their evaluation. Of particular importance are the measurements of the actual cross section as these are affected by distortion or corrosion.

The bridge evaluation method of the Ontario code is appropriate "only to those bridges which have proven capable of sustaining wind, ice forces and other environmental loads." Serviceability limit states are not considered. Rating is thus performed for the actual dead load on the bridge plus the live load due to vehicles. The vehicle could be a special heavy vehicle, called the "Controlled Vehicle," for which it is necessary to determine the capacity of the bridge, or it could be the inventory rating vehicle called the "Ontario Bridge Evaluation" (OBE) vehicle. This latter load is either the OHBD (Ontario Highway Bridge Design) truck, or one of two truncated subsets of it. Figure 2.2 shows the three trucks, designated as Levels I, II, and III. Level III is the OHBD vehicle. All three levels must be considered in the evaluation. In addition to the three Ontario Bridge Evaluation vehicles, there are also three companion lane and point load combinations to be considered.

The OBE vehicles may be compared to the corresponding AASHTO rating vehicles shown in Figure 2.1. The total truck length and clustering of axles is similar; however, the Ontario loads are far heavier than the AASHTO loads. This is one of the significant differences between the two rating methods. The other difference is that the Ontario impact factor is determined as a function of the first flexural frequency of the bridge. Over a certain range of frequency, the Ontario impact factor exceeds the maximum AASHTO value by 33 percent (i.e., max. of 0.4



Ontario design and evaluation loads: (*a*) design loading and level 3 evaluation loading; (*b*) level 2 evaluation loading; (*c*) level 1 evaluation loading.

Figure 2.2. Ontario Bridge Design Code rating vehicles (11).

versus 0.3). However, it has been reported that the next edition of the Ontario code will not contain this distinction.

The limit states in the Ontario code are the same ultimate limit states as those that apply in design. Thus only member or component failure rather than system failure is contemplated as the limit of structural usefulness. The only difference between design and evaluation is that in the latter process the actual, possibly deteriorated, cross-sectional properties are used.

Evaluation of existing bridges is performed for the same load factors and resistance factors as are specified for design. These factors were determined by calibration to provide a uniform reliability against exceeding a limit state for each structural member or component. This common reliability is approximately equal to a reliability index of 3.5, or, the chance of exceeding a limit state is roughly 1 percent during the bridge lifetime of 50 years. Member reliability rather than systems reliability is the basis of the load and resistance factors.

As in the AASHTO rating manual, the philosophy in the Ontario bridge rating criteria is to use the same behavioral models and limit states in both design and evaluation.

2.2.3.3 The British Code (12)

The British document titled *The Assessment of Highway Bridges and Structures*, which consists of a Standard and a Commentary, is very similar in its requirements to the two corresponding documents from North America. One of the differences, however, is the emphasis in the British document on much older and different bridges, particularly on stone and masonry bridges.

The initial process is a thorough inspection of the bridge to be evaluated in order to determine its condition. Field observations and measurements, as well as original drawings and material data, are used to give the engineering assessor the data needed to make the rating.

In this rating method, the actual dead loads and the actual cross-sectional properties are to be used. With regard to live loading, only loads due to vehicles need to be considered. The vehicle loading criteria represent the effects of the full range of vehicles allowable under the general British "Construction and Use Regulations" (*16*). General bridge assessment is thus made for the same live loading as is used in design.

The standard loading per lane is composed of a uniformly distributed load that acts over the "loaded length," and a concentrated force of 120 kN (27 kips). The "loaded length" is defined as "the base length of that area under the live load which produces the most adverse effect at the section being considered." The magnitude of the distributed load is given as the conservative envelope from all legal vehicles by the following equation:

$$W = 260L^{-0.6} \qquad \text{(Eq. 2.1)}$$

where W is the uniformly distributed load in kN/m, and L is the loaded length in meters. Thus for a loaded length of 50 ft (15.24 m), W = 50.72 kN/m or 3.475 kip/ft. This loading includes allowances for impact, overloading, and "lateral bunching" (two vehicles passing or traveling side-by-side).

The method of assessing the capacity of a bridge is to use ultimate limit states with load factors and resistance factors. These ultimate limit states are those of the corresponding design standard, which is British Standard BS 5400 "Steel and Concrete Bridges" (*17*). This is a modern probability-based Limit States Design specification, similar to the Ontario Code. The development of the steel bridge design part of BS 5400 is documented in References *18* and *19*. The limit states are those of the individual members and components of the bridge, rather than the limit states of the bridge system. The aim of BS 5400 is that all components of the bridge should have a common reliability. The load and resistance factors were obtained by calibration to bridges designed by the previous code.

The British rating code gives relatively little guidance for the analysis of steel structures, concentrating in greater detail on the modeling of masonry arch bridges. The Commentary provides charts and formulas for determining the lateral distribution of the lane load. This rating method is, again, an extension of the design criteria.

2.2.4 Comparison of the Three Rating Methods

To compare and contrast these documents, a simple example bridge will be evaluated by the AASHTO, Ontario, and British rating methods for an overload condition. Following are the details of the bridge:

- 100-ft-long (30.5 m) simple span
- 24-ft (7.5 m) wide (2 lanes)
- 6-ft (1.75 m) girder spacing (interior girder)
- 7-in. (179 mm) concrete slab and a 3-in. (76 mm) wearing surface
- Dead loads: Steel = 0.3 kip/ft of one girder
  Concrete = 0.524 kip/ft
  Wearing surface, curbs and rails = 0.2 kip/ft

The results are summarized in Table 2.1.

AASHTO Method, Using Load Factor Design (9)

The factored DL per ft/girder is equal to:

$$1.3\ DL = 1.3\ (0.3 + 0.525 + 0.2) = 1.33 \text{ kip/ft}$$

Table 2.1. Rating example summary

		AASHTO	ONTARIO	BRITAIN
Factored Dead Loads = L.F. x w_D (k/ft.)	Steel	1.3 x 0.30	1.1 x 0.30	1.05 x 0.30
	Concrete	1.3 x 0.52	1.2 x 0.52	1.15 x 0.52
	Others	1.3 x 0.20	1.5 x 0.20	1.75 x 0.20
Total Dead Loads (k/ft.)		1.33	1.26	1.27
Live Load Factor		1.66	1.40	1.50
Impact Factor		0.222	0.400	....
Distribution Factor		1.09	0.96	0.55
Axle load multiplier (to wheel)		1.444	0.941	....
Controlling Vehicle or Loads		Type 3-3	Vehicle III	w = 3.16 k/ft. P = 22.3 k
Design Moment (ft.-kips)		3,580	4,430	5,700

L.F. = Load Factor
w = Load / Unit Length
w_D = Dead Load / Unit Length

The impact factor is:

$$I = 50/(100 + 125) = 0.222$$

The distribution factor is:

$$K = 6/5.5 = 1.09$$

The factor by which one axle load is to be multiplied equals:

$$1.3\ (5/3)(1.09)(1/2)(1 + 0.222) = 1.444.$$

The controlling truck is the Type 3-3 AASHTO unit (AASHTO truck III in Figure 2.1), and the design moment (including dead load and vehicle load with impact) is 3580 ft-kips. The bridge girder must have a nominal resistance equal to or larger than this value.

The Ontario Code (11)

The Ontario rating criterion is expressed by

$$\sum_{i=1}^{n} \alpha_{Di}\, D_i + \alpha_L\, L = \phi R_n \qquad \text{(Eq. 2.2)}$$

where α_{Di} = partial load factors, D_i = dead load, ϕ = resistance factor, and R_n = the nominal resistance. For beams and girders, ϕ = 0.9 is the appropriate value from the Ontario code.

The factored dead load per girder is determined with the following partial load factors: 1.1 for the weight of the steel, 1.2 for the weight of the concrete, and 1.5 for the weight of the wearing surface and the bridge "furniture" (guardrails, lights, curbs, sidewalks, etc.). The factored dead load is thus:

$$\sum \alpha_{Di}\, D_i = 1.1 \times 0.3 + 1.2 \times 0.525 + 1.5 \times 0.2 = 1.26 \text{ kip/ft}$$

The live load factor α_L is equal to 1.4. The distribution factor is the girder spacing (in meters) divided by 1.9, or:

$$K = 6 \text{ ft} \times 0.3048/1.9 = 0.96$$

For spans of 22 to 60 meters, an impact factor of 0.4 is recommended. The factor by which axle loads are to be multiplied for analysis is thus:

$$1.4\ (1 + 0.4)(0.96)(1/2) = 0.941.$$

The controlling vehicle is the full Ontario truck (vehicle III in Figure 2.2), and the required reduced nominal resistance moment ϕR_n = 4430 ft-kips.

The British Code (*12*)

The dead load factors in the British code are 1.05, 1.15, and 1.75 for the steel, the concrete, and the surfacing and bridge furniture, respectively. The factored dead load is, therefore:

$$\sum \alpha_{Di}\, D_i = 1.05 \times 0.3 + 1.15 \times 0.525 + 1.75 \times 0.2 = 1.27 \text{ kip/ft}$$

The live load is given as a uniformly distributed and knife edge lane load. For a simple span the influence length is equal to the span, which is 100 ft, or 30.5 meters. The uniform load is thus:

$$260(1/30.5)^{0.6} = 33.5 \text{ kN/m} = 2.29 \text{ kip/ft}$$

The knife-edge load is 27 kip. These loads already contain an allowance for impact. The distribution factor is 0.55 and the live load factor is 1.5.

The loads for analysis are thus as follows:

1. Uniformly distributed load = 1.27 + 1.5 × 2.29 × 0.55 = 3.16 kip-ft;
2. Concentrated load at the center = 1.5 × 27 × 0.55 = 22.3 kip.

The required maximum moment is thus:

$$M_u = 3.16 \times 100^2/8 + 22.3 \times 100/4 = 4510 \text{ ft-kip}$$

This required bending moment must yet be multiplied by a factor of 1.1 to account for the uncertain features in the determination of the load effect.

The resisting nominal moment M_n must be multiplied by a factor, less than or equal to unity, which accounts for the condition of the bridge. If all of the cross-sectional dimensions are actual measured values, then this factor is unity. This will be the case assumed here. An additional factor is specified by which the yield stress is divided to account for the uncertainty of the material property. This factor is 1.15 (*11*). The British design criterion is, therefore:

$$1.1 \times 4510 = 1.0\ M_n/1.15 \text{ or } M_n = 5700 \text{ kip-ft}$$

AASHTO Guide Criteria (*10*)

The *NCHRP Report 301*, "Load Capacity Evaluation of Existing Bridges" (*3*), on which the AASHTO *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* (*10*) is based, makes recommendations for bridge rating that are based on extensive statistical and probabilistic studies of bridge loading and resistance. The report recommends the same procedures for structural analysis and truck load configurations as the AASHTO bridge design specification and the AASHTO rating manual, and it uses the same limit states. However, a wide range of judgmental factors are suggested to account for the existing condition of the bridge. The general limit state equation is

$$\tau_D D_n + \tau_L (DF) L_n (1 + I) \leq \phi R_n \qquad \text{(Eq. 2.3)}$$

where D_n and L_n are the nominal dead and live load effects, R_n is the nominal resistance, or member capacity, DF is the distribution factor, and τ_D = 1.2 is the dead load factor.

The live load factor takes on different values depending on the traffic volume and the overload enforcement levels. For example, for low volume roadways (i.e., average daily truck traffic (ADTT) less than 1000), τ_L = 1.30 if there is reasonable enforcement, and τ_L = 1.65 if there is a lack of effective enforcement. If the ADTT is above 1000, the corresponding live load factor becomes equal to 1.45 and 1.80, respectively.

The impact factor I depends on the condition of the wearing surface. It equals 0.1 if the wearing surface is in good to fair condition, 0.2 if it is in need of repair in order to continue functioning as designed, and 0.3 when it is in critical condition, i.e., it no longer functions as designed.

The resistance factor ϕ = 0.95 for redundant steel members (a multigirder bridge is defined as being redundant). However, ϕ becomes equal to 0.85 when field inspection reveals slight deterioration with a slight loss of section, and 0.75 when significant deterioration and heavy section loss is observed.

For the 100 ft beam of the previous problem, the unfactored dead load equals 1.025 kip/ft. The factored dead load then becomes:

$$1.2\ DL = 1.2 \times 1.025 = 1.23 \text{ kip/ft} = M_D$$

The live load portion of Equation 2.3 becomes, for τ_L = 1.45 (heavy volume roadway, good control of overloads), I = 0.2 (poor wearing surface), and DF = 6/5.5, equal to:

$$\tau_L (DF) L_n (1 + I) = 1.45 \times (6/5.5)(1/2) \times (1 + 0.2) M_L = 0.949 M_L$$

For a slightly deteriorated girder section the resistance factor is 0.85. Thus

$$M_D + 0.949 M_L \leq 0.85\ M_n$$

expresses the rating condition; M_D is the moment due to the dead load of 1.23 kip/ft, M_L is the moment from the critical AASHTO rating truck, using the appropriate axle loads, and M_n is the required nominal resistance moment. The resulting value of M_n is 3305 ft-kips for the example bridge. This value compares with the required nominal required moment capacity of 3580 ft-kips for the usual AASHTO rating according to Reference *9*.

Table 2.2. Influence of judgmental factor

I	ϕ	γ	Nominal Moment (ft.-kip)	Condition
0.1	0.95	1.30	2719	EXCELLENT
		1.45	2846	
		1.65	3015	
		1.80	3142	
0.2	0.85	1.30	3150	POOR
		1.45	3305	
		1.65	3512	
		1.80	3666	
0.3	0.75	1.30	3697	VERY POOR
		1.45	3887	
		1.65	4141	
		1.80	4331	

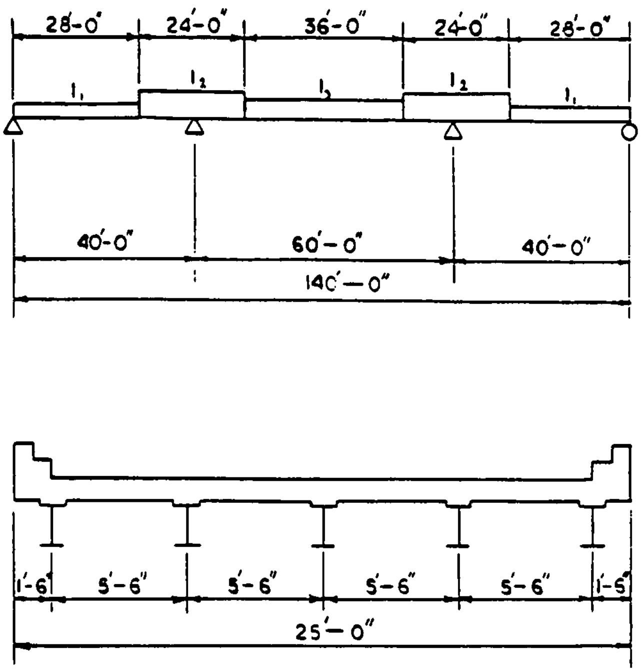


Figure 2.3. Dimensions of bridge rated in Reference 4.

There are three major judgmental factors in the rating method of Reference *10:* the determination of (1) the impact factor, (2) the live load factor, and (3) the resistance factor. All three require subjective judgement based on experience and comparison. The results from the full range of parameter variations are shown in Table 2.2. By comparing these values to the standard AASHTO rating of Reference *9* (i.e., M_n = 3580 ft-kips), it is evident that the AASHTO method is biased toward bridges in poor condition. The official rating procedures (AASHTO, Ontario, British), therefore, do not have the degree of flexibility that is provided by the methodology of Reference *10.*

A more detailed comparison of the three rating methods is given in Reference *4,* where the bridge shown in Figure 2.3 was investigated. This three-span five-girder bridge was designed by the 1941 AASHTO bridge design specifications using A7 steel (F_y = 33 ksi) and shapes found in the 1937 AISC

Table 2.3. Comparison of rating methods

	AASHTO	OHBDC	BRITISH
γ/θ	2.17	1.56	1.90
Distribution Factor	1.0	0.82	0.53
Rating Factor	1.85	1.59	1.00

Manual. This bridge was rated by the AASHTO (1983), the Ontario, and the British criteria, using the respective vehicles and factors of each document. Table 2.3 gives the various factors, load effects, and rating results.

A quick observation of the rating factors indicates that the bridge rates the highest by the AASHTO method. This might lead one to think that this is the least conservative method; however, a closer inspection of the above information will shed some light on why this method rates the bridge so high.

The factored dead loads compare closely for each method, and for the AASHTO and Ontario methods, the impact factors are similar. The British impact factor is difficult to compare directly since it is applied differently from the other two methods. The distribution factor calculated by the Ontario method is less conservative and probably more accurate than the AASHTO factor. Again, a direct comparison with the British factor cannot be made because their factor is applied to an entire lane of loading while the other two are applied to half of a truck. However, the corresponding load per lane for the other methods is a full truck, so a comparison could be made by doubling the British number or halving the others. This would give a distribution factor comparable to the AASHTO value (1.05 vs. 1.0).

A comparison of the combined load and resistance factors shows that the AASHTO factors are the most conservative, exceeding the British values by about 15 percent and the Ontario numbers by about 40 percent. This still does not explain the results of the ratings. In fact, it contradicts the earlier conclusion based only on the rating factors. The major difference can be found by comparing the applied loadings. The best comparison can be made by looking at the live load effect of each truck model after the lateral distribution factors and dynamic effects have been included.

The resulting live load moments for each method then become:

AASHTO—180 ft-kips (1.0)(1.27) = 229 ft-kips (100%)
Ontario—349 ft-kips (0.82)(1.3) = 372 ft-kips (162%)
British—903 ft-kips (0.525) = 474 ft-kips (207%)

The Ontario load effects exceed the AASHTO load effects by more than 60 percent while the British load effects exceed them by more than twice as much. Although the other factors are significant, it is the loadings that most influence the final results.

The British method provides the greatest loadings (which include overloads) perhaps to account for the lack of provisions for permit vehicles. The Ontario method provides the lowest combined factors reflecting the greater certainty in their load models, and in the extensive work involved in developing these factors. While the AASHTO loads are much lighter, the method gives larger load factors. The Ontario and British methods seem to represent their loads more realistically than the AASHTO method, while the AASHTO method attempts to account for a greater uncertainty in the loading conditions by applying larger load factors. The AASHTO provision allows for the use of more realistic loadings if the provision to adapt rating vehicles to meet local conditions is used.

2.2.5 Observations Regarding the Evaluation Methods

One common feature of the three national rating methods is the requirement of thorough and frequent inspection. This process must be performed by experienced engineers and accurate records must be kept. Much depends on the observations and judgments made in the field, and in the way information is transmitted to those who later must perform analyses and evaluations based on the field data.

In addition, the three rating methods have the following two features in common:

1. Only dead loads and vehicle loads are considered to be of consequence in rating. Such loads as ice, wind, snow, earthquake, and other environmental effects are not counted in rating, except when unusual special conditions warrant inclusion. It is tacitly assumed that the chance of a simultaneous occurrence of an exceptionally heavy traffic loading and a major natural disaster is negligibly small.
2. The modeling of bridge behavior under load is made with the assumption that the material is elastic up to the limiting capacity of the member. The behavior model and the limit states of rating an existing bridge are the same as those used in current design. An older existing bridge is thus expected to conform to the current standards of design and it must support current traffic. While the AASHTO rating manual recognizes that there are differences between designing a new bridge and rating an old one, the fact is that all three evaluation methods assume the same level of reliability for a new bridge as for an existing bridge, which has a far lesser level of uncertainty. After all, an older existing bridge has already survived many extremes of loading.

Only the rating scheme of Reference *10* has a different reliability index for existing bridges.

The limit states of rating, as well as design, are the limiting capacities of the members and elements, not of the whole structural system. At present, there is no penalty imposed on a nonredundant system. The scheme in Reference *10* to reduce the basic resistance factor ϕ from 0.95 for a redundant system to 0.80 for a nonredundant system is an attempt to cope with differences between "member" and "system" reliability.

The two rating exercises in the previous part of this report have indicated that the authorities in the U.S., Canada, and Great Britain expect different levels of performance of their existing bridges. These differences are in part due to differences in the rating vehicles. The AASHTO requirements apparently rely on control to keep the loads below the legal limits while the other two countries appear to be somewhat more realistic in appraising the actual conditions on the road. In addition to the differences in live loads, there are differences in the magnitudes of the impact factors (Ontario and Great Britain requirements are generally higher than those of AASHTO), the distribution factors, the load factors, and the resistance factors.

Each of the three rating documents stresses the importance of judgment and experience in the inspection phase of the evaluation process, but once the field data are evaluated the remaining calculation procedures follow fairly explicit rules.

The three national rating methods described in this report represent the current typical evaluation procedures. Fundamentally, the assessment of an existing bridge is done by the same procedures, with the same assumptions regarding loads and behavior, and for the same reliabilities and limit states, as the design of a new bridge. Experience has demonstrated that these methods produce satisfactory results. There are questions as to whether they are too conservative, and whether they are flexible enough to deal with different conditions of traffic control and bridge deterioration. Reliability is focused on individual components rather than on the total system.

However, there are a number of research issues that can be considered in order to improve bridge rating, including:

- Development of simple methods accounting for the reliability of the total bridge system, i.e., rational assessment of the effects of redundancy.
- Development of rational resistance criteria that would permit the ability to carry higher loads by using inelastic limit states.

2.3 ANALYSIS AND EVALUATION OF RESEARCH

2.3.1 Introduction

This section summarizes a survey of relevant research. The areas investigated include the behavior of composite and plate girder bridges; analytical approaches to bridge rating; plastic design, shakedown and autostress design; bridge tests to ultimate; probabilistic methods; and repair, rehabilitation and retrofit. The literature reviewed was restricted to straight, simple and continuous span, composite and noncomposite beam and girder bridges. These types of bridges constitute the vast majority of older steel beam and girder bridges that this investigation intends to address.

2.3.2 Composite Bridges and Plate Girders

Straight highway bridges composed of steel girders, a composite concrete deck, and diaphragms at close intervals are highly redundant and safe structures (*20,21*). The substantial ultimate overstrength that these structures exhibit over design loads stems from both the large number of load paths available in multigirder bridge systems and the large ductility and compactness of typical members. The advantages of composite construction in highway bridge design have long been recognized and were translated into practice following the pioneering work of Newmark *et al.* (*22*). Since then, a large amount of experimental and analytical work aimed at developing and evaluating composite girder systems has been carried out in the U.S., England, Australia, and Japan. Research on composite beam bridges relevant to rating can be subdivided into two areas:

1. Studies on the behavior of individual beams or girders where the loading effects are almost exclusively longitudinal. Until the late 1960s, most of the work in this area had been experimental, and geared to identifying failure modes, studying continuity of the system and shear transfer at the steel-concrete interface, and deriving design provisions to produce ductile failures. Since the early 1970s, there has been considerable work aimed at describing the behavior with analytical tools such as the finite element and finite difference techniques.
2. Behavior of assemblages of beams and girders where both the transverse and longitudinal load effects are included. The research in this area has focused on the development of simplified analytical models to form the basis of design procedures such as those in the AASHTO (*15*) and OHBD (*11*) design codes. Much of the recent work has focused on using sophisticated three-dimensional linear and nonlinear finite element codes to verify and calibrate two-dimensional orthotropic models suitable for design. The latter have been further simplified into the "distribution factor" procedures currently at the heart of design provisions in North America.

Both of these research areas are directly applicable to inelastic rating because the former insures sufficient ductility for the formation of plastic mechanisms, while the latter provides simple methods of calculating the ultimate capacity of the bridge. A short discussion of the current issues in these two areas follows.

2.3.2.1 Behavior of Composite Girders

The strength characteristics of individual composite girders with full interaction are well understood in both the elastic and inelastic range. A large number of experimental and analytical studies have been carried out over the past 25 years (*23–33*) and a large database is available for parametric studies. Very simple expressions to calculate the yield and ultimate capacity of composite sections are available and have been verified.

For composite beams with partial interaction, the calculations are slightly more involved, but even for that case solutions are readily available (*34*). Of interest in rating older bridges is the case of no interaction, or the absence of shear connectors. For this particular case, most data indicate that the structure tends to behave compositely in the elastic range (*35*). At ultimate, however, the friction between the slab and girders is overcome and the bridge tends to behave noncompositely (*36*).

Although the behavior of composite beams at ultimate is well understood, the same cannot be said for the serviceability criteria of deflection and cracking. The deflection characteristics of composite beams, and particularly the long-term effects due to creep, shrinkage, and thermal cycling, have been addressed for the case of buildings (*37*) and for bridges by Roik *et al.* (*38*) and Mangerig (*39*). Unfortunately simple accurate formulas cannot be developed because the long-term effects and nonlinear response of the shear studs lead to complex formulations. Some procedures, such as those in the LRFD Manual (*34*) and those suggested by Vallenilla and Bjorhovde (*40*), can be utilized to successfully estimate the short-term deformations, and this may be sufficient for rating bridges for overloads. If more accurate predictions for deflections in the inelastic range are needed, three-dimensional finite element analysis or a modified orthotropic plate theory must be used.

Because some inelastic action is anticipated, the cracking of the slab over interior supports could become an important serviceability criterion. This cracking arises from both the material characteristics of concrete (i.e., settlement of plastic concrete during hardening and volumetric instabilities due to changes in temperature and moisture) and the structural action of the beam (i.e, negative moments and unintended restraint). In inelastic bridge rating, the main concern will be with those cracks produced by negative moments and the formation of plastic hinges at supports. Most test data indicate that these cracks will close once the load is removed, and that the modest ductility requirements likely under overload conditions do not result in excessive crack widths.

A third serviceability criterion that has received considerable attention in Europe recently is the fatigue capacity of shear studs (*41,42*). The problem is more severe in railway bridges because of the large stress ranges encountered, but in some cases highway bridges could be sensitive to this kind of failure. Because fatigue does not fall within the areas to be addressed by this research, the issue was not pursued further.

The primary intent of most of the recent work on individual composite girders has centered on identifying nonductile failure modes such as concrete crushing (positive moment) and local web and flange buckling as well as lateral torsional buckling (negative moment) (*29*). The ductility and rotational capacity of composite sections are key parameters in inelastic rating because the procedure will, by definition, accept some yielding of the section. These characteristics of composite behavior are utilized in the subsequently described rating procedure.

The issue of ductility is of particular importance in continuous bridges because elastic moment envelopes typically give larger moments over the supports than at midspan whereas the composite beam is stronger at midspan (positive moment) than over the support (negative moment). This is compounded by the fact that shape factors, or the ratio of plastic to yield moment capacity, are usually in the range of 1.25 and 1.35 for composite beams under positive moment, while they seldom exceed 1.20 under negative moment. If we assume that inelastic action will take place and that a full or partial plastic mechanism will form, the required ductility over the support would be large.

Table 2.4 summarizes the current approaches taken by design codes to minimize ductility problems, while detailed descriptions of applicable rules can be found in the work of Kemp (*31*), Ansourian (*23*), and Rotter and Ansourian (*32,33*). A review of these and other pertinent work indicates that most current design procedures insure adequate ductility, and that guidelines for ductility rating can be reliably formulated. Ansourian's work (*25*) is taken as the basis for the ductility determination in the proposed rating method.

Recently the following criteria have been proposed to insure ductility in composite beams under the current AASHTO provisions (*43*):

> 10.50.1.1.2. Composite beams qualify as compact when their steel section meets the requirements of Article 10.48.1.1 (b). D

Table 2.4. Approaches to minimize ductility problems (*31*)

Region of Beam	Mode of Failure	Variables Controlled in Design Codes	Other Relevant Variables
Positive moment (sagging)	Concrete failure in compression	Neutral axis depth reflecting ratio of extreme fiber strains in concrete and steel	Biaxial restraint and transverse bending in slab Concentrated loads
Negative moment[1] (hogging)	Local buckling of bottom flange	Flange breadth-to-thickness ratio $b_f/t_f < 16$ to 20	Moment gradient (L_i/t_f) Axial force balancing reinforcement force
	Local buckling of web	Web depth-to-thickness ratio Increased web depth in compression due to slab reinforcement $N/f_y h_w t_w$	Moment gradient (L_i/h_w)
	Lateral-torsional buckling of steel section	Slenderness ratio between lateral restraints $L_b/r_y < 35$ to 63 Moment gradient L_b/L_i	Axial force balancing reinforcement force

[1] Considerable interaction occurs between modes of failure in negative moment region.

L_i - length from section of maximum moment to adjacent point of section
t_f - flange thickness
b_f - flange width
N - axial compressive force
h_w - web depth
t_w - web thickness
L_b - braced length, in.
r_y - radius of gyration about minor axis, in.

in Eq. (10.93) may be replaced by $2D_{cp}$ for composite beams and girders used in simple spans only. D_{cp} is the distance from the compression flange to the neutral axis in plastic bending. The compression depth of the composite section including the slab must not exceed:

$$(D + t_s)/7.5 \qquad \text{(Eq. 2.4)}$$

where D is the depth of the steel beam or girder and t_s is the thickness of the slab. The stress-strain diagram of the steel shall exhibit a yield plateau followed by a strain hardening range.

Table 2.5. Analysis methods for steel girder bridges (*44*)

Method of Analysis	Bridge Types for which the Method is Permitted	Limitations of Applicability
Orthotropic Plate Theory Methods	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss, Arch, Rigid Frame Bridges incorporating Longitudinal Wood Beams Box Girder--Multi-Spine	Structure must meet requirements for simplified methods and also, (i) edge stiffening, if present, to be idealized as specified, and (ii) multi-spine bridges to be of concrete construction and have four or more spines.
Finite Element Method	All bridge types	For shear-connected beam bridges, special elements having zero transverse rigidity, may be required.
Finite Strip and Folded Plate Methods	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss, Arch, Rigid Frame Bridges incorporating Longitudinal Wood Beams Box Girder--Single Cell Box Grider--Multi-Spine	Not applicable to bridges with any of the following: (i) skew angle greater than 20°; (ii) longitudinal variations in transverse directions; (iii) support conditions other than line supports; (iv) intermediate diaphragms. For shear-connected beam bridges, special elements having zero transverse rigidity, may be required.
Simplified Method for Dead Load (Beam Analogy)	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss, Arch, Rigid Frame Bridges incorporating Longitudinal Wood Beams Box Girder--Single Cell Box Girder--Multi-Spine	Bridges that carry load due to bending predominantly in one direction and have the following characteristics: * Constant width * Line support * Skew less than 20° * Curvature limitations * Limits on deck overhang
Simplified Methods for Live Load (Design Charts)	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss, Arch, Rigid Frame Bridges incorporating Longitudinal Wood Beams	Bridges with the following characteristics: * Constant width * Line supports * Skew less than 20° * Curvature limitations * Cross section variation limitation * Limits on deck overhang
Simplified Methods for Live Load (Design Charts with Special Requirements)	Box Girder--Multi-Cell Box Grider--Multi-Spine	Bridges with the following characteristics: * Constant width * Line supports * Skew less than 20° * Curvature limitations * Cross section variation limitation * Limits on deck overhang * Limitations on cell size and stiffening
Grillage Analogy	Slab Voided Slab Slab-on-Griders Shear-Connected Beams Floor Systems of Truss, Arch, Rigid Frame Bridges incorporating Longitudinal Wood Beams Box Girder--Multi-Cell Box Girder--Multi-Spine	Not applicable to voided slab and box girder bridges in which the number of cells is less than three.

2.3.2.2 Behavior of Composite Girder Bridges

The extension of the large amount of work done on individual composite beams and girders to full-scale composite bridges is not straightforward. Because of the strength and stiffness of the floor slab, composite bridges tend to redistribute loads in a complex manner. While a finite difference or three-dimensional finite element formulation can be used, a simplified orthotropic plate or grillage approach offers the most cost effective solution to finding forces and deformations in real structures.

Currently two simplified methods are available in North America for the design of bridges: those specified in the AASHTO and the OHBD design codes. Because any rating scheme will require a tier of analytical approaches and many of the bridges to be investigated in this project are quite regular in plan and do not contain large stiffness discontinuities, it is important to determine whether the current distribution factor approach can be extended to the inelastic range.

A brief review of the simplified methods follows, with emphasis only on distribution factors for longitudinal moments. The orthotropic plate techniques described are applicable to both composite and noncomposite bridges, but only the former will be discussed. Similar distribution factors can be derived for transverse moments and longitudinal and transverse shears. Other applicable analytical methods, which have been shown to yield reasonable results but at substantially larger computational effort, are shown in Table 2.5) (*44*).

AASHTO Distribution Factors

The orthotropic plate approach was originally used by Sanders and Elleby (*45,46*) and Heins and Kuo (*47,48*) to verify AASHTO distribution factors. The concept of a distribution factor is discussed in Reference *35*, and the AASHTO specifications compute it as $D = S/K$ where D is the proportion of the total longitudinal moment due to a single line of wheels carried by a beam, S is the spacing in feet and K is a constant. For elastic design of composite bridges, AASHTO has fixed the value of K at 5.5. The orthotropic plate analysis developed by Sanders showed K values ranging from 4.42 for short-span, wide, torsionally soft bridges to 7.52 for medium-span, narrow, torsionally stiff bridges. The finite difference approach developed by Heins and Kuo (*47*) showed elastic values of K ranging from 7.11 for long narrow bridges with widely spaced girders, to 4.98 for wider bridges with closely spaced girders. The value of 5.5 was found to be adequate for most cases but not always conservative. The fixed K method used by the AASHTO method cannot account for differences in the geometry of the bridge nor the difference in torsional stiffness of common composite bridges.

The AASHTO approach can be modified for the inelastic range as suggested by Heins and Kuo (*47*) by computing an inelastic distribution factor, $(DF)_p$, as a function of γ, where γ is the ratio of the number of girders to the number of lanes:

$$(DF)_p = 3.45 + 1.809\gamma + 0.315\gamma^2 \qquad \text{(Eq. 2.5)}$$

For the most common ranges of composite bridges, this results in an inelastic value of K of approximately 7.0. In contrast to the elastic case, the inelastic values of K varied from 9.45, for a medium length (70 ft) two-lane bridge with widely spaced girders, down to 5.90 for a four-lane bridge of the same length but with closely spaced girders.

Ontario Highway Bridge Design Code

To avoid the limitations of the AASHTO code, the Ontario design code has maintained the orthotropic plate method in a more complex form. In this formulation the bridge is replaced by a plate with the longitudinal stiffness of the composite beam (D_x) and the transverse stiffness of the floor slab (D_y). Appropriate coupling stiffnesses between the longitudinal (D_{xy}) and transverse direction (D_{yx}) are included, and recently formulations for the inclusion of the stiffening effect of cross-bracing have been proposed (*49*). This approach (*35,50*) basically divides the bridge according to the torsional stiffness of its members (α) and its geometry (θ).

This approach is based on two main assumptions for a given value of α and θ. The assumptions are as follows and have been verified by extensive parametric studies: (1) The manner in which the longitudinal moments resulting from vehicle loads are transversely distributed in a bridge is independent of the actual values of the span but depends on the relative width of the vehicle with respect to the width of the bridge. (2) The transverse distribution of longitudinal moments due to vehicle loads remains constant throughout the length of the span in which the superstructure is designed mainly for longitudinal moments.

A typical diagram of an α-θ space (*50*) indicates that most bridges of interest for this research fall within a narrow band of $0.06 < \alpha < 0.20$ and $0.5 < \theta < 2.0$. By studying many loading cases and bridge configurations, a value for the distribution factor can be computed. Canadian researchers have presented these values as graphs, with a correction factor C_f.

The typical value for K for composite bridges in the Ontario code varies from 6.25 for one- and two-lane bridges, to 6.75 for three-lane bridges and 7.05 for four-lane bridges. An important difference in the Ontario code is the recognition of the difference in distribution factors between internal and external girders, the effect of edge distances, and effects of edge stiffening. The Ontario code factors, however, are based entirely on linear elastic analysis. As with the case of AASHTO, there is a philosophical inconsistency in using these factors to estimate the ultimate load capacity of an entire bridge as substantial redistribution will occur after first yielding. In general the use of such a factor will lead to a conservative estimate of ultimate capacity.

Much work has been done in Canada on three-dimensional finite element and small scale bridge models to verify and calibrate the Ontario code provisions (*51–53*). Extensive finite element modeling has shown that (*49*):

1. The α-θ method can be used accurately provided there are two (third point) or three (quarter and mid-span) intermediate diaphragms per span.
2. In calculating the values of α and θ, the value of D_y should be calculated as the total rigidity of the deck plus diaphragms divided by the span length.
3. In calculating the value of α, the torsional rigidity of the diaphragms should be ignored.
4. In the case of plate or beam-type diaphragms rigidly connected to the longitudinal girders, the effective flexural rigidity is to be determined by treating the diaphragm as a beam bent about its own longitudinal neutral axis.
5. In the case of cross-bracing, the effective flexural rigidity should be calculated as:

$$BI_d = \{(a_d\, s^3\, h^2) \,/\, (6 l_d^{\,3})\} \qquad \text{(Eq. 2.6)}$$

$$B = a_d\, s^3 \,/\, (3a\, l_d^{\,3}) \qquad \text{(Eq. 2.7)}$$

where a_d is the cross-sectional area of the diagonal members, a is the cross-sectional area of the horizontal members, s is the girder spacing, h is the height of the cross-bracing, and l_d is the length of the diagonal members. A typical value of B is 0.20.

More recently the ultimate load distribution characteristics have been investigated by Cheung *et al.* (*52*) with finite element analysis and experimental work on a ¼-scale bridge model. The results indicate:

1. A finite element model utilizing linear elastic shell elements to model the concrete deck, three-dimensional beam elements to model bracing, and three-dimensional thin-walled elasto-plastic elements for the longitudinal steel girders can model the ultimate behavior adequately.
2. The model bridge tested had a relatively stiff deck, which minimized the effects of the interior diaphragms on redistribution.
3. The distribution factors changed considerably between the linear elastic range and the first plastic hinge. For a conservative

estimate, this reduction is at least equal to the shape factor of the girder section.

4. Load redistribution is insignificant before the formation of the first plastic hinge.

5. Load redistribution after first yielding is extensive, but it is a complicated function including the girder section, geometry of the bridge, relative stiffness of the girder to the deck, internal bracing, load pattern, and even the size of the load step used in the analysis.

6. Further studies are needed to generate simplified approaches to develop inelastic distribution factors. These studies can be performed most efficiently by analytical methods such as the finite element modeling described.

Proposed New AASHTO Distribution Factors

As a result of NCHRP Project 12-26, "Distribution of Wheel Loads on Highway Bridges," new load distribution formulas have been proposed (*44*). The proposed distribution factors are as follows:

$$K = 0.1 + [(S/4)^{0.4} (S/L)^{0.3} (K_g/Lt_s^3)^{0.1}] \text{ (1 lane)} \quad \text{(Eq. 2.8)}$$

$$K = 0.15 + [(S/3)^{0.6} (S/L)^{0.2} (K_g/Lt_s^3)^{0.1}] \text{ (2 or more lanes)} \quad \text{(Eq. 2.9)}$$

where

K = wheel load distribution factor,
S = girder spacing (ft),
L = span length (ft),
$K_g = n(I_g + A_g e^2)$,
t_s = slab thickness (ft),
I_g = transformed gross moment of inertia of the girder only in terms of the equivalent slab material (ft^4),
A_g = transformed area of the girder only in terms of the equivalent slab material (ft^2),
n = modular ratio of the girder material to the slab material, and
e = distance from the neutral axis of the girder to the middle surface of the slab (ft).

These factors are multiplied by 1.05 for positive moment and 1.10 for negative moment if continuous supports are present, and by the following formula for skewed supports:

$$C_{MS} = 1 - \{0.25[(S/L)^{0.5}(K_g/Lt_s^3)^{0.25}(\tan\theta)^{1.5}]\} \quad \text{(Eq. 2.10)}$$

The factors derived by Imbsen and Associates (*44*) come from a statistical analysis [(Multidimensional Space Interpolation (MIS)] of many computer runs using an eccentric orthotropic finite element model. The factors are basically elastic because the loads applied to the models were the usual AASHTO trucks. The results presented above, however, gave very accurate and reliable results.

Use of these factors for rating is considered to be conservative for the usual AASHTO vehicles and Types 3, 3S-2, or 3-3. They are not applicable for special vehicles in which case more sophisticated analysis is recommended.

2.3.2.3 Summary of Composite Girder and Bridge Research

It is possible to accurately predict failure modes, calculate yield and ultimate strengths, and predict deformations and rotations of individual composite beams. Perhaps the main area of further research is the extension of the ductility requirements, derived from tests on building configurations, to typical bridge geometries.

Insofar as the *analysis* of assemblies of girders (or actual bridges) is concerned, there are computer programs available capable of predicting the behavior well into the inelastic range and perhaps to collapse (see Appendix D). The major problem is the time required for input, computation, and interpretation of the results. Most composite bridges rated are quite regular in plan and stiffness distribution; thus such a sophisticated analysis is probably not warranted from the structural standpoint. A simplified method based on the grillage model will be presented in a subsequent part of this report.

2.3.2.4 Behavior of Plate Girders

Plate girders are used extensively in highway bridges because they can be fabricated to span large distances with little or no intermediate support. A plate girder is constructed with large flanges, which effectively resist bending moment and a deep web that serves to carry shear and enables the system to act as a unit. The web is kept very thin in order to reduce the plate girder weight.

The main difference between composite wide-flange beams and composite plate girders is in the behavior of the web, which is much more sensitive to buckling in plate girders due to the large web plate slenderness ratios allowed. Although the ultimate strength of the composite plate girders can be reached and exceeded by using stiffeners and tension field action, the question of available rotational ductility of the plate girders has not yet been thoroughly researched. It should also be pointed out that plate girders, because of the use of stiffeners and bracing, are very sensitive to fatigue problems.

Prior to 1961 the AISC Specification was based on the premise that elastic buckling should be prevented in plate elements (*54*). This is a conservative design philosophy because web buckling results in a redistribution of stresses. The web cannot actually fail by buckling unless the surrounding elements, flanges and stiffeners, also collapse. Tests by Basler and Thurlimann at Lehigh (*55,56*) have led to the present AISC and AASHTO-LFD Specifications (*34,57*), which consider post-buckling strength. The plate behaves similar to a truss: a diagonal tension field is created in the web and the stiffeners act as compression struts (tension field action).

The ASD versions of the AREA and AASHTO Specifications are generally conservative because they maintain the use of classical buckling theory for design and thus neglect post-buckling strength. In these design procedures, limiting values of web slenderness are used to control flexural buckling with and without longitudinal stiffeners and shear buckling with and without transverse stiffeners. The reserve capacity of plate girders is partially taken into consideration by using a lower factor of safety against web buckling compared with those used for the overall strength of the member.

Basler and Thurlimann (*55,56*) were the first to successfully formulate a post-buckling model for plate girders. Since then many other formulations accounting for different boundary conditions, unequal flanges, longitudinal stiffeners, and the effects of shear and moment gradients have been developed (*58–63*).

In comparison with composite beams, few composite plate girders have been tested. Although composite plate girders are commonly used in bridges, no experimental database exists for the current provisions; it is assumed (correctly, by the way) that provisions for noncomposite plate girders will give conservative results for composite ones. Most researchers feel that the rigid concrete slab and the large tension flange of the girder provide equal or better rigidity to the web panel than in noncomposite plate girders. While this may be conservative, it neglects several important behavioral differences between composite and noncomposite plate girders (*64*):

- The usual compactness requirements for webs in plate girders are meant to prevent local buckling due to high compressive stresses. In composite sections, the neutral axis is typically very close to the top of the flange and thus only a small portion of the web us subjected to low compressive stresses. Thus the requirements for web buckling in plate girders need to be revised for composite plate girders.
- The proportioning of a composite plate girder is such that the boundary conditions for the web (rigid slab at the top and large tension flange at the bottom) result in fixed conditions along the two edges of the web. Current design formulas are based on conservative estimates for unstiffened plates and assume simple supports along the boundaries. Thus these provisions are probably very conservative if applied to composite plate girders.
- The contribution of the flexural rigidity of the slab to the post-buckling shear strength is not incorporated into any design formulas.

To address these and other basic questions on composite plate girder design, AISI has funded an experimental project at the University of Texas (*65*). The conclusions of this study are summarized here in some detail:

1. An unstiffened plate with a web slenderness of 156 reached approximately three times the maximum factored shear allowed by current AASHTO specifications. The test also showed that the buckling load was very close to a theoretical prediction assuming fixed edge conditions along the top and bottom of the web.
2. Two stiffened panels with aspect ratios of 1.8 reached approximately five times the maximum factored shear allowed by AASHTO and developed extensive tension field action. Currently a maximum aspect ratio of 1.5 is allowed, and thus these panels would have been considered as unstiffened. The research indicates that 1.5 is not a realistic limit for aspect ratios if fatigue considerations are included in the design.
3. In flexural tests the composite plate girder exceeded the capacity at first yield by 19 percent, and the support rotations at ultimate were 2.6 times the rotations at yield. All plate girders tested under positive moment exceeded the plastic moment capacity of the composite section. The failure under positive loading was triggered by crushing of the concrete at stresses close to $0.9 f'_c$.
4. Only one of the tests was conducted to investigate negative moment capacity. This specimen, with a web slenderness of 121, reached its plastic moment capacity and showed very good ductility.
5. The shear capacity of the unstiffened panels in the negative moment region was also underestimated by current specifications, but the capacity in this region was low compared with those in the positive moment region.
6. Current stress limitations of $0.8\ F_y$ for the negative moment region were found to be overly conservative.
7. Under cyclic loading web behavior of composite plate girders does not differ greatly whether designed for shored or unshored construction.
8. In an unstiffened plate girder with large moment, the yielding of the cross section reduces the shear capacity of the panel, because of the loss of stiffness of the tension flange.
9. The shear capacity of stiffened plates is not adversely affected by yielding of the cross section due to moment.

These tests led Frank (*43*) to propose extensive revisions to the Alternate Load Factor Design Specification (*57*) in order to ensure adequate ductility and strength to bridges designed by the autostress method.

As part of the development of the autostress method, several experimental investigations have been undertaken regarding composite beams and plate girders. Most of the work has been done by Schilling on individual noncomposite transversely stiffened plate girders without longitudinal stiffeners (*66*). All three of the specimens tested were proportioned to obtain a relatively high shear load at their plastic moment capacity. The results indicated that plate girders can undergo very large rotations, and a preliminary lower-bound curve for rotational capacity was proposed.

Of great interest is the $\frac{4}{10}$th-scale model of a continuous two-span composite bridge, which was tested at the Federal Highway Administration's Turner-Fairbank Highway Research Center (*67*). The plate girders had a b/t ratio of 11.25 and an h/t ratio of 108. The results of the tests have been analyzed and comparisons to the proposed method of bridge rating will be presented later in this report. The bridge behaved very much as predicted by the autostress design procedure. During overload tests, the structure developed only minor yielding of the bottom flange near the pier, extending longitudinally slightly more than expected but with very minor residual deformations. Only slight cracking over the supports was evident, and full composite action over the negative moment region was achieved. The deck was prestressed biaxially, and therefore cracking was inhibited. The results have verified the autostress design philosophy.

When the large amount of work done on composite beams is compared with that carried out on composite plate girders, it would seem that much experimental and analytical work remains to be done for composite plate girders. The work by Schilling, Moore, and Frank, however, indicates that good models have been developed and that our understanding of the behavior of composite plate girders has improved substantially over the past 5 years.

2.3.3 Analytical Approaches to Bridge Rating

Although simplified methods of analysis, such as the load distribution factor procedure in the AASHTO specifications or improvements on this technique, can be used to predict the ultimate capacity of a bridge, they are likely to give (a) conservative or lower bound solutions and (b) very little information on the amount of damage caused to the structure as the ultimate load is approached. The basic limitations of most of these procedures stems from the fact that the structures are treated as one- or two-dimensional continua with elastic perfectly plastic material properties. Thus the influence of secondary members, buckling, fatigue, strain-hardening and strain-softening of the materials cannot be properly modeled with such techniques. For design of new bridges, of course, these limitations may not be very important. For rating and evaluation, on the other hand, they are very important.

With the advent of digital computers, more sophisticated models based primarily on the grillage analogy or on orthotropic plate theory have become quite popular (*35*). These procedures, while still using elastic or at best elasto-plastic analysis, give more accurate solutions for the ultimate strength of a bridge by taking into account some of the three-dimensional aspects of bridge behavior. However, while they can detect the amount of yielding in the steel and compute deflections, they still cannot be easily adapted to predict the damage (particularly to the concrete slab) or permanent set due to overloads.

Because of these shortcomings in predicting structural damage or residual deflections, methods that utilize elastic analysis cannot be considered very promising rating tools if the inelastic capacity of the bridge and three-dimensional action is to be used. On the other hand, methods utilizing elasto-plastic idealizations are very promising, if some limit states for damage can be formulated. These methods (plastic analysis and shakedown) are simple to use and give good estimates of overload and ultimate strength.

Over the past 20 years, several other approaches have been suggested, primarily utilizing finite element methods (FEM). The primary advantages of these approaches are the ability to model the three-dimensional behavior of the bridge and the nonlinearities of the materials. However, most FEM programs developed to investigate bridge behavior were cumbersome to use and required large computing capacity. This effectively put such programs out of everyday use by bridge rating engineers. A very thorough review of these techniques and available computer programs is given by Nutt *et al.* (*44*) in *NCHRP Research Results Digest 187* and will not be repeated here.

As part of this NCHRP project it was decided that one such program should be implemented, either as the basis for a sophisticated rating program or as a check on the proposed simplified procedure. After looking at many of the programs available for bridge design and analysis, BOVAS was selected for use in this project. BOVAS was written by C. Kostem *et al.* at Lehigh University (*68–71*), under a contract to PennDOT and FHWA, to analyze simple or continuous steel multigirder bridge superstructures with a reinforced concrete deck slab and steel girders.

BOVAS was selected because it was written specifically for the type of bridge under investigation, and it has proven successful at predicting not only the overall bridge behavior but also the damage to the structure. The version of BOVAS obtained was key to the AASHTO specifications current at the time and has the following important limitations:

1. Girder spacing must be constant for a given bridge.
2. The diaphragm and cross-bracing do not contribute to the structural stiffness of the superstructure.
3. Many important variables have been internally defined.
4. Torsion and minor axis bending are not included.
5. Shear punching failure of the slab is not modeled.
6. Impact is not modeled except as the usual AASHTO impact factor.

The main advantages of BOVAS are:

1. The bending and axial forces in the slab are modeled.
2. Beams of constant or varying cross section may be used.
3. Material nonlinearities are incorporated in the analysis.
4. The amount of composite action can be varied.
5. Local buckling is included in the analysis.
6. Fatigue checks can be carried out.
7. It has been extensively checked for the types of structures of interest in this study.

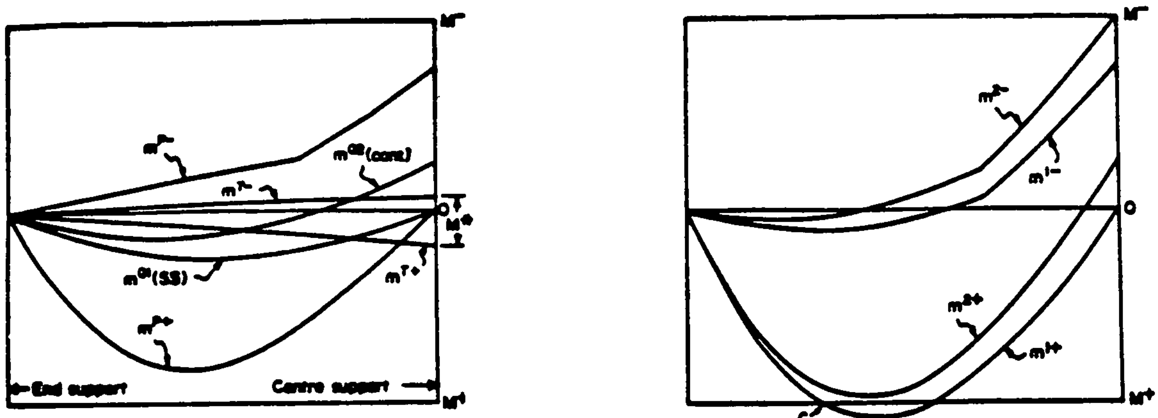
2.3.4 Plastic Design, Shakedown and Autostress Design

Plastic design of structures, particularly continuous beams and low-rise frames, has long been accepted by design codes since its introduction in England by Baker (see Reference *72*). It is currently recognized in the U.S. in the AISC Specifications (*34*) and the AASHTO Alternate Load Factor design procedure (*57,73,74*). Application of plastic design and shakedown theory to the design of bridge-type structures has been proposed by Eyre and Galambos (*75,76*), Grundy (*77,78*), and Gurley (*79,80*). A very complete and detailed procedure for bridges, the Autostress Design Method, was proposed by Haaijer *et al.* (*81–85*).

This section highlights the important assumptions in shakedown theory and autostress design as they relate to rating of composite bridges. Some familiarity with plastic analysis and design principles is assumed and these basics will not be repeated here (*86–88*). The two key concepts required from plastic analysis are the kinematic theorem (or formation of mechanisms) and the rotational ductility needed to form those mechanisms.

2.3.4.1 Shakedown of Structures

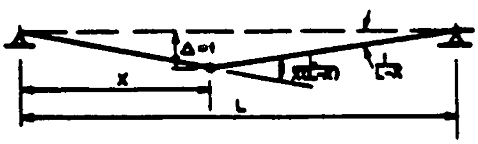
The concept of shakedown, or adaptation, provides a powerful analytical tool in inelastic analysis and plastic design (*89–93*). Shakedown is deemed to have occurred in a structure after some finite amount of plastic work has taken place and the structure responds in a purely elastic manner from a residual



Elastic Moments due to P = 4M/L, Q = 2M/L
M^T (temperature) = M/4

Elastic Moment Envelopes due to
P(= 4M/L) + Q(= 2M/L)

Figure 2.4. Moment envelopes for shakedown calculations (77).



Collapse Mechanism

Figure 2.5. Plastic collapse mechanism for one span of a two-span bridge (77).

stress distribution corresponding to a locked-in permanent deformation (*89*). In simpler terms, it means that after some initial deformations in the plastic range due to overloading, the structure will adapt or shakedown, and respond in the elastic range to working loads.

It is reasonable to assume that any bridge that has been in service for some years has seen several large legal or illegal overloads, and thus has achieved this state. Shakedown is important in plastic design because it allows the designer to check for repeated and nonproportional loading, which the usual mechanism approach cannot handle. The concept of shakedown is also useful in estimating both the actual safety factors and bounds to maximum deformations in overloaded structures (*83–85*).

To ensure safety in plastic design, it is important to establish whether shakedown will occur for a particular type of structure. It has been shown that for the shakedown criteria to be significant some conditions not commonly found in rigid frames must exist. The most important of these conditions are a large margin between the elastic limit and the collapse load, large load reversals, and many repetitions of the loading (*77*). Among the types of structures that do fulfill these conditions are bridge deck systems, offshore structures, and structures supporting cranes.

If shakedown does not occur, three possibilities arise. First, the plastic deformations will continue to increase to infinity leading to the so-called incremental collapse (IC). Second, the structure may undergo yielding in both directions, exhaust the local plastic work capacity, and result in alternate plasticity (AP) and failure by low-cycle fatigue. The third possibility is a combination of the first two.

Calculation of the shakedown load, for rating purposes, does not represent additional computational effort on the part of the engineer. The basic steps necessary to obtain the shakedown load can be summarized as follows (*78*):

1. Obtain the elastic moment envelope for the structure given the loading vehicle and geometry of the bridge. Shown in Figure 2.4 is half of the moment envelope for a two-span bridge. Three load effects are shown for two different construction sequences. The superscripts *P, Q,* and *T* refer to the effects of live load, dead load, and changes in temperature on the moment (m). The superscripts 1 and 2 on the dead load moments refer to whether the bridge was assumed as simply supported (1) or continuous (2) for dead load. The effects of changes in temperature could lead to either positive or negative moments, and can be visualized as a change in the reactions, and thus a linear variation of moment with length.

2. A kinematically admissible mechanism is assumed. This is the equivalent of drawing the plastic collapse mechanism shown in Figure. 2.5.

3. The load factor against incremental collapse, or the shakedown load factor (τ_{IC}), can be calculated from the following equation:

$$\tau_{IC}\left[\textstyle\sum_i (m^{Q+} + m^{T+})\,\theta_i^+ + \sum_j (m^{Q-} + m^{P-} + m^{T-})\,\theta_j^-\right] = \left[\textstyle\sum_i M^+\theta_i^+ + \sum_j M^-\theta_j^-\right] \qquad \text{(Eq.2.11)}$$

where, M is the ultimate moment capacity; i refers to the positive moment hinges; j refers to the negative moment hinges; θ refers to the rotation at the hinge. The right-hand side of the equation is the usual expression for plastic dissipation of energy, while the left-hand side represents the work done during incremental collapse.

It is clear that shakedown represents several different limit states in the design of structures. Shakedown is a limit state with respect to deflections. The fact that shakedown will occur is not enough to insure the adequacy of the design, because the

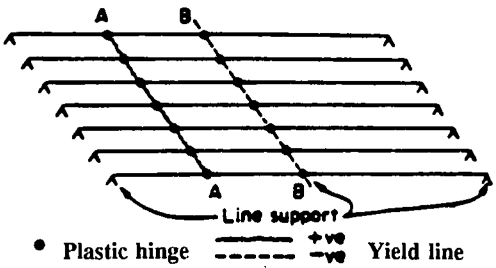


Figure 2.6. Overall collapse mechanism for a grillage (77).

deformations associated with shakedown might be excessive. Moreover, shakedown must be associated with cumulative and irreversible damage to the structure. Therefore, it is also required that a careful check for the possibility of local failure due to alternating plasticity and low-cycle fatigue be carried out. Procedures to check this limit, labeled "incremental collapse toughness," have been proposed (*93*).

As an ultimate limit state, shakedown will provide lower bounds to the collapse mechanism strength. Because the reserve capacity of multigirder composite bridges is very large, it is unlikely that this limit will ever be achieved. The most careful calculations run on existing structures about to be tested or overloaded indicate that it is impossible to find vehicles capable of causing a collapse of such structures (*94*).

The tests on beams and grids (*75,76*) demonstrated that shakedown could be achieved at loads greater than the simple plastic load even when the beams were susceptible to lateral torsional buckling. For individual beams, loads equal to or less than the simple plastic load will cause only minor structural damage as reflected by the residual deflections being no larger than the deflections which occur at the limit of elastic action. In grids, an analysis procedure ignoring torsion and strain-hardening indicated that the shakedown load falls between the load necessary to cause first yield and the fully plastic load required to cause a mechanism.

The detailed work by Grundy (*78*) has shown that incorporating the effect of the deck will result in a significant contribution to the strength of the structure, and relatively simple plastic analysis techniques can be used to investigate this effect. For the analysis of composite bridges, Grundy has shown that the mechanisms associated with shakedown are global mechanisms, such as those shown in Figure 2.6, rather than the partial or local collapse mechanisms usually associated with orthotropic grids. Thus the calculation of a shakedown load may be simpler than that for the static collapse load.

The concept of shakedown will be utilized in the inelastic rating method to be proposed herein.

2.3.4.2 The Autostress Method

The current AASHTO design philosophy for highway bridges allows up to a 10 percent redistribution of moments at intermediate supports if the girders comply with certain compactness criteria. The Autostress Design method provides a more realistic approach to the utilization of this reserve capacity of the structure by assuming that some yielding will occur in the structure and by incorporating the forces generated in the process (the automoments) into the design procedure. In addition, Autostress Design provides initial estimates of permanent deformation due to overloads, information that could be used to compute initial camber of the beams.

"Autostress" design is an extension of the "Load Factor Design" (LFD) method of the AASHTO Standard Specifications for Highway Bridges. The AASHTO LFD method provides criteria for resistance against the following three levels of loading: service load, overload, and maximum load. The difference between these levels is in the magnitude of the load factors: 1.00, 1.66, and 2.17, respectively, for service load, overload and maximum load. The underlying limit state philosophy of the LFD method is (*95*): (1) No damage is expected during the intended lifetime of the structure under cyclic stress caused by service loadings; (2) "Overloads are the line loads that can be allowed on a structure on infrequent occasions without causing permanent damage"—elastic behavior under an overload vehicle is stipulated; and (3) Maximum loads will cause damage to a critical cross section of the bridge.

In 1986 AASHTO issued the *Guide Specifications for Alternate Load Factor Design Procedure for Steel Beam Bridges Using Braced Compact Sections* (*57*). These new design criteria contain the "Autostress" method. This alternate design procedure differs from the LFD method in the following respects: (1) The ultimate limit state under the maximum load condition is the formation of a plastic mechanism. (2) Under negative bending a reduced plastic moment is defined for sections where flanges and webs do not meet the compactness criteria. (3) In order to extend the elastic range under overload, the bridge is permitted to develop "automoments" during overload.

The Autostress design method is the result of more than 10 years of research conducted at the U.S. Steel Technical Center in Monroeville, PA (*66,74,85*), with additional field testing at the University of Washington in Seattle (*96*), research at the Federal Highway Administration's Turner-Fairbank Highway Research Center, McLean, VA (*67*), and research by AISC Marketing, Inc.

The research leading up to the publication of the AASHTO Guide Specification was summarized by Grubb (*74*). This research comprised experimental studies on noncomposite and composite members with noncompact webs or flanges to establish the actual moment capacity of the composite section at the support. The research addresses essentially two problems:

1. Can a span in a continuous bridge fail as a plastic mechanism? As long as the critical sections are braced and compact, and the loading is static, there is no doubt that the answer to the question is affirmative. The applicability of plastic analysis, behavior, and design has been well documented (*97*), and plastic design is an established design method for building structures. The most significant results of the U.S. Steel research was the demonstration that sufficient hinge rotation can be developed, even for noncompact flanges or webs, if a reduced plastic moment rather than the full value is used. Simple formulas were presented for the reduced plastic moments. A study of two-span girders using (a) elasto-plastic moment-curvature (M-θ) relations with the reduced plastic moments and (b) more realistic elastic-softening M-θ curve showed that the simplified method proposed in the Autostress design is always conservative (*98*). Additional proof of the plastic design approach to

resisting the maximum load level has been provided by the test of the bridge at the FHWA Turner-Fairbank Highway Research Center.

2. The second emphasis of the Autostress research concerns behavior under overload. The argument is made that a continuous bridge will shakedown after several cycles of load excursions into the inelastic range, and thereafter loads that are less than or equal to the shakedown limit load will be accommodated elastically. The theory of shakedown is especially applicable to continuous bridges. Shakedown will occur naturally when a bridge is loaded into the inelastic range when the load is below the shakedown limit load. A residual moment field is set up in the bridge, thus enlarging the available elastic moment range.

The proposed rating method is an extension of the Autostress design procedure.

2.3.5 Probabilistic Methods

2.3.5.1 Introduction

Modern bridge design is based on the principles of Limit States Design (LSD). This design method uses the following design checking format for the prevalent load case of dead load plus live load:

$$\phi R_n > \tau_D D + \tau_L L_o \qquad \text{(Eq. 2.12)}$$

the right side of this equation defines the factored loading, while the left side is the reduced resistance of the element under consideration. The individual terms of Equation 2.12 are described as follows:

- R_n is the "nominal resistance" of the structural element (beam, girder, bolt, or weld), and it is determined from the relevant equations in a standard bridge specification for nominal material properties (e.g., the minimum specified yield stress).
- ϕ is the "resistance factor," less than or equal to unity, and it accounts for uncertainties inherent in the determination of nominal resistance: (a) variability of the material properties, (b) variability of the cross-sectional dimensions, and (c) inaccuracies introduced by modeling idealizations of structural behavior.
- D is the "dead load effect," i.e., the moment, shear, axial force, or stress in the element under consideration because of the dead load.
- L_o is the "live load effect," including the impact factor.
- τ_d is the "dead load factor," which accounts for the uncertainties of the dead load as well as for the idealization inherent in translating load to load effect.
- τ_L is the "live load factor," which accounts for the uncertainties in live load and the load effect modeling.

The factors ϕ, τ_d and τ_L reflect our belief on how uncertain future events will affect the structure.

The determination of the load factors and resistance factors, and also of the factors of safety used in the Allowable Stress Design method, involves the following three ingredients: (1) Past successful or unsuccessful experience, (2) Judgment, and (3) Implicit or explicit use of probability.

The exercise of bridge rating is performed to evaluate the future load capacity of an existing structure. Many of the uncertainties of design no longer apply to an existing bridge, but there are still many random factors that need to be considered. Bridge rating, as design, is best based on the principles of Limit States Design, with the load factors and resistance factors determined by probabilistic methods.

Three bridge rating methods have been examined in previous parts of this report:

- The AASHTO Rating Manual
- The Ontario Bridge Code
- The British Bridge Assessment Standard

Each of these documents uses the format of Equation 2.12 for the determination of the rating load, and each specifies the same factors as are applicable in design. In view of the fact that there are significant differences between designing a new bridge for a 50-year life-span, and rating an existing bridge for a 2-year period, it is desirable that any probability-based assessment procedure account for these differences.

NCHRP Reports 292 and 301 (*3,99*) presented recommendations that include an explicit treatment of the reliability of existing bridges. This report will concentrate on those two references, which deal directly with the probability-based bridge assessment, because work under NCHRP Project 12-28(12) was a direct extension of the foregoing research.

2.3.5.2 Review of Probabilistic Bridge Rating Studies

Of the two previous NCHRP-sponsored studies on the probability-based load capacity evaluation of existing bridges, one (*99*) concerns reinforced concrete bridges, and the other (*3*) deals with steel and prestressed concrete bridges. The latter reference is most relevant to this study, but both will be presented for the purpose of comparison.

The proposed evaluation format in both of these studies is identical to Equation 2.12, from which the following rating factor can be derived (*3*):

$$RF = \frac{\phi R_n - \gamma_D D}{\gamma_L L\ (1 + I)} \qquad \text{(Eq. 2.13)}$$

In this equation $L_o = L\ (1 + I)$ is introduced to explicitly identify the impact factor I. When RF is equal or greater than 1, the bridge is satisfactory; otherwise some kind of remedial action is necessary.

The following is a description of the terms in Equation 2.13:

- R_n, the nominal resistance, is to be taken as the appropriate limit state in the applicable part of the AASHTO Design Specification, but modified as necessary, for possible corrosion or deterioration. Only the ultimate limit states are to be considered in the evaluation.
- ϕ, the resistance factor (called the "capacity reduction factor" in Reference *99*) is dependent on: (1) whether the

geometric data have been determined by field measurement or obtained from the plans; (2) inspection frequency; (3) degree of deterioration; and (4) degree of preventative maintenance. The resistance factor may vary from 0.94 down to 0.54 for concrete bridges (*99*), and from 0.95 to 0.55 for steel and prestressed concrete bridges. No penalty for lack of redundancy is prescribed in Reference *99* for concrete bridges; Reference *3* assesses a 0.2 reduction of ϕ for a nonredundant bridge member.

- τ_D, the dead load factor, is equal to 1.2 for steel and prestressed bridges, while it is variable for concrete bridges (1.2 for concrete, 1.05 for factory produced components, and 1.4 for the wearing surface).
- τ_L, the live load factor, depends on the traffic volume (light, moderate, or heavy (*99*), and low or high volume (*3*)), the presence of overload enforcement (enforced or unenforced), and the method of lateral load distribution analysis. This latter effect is considerably different for the three types of bridges (concrete, steel, prestressed concrete) because of the distinct lateral load distribution mechanisms. The live load factors can vary from 1.44 to 2.12 for concrete bridges, from 1.35 to 1.85 for steel bridges, and from 1.35 to 1.85 for prestressed bridges.

The two evaluation criteria (*3,99*) are very similar in their origins (i.e., same probabilistic method). The criteria provide great benefits for bridges that are (a) carefully monitored for corrosion and deterioration, (b) inspected regularly and maintained vigorously, and (c) on highways where weight limits are enforced rigorously. On the other hand, there are heavy penalties for bridges that are neglected by both the bridge department and the state police.

The impact factor, *I*, is dependent on the smoothness of the roadway surface on the bridge. It may vary from 0.1 for a surface in good to fair condition, to 0.2 (poor condition) and 0.3 (critical condition).

Judgmental factors which cannot be easily quantified, such as decisions between light or medium truck traffic, whether adequate weight control exists, the degree of inspection and maintenance, and the amount of deterioration, can have major effects on the magnitude of the rating factor, and thus on the consequence of not achieving a factor above unity.

The basis for determining the load and resistance factors is the first-order second-moment (FOSM) probabilistic method. This theory uses relatively simple probabilistic manipulations (thus "first-order") of the means and the standard deviations (the standard deviation is related to the second moment of the area under the probability density function—thus "second-moment") of the applicable random variables (e.g., loads, load effects, resistances) to arrive at an estimate of the probability of exceeding a limit state criterion. By itself, this probability means nothing, but as a measure of comparison between two structural elements it can say whether one element is more, or less, reliable than the other one. If one of these elements has been designed by a true and tested method, while the other one has been proportioned by a new and untried criterion, then a statement can be made about the relative reliability of the new versus the old method of design or analysis. This comparison and the resulting adjustment is called "calibration."

The purposes of "calibration," also appropriately named "code optimization," are (a) the determination of the characteristic reliability index for a structural member according to the current evaluation criterion (e.g., AASHTO Rating Manual) and (b) the development of load factors and resistance factors in the new criterion (e.g, rating method recommended in References *3* and *99*), which will give the same reliability index over the whole range of relevant variables.

The reliability index, β, is a convenient replacement for the concept of a probability of exceeding a limit state. If we know the precise probability distribution, then it is possible to transform from one to the other. However, it is psychologically more helpful to work with a "reliability index" than with a "probability of exceeding a limit state."

The reliability index of any structural element may be determined by mathematical manipulations from a knowledge of the two statistical properties, for example, the mean value and the standard deviation, of all the random parameters affecting the evaluation. In its simplest form, assuming we have only two quantities that define the safety of the structure, namely the resistance *R* and the load effect *Q*, the structure is "safe" if:

$$R - Q > 0 \qquad \text{(Eq. 2.14)}$$

The measure of the degree of reliability is the reliability index (also often called the "central safety factor"):

$$\beta = \frac{\overline{R}}{\overline{Q}} = \frac{\overline{R} - \overline{Q}}{(\sigma_R^{\,2} + \sigma_Q^{\,2})^{1/2}} \qquad \text{(Eq. 2.15)}$$

where $\overline{R}$ and $\overline{Q}$ are the mean values of *R* and *Q*, respectively. The right-hand side of Equation 2.15 indicates that the standard deviations of *R* and *Q*, namely σ_R and σ_Q, respectively, also influence reliability.

The actual problem is, of course, far more complex because the resistance and the load effect each consist of many other variables. Methods of dealing with such complications are available and described in many references, including References *3* and *99*.

Knowing the statistical properties of the resistance and the load effects, it is possible to determine β for any member in a bridge, as illustrated below:

$$RF = \frac{\phi R_n - \tau_D D_n}{\tau_L L_n} \qquad \text{(Eq. 2.16)}$$

This member is just at the limit of its structural usefulness when $RF = 1.0$. We can then solve for:

$$R_n = \frac{\tau_L L_n + \tau_D D_n}{\phi} \qquad \text{(Eq. 2.17)}$$

where, R_n is the nominal resistance which will be just adequate to resist the factored nominal load effects, D_n and L_n. The equation for the reliability index β for this case is:

$$\beta = \frac{(\overline{R}/R_n)\, R_n - \overline{D} - \overline{L}}{(\sigma_R^{\,2} + \sigma_D^{\,2} + \sigma_L^{\,2})^{1/2}} \qquad \text{(Eq. 2.18)}$$

From the knowledge of the statistics $\overline{R}/R_n$, $\overline{D}$, $\overline{L}$, σ_R, σ_L, and σ_D, we can find β for various combinations of ϕ, τ_D and τ_L. If we know what β should be, we can obtain the corresponding values of ϕ, τ_D and τ_L by the methods described in References *3* and *99*. The key issue is thus what the "target reliability index" β_T should be.

The argument is made in Reference *3*, for steel and prestressed concrete bridges, that the target reliability index should be calibrated to the AASHTO Rating Manual procedure for allowable stress design (ASD) for the operating stress level when the bridge is frequently inspected and loads are reasonably controlled. Selection of the ASD procedure and the operating stress level will give the nominal resistance for the bridge member, and the stipulation of frequent inspection and reasonable load control defines the live load statistics.

This calibration indicated that the target reliability index for this case is $\beta_T = 2.5$ for steel and prestressed concrete bridges. The target reliability of $\beta_T = 2.8$ was found to be applicable for concrete bridge members (*99*). The resistance factors and load factors recommended in References *3* and *99* were developed to meet these target values.

The issue of nonredundant members is not considered in Reference *99* (concrete bridges). The issue is addressed, however, for steel bridges in Reference *3*, where a target reliability is determined for the allowable stress rating method for the inventory load level when the truck traffic is heavy and unenforced. The argument is made that this is the worst possible loading case and since it has historically resulted in satisfactory performance, it should be used as a calibration point. The resulting target reliability index is $\beta_T = 3.5$. Rather than defining new load factors for nonredundant members, the adjustment is made in the resistance factor ϕ for the higher required reliability index, thus reducing $\phi = 0.95$ for uncorroded redundant members to $\phi = 0.80$ for nonredundant members.

Reference 3 provides a comprehensive and rational live load model that can be used for further probabilistic studies. (It has also been used for concrete bridges in Reference *99*.) This model is expressed by the following equation:

$$M = aW_{.95}\ m\ H\ (1 + I)\ g \qquad \text{(Eq. 2.19)}$$

where

- M = predicted maximum live load effect
- a = deterministic constant relating load effect to reference vehicle load
- $W_{.95}$ = characteristic truck weight at a site
- m = axle spacing and truck type effect
- H = multiple presence factor variable ("headway" influence)
- I = impact allowance variable
- g = structural analysis variable.

The reference also develops the statistical properties for these variables. Mean values and standard deviations are given for the variables W_{95}, H, m, I, and g. This model, used in this research, is a very useful tool for further probabilistic studies with new types of limit states.

A proven probabilistic methodology was used to develop load models, resistance models, statistical data, and finally load factors and resistance factors, which meet reasonable and rational target reliabilities. Thus a framework has been established on which further developments can be based.

Table 2.6. Details of the AASHO steel bridges (*36*)

Bridge No.	Type	Nominal Beam Size	Length of Top CP	Length of Bottom CP
1A	Noncomp.	18WF55	0	20ft. 6in.
1B	Noncomp.	18WF50	0	0
2A	Noncomp.	18WF55	0	0
2B	Comp.	18WF50	0	14ft. 0in.
3A	Noncomp.	21WF62	0	0
3B	Comp.	18WF60	0	18ft. 6in.
4A-B	Noncomp.	18WF60	0	19ft. 0in.
9A-B	Noncomp.	18WF96	17ft. 0in.	17ft. 0in.

CP - Cover plate

2.3.6 Bridge Tests to Ultimate

2.3.6.1 Introduction

Bridge tests have been conducted to investigate load distribution among the various members of the bridge, dynamic (fatigue) response, and load capacity (*100–103*). The results of these tests (e.g., load distribution, degree of composite action observed, etc.) provided insight for the development of the inelastic bridge rating methods. The results (e.g., onset of damage, ultimate loads, etc.) were also used as a benchmark to calibrate the results of the inelastic bridge rating procedures. Because of these reasons, a detailed description of some of the test results is included.

References *100* and *101* contain extensive lists of bridge tests that have been performed. This report will concentrate on the results of three of those investigations as they pertain to inelastic bridge rating methods. These are the AASHO (*36*), University of Tennessee (*104*), and Baldwin tests (*105*).

2.3.6.2 Description of Tests

AASHO Bridge Tests (*36*). The AASHO tests were conducted on 18 simply-supported slab-beam bridges subjected to test traffic. Ten of the bridges comprised rolled steel sections and reinforced concrete deck slabs. Eight were completely noncomposite (no bond between the girders and the slab); two were composite (connected to the slab with channel connectors). Three of the noncomposite (1A, 4A and B) and both composite bridges (2B, 3B) had partial length cover plates welded on the bottom flange only. Two of the noncomposite bridges (9A and B) had partial length cover plates welded on both the top and bottom flanges and three had no coverplates (1B, 2A, 3A). See Table 2.6 for more details.

Initial Reference Tests. The bridges were initially subjected to a reference test consisting of numerous cycles of trucks carrying constant load. The traffic caused permanent sets in all of the bridges (Table 2.7). The largest permanent sets occurred in five of the noncomposite bridges (1B, 2A, 3A, 4A and B); four of the bridges (1B, 2A, 4A and B) were deemed failed when the mid-span deflection exceeded 3 in. during the reference tests. (Two of the bridges did not contain cover plates, and two contained coverplates on the bottom flange only.) In all

Table 2.7. Deflections during initial AASHO reference tests (*36*)

Bridge No.	LL Deflection[1] Interior	Center	Exterior	Permanent Set Interior	Center	Exterior
1A	1.89	1.71	1.59	0.42	0.45	0.45
1B	1.85	2.01	1.92	1.81	1.85	1.89
2A	2.49	2.49	2.21	3.19[2]	3.15	3.19
2B	1.08	1.03	1.08	0.29	0.29	0.29
3A	1.61	1.65	1.72	1.19	1.16	1.19
3B	0.70	0.79	0.76	0.10	0.06	0.10
4A	2.48	2.37	....	3.50	3.36	3.26
4B	2.56	2.68	2.77	3.94	3.71	3.54
9A	1.60	1.69	1.80	0.31	0.32	0.32
9B	1.51	1.44	1.42	0.25	0.29	0.29

[1] All deflections given in inches. Live load deflection was measured as the vehicles were traveling 30 mph.

[2] Failure usually corresponded to a midspan deflection of 3 in. (L/200).

other bridges the permanent set was less than 0.5 in.; the composite bridges (2B, 3B) generally had the least permanent set.

At the end of the initial reference tests, yielding (slip lines) had developed in all of the bridges. Yielding was observed in the tension flange of the composite girders, in the compression flanges of 9A and B, and in both top and bottom flanges of the other noncomposite girders. The reason for the yielding in the compression flanges of 9A and 9B was thought to be a result of high residual stresses measured in the compression flange of these girders. The yield point was exceeded in all three beams of tests 2A, 4A, and 4B, which had permanent deflections exceeding 3 in. Two of the girders exceeded the yield point in 1B. The actual stress in the girders of these three bridges exceeded the design stress on the order of 10 percent because of the lack of composite action in these bridges (10 percent was a figure assumed in design).

Details of Damage to Beams Deemed Failed—Yield lines visible on the bottom flange of Bridge 2A spread 14 ft from the midspan toward the supports. Bridges 4A and B suffered severe yielding in the tension flange to a distance within 5 ft of each end of the cover plates toward the supports. Yield lines were also observed in the compression flange and a few were even evident in the cover plates of Bridge 4A. Test 1B is of special note; at the end of the initial reference tests, it maintained an average permanent midspan deflection of 1.85 in. After 235 trips of the regular test traffic, it was subjected to an overload, which caused a 40 percent increase in live load moment. This overload resulted in a permanent set exceeding 3 in., which was the limit criterion for the bridges. Yielding was observed in the bottom flanges of the girders extending 14 ft from the midspan toward both supports.

Tests with Regular Traffic. Bridges 1A, 2B, 3A and B, and 9A and B, were subjected to approximately 400,000 to 600,000 trips of regular test vehicles. The midspan transient live load deflections measured at the beginning and end of the tests, and the dead load permanent set deflections are listed in Table 2.8. The worst deflection occurred in Bridge 3A, which suffered damage from a major vehicle accident. All other bridges suffered permanent deflections less than *L*/500. The deflection of 3B (composite bridge) was less than *L*/1000.

The regular test traffic caused yielding in all steel beams and fatigue cracking in several beams with partial length cover plates. Yielding was observed immediately after the first few trips of the regular test traffic and most of it took place early in the tests. The fatigue cracks were observed shortly before the end of the test traffic.

Yielding developed within the bottom flange of Bridges 1A, 2B, and 3A and B. The yield lines reached within 8, 9, 7 and 10 ft of the supports of the aforementioned bridges, respectively. The most extensive yielding occurred at the critical sections. Because of the difference in residual stresses, Bridges 9A and B did not experience any tensile yielding, only light compression yielding in the top flanges. Light yielding was also found on the compression flange of the other noncomposite bridges (1A, 3A). No yielding was observed in the cover plates.

Fatigue cracks were observed in Bridges 1A, 2B, 3B, and 9A and B. At the conclusion of the test, 10 cracks were observed in 1A (cracks developed at 10 of the 12 ends of the coverplate welds). Bridge 2B had five cracks; the most extensive crack (which completely penetrated the flange) occurred in this bridge. Bridge 3B had two cracks, one of which was the second largest observed. Bridge 9A had two cracks; Bridge 9B had one crack.

Two of the bridges (1A and 3A) were damaged by accidents during the tests. An accident on 1A caused a 1-in. increase in permanent set. An accident on 3A caused permanent sets in the three girders of 0.42, 0.78 and 1.17 in., respectively. The total permanent set of 3A, following the accident, exceeded the limiting condition of 3 in., thereby taking the bridge out of service.

Table 2.8. Deflections during AASHO tests with regular vehicles (*36*)

Bridge No.	Permanent Set[1] Interior	Center	Exterior
1A	1.73	2.05	2.39
2B	0.80	0.68	0.52
3A	3.94	4.22	4.53
3B	0.50	0.29	0.19
9A	0.70	0.65	0.60
9B	0.68	0.70	0.50

[1] Failure usually corresponded to a midspan deflection of 3 in. (L/200).

Following the tests of the regular traffic, Bridge 2B was subjected to accelerated fatigue tests during which it failed.

Overload During Regular Traffic—At least three of the bridges experienced overloads during the regular test traffic. Bridge 1A was loaded 8 times by heavier than regular vehicles, which produced a 40 percent increase in midspan moment caused by the regular vehicles. The permanent set resulting from the overload and 117 regular trips caused a 0.29 in. permanent set. Five more trips of the heavier vehicle at a later date did not produce any increase in permanent deformations. This information is very useful with respect to our investigation of automoments. Bridges 3A and B also experienced overloads. The overload of Bridge 3A produced an increase of 25 percent in the live load moment. Neither of these overloads created any detectable permanent deformations.

Tests to Ultimate. Four of the bridges (three noncomposite, 1A, 9A and B; one composite, 3B) were tested to failure with increasing loads. These tests were conducted to investigate the failure mechanisms and to determine the ultimate capacity of the bridges. The bridges were each loaded with different incremental loadings (30 passes were made by the vehicle at each incremental load level) using different test vehicles. Once permanent deformations were observed to occur in the incremental loading tests, they continued to develop in nearly every subsequent vehicle trip. For constant speed, the first few trips tended to produce greater permanent deformations than later trips at the same load (except at maximum loads). The results of the tests to failure are shown in Figures 2.7 to 2.10 for each of the bridges.

Noncomposite Bridges Tested to Ultimate—Bridge 1A had only bottom flange cover plates. During the test to failure, plastic hinges developed at both ends of the cover plates and a large amount of yielding occurred in the top flange in the middle 30 ft. Bridges 9A and B had top and bottom flange cover plates. Plastic hinges developed in the approach end of the cover plates in 9A and B. Tensile yielding was observed at high load levels and was generally confined to the cover plates.

Fatigue cracks, which had been noted in the bridges after the reference tests, became wider but did not increase in length in the incremental tests to failure. Fatigue cracks at the toes of the cover plate welds developed during the incremental tests to failure at all such locations.

At the end of the tests, all three noncomposite bridges had permanent sets exceeding 12 in., a large portion of which (2 in.) occurred at the maximum load.

Composite Bridge Tested to Ultimate—The result of the composite bridge tested to failure (3B) was compared with a finite element analysis using the program BOVAS described in Section 2.3.3 (Table 2.9, Fig. 2.11). At the end of the reference tests, this bridge had a small permanent set (0.19–0.5 in.) and two fatigue cracks, 1.5-in. long. The deformation measurements toward the end of the test traffic did not exhibit any effect of the fatigue cracking on the stiffness.

During the seventh incremental loading, the cracks were observed to increase in length (1.5 to 2 in.) as well as in width. The cracks were repaired with a butt weld after this increment. The first appreciable permanent set was observed to occur in the third load increment. During the fifth, yield lines appeared in the bottom plate; first near the approach end and later near the exit end of the cover plates. During the sixth increment, yield lines developed all along the bottom plate (except near the ends of the spans and on the bottom cover plate). In the seventh, the lines spread everywhere but the slab. In the eighth, extensive yielding was observed in the web especially near the ends of the cover plates. An increase in permanent set of 1.1 and 0.2 in. developed during the ninth and tenth load increments, respectively.

The test was terminated after a total permanent set of 13 in. was observed. Tensile yielding had developed through the full depth outside both ends of the cover plates. At midspan, tensile cracking was observed in the concrete within 2 in. of the slab surface. There was no sign of crushing at the top surface.

The large maximum static moment obtained by Bridge 3B indicated the strengthening action of the composite section. The increased stiffness of the composite section was evident in the moment-deflection plot. The magnitude of the yield load is less of a critical quantity in composite bridges. Large load increments correspond to small permanent deformations in composite bridges, which is in contrast to the results obtained in the noncomposite tests.

When the loads corresponding to the effective yield stress (yield point – average residual stress in flange) were exceeded, large permanent sets accumulated with relatively few vehicle trips. This was particularly the case in bridges without coverplates. Large permanent sets occurred in all bridges when the yield point was exceeded. When the applied moment exceeded the ultimate moment of the bridge, the permanent deformations increased with each successive trip of the same load. Fatigue cracking was observed at the toes of the coverplate welds, which was caused by large number of cycles of transient stresses.

The static capacity of any cross section of the test bridges (at first yielding, cracking, or failure) was determined assuming the bridge responded as a beam. The assumptions were justified by the generally uniform response to transient loads (applied to the centerline) by all three bridge beams.

Complete interaction was assumed in the composite bridges. In noncomposite bridges, the slab and beams were treated completely independently. The external moments calculated to produce first permanent set in the bridges were on the order of 10 to 20 percent less than the applied moment—except for that of the composite Bridge 3B for which case substantial increases in load produced relatively small increases in permanent deformation.

University of Tennessee Tests (*104*). Tests were conducted on four deck-girder highway bridges in Tennessee. The bridges were subjected to three types of loading: vibration, rolling loads (HS20 and approximately two times HS20), and static tests to failure (using a rock-anchor system). Two of the bridges were constructed with rolled steel beams. The first was a composite four-span continuous bridge, and the second was a noncomposite three-span continuous bridge.

The objectives of the tests were to determine the lateral load distribution, the dynamic response, and the ultimate strength and failure mode.

Tests to Failure. The first bridge, B1, was loaded with the rock-anchor system to simulate an HS truck in each lane located

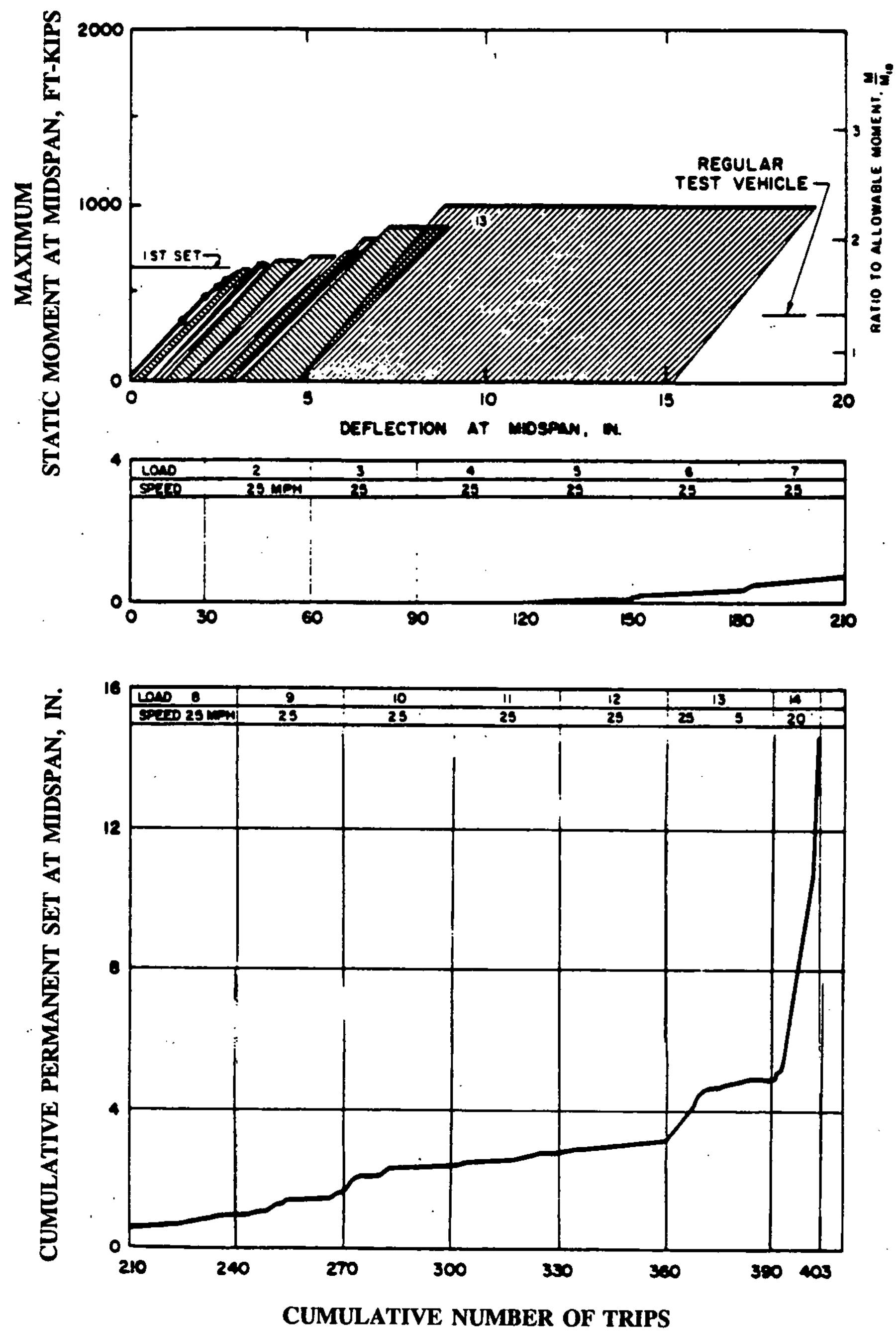


Figure 2.7. Results of AASHO bridge tests—Bridge 1A (36).

in an approximate position assumed to cause the maximum positive moment near the centerline of the span. The second bridge, B4, was loaded without the simulated front wheel loads. For both of these tests, the first evidence of distress was exhibited in the diaphragms. Noticeable and audible slip occurred between the diaphragm and the girders as the load increased.

Bridge B1 behaved linearly elastically until yielding occurred at a section under the applied loads near the center of the span. Tensile cracks visible in the deck slab over the first pier developed shortly thereafter. Soon after yielding began and the loading was increased further, the bridge "lifted off" the abutment nearest the applied load, thus making it impossible to develop more moment at the first pier. The deflections increased following yielding, and a plastic hinge formed at a section near the center pier at the end of the cover plates on the side of the pier away from the loaded span. Web buckling of Girder 4 occurred at the formation of the hinge. Shortly after, secondary compression failure of the curb occurred at the section of maximum positive moment. The response of Bridge B1 to failure was compared with the finite element analysis program BOVAS (*68–70*) (Table 2.10, Fig. 2.12).

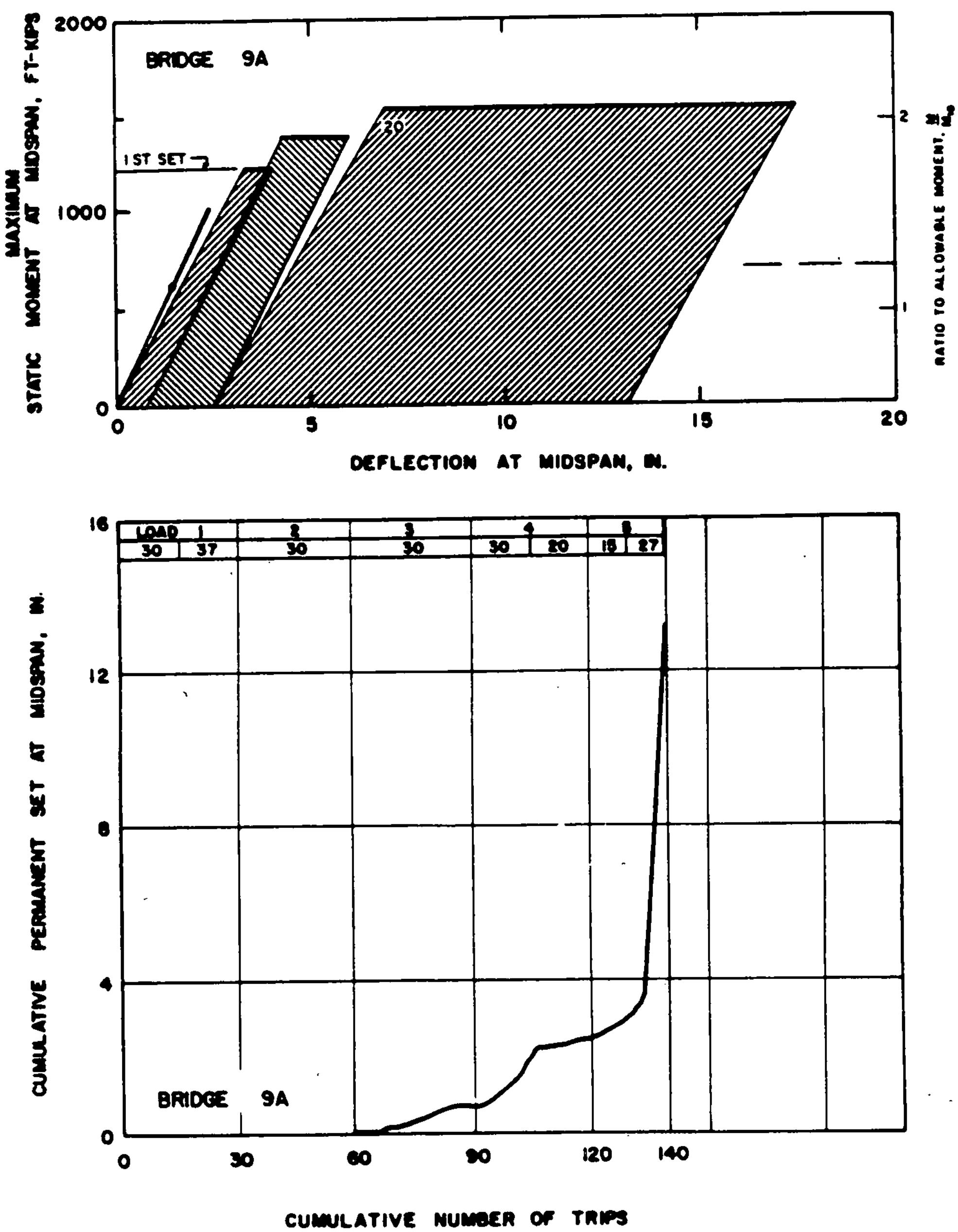


Figure 2.8. Results of AASHO bridge tests—Bridge 9A (36).

Bridge B4 was designed to act noncompositely; however, strain readings indicated a large degree of composite action existed prior to yielding of the bridge. A plastic hinge developed, after which, considerable rotation and deflection occurred with only nominal increases in load. Yielding occurred in the negative moment regions. Plastic hinges formed in the girders at the end of the cover plates near two piers on the sides away from the loaded span.

There was quite a bit of diversity in the composite action observed in Bridge B4, which was designed to act noncompositely. Strain readings indicated that a large degree of composite action existed until first yield. This composite action existed despite rigorous vibration and heavy rolling loads which were imposed on the bridge before the tests to ultimate.

The results from the rolling load tests indicated that the lateral distribution of load was a function of the lateral position of load. The axle spacing, load magnitude, and vehicle speed had a measurable effect on the total moment, but only a minor effect on the lateral distribution of load. Similar results were observed in the static tests to first yield. The distribution factors outlined in the AASHO specification appeared to be conservative.

Tests indicated that the curbs acted as an integral part of the composite bridges. The structures including the curbs were treated as wide beams in calculating the ultimate capacity of the sections. The method predicted the ultimate capacity of the two bridges (flexural failure) within 9 percent. Ultimate loads calculated with the AASHO assumptions (which fail to account

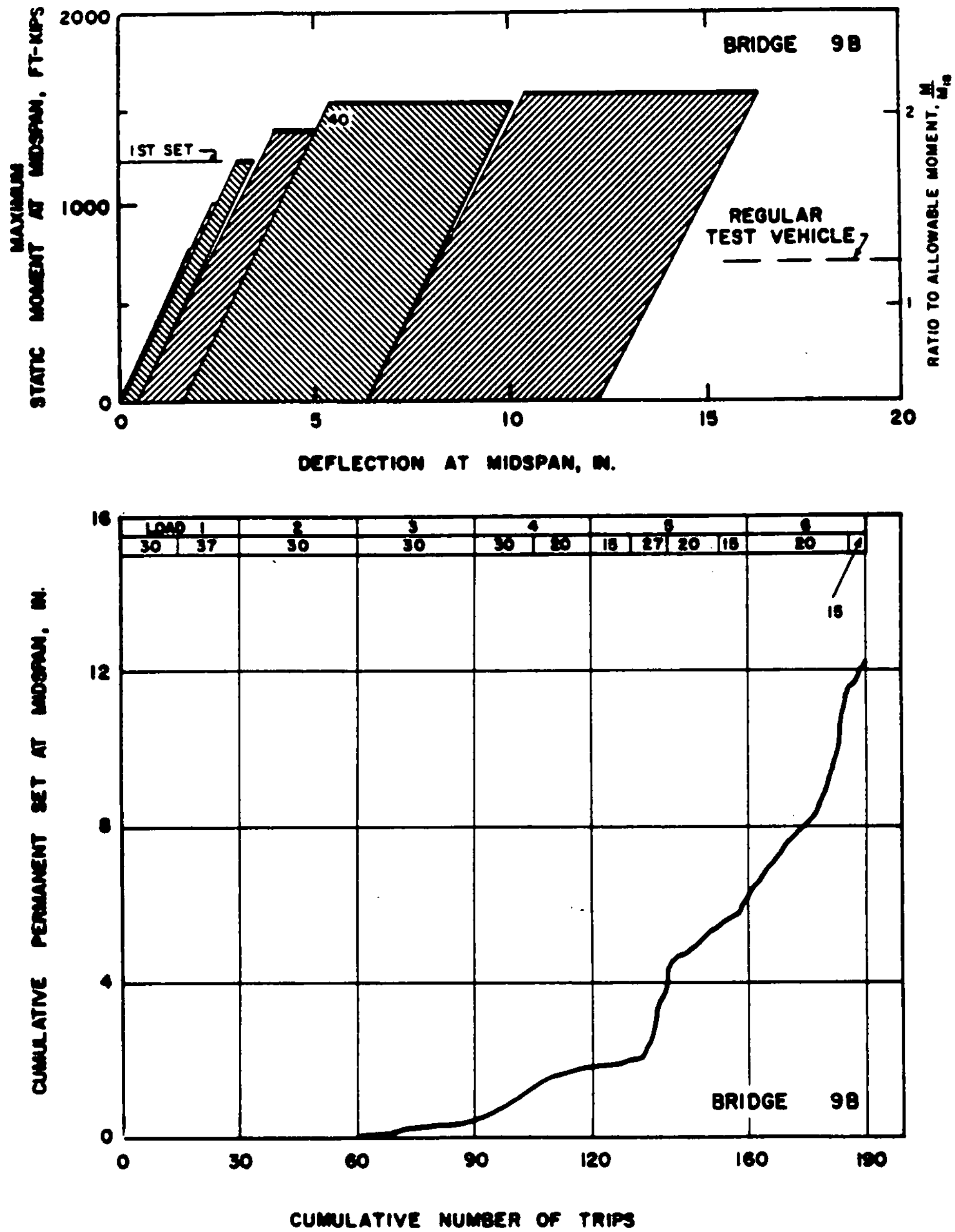


Figure 2.9. Results of AASHO bridge tests—Bridge 9B (36).

for redistribution of moment in continuous bridges) were lower than the actual values.

The load causing "first permanent set" is less readily identifiable than the ultimate load. Elastic theory and measured steel yield strength were used to calculate the loads causing "first permanent set." These calculated values were then compared with the measured values. The computed load for Bridge B1 compared reasonably well with that indicated by the measured load-deflection curve. The load computed for Bridge B4 was calculated assuming that the bridge acted noncompositely; the calculated value was approximately 75 percent of the measured load. The difference was attributed to the high degree of composite action observed before first yield. AASHO Specifications for limiting the overload on the basis of the first permanent set seemed reasonable according to the investigators.

Tests by Baldwin (*105*). A three-span continuous I-beam bridge with a simply-supported approach span was subjected to static and dynamic tests. The bridge was designed for H15-44 as a noncomposite structure.

The bridge was subjected to a series of five tests. The first three were conducted to investigate the lateral distribution of load; the other two were conducted to investigate the effect of induced roughness (truck shock absorbers served to alleviate

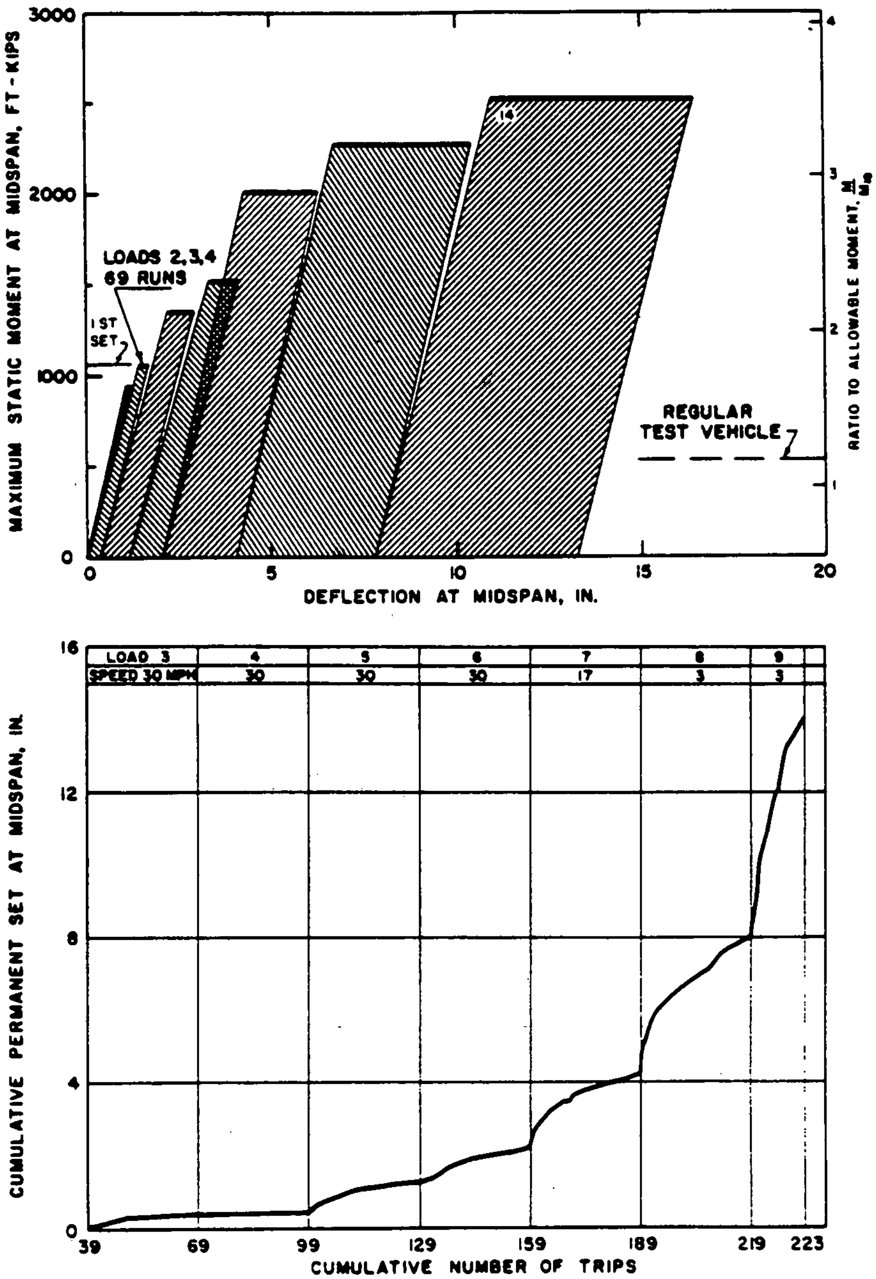


Figure 2.10. Results of AASHO bridge tests—Bridge 3B (36).

the detrimental impact effects). Mean moments and deflections appeared to be independent of the speed of the vehicles.

The actual behavior of the bridge was closer to composite than to noncomposite action. This action was attributed to friction between the steel and concrete. The deflections in the center span corresponded approximately to those assuming composite action. The deflections in the end span were equivalent to those approximately a third of the way between noncomposite and composite assumptions. The simply supported span exhibited much less composite action.

The lateral load distribution was compared with AASHO and theoretical analysis. The AASHO lateral load distribution seemed reasonable.

2.3.6.3 Summary of Bridge Tests

The available information on bridge tests provided data for the calibration of our inelastic bridge rating procedures. These will be described later in this report.

From the tests, it appears that a wide beam analysis may be quite sufficient in determining the limiting ultimate capacity of the bridges. In addition, these field test results provide insight regarding the range of composite action that may be expected from bridges designed to be noncomposite. Although composite action initially may be quite significant in bridges designed to be noncomposite (without mechanical shear connectors)

Table 2.9. Damage to AASHO Bridge 3B as computed by BOVAS (*69*)

LOAD VS. DAMAGE RECORD - EXAMPLE NO. 1

Load (kip-ft.)	Damage - Test	Load (kip-ft.)	Damage - BOVAS
		762	Yielding of exterior beam bottom flange @ midspan
		906	Yielding of interior beam bottom flange @ midspan
		1059	Yielding of coverplate of exterior beam @ midspan
		1156	Yielding of exterior & interior beam bottom flange at end of coverplate
1333	Yielding of bottom flange near ends of coverplate	1364	Complete yielding of interior beam coverplate. 85% of interior coverplate beam bottom flange has yielded
1493	Almost complete yielding of bottom flange	1455	Complete yielding of interior beam coverplate. 85% of interior except near supports beam bottom flange has yielded extensive coverplate yielding
		1662	Bottom layer of slab has a transverse crack all the way across at midspan
		1883	The web of exterior beam has yielded over 70% of its depth
2000	Web yielding is clearly evident	1919	The web of interior beam has yielded over 70% of its depth
2277	Extensive web yielding and tension cracks in slab halfway through depth in coverplated section	2296	The slab has a transverse crack through 50% of its depth at midspan and 33% through depth in coverplated section. The web has yielded through 86% of depth at midspan

(*104,105*), the interaction breaks down as the structure is taken into the inelastic range (*104*).

2.3.7 Repair, Rehabilitation, and Retrofit

2.3.7.1 Introduction

There are three possible options for bridges that have insufficient capacity to carry normal loading: (1) do nothing and post if necessary, (2) repair or upgrade, or (3) replace.

NCHRP Report 271 "Guidelines for Evaluation and Repair of Damaged Steel Bridge Members" (*106*), *NCHRP Report 293* "Methods of Strengthening Existing Highway Bridges" (*13*), and several publications from the IABSE Symposium on "Maintenance, Repair, and Rehabilitation of Bridges" (*107*) provide excellent sources and guidelines for choosing the appropriate option.

It is usually much more economical to repair or strengthen bridges than it is to replace them. The funds that are available to maintain the infrastructure are generally insufficient to replace every bridge which may be a possible candidate for replacement. Thus the most economically feasible options previously mentioned are the "do nothing/post" or "repair/upgrade" approaches.

Once the situation has been assessed and the appropriate option chosen, the next step of the "do nothing/post" approach is self-explanatory (*108*). If the "repair/upgrade" option is chosen, the next step is to select the most appropriate repair method.

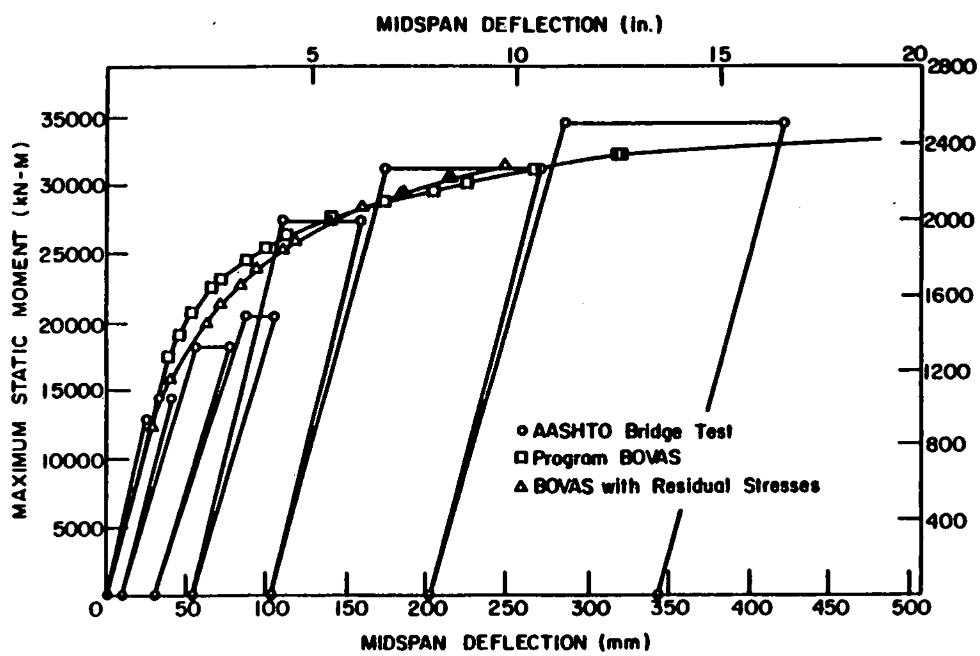


Figure 2.11. Measured versus predicted deflection for AASHO 3B (69).

Table 2.10. Damage to Tennessee bridge as computed by BOVAS (69)

Load (kip-ft.)	Damage - Test	Load (kip-ft.)	Damage - BOVAS
		259.5	Up to this point there has only been longitudinal cracking of the slab in the bottom layers at the centerline under or near the load
		402.5	The first transverse crack appears in the top layer of the slab near the first pier
		446.7	Transverse cracks appear in the top of slab near the second pier
		556.4	First yielding begins in bottom flange of interior beams in area under the load
		590.9	First yielding begins in bottom of web of interior beams in area under the load
620	First yielding of steel appears to occur at this load - shortly after yielding started the bridge "lifted off" the abutment nearest the load	625.5	Transverse crack over first pier penetrates through 50% of the slab depth
650	Tension cracks visible in deck slab over first pier		
700	Tension cracks extend across the slab and through the curb at second pier	710.4	First transverse crack in the bottom of the slab appear under the load
		757.5	Slab over first pier is completely cracked longitudinally through the complete depth; however, the reinforcement is still functional
		767.8	Slab over second pier is cracked completely through the depth in the longitudinal direction
		819.3	Yielding of the bottom flange of the exterior beams in the area of the load begins
		851.6	Slab between the interior and exterior beam at the second pier is now cracked through 60% of its depth in the longitudinal direction
		925.4	Bottom transverse steel in the slab yields in tension near the load
		1029.2	The transverse crack in the bottom of the slab under the load is halfway through the slab depth in the area near the center of the bridge
		1072.6	The web of interior beam under the load is now fully yielded
		1119.9	First crushing of slab at load point
		1202.3	Yielding in compression of top transverse slab reinforcement in area under load. Yielding in tension of top longitudinal slab reinforcement near the first pier. Yielding in tension of bottom longitudinal slab reinforcement in area under the load
		1221.2	The interior beam in the area under the load has fully yielded forming a plastic hinge in the beam
1265	Maximum load reached Compression failure of curb section	1254.7	The web of exterior beam under point of loading has fully yielded

2.3.7.2 *Selection of Repair/Upgrade Method*

NCHRP Report 271 (*106*) is essentially a manual of practice for the repair of damaged steel bridges. It provides a list of repair techniques, guidelines for selection, listing of necessary equipment, and examples of use. The following is a brief description of repair methods for use with steel members.

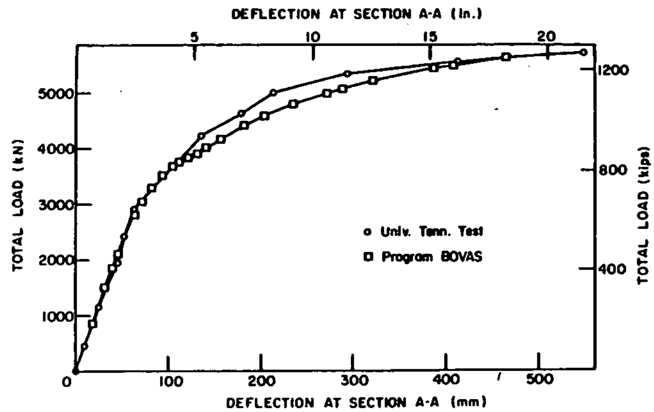


Figure 2.12. Measured versus predicted deflection for Tennessee bridge (68).

Flame Straightening. With this technique, heat provides the mechanism for straightening a damaged or bent member. It does not significantly degrade the steel properties and is recommended for use on any type of bent member.

Hot Mechanical Straightening. Heat is applied to all sides of the bent member in this case, and an applied force is used to straighten the member. The results of this technique are unpredictable, and it is only recommended for use on primary compression members and secondary members.

Cold Mechanical Straightening. A force is applied to the member without heat. It is difficult to determine the detrimental effects of this method. It is only recommended for use once. The plastic strain and number of cycles used to straighten the member should be limited.

Welding. This method is only to be used with weldable steel. It is very useful in the repair of defects, cracks, replacement of sections, and straightening. Nondestructive tests and thorough inspection should accompany the welding.

Bolting. This method is recommended for the replacement of a damaged section. A new steel segment may be bolted across the region with high-strength bolts. This is probably the safest repair technique.

The selection of the appropriate repair/upgrade method should be based on the material properties, type of member, and type of damage. Other considerations include: relative cost, user inconvenience, durability and strength. Combinations of the aforementioned methods should also be an option.

2.3.7.3 Strengthening of Bridges

NCHRP Report 293 (*13*) proposes the following alternatives to strengthen or upgrade the capacity of existing bridges:

1. Replacement of the existing deck with a lightweight deck—such techniques include the use of lightweight concrete, exodermic decks (prefabricated reinforced concrete joined to a lower layer of steel grating), precast concrete panels, and aluminum or steel orthotropic plates. Many of these techniques are already used.
2. Increase the composite action—this could be easily accomplished in bridges for which deck replacement is planned. In existing decks, the slab could be cored to accommodate shear stud placement or epoxies could be used to provide better bond between the steel and concrete. The latter technique has shown only limited effectiveness.
3. Increase the transverse stiffness of the bridge—this could be achieved by adding additional bracing or transverse post-tensioning. These methods would result in a more favorable distribution of stresses to the beams and represent economical methods of improving the load carrying capacity of bridges.
4. Improving the strength of individual members—this could be achieved by adding cover plates and external shear reinforcement to existing members.
5. Adding or replacing members—this is expensive but can be economically advantageous if only a single or very few members in the bridge are damaged or deteriorated.
6. Post-tensioning of various bridge components—post-tensioning of the longitudinal members and decks can result in substantial stiffening of the structure as well as reductions in deflections and cracking. The development of external post-tensioning techniques for bridges makes this method particularly attractive.
7. Strengthening of connections—this can provide more end restraints to members and result in better load redistribution and reduced deflections.
8. Developing continuity over interior supports or providing additional supports to the structure.

The IABSE Symposium contains a reference (*107*) that describes the use of the longitudinal post-tensioning technique to strengthen a series of simply supported composite bridges. The results of the tests appeared quite successful.

2.4 INELASTIC RATING PHILOSOPHY

2.4.1 Recapitulation of Research Findings

The review of relevant research in this chapter showed that it is within the capability of the current state of the art to predict the post-elastic or the post-buckling behavior of individual steel composite or noncomposite beams and plate girders. Both the theoretical knowledge and the experimental verification exist to be able to confidently forecast the inelastic strength and deformation capacity of these longitudinal bridge elements. It is also well known what the required limit of plate slenderness and lateral bracing spacing is to ensure adequate ductility so that the full strength of the member can be achieved. Simple empirical rules have been developed and tested for these geometric limit ratios.

Methods of analyzing bridge systems vary in complexity from the customary simple and conservative procedure of analyzing individual longitudinal bridge members (i.e., beams or girders) using empirical distribution factors to allocate the axle load to the individual member to more accurate and complicated distribution schemes. The next level of complexity is to use a grid or orthotropic plate type analysis. Efficient and proven means exist for calculating the reactions, shears and moments in a bridge member. The final sophistication in analysis is the finite element method, which can be used for a three-dimensional calculation of forces, deformations, and strains in the various parts of the bridge.

It was found that a well-documented, theoretically consistent, and practically reasonable method exists to rate a bridge. The method offers consistent reliability in bridge rating. A full set of load factors and resistance factors has been developed and a rating scheme based on these factors has been published by AASHTO. This new methodology is based on first-order second-moment probability concepts (*3*).

In the examination of the literature, it was shown that there is a great fund of experience in designing structures to inelastic limit states. Plastic design of steel buildings against static and dynamic loads is one such area. Another area is seismic design criteria, which depend greatly on the inelastic deformability, i.e., ductility, of the steel frames.

2.4.2 Selection of a Limit State for Bridge Rating

The purpose of bridge rating is to ensure that the chance of exceeding the applicable limit state is acceptably small. This aim is achieved by applying probabilistically determined resistance factors to the resistance, and load factors to the load effect. These factors are obtained by calibration to furnish the same reliability as that possessed by a representative set of bridges that were found to be adequate by the conventional rating method. This calibration exercise and the development of the factors, which accounts for the variability of the resistance and the load effect, was accomplished by Moses and reported in *NCHRP Report 301* "Load Capacity Evaluation of Existing Bridges" (*3*). That research was carefully evaluated for its suitability for this project on inelastic bridge rating and was found to be applicable. Thus the same framework of load and resistance factors was adapted for this project as that adopted in the AASHTO *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* (*10*). The only change that was recommended was a change of limit states.

In the search for appropriate limit states, the following considerations entered into the decision process:

1. Existing steel beam and girder bridges were shown to be stronger than predicted by traditional rating methods.
2. The "global" bridge system is stronger than the individual member or component.
3. The material and most members in a steel bridge are ductile.
4. Multiple beam and girder systems are very robust and redundant.

5. At the time of rating, the bridge is likely to have already survived many large illegal loads and therefore there is in existence a residual moment field as well as a set of moderate permanent deformations, which are so small as to be undetectable to the untrained naked eye.

The following choices are available for limit states:

1. *No residual damage is permitted under many applications of the factored rating vehicle.* The limit state thus is the plastic moment M_p, the yield moment M_y, or the critical moment M_r when the moment distribution is determined by linear elastic analysis. This limit state is the criterion of the contemporary rating methods in use around the world. Essentially no damage exists at the limit.
2. *The bridge members form a plastic mechanism under one application of the factored rating vehicle.* This limit state is the same as that of the AASHTO "Alternate Load Factor Design Specification" (*57*) under ultimate loading. In this design method, the mechanism limit state never governs because of the stress check in positive bending after the formation of the automoment. The attainment of the plastic mechanism is the theoretical maximum load that can be supported by the bridge, and it corresponds to total damage. In reality the actual deformations at the plastic mechanism load are still relatively small, and the bridge does not physically "collapse." This was demonstrated by the Tennessee bridge tests (*104*) and by the final loading regime of the FHWA test bridge (*67*). The same observation was made in the Iowa bridge that was tested as part of the present project (see Appendix E). It should be noted, however, that all three of the above referenced bridges were loaded by static loads applied by hydraulic jacks and not by moving vehicles. However, both strength and ductility reserves are believed to exist beyond the plastic mechanism load because of strain hardening. Nevertheless, it is only prudent and reasonable to limit the maximum rated capacity of a bridge to the load corresponding to the formation of the global plastic mechanism under the factored rating vehicle.
3. Although from a detached standpoint it is perfectly logical to base bridge rating on a total damage concept, it was felt that for a method which could be acceptable to bridge engineers, a somewhat less radical approach would be desired. The limit state, which is recommended herein, is that *the bridge develops, after several passes of the factored rating vehicle, a residual moment field [also often called a set of automoments (57)] such that all subsequent passes are accommodated by elastic response.* The bridge is rated by this method to *shakedown* to an elastic state.

The spectrum of damage from the first hinge formation (no damage) to the formation of the plastic mechanism (complete damage) is shown in Figure 2.13. The shaded area in this diagram is the region in which the proposed rating method has its limit state. The greatest degree of damage permitted is the "shakedown limit." If the load exceeds this value, each subsequent passage of the vehicle will cause additional plastic deformation, i.e., the structure will *not* shakedown. For any load level at or below the shakedown limit the structure will shakedown, i.e., the plastic action will stop, and all further passages of the load are resisted elastically. The shakedown process is also illustrated in Figure 2.14 where the number of cycles of

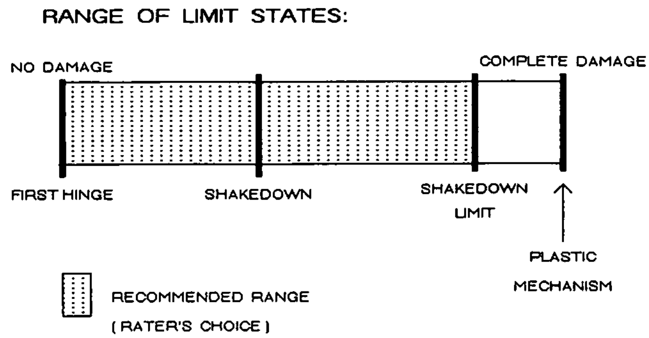


Figure 2.13. Range of damage states suitable for rating.

load application is plotted against the residual deformation. If shakedown occurs, then the curve levels off; if no shakedown occurs, the deformations continue to increase. It is recommended that rating under the factored rating vehicle be limited by a prescribed permanent deflection. The amount of deflection is up to the rating engineer. A suggested maximum permanent, or residual deflection in inches is $L/300$, where L is the span length in inches. This is the limit of visual observation.

The recommended limit state of a maxium permanent set of $L/300$ is a condition that has only a small chance of being exceeded once in the remaining life of the bridge. In the rating exercise, this limit is being checked against the factored maximum vehicle load. The proposed deformation limit state is less severe than that inherent in the approved Alternate Load Factor Design (also known as the Auto-Stress Design) method where at the factored ultimate loads the formation of a plastic mechanism is permitted for compact members. Instead of expecting plastic collapse at ultimate, the proposed method will result in a small permanent set, which is not noticeable with the naked eye. Almost none of the bridges will see such deformations during their remaining life. Although it would not be appropriate to design a new bridge this way, rating an existing older bridge in this manner is an acceptable risk because it can be assumed that the structure has successfully survived all the serviceability limit states.

2.4.3 Summary of Rating Philosophy

Based on the evaluation of past and present research and practice, the following philosophy is recommended for rating existing steel beam and girder bridges:

- Recommended limit state: Global shakedown of the multigirder bridge system; in general, the residual permanent maximum deflection shall not exceed $L/300$ (inches).
- Recommended rating vehicles: AASHTO vehicles (*9*).
- Recommended reliability level: Same reliability against shakedown as that inherent in the AASHTO *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* (*10*) against the formation of the first plastic hinge; this is accomplished by using the same load, resistance and impact factors.

The following section of this report will describe the tools by which this rating philosophy is implemented.

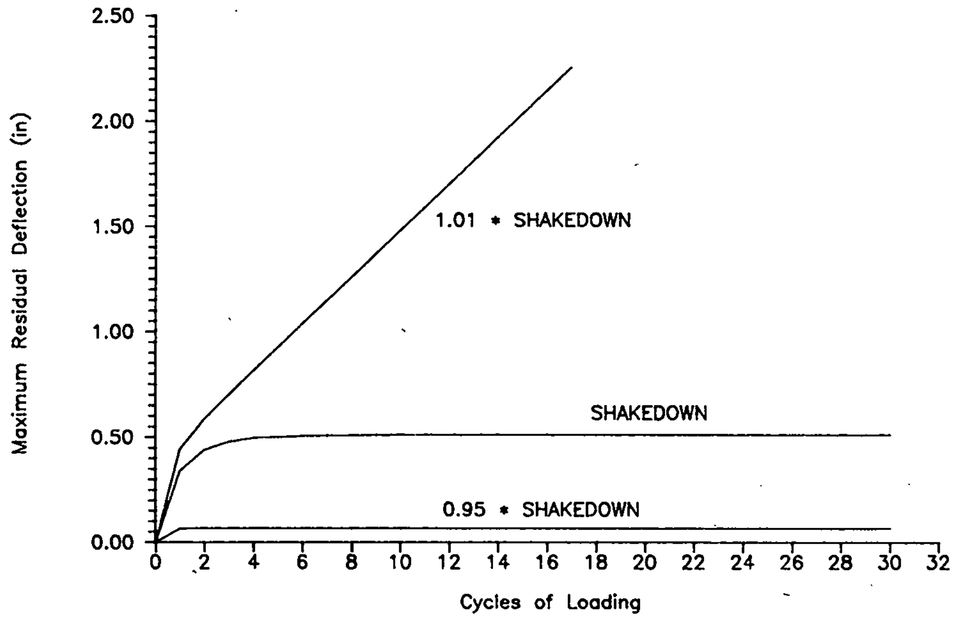


Figure 2.14. Inelastic behavior of two-span example bridge.

2.5 INELASTIC RATING METHODOLOGY

2.5.1 Statement of the Requirements

Bridge rating consists of the mathematical determination of the rating factor *RF*, and then of experimental, judgmental, and administrative decisions on an appropriate action if the value of *RF* turns out to be less than unity. The objective of this research project was to develop a method of calculating the rating factor *RF* such that some of the inelastic reserve of the steel bridge can be utilized.

The rating factor *RF* is the number by which the weight of the factored rating vehicle is multiplied such that a limit state is just attained. For example, if the limit state is the plastic moment M_p of the compact cross section, then the checking condition can be written as:

$$\gamma_D M_D + \gamma_L (DF)(RF) M_L (1 + I) \leq \phi M_p \qquad \text{(Eq. 2.20)}$$

In this equation γ_D, γ_L and ϕ are the dead and live load factors, and the resistance factors obtained from Reference *10*, respectively; M_D is the dead load moment, M_L is the live load moment from the rating vehicle, *I* is the appropriate impact factor from Reference *10*, and *DF* is the distribution factor. The rating factor can then be calculated from Equation 2.20.

This equation makes use of the modern load and resistance factors that were obtained from a thorough probabilistic analysis of the loads on and the strengths of bridges (*3*). They can account for the observed bridge traffic and weight limit enforcement conditions.

A considerable improvement in the determination of the moments acting at the critical locations can be achieved by using a two- or three-dimensional idealization of the bridge structure. After a thorough evaluation of the available methods, it was decided that the multi-beam deck-slab bridge should be analyzed as a two-dimensional grillage system. The grillage members were idealized according to the models proposed by Bakht and Jaeger (*35*), and the analysis was patterned after the suggestions in the text by Weaver and Gere (*109*). With the grillage idealization as a basis, a framework was set up to attack the requirements of inelastic bridge rating as they are enumerated at the end of Section 2.4. Thus a tool was developed to determine the shakedown criterion for a bridge system. In addition, a separate but independent method was developed to calculate residual deformations of compact and noncompact members. These analytical tools will be described in the following two sections of this chapter.

2.5.2 Determination of the Shakedown Limit of the Bridge

The details of the development of the grid analysis program, as well as the details of modeling the beams, the slabs, the diaphragms, and the connections between these elements, are discussed in Appendix C "Two-Dimensional Shakedown Limit."

Four PC-based computer programs were developed during the course of project: SETUP.EXE, EGRID.EXE, SHAKE.EXE, and IGRID.EXE. See Appendix H for information on obtaining this software and instructions for its use.

SETUP.EXE performs four tasks, which produce the required information for a shakedown analysis: It discretizes the bridge into longitudinal and transverse elements, produces the dead load equivalent fixed end force vector, develops the live load scheme to move the truck loads across the structure for different transverse positions, and produces the required longitudinal member moment capacity information.

EGRID.EXE uses the information from SETUP.EXE to perform an elastic analysis of the bridge. The program produces the elastic envelopes for the moving trucks at all their lateral positions. The dead load force effects are analyzed separately. The results of this program are used by SHAKE.EXE to find the elastic limit, the alternating plasticity limit, and the shakedown limit rating factors. It also produces the grid distribution factors for the particular bridge configuration.

SHAKE.EXE uses the longitudinal member moment capacity information from SETUP.EXE and the live load moment envelopes and the dead load moments from EGRID.EXE. Verification of the limit state, finding the residual moment field, and estimating the residual damage is accomplished by executing the inelastic grillage analysis program IGRID.EXE.

IGRID.EXE is a self-contained linear segmental grillage analysis program that performs a step-by-step elastic-plastic analysis. The output of this program contains the forces (moments, shears, reactions) and deformations (rotations and deflections) at the level of the load for which the analysis is performed, and the corresponding quantities after the vehicles have been removed. These residual deformations and forces are those that represent (a) the damage incurred and (b) the internal force system that helps the structure to resist subsequent live loads elastically. The analysis can be repeated as often as necessary to ascertain whether or not the system shakes down (see Figure 2.14, for a sample output from program IGRID.EXE).

The shakedown limit rating factor (RFSD) is the smallest value of Γ of all spans of a longitudinally continuous bridge, where Γ for each span is evaluated from the following three simultaneous equations (see also Appendix C):

$$\Gamma\sum M_{el+}{}^{(i)} + \sum M_d{}^{(i)} + \sum m_{r+}{}^{(i)} = \sum M_{p+}{}^{(i)} \quad \text{(Eq. 2.21a)}$$

$$\Gamma\sum M_{el-}{}^{(j)} + \sum M_d{}^{(j)} + \sum m_{r-}{}^{(j)} = \sum M_{p-}{}^{(j)} \quad \text{(Eq. 2.21b)}$$

$$\sum m_{r+}{}^{(i)}\,(1/\alpha) - \sum m_{r-}{}^{(j)} = 0 \quad \text{(Eq. 2.21c)}$$

Equation 2.21a states that the sum of the positive elastic live load and dead load moments (ΣM_{el+} and ΣM_d obtained from the elastic grid analysis program) and the residual moments (Σm_{r+}) across all the girders of the span equals the sum of the respective plastic moment capacities. Equation 2.21b describes the same equilibrium condition for the negative moment region. Equation 2.21c defines the equilibrium of the residual moments. The term αL is the location of the maximum positive moment in the interior of the span. Equations 2.21a–c are for an exterior span where there is only one negative moment region. For an interior span with negative moments on both ends another equation must be added. All elastic moments are calculated for the factored loads, and all plastic moments include the resistance factor.

The basis for the shakedown analysis is the assumed elastic-plastic moment-curvature curve (see Figure 2.15). Such a relationship is assured if the cross section is compact (i.e., unbraced length and the width-thickness ratios of the flanges and the webs are less than the limits specified in Section 10.48.1 of the AASHTO Standard Specifications). Strictly speaking, the shakedown limit analysis by the grid method applies only to noncomposite compact sections. However, in Chapter 3 it will be explained how the program could be used to give a conservative shakedown limit rating factor also for composite and for noncompact beams. It is really not necessary to do this, because

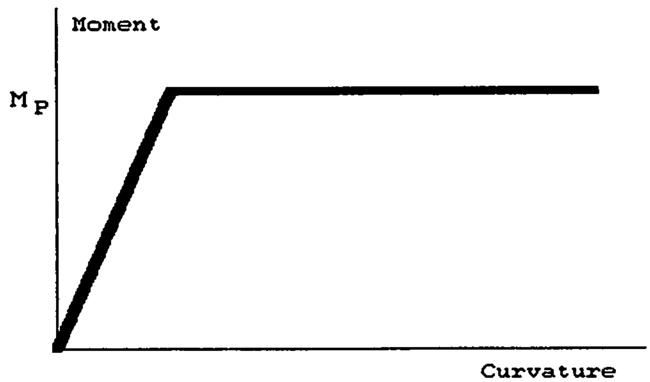


Figure 2.15. Elastic-plastic moment curvature relationship.

the Residual Damage Analysis program was specifically developed to deal with composite or noncompact beams.

The grid analysis uses a global incremental collapse mechanism for each span (Figure 2.16). This means that the incremental plastic hinge forms across the whole width of the bridge. This is the most critical condition and it results from two vehicles moving side-by-side along a two-lane bridge. It is assumed that the concrete deck does not deteriorate in the transverse direction. This is reasonable for the loadings considered on a two-lane bridge. The shakedown limit is based on the elastic response from loads placed in many transverse positions along the length of the bridge. It is not governed by a very heavy load in any one position, which would be more critical for a local slab type failure.

In summary, the grid analysis model and the resulting computer programs give the global shakedown limit rating factor of multi-beam slab-type bridges erected of compact shapes. The shakedown limit is the load level at which the structure just shakes down (see Figure 2.14). By-products of this analysis are the elastic moment envelopes, the rating factor for the first hinge formation, the global plastic mechanism rating factor, the alternating plasticity rating factor, and the residual moment distribution, the grid distribution factor, the residual deformations. The most important information for the rating engineer is the shakedown limit rating factor. The relationship of the various kinds of information from this analysis is depicted in the flow chart of Figure 2.17.

2.5.3 Residual Damage Analysis

The previously described grid analysis method is able to make an analysis of the whole bridge system consisting of the girders, the slab, and the diaphragms. Because of its comprehensiveness, there had to be some restrictions as to the details of what could be analyzed, therefore only right bridges (i.e., no skew) and compact noncomposite shapes can be analyzed. While one can "tweak" the programs to solve for less restrictive cases, this is not convenient for a rating engineer who may not be fully cognizant of the full moment-curvature relationships for composite and noncompact sections. Furthermore, the faster analysis (programs EGRID.EXE and SHAKE.EXE) gives only

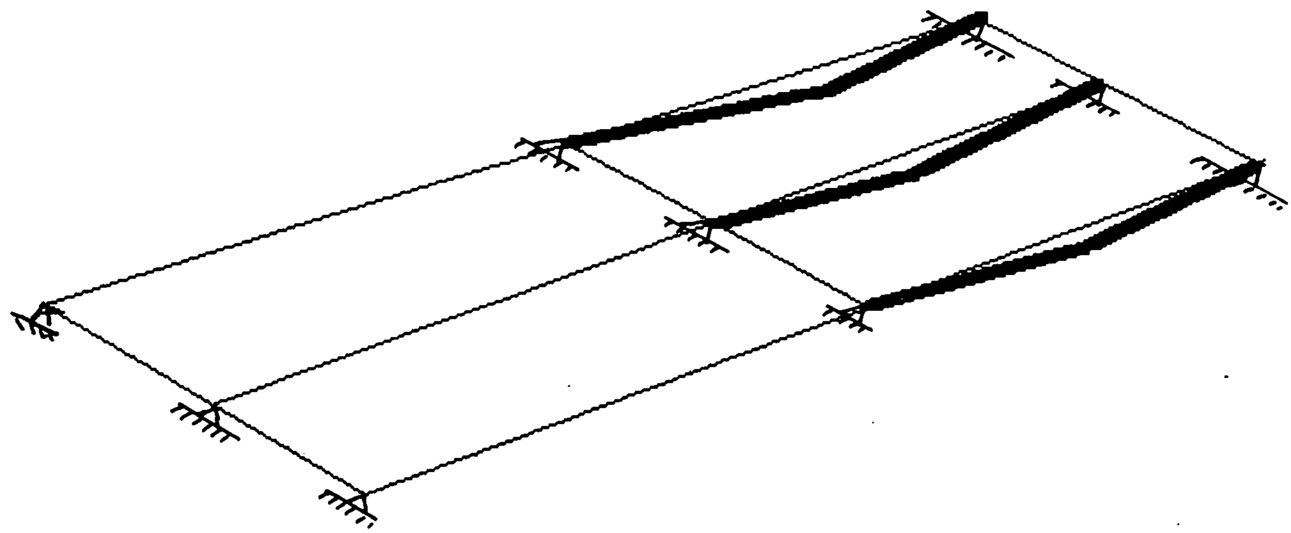


Figure 2.16. Global shakedown mechanism.

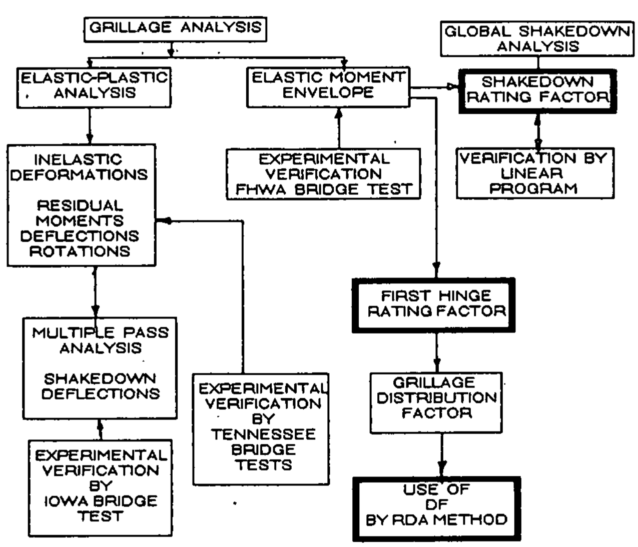


Figure 2.17. Grid analysis flow-diagram.

the shakedown limit rating factor and one must engage the much longer working program IGRID.EXE to calculate deflections. This is not really suitable for routine rating exercises. The program is best used for in-depth investigations. Thus is was necessary to develop a faster analysis scheme that can also handle compact as well as noncompact composite and noncomposite beams and that can furnish the residual deflections after shakedown (i.e., the rater can assess the damage under the factored rating vehicle). Such a method was developed (*5*) and is called "Residual Damage Analysis," or RDA.

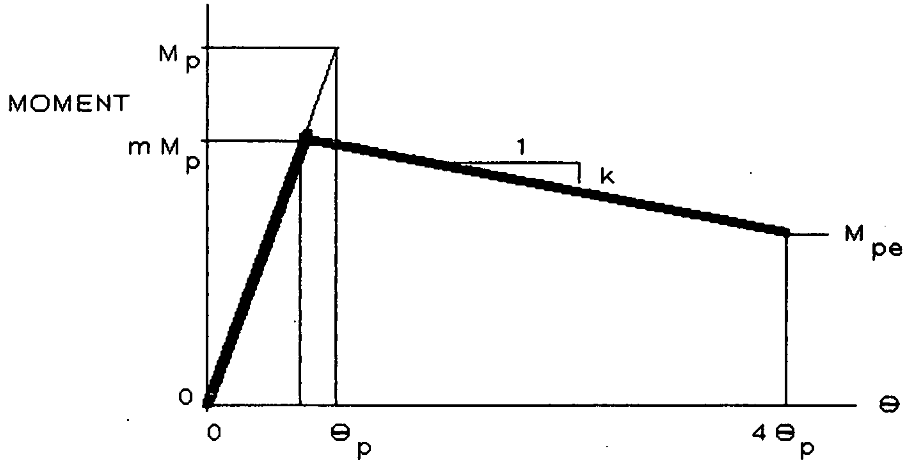
RDA can satisfy all of the above stated conditions; however, this is done at the cost of being able to analyze only one girder. The pertinent details of the method are furnished in Appendix B "Residual Damage Analysis."

The connection between the individual girder analysis of RDA and the total bridge system is provided by the distribution factor *DF*. This factor in an output of the grid shakedown analysis (from program SHAKE.EXE). In case it is not desired, or inappropriate, to perform the grid analysis, one can use the various available distribution factor schemes (e.g., see References *44–48*) or by default use the conservative AASHTO distribution factors. The RDA method is able to accommodate any appropriate distribution factor scheme.

RDA is based on a structural analysis that is performed at successive positions of the factored rating truck along the beam, which is also loaded by the factored dead load. When the plastic moment capacity is exceeded, an inelastic analysis is performed that calculates the residual moments and the elastic and inelastic deformations. The residual forces and moments are considered in the further analysis as the vehicle is moved to the next position. When the vehicle is totally off the bridge, then the remaining moments and deflections are the representation of the damage to the structure.

The inelastic analysis method is described in detail in Reference *5* and in Appendix B. It is based on an application of the conjugate beam method. The analysis is able to accommodate noncompact noncomposite cross sections, which have a descending inelastic moment rotation curve (Figure 2.18) and which are characteristic of the negative moment region at the interior supports of composite or noncomposite bridges, as well as composite cross sections under positive moment, which have an increasing moment rotation curve in the inelastic range (Figure 2.19). These moment rotation relationships are conservative linearizations of empirical models based on many experiments (see Appendix B and References *23,66,74*). Until the results of contemporary research permit a more accurate characterization, the curves of Figures 2.18 and 2.19 can serve as a convenient and conservative tool for inelastic rating.

In summary, Residual Damage Analysis will give the residual deflections (i.e., the damage under factored loads) and residual moments for a given rating vehicle and a predefined rating factor. The analyst can then decide if the resulting level of damage is above or below a limiting value. RDA must be repeated until it is ascertained that the beam has shaken down to elastic responses to all subsequent loads of the same magnitude.



M_p = FULL PLASTIC MOMENT OF STEEL SECTION

$m M_p$ = NOMINAL MOMENT CAPACITY (AISC-LRFD SPECIFICATION FOR FLB & WLB LIMIT STATES)

θ_p = ELASTIC ROTATION AT FULL PLASTIC MOMENT

M_{pe} = EFFECTIVE PLASTIC MOMENT INCLUDING FLB & WLB REDUCTIONS (AASHTO AUTOSTRESS DESIGN CRITERIA)

k = SLOPE OF UNLOADING CURVE

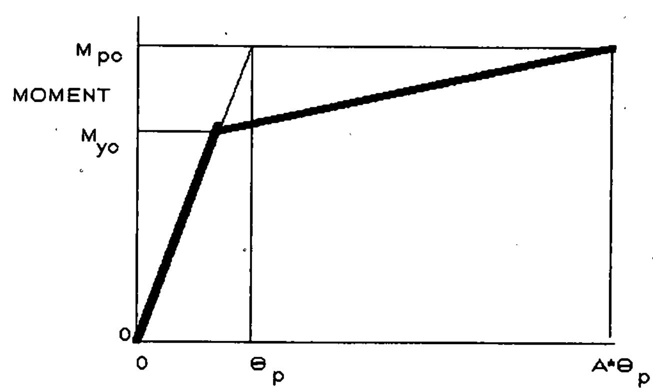
NOTES: 1) for a compact beam m=1, k=0
2) bracing spacing must obey AASHTO LFD provisions for attaining the full plastic moment

Figure 2.18. Moment-rotation curve for noncomposite noncompact section.

2.5.4 Method of Inelastic Bridge Rating

Two methods of inelastic bridge rating were developed in this project: (1) The *grid analysis,* which is applicable to complete beam-and-slab bridges with compact beams. It provides the rating factor at the shakedown limit. (2) The *residual damage analysis,* which is applicable to individual beam members of the bridge made of compact or noncompact composite or noncomposite sections. This analysis provides the residual damage for a given rating factor.

A PC-based computer program was developed to incorporate both of these inelastic rating methods. The following chapter will describe this program (applicability, use, limitation). First, however, the two proposed rating methods will be verified by comparison to independent analyses and to laboratory and field tests.



A = FACTOR DETERMINED BY ANSOURIAN FROM THEORY AND TESTS (6)

A = FUNCTION OF CONCRETE STRENGTH
STEEL YIELD STRESS
AREA OF STEEL GIRDER
DEPTH OF STEEL
DEPTH OF SLAB

Figure 2.19. Moment-rotation curve for composite section.

CHAPTER 3

INTERPRETATION, APPRAISAL, AND APPLICATION

3.1 INTRODUCTION

This chapter will further explain the inelastic rating scheme that was developed by providing backup verification—both analytical and experimental—and will conclude by developing applications of the methods. The computer program IBR will be described and then used in rating example bridges and in prescribing modifications in order to increase the rating factor of deficient bridges. Finally, a recommended addendum to the AASHTO *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* (*10*) will be presented so that inelastic bridge rating can be performed.

3.2 VERIFICATION OF THE INELASTIC RATING METHODS

3.2.1 Verification of the Grid Method

This method uses a two-dimensional grid model to represent the bridge. It is described in Appendix C of this report (and in more detail in Reference *6*). The grid method uses an elastic-plastic incremental analysis to determine the forces and deformations both under load and after the removal of the live load of two side-by-side trucks for any location. By using the elastic moment envelope and the global shakedown conditions, the method can predict the shakedown limit of the bridge system. This shakedown limit can also be predicted in a totally independent manner by formulating the equations as a linear program (*110*). Barker (*6*) demonstrated that for a three-beam two-span bridge, the two methods give identical results. In fact, if one would only wish to determine the shakedown limit, either the grillage analysis or the linear program could have been used for a rating method. The decision to use the grillage approach was made because this method gives much more information than the linear programming one, and because the approach is more familiar to bridge engineers.

A necessary test of any analytical scheme is its comparison to test results. There are not many tests available on entire bridge systems where loading is continued up to the ultimate load. There are even fewer tests where the loading is a repetitive, moving, damage-producing set of trucks. Following is a comparison of analytically determined results with test data on various aspects of the grid method. The details of the comparisons are presented in Reference *6*.

Elastic Moment Envelope. Given any regime of live loading, the grid method can determine the envelope of elastic moments, reactions, and deflections. Deflection and strain data from the model bridge test at the Turner-Fairbanks Laboratory of the Federal Highway Administration (*67*) was made available (*111*) for our use. In addition, a set of finite element predictions was also available from a study of that data (*112*). The test responses (reactions, moments, deflections) are compared to the predictions from the grid analysis and the Maryland finite element analysis in Reference *6*. A typical comparison for the moments in an exterior girder of the three-girder two-span model bridge is shown in Figure 3.1. The left figure compares the moments when the load is on the exterior girder, and the right one applies when the interior girder is loaded. The comparison is typically good, indicating that the analytical procedures model the elastic bridge properties quite adequately.

Shakedown Behavior. Because there were no tests on a bridge system subjected to repeated inelastic loading, the opportunity was seized when a model bridge at Iowa State University became available for testing. The bridge, as well as the tests which were performed on it, are described in detail in Appendix E of this report, and also in Reference *8*. The bridge was a $\frac{1}{3}$-scale model. The model had three spans (154-196-154 in.) and four beams spaced at 31 in. It was loaded with three hydraulic jacks and two concrete blocks so that the dead load and a varying live load could be simulated. Figure 3.2 shows a plot of the residual deflection in the center beams in the center of the middle span versus the load stage number. Each load stage represents a cycle of loading passing over the bridge via the hydraulic jacks. At every load level the cycling was repeated until the deflection no longer increased, i.e., the system was shaken down. The dashed line represents the experimental behavior. Four levels of loading are represented, each causing initially some inelastic action, but eventually shakedown occurred at each level, as attested by the arrests of further residual deformations. The solid curves show the predictions of the grid analysis. The lower curve is for the assumption that the slab is fully intact, while the upper curve is for the case where the slab is so damaged that it is ineffective. At the lower levels of inelastic loading the slab appears to be participating, whereas for the last level it is obvious that the slab has been severely damaged.

While the comparison between prediction and test is marred by the excessive damage of the slab, the conclusions are that (1) the bridge did shake down physically and (2) the behavior can be predicted. Further details of the test and other comparisons are provided in Appendix E and Reference *8*.

Ultimate Capacity. The Iowa State University bridge was eventually tested to failure by monotonic loading in the center

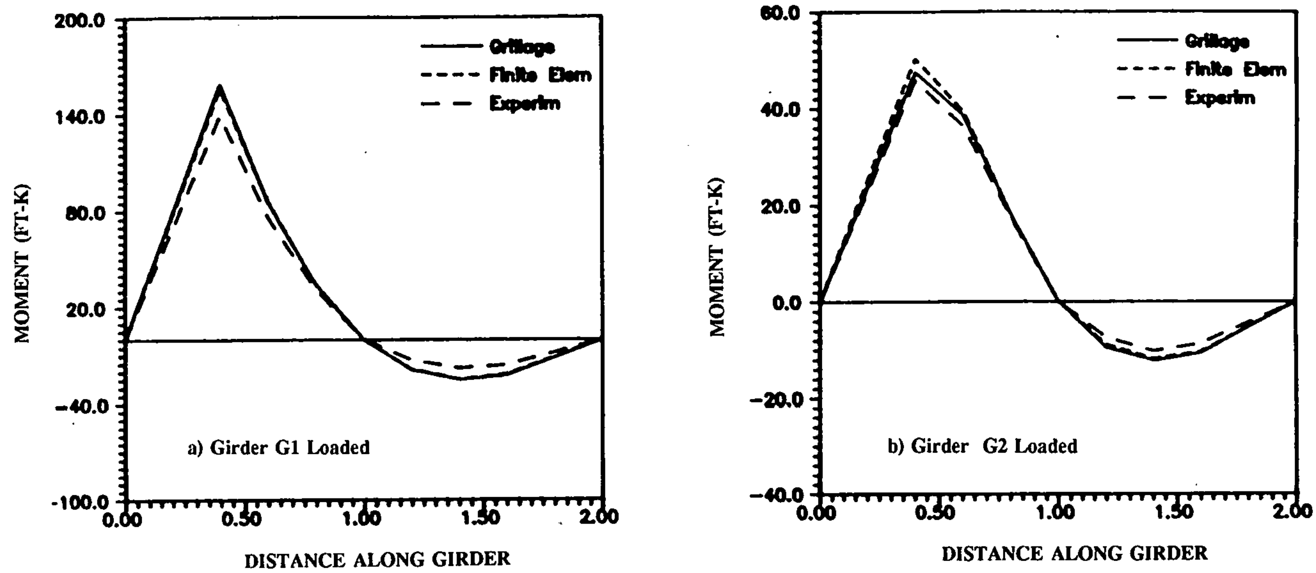


Figure 3.1. Girder G1 moment influence lines at 0.4L.

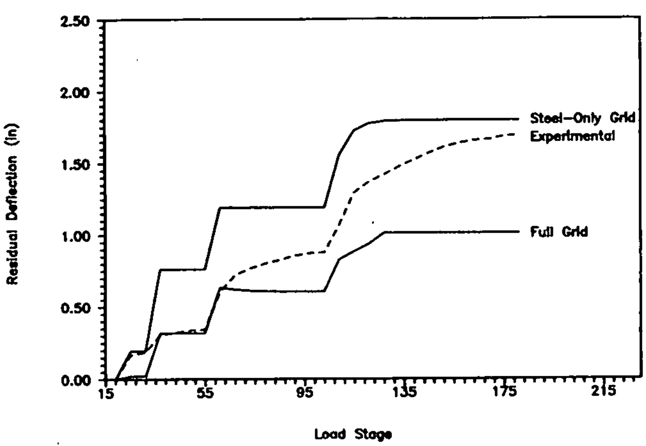


Figure 3.2. Shakedown tests of the one-third scale bridge.

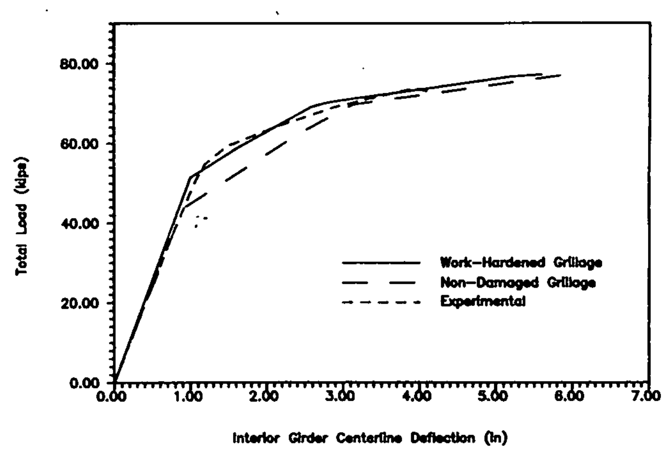


Figure 3.3. One-third scale bridge ultimate load-deflection test.

span. The comparison between the experimental load-deflection curve and the prediction by the grillage method is shown in Figure 3.3. Two analyses are shown: one which does and one which does not account for the previous inelastic load history. The former approach represents the actual behavior very well. A similar comparison of the ultimate load test of a full scale bridge test in Tennessee (*104*) also showed good comparison between theory and test.

These three comparisons of the elastic, shakedown, and ultimate behavior of multi-beam continuous beam-slab type bridges verify the application of the grid method to bridge rating (see Figure 2.17 for the relationship between analysis and test verification).

3.2.2 Verification of the Residual Damage Analysis Method

The Residual Damage Analysis (RDA) program performs an inelastic analysis of an individual bridge beam where the contribution of the system is accounted for by the use of the traditional distribution factor (*DF*). When the limit state is the termination of elastic action (e.g., when the plastic moment is reached at the location of highest moment), then the corresponding first-hinge rating factor should also be predicted by the grillage program. This is an appropriate check only since the grid model is a two-dimensional analysis. Three bridges, for which there were also totally independent rating results, are compared in Table 3.1. Two of these are bridges were specifically designed by the investigators [Galambos, Sauser (*4*)], while the last one (AASHO) was one of the test bridges in the AASHO Road Test conducted in 1957–1959 at Ottawa, Illinois. Although the comparison is not absolutely perfect, one can see that the three methods (independent rating, RDA, and grid method) are quite similar. The independent rating was performed by the project staff members (*4,7*).

Experimental verification of the RDA method is discussed in Reference 5 by evaluating tests performed on two-span com-

Table 3.1. Verification of IBR for first hinge limit state

Bridge Name	TVG	SAUSER	AASHO
Bridge Type	3-SPAN	3-SPAN	1-SPAN
AASHTO Rating Vehicle	3S2	HS20	3
Compact Section, Y/N	Y	Y	Y
Composite Section, Y/N	N	N	Y
Resistance Factor, ϕ	0.9	1.0	0.7
Impact Factor	0.2	0.27	0.1
Live Load Factor	1.65	2.17	1.80
Dead Load Factor	1.2	1.3	1.2
Rating Factors			
Independent Analysis	1.45	2.34[1]	1.27[2]
RDA	1.53	2.39	1.14[3]
GRID	1.57	2.50	....

[1] Operating Rating (4)
[2] Reference 7
[3] Maximum Elastic Moment Occurs at End of Coverplate

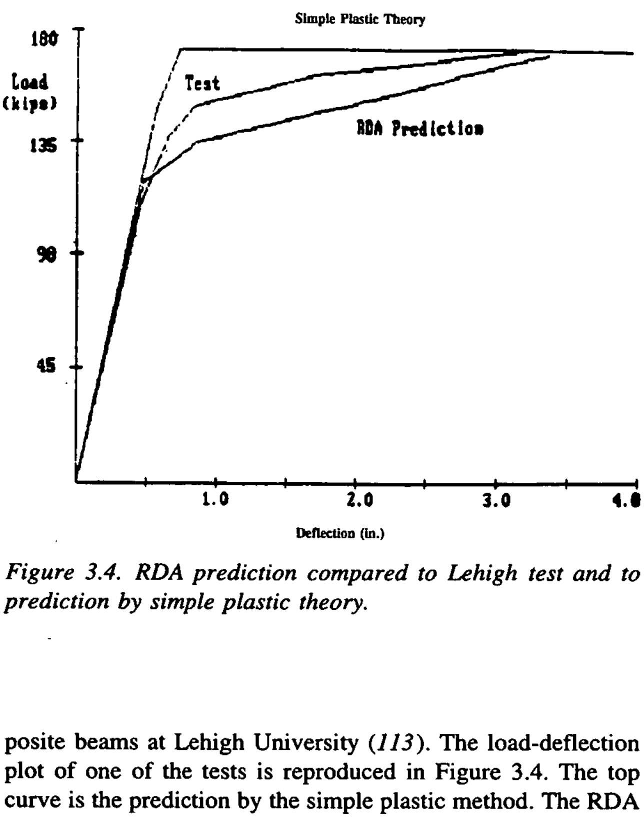


Figure 3.4. RDA prediction compared to Lehigh test and to prediction by simple plastic theory.

posite beams at Lehigh University (*113*). The load-deflection plot of one of the tests is reproduced in Figure 3.4. The top curve is the prediction by the simple plastic method. The RDA prediction is the lower curve. The experimental curve is between the two extremes. Both methods predict the ultimate strength adequately. However, in the range in which RDA operates in practice (which is the region not far above the onset of inelastic behavior) RDA is conservative.

3.3 APPLICATIONS

3.3.1 Computer Program IBR

One of the chief application features of this research project is the computer program "Inelastic Bridge Rating" (IBR). More details about its development are given in References *5* and *6* and in Appendixes B and C of this report.

The major components of the IBR program are described in the flowchart shown in Figure 3.5. It consists of essentially three parts: Input, Output, and Help Hypertext. The program is designed for use with IBM personal computers on the DOS operating system, or compatible machines.

The Help Hypertext is accessed by the F1 key of the IBM keyboard, and it puts information about the various parts of the program on the screen for the perusal of the rating engineer. It provides general comments, detailed definitions, and problem size limitations. It is not a very extensive textbook, but it gives the experienced rating engineer enough hints and instructions so as to acquaint him or her with the basic features and assumptions of the program. Any user agency can add to the help material as desired.

The program consists of a number of pop-up menus, which permit the efficient entering of the data needed to perform the analysis. The various components of the data are enumerated on the top of Figure 3.5. The load factors and the resistance factors are automatically computed in accordance with the AASHTO *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* (*10*). However, there is an opportunity to override the vehicles and factors to suit the rating engineer. Judgment must be exercised in entering data for compact members; this will be illustrated in the next section of this report.

The IBR program performs two operations: it executes the Grid Program and the RDA (Residual Damage Analysis) Program, as desired. In typical rating exercises, the RDA Program would probably be used more than the Grid Program because the former takes much less time, and it is more general in its applicability.

The Grid Program performs the following operations:

1. It calculates the moments in each longitudinal element of the bridge, giving finally an elastic moment envelope for two trucks moving side-by-side across the bridge at different transverse positions. By setting the maximum calculated elastic moment equal to ϕM_{max}, where ϕ is the resistance factor and M_{max} is the moment capacity of the critical section, the Grid First Hinge Rating Factor RF^* is calculated.
2. It determines the rating factor for a single interior beam using a Distribution Factor $DF = 1.0$ against the factored capacity ϕM_{max} of this beam. This factor is called $RF_{\text{single girder}}$. This step also determines the corresponding $RF_{\text{single girder, shakedown}}$.
3. It calculates the shakedown limit from the elastic moment envelopes and the global shakedown mechanism (see Figure 2.16). This operation also gives the residual moment field in the grid. The resulting rating factor is $RF^*_{\text{shakedown}}$.
4. The following quantities are also calculated:

(a) $RF_{NCHRP} = RF_{\text{single girder}}/DF_{AASHTO}$ (Eq. 3.1)

where DF_{AASHTO} is the AASHTO Distribution Factor used in the AASHTO design specification (*17*).

(b) $DF_{\text{elastic}} = RF_{\text{single girder}}/RF^*$ (Eq.3.2)

(c) $DF_{\text{inelastic}} = RF_{\text{single girder, shakedown}}/RF^*_{\text{shakedown}}$ (Eq.3.3)

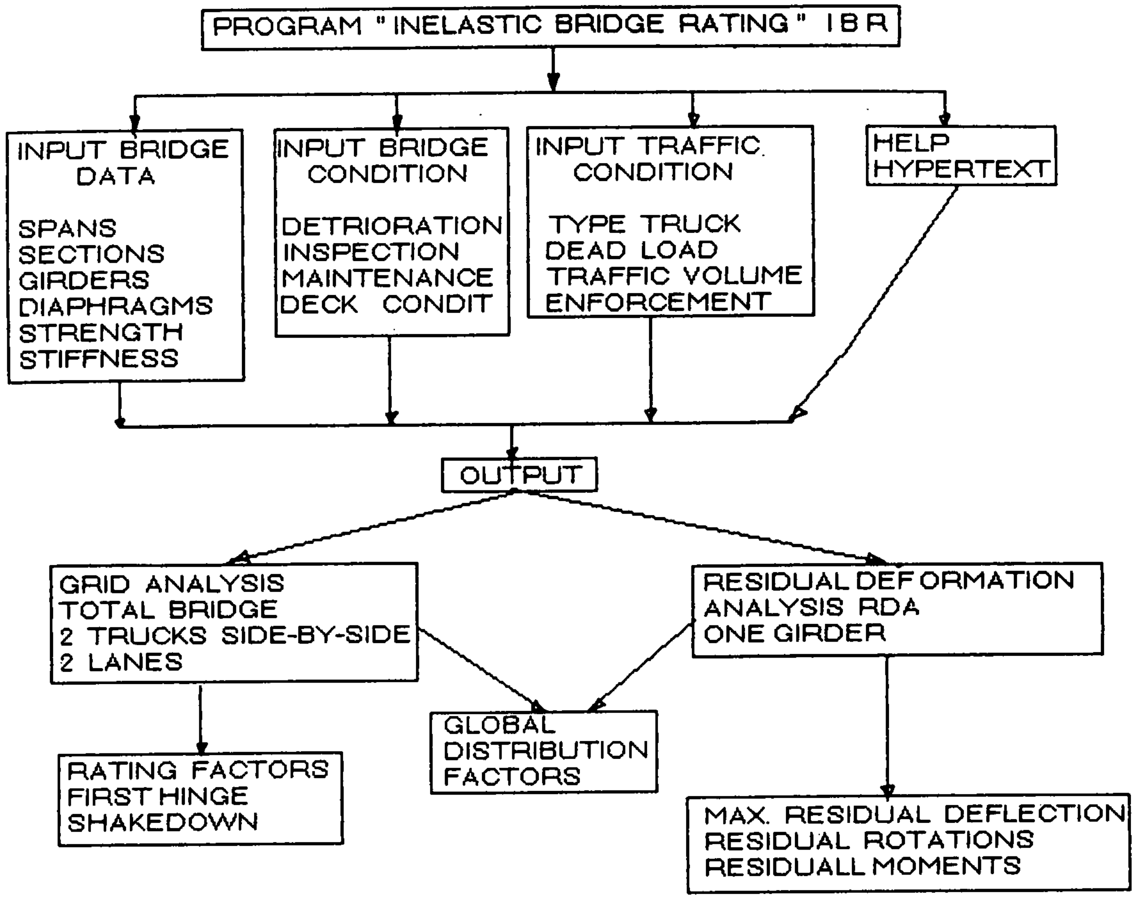


Figure 3.5. Flowchart of IBR computer program.

The quantities RF^*, $RF^*_{\text{shakedown}}$, and RF_{NCHRP} define the grid first-hinge rating factor, the grid shakedown rating factor, and the rating factor one would obtain from a traditional single-girder rating exercise, respectively. The distribution factors DF_{elastic} and $DF_{\text{inelastic}}$, respectively, are to be used in the RDA analysis, as will be explained subsequently.

The Grid Program is most suited for a bridge system composed of noncomposite compact beams. If the beams are noncompact, the default moment capacity can be altered to mM_p (see Figure 2.18) to account for the reduced moment capacity due to flange or web local buckling. In such a case, of course, the computed shakedown limit rating factor has no worth because the system cannot reach this load level. In the case of a composite cross section a similar adjustment can be made ($mM_p = M_{yc}$ in Figure 2.19). By default the Grid Program will use M_{pc} as the maximum moment capacity. In this case it is advisable to check the residual deflection by the RDA Program, as illustrated later.

The Grid Program correctly treats the bridge as a grid system. Its shortcomings are: (1) it takes a long time to run, (2) it has size limitation (e.g., a maximum of five girders for a three-span bridge—see the Help Hypertext for further such limits), and (3) it cannot really give any reliable inelastic evaluations except for noncomposite compact beams where the shakedown limit is an attainable practical rating criterion. The chief benefits of the Grid Program are the grid distribution factors DF_{elastic} and $DF_{\text{inelastic}}$ for use in the RDA program (see also Figure 2.17).

The RDA program determines the residual inelastic deformation of a bridge idealized as a single girder. The effect of the bridge as a two-dimensional grid system is accounted for by using distribution factors DF. When no analysis is made with the Grid Program, then only the standard DF_{AASHTO} is available. However, if they are available from the grid analysis it is preferable to use the factors DF_{elastic} as long as the response of the bridge is elastic, and $DF_{\text{inelastic}}$ when the bridge goes into the inelastic range, i.e., there is some residual damage. In the case that distribution factors from the Grid Program are used, the live load factor must be multiplied by 1.07 (*3*). This is unfortunately not automatic with the RDA program and must be entered into the rating equation. If the *DF*s from the Grid Program are used, the RDA program correctly analyzes an interior girder. However, if DF_{AASHTO} is used, then both the interior and the exterior girder must be checked if they are not identical.

A typical application of the RDA program is to set the rating factor equal to 1.0, choosing a suitable Distribution Factor from the three possibilities, then running the program. If the program runs through without stopping, the bridge is elastic and no damage is sustained. If the program indicates inelastic action, it has to be continued by tapping any key. It finally gives the magnitude and location of the maximum residual deflection. If this does not exceed the desired permissible value, say $L/300$ (where L is the span length in inches), then the bridge has passed the rating test.

The program can also be used to probe for the rating factor at which the first hinge forms (point mM_p in Figure 2.18 or M_{yc} in Figure 2.19), when the residual deflection is $L/600$, $L/300$, or any similar limit that the rating engineer may select.

The real workhorse of the rating exercise is thus the RDA Program. It can accommodate composite and noncomposite, and compact and noncompact cross sections. Care must be exercised if the critical section is not near the center of a span or at a support. Then it is necessary to roll through the length of the bridge with the cursor key to locate the moment which exceeds ϕM_{max}.

3.3.2 Inelastic Rating of Bridges

This report has described one possible tool to evaluate the inelastic capacity of bridges and to rate them—the computer program IBR. One part of the application package, the Grid Program, is suited for calculating the shakedown rating factor for multigirder bridges with compact noncomposite beams. The program also provides the first hinge rating factor for non-compact or composite multi-beam bridges. It outputs, of course, the value of the shakedown rating factor, but this factor should be viewed as an upper bound only. The Grid Program also calculates the Distribution Factors based on the two-dimensional idealization of the bridge for the use of the other program, called Residual Damage Analysis. This program determines the inelastic residual deformation for a bridge idealized as a single girder for a user-specified rating factor. The analysis can be performed for composite or noncomposite, compact or noncompact beams. Caution is recommended with this program if plastic moments form at the ends of coverplates not near the centers of the beams or at the supports. The program permits the search for the maximum moment check. When this happens, then the limit state is the formation of the first hinge, i.e., no inelastic reserve should be counted on.

This previously described method of rating analysis is only one of several other approaches that can be employed to utilize the inelastic reserve of steel-and-concrete slab bridges. Another simple manual procedure is to use formulas, as shown in Appendix B. The formulas are based on the same premises as the RDA method, and they can be used if only one plastic hinge forms in the bridge. Another method is the direct application of plastic analysis, as permitted in the AASHTO Alternate Load Factor Design Method (*57*).

The IBR bridge rating method is applied to the analysis of several theoretical and actual bridges in Appendix F, where sample rating exercises are detailed, and in Appendix G, which discusses bridge modification procedures that will allow a structure to receive the benefit of inelastic load distribution. In the following paragraphs, several of the rating results from Appendix F will be briefly discussed to illustrate the application of the IBR program.

Table 3.2 summarizes the rating of a bridge designed by Sauser (*4*), based on the 1941 AASHO Specification. It is evident that the bridge is conservatively designed because the rating factors are all in excess of 3.2. The first hinge rating

Table 3.2. Rating data

Bridge Name:	SAUSER (4)
Bridge Type:	3-Span, 40ft.-60ft.-40ft.
	5 Beams, Spaced 5ft.-5in.
Material:	A7 Steel, F_y = 33 ksi
Rating Truck:	AASHTO No. 3
Cross Section:	Compact, Noncomposite

ϕ = 0.95, Dead L.F.=1.2, Live L.F.=1.45, Impact Factor=0.1

RATING FACTORS:

GRID Analysis	NCHRP	3.35
	FIRST HINGE	3.37
	SHAKEDOWN	3.99
RDA, AASHTO DF	FIRST HINGE	3.22
	SPAN/DEFL=600	3.57
	SPAN/DEFL=300	3.80
RDA, GRID DF[1]	FIRST HINGE	3.24
	SPAN/DEFL=600	3.80
	SPAN/DEFL=300	4.02

Same Data as Above, Except Bridge Deck Made Composite

RATING FACTORS:

RDA, AASHTO DF	FIRST HINGE	3.50
	SPAN/DEFL=600	4.85
	SPAN/DEFL=300	5.35

[1] Live Load Factor Not Multiplied by 1.07

Table 3.3. Rating data

Bridge Name:	MN 9413 (MANKATO)
Bridge Type:	3-SPAN, 105ft.-108ft.-105ft.
	4 Girders, Spaced 10ft.
Material:	High Strength Steel
Rating Truck:	AASHTO No. 3S2
Cross Section:	Noncompact, Composite; Slab Steel in Negative Moment Region Ignored

ϕ = 0.95, Dead L.F.=1.2, Live L.F.=1.40, Impact Factor=0.1

RATING FACTORS:

RDA, AASHTO DF	FIRST HINGE	2.50
	SPAN/DEFL=600	2.90

factors are essentially comparable for all the three methods used, as are the shakedown and the RDA rating factors for a residual deflection of $L/300$. For practical purposes this bridge is rated to four times the AASHTO rating truck No. 3. This rating factor is further increased if the bridge deck is made composite with the beams.

Table 3.3 gives the rating factors of a bridge at Mankato, Minnesota. This is a noncompact, composite steel bridge, and thus the RDA method is the correct method of rating. For a residual deflection of $L/600$ the rating factor is 2.9.

Rating factors for Minnesota Bridge B9055 are given in Table 3.4. This bridge is reported to be in excellent condition, the load surveillance is vigorous and the riding surface is

Table 3.4. Rating data

Bridge Name:	MN B9055	
Bridge Type:	3-Span, 44ft.-47ft.-44ft. 5 Girders, Spaced 8ft.-6in.	
Material:	A7 Steel, F_y = 33 ksi	
Rating Truck:	AASHTO No. 3	
Cross Section:	Composite, Compact; Slab Steel in Negative Moment Region Ignored	
ϕ = 0.90, Dead L.F.=1.2, Live L.F.=1.45, Impact Factor=0.1		
RATING FACTORS:		
GRID Analysis	NCHRP	1.66
	FIRST HINGE	1.93
	SHAKEDOWN	2.27
RDA, AASHTO DF	FIRST HINGE	1.60
	SPAN/DEFL=600	1.85
	SPAN/DEFL=300	2.05
RDA, GRID DF[1]	FIRST HINGE	1.80
	SPAN/DEFL=600	2.04
	SPAN/DEFL=300	2.25

Same Data as Above, Except
ϕ = 0.7, Live L.F.=1.85, Impact Factor=0.3

For a Rating Factor of RDA = 1.0, SPAN/DEFL = 574

[1] Live Load Factor Multiplied by 1.07

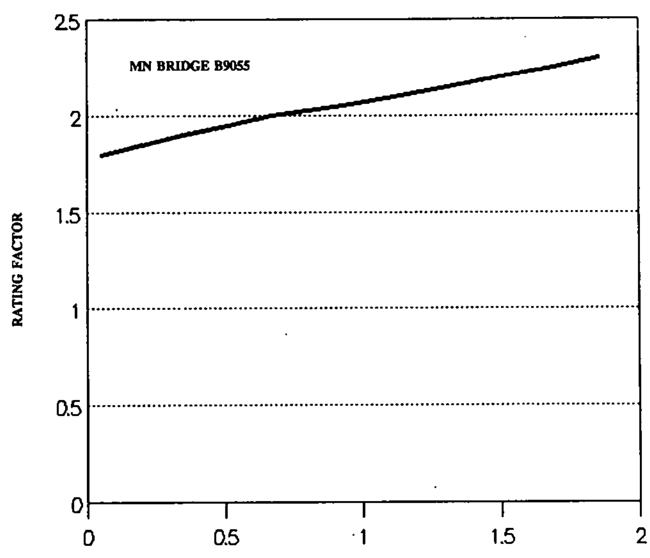


Figure 3.6. Variation of the residual deflection with the rating factor.

smooth. The applicable rating factor is 2.25. The variation of the residual deflection with the rating factor for this bridge is shown in Figure 3.6. Also shown in Table 3.4 is the hypothetical case of a situation where this bridge is in very poor shape, and its traditional first-hinge rating factor is less than unity. It can be seen that by using the RDA method, the maximum residual deflection under the factored rating vehicle is *L*/574, which is acceptable since it is less than the limit of visible deflection (*L*/300, approximately).

3.3.3 Bridge Modification Using IBR

The IBR method can be used effectively to assess the consequence of modifying the bridge to enhance its rating factor. Appendix G presents a number of ways to modify the strength of the bridge, such as making it composite when the deck is replaced, adding lateral bracing to make the bridge compact in the negative moment regions, adding stiffeners or doubler plates to strengthen the web, or strengthening the cross section by adding cover plates to the bottom flanges. The analytical procedures require that a bridge's compression flanges have adequate lateral bracing to ensure plastic redistribution. "The Resistances—Strength/Stiffness" menu of the IBR program includes an output showing the maximum distance between braced points based on the AASHTO LFD provisions. A number of case studies are presented in Appendix G illustrating the use of IBR.

3.3.4 Recommended Rating Manual

Appendix A contains a recommended addendum to the AASHTO *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* (*10*) to expand that bridge rating document to include the Inelastic Bridge Rating (IBR) method, described in the previous sections of this chapter. This addendum consists of a manual of instruction for IBR, containing formulas to apply IBR by manual calculation, and an example problem.

3.4 SUMMARY

From the example bridges that were studied in this project, it is evident that the types of bridges that were considered, i.e. multi-beam steel bridges, have a very large inelastic reserve capacity. By using only a small portion of this reserve, it may be possible to increase the load rating of a large number of these bridges. The benefit of the new recommendations is that the rating engineer not only has control over the strength of the bridge, but also over the amount of damage that can be tolerated under a rarely expected extreme overload. This is an advantage that no previous rating method possesses.

The beauty of IBR is that it provides rating engineers with the information necessary to rank their own strengthening schemes for each bridge deemed to be insufficient. The engineer may easily modify the program input to investigate the effect of modification schemes (e.g., composite vs. noncomposite decks, light-weight vs. normal-weight decks, addition of coverplates, etc.) on the rating factors. The rating engineer may then refer to *NCHRP Report 293*, "Methods of Strengthening Existing Highway Bridges," for cost information on the various strengthening schemes. Using this information, in combination with information on the site-specific user costs incurred with detours/posting/closing, rating engineers may use a cost/benefit analysis to rank order strengthening/posting/closing options for each deficient bridge in their inventory.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

A rating methodology was developed for existing bridges that incorporates some of the inelastic capacity present in most multigirder bridges. From the analysis of the data discussed in Chapter 2, the following general conclusions can be made about current rating methods:

1. Current rating procedures differ little from design provisions for new bridges. This is true not only in the U.S. and Canada, but throughout the world. Design and rating are treated as interchangeable primarily because of a lack of extensive research, until very recently, on important issues related to rating such as: (a) distribution of truck weights, extreme loads, and multipresence of trucks on a bridge; (b) simplified but accurate methods of predicting durability of the bridge components; (c) simplified analysis tools to account for deterioration; and (d) realistic lateral distribution factors. In the absence of such research, most rating agencies have preferred a rating process based on the design provisions for new bridges, which seems to be a conservative approach. However, in the past 5 years much research has taken place in the area of loading and modeling of deterioration (*3,99*) and it is now possible to formulate rating procedures that reflect the differences between rating and design.
2. An important corollary of (1.) is that rating procedures have looked at the bridge as a series of isolated longitudinal beams. This view ignores not only the redistribution capacity inherent in these structures, but casts the problem in a "member" rather than a "system" formulation. Thus the calculated failure load of the weakest member is the maximum load that can be applied to the structure. In reality as soon as one member begins to yield or fail, the load will be shed by the floor system to the adjacent, stiffer elements. This redistribution will take place without excessive deflections or permanent set, and thus the rating process should be cast in terms of a "system" rather than a "member" failure. As for the case of (1.), it is only recently that tools to account for the "system" effects in reliability-based procedures have been developed.
3. While the general principles behind the rating process are the same throughout the world, rating codes from the U.S., Canada, and Great Britain will give very different rating factors for the same bridge. The main difference is in the loads used for rating. In the AASHTO *Manual for Maintenance Inspection of Bridges,* the loads are relatively light, but load factors are higher. This is consistent with the assumption that enforcement of weight limits is strict. In Canada and Great Britain much larger (and more realistic) loads are used while the load factors are lower. This is consistent with the assumption that enforcement of weight limits is poor. The recent proposals by Verma and Moses (*3*) have shown that the rating vehicle weights should be increased, and that the load factors should account for degree of enforcement, deterioration of the members and wearing surface, and number of large trucks crossing the structure. This will reward authorities that have strict enforcement and good maintenance programs.
4. Current AASHTO rating methods are very conservative when applied to regular, straight, composite or noncomposite multigirder bridges. Much of the conservatism stems from ignoring the fact that rating and design should have different load and resistance factors because much more is known about the material properties and loading of existing bridges. Another large source of conservatism is the lateral distribution factors, which were based on elastic analysis. With the advent of personal computers, the calculation of more exact distribution factors based on a grillage analogy is a simple task. Thus techniques to calculate better load, resistance, and distribution factors should be incorporated into the rating process.
5. Experimental studies and field tests indicate that both composite and noncomposite compact beams not only exceed their plastic moment capacity when loaded to ultimate, but also show excellent ductility and rotational capacity. Thus some limited yielding in these structures should be permitted under large overloads. Even structures with noncompact sections can utilize the remaining post-buckling strength if the moment-rotation characteristics of the section are known. Formulas to calculate all pertinent values required to derive moment-rotation curves are now available.
6. Most composite bridges will show a large inelastic reserve capacity if the comparison is made between the yield and the ultimate load (i.e., between the design vehicle and the collapse load). Because of the cyclic nature of the loads applied to a bridge, an ultimate strength limit state is unconservative and should not be used to rate bridges. Rather *shakedown,* or that load causing a set of residual moments throughout the structure such that the bridge responds to subsequent loads of the same magnitude or smaller in an elastic fashion, is the correct limit state to be used when cyclic loads are present.
7. It is not possible, at this point, to make very general statements about the magnitude of rating factors based on shakedown versus those based on ultimate strength. However, all research done to this date indicates that the shakedown limit state will be reached with the formation of a global failure mechanism rather than a local one. This implies that (a) a three-dimensional structure can be simplified to a two-dimensional one without sacrificing accuracy or safety, and (b) the rating factors will probably be closer to those based on the individual

member ultimate strength rather than on the system ultimate strength.

8. Although the concept of shakedown is not familiar to most rating engineers, it is easy to grasp from the conceptual point of view. In addition, the calculation of the shakedown loads follows directly from the elastic calculations made for the analysis of the bridge, and represents very little, if any, additional effort on the part of the rating engineer.

9. Probabilistic and reliability-based procedures as applied to bridge rating and design have reached maturity, and there should not be any hesitancy to include them in codes and specifications in a more explicit form than before.

10. Computer programs to rate bridges inelastically utilizing the shakedown limit state can be developed and successfully implemented in personal computers, as evidenced by the prototype programs developed as part of this project.

The research performed under this project clearly indicates that the use of inelastic action in rating straight, composite and noncomposite multigirder bridges is justified. The permanent deformations expected are very small, less than what is visually evident, and the members possess, in general, more than adequate ductility to allow the required rotations. Rating factors utilizing inelastic action not only yield a more realistic assessment of the structure's capacity but also provide means to determine economical and reliable strengthening and repair methods.

4.2 RECOMMENDATIONS

As a result of this project, the researchers recommend that:

1. Inelastic Bridge Rating (IBR) can be adopted as an addendum to the AASHTO *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* (*10*). The methodology behind IBR was developed in this project, and the necessary analytical and experimental verification has been carried out.

2. Given the large number of bridges that will need to be rated and inspected in the future, automated methodologies need to be developed. The prototype programs written as part of this project, while limited in scope and requiring further development, prove that rating of bridges by inelastic methods can be accomplished in PC-type machines with reasonable computation time. The Hypertext technology utilized in IBR makes use of sophisticated procedures possible even for engineers not fully conversant with computers.

3. The rating process should be streamlined such that the data required to carry out the evaluation is acquired during the field inspection, and the data should be input directly into the computer database in a format accessible to programs like IBR. With the arrival of powerful laptop computers, it is possible that "field" rating of bridges (or rating done simultaneously with the inspection) will be possible within a few years. This approach will provide the opportunity for the rater to see immediately whether his/her assessment will lead to a posting or not, and if so lead him/her to suggest some possible modification schemes.

4.3 SUGGESTIONS FOR FUTURE RESEARCH

Although much progress has been made in this project toward developing a rating methodology that incorporates inelastic action, there are several areas where further developments are needed. Some of these areas include:

1. Experimental work on inelastic distribution factors under moving loads. Although strain hardening is assumed to provide a substantial safety margin over the shakedown limit calculated by the procedures proposed in this project, some experimental verification (possibly on a bridge scheduled for replacement) is needed.

2. Additional work is needed on possible local failure mechanisms, including service load criteria such as fatigue. Although most of the research indicates that the failure will occur by a global rather than a local failure, the interaction between strength and serviceability limit states needs to be explored further. For example, what is the effect of a large load (below the shakedown) on an existing fatigue crack?

3. Better quantification of impact effects. The current procedures from the U.S., Canada, and Great Britain differ significantly on this point. Research toward discovering the basic mechanistic principles involved should be carried out.

4. The methodology developed in this project should be extended to include multi-beam steel bridges, trussed bridges, and open and closed cross section bridges.

5. The computer program IBR, as presented in this report, is but a prototype of a more extensive, more robust, more efficient, and more user-friendly program that should be developed for extensive use.

6. A great deal of educational effort needs to be invested in teaching the many bridge engineers involved in rating and maintaining the nation's bridges about the rudiments and applications of inelastic behavior. Although much has been learned about bridge behavior from recent research, this knowledge needs to be transferred to the users who could benefit from it.

7. Additional analytical and experimental research on the strength and rotational ductility is needed for noncompact composite and noncomposite girders.

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APPENDIX A

PROPOSED ADDENDUM TO AASHTO *GUIDE SPECIFICATIONS FOR STRENGTH EVALUATION OF EXISTING STEEL AND CONCRETE BRIDGES, 1989*

The following comprises a proposed addendum to the AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, 1989 (6). This addendum may be included as the fourth chapter of the aforementioned AASHTO Guide Specifications, and the following sections are numbered accordingly.

This material presents the procedure that is followed for the inelastic rating of highway bridges, with emphasis on steel girder structures.

INELASTIC RATING OF PARALLEL GIRDER HIGHWAY BRIDGES

4.1 General

In addition to the potential gains that may result from the application of probabilistic load and resistance factors to the rating of highway bridges, additional benefits may be realized if the inelastic reserve strength of bridge structures is called upon to resist the factored rating vehicles. When rating highway bridges, load factors are applied to the actual truck weights to ensure that a particular structural limit state will seldom, if ever, be reached. Elastic bridge rating methods restrict factored truck loads to the maximum level at which all load-induced deflections will vanish once the load is removed, i.e., the elastic load limit. Using an inelastic rating methodology,

more liberal load allowances can be achieved by allowing a modest amount of permanent deflection to remain after the factored loads are removed. Because load factors are used, there is the same assurance that seldom, if ever, will this residual damage actually be realized.

The inelastic reserve strength of most multi-girder highway bridges can be used to obtain higher load ratings provided that (1) a resisting moment versus inelastic rotation model can be defined for every section along the longitudinal girder of the bridge, and (2) an acceptable inelastic deflection limit C, defined by the ratio of the critical span length to its midspan permanent deflection ($C = L/D$), is established.

The method to be used for the inelastic rating of multi-girder highway bridges is known as Residual Damage Analysis. This rating method combines the conjugate beam method of elastic structural analysis with a moment versus inelastic rotation model; it allows for the rating of bridge structures against an inelastic deflection limit.

4.2 Moment versus Rotation Model

The load-deformation behavior that is characteristic of a bridge girder can be represented by a moment-versus-rotation (M-θ) curve of the form shown in Figure 4.1. Such a relationship provides the moment-rotation response of a particular girder cross section as it is loaded throughout its elastic and inelastic ranges; this relationship is influenced by all of the cross-sectional parameters—material strengths, slab dimensions, girder dimensions, etc. Due to the complexity of the interaction of all such parameters, tests have been performed to determine the functional M-θ response for particular girder types.

Generalized models can be developed from the results of these moment-rotation tests (1,2). Such models may be expressed as complicated functional relationships among the many cross-sectional parameters; however, for the rating of existing bridges, it may be more desirable to employ a simple linear relationship for both the elastic and inelastic portions of the load-deformation response range. A piecewise linear function of this type is shown in Figure 4.2.

In this non-dimensionalized graph 'm' is the fraction of the full plastic moment, M_p, which can be elastically attained, and 'k' is the slope of the inelastic branch of the moment versus rotation (M-θ) relationship. θ_p is the maximum elastic rotation associated with the plastic moment, M_p. The elastic and inelastic portions of the M-θ functional model are shown in the figure, as well as the relationship between the moment (M_i) and the inelastic rotation (θ_i). The latter relates the value of the inelastic portion of the total rotation that results from a post-elastic moment.

4.2.1 Moment versus Rotation Model for Steel Sections

For steel sections alone, or for steel sections acting compositely with the slab reinforcement in the negative moment region, $m = M_n/M_p$, where M_n is the nominal flexural strength of the steel section. The

value of M_n will be governed by the most critical of the limit states of either flange local buckling (FLB), or web local buckling (WLB). Lateral bracing must be provided in accordance with the provisions of Section 10.48 of the AASHTO Guide Specification (3).

This provision states that the unbraced length Lb may not exceed the limiting value

$$L_{pb} = (3600-2200(M_1/M_p))/(F_y*r_y) \qquad \text{(Eq. A1)}$$

where L_{pb} is in inches, F_y is the yield stress in ksi, r_y is the minor axis radius of gyration of the total cross section, M_1 is the smaller moment at the end of the unbraced length, and M_p is the full plastic moment. No inelastic reserve can be counted on when L_{pb} is exceeded.

The value of M_n for both FLB and WLB limit states is:

$$M_n = M_p - (M_p - M_r)\left(\frac{L - L_p}{L_r - L_p}\right) \le M_p \qquad \text{[Eq. A2]}$$

where M_n and M_p are the nominal flexural strength and the full plastic moment, respectively. M_r is the limiting moment below which elastic buckling controls the strength. No inelastic reserve can be counted on when the slenderness parameter L_r is exceeded.

Limit State Flange Local Buckling:

Slenderness Parameter,	$L = b_f/(2*t_f)$	(Eq. A3)
	$L_p = 65/(F_y)^{0.5}$	(Eq. A4)
For Rolled Shapes,	$M_r = (F_y-10)*S_x$	(Eq. A5)
	$L_r = 141/(F_y-10)^{0.5}$	(Eq. A6)

For Welded Shapes, $M_r = (F_y-16.5)*S_x$ (Eq. A7)

$L_r = 106/(F_y-16.5)^{0.5}$ (Eq. A8)

Limit State Web Local Buckling:

Slenderness Parameter, $L = h_c/t_w$ (Eq. A9)

$L_p = 640/(F_y)^{0.5}$ (Eq. A10)

$L_r = 970/(F_y)^{0.5}$ (Eq. A11)

$M_r = F_y*S_x$ (Eq. A12)

where b_f is the flange width, t_f is the flange thickness, h_c is twice the depth of web in compression, and t_w is the web thickness. The terms (F_y-10) and $(F_y-16.5)$ represent a reduction in the nominal yield stress, F_y, due to residual stresses locked into the section at the time of manufacture. S_x is the elastic section modulus.

These parameters are applicable to rolled and welded steel shapes; they are from the <u>AISC Manual of Steel Construction Load and Resistance Factor Design Specifications</u> (see Reference 4, Appendix F), where additional criteria are also given for hybrid beams.

The slope of the inelastic branch, k, is given as:

$$K = \frac{m - m_e}{4 - m_e} \qquad \text{[Eq. A13]}$$

where m_e is given as the ratio of the effective plastic moment to the full plastic moment, $m_e = M_{pe}/M_p$. M_{pe} is computed in accordance with the provisions of Section 10.50 of the AASHTO Guide Specification (3).

$$M_{pe} = R_f*M_{pf} + R_w*M_{pw} \qquad \text{(Eq. A14)}$$

where M_{pf}, M_{pw} are the flange and web components of the full plastic moment, respectively, and

$$R_f = 0.0845\ (E/F_y)\ (t_f/2b_f)^2 \le 1$$

$$R_w = 1.32*(E/F_y)*(t_w/D_{cp})^2 \le 1 \qquad \text{(Eq. A15)}$$

D_{cp} is the distance to the compression flange from the neutral axis for plastic bending. E is the modulus of elasticity and F_y is the yield stress of the flange steel.

4.2.2 <u>Moment versus Rotation Model for Composite Steel Sections</u>

For composite sections in positive moment bending, $m = M_y/M_p$, which is the ratio of the composite section yield moment, M_y, to its full plastic moment, M_p, both of which can be computed in accordance with AASHTO Specifications (see Reference 7, Chapter 10, Part D).

The inelastic reserve of a composite beam cross section can be counted on provided that the beam cross section has a ductility factor, X, greater than unity (1,5), where

$$X = \frac{0.17 * f'_c * A_g}{F_y * A_s} \qquad \text{[Eq. A16]}$$

When the ductility factor for a composite beam section is less than unity, no inelastic rotation is allowed.

If the ductility factor for a composite beam section is at least 1.0, then the section will reach its full plastic moment capacity, M_p, and the inelastic to maximum elastic rotation ratio is

$$A = \frac{0.41 * f'_c * A_g}{F_y * A_s} - 1.6 \qquad \text{[Eq. A17]}$$

In these expressions: f'_c = concrete compressive strength (ksi); F_y = steel girder yield strength (ksi); A_s = cross section area of steel girder (in^2); and, A_g = gross area of the composite section = effective slab width times overall depth, i.e., the slab thickness plus depth of steel beam, (in^2).

The factor, A, given here is for beams loaded with concentrated loads only; for typically loaded bridge spans, where the moment gradients are more uniformly distributed across the midspan regions, the factor, A, should be increased by 1.6.

The slope of the inelastic branch will be given by:

$$k = \frac{1 - m}{1 + A - m} \qquad \text{[Eq. A18]}$$

4.3 Residual Deflection Limit

The inelastic rating serviceability limit state (acceptable level of permanent deformation) must be established by the governing bridge rating authority. This limit is expressed as the minimum allowable span length to permanent midspan deflection ratio, or C = L/D.

In most cases, the value of C should be limited to a value not less than 300 — the limit of visible detection by the human eye.

4.4 Residual Damage Analysis

Residual Damage Analysis (RDA) is a method for the inelastic rating of multi-girder bridges (5). RDA is based on an elastic structural analysis technique familiar to structural engineers — the conjugate beam method. The moment-rotation (M-θ) model developed for RDA is based on research tests that determine the inelastic behavior of highway bridge girders.

With this M-θ model, RDA extends the usefulness of the conjugate beam analysis method beyond the elastic range, and into the inelastic range of structural load-deformation response.

Use of RDA to rate highway bridges against an inelastic serviceability limit state (C = L/D) provides a more meaningful way of usefully taking advantage of the inelastic reserve strength of multi-girder highway bridge structures.

Residual Damage Analysis is used to perform a single girder analysis to inelastically rate a bridge subjected to moving truck loads. With RDA, the conjugate beam is loaded with the moment diagrams of the factored dead and live loads of the actual bridge. At sections where inelastic rotations form, the moment versus inelastic rotation, M_i-θ_i, relationship shown in Figure 4.2 is invoked to solve for the additional unknown on the conjugate beam — the inelastic rotation, θ_i. The parameters, m and k, needed in the M_i-θ_i relationship are determined in accordance with the provisions of Section 4.2, above.

When multiple hinges form as the result of the moving rating truck load, a computer version of RDA must be used, since the interplay between increments of inelastic rotation and their associated residual moment field must be allowed to run its course with multiple truck passages, i.e., the bridge must be allowed to shakedown. This software is provided within this document and is called INELASTIC BRIDGE RATING (IBR).

When only one inelastic hinge rotation occurs, RDA can be used to manually rate a steel girder bridge, since, in this instance, shakedown occurs with a single pass of the load.

When manually rating against a specific level of residual damage, which is defined as the ratio of the length of span to the midspan permanent deflection, C = L/D, the following steps are followed:

(1) Determine the required value of inelastic rotation θ_i, to achieve the inelastic deflection limit, C = L/D. With this value of θ_i, determine the accompanying residual moment, M_r. This relationship is obtained using the conjugate beam "loaded" with the inelastic rotation "force," θ_i.

(2) Determine the parameters necessary to define the moment versus inelastic rotation, $M_i - \theta_i$ model.

(3) Determine the dead load moments, M_d, and the live load

elastic moment envelope, M_l.

(4) Equate θ_i of Step 1 with the expression obtained in Step 2. Solve for the hinge resisting moment, M_i.

(5) Determine the inelastic rating factor, IRF, by applying the following formula at the hinge point:

$$(IRF) * M_l + M_d + M_r = M_i \qquad \text{(Eq. A19)}$$

(6) Check other potential hinge locations to ensure that the factored loads do not form a second hinge.

4.5 <u>Illustrative Examples</u>

To demonstrate the Residual Damage Analysis procedure, a simple-span composite bridge (L=57 ft.), a three-span noncomposite bridge (L=40-75-40 ft.), and a two-span composite bridge (L=69-69 ft.) were each rated for the AASHTO 3S2 vehicle (72 kips). The two-span case required the use of the INELASTIC BRIDGE RATING software, while the former two bridges were manually rated. The inelastic deflection limit was C = L/D = 300 for each case.

Using the load and resistance rating format suggested in the AASHTO Guide Specifications (6), the elastic limit rating equation is given as:

$$F_r * R_n = F_d * D + F_l * DF * L * (1+I) * RF \qquad \text{(Eq. A20)}$$

or,

$$RF = \frac{F_r * R_n - F_d * D}{F_l * L * DF * (1+I)} \qquad \text{[Eq. A21]}$$

where, RF = rating factor
F_r = resistance factor
R_n = nominal strength or resistance
F_d = dead load factor
M_d = nominal dead load moment
F_l = live load factor
M_l = nominal live load moment (truck wheel line)
DF = lateral load distribution factor
I = impact factor

For the three examples, the following load and resistance factors were assumed: no deterioration with good inspection and maintenance (F_r=0.95); smooth deck (I=0.1); heavy volume roadway (average daily truck traffic greater than 1000) and reasonable enforcement in the control of overloads (F_l=1.65); no asphalt overlay (F_d=1.2); and the AASHTO lateral load distribution factor was used to give DF=7.0/5.5=1.27.

For all examples, a W21X101 steel girder was used; F_y=36 ksi. The slab thickness was 7 in.; f'_c=4 ksi. Within the effective slab width, 6 sq. in. of longitudinal rebars were contained; F_y=60 ksi. Girders were spaced 7 ft. o.c. The uniform dead load was 0.75 klf. Section properties and RDA parameters are given below for the composite and noncomposite sections:

Noncomposite:
M_p = 759 ft.-k
EI = 487,000 k-ft.^{2}
m = 1.0
k = 0

Composite (includes rebars):

Midspan:	Support:
M_p = 1397	M_p = 1033
EI = 1,411,000	EI = 688,000
m = 0.72	m = 1.0
k = 0.08	k = -0.05 (noncompact)

4.5.1 Useful Formulas Derived from Application of RDA

The permanent, inelastic midspan deflection of a beam span can be expressed as a function of the span length, L, and the inelastic rotation (angular discontinuity of the beam elastic curve), θ_i, that exists at a support or in the midspan region as a result of applied loads in excess of the elastic beam capacity.

In the case of a simple beam, for example, the permanent centerspan deflection, D, that results from an inelastic rotation at the midspan is $D=\theta_i*L/4$. This simple relationship is obtained by placing the concentrated "load," θ_i, at L/2 on the conjugate beam that corresponds to the real simple beam. The "reactions" at either end of the conjugate beam are each $\theta_i/2$, and the deflection at the centerline of the real beam is equivalent to the moment at the same point on the conjugate beam, which is $D=\theta_i/2*L/2$, or $D=\theta_i*L/4$.

Such a relationship can be useful for making quick estimates of the inelastic rotation required to achieve a specific permanent deflection limit. Let C=L/D define the span to permanent deflection ratio: if the limit of visible deflection, D=L/300, is used as an inelastic deflection limit state, then C=L/D=300, and knowing that $D=\theta_i*L/4$, we have θ_i=4/300=0.0133 radians — this is the required inelastic rotation to achieve the limit state of visible deflection at midspan of a simple beam. This approach will be useful in the inelastic rating of bridges.

In similar fashion, the conjugate beam can be used to develop formulas for multispan beams which are similar to the simple beam relationship, $\theta_i=4/C$. For example, for a three-span continuous beam whose span lengths are AL, L, and BL, the formula relating the permanent midspan deflection of the interior span that results from an inelastic rotation, θ_i, at the interior span centerline is (5):

$$\theta_i = \frac{1}{C} * \frac{A/3+1/2+(B/3+1/2)*(A+1/2)/(B+1/2)}{1/48*(A+1/2)/(B+1/2)+5/48+A/6} \qquad \text{[Eq. A22]}$$

The residual moments at the two interior supports are given as:

$$M_1=\theta_i*(EI/L)/(-A/3-1/2-(B/3+1/2)*(A+1/2)/(B+1/2)) \qquad \text{(Eq. A23)}$$

$$M_2=M_1*(A+1/2)/(B+1/2) \qquad \text{(Eq. A24)}$$

In these formulas, a negative value of θ_i corresponds with a positive moment, M_1, and vice versa.

4.5.2 Simple-Span Composite Beam

The factored dead and live load moments at the critical midspan section are $M_d=365$ and $M_l=-653$ ft.-k, respectively. Substituting these values into the elastic limit rating equation,

$$0.95*(0.72*1397) = 365 + (653)*RF$$

the elastic limit rating factor is RF=0.90.

For the C = 300 deflection limit, it was shown that the required inelastic midspan rotation is $\theta_i=0.0133$ rad. The maximum elastic

rotation, $m*\theta_p$, occurs as the moment at the critical midspan section reaches the yield limit, $m*F_r*M_p$. For simple spans, this may be approximated as the area enclosed by a parabola of height, $m*M_p/EI$, and length, L. In this case, $m*\theta_p=2/3(0.72*0.95*1397)/1411000*57$, and $\theta_p=0.036$ rad.

The resisting moment of the composite section as it undergoes the inelastic rotation, θ_i, is obtained from the $M_i-\theta_i$ relationship shown in Figure 4.2:

$$0.0133=(0.036/(0.95*1397))(1/0.08-1) * M_i$$
$$+(0.036)(0.72-0.72/0.08)$$

Thus, the factored resisting moment is, $M_i=998$ ft.-k. From the inelastic rating formula given previously,

$$IRF * (653) + 365 + 0 = 998$$

so that IRF=0.97 (O.K.).

The result is a 7% strength increase beyond the elastic limit, and the bridge will not need to be posted.

4.5.3 Three-Span Noncomposite Bridge

The negative moment regions of the interior supports are made from $F_y=50$ ksi steel and the resulting M_p was 1054 ft.-k; the section is compact. The resisting moment at the critical midspan section is $M_i=F_r*M_p=721$ ft.-k for the compact beam. The factored moments at the critical midspan section of the center span are $M_d=272$ and $M_l=549$ ft.-k; $M_d=361$ and $M_l=501$ ft.-k at the supports. The elastic rating factor based on the critical hinge at the centerline of the interior span becomes

$$RF = (721 - 272)/549 = 0.82$$

Using the aforementioned θ_i versus C relationship for the three-span bridge, and setting A=B=40/75=0.53, θ_i=6.34/C =6.34/300=0.021 rad. From this value of θ_i, the residual moments become $M_{r1}=M_{r2}=M_r=(\theta_i*EI)/(-1.35*L)$, M_r=-95 ft.-k. Substituting into the inelastic rating formula,

$$IRF * (549) + 272 - 95 = 721$$

and IRF = 0.99 (O.K.)

At the critical support section, the applied moment is 361 + (0.99) * 501 + 95 = 952 ft.-k, which is less than (0.95*1054): no second hinge forms. Therefore, in this case, the inelastic strength provides a 22% increase over the elastic limit.

4.5.4 <u>Two-Span Composite Bridge</u>

For the two-span example, all three possible hinges will form and the INELASTIC BRIDGE RATING Software is needed so that multiple truck crossings can be examined to allow shakedown of the bridge. The critical section is located 29 ft. from the right end of the bridge; the dead and live load moments at this section are M_d=347 and M_l=753 ft.-k, respectively. The elastic limit rating factor at this section is given as:

$$RF = ((0.95*0.72*1397) - 347) / 753 = 0.81$$

Shakedown occurs with an inelastic rating factor of IRF=1.0 after 4 passages of the load. After the fourth truck crossing, the final values of the three inelastic rotations are: θ_{ic1}=0.013 rad, θ_{ic2}=0.017 rad, and θ_{in}=-0.0032 rad. The residual moment at the support is M_r= -210 ft.-k.

The $M_i-\theta_i$ relationship can be used to check the last hinge to change during the shakedown (the support hinge in this case). Here, the elastic live and dead load moments are 351 and 417 ft.-k, respectively. The value of the maximum elastic rotation at the support hinge, θ_{pn}, is θ_{pn}=-0.0267 rad (occurs when the yield moment is 0.95*1033 = 981 ft.-k):

$$\begin{aligned}-0.0032 = &(-0.0267/-981)\ (1/-0.05 - 1) * M_i \\ &+ (-0.0267)\ (1 - 1/-0.05)\end{aligned}$$

From this relationship, M_i = -975 ft.-k. This is the moment that is resisting the dead, live, and residual moments as given by the inelastic rating formula:

$$(1.0)*(-351) + -417 + -210 = -978 \text{ (check)}$$

The maximum deflection occurs 29 ft. from the right; it is D=2.6 in. With the inelastic deflection limit of C=319 and a rating factor of IRF=1.0, RDA again provides a substantial increase in load rating over that given by the elastic limit — 19% in this case.

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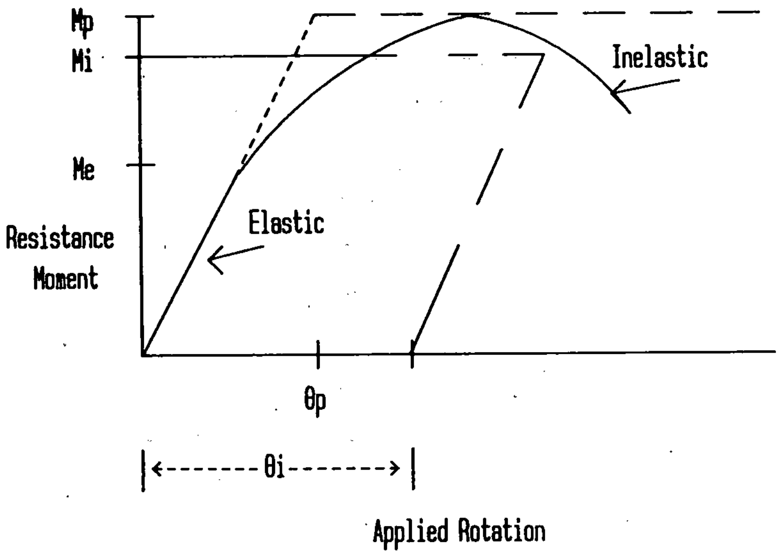
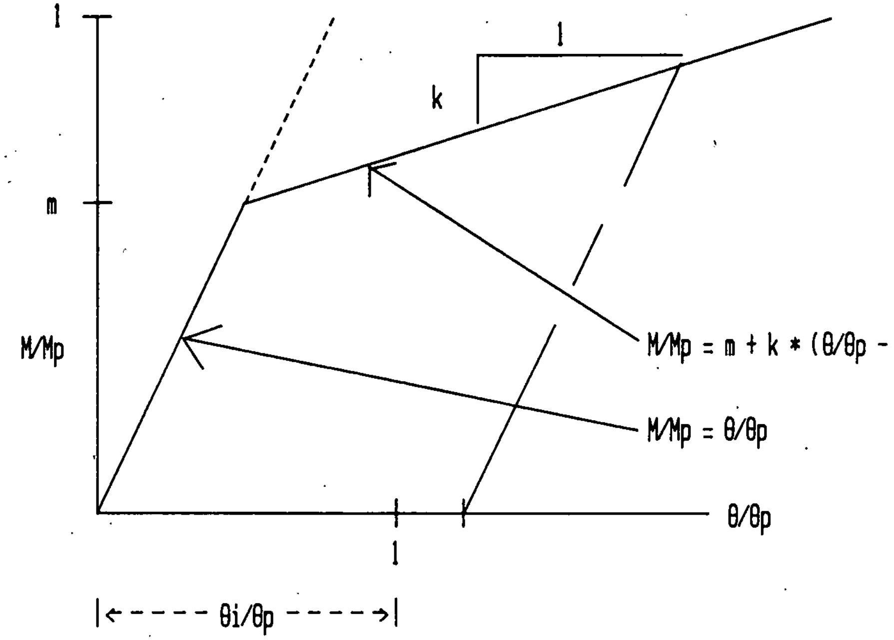


$\theta i = f\,(\,Mp\,.\,\theta p,\,Me\,)$

Figure A4.1 - Moment versus Rotation Model



$\theta i = \theta p/Mp * (1/k - 1) * Mi + \theta p * (m - m/k)$

Mi = a post - elastic resisting moment causing the inelastic component of rotation, θi

Figure A4.2 - Nondimensionalized Piecewise Linear Moment versus Rotation Model

APPENDIX B

RESIDUAL DAMAGE ANALYSIS

B.1 INTRODUCTION

In this Appendix a method is presented which allows for the inelastic rating of composite and noncomposite steel girder bridges. The method is called Residual Damage Analysis (RDA).

RDA can be used to compute higher bridge rating factors by relying on the inelastic reserve strength of steel girders, which are capable of resisting additional factored loads as they sustain acceptably low levels of inelastic deflection while subjected to these factored load levels. The low probability of actually realizing these inelastic deflection limits is ensured by the use of bridge rating load and resistance factors.

RDA uses the conjugate beam elastic analysis method combined with a moment-versus-rotation model for sections that are required to inelastically rotate. The relatively simple yet powerful analysis method of RDA provides a versatile means of inelastically rating bridges: RDA can be used to provide higher rating factors for such basic structures as simple span composite bridges, for which a single midspan hinge undergoes inelastic rotation; or, RDA can also be employed to perform a shakedown analysis in order to inelastically rate multispan, asymmetrical, noncomposite, noncompact structures for which numerous inelastic hinges rotate as multiple truck crossings occur.

A software system for the inelastic rating of steel girder bridges, INELASTIC BRIDGE RATING (IBR), is also presented. This software package is a microcomputer-based rating tool that contains the single girder analysis procedure of RDA, as well as a two-dimensional inelastic grillage analysis. IBR implements pull-down menus, pop-up data entry windows, dialogue boxes, and context-sensitive help available as part of an integral hypertext training package, to allow experienced bridge engineers, as well as new trainees, to quickly model, load, and rate existing steel girder bridges.

B.2 BACKGROUND

A new method has been developed to assess the residual moments and deformations that are set up in a beam that has been loaded into the post-elastic range. It is called Residual Damage Analysis, or RDA. RDA combines classical elastic conjugate beam theory with piecewise linear moment-rotation relationships for both midspan inelastic positive moment and interior support inelastic negative moment hinging. This method will allow bridge engineers to more easily and systematically incorporate the "automoment" concept of the Alternate Load Factor Design (ALFD) Method as presented in the ASSHTO Guide Specifications (1) into their inelastic bridge analysis. While ALFD method is useful for the design of bridges, it is not well-suited to the task of analysis (for the purpose of rating) of an existing, older bridge. Existing structures usually have asymmetric span layouts and girder cross sections that preclude the application of ALFD.

The following terms are defined for use in the RDA method:

- M - bending moment at the critical beam cross section
- M_p - plastic moment capacity of a beam section
- M_i - bending moment causing inelastic rotation at a critical beam section
- θ - rotation angle = difference between the two slopes of the elastic curve at the two moment diagram inflection points to either side of a critical beam section
- θ_e - elastic component of the rotation angle
- θ_i - inelastic component of the rotation angle (θi) max - largest value of inelastic rotation angle attained
- θ_p - elastic rotation angle associated with an assumed elasto-plastic beam section subjected to its maximum elastic bending capacity, assumed equal to the plastic moment
- θ_{ei} - slopes of the elastic curve at beam support i
- θ_{icj} - angular discontinuity at beam midspan section j that has undergone post-elastic bending
- θ_{nj} - angular discontinuity at interior beam support j that has undergone post-elastic bending
- M_{nj} - redundant bending moment at interior support j of the loaded beam
- M_{rj} - residual moment at interior support j of the unloaded beam that has been subjected to inelastic loading
- m - fraction of the full plastic moment of a beam section that can be elastically attained
- k - slope of the inelastic branch of the moment-rotation relationship of a beam section

The procedure used in applying RDA will be outlined below. Determination of the parameters necessary for forming an appropriate moment-rotation model will be covered later.

B.3 MOMENT-ROTATION MODEL FOR RDA

Kubo and Galambos (2) have shown that the moment versus rotation (M-θ) relationship for a steel or composite section in the region of a plastic hinge can be modeled as a piecewise linear function of the form (see Figure B1):

$$\frac{M}{M_p} = \frac{\theta}{\theta_p}; \quad \frac{\theta}{\theta_p} < m \qquad \text{[Eq. B1]}$$

$$\frac{M}{M_p} = m + k * \left(\frac{\theta}{\theta_p} - m\right); \quad \frac{\theta}{\theta_p} > m \qquad \text{[Eq. B2]}$$

For the negative moment bending of steel sections at interior supports, the parameters m and k respectively define: the section's capacity to develop full plastic moment, M_p, as computed using the AISC Load and Resistance Factor Design (LRFD) Specification (3); and the slope of the post-yield, or inelastic, branch of the section moment-rotation curve. A negative k value denotes softening. To achieve inelastic behavior, the positive moment regions of noncomposite sections must be fully compact, i.e., have relatively stocky webs and flanges, in which case m=1 and k=0. For positive moment bending in composite sections, m is the ratio of the yield to the plastic moment, M_y/M_p; k is the slope of the inelastic branch of the M-θ function, which will be positive for composite sections in positive bending. These two parameters must be empirically determined and will be described in detail latter in this Appendix.

At a particular beam section with a maximum moment capacity, M_p, and for which the applied moment is a local maximum or minimum value, the maximum elastic rotation, θ_p, is defined as the total change in the slope of the beam elastic curve that occurs over the positive or negative moment region that contains the section. In this definition, the moment-rotation behavior is idealized as elastic-perfectly plastic (elasto-plastic). Thus, θ_p is the area under the M/EI curve (the beam bending moment diagram divided by the beam stiffness) in the positive or negative moment region surrounding the section at which the bending moment equals M_p; if $m*M_p$ is the maximum elastic moment capacity in this region, then the area under the M/EI diagram will equal $m*\theta_p$.

The M-θ relationships can be rearranged to give the elastic, θ_e, and the inelastic, θ_i, portions of θ as follows:

Elastic

$$\frac{M_p}{\theta_p} = \frac{M}{\theta_e} \Longrightarrow \theta_e = M * \frac{\theta_p}{M_p} \qquad \text{[Eq. B3]}$$

Total (rearranging Eq. B2):

$$\theta = \left(\frac{\theta_p}{k*M_p}\right) M + \theta_p \left(m - \frac{m}{k}\right) \qquad \text{[Eq. B4]}$$

Because $\theta = \theta_i + \theta_e$, Equations B3 and B4 can be combined to give:

$$\theta_i = \frac{\theta_p}{M_p} * \left(\frac{1}{k} - 1\right) M_i + \theta_p \left(m - \frac{m}{k}\right) \qquad \text{[Eq. B5]}$$

This M_i-θ_i relationship holds at a section only after first yielding occurs and for as long as the loads continue to increase the inelastic rotation at that section. The moment is now termed M_i to indicate that it is creating inelastic deformations. The maximum value of θ_i attained at a section is permanently kept there. Thus, as a post-yielded section begins to attract a decreasing magnitude of bending moment as the result of a moving load, i.e., as the section begins to

unload, the value of θ_i just prior to unloading equals $(\theta_i)_{max}$ and it is kept in place throughout all subsequent loading; however, it is allowed to increase in response to subsequent reloadings.

B.4 THE CONJUGATE BRIDGE BEAM

The conjugate beam method is used to solve the set of unknown moments and rotations in a bridge girder. Most elementary structural analysis texts present this method. The real beam and the conjugate beam (CB) for a three-span bridge girder subjected to uniformly distributed dead load and concentrated live load are shown as Figure B2.

The interior pin or roller supports of the real beam become internal hinges for the conjugate beam; the exterior supports are the same for both. There are unknown moments, elastic rotations, and possible inelastic rotations at each interior support; also, there are possible unknown inelastic rotations near each of the midspans. Note that rotations appear as concentrated loads on the CB. There is also an unknown elastic rotation at each of the two exterior supports. So, for a three-span bridge case, there would be a maximum possible $3*2+3+2=11$ unknowns. Two equilibrium equations are available for each span of the CB. There is also a moment-rotation relationship that holds at each interior support and at the location of each midspan hinge. So, there are $3*2+3+2=11$ equations available for a three-span beam, enough to solve for all unknowns.

The unknowns are placed on the CB according to their positive sign convention as shown in Figure B3. The unknowns are: θ_{eo}, θ_{e1}, θ_{e2}, θ_{e3}, θ_{in1}, θ_{in2}, θ_{ic1}, θ_{ic2}, θ_{ic3}, M_{n1}, and, M_{n2}. For each span there are two equilibrium equations: the sum of the vertical forces must equal zero, and the sum of the moments must be zero. For each hinge location, the moment-rotation relationship is given as either of the following:

If post-yield loading,

$$\theta = \frac{\theta_p}{M_p} * \left(\frac{1}{k} - 1\right) * M_i + \theta_p * \left(m - \frac{m}{k}\right)$$

Otherwise,

$\theta_i = (\theta_i)_{max}$ = largest value of angular discontinuity thus far attained.

B.5 AUTOMOMENTS AND RESIDUAL DEFORMATIONS

After application of loads that have caused some inelastic rotations, all loads (dead and live) are stripped from the bridge, and the CB is loaded with the previously computed $(\theta_i)_{max}$ values as depicted in Figure B4.

This new set of resulting moments and rotations comprise the automoments and residual deformations. There are six unknowns: θ_{eo}, θ_{e1}, θ_{e2}, θ_{e3}, M_{r1}, and M_{r2}. There are six available equilibrium equations (two per span). The residual deflection at any point of the real beam is computed as the moment at the same location on the CB as given by conjugate beam theory.

B.6 DEFLECTION LIMITS

AASHTO Specifications (4) give two means of limiting the

deflection of bridges: first, by imposing beam depth/span limitations (H/L), and second, by limiting the live load deflection to some divisor, C, of the span length (D=L/C).

The depth/span ratios are 1/10 for trusses, 1/25 for steel beams and girders, and a 1/30 limit is set for composite beams. The origins of the H/L ratios trace back to turn-of-the-century railroad bridge design specifications. These limits were mainly based on economic member sizing, although deflection control was also considered. These limits have changed very little over the years.

The live load deflection limitations stated as a divisor, C, of the span length (where C=L/D) were originally put in place to limit traffic induced bridge structural vibrations that were judged by pedestrians to be objectionable; these live load deflection limits were based on a limited statistical study. AASHTO currently limits the elastic live load deflection to L/900, and preferably L/1000 in urban areas (where there is a higher percentage of pedestrian traffic). By way of comparison, it has been reported that when deflections are on the order of L/300, they become visibly noticeable (5).

The AASHTO serviceability and overload limit states provide guidelines that are intended to limit recoverable and permanent deflections, respectively. The stress limits imposed by the LFD and ALFD overload limit states are the result of experimental studies of the permanent deflection observed in several AASHTO bridge tests (6), which indicated that little permanent deflection occurs when stresses are kept below these stress limits. In these tests, AASHTO officials implicitly set a permanent deflection limit of D=L/600 by imposing a one-inch restriction on the allowable permanent set for the 50 ft. (600 in.) span test bridges.

In order to develop an inelastic bridge rating methodology, it is necessary to establish a deflection limit for the permanent deflections that will result from allowing inelastic behavior (upon exceeding the assumed linear elastic limit). The dead load deflection will generally have been removed by cambering the bridge spans at the time of construction. The live load deflection that results from the factored vehicle for which the bridge is being rated by inelastic means will cause both recoverable, elastic deflection and permanent, inelastic deflection.

The proposed load and resistance rating factors developed in NCHRP Project 12-28(1) (7) have been established to provide an acceptably low probability that a structural strength limit state will be exceeded. A bridge with a rating factor (RF) of unity defines a situation in which rarely, if ever, will the factored loads exceed the factored strength limit state (first yield, for example). If this bridge was rated against the inelastic serviceability limit state, which allows specified post-yield deformation to result from localized yielding, with accompanying redistribution of moments (residual moments), a somewhat higher rating factor (inelastic rating factor, IRF) could be realized.

The redistribution of moments, or the shedding of moments away from sections of localized yielding, allows the higher loads to be imposed on the bridge, as less stressed sections of the structure are called upon to resist a greater share of the imposed loads.

For example, for a bridge subjected to a set of factored loads and being rated against the strength limit state of first yielding, if the value of RF were to be unity, then the probability of the factored loads exceeding the strength limit would be that which is implicit in the load and resistance factors. For this example, if the value of IRF based on an inelastic serviceability limit were computed to be, say 1.1, then the probability of the factored load causing permanent deformations is seen to be less than the probability of yielding.

If, on the other hand, a bridge subjected to factored loads is rated against the inelastic serviceability limit state, and IRF were unity, while the value of RF were, say 0.9, then the probability of incurring some inelastic deformation would be that which is implicit in the load and resistance factors, while the probability of reaching the first yield strength limit would be higher. However, because of the redistribution of stresses away from the sections of localized yielding to other, noncritical points, strength is not compromised -- the probability of yielding is greater than that implicit in the load factors, but adequate strength is still maintained.

The inelastic rating serviceability limit state (acceptable level of permanent deformation) must be established by the governing bridge rating authority. One such limit could be that of the limit of visible detection by the human eye (D=L/300), while the aforementioned AASHTO permanent deflection limit of L/600 could likewise serve as a recommended inelastic serviceability limit state. Once established, the inelastic serviceability limit state would govern the bridge rating factor, providing that all other strength limit states (shear, bearing, etc.), as checked by the rating equation of Reference 7, do not control.

B.7 INELASTIC ROTATION CAPACITIES

In order for substantial inelastic redistribution of moments to occur, and especially for the purpose of forming a plastic mechanism, beam cross sections must be capable of accommodating considerable inelastic rotation (plastic hinge rotation). The rotational capacity of steel and composite sections is discussed in the following sections.

B.8 STEEL SECTIONS AND THE EFFECTIVE PLASTIC MOMENT

For steel sections, the two primary factors affecting available hinge rotations are: lateral bracing, and web and compression flange slenderness ratios.

The AASHTO ALFD Specifications (Autostress Design) require that, in order to take advantage of the inelastic reserve strength of steel girders, full lateral bracing as stipulated in the AISC LRFD (3) or AASHTO LFD provisions (4) must be provided in order to preclude lateral-torsional buckling (LTB) from becoming the governing limit state. This

will also be a requirement for the inelastic rating of older bridges.

Provided that adequate lateral bracing exists, the rotation capacity of a steel section will be governed by the slenderness ratios of its plate components. For the purposes of plastic design, the <u>AISC Manual of Steel Construction</u> - Part 2 (8) requires that in order for a steel section to be adequate for use in plastic design, it must be capable of sustaining the full plastic moment, M_p, while undergoing inelastic rotations of at least three times the maximum elastic rotation (previously defined as θ_p). This will occur when the web slenderness ratio, d_w/t_w, satisfies the following:

$$\frac{d_w}{t_w} \le 412/(Fy)^{0.5} \qquad \text{[Eq. B6]}$$

The AISC Part 2 rules consider a steel flange with a 50 ksi yield stress to be compact if its flange slenderness ratio, $b_f/(2*t_f)$, is less than or equal to 7.0. For other yield strengths, this value is approximately:

$$\frac{b_f}{2*t_f} \le 7.0*(50/F_y)^{.5} = 49.5/(F_y)^{.5} \qquad \text{[Eq. B7]}$$

Here, b_f, t_f, d_w, and t_w are the flange width, flange thickness, web depth and web thickness, respectively, and F_y is the steel yield stress (ksi).

In the case of sections that are nonsymmetric, either because of unequal top and bottom flange areas, or, in the negative moment regions of composite beams, because the slab rebars act compositely with the steel girder (effectively increasing the tension flange area), the plastic neutral axis shifts towards the larger flange area. For these situations, d_w is replaced with $2*d_{wcp}$, where d_{wcp} is the total depth of the web in compression, i.e., the depth of the web from the plastic neutral axis to the inner face of the bottom, compression flange.

These limits exclude many sections of existing bridges for use in plastic design. Also, in the negative moment region of composite sections, where the plastic neutral axis moves upwards due to the slab steel acting compositely with the steel section, the resulting effective web slenderness ratio $(2*d_{wcp}/t_w)$ increases above the web slenderness of the girder alone (d_w/t_w), and will thus often disqualify a section for use in plastic design.

Instead of excluding sections from use in the maximum load limit analysis of the ALFD (Autostress) design provisions, an effective plastic moment, M_{pne}, is defined by equating actual flange and web slenderness ratios to the aforementioned AISC Part 2 limits, and solving for an "effective" flange or web yield stress that is less than or set equal to the nominal flange yield stress (9). The effective flange and web yield stresses, F_{yfe} and F_{ywe} are based on the flange nominal yield strength, F_y (ksi), and are respectively given as:

$$F_{yfe}=9800*(t_f/b_f)^2 < F_y$$

$$F_{ywe}=38300*(t_w/d_{wcp})^2 < F_y \qquad \text{(Eq. B8)}$$

Reduction factors of $R_f=F_{yfe}/F_y$ and $R_w=F_{ywe}/F_y$ are computed and the

effective plastic moment is then given as:

$$M_{pne} = R_f * M_{pnf} + R_w * M_{pnw} \qquad \text{(Eq. B9)}$$

where M_{pnf} and M_{pnw} are, respectively, the flange and web contributions to the total plastic moment, M_p.

Numerous tests of various steel cross section shapes (10,11,12,13) have provided extensive moment-rotation data. The resulting M-θ curves show that, at the computed value of M_{pne}, the softening branch of the moment-rotation curve will provide inelastic rotations of about three times the maximum elastic rotation.

B.9 COMPOSITE SECTIONS

For the positive hinge rotation capacity of composite beams, Ansourian (14) has developed a ductility factor, X, which is given by

$$X = \frac{0.72 * f'_c \; B \; e_u \; (D_s + D_c)}{F_y * A_s * (e_u + e_{sh})} \qquad \text{[Eq. B10]}$$

where $f'c$ = concrete strength

F_y = steel yield stress

B = effective slab width

D_s = depth of the steel section

D_c = slab thickness

A_s = area of steel girder cross section

e_u = concrete crushing strain

e_{sh} = steel strain at first strain hardening

This formula was developed after studying the parameters that affect the cross-sectional behavior of composite beams such as geometry, steel yield stress and concrete strength, length of steel yield plateau, strain hardening modulus, steel to concrete interface slip, and residual stresses. Using typical values of e_u=0.0035 and e_{sh}=0.012, and letting the overall beam width times depth be denoted by A_g=B*(D_s+D_c), the formula for X can be rewritten as:

$$X = \frac{0.17 * f'_c * A_g}{F_y * A_s} \qquad \text{[Eq. B11]}$$

The ductility factor represents the ratio of the limiting neutral axis depth (depth of neutral axis when the concrete strain reaches its crushing strain and the steel strain reaches its strain hardening strain simultaneously) to the neutral axis depth at failure (closely approximated as the conventional neutral axis depth at full plastic moment, M_p). Provided that a composite section possesses a ductility factor of at least unity, the section will be able to reach its full plastic moment, M_p, prior to crushing of the concrete slab. If the ductility factor is below unity, the section cannot be allowed to rotate inelastically. Its load capacity is then based on its elastic limit.

For a composite section that possesses a ductility factor, X ≥ = 1.0, the following ratio of inelastic to elastic rotation capacity (elastic rotation capacity defined previously as θ_p) was developed:

$$A = \frac{0.41 * f'_c * A_g}{F_y * A_s} - 1.6 \qquad \text{[Eq. B12]}$$

The inelastic/elastic rotation ratio, A, given above is based on mean regression lines based on the results of analytical and experimental studies performed on over sixty different composite beams loaded with single point loads. Ansourian recommends that the ratio, A, be increased if the loading results in more uniform moment gradients: for the case of bridges loaded with concentrated loads and substantial uniform dead load, the ratio should be increased by a factor of 1.6.

B.10 DETERMINATION OF PARAMETER m

The parameter, m, defines the moment limit of elastic moment-rotation behavior for the M-θ relationship used in RDA; m is the fraction of the theoretical full plastic moment, M_p, that can be elastically attained (ignoring the effect of residual stresses). For positive bending and negative bending regions, respectively, m is termed as m_c and m_n (see Figures B1, B5, and B6).

For composite sections in positive moment bending, m_c=M_y/M_p, which is the ratio of the composite section yield moment, M_y, to its full plastic moment, M_p, both of which can be computed by conventional means. The yield moment, M_y, is reached when the lower steel beam flange fibers first yield; it is computed differently depending on whether the steel girder was shored or unshored at the time of construction (16). M_p is computed by assuming a fully yielded steel cross section; the Whitney stress block (15) is used for computing the concrete slab contribution (1).

For steel sections alone, or for steel sections acting compositely with the slab reinforcement in the negative moment region, $m_n = M_n/M_p$, where M_n is the nominal flexural strength of the steel section, and is based on the AISC LRFD specifications (3) for beams found in Chapter F, "Beams and other Flexural Members." AASHTO requires that steel sections be able to reach their yield moment, M_y. In the LRFD specifications, this same criterion defines the cutoff point between beams (Chapter F) and plate girders (Chapter G); thus, Chapter F of the LRFD specifications will apply to rolled beam and transversely stiffened plate girder bridges designed according to AASHTO specifications.

The value of M_n will be governed by the most critical of the limit states of either flange local buckling (FLB), or web local buckling (WLB), as full lateral bracing must be provided. The LRFD formula for computing M_n for both FLB and WLB limit states is:

$$M_n = M_p - (M_p - M_r) * \left(\frac{L - L_p}{L_r - L_p}\right) \leq M_p \qquad \text{[Eq. B13]}$$

where M_n and M_p are the nominal flexural strength and the full plastic moment, respectively. M_r is the limiting buckling moment which is equal to $(F_y-10)*S_x$ for FLB and F_y*S_x for WLB. S_x is the section modulus of the beam section. The slenderness parameters for both limit states are as follows:

for FLB, $L=b_f/(2*t_f)$, (Eq. B14)

$L_r = 141/(F_y-10)^{0.5}$ (Eq. B15)

$Lp=65/(F_y)^{0.5}$ (Eq. B16)

for WLB, $L=h_c/t_w$ (Eq. B17)

$L_r=970/(F_y)^{0.5}$ (Eq. B18)

$L_pp=640/(F_y)^{0.5}$ (Eq. B19)

For these parameters, the flange width, flange thickness, twice the depth of web in compression, and web thickness are, respectively, b_f, t_f, h_c, and t_w. (F_y-10) represents a 10 ksi reduction in the nominal yield stress, F_y, due to residual stresses locked into the section at the time of manufacture. These parameters are applicable to rolled steel shapes; slightly different values for M_r and L_r are given for built-up sections in the LRFD specifications. In summary, then, $m_n=M_n/M_p$, where M_n is determined from Appendix F of Reference 3.

B.11 DETERMINATION OF PARAMETER k

The slope of the inelastic branch of the moment-rotation relationship used in RDA is called k_c for positive bending and k_n for negative bending. This slope will be positive for composite sections in positive moment bending, and either zero or negative for steel sections in negative bending. If steel sections in positive bending are noncomposite, the section must have flange and web slenderness ratios that meet the AISC Part 2 slenderness limits for plastic design (see above) in order to allow any inelastic behavior in the beam midspan. In this case, $m_c=1.0$ and $k_c=0$.

To determine, k_c, for composite sections, refer to Figure B5 which shows the non-dimensionalized M-θ relationship for composite sections in positive moment bending. The slope of the inelastic branch is determined as the rise over the run. The rise is given as $1-m_c$, where

m_c-M_y/M_p as given above. The run is seen as 1-m_c plus the ratio of the inelastic to the elastic rotation given by Ansourian (Eq. B12). Thus,

$$k_c = \frac{1 - m_c}{1 - m_c + A} \qquad \text{[Eq. B20]}$$

For k_n of steel sections in negative moment regions, refer to Figure B6. The rise is shown as m_{ne}-m_n, where m_n was previously defined as M_n/M_p, and m_{ne} is the ratio of the effective plastic moment to the full plastic moment, or M_{pne}/M_p. As mentioned above a steel section possesses a ratio of inelastic to elastic rotation of at least three at the effective plastic moment. Thus, the run for k_n is seen to be 3+1-m_n, or 4-m_n, and

$$k_n = \frac{m_{ne} - m_n}{4 - m_n} \qquad \text{[Eq. B21]}$$

B.12 RELATIONSHIP BETWEEN INELASTIC ROTATION AND RESIDUAL MOMENTS

The permanent, inelastic midspan deflection of a beam span can be expressed as a function of the span length, L, and the inelastic rotation (angular discontinuity of the beam elastic curve), θ_i, that exists at a support or in the midspan region as a result of applied loads in excess of the elastic beam capacity. In the case of a simple beam, for example, the permanent centerspan deflection, D, that results from an inelastic rotation at the midspan is $D=(\theta_i/2)*(L/2)$, or $D=\theta*L/4$. This simple relationship is obtained by placing the concentrated "load" (θ_i) at L/2 on the conjugate beam that corresponds to the real simple

beam. The "reactions" at either end of the conjugate beam are each $\theta_i/2$, and the deflection at the centerline of the real beam is equivalent to the moment at the same point in the conjugate beam, or $D=\theta_i*L/4$.

Such a relationship can be useful for making quick estimates of the inelastic rotation required to achieve a specific permanent deflection limit. Let C=L/D define the span to permanent deflection ratio: if the limit of visible deflection, D=L/300, is used as an inelastic deflection limit state, then C=L/D=300, and knowing that $D=\theta_i*L/4$, we have $\theta_i=4/300=0.0133$ radians. This is the required inelastic rotation to achieve the limit state of visible deflection at midspan of a simple beam. This approach will be useful in the inelastic rating of bridges.

The RDA method of computing the residual moments in continuous beams by applying inelastic rotations (in the form of concentrated angular discontinuities) to the corresponding conjugate beam can be used to develop formulas for multispan beams which are similar to the simple beam relationship, $\theta_i=4/C$. For example, for a three-span continuous beam with span lengths AL, L, and BL, the formula relating the permanent midspan deflection of the exterior span that results from an inelastic rotation, θ_i, at the first interior support is developed in Figure B7. The inelastic rotation and residual moments are shown in the positive mathematical sense. A typical support hinge to negative bending creates an angular discontinuity which will actually be a downward directed arrow on the conjugate beam, although the residual moment that corresponds to this negative support angular discontinuity will be positive. Noting that AL/D would be the deflection limit, C, this formula can be written as:

$$\theta_i = \frac{16/A*(A/3+1/2-(1/4+B/6)/(B+1))}{C} \qquad \text{[Eq. B22]}$$

In similar fashion, the θ_i vs. C relationship for the inelastic rotation at the first interior support and permanent deflection at the midspan of the interior span (here, C=L/D) is:

$$\theta_i = \frac{1}{C} * \frac{(A/3+1/2-(1/4+B/6)/(B+1))}{((5/96+B/12)/(B+1))-1/48} \qquad \text{[Eq. B23]}$$

For midspan hinge (due to positive inelastic bending) in the exterior span of a three-span beam, the span to midspan permanent deflection ratio is, C=AL/D, and:

$$\theta_i = \frac{1}{C} * \frac{32/A*(A/3+1/2-(1/4+B/6)/(B+1))}{8/A*(A/3+1/2-(1/4+B/6)/(B+1)) - 1} \qquad \text{[Eq. B24]}$$

For a midspan hinge in the interior span and midspan permanent deflection of the interior span (C=L/D) we get:

$$\theta_i = \frac{1}{C} * \frac{A/3+1/2+(B/3+1/2)*(A+1/2)/(B+1/2)}{1/48*(A+1/2)/(B+1/2)+5/48+A/6} \qquad \text{[Eq. B25]}$$

To demonstrate these relationships, for a three-span beam with spans of 0.7L, L, and 0.7L, θ_i, as given by Formulas B22-B25 is, respectively, 11.83/C, 11.72/C, 4.81/C, and 6.07/C.

In the general case where the midspan hinge of an exterior span is

not at the centerline of the span, the following formula can be used to relate the near midspan angular discontinuity, θ_i, located at some fraction, X, from the outer end of the span AL, to the ratio of the span to the permanent deflection (C=AL/D), where the deflection is at the hinge point:

$$\theta_i = \frac{1}{C} * \frac{1}{X-X^2-(A/6)*(X^2-X^4)/(A/3+1/2-(B/6+1/4)/(B+1))} \qquad \text{[Eq. B26]}$$

A quick check of Formula B26 reveals that, for X=0.5, and A=B=0.7, θ_i=4.81/C. When X=0.4, this becomes 5.08/C.

For a two-span continuous beam with span lengths of AL and L, the formulas above can be applied if the span, BL is made very long in relation to AL and L: this can be done by setting B=1000. For example, consider a symmetrical two-span beam with a hinge at the support. Here, A=1 and B=1000 and from Formulas B22 or B23, we get the relationship between the inelastic rotation and the span length to midspan permanent deflection ratio to be θ=10.66/C. Likewise, for a midspan angular discontinuity and midspan deflection, Formula B24,B25 or B26 gives θ_i=4.92/C.

In addition to the θ_i vs. C relationships above, it is useful to develop relationships between the residual moments at the interior supports as a function of EI/L and θ_i. For the case of a hinge at an interior support, these relationships for the support residual moments, M_1 and M_2, are (see Figure B7):

$$M_1 = \theta_i*(EI/L)/(-A/3-1/2+(B/6+1/4)/(B+1)) \qquad \text{(Eq. B27)}$$

$$M_2=-M_1/(2*B+2) \quad \text{(Eq. B28)}$$

For a hinge in the midspan region of the exterior span located a distance X*AL from the outer end of the span:

$$M_1= \theta_i*(X*EI/L)/(-A/3-1/2+(B/6+1/4)/(B+1)) \quad \text{(Eq. B29)}$$

$$M_2=-M_1/(2*B+2) \quad \text{(Eq. B30)}$$

Also, for an angular discontinuity in the middle of the interior span, these relationships become:

$$M_1=\theta_i*(EI/L)/(-A/3-1/2-(B/3+1/2)*(A+1/2)/(B+1/2)) \quad \text{(Eq. B31)}$$

$$M_2=M_1*(A+1/2)/(B+1/2) \quad \text{(Eq. B32)}$$

In these formulas, a negative value of θ_i corresponds with a positive moment, M_1, and vice versa.

Formulas 22 through 32 were developed to analyze multispan prismatic beams subjected to an inelastic rotation at a single section of the girder. These formulas can also be used to analyze nonprismatic beams when the location of the plastic hinge occurs at a section of lower stiffness relative to the surrounding portions of the beam (at the midspan region of the noncomposite girder with cover plates at the interior supports, for instance, or at the interior supports of composite girders, where the cracked concrete due to negative bending creates a reduced moment of inertia compared to that of the midspan section). For these cases, Formulas 22 through 32 can be used to conservatively analyze continuous-span beams using an equivalent moment of inertia based on the average of the computed elastic stiffnesses for each span. For more complicated girder geometries, the computer program, IBR, should be used to perform the inelastic analysis.

B.13 BRIDGE RATING EXAMPLE WITH RDA

The relationships developed above will be used in this section to manually rate a three-span, noncomposite structure, with span lengths of 45-60-45 ft. The longitudinal beams are spaced 7 ft. 4 in. o.c.

The three-span beam consists of a W27X102 section, except at the interior support regions, where a W27X94 section with 10 in. X 3/4 in. cover plates is used. The steel yield strength is 40 ksi; the moment of inertia and ultimate moment capacity of the W27X102 section are 3620 $in.^4$ and 1017 ft.-kips, respectively. The moment of inertia at and ultimate moment capacity at the interior supports is 6110 $in.^4$, and 1610 ft.-kips, respectively.

Formulas 22-32, above, are for prismatic girders, while this girder is nonprismatic. Therefore, for the following analysis, an equivalent uniform moment of inertia is computed for each nonprismatic span of the continuous girder. The average of these three values gives the equivalent prismatic moment of inertia for the girder as 1.17 times the midspan moment of inertia, or $1.17*3620=4240$ in^4.

The manual RDA bridge rating procedure will be applicable whenever <u>only one inelastic hinge</u> is allowed. The elastic live load moment diagram (or moment envelopes) along with the dead load moment diagram must first be obtained, then, based on an accepted span to inelastic deflection limit, C, the applicable Formulas from 11 through 18 are applied to compute a residual moment, and the following rating formula can be applied:

$$IRF \quad M_l + M_r + M_d = M_i \qquad \text{[Eq. B33]}$$

Here M_l, M_d, and M_r are the live, dead, and residual moments described above; M_l and M_d are factored by the rating factors presented in the Final Report of NCHRP Project 12-28(1). IRF is the inelastic rating factor, or multiple of the live load needed to reach the inelastic deflection limit state, and M_i is the factored resisting moment that is associated with the inelastic rotation, θ_i, and which comes from application of Formula B5.

In this example, all sections are compact, so that $m_c=m_n=1$ and $k_c=k_n=0$. The bridge will be rated for the AASHTO 3S2 truck (7) the critical rating vehicle for this bridge configuration, and an inelastic deflection limit of C=600. The live load moment envelope, the dead load moment diagram, and the residual moment diagram for an inelastic rotation of θ_i=6/600=0.010 radians at the critical section (middle of interior span) are presented in Figure B8.

In order to apply the rating factors of NCHRP 12-28(1), the following assumptions will be made: low volume roadway (average daily truck traffic less than 1000) with significant sources of overloaded vehicles (F_l=1.65); rough deck and approaches (I=0.2); no asphalt overlay (F_d=1.2); adequate inspections reveal slight deterioration with some section loss due to corrosion (F_r=0.85); and the AASHTO live load distribution factor, DF=S/5.5=7.33/5.5 or DF=1.33. From these assumptions, the live and dead load moments at the midspan section are

(see Figure B8) 462 and 148 ft.-kips, respectively.

The resistance moment is M_i=0.85*M_p=864 ft.-kips. The residual moment that will result from the inelastic rotation of θ_i=0.010 radians can be obtained from Formula 31 using A=B=0.75, so that M_r=θ_i*EI/(-1.5*L), which, for θ_i=0.010 radians and EI=853500 k-ft.^{2}, gives M_r=-95 ft.-kips. Thus, use of Formula 33 gives (IRF)*(462)-95+148=864, from which IRF=1.76.

A quick check at the left interior support shows that the loading is 1.76*(401)+95+257=1056, which is less than the resistance, 0.85*1610. At the critical section of the positive moment region of the left exterior span, the loading is 1.76*401-40+114=778, which is less than 0.85*1017. Therefore, with the IRF of 1.76, only one hinge forms in the girder, at the midspan.

If no inelastic action were allowed, i.e., if a first yield limit state (elastic limit) were used, the rating factor would be given from the elastic rating formula, (RF)*M_l+M_d=M_e, where M_e is the elastic moment resistance. From this relationship, we have RF=(864-148)/462=1.55. Thus, the inelastic rating based on a limit state of C=600 provides a (1.76/1.55-1)*100%=14% increase in load capacity over the elastic limit.

B.14 INELASTIC BRIDGE RATING SOFTWARE

As stated in the Interim Report of Project NCHRP 12-28(12), "Inelastic Rating Procedures for Steel Beam and Girder Bridges," the emphasis of the project is to take the results of previous and currently ongoing research, and to "creatively and imaginatively" combine them to produce rating tools that extend current bridge rating methods (17). To this end, the research work herein has been, in addition to developing an inelastic rating method (RDA), to develop a software package that facilitates the rating engineers by: aiding them in making correct judgements about the input data of the bridge being rated; performing inelastic rating analysis of the bridge; providing guidance through the steps of the rating process; and interpreting the data that results from the rating exercise.

This software consists of easy-to-use, personal computer (PC) based, inelastic structural analysis routines, as well as a component for helping to train engineers to use these routines as rating tools. This software package is called INELASTIC BRIDGE RATING.

The framework for this package consists of a windowing interface with drop-down menus and context sensitive access to on-line hypertext help/training information. The inelastic bridge analysis subprograms consist of a two-dimensional grid analysis routine, and the one-dimensional RDA routine described previously. The reliability-based load and resistance factors for bridge rating recently developed by Moses (7) have been included as part of the package.

In order that this software be transferable to as many users as

possible (within the civil engineering university community and in industry), the software was required to run under the PC/MS-DOS operating system on an IBM-PC (or compatible) machine with 512 kilobytes of memory and dual floppy disk drives.

The look and feel of the software functions have, as closely as possible, been modeled after the current state-of-the-art in microcomputer applications user-interfaces. This implies data input masking, error checking, windows, drop-down menus, easy disk access from within the program, context-sensitive help, etc. Every menu choice form within the program is hot-linked to a hypertext node. Thus, with the press of the F1-help key from any point in the main program, the user will "jump" into the hypertext in a context-sensitive manner. From there, any trail of associative links may be followed; the links are two-way, and a record is kept of which links were pursued -- this allows for backtracking. Once done with following a trail of links, the user can simply jump back into the main program with the touch of a key.

The program uses pull-down menus to access pop-up data entry windows that contain masked fields for quick, accurate data input. This data includes the overall geometry of the bridge, as well as that of its superstructure components, material properties, the level of deterioration, and the loads to be carried.

To evaluate the software package, INELASTIC BRIDGE RATING, opinions were obtained from bridge engineers from academia, industry, and government agencies. The following is a list of those agencies that were contacted:

- Howard, needles, Tammen, and Bergendoff Architects, Engineers, and Planners
- Bakke, Kopp, Ballou, and McFarlin, Inc. Professional Engineers
- Minnesota Department of Transportation
- U.S. Army Corps of Engineers
- University of Texas at Austin
- American Institute of Steel Construction
- Members of the NCHRP Project 12-28(12) Evaluation Panel

Engineers from these agencies were contacted either in person, or by mail. Copies of the software, a user's guide, and an evaluation questionnaire were provided to the evaluators.

As a result of the software evaluation, two primary observations can be made:

First of all, the use of microcomputers by bridge engineers is not as common as was expected. In several cases, the evaluation was conducted on a computer that was not dedicated to the bridge engineer's personal use, and the engineers were most often unfamiliar with basic microcomputer terminology and functions (e.g., hypertext, context-sensitive help, what graphics capability does the machine have, how to create a subdirectory, etc.). In one instance, the evaluator did not

have a microcomputer, and furthermore, stated that he was not able to locate one! For the few instances when the engineer was accustomed to using the microcomputer, the evaluator reacted very favorably to the modern, windowing interface, and the context-sensitive help system -- both of which are software features common to today's commercial programs.

Secondly, aside form a basic understanding of the plastic collapse load of structures, the majority of bridge engineers are not familiar with inelastic structural analysis techniques. In most cases, before any sense could be made of the capabilities of the software, it was first necessary to explain such terminology as structural limit states, shakedown, autostress design, etc. This is very understandable, since the AASHTO Specifications of the past did not address this subject matter.

In general, the evaluators were favorably impressed with the software package. However, the aforementioned observations generally limited them in their ability to provide a critique of the system's performance or to provide recommendations for future enhancements to such a rating/training package.

The evaluation process clearly revealed the need for (1) increased microcomputer literacy within the bridge engineering community; and (2) the need to mount a major educational effort within this community, in order to inform bridge engineers about inelastic analysis methodology.

These two situations must be properly addressed if any benefits are to be gained by using microcomputer analysis routines to take advantage of the inelastic reserve strength of steel girder bridges.

B.15 Conclusion

RDA is a flexible, straightforward analysis tool that provides a means of simple manual rating of structures for which one hinge must rotate considerably before a second hinge forms. In the case of continuous-span structures for which several hinges form at about the same load level, the computer implementation of RDA contained in the software package, INELASTIC BRIDGE RATING, can be used to provide a suitable means of analysis.

Inelastic strength limit states of shakedown or plastic mechanism formation are often defined for analysis of steel girder structures; these provide the engineer with substantially higher load carrying factors as compared with the elastic strength limit of first yield. However, the level of permanent deformation associated with these inelastic strength limit loads is considerable, and will usually be in excess of an acceptable level of serviceability (L/D of 300 or more). By application of RDA, and by establishing an inelastic serviceability limit state defined by limiting permanent set under factored live load to some portion of the span length, or C=L/D, a rationale now exists that will allow bridge engineers to take advantage of the inelastic reserve strength of steel girder bridges.

To implement RDA, it was necessary to develop a method of computing the cross-sectional parameters (termed m and k) for use in defining the moment vs. rotation model. The method established herein is a direct application of recently conducted research into the load-deformation behavior of post-yielded structural components, namely, the composite beam research of Ansourian (14) and the moment-rotation tests (9,10,11,12,13) required to develop the Alternate Load Factor (Autostress) Design Method (1). As a result, RDA serves as a means to apply research findings to the task of extending the load-carrying capacity of steel girder highway bridges by means of inelastic bridge rating.

Combined with the proposed bridge rating factors of NCHRP Project 12-28(1) (7), an inelastic bridge rating method using RDA and an inelastic serviceability limit state provides bridge engineers with a means of realizing higher load rating factors for steel girder bridges, while maintaining the same acceptably low probability of exceeding the given limit state that is implicit in the NCHRP factors.

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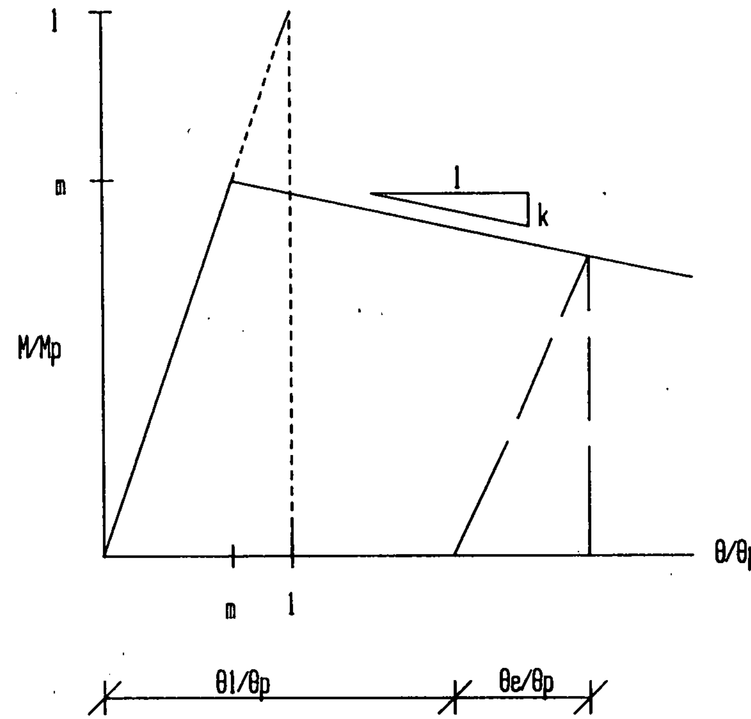


Figure B1 - Moment-Rotation Model for RDA

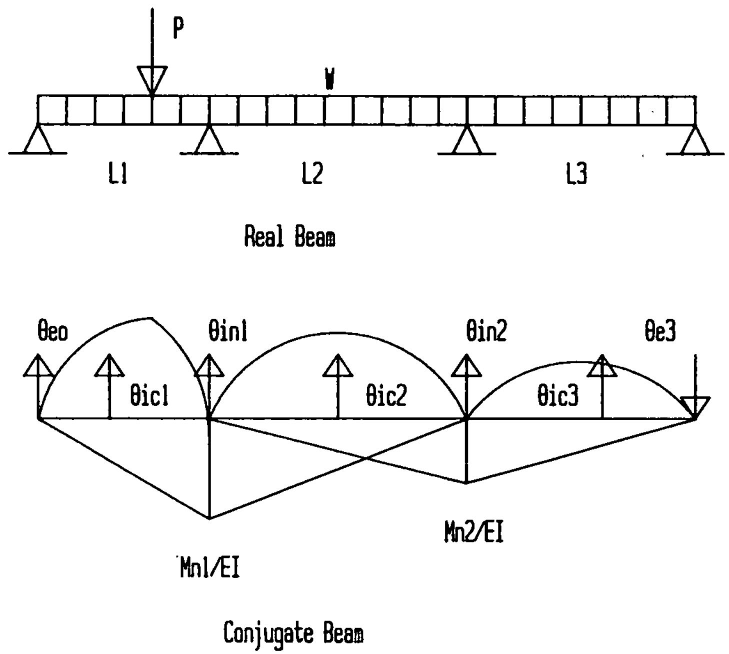
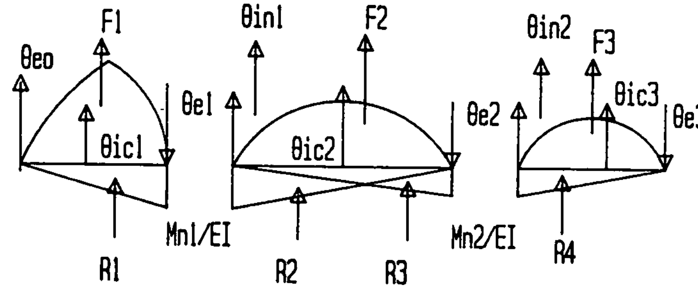


Figure B2 - Real and Conjugate Three Span Beam



$$F1 = \frac{P*L1^2}{KEI} + \frac{W*L1^3}{12EI} \qquad (K>=8)$$

$$F2 = \frac{W*L2^3}{12EI}$$

$$F3 = \frac{W*L3^3}{12EI}$$

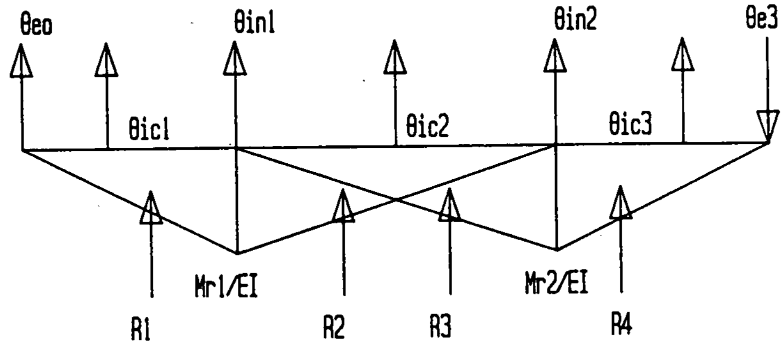
$$R1 = \frac{Mn1*L1}{2Ei}$$

$$R2 = \frac{Mn1*L2}{2EI}$$

$$R3 = \frac{Mn2*L2}{2EI}$$

$$R4 = \frac{Mn2*L3}{2EI}$$

Figure B3 - Positive Orientation of Conjugate Beam "Forces"



$$R1 = \frac{Mr1*L1}{2EI}$$

$$R2 = \frac{Mr1*L2}{2EI}$$

$$R3 = \frac{Mr2*L2}{2EI}$$

$$R4 = \frac{Mr2*L3}{2EI}$$

Figure B4 - Conjugate Beam Loaded with Residual Moments and inelastic Rotations

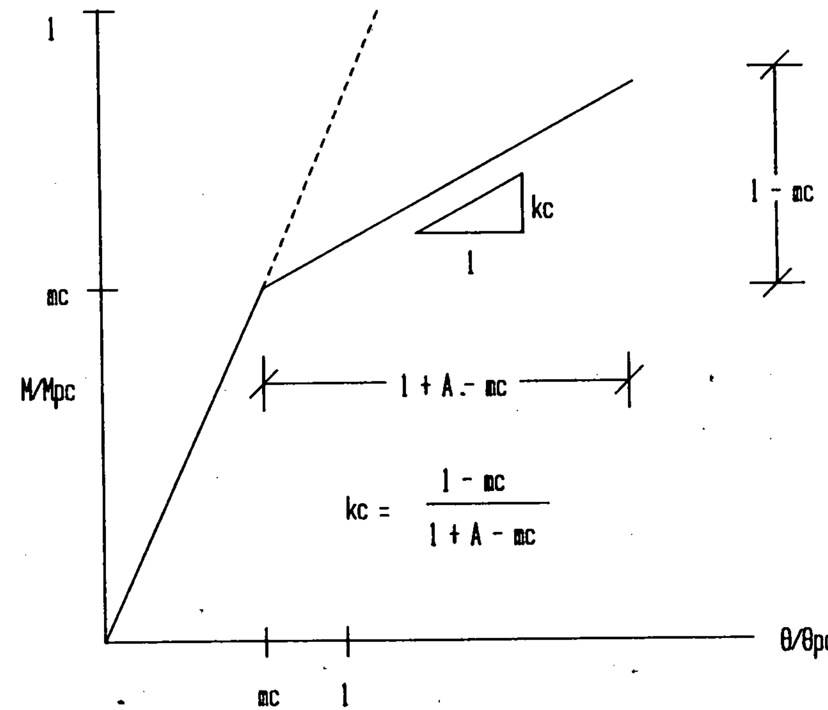


Figure B5 - Moment vs. Rotation and Midspan of Composite Beam

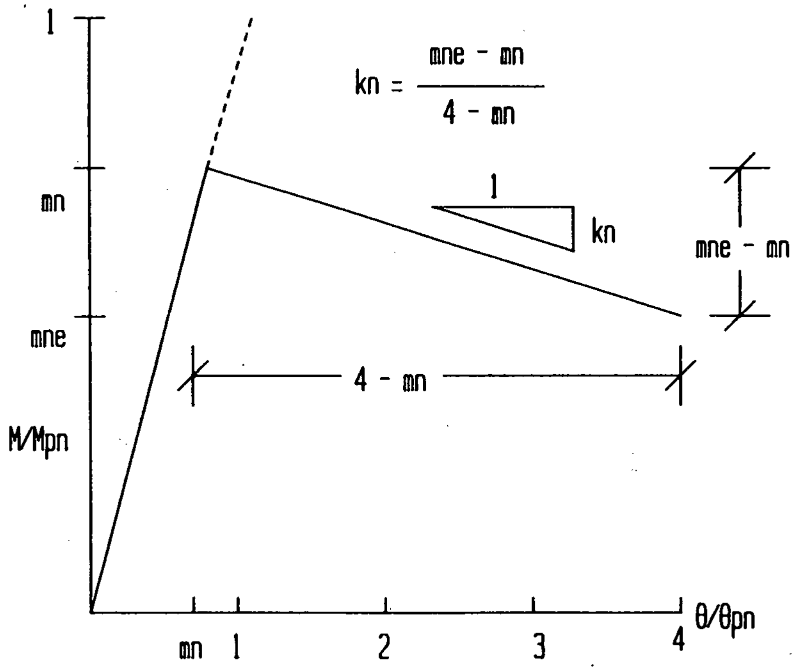


Figure B6 - Moment vs. Rotation at Supports

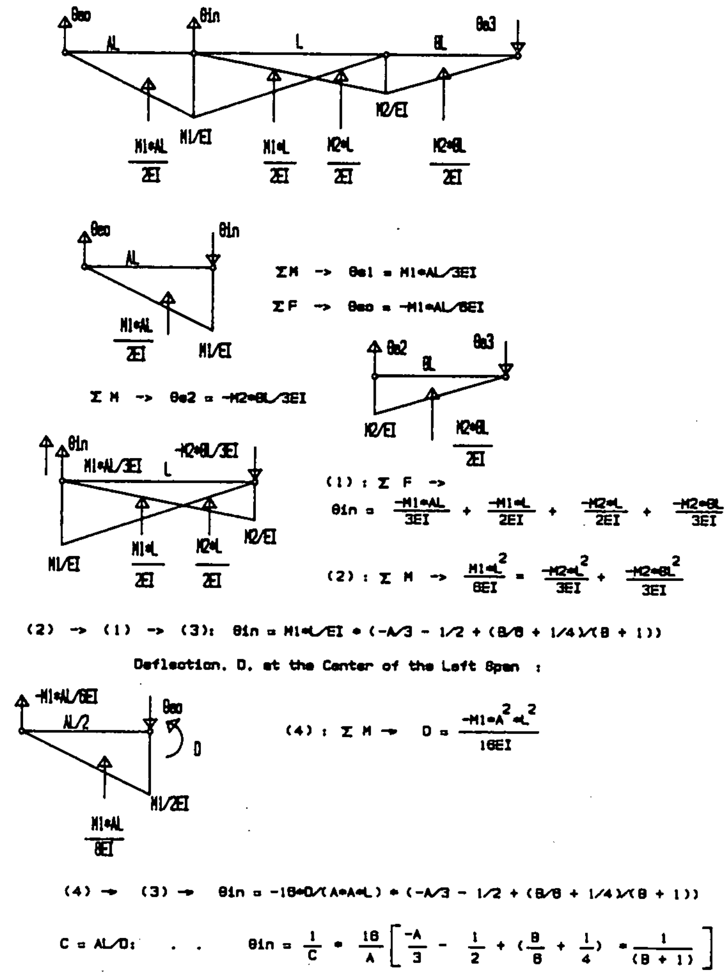


Figure B7 - Derivation of θi vs. C Equation for the External Span with an inelastic Rotation at Support 1

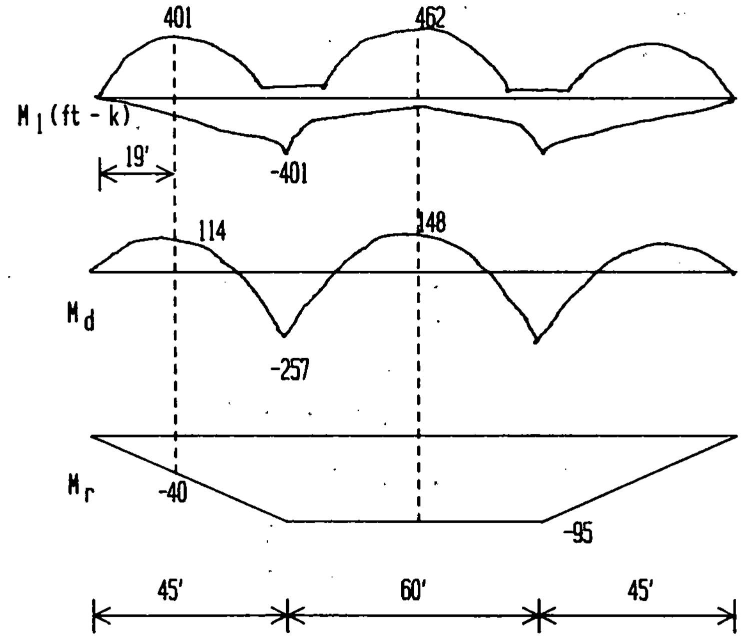


Figure B8 - Moment Diagrams for AASHTO 3S2 Truck

APPENDIXES C, D, E, AND G

UNPUBLISHED MATERIAL

Appendixes C, D, E, and G contained in the report as submitted by the research agency are not published herein. Their titles are listed here for the convenience of those interested in the subject area. Qualified researchers may obtain loan copies of the agency report by writing to the Transportation Research Board, Business Office, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

The available appendixes are titled as follows:

Appendix C—Two-Dimensional Shakedown Limit
Appendix D—Rating by 3-D Finite Element Methods
Appendix E—Shakedown Tests on a $\frac{1}{3}$-Scale Three-Span Composite Bridge
Appendix G—Strengthening and Retrofitting

Appendixes F and H follow this section.

APPENDIX F

SAMPLE RATING EXERCISES

F.1 RATING EXAMPLES

The following pages contain information and results of six bridges rated with the Grid/Shakedown and RDA procedures of the IBR program. The six bridges rated included three bridges from the Minnesota Department of Transportation Bridge Inventory, two fictitious noncomposite bridges, and one bridge from the AASHO Road Test. The bridges are as follows:

1. Minnesota Bridge 3708, simply supported noncomposite bridge;
2. Minnesota Bridge 9779, two-span composite bridge;
3. Minnesota Bridge 9055, three-span composite bridge with coverplates;
4. EX2SPAN, fictitious two-span noncomposite bridge;
5. SAUSER, fictitious three-span noncomposite bridge; and
6. AASHO 3B, simply supported composite bridge with coverplates.

For each of these bridges, the IBR input and output screens are shown. There are approximately six pages per bridge, with two pages for the geometry, one for the resistances, two for the loading, and one for the rating factors. Following are summary sheets of the rating results from the original input and from a variety of modification schemes. A summary and description of each of the cases are listed in the notes below the results.

F.2 MODIFICATION SCHEMES

Modification schemes investigated included adding lateral support in the negative moment regions; making noncompact sections compact; using lighter-weight replacement decks (reducing the dead loads); making noncomposite sections composite; and enhancing section capacities with coverplates. Each of these modification schemes is briefly summarized in Appendix G, "Strengthening and Retrofitting." For a more detailed description of modification schemes, design guidelines, and general cost information, refer to *NCHRP Report 293,* "Methods of Strengthening Existing Highway Bridges" (*1*).

To take advantage of IBR as it is presently exists, the bridges to be rated must be straight composite or noncomposite multigirder bridges. In addition, the bridges must have lateral support in the negative moment regions as specified in the AASHTO LFD provisions (*2*) to ensure plastic redistribution. The required spacing of the lateral bracing is output by IBR under the "RESISTANCES, Strength Stiffness" screen. If the bridge does not have adequate lateral bracing, as specified, this would be the first modification scheme required to take advantage of inelastic bridge rating.

Bridges that meet the aforementioned requirements, and which develop plastic hinges in the interior girders in regions of maximum moment, are well suited for the Residual Damage Analysis. If the bridges contain exterior girders of smaller cross section, they must be checked to ensure they do not govern the capacity of the bridge. If the bridges contain coverplates, the bridge engineer must compare the moment capacity at the ends of the coverplates with the moment envelope, because the plastic hinge region may shift away from the location of maximum moment to the end of the coverplates.

The grid analysis assumes the bridge girders to be compact and noncomposite. In other words, the moment-rotation characteristics of the section are assumed to be elastic-perfectly plastic. If the bridge girders are composite, a lower-bound solution would be to assume the yield moment, M_y (mM_p), as the limiting capacity of the girder. The grid analysis is not applicable to noncompact girders.

With the aforementioned caveats in mind, the six example bridges were rated with IBR. The input and results are presented in the following sections, pages F4 to F53.

F.3 IBR SAMPLE INPUT AND OUTPUT SCREENS

INPUT FOR IBR PROGRAM EXAMPLES:

G E O M E T R Y : B3708 (Noncomposite Simple-Span)

Spans:

Spacing and Spans:

Deck Thickness (in)..................:	5.0
Concrete Strength (ksi)..............:	3.0
Center-Center Girder Spacing (ft)....:	2.38
Number of Girders per Span...........:	13
Length of Span 1 (ft)................:	51
Length of Span 2 (ft)................:	-
Length of Span 3 (ft)................:	-
Length of Span 4 (ft)................:	-
Length of Span 5 (ft)................:	-

Give span lengths to nearest foot.
(See Hypertext for limitations on # of girders.)

Sections:

Define Total Number of Different Cross Sections of the Bridge:
How Many Different Sections Are There?:

Cross Section # of 2	1 (int)	2 (ext)
B (Effective Width of Slab) (in).......:	0.0	0.0
t (Depth of Slab) (in)..................:	5.0	5.0
B_{tf} (Top Flange Width) (in)...........:	14.04	10.04
T_{tf} (Top Flange Thickness) (in).......:	1.02	0.86
D_{web} (Web Depth) (in).................:	22.45	25.42
T_{web} (Web Thickness) (in).............:	0.61	0.54
B_{bf} (Bottom Flange Width) (in)........:	14.04	10.04
T_{bf} (Bottom Flange Thickness) (in)....:	1.02	0.86
As (Area of Slab Steel) (in^2).........:	0.61	0.56
Fs (Yield Stress of Slab Steel) (ksi).:	40.0	40.0
Fyf (Yield Stress of Flange) (ksi)....:	30.0	30.0
Fyw (Yield Stress of Web) (ksi).......:	30.0	30.0

Girders:

Section Number Associated with Interior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	1	1	1	1	1	1	1	1	1	1
Length (ft)	50.8									

Section Number Associated with Exterior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	2	2	2	2	2	2	2	2	2	2
Length (ft)	50.8									

Diaphragms:

Define Diaphragm Properties:

For X-Bracing Diagphragms (X), Give Diagonal Area, Depth:
Area (in) Depth (in)

For Fixed-Ended, Solid Member Diaphragms (M), Give Ix:
Ix (in^4) 67

Location of Diaphragm along Span (ft):
Input Diaphragm Type (X/M):

Diaphragm #	1	2	3	4	5	6	7	8	9	10
Limit 10										
Span 1	1	17	34	51						
Length (ft)	50.8									

Give location of diaphragm from left end of span (ft).

R E S I S T A N C E S :

Deterioration:

Select One of the Following:

Field inspection and condition surveys indicate no deterioration.
Inspection and condition surveys indicate slight deterioration.
Inspection and condition surveys indicate significant deterioration.
Instead of inspection, bridge records show superstructure condition: 5-6.
Instead of inspection, bridge records show superstructure condition: ≤ 4. *

Inspection:

Bridge records used instead of field inspection
$\phi = 0.75$

Maintenance:

$\phi = 0.75$

Strength Stiffness:

Nominal Section Properties

Section #	1 (int)	2 (ext)
Ix..............................:	4650	3862
Mp..............................:	1061	814
m = Max elastic M / plastic M...:	1.00	1.00
k = Inelastic Branch Slope......:	0.00	0.00
See Moment-Rotation Curve (YN)?.:		
Lateral Bracing Required for -M Regions (ft)...:	12.96	8.41

L O A D I N G :

AASHTO Trucks:

Select One of the Following: Truck Direction -->

Type 3 (50k)
Type 3S2 (72k) *
Type 3-3 (80k)
HS-20 (72k)

Special Permit:

Special Vehicle: Truck Direction -->

First (Front) Axle Load (kips).........:
Distance to 2nd Load (ft)..............:
2nd Axle Load (kips)...................:
Distance to 3rd Load (ft)..............:
3rd Axle Load (kips)...................:
Distance to 4th Load (ft)..............:
4th Axle Load (kips)...................:
Distance to 5th Load (ft)..............:
5th Axle Load (kips)...................:
Distance to 6th Load (ft)..............:
6th Axle Load (kips)...................:

Dead Loads:

Dead Load of Span 1 (klf)..............: 0.10

NOTE: If overlay thicknesses aren't verified by taking core samples then increase the overlay portion of the dead load by 20%.

The Dead Load Factor, $rd = 1.20$ (1.2 is Recommended)

Impact:

Select One of the Following:

Deck and approach are smooth.
Deck and approach are rough with bumps. *
Deck and approach very distressed and speeds are high.

Traffic Factors:

Select One of the Following:

Low volume road, reasonable enforcement, apparent control of overloads.
Heavy volume road, reasonable enforcement, apparent control of overloads. *
Low volume road, significant overloads without effective enforcement.
Heavy volume road, significant overloads without effective enforcement.

A N A L Y S I S:

Grid Analysis:

System Factors Analysis:

Execute System Analysis Immediately (Y/N)?.........: **N/A # girders exceeds limit**

System Factors from Grid:

Limit States Rating Factors:

The following Rating Factors (R.F.) are from the Grid Analysis: They are based on ALL SECTIONS being Elastic-Perfectly Plastic.

Limit State: NCHRP First Hinge :
Limit State: Grid First Hinge :
Limit State: Grid Shakedown :

If sections are not compact and composite, see hypertext.

Lateral Load Distribution:

Suggested Lateral Distribution Factors:

AASHTO Elastic Distribution Factor (A) [S/5.5]....: 0.43
Grid Elastic Distribution Factor (E)..............:
Grid Inelastic Distribution Factor (I)............:
Choose Either (A), (E), or (I)....................: A

Rating Equation:

The Rating Equation is:

$\phi * Rn = \gamma_d * D + \gamma_l * DF * L * (1 + I) * RF$

Which, in this case becomes:

$0.75 * Rn = 1.20 * D + 1.45 * 0.43 * L * (1 + 0.20) * RF$

Residual Damage Analysis:

Rating Factor (R.F.) to be Used in RDA...........: 3.00 (Ext. girder controlled)
Distance Truck Advances at Each Stop.............: 1
Number of Passes Truck Makes (5 max).............: 1
Execute RDA Immediately (Y/N)?...................: Y

INPUT FOR IBR PROGRAM EXAMPLES:

G E O M E T R Y :   B9779 (Composite 2-Span)

Spans:

Spacing and Spans:

Deck Thickness (in)..................:	6.5
Concrete Strength (ksi).............:	4.0
Center-Center Girder Spacing (ft)...:	8.21
Number of Girders per Span..........:	6
Length of Span 1 (ft)...............:	94.0
Length of Span 2 (ft)...............:	94.0
Length of Span 3 (ft)...............:	-
Length of Span 4 (ft)...............:	-
Length of Span 5 (ft)...............:	-

Give span lengths to nearest foot.
(See Hypertext for limitations on # of girders.)

Sections:

Define Total Number of Different Cross Sections of the Bridge:
How Many Different Sections Are There?:

Cross Section # of 4	1 (int)	2 (int)	3 (ext)	4 (ext)
B (Effective Width of Slab) (in)......:	78.0	0.0	70.0	0.0
t (Depth of Slab) (in)................:	6.5	6.5	6.5	6.5
B_{tf} (Top Flange Width) (in)...........:	16.6	15.9	16.6	15.9
T_{tf} (Top Flange Thickness) (in)........:	1.57	1.95	1.57	1.95
D_{web} (Web Depth) (in)..................:	33.4	39.4	33.4	39.4
T_{web} (Web Thickness) (in).............:	0.89	0.89	0.89	0.89
B_{bf} (Bottom Flange Width) (in)........:	16.6	16.5	16.6	16.5
T_{bf} (Bottom Flange Thickness) (in)....:	1.57	1.75	1.57	1.75
As (Area of Slab Steel) (in^2).........:	2.66	2.66	2.0	2.0
Fs (Yield Stress of Slab Steel) (ksi).:	40.0	40.0	40.0	40.0
Fyf (Yield Stress of Flange) (ksi)....:	33.0	33.0	33.0	33.0
Fyw (Yield Stress of Web) (ksi).......:	33.0	33.0	33.0	33.0

Girders:

Section Number Associated with Interior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	1	1	1	1	1	1	1	1	1	2
Length (ft)	94									
Span 2	2	1	1	1	1	1	1	1	1	1
Length (ft)	94									

Section Number Associated with Exterior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	3	3	3	3	3	3	3	3	3	4
Length (ft)	94									
Span 2	4	3	3	3	3	3	3	3	3	3
Length (ft)	94									

Diaphragms:

Define Diaphragm Properties:

For X-Bracing Diagphragms (X), Give Diagonal Area, Depth:
Area (in)   Depth (in)

For Fixed-Ended, Solid Member Diaphragms (M), Give Ix:
Ix (in^4)   315

Location of Diaphragm along Span (ft):
Input Diaphragm Type (X/M):

Diaphragm # Limit 10	1	2	3	4	5	6	7	8	9	10
Span 1	1	24	48	73	94					
Length (ft)	94									
Span 2	1	22	46	70	94					
Length (ft)	94									

Give location of diaphragm from left end of span (ft).

R E S I S T A N C E S :

Deterioration:

Select One of the Following:

Field inspection and condition surveys indicate no deterioration.
Inspection and condition surveys indicate slight deterioration. *
Inspection and condition surveys indicate significant deterioration.
Instead of inspection, bridge records show superstructure condition: 5-6.
Instead of inspection, bridge records show superstructure condition: ≤ 4.

Inspection:

Select One of the Following:

As a result of the inspection, careful estimates of section loss are made.
No careful estimates of section loss are made as result of inspection. *

Maintenance:

Select One of the Following:

Vigorous maintenance likely to inhibit future section loss. *
Intermittent maintenace not likely to inhibit future section loss.

Strength Stiffness:

Nominal Section Properties

Section #	1 (int)	2 (int)	3 (ext)	4 (ext)
Ix.............................:	35486	31531	34400	31156
Mp.............................:	4619	4545	4753	4496
m = Max elastic M / plastic M...:	0.76	1.00	0.74	1.00
k = Inelastic Branch Slope......:	0.25	0.00	0.42	0.00
See Moment-Rotation Curve (YN)?.:				
Lateral Bracing Required for -M Regions (ft)...:	13.52	13.12	13.52	13.12

L O A D I N G :

AASHTO Trucks:

Select One of the Following: Truck Direction -->

Type 3 (50k)
Type 3S2 (72k) *
Type 3-3 (80k)
HS-20 (72k)

Special Permit:

Special Vehicle: Truck Direction -->

First (Front) Axle Load (kips).........:
Distance to 2nd Load (ft)..............:
2nd Axle Load (kips)...................:
Distance to 3rd Load (ft)..............:
3rd Axle Load (kips)...................:
Distance to 4th Load (ft)..............:
4th Axle Load (kips)...................:
Distance to 5th Load (ft)..............:
5th Axle Load (kips)...................:
Distance to 6th Load (ft)..............:
6th Axle Load (kips)...................:

Dead Loads:

Dead Load of Span 1-2 (klf)............: 0.67

NOTE: If overlay thicknesses aren't verified by taking core samples then increase the overlay portion of the dead load by 20%.

The Dead Load Factor, γd = 1.20 (1.2 is Recommended)

Impact:

Select One of the Following:

Deck and approach are smooth. *
Deck and approach are rough with bumps.
Deck and approach very distressed and speeds are high.

Traffic Factors:

Select One of the Following:

Low volume road, reasonable enforcement, apparent control of overloads.
Heavy volume road, reasonable enforcement, apparent control of overloads. *
Low volume road, significant overloads without effective enforcement.
Heavy volume road, significant overloads without effective enforcement.

A N A L Y S I S:

Grid Analysis:

System Factors Analysis:

Execute System Analysis Immediately (Y/N)?.........: Y

System Factors from Grid:

Limit States Rating Factors:

The following Rating Factors (R.F.) are from the Grid Analysis: They are based on ALL SECTIONS being Elastic-Perfectly Plastic.

Limit State:	NCHRP First Hinge :	2.36	Note that these values were
Limit State:	Grid First Hinge :	2.62	based on m*Mp (rather than
Limit State:	Grid Shakedown :	3.48	full Mp).

If sections are not compact and composite, see hypertext.

Lateral Load Distribution:

Suggested Lateral Distribution Factors:

AASHTO Elastic Distribution Factor (A) [S/5.5]....: 1.49
Grid Elastic Distribution Factor (E)..............:
Grid Inelastic Distribution Factor (I)............:
Choose Either (A), (E), or (I)....................: A

Rating Equation:

The Rating Equation is:

$\phi * Rn = \gamma_d * D + \gamma_l * DF * L * (1 + I) * RF$

Which, in this case becomes:

$0.90 * Rn = 1.20 * D + 1.45 * 1.49 * L * (1 + 0.10) * RF$

Residual Damage Analysis:

Rating Factor (R.F.) to be Used in RDA...........: 2.28
Distance Truck Advances at Each Stop.............: 1
Number of Passes Truck Makes (5 max).............: 1
Execute RDA Immediately (Y/N)?...................: Y

INPUT FOR IBR PROGRAM EXAMPLES:

G E O M E T R Y : B9055 (Composite 3-Span)

Spans:

Spacing and Spans:

Deck Thickness (in)..................:	6.5
Concrete Strength (ksi).............:	4.0
Center-Center Girder Spacing (ft)...:	8.5
Number of Girders per Span..........:	5
Length of Span 1 (ft)...............:	44.0
Length of Span 2 (ft)...............:	47.0
Length of Span 3 (ft)...............:	44.0
Length of Span 4 (ft)...............:	-
Length of Span 5 (ft)...............:	-

Give span lengths to nearest foot.
(See Hypertext for limitations on # of girders.)

Sections:

Define Total Number of Different Cross Sections of the Bridge:
How Many Different Sections Are There?:

Cross Section # of 4	1 (int)	2 (int)	3 (ext)	4 (ext)
B (Effective Width of Slab) (in)......:	78.0	0.0	77.4	0.0
t (Depth of Slab) (in)................:	6.5	6.5	6.5	6.5
B_{tf} (Top Flange Width) (in)...........:	10.0	9.6	10.0	9.6
T_{tf} (Top Flange Thickness) (in).......:	0.75	1.25	0.75	1.25
D_{web} (Web Depth) (in).................:	25.41	25.41	25.41	25.41
T_{web} (Web Thickness) (in).............:	0.50	0.50	0.50	0.50
B_{bf} (Bottom Flange Width) (in)........:	10.0	10.0	10.0	10.0
T_{bf} (Bottom Flange Thickness) (in)....:	0.75	0.75	0.75	0.75
As (Area of Slab Steel) (in^2).........:	3.03	3.03	2.39	2.39
Fs (Yield Stress of Slab Steel) (ksi).:	40.0	40.0	40.0	40.0
Fyf (Yield Stress of Flange) (ksi)....:	33.0	33.0	33.0	33.0
Fyw (Yield Stress of Web) (ksi).......:	33.0	33.0	33.0	33.0

Girders:

Section Number Associated with Interior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	1	1	1	1	1	1	1	1	2	2
Length (ft)	43.5									
Span 2	2	2	1	1	1	1	1	1	2	2
Length (ft)	47.0									
Span 3	2	2	1	1	1	1	1	1	1	1
Length (ft)	43.5									

Section Number Associated with Exterior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	3	3	3	3	3	3	3	3	4	4
Length (ft)	43.5									
Span 2	4	4	3	3	3	3	3	3	4	4
Length (ft)	47.0									
Span 3	4	4	3	3	3	3	3	3	3	3
Length (ft)	43.5									

Diaphragms:

Define Diaphragm Properties:

For X-Bracing Diagphragms (X), Give Diagonal Area, Depth:
Area (in) Depth (in)

For Fixed-Ended, Solid Member Diaphragms (M), Give Ix:
Ix (in^4) 315

Location of Diaphragm along Span (ft):
Input Diaphragm Type (X/M):

Diaphragm #	1	2	3	4	5	6	7	8	9	10
Limit 10										
Span 1	1	23	44							
Length (ft) 43.5										
Span 2	1	24	47							
Length (ft) 47.0										
Span 1	1	21	44							
Length (ft) 43.5										

Give location of diaphragm from left end of span (ft).

R E S I S T A N C E S :

Deterioration:

Select One of the Following:

Field inspection and condition surveys indicate no deterioration.
Inspection and condition surveys indicate slight deterioration. *
Inspection and condition surveys indicate significant deterioration.
Instead of inspection, bridge records show superstructure condition: 5-6.
Instead of inspection, bridge records show superstructure condition: ≤ 4.

Inspection:

Select One of the Following:

As a result of the inspection, careful estimates of section loss are made.
No careful estimates of section loss are made as result of inspection. *

Maintenance:

Select One of the Following:

Vigorous maintenance likely to inhibit future section loss. *
Intermittent maintenace not likely to inhibit future section loss.

Strength Stiffness:

Nominal Section Properties

Section #	1 (int)	2 (int)	3 (ext)	4 (ext)
Ix.............................:	8858	4627	8828	4499
Mp.............................:	1388	1013	1388	933
m = Max elastic M / plastic M...:	0.70	1.00	0.70	1.00
k = Inelastic Branch Slope......:	0.06	-0.06	0.06	-0.05
See Moment-Rotation Curve (YN)?.:				
Lateral Bracing Required for -M Regions (ft)...:	7.51	7.75	7.51	7.75

L O A D I N G :

AASHTO Trucks:

Select One of the Following: Truck Direction -->

Type 3 (50k) *
Type 3S2 (72k)
Type 3-3 (80k)
HS-20 (72k)

Special Permit:

Special Vehicle: Truck Direction -->

First (Front) Axle Load (kips).........:
Distance to 2nd Load (ft)..............:
2nd Axle Load (kips)...................:
Distance to 3rd Load (ft)..............:
3rd Axle Load (kips)...................:
Distance to 4th Load (ft)..............:
4th Axle Load (kips)...................:
Distance to 5th Load (ft)..............:
5th Axle Load (kips)...................:
Distance to 6th Load (ft)..............:
6th Axle Load (kips)...................:

Dead Loads:

Dead Load of Span 1-3 (klf)............: 1.01

NOTE: If overlay thicknesses aren't verified by taking core samples then increase the overlay portion of the dead load by 20%.

The Dead Load Factor, γd = 1.20 (1.2 is Recommended)

Impact:

Select One of the Following:

Deck and approach are smooth. *
Deck and approach are rough with bumps.
Deck and approach very distressed and speeds are high.

Traffic Factors:

Select One of the Following:

Low volume road, reasonable enforcement, apparent control of overloads.
Heavy volume road, reasonable enforcement, apparent control of overloads. *
Low volume road, significant overloads without effective enforcement.
Heavy volume road, significant overloads without effective enforcement.

A N A L Y S I S:

Grid Analysis:

System Factors Analysis:

Execute System Analysis Immediately (Y/N)?.........: Y

System Factors from Grid:

Limit States Rating Factors:

The following Rating Factors (R.F.) are from the Grid Analysis: They are based on ALL SECTIONS being Elastic-Perfectly Plastic.

Limit State:	NCHRP First Hinge	:	1.68	Note that these values were
Limit State:	Grid First Hinge	:	1.96	based on m*Mp (rather than
Limit State:	Grid Shakedown	:	2.36	full Mp).

If sections are not compact and composite, see hypertext.

Lateral Load Distribution:

Suggested Lateral Distribution Factors:

AASHTO Elastic Distribution Factor (A) [S/5.5]....: 1.55
Grid Elastic Distribution Factor (E)..............:
Grid Inelastic Distribution Factor (I)............:
Choose Either (A), (E), or (I)....................: A

Rating Equation:

The Rating Equation is:

$\phi * Rn - \gamma_d * D + \gamma_l * DF * L * (1 + I) * RF$

Which, in this case becomes:

$0.90 * Rn - 1.20 * D + 1.45 * 1.55 * L * (1 + 0.10) * RF$

Residual Damage Analysis:

Rating Factor (R.F.) to be Used in RDA...........: 1.65
Distance Truck Advances at Each Stop............: 1
Number of Passes Truck Makes (5 max)............: 1
Execute RDA Immediately (Y/N)?..................: Y

INPUT FOR IBR PROGRAM EXAMPLES:

G E O M E T R Y : SAUSER (Noncomposite 3-Span)

Spans:

Spacing and Spans:

Deck Thickness (in)................:	7.5
Concrete Strength (ksi)............:	3.0
Center-Center Girder Spacing (ft)...:	5.5
Number of Girders per Span.........:	5
Length of Span 1 (ft)..............:	40.0
Length of Span 2 (ft)..............:	60.0
Length of Span 3 (ft)..............:	40.0
Length of Span 4 (ft)..............:	-
Length of Span 5 (ft)..............:	-

Give span lengths to nearest foot.
(See Hypertext for limitations on # of girders.)

Sections:

Define Total Number of Different Cross Sections of the Bridge:
How Many Different Sections Are There?:

Cross Section # of 3	1	2	3
B (Effective Width of Slab) (in)......:	66.0*	0.0	66.0*
t (Depth of Slab) (in)................:	7.5	7.5	7.5
B_{tf} (Top Flange Width) (in)...........:	12.04	14.09	14.02
T_{tf} (Top Flange Thickness) (in).......:	0.85	1.13	0.98
D_{web} (Web Depth) (in).................:	22.45	22.45	22.45
T_{web} (Web Thickness) (in).............:	0.51	0.65	0.59
B_{bf} (Bottom Flange Width) (in)........:	12.04	14.09	14.02
T_{bf} (Bottom Flange Thickness) (in)....:	0.85	1.13	0.98
As (Area of Slab Steel) (in^2).........:	-	-	-
Fs (Yield Stress of Slab Steel) (ksi).:	-	-	-
Fyf (Yield Stress of Flange) (ksi)....:	33.0	33.0	33.0
Fyw (Yield Stress of Web) (ksi).......:	33.0	33.0	33.0

* Effective width was initially assumed to be zero for this example to reflect that the structure was noncompact.

Girders:

Section Number Associated with Interior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1 Length (ft) 40.0	1	1	1	1	1	1	1	2	2	2
Span 2 Length (ft) 60.0	2	2	3	3	3	3	3	3	2	2
Span 3 Length (ft) 40.0	2	2	2	1	1	1	1	1	1	1

Section Number Associated with Exterior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1 Length (ft) 40.0	1	1	1	1	1	1	1	2	2	2
Span 2 Length (ft) 60.0	2	2	3	3	3	3	3	3	2	2
Span 3 Length (ft) 40.0	2	2	2	1	1	1	1	1	1	1

Diaphragms:

Define Diaphragm Properties:

For X-Bracing Diagphragms (X), Give Diagonal Area, Depth:
Area (in) Depth (in)

For Fixed-Ended, Solid Member Diaphragms (M), Give Ix:
Ix (in^4) 0.00

Location of Diaphragm along Span (ft):
Input Diaphragm Type (X/M):

Diaphragm #	1	2	3	4	5	6	7	8	9	10
Limit 10										
Span 1-3	None									

Give location of diaphragm from left end of span (ft).

R E S I S T A N C E S :

Deterioration:

Select One of the Following:

Field inspection and condition surveys indicate no deterioration. *
Inspection and condition surveys indicate slight deterioration.
Inspection and condition surveys indicate significant deterioration.
Instead of inspection, bridge records show superstructure condition: 5-6.
Instead of inspection, bridge records show superstructure condition: ≤ 4.

Inspection:

Select One of the Following:

As a result of the inspection, careful estimates of section loss are made. *
No careful estimates of section loss are made as result of inspection.

Maintenance:

Select One of the Following:

Vigorous maintenance likely to inhibit future section loss. *
Intermittent maintenace not likely to inhibit future section loss.

Strength Stiffness:

Nominal Section Properties

Section #	1	2	3
Ix.............................:	8435	5039	10358
Mp.............................:	1443	1251	1761
m - Max elastic M / plastic M...:	0.74	1.00	0.77
k - Inelastic Branch Slope......:	0.16	0.00	0.31
See Moment-Rotation Curve (YN)?.:			
Lateral Bracing Required for -M Regions (ft)...:	9.84	11.91	11.75

L O A D I N G :

AASHTO Trucks:

Select One of the Following: Truck Direction -->

Type 3 (50k) *
Type 3S2 (72k)
Type 3-3 (80k)
HS-20 (72k)

Special Permit:

Special Vehicle: Truck Direction -->

First (Front) Axle Load (kips).........:
Distance to 2nd Load (ft)..............:
2nd Axle Load (kips)...................:
Distance to 3rd Load (ft)..............:
3rd Axle Load (kips)...................:
Distance to 4th Load (ft)..............:
4th Axle Load (kips)...................:
Distance to 5th Load (ft)..............:
5th Axle Load (kips)...................:
Distance to 6th Load (ft)..............:
6th Axle Load (kips)...................:

Dead Loads:

Dead Load of Span 1-3 (klf)............: 0.59

NOTE: If overlay thicknesses aren't verified by taking core samples then increase the overlay portion of the dead load by 20%.

The Dead Load Factor, γd = 1.20 (1.2 is Recommended)

Impact:

Select One of the Following:

Deck and approach are smooth. *
Deck and approach are rough with bumps.
Deck and approach very distressed and speeds are high.

Traffic Factors:

Select One of the Following:

Low volume road, reasonable enforcement, apparent control of overloads.
Heavy volume road, reasonable enforcement, apparent control of overloads. *
Low volume road, significant overloads without effective enforcement.
Heavy volume road, significant overloads without effective enforcement.

A N A L Y S I S:

Grid Analysis:

System Factors Analysis:

Execute System Analysis Immediately (Y/N)?.........: Y

System Factors from Grid:

Limit States Rating Factors:

The following Rating Factors (R.F.) are from the Grid Analysis: They are based on ALL SECTIONS being Elastic-Perfectly Plastic.

Limit State:	NCHRP First Hinge :	3.35	Assuming section to be
Limit State:	Grid First Hinge :	3.37	noncomposite, B = 0.
Limit State:	Grid Shakedown :	3.99	

If sections are not compact and composite, see hypertext.

Lateral Load Distribution:

Suggested Lateral Distribution Factors:

AASHTO Elastic Distribution Factor (A) [S/5.5]....:	1.00
Grid Elastic Distribution Factor (E)..............:	0.99
Grid Inelastic Distribution Factor (I)............:	0.94
Choose Either (A), (E), or (I)....................:	A

Rating Equation:

The Rating Equation is:

$\phi * Rn = \gamma_d * D + \gamma_l * DF * L * (1 + I) * RF$

Which, in this case becomes:

$0.95 * Rn = 1.20 * D + 1.45 * 1.00 * L * (1 + 0.10) * RF$

Residual Damage Analysis:

Rating Factor (R.F.) to be Used in RDA...........:	3.22	Assuming section
Distance Truck Advances at Each Stop.............:	1	to be noncomposite,
Number of Passes Truck Makes (5 max).............:	1	B = 0.
Execute RDA Immediately (Y/N)?...................:	Y	

INPUT FOR IBR PROGRAM EXAMPLES:

G E O M E T R Y : EX2SPAN (Noncomposite 2-Span)

Spans:

Spacing and Spans:

Deck Thickness (in)..................:	7.0
Concrete Strength (ksi).............:	3.5
Center-Center Girder Spacing (ft)...:	7.0
Number of Girders per Span..........:	5
Length of Span 1 (ft)...............:	55.0
Length of Span 2 (ft)...............:	55.0
Length of Span 3 (ft)...............:	-
Length of Span 4 (ft)...............:	-
Length of Span 5 (ft)...............:	-

Give span lengths to nearest foot.
(See Hypertext for limitations on # of girders.)

Sections:

Define Total Number of Different Cross Sections of the Bridge:
How Many Different Sections Are There?:

Cross Section 1 of 1 1

B (Effective Width of Slab) (in)......:	0.0
t (Depth of Slab) (in)................:	7.0
B_{tf} (Top Flange Width) (in)...........:	9.99
T_{tf} (Top Flange Thickness) (in).......:	0.75
D_{web} (Web Depth) (in).................:	25.6
T_{web} (Web Thickness) (in).............:	0.49
B_{bf} (Bottom Flange Width) (in)........:	9.99
T_{bf} (Bottom Flange Thickness) (in)....:	0.75
As (Area of Slab Steel) (in^2).........:	-
Fs (Yield Stress of Slab Steel) (ksi).:	-
Fyf (Yield Stress of Flange) (ksi)....:	36.0
Fyw (Yield Stress of Web) (ksi).......:	36.0

Girders:

Section Number Associated with Interior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	1	1	1	1	1	1	1	1	1	1
Length (ft)	55									
Span 2	1	1	1	1	1	1	1	1	1	1
Length (ft)	55									

Section Number Associated with Exterior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	1	1	1	1	1	1	1	1	1	1
Length (ft)	55									
Span 2	1	1	1	1	1	1	1	1	1	1
Length (ft)	55									

Diaphragms:

Define Diaphragm Properties:

For X-Bracing Diagphragms (X), Give Diagonal Area, Depth:
Area (in) Depth (in)

For Fixed-Ended, Solid Member Diaphragms (M), Give Ix:
Ix (in^4) 0.00

Location of Diaphragm along Span (ft):
Input Diaphragm Type (X/M):

Diaphragm #	1	2	3	4	5	6	7	8	9	10
Limit 10										
Span 1-2	None									
Length (ft)	55									

Give location of diaphragm from left end of span (ft).

R E S I S T A N C E S :

Deterioration:

Select One of the Following:

Field inspection and condition surveys indicate no deterioration.
Inspection and condition surveys indicate slight deterioration. *
Inspection and condition surveys indicate significant deterioration.
Instead of inspection, bridge records show superstructure condition: 5-6.
Instead of inspection, bridge records show superstructure condition: ≤ 4.

Inspection:

Select One of the Following:

As a result of the inspection, careful estimates of section loss are made.
No careful estimates of section loss are made as result of inspection. *

Maintenance:

Select One of the Following:

Vigorous maintenance likely to inhibit future section loss. *
Intermittent maintenace not likely to inhibit future section loss.

Strength Stiffness:

Nominal Section Properties

Section # (int)	1
Ix.............................:	3270
Mp.............................:	834
m = Max elastic M / plastic M...:	1.00
k = Inelastic Branch Slope......:	0.00
See Moment-Rotation Curve (YN)?.:	
Lateral Bracing Required for -M Regions (ft)...:	6.87

L O A D I N G :

AASHTO Trucks:

Select One of the Following: Truck Direction -->

Type 3 (50k)
Type 3S2 (72k) *
Type 3-3 (80k)
HS-20 (72k)

Special Permit:

Special Vehicle: Truck Direction -->

First (Front) Axle Load (kips).........:
Distance to 2nd Load (ft)..............:
2nd Axle Load (kips)...................:
Distance to 3rd Load (ft)..............:
3rd Axle Load (kips)...................:
Distance to 4th Load (ft)..............:
4th Axle Load (kips)...................:
Distance to 5th Load (ft)..............:
5th Axle Load (kips)...................:
Distance to 6th Load (ft)..............:
6th Axle Load (kips)...................:

Dead Loads:

Dead Load of Span 1-2 (klf)............: 0.75

NOTE: If overlay thicknesses aren't verified by taking core samples then increase the overlay portion of the dead load by 20%.

The Dead Load Factor, γd = 1.20 (1.2 is Recommended)

Impact:

Select One of the Following:

Deck and approach are smooth.
Deck and approach are rough with bumps. *
Deck and approach very distressed and speeds are high.

Traffic Factors:

Select One of the Following:

Low volume road, reasonable enforcement, apparent control of overloads.
Heavy volume road, reasonable enforcement, apparent control of overloads.
Low volume road, significant overloads without effective enforcement.
Heavy volume road, significant overloads without effective enforcement. *

A N A L Y S I S:

Grid Analysis:

System Factors Analysis:

Execute System Analysis Immediately (Y/N)?.........: Y

System Factors from Grid:

Limit States Rating Factors:

The following Rating Factors (R.F.) are from the Grid Analysis: They are based on ALL SECTIONS being Elastic-Perfectly Plastic.

Limit State:	NCHRP First Hinge :	0.94	Note that these values were
Limit State:	Grid First Hinge :	1.01	based on m*Mp (rather than
Limit State:	Grid Shakedown :	1.13	full Mp).

If sections are not compact and composite, see hypertext.

Lateral Load Distribution:

Suggested Lateral Distribution Factors:

AASHTO Elastic Distribution Factor (A) [S/5.5]....: 1.27
Grid Elastic Distribution Factor (E)..............: 1.19
Grid Inelastic Distribution Factor (I)............: 1.09
Choose Either (A), (E), or (I)....................: A

Rating Equation:

The Rating Equation is:

$\phi * Rn = \gamma_d * D + \gamma_l * DF * L * (1 + I) * RF$

Which, in this case becomes:

$0.90 * Rn = 1.20 * D + 1.80 * 1.27 * L * (1 + 0.20) * RF$

Residual Damage Analysis:

Rating Factor (R.F.) to be Used in RDA...........: 1.27
Distance Truck Advances at Each Stop.............: 1
Number of Passes Truck Makes (5 max).............: 1
Execute RDA Immediately (Y/N)?...................: Y

INPUT FOR IBR PROGRAM EXAMPLES:

G E O M E T R Y :   AASHO (Composite Simple-Span)

Spans:

Spacing and Spans:

Deck Thickness (in)..................:	6.5
Concrete Strength (ksi)..............:	5.7
Center-Center Girder Spacing (ft)...:	3.0
Number of Girders per Span..........:	3
Length of Span 1 (ft)...............:	50
Length of Span 2 (ft)...............:	-
Length of Span 3 (ft)...............:	-
Length of Span 4 (ft)...............:	-
Length of Span 5 (ft)...............:	-

Give span lengths to nearest foot.
(See Hypertext for limitations on # of girders.)

Sections:

Define Total Number of Different Cross Sections of the Bridge:
How Many Different Sections Are There?:

Cross Section # of 2	1 (int)	2 (ext)
B (Effective Width of Slab) (in)......:	60.0	60.0
t (Depth of Slab) (in)................:	6.5	6.5
B_{tf} (Top Flange Width) (in)...........:	7.5	7.5
T_{tf} (Top Flange Thickness) (in).......:	0.67	0.67
D_{web} (Web Depth) (in).................:	16.9	16.9
T_{web} (Web Thickness) (in).............:	0.36	0.36
B_{bf} (Bottom Flange Width) (in)........:	7.5	6.9
T_{bf} (Bottom Flange Thickness) (in)....:	0.67	1.11
As (Area of Slab Steel) (in^2).........:	0.74	0.74
Fs (Yield Stress of Slab Steel) (ksi).:	61.2	61.2
Fyf (Yield Stress of Flange) (ksi)....:	35.1	36.0
Fyw (Yield Stress of Web) (ksi).......:	39.9	39.9

Girders:

Section Number Associated with Interior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	1	1	1	2	2	2	2	1	1	1
Length (ft)	50									

Section Number Associated with Exterior Span 10th Point
10th Point of Span:

	1	2	3	4	5	6	7	8	9	10
Span 1	1	1	1	2	2	2	2	1	1	1
Length (ft)	50									

Diaphragms:

Define Diaphragm Properties:

For X-Bracing Diagphragms (X), Give Diagonal Area, Depth:
Area (in)   Depth (in)

For Fixed-Ended, Solid Member Diaphragms (M), Give Ix:
Ix (in^4) 129

Location of Diaphragm along Span (ft):
Input Diaphragm Type (X/M):

Diaphragm # Limit 10	1	2	3	4	5	6	7	8	9	10
Span 1	1	16	34	50						
Length (ft) 50.0										

Give location of diaphragm from left end of span (ft).

R E S I S T A N C E S :

Deterioration:

Select One of the Following:

Field inspection and condition surveys indicate no deterioration. *
Inspection and condition surveys indicate slight deterioration.
Inspection and condition surveys indicate significant deterioration.
Instead of inspection, bridge records show superstructure condition: 5-6.
Instead of inspection, bridge records show superstructure condition: ≤ 4.

Inspection:

Select One of the Following:

As a result of the inspection, careful estimates of section loss are made. *
No careful estimates of section loss are made as result of inspection.

Maintenance:

Select One of the Following:

Vigorous maintenance likely to inhibit future section loss. *
Intermittent maintenace not likely to inhibit future section loss.

Strength Stiffness:

Nominal Section Properties

Section #	1 (int)	2 (ext)
Ix..............................:	3039	3964
Mp..............................:	719	911
m = Max elastic M / plastic M...:	0.66	0.70
k = Inelastic Branch Slope......:	0.04	0.05
See Moment-Rotation Curve (YN)?.:		
Lateral Bracing Required for -M Regions (ft)...:	5.68	5.50

L O A D I N G :

AASHTO Trucks:

Select One of the Following: Truck Direction -->

Type 3 (50k) *
Type 3S2 (72k)
Type 3-3 (80k)
HS-20 (72k)

Special Permit:

Special Vehicle: Truck Direction -->

First (Front) Axle Load (kips).........:
Distance to 2nd Load (ft)..............:
2nd Axle Load (kips)...................:
Distance to 3rd Load (ft)..............:
3rd Axle Load (kips)...................:
Distance to 4th Load (ft)..............:
4th Axle Load (kips)...................:
Distance to 5th Load (ft)..............:
5th Axle Load (kips)...................:
Distance to 6th Load (ft)..............:
6th Axle Load (kips)...................:

Dead Loads:

Dead Load of Span 1 (klf)..............: 0.48

NOTE: If overlay thicknesses aren't verified by taking core samples then increase the overlay portion of the dead load by 20%.

The Dead Load Factor, γd = 1.20 (1.2 is Recommended)

Impact:

Select One of the Following:

Deck and approach are smooth. *
Deck and approach are rough with bumps.
Deck and approach very distressed and speeds are high.

Traffic Factors:

Select One of the Following:

Low volume road, reasonable enforcement, apparent control of overloads. *
Heavy volume road, reasonable enforcement, apparent control of overloads.
Low volume road, significant overloads without effective enforcement.
Heavy volume road, significant overloads without effective enforcement.

A N A L Y S I S:

Grid Analysis:

System Factors Analysis:

Execute System Analysis Immediately (Y/N)?.........:

System Factors from Grid:

Limit States Rating Factors:

The following Rating Factors (R.F.) are from the Grid Analysis: They are based on ALL SECTIONS being Elastic-Perfectly Plastic.

Limit State: NCHRP First Hinge :
Limit State: Grid First Hinge :
Limit State: Grid Shakedown :

If sections are not compact and composite, see hypertext.

Lateral Load Distribution:

Suggested Lateral Distribution Factors:

AASHTO Elastic Distribution Factor (A) [S/5.5]....: 0.91
Grid Elastic Distribution Factor (E)..............:
Grid Inelastic Distribution Factor (I)............:
Choose Either (A), (E), or (I)....................: A

Rating Equation:

The Rating Equation is:

$\phi * Rn = r_d * D + r_l * DF * L * (1 + I) * RF$

Which, in this case becomes:

$0.70 * Rn = 1.20 * D + 1.80 * 0.91 * L * (1 + 0.10) * RF$
[Factors have been overridden.]

Residual Damage Analysis:

Rating Factor (R.F.) to be Used in RDA...........:
Distance Truck Advances at Each Stop.............:
Number of Passes Truck Makes (5 max).............:
Execute RDA Immediately (Y/N)?...................:

F.4 SUMMARY OF RESULTS

To utilize IBR, lateral bracing was assumed to be provided according to the AASHTO LFD specifications (2) in the negative moment regions. This bracing must be provided to insure plastic redistribution. The value of maximum lateral bracing spacing is output by IBR (see Section F.3).

The GRID analysis is based on compact noncomposite sections (elastic-plastic section properties). If a structure contains noncompact sections, the sections must be made compact to utilize the GRID analysis. This would be a <u>required</u> modification scheme to utilize the GRID procedure. If the section is composite, an assumption must be made regarding the maximum capacity of the section. The value "mM_p" would provide a lower bound value for the rating factor; whereas, "M_p" would provide an upper bound. The RDA procedure is based on the actual moment-rotation properties of the cross section, and can thus accommodate noncompact composite sections without modification.

Caution is advised when using RDA if it is suspected that the hinge will form away from the location of maximum moment (e.g. end of coverplate). RDA is based on the assumption that the plastic hinge will develop at the location of maximum moment. Caution is also advised when using RDA with the AASHTO DF because the RDA procedure has been written to automatically check the capacity of an interior girder. If the exterior girder is of smaller cross section, it may control the rating results as in the case of Bridge 3708. This must be checked by the

rating engineer. RDA using GRID DF gives correct results for the case of an exterior girder controlling the rating factor.

Note that only one truck pass was used for RDA in these examples. At least five passes should ordinarily be used to insure that the structure has shaken down.

RATING STUDY AND MODIFICATION SCHEMES

Bridge Name : 3708
Bridge Type : 1-span
Truck Type : 3
Compact : Yes
Composite : No

Factors ϕ = 0.75
γ_d = 1.20
γ_l = 1.45
I = 0.20

Case No.	1
GRID:	
NCHRP	N/A
1st Hinge	N/A
Shakedown	N/A
D.F.:	
(A) AASHTO	
(GE) Grid Elastic	
(GI) Grid Inelastic	
RDA:	
(A) 1st Hinge	3.00
(A) L/D = 600	—
(A) L/D = 300	—
RDA:	
(GE) 1st Hinge	N/A
(GI) L/D = 600	N/A
(GI) L/D = 300	N/A

Notes:

Case No. 1 – Original bridge. To utilize IBR, lateral bracing was assumed to be provided according to the AASHTO LFD specifications (2) in the negative moment regions. The maximum spacing was computed by IBR to be 12.96 ft. and 8.41 ft. for the interior and exterior girders, respectively (refer to Section F.3).

The GRID analysis was not applicable (N/A) to this bridge because the thirteen girders exceeded the current programming dimension limitation of seven girders for a simple-span bridge.

The RDA procedure has been written to automatically check the capacity of an interior girder. In the case of this bridge, the exterior girders were of smaller cross section and controlled the rating results.

RATING STUDY AND MODIFICATION SCHEMES

Bridge Name : 9779
Bridge Type : 2-span
Truck Type : 3S2
Compact : Yes
Composite : Yes

Factors ϕ = 0.90
γ_d = 1.20
γ_l = 1.45
I = 0.10

Case No.	1	2
GRID:		
NCHRP	2.36	3.22
1st Hinge	2.62	3.71
Shakedown	3.48	4.40
D.F.:		
(A) AASHTO		
(GE) Grid Elastic		
(GI) Grid Inelastic		
RDA:		
(A) 1st Hinge	2.28	3.14
(A) L/D = 600	3.10	—
(A) L/D = 300	3.42	—
RDA:		
(GE) 1st Hinge	2.52	3.47
(GI) L/D = 600	3.80	—
(GI) L/D = 300	4.19	—

Notes:

As mentioned earlier, the bridges were all rated on the assumption that adequate lateral bracing was present in the negative moment regions (approximately 20 ft. intervals for Bridge 9779, see Section F.3).

The exterior girder has not been checked to see if it controls in this example for the AASHTO DF case.

Case No. 1 – Original bridge. For this composite bridge, the section properties were modified such that "mM_p" (yield moment) was used as the maximum capacity for the GRID analysis (which is based on elastic-plastic section properties). The GRID rating results based upon this assumption serve as a "lower bound" rating factor.

Case No. 2 - "M_p" (ultimate composite moment) was assumed as the maximum section capacity for the elastic-plastic GRID analysis. This assumption gives an upper bound rating factor. "M_p" was also assumed for the first hinge rating factor obtained using RDA.

RATING STUDY AND MODIFICATION SCHEMES

Bridge Name : 9055
Bridge Type : 3-span
Truck Type : 3
Compact : (see notes below)
Composite : Yes

Factors ϕ = 0.90
γ_d = 1.20
γ_l = 1.45
I = 0.10

Case No.	1	2	3	4	5
GRID:					
NCHRP	1.68	2.62	1.66	2.58	2.72
1st Hinge	1.96	3.04	1.93	3.01	3.16
Shakedown	2.36	3.29	2.27	3.20	3.44

D.F.:
(A) AASHTO
(GE) Grid Elastic
(GI) Grid Inelastic

RDA:	1	2	3	4
(A) 1st Hinge	1.65	2.56	1.60	2.50
(A) L/D = 600	1.89	—	1.85	—
(A) L/D = 300	2.15	—	2.05	—

RDA:	1	2	3	4
(GE) 1st Hinge	1.91	2.95	1.87	2.91
(GI) L/D = 600	2.21	—	2.16	—
(GI) L/D = 300	2.44	—	2.40	—

Notes:

Bridge 9055 was assumed to have adequate lateral bracing (maximum spacing of 7.51 ft.). The exterior girder was not checked to see if it controlled the RDA AASHTO DF case.

Case No. 1 - Because this structure was noncompact, it was not appropriate to use the GRID analysis. To employ the GRID analysis, the sections must be modified to be made compact (which is not a trivial retrofitting scheme). Once the sections are made compact, an assumption must be made regarding the composite action because the GRID analysis uses elastic-plastic section properties. For this case (Case No. 1), "mM_p" was used as the maximum section strength. This assumption gave a lower bound rating factor of the composite bridge modified to be compact.

The RDA analysis can accommodate noncompact composite structures. Consequently, the RDA solution for this case was based on the original structure. The calculated M-theta curve was used for RDA.

Case No. 2 – Similar to Case No. 1. The sections were assumed to be made compact for the GRID analysis. In this case "M_p" was used as the maximum section strength to give an upper bound rating factor. "M_p" was assumed for the first hinge rating factor of RDA.

Case No. 3 – The sections are compact if the slab steel is ignored. Thus while ignoring the slab steel, "mM_p" was used for GRID. While ignoring the slab steel, the calculated M-theta curve was used for RDA.

Case No. 4 – Similar to Case No. 3, the slab reinforcement was ignored while assuming "M_p" for GRID and "M_p" for the first hinge rating factor of RDA.

Case No. 5 – Modification scheme – The dead load was reduced by 20% (1.01 to 0.8 klf) by replacing deck with a light-weight deck.

RATING STUDY AND MODIFICATION SCHEMES

Bridge Name : EX2SPAN
Bridge Type : 2-span
Truck Type : 3S2
Compact : Yes
Composite : No

Factors ϕ = 0.90
γ_d = 1.20
γ_l = 1.80
I = 0.20

Case No.	1	2	3	4	5
GRID:					
NCHRP	0.94	1.03	1.38	1.95	2.03
1st Hinge	1.01	1.12	1.46	2.09	2.17
Shakedown	1.13	1.30	1.55	2.20	2.30
D.F.:					
(A) AASHTO	1.27	1.27	1.27	1.27	1.27
(GE) Grid Elastic	1.19	1.18	1.20	1.19	1.19
(GI) Grid Inelastic	1.09	1.10	1.13	1.13	1.13
RDA:					
(A) 1st Hinge	0.91	1.03	1.23 1.34	1.25 1.90	1.32 2.01
(A) L/D = 600	0.99	1.08	1.34	1.50	1.58
(A) L/D = 300	@ 1 GONE	1.14 @ 1.40 GONE	1.39	1.59	1.68
RDA:					
(GE) 1st Hinge					
(GI) L/D = 600					
(GI) L/D = 300					

Notes:

Case No. 1 – The original noncomposite compact section was used for GRID and RDA.

Case No. 2 – Modification scheme – Add 3/8" cover plates to top and bottom of girders in negative moment region (location of first hinge without coverplates).

Case No. 3 – Modification scheme – Without the addition of coverplates make the section composite in the positive moment region (ignore slab steel). Assume "M_p" for GRID. Use calculated M-theta curve for RDA. Highlighted number indicates "M_p" assumption for first hinge rating factor of RDA.

Case No. 4 – <u>Modification scheme</u> – Combination of Case Nos. 2 and 3 (add coverplates in negative moment region, make slab composite in positive moment region).

Case No. 5 – <u>Modification scheme</u> – reduce dead load of Case No. 4 by 20% (0.75 to 0.60 klf) by replacing deck with a light-weight deck.

<u>RATING STUDY AND MODIFICATION SCHEMES</u>

Bridge Name : SAUSER
Bridge Type : 3-span
Truck Type : 3
Compact : Yes
Composite : No (Except Cases 2-3)

Factors ϕ = 0.95
γ_d = 1.20
γ_l = 1.45
I = 0.10

Case No.	1	2	3
<u>GRID:</u>			
NCHRP	3.35	4.81	4.91
1st Hinge	3.37	4.82	4.93
Shakedown	3.99	5.66	5.81
<u>D.F.:</u>			
(A) AASHTO	1.00	1.00	1.00
(GE) Grid Elastic	0.99	1.00	1.00
(GI) Grid Inelastic	0.94	0.95	0.95
<u>RDA:</u>			
(A) 1st Hinge	3.22	3.50 4.60	3.55 4.72
(A) L/D = 600	3.57	4.86	4.92
(A) L/D = 300	3.80	5.36	5.43
<u>RDA:</u>			
(GE) 1st Hinge	3.24		
(GI) L/D = 600	3.80		
(GI) L/D = 300	4.02		

<u>Notes:</u>

Case No. 1 – Original noncomposite compact section for GRID and RDA.

Case No. 2 – <u>Modification scheme</u> – Make the section composite in the positive moment region (ignore slab steel). Assume "M_p" for GRID. Use calculated M-theta curve for RDA. Highlighted number indicates "M_p" assumption for first hinge rating factor of RDA.

Case No. 3 – <u>Modification scheme</u> – reduce dead load of Case No. 2 by 20% (0.59 to 0.47 klf) by replacing deck with a light-weight deck.

RATING STUDY AND MODIFICATION SCHEMES

Bridge Name : AASHO 3B
Bridge Type : 1-span
Truck Type : 3
Compact : Yes
Composite : Yes

Factors ϕ = 0.70
r_d = 1.20
r_l = 1.80
I = 0.10

Case No. 1

GRID:

NCHRP	1.08
1st Hinge	0.72
Shakedown	0.73

D.F.:

(A) AASHTO 0.91 (Should be S/7=0.71; used this value)
(GE) Grid Elastic 1.35
(GI) Grid Inelastic 1.33

RDA:

(A) 1st Hinge 1.27 (Note RDA does not predict this hinge because it is located at the end of the coverplate rather than region of maximum moment)
(A) L/D = 600 —
(A) L/D = 300 —

RDA:

(GE) 1st Hinge
(GI) L/D = 600
(GI) L/D = 300

Notes:

Case No. 1 – Note that AASHTO distribution factor given by program IBR is always S/5.5. This value may be overridden in the rating equation as was done in this case.

Another caution is that RDA assumes that the plastic hinge forms at location of maximum positive moment or at the interior supports. In this case, the hinge formed at the end of the cover plates. Extending the coverplates across the span to force hinging at location of maximum positive moment gives an RDA first hinge rating factor of 1.37 assuming "M_p". Coverplate extension may be considered a Modification scheme.

REFERENCES

1. Klaiber, F.W., Dunker, K.F., Wipf, T.J. and Sanders, Jr., W.W., "Methods of Strengthening Existing Highway Bridges," NCHRP Report 293, Transportation Research Board, National Research Council, Washington, D.C., September 1987, 114 pp.

2. American Association of State Highway and Transportation Officials "Guide Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections" AASHTO, Washington D.C., 1986.

APPENDIX H

GUIDE TO INELASTIC BRIDGE RATING SOFTWARE PACKAGE

PURPOSE OF INELASTIC BRIDGE RATING

INELASTIC BRIDGE RATING (IBR) is a very user-friendly computer program for performing inelastic ratings of composite or noncomposite steel girder bridges of from one to five continuous spans. For noncomposite bridges made from compact girders, the two-dimensional grillage analysis routing of IBR can be used to provide elastic and inelastic rating factors based on the strength limit states of first yield and shakedown, as well as elastic and inelastic lateral load distribution factors.

The lateral load distribution factors can be useful when performing a single girder analysis. For composite and/or noncompact systems (as well as noncomposite, compact structures), IBR provides the single girder analysis method of Residual Damage Analysis (RDA), which is used to rate a bridge against an inelastic serviceability limit state, which is defined as the ratio of span length to maximum inelastic residual span deflection, $K = L/D$.

FEATURES OF INELASTIC BRIDGE RATING

The IBR software package consists of easy-to-use, personal computer (PC)-based, inelastic structural analysis routines, as well as a component for helping to train engineers to use these routines as rating tools.

The framework for this package consists of a windowing interface with drop-down menus and context-sensitive access to on-line hypertext help/training information. The inelastic bridge analysis subprograms consist of a two-dimensional grillage analysis routine, and the one-dimensional RDA routine. The reliability-based load and resistance factors for bridge rating developed by Moses and recently published as AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, 1989* (Reference *10* in this report) (hereafter referred to as the *Guide Specifications*) have been incorporated into IBR.

The look and feel of the software's functions have, as closely as possible, been modeled after the current standards in microcomputer applications user-interfaces. Every menu choice from within the program is hot-linked to a hypertext node. Thus, with the press of the F1-help key from any point in the main program, you can "jump" into the hypertext in a context-sensitive manner. From there, any trail of associative links may be followed; the links are two-way, and a record is kept of which links were pursued—this allows for backtracking. Once done with following a trail of links, you can simply jump back into the main program with the touch of the SPACEBAR.

System Requirements

In order that this software be transferable to as many users as possible, it was developed to run under the PC/MS-DOS operating system on an IBM-PC, XT, AT (or compatible) machine with 512 kilobytes of memory, graphics capability (Hercules, CGA, EGA, or VGA), and a hard disk. To use IBR, just create a subdirectory on your hard disk called IBR. Copy both diskettes into this directory. Type IBR to run the program.

Sample Rating Exercise

As a quick introduction to IBR, the two-span example design bridge that appears in the AASHTO *Guide Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections,* 1986 (see Reference *73* in this report) will be rated for the AASHTO 3S2 truck.

From within the IBR subdirectory, type IBR and you will enter the program in the FILES window under LOAD. Press RETURN to see the name of the last referenced file, which in this case is ALFD100. Press the ESCAPE key to load the data of ALFD100. Now you are free to examine and/or modify the data of this bridge. In most cases, each pop-up window will prompt you as to which key needs to be pressed; also, note that the upper right corner of the screen indicates which keys are available.

Whenever the F1=HELP message appears in the upper corner of the screen, context-sensitive system help in the form of a hypertext pops up at the press of the function key, F1. While reading through a help window, you will notice many significant bridge rating terms that are followed by a bracketed number (e.g., [3]); this indicates that further help is available by pressing the function key of the number in brackets (the F3 key, in this instance). The help screen that pops up to describe the bridge rating term in question will itself contain terms followed by bracketed numbers, each of these terms has its own descriptive help screen that can be accessed in the same manner as just described. The PgUp and PgDn keys are used to scroll through large help files that fill more than one screen. The trail of help files that is pursued can be backtracked using the HOME key. The SPACEBAR exits you from the help hypertext.

Windows Under the GEOMETRY Main Menu Heading

Use the arrow keys to enter the SPANS window under the GEOMETRY main menu heading, where you'll see that a 7" slab of 4 ksi concrete sits on 4 parallel two-span (L = 100') continuous girders spaced 8.33' o.c.

In the SECTIONS window, you'll see that two different cross sections will be used to model the bridge. Press ESC to see the data of Section #1 that is used in the positive moment regions.

Notice that Section #1 is a composite beam with an 84" effective width of slab connected to a W36×170 steel section made from 50 ksi steel; 6.16 sq. in. of 60 ksi rebar are located in the effective width. Press ESC again to see that Section #2, which will be used at the interior support, is identical to Section #1 except that the contribution of the concrete is removed by making the effective width equal to zero.

Enter the GIRDERS window to see that all interior and exterior girders use Section #1 everywhere except at the $\frac{1}{10}$th span segments adjacent to the interior support. Enter the DIAPHRAGMS window to see that up to 10 X-bracing or solid welded-end diaphragms can be placed along each span. The diaphragms were neglected in this example.

Windows Under the RESISTANCES Main Menu Heading

Under the RESISTANCES window, the first three submenus, DETERIORATION, INSPECTION, and MAINTENANCE, are used to define the resistance factor in accordance with the rules set forth in the *Guide Specifications*. In this case notice that for no deterioration, careful inspection, and vigorous maintenance, the resistance factor is 0.95 for this redundant structure.

In the STRENGTH/STIFFNESS window, the moment of inertia, *Ix*, the nominal (unfactored) positive and negative plastic moment capacities, *Mp* and *–Mp*, respectively, the percentage of the full plastic moment, *Mp*, that can be attained elastically, *m*, and the slope of the inelastic branch of the bi-linear moment-rotation curve, *k*, are all calculated based on the data given for each cross section in the SECTIONS window of GEOMETRY. The required distance between lateral brace points to prevent the section from lateral buckling is also computed. The bi-linear moment-rotation curve for each section may be graphically viewed—notice that neither of the cross sections is elastoplastic, and that Section #2 is noncompact due to the rebars shifting the N.A. See the hypertext for further information.

Before leaving the STRENGTH/STIFFNESS window, enter the screen for the negative bending section, Section #2, and use the BACKSPACE key to change Ix = 12929 in.^{4} to the value of *Ix* for the positive bending Section #1, Ix = 24697. This will result in a prismatic analysis of the two-span composite structure.

Windows Under the LOADING Main Menu Heading

Under the AASHTO TRUCKS window, a 3S2 rating vehicle is selected. A special configuration of loads can be defined in the SPECIAL PERMITS window, where you see the loads and axle spacings of the truck selected previously. (Note that these are full axle loads—prior to running the analysis routines, these loads are divided by two to get a single wheel line.)

The DEAD LOADS window reveals that 1.13 k/ft of dead load is placed on all girders of each span. The recommended dead load factor from the *Guide Specifications*, 1.2, is used here.

The *Guide Specifications* provide the rules for determining IMPACT FACTORS, which are based on the roughness of the bridge approaches and deck; a smooth deck is assumed here, so that I = 0.1. The TRAFFIC FACTORS window is used to select a live load factor; for the four options presented, this factor is (top to bottom) 1.3, 1.45, 1.6, and 1.8. A heavy volume roadway with good enforcement is assumed here, and the live load factor is 1.45.

Windows Under the ANALYSIS Main Menu Heading

For this composite, noncompact bridge example, the grillage analysis under the GRID ANALYSIS window can be used to provide refined elastic and inelastic lateral distribution factors for use in a single girder analysis (see the LATERAL LOAD DISTRIBUTION window of this menu). Also, upperbound rating factors are given in the SYSTEMS FACTORS FROM GRID window. The three rating factors are based on the strength limit states of first yield, and shakedown—they correspond with the three lateral distribution factors. The first two of these three rating factors are elastic rating factors: the first is based on the AASHTO S/5.5 distribution factor; the second is based on the elastic distribution factor computed using the grillage analysis.

The "FILENAME: ALFD100" prompt reminds you of the name of the file currently being used: before running the analysis routines, IBR always saves all data back to the file with this name. If you want to save to another filename, go to the FILES menu, enter the SAVE window, use the BACKSPACE key to erase the old filename, then type in the new filename (must be no longer than seven letters)—the active file will now be that of this newly created file. The analysis routines of IBR create many additional files during their execution. Only the file with the extender ".RAT" needs to be preserved for future IBR runs. This means that, if you wish to save space on your hard disk, you can delete all of the other files created by IBR.

The grillage analysis is performed with the load and resistance factors previously defined. (These may be checked in the RATING EQUATION window, discussed below.) At this time, you may enter the GRID ANALYSIS window and run the grillage analysis, to see graphically how the moment envelope surface is generated from the multiple placements of two side-by-side 3S2 rating trucks. The grillage performs a single girder analysis, as well as a system analysis, so that lateral distribution factors can be established. The trucks are placed at various longitudinal and transverse positions on the bridge deck, and the loads are apportioned to their nearby grillage nodes.

The RATING EQUATION window summarizes all of the load factors that have been defined this far, and puts them in the rating format stipulated in the *Guide Specifications*. You may change these values in this window.

Because the bridge is not made from elastic-perfectly plastic sections, the rating factors from the grillage analysis must be reduced, and a single girder analysis is performed in the RESIDUAL DAMAGE ANALYSIS window. The bridge is then rated against an inelastic serviceability limit state. For this example, we will be rating against a permanent deflection limit state such that the span to permanent span deflection ratio, $K = L/D$, cannot fall below 300—the level of visible detection.

For this example, the inelastic distribution factor of 1.28 was chosen from the LATERAL LOAD DISTRIBUTION window, and will be used to perform the RDA single girder analysis. (If you have forgotten the factors being used, exit the RDA window and look over the other windows under this ANALYSIS menu.) The shakedown limit rating factor from the grillage is 2.95: a value of 2.24 will be entered for R.F. in the RESIDUAL DAMAGE ANALYSIS window. For purposes of speed, the truck will advance 5 ft with each truck placement (1-ft increments take longer to run, but greater accuracy may result). The maximum number of passes of the truck (five) will be selected so that shakedown can be observed.

Be sure that you've changed the values of *Ix* for Section #2 to equal that of Section #1 in the SECTIONS window of GEOMETRY, as mentioned before.

Run RDA now. The elastic envelopes are first generated in order to locate potential midspan hinge locations. Next, the results of the shear and moment envelopes are reported and you may scan these values using the left and right arrow keys. Now, the truck will make five crossings, allowing inelastic rotations to take place. The inelastic rotations for each potential hinge are shown—these values are for the next truck advance (i.e., when the truck is at 45', these values are the results of the truck being placed at 50'). Whenever yielding takes place, the program stops and the maximum elastic rotation is computed (this is the rotation associated with the full plastic moment assuming an elastoplastic moment-rotation curve—it is a necessary parameter for solving the conjugate beam equations of RDA). In this case, the first inelastic pass results in yielding at all three possible hinge points.

Note that no change to the inelastic rotations occurs between the fourth and fifth pass, so that shakedown has occurred. The next screen provides the residual damage deflections, moment, and shear. The elastic, residual, and total moment envelopes for the 3S2 loading follow. Finally, the inelastic serviceability limit state, $K = 300$, is seen to be satisfied since L/D is 314 for this case. Note the relative magnitudes of the inelastic and the maximum elastic rotations, which are summarized on this final RDA screen.

Additional Examples

A three-span beam subjected to a single concentrated shakedown load is given as THREE.RAT. Also, a simple span composite beam is loaded with its theoretical collapse load in SIMPLE.RAT—note that for the case of ascending inelastic branches of composite beams, it is necessary to check the resulting moment diagrams of these structures, in order to verify that *Mp* has not been exceeded. In both of these examples, the shakedown strength limit load is seen to result in $K = L/D$ ratios below 300. Therefore, RDA should be rerun several times, each time with a lower value of R.F., until K is at the desired level.

SOFTWARE PACKAGE DISTRIBUTION

The original version of the "Inelastic Bridge Rating Program" software developed by the University of Minnesota for this project is available through the Transportation Research Board. However, TRB assumes no responsibility for the performance of the software and offers no software technical support.

The software is available only in the $3\frac{1}{2}$ high-density IBM-PC floppy diskette format. To order a copy send a check for $5.00, payable to the Transportation Research Board, to cover duplication, handling and shipping costs. Include the name of this NCHRP report and the above title with your request and mail to:

CRP-Software, 12-28(12)
c/o Transportation Research Board
2101 Constitution Avenue, NW
Washington, D.C. 20418

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