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# Manual for Bridge Rating Through Load Testing

*This NCHRP digest describes the research findings from NCHRP Project 12-28 (13) A, "Bridge Rating Through Nondestructive Load Testing," conducted by A.G. Lichtenstein and Associates, Inc. The project developed and documented processes for performing load tests and using the test results to calculate bridge ratings. The research results are presented in the form of a manual, which provides guidelines for integrating the load testing of bridges with their load rating. The manual is supplemented with a technical report, which presents detailed data on two major technical areas—evaluating unintended composite action and establishing target proof load levels.*

## FOREWORD

This manual presents the results of a study on the use of nondestructive load testing to evaluate the carrying capacity of bridges. Recommended procedures for performing load tests and for using the results to calculate load ratings are included. The contents of this manual will be of immediate interest to bridge and structural engineers, bridge inspectors, traffic engineers, and others interested in bridge safety and the movement of traffic across bridges.

Nondestructive load testing of bridges has been used primarily as a research tool to provide better understanding of the way in which loads are carried by, and distributed through, the bridge structure. In some cases, load testing has been used to assist in the determination of bridge load-carrying capacity. From such tests, some structures have been found to possess greater load-carrying capacity than predicted by conventional analytical load-rating procedures. Load-rating procedures that incorporate load test results have potential for demonstrating higher load capacity for many structures that would otherwise be determined to require load-posting based on conventional analysis alone.

NCHRP Project 12-28(13), "Nondestructive Load Testing for Bridge Evaluation and Rating," was initiated in 1987 with the objective of developing guidelines for nondestructive load testing of highway bridges to augment the analytical rating process. A 1990 follow-on project, NCHRP Project 12-28(13)A, "Bridge Rating Through Nondestructive Load Testing," developed and documented processes for performing load tests and using the test results to calculate bridge ratings. Project 12-28(13)A is the basis for this manual. The research was performed by A.G. Lichtenstein and Associates, Inc., of Paramus, New Jersey.

The research results are presented in the form of a manual that introduces the concept of nondestructive load testing (including the two major types of tests: diagnostic tests and proof tests), describes the appropriate selection of candidate bridges, provides detailed procedures for load testing, and describes how to use load test results to develop a load rating for a bridge. The manual also includes illustrative examples of both diagnostic-load-test and proof-load-test procedures. Finally, information on special topics, including evaluations for live load impact; fatigue life testing of steel bridges; and unintended composite action of bridges (and how it may improve the load rating for a bridge) is presented in the appendices.

A workshop related to the load rating manual also was developed, and the workshop materials include an instructor's notebook and a student's notebook. Two pilot workshops were presented during the course of the research (one in Irvine, California and one in Albany, New York). The workshop materials have been turned over to the Federal Highway Administration for a training course that may be offered through the National Highway Institute.

Contained herein as a supplement to the manual is a technical report, which presents detailed data on two major technical areas—evaluating unintended composite action and establishing target proof load levels. The pages of the technical report are shaded in the margins to help the reader more easily distinguish it from the manual.

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# CHAPTER 1

# INTRODUCTION

## 1.1 OBJECTIVES OF NONDESTRUCTIVE LOAD TESTS

The actual performance of most bridges is better than theory dictates. When a structure's computed theoretical safe service live load capacity is less than desirable, it may be beneficial to the owner to take advantage of some of the bridge's inherent extra capacity. The objectives of nondestructive load testing are to quantify in a scientific manner the enhanced capacity and determine the portion of this enhanced capacity that can be reliably used to establish the bridge's load rating. The theoretical bridge ratings for the Inventory and Operating levels can then be adjusted to reflect the results of the nondestructive load test.

The objectives of the Manual are to present recommended procedures for performing nondestructive bridge load tests and for incorporating the load test results into the bridge load rating process.

## 1.2 WHAT IS A NONDESTRUCTIVE LOAD TEST?

Nondestructive load testing is the observation and measurement of the response of a bridge subjected to controlled and predetermined loadings without causing changes in the elastic response of the structure. The principle of load testing is simply the comparison of the field response of the bridge under the test loads with its theoretical performance as predicted by analysis.

Basically, two types of nondestructive load tests are available: diagnostic and proof. Both types utilize loads, instruments and calculations; but they differ in the manner in which the test results are applied to obtain the live load rating of the tested structure.

Under the diagnostic type test, the selected load is placed at designated locations on the bridge and the effects of this load on individual members of the bridge are measured by the instrumentation attached to these members. The resulting field measured effects are then compared to effects computed based on the applied loading and standard engineering analysis principles and practices.

For proof load tests, the bridge is carefully and incrementally loaded in the field until the bridge approaches its elastic limit. At this point, the loading is stopped and the maximum applied load and its position on the bridge is recorded. In some instances, a target proof load is established by office computations, and the load test is discontinued when this goal is reached.

For both the diagnostic and proof tests, the results from the load test are then studied in the office and the original calculated load ratings for the bridge are adjusted or refined accordingly.

## 1.3 WHY USE NONDESTRUCTIVE LOAD TESTS?

Nondestructive load tests may provide sufficient data to establish safe service live load levels for older bridges. In some instances the make-up of the bridge members and/or the members response to loading cannot be determined because of lack of existing as-built information. In other cases theoretical rating calculations may result in a low live load capacity requiring posting of the rated bridge, and nondestructive load test may provide a more realistic safe service live load capacity. In some instances, the test results will indicate that the actual safe service live load capacity is less than computed, thus alerting the bridge owners to speedy action to reinforce or close the bridge. Existing bridges that have been strengthened over the years, may not be accurately load rated due to the unknown interaction of the various elements of the repaired structure in supporting live loads. Again, nondestructive load tests can help clear up the performance of such a bridge, and generally improve its load rating.

## 1.4 APPLICATION OF NONDESTRUCTIVE LOAD TESTS

Nondestructive load testing of bridges has been in practice in the USA and many other parts of the world for many years. Most bridge testing is considered an art performed by experienced engineers familiar with their structures and behavior, who then evaluate and interpret the test results based on their knowledge and experience rather than through prescribed procedures and formulae.

Nondestructive bridge load testing should not be attempted by inexperienced personnel. Common sense, good engineering judgment and sound analytical principles are not to be ignored. The load path through the bridge must be clearly identified before beginning a test. The various conditions that may contribute to an enhanced capacity of the bridge should be identified and understood when performing diagnostic tests and the application of loads and measurement of response during a proof test must be done with care.

Bridge load testing can be a very useful tool for bridge owners. It can save money by permitting the continued use of older bridges at a higher service load level and/or by reducing replacement/upgrading costs. The test results can also issue a warning when the bridge is not performing properly.

On the other hand, unfamiliarity with nondestructive load testing practices causes some bridge owners to be apprehensive that damage may be inflicted to the bridges by the testing activities. Other bridge owners are concerned that the evaluation of test results is too arbitrary and may result in unsafe conclusions. Some guidelines and procedures are needed to encourage and standardize bridge load testing.

## 1.5 OVERVIEW OF MANUAL

The Manual for Bridge Rating through Load Testing is intended for use by bridge owners as a guideline for the establishment of a realistic safe service live load capacity for their bridges through the use of Nondestructive Field Load Testing. The intent and an overview of the Manual are presented in Chapter 1, "Introduction". Chapter 2, "General Considerations", describes the diagnostic and proof tests and the various types of loading vehicles, recording equipment and other related items. Also included in this chapter is a discussion of the types of bridges that could be tested beneficially, as well as a section on when not to utilize load tests.

Chapter 3 contains important provisions on how to explain the variations between the theoretical calculations made in the office and the actual measurements produced by the nondestructive load tests in the field. Chapter 4, "General Load Testing Procedures," covers the planning of the actual load test activity, execution of the load test, evaluation and preparation of a Report. A generic description of the test equipment and types of measurements is included in Chapter 5.

The detailed procedures for diagnostic load tests including the interpretation of the test results, both for Inventory and Operating levels, are included in Chapter 6. Similarly, the procedures for proof load testing are included in Chapter 7.

Chapter 8 provides assistance to the bridge owners on how to utilize the results of the load tests in the posting decisions and/or issuance of permits.

Chapter 9 contains examples illustrating nondestructive load tests and the resulting live load rating for both diagnostic and proof load cases when applied to typical highway bridges.

The appendices include listings of pertinent bridge testing literature, field procedures for evaluation of live load impact, and suggestions on the implementation of fatigue life testing of steel bridges.

## 1.6 STANDARD REFERENCES

Sources used in this Manual have been included in the "References" section and have been numbered for ease of reference. The report by Pinjarkar, et. al. (Ref. 22) was a major source of information for portions of this Manual. In addition, a number of standard references are used throughout this Manual. In the following chapters, "Manual" refers to the manual for "Bridge Rating Through Load Testing," "AASHTO Specifications" refers to the AASHTO "Standard Specifications for Highway Bridges" (36), "C/E Manual" refers to the AASHTO "Manual for Condition Evaluation of Bridges" (35), "Guide Specifications" refers to the AASHTO "Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges" (37), and "Fatigue Guide" refers to the AASHTO "Guide Specifications on Fatigue Evaluation of Existing Steel Bridges" (38).

# CHAPTER 2

## GENERAL CONSIDERATIONS

### 2.1 INTRODUCTION

The basic formula for the theoretical rating of a bridge member, as expressed in the C/E Manual, Article 6.5.1, is as follows:

$$RF = \frac{C - A_1 D}{A_2 L_R (1 + I)} \tag{2-1}$$

where:

C denotes the capacity of the member to resist the applied load effects and is usually defined from available information in the Bridge Record. If more accurate characteristics are required, such as yield or ultimate strength of steel, they can be obtained by laboratory testing of specimens removed from the structure, or other similar means.

D is the dead load effect on the member and is calculated from data on the plans supplemented by field measurements.

$L_R$ is the live load effect on the member.

I is the impact factor to be used with the live load effect. The impact factor is generally the AASHTO Specifications formula impact unless field tests are performed in accordance with Appendix B to establish an impact value based on site conditions.

$A_1$ and $A_2$ are factors applied to dead load and live load effects respectively. These factors vary depending on the rating method used (Allowable Stress, Load Factor or Load and Resistance Factor).

RF is the rating factor for live-load effect on the member being evaluated. The rating factor multiplied by the rating vehicle weight (in tons) gives the rating of the member for that vehicle configuration. The lowest rated member should be used as the overall bridge load rating.

Load tests can be utilized to improve the rater's understanding of the bridge member's response to live load applications by directly measuring the live load effect, $L_R$, in equation 2-1. Load tests can also provide a more realistic determination of the capacity of the existing bridge to carry live loads by measuring the equivalent of the quantity ($C - A_1 D$) in equation 2-1.

Nondestructive load tests can be diagnostic types that determine how a bridge responds under known applied loads, and proof tests that establish that some level of capacity actually exists in the bridge by virtue of its performance. Load rating through load testing should also address the different rating methods and rating levels used by bridge owners.

## 2.2 DIAGNOSTIC TESTS

Diagnostic testing measures the load effects (moment, shear, axial force, stress or deflection) occurring in bridge members in response to the applied loads. In terms of equation 2-1, the live load effect in the member is measured directly during a diagnostic test. Load tests to verify predicted load effects are the most frequent examples of the types of bridge testing conducted in recent years. Such tests are generally performed under controlled and known loads with traffic temporarily suspended. In some tests, random traffic is used with the bridge response recorded in the form of statistical data.

Diagnostic load tests include the measurement of load effect in one or more critical bridge members and comparing the measured load effects with that computed by an analytical model (theory). The difference between the theoretical and measured load effects will then be utilized in the establishment of the load rating for the bridge member tested.

Diagnostic tests are usually associated with one of the following situations:

1. Uncertainties about bridge behavior. Bridge structural analysis requires assumptions about material properties, boundary conditions, effectiveness of repairs, unintended composite actions, and the influence of damage and/or deterioration. Diagnostic tests can be used to verify some of the assumptions made by the rating engineer.

2. Routine parametric determinations. Several parameters, such as load distribution and impact factors, are routinely used in load-rating bridges. Generally, the design provisions of the AASHTO Specifications are used in determining values for these parameters. Diagnostic field tests can provide a more accurate determination of the above noted parameters and specification requirements.

Diagnostic tests serve to verify and adjust the predictions of an analytical model. Measured responses should agree with predictions or some rational explanation for any differences that are known to be conservative should be provided. In addition to model comparison, diagnostic tests should include the repetition of load cases in order to establish conservative values for the load effects measured in the field. Typical diagnostic load tests are described in references 1-7 and 28-32. These tests are reviewed in Appendix A.

There are many reported examples of contributions from nonstructural components, such as noncomposite deck slabs or parapets enhancing a structural member's behavior at low load levels, but which may cease to participate at high load levels. During a diagnostic load test, the applied load should be sufficiently high to properly model the physical behavior of the bridge at the rating load level.

## 2.3 PROOF TESTS

The historic "dramatic" form of bridge testing is by proof loading in which the bridge is subjected to specific loads, and observations are made to determine if the bridge carries these loads without damage. In effect, proof testing measures the capacity of the bridge to carry live load, at least with regard to a particular test load pattern. In terms of equation 2-1, the net capacity to carry live load, ($C$-$A_1D$), is measured during a proof test. Loads should also be applied in increments and the bridge monitored to provide early warning of possible distress or nonlinear behavior.

The proof test is terminated when: (1) a predetermined maximum load has been reached; or (2) the bridge exhibits the onset of non-linear behavior, or other visible signs of distress. Formulas for load rating through proof tests are given in Chapter 7. Although simple in concept, proof testing will in fact, require careful preparation and experienced personnel for implementation. Caution is required to avoid causing damage to the structure or injury to personnel or the public.

Despite these difficulties, proof testing, when applied correctly and carefully, has become a valuable tool in checking the load capacity of existing bridges in service. Proof testing existing bridges has been widely used by the Ontario Ministry of Transportation and the Florida Department of Transportation. In Switzerland, every new bridge is subject to a proof test before its opening to traffic. Typical proof-load tests are described in references 11-14. These tests are reviewed in Appendix A.

## 2.4 OTHER TESTS

### 2.4.1 General

The primary tests used for load rating bridges through nondestructive load tests are the diagnostic and proof load tests described above. Other tests may be performed in conjunction with or independent of the diagnostic and proof load tests to provide additional information on the dead and live loads carried by the bridge and the dynamic response characteristics of the bridge. Some of these other tests are described in this Section.

Field and laboratory tests may be used to give information on material characteristics as well as the extent of deterioration. These tests have utilized acoustic emissions, ultrasonics, magnetic crack definer, radar, and similar techniques. Such tests are described in the C/E Manual.

### 2.4.2 Load Identification

The margin of safety in load rating should provide for possible overloading, the volume of trucks, and the number of heavy trucks. The actual site survey of truck weights and frequency can be determined by weigh-in-motion systems (WIM) including devices which make use of the bridge as the scale. WIM techniques utilize axle sensors and the assumed linear load-response parameters of the bridge to determine axle and gross loads of passing vehicles. Numerous tests have been done to confirm the weigh-in-motion concept and recent tests by states and FHWA have shown how truck related statistics can be obtained and utilized in bridge response validation. The AASHTO Fatigue Guide also indicates how WIM data can be utilized in such applications.

### 2.4.3 Unusual Forces

Tests for the effects of forces resulting from stream flow, ice, wind pressure, seismic action and thermal response have also been conducted. Since such forces are not part of the usual load rating procedures, these tests are not considered further in this Manual.

### 2.4.4 Dead Load Effects

Dead load stresses play a major role in load ratings. Since the loads are already applied it is difficult to measure their effects. One approach is the use of residual stress gages, which are designed to obtain the dead load stresses present in a steel member. The dead load effects could also be established by jacking the structure but this procedure is dangerous and not recommended.

### 2.4.5 Dynamic Effects

A bridge may be tested under dynamic loadings for several reasons. Earthquake response is strongly influenced by bridge frequency and damping. Another dynamic behavior concerns fatigue assessment where damage may be influenced by repeated stress oscillations. The principal results of a dynamic response test may be the bridge natural frequencies and corresponding mode shapes as well as damping values.

Dynamic tests may be conducted by means of moving loads, portable sinusoidal shakers, sudden release of applied deflections, sudden stopping of vehicles by braking and impulse devices such as hammers.

### 2.4.6 Impact

Normally, the AASHTO Specifications impact factor will be used in load rating bridges based on nondestructive load tests. The actual impact factor is influenced primarily by the surface roughness of the deck and the presence of bumps on the bridge approach and to a lesser extent by the bridge frequency. Procedures for the field evaluation of live load impact are contained in Appendix B.

### 2.4.7 Fatigue

Load rating is separate and distinct from the evaluation of safe remaining fatigue life of steel highway bridges. In assessing the safe remaining fatigue life of steel bridges, both the range of stress and the number of stress cycles acting on a member need to be evaluated. Thus, field load testing can provide data for both of these parameters. It should be noted that stresses calculated in accordance with AASHTO Specifications are usually higher than actual measured stresses. However, the actual number of stress cycles in the field may be higher than those required by the AASHTO Specifications. The AASHTO Fatigue Guide provides that measured stresses can be used in place of computed stresses in making remaining life assessments.

Field tests may be the only accurate way to determine stress spectra in older bridges. In addition, stress spectra may be obtained for distortion induced stresses which have been found to be a major cause of distress in steel bridges and can lead to cracking of components and eventual failure. Also, tests may be performed before and after instituting retrofits to check the efficiency of such changes. Appendix C describes procedures for the fatigue life testing of steel bridges.

## 2.5 LOAD APPLICATION

### 2.5.1 General

It is important that any loading system consider the safety of personnel and the avoidance of heavy damage or catastrophic failure of the structure. A convenient application of load for either a diagnostic or a proof test is by static loads, stationary or movable. If a stationary load is applied to the bridge, the load cannot be easily reduced once a peak capacity level is reached.

A good loading system for both diagnostic and proof load tests should possess the following desirable characteristics: 1) it should be representative of the rating vehicles; 2) the load should be adjustable in magnitude; 3) loads should be maneuverable and; 4) loads should allow for repeatability so that both linearity of bridge response with repeated loading as well as return of response to zero following load removal can be checked. Typical loading systems are described in this section.

### 2.5.2 Stationary Loading

Stationary loads have been applied to bridges by placing blocks of known weight by means of a crane positioned outside the bridge (Fig. 2-1). There are several disadvantages to stationary loads in terms of their maneuverability to different load positions and their removal, if needed. Also, if the loads are applied very slowly, temperature effects should be considered for certain types of structures.

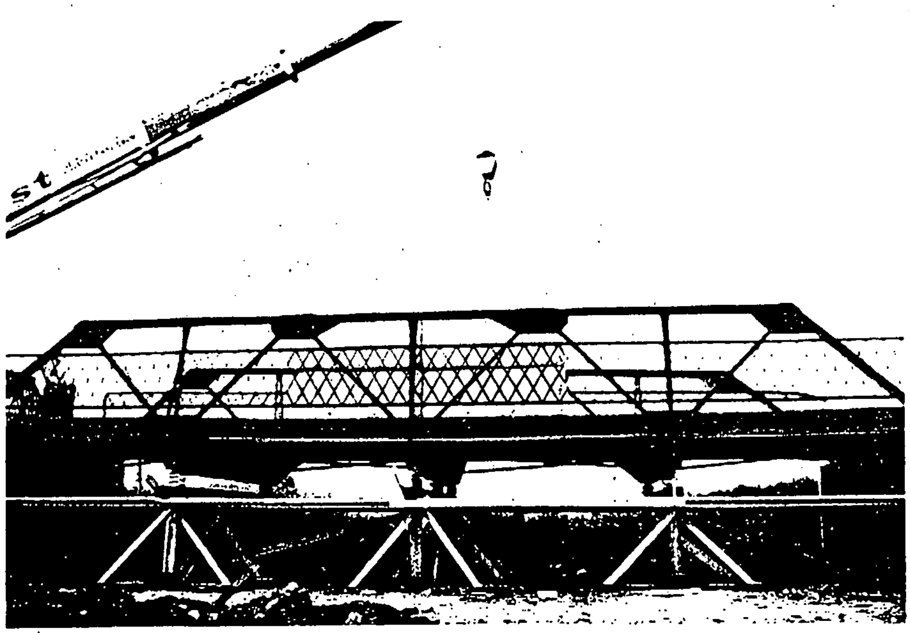


FIGURE 2-1: Concrete blocks used for load testing a bridge. Note safety scaffolding underneath the bridge.

Another stationary load method which has been used especially for destructive tests of bridges is the use of hydraulic jacks with the load applied through cables anchored in the ground or with heavy weights (Fig. 2-2). Load is monitored with calibrated load cells. One advantage of this approach is that as large deflections occur, the applied load will automatically be reduced avoiding damage to the test equipment and the collapse of the bridge.

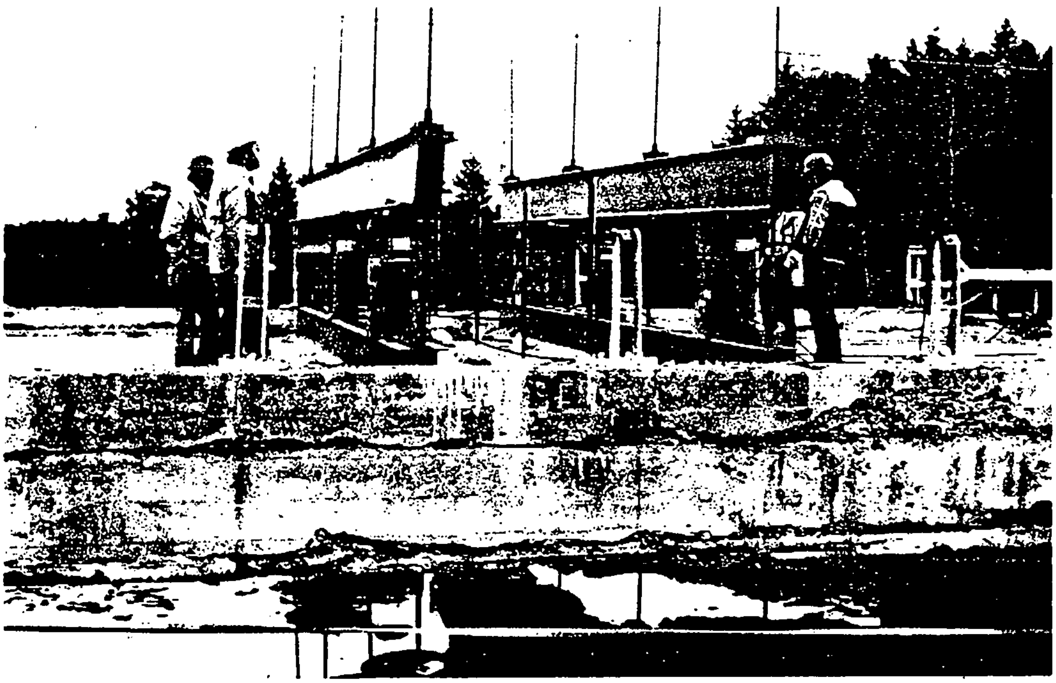


FIGURE 2-2: Example showing the use of a hydraulic jack used in load testing of bridges by the New York State DOT.

### 2.5.3 Movable Loading

A movable load is one that can be easily applied at different positions, both transversely and longitudinally along the bridge to simulate all possible load cases. These positions should be determined by the engineer prior to the test. The use of a movable load can provide information for constructing influence lines and moment, shear and axial load envelopes for individual bridge members. Generally, one or more dump trucks or specially-designed test trucks are used.

In this type of loading, the test vehicle should be brought onto the bridge at crawling speed (5 mph or less), and the structural response should be monitored continuously. The test vehicle may be stopped in predetermined positions on the bridge and the response measured under static load conditions. A vehicle of known axle loads and spacings which simulates either the AASHTO Specifications load model, the C/E Manual legal loads, or other legal vehicles is an example of a typical test vehicle. The vehicle may be fixed in total weight (Fig. 2-3) or else may have provisions for addition of blocks to change its weight during testing (Fig. 2-4). Also, provision for shifting weights to the different axles may allow for changing loads on parts of the structure. If the test loads exceed the legal load limit, some difficulty will be encountered in transporting the loads to the test site. One approach is to use water as the load medium, although its low density relative to concrete blocks may require a more bulky test vehicle.

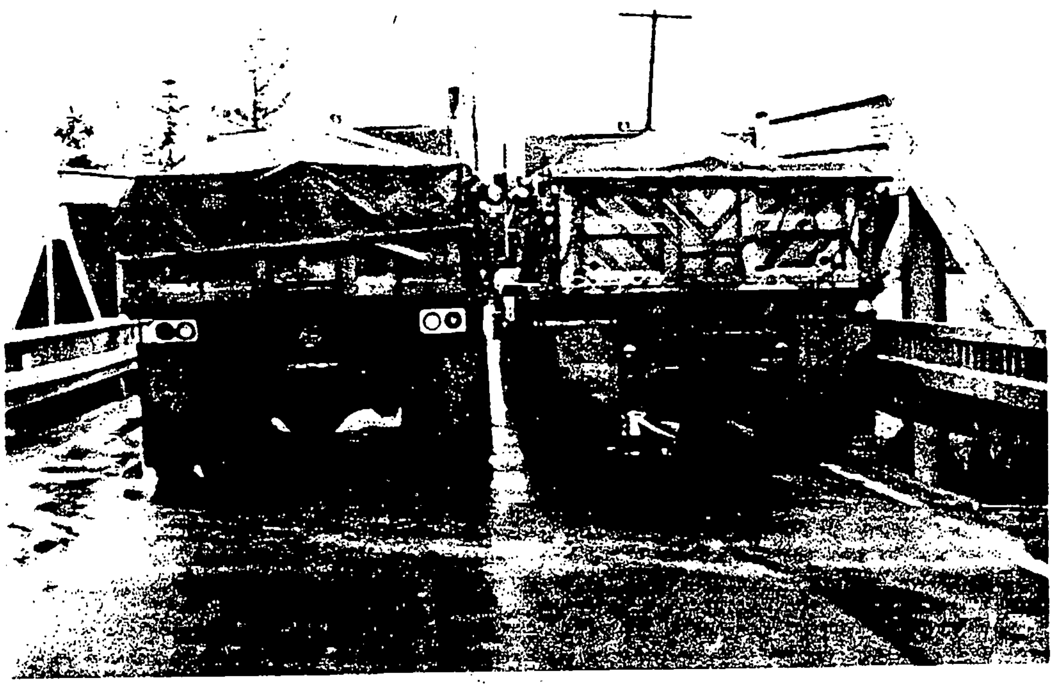


FIGURE 2-3: Dump trucks, filled with sand, used by the New York State DOT for load testing of bridges.

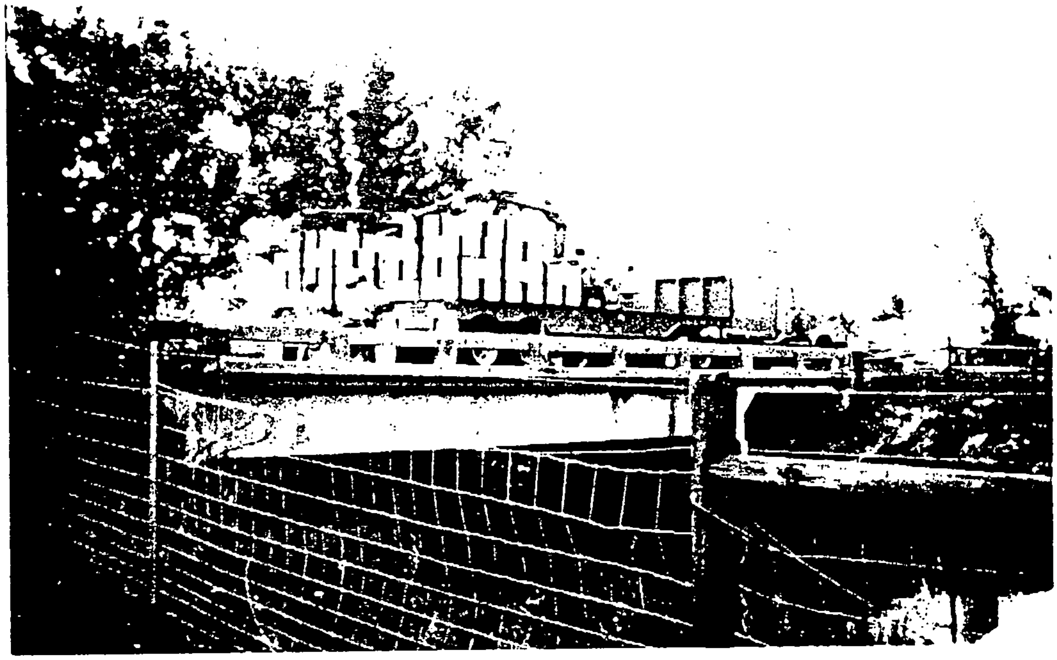


FIGURE 2-4: Proof load testing of Sunshine State Parkway Bridge No. 37, Florida

### 2.5.4 Moving Vehicle Tests

Another means of applying test loads to a bridge is by using a test vehicle of known dimensions and running it across the bridge at normal operating speeds. This should be done along different transverse paths on the bridge to create an influence surface for subsequent load checking. The speed of the test vehicle should also be varied to envelop the maximum impact effects. Moving loads may also provide information on the impact allowance (see Appendix B) as well as the bridge frequency. Comparisons between static, crawl, and moving load effects are then needed. It is important whenever comparisons of static and dynamic results are made that the identical load path is maintained. This is especially important when the output is measured in terms of stress response, since small deviations in the path of a vehicle can change the lateral load distribution and hence the stresses in individual components.

## 2.6 BRIDGES WHICH COULD BENEFIT FROM LOAD TESTS

### 2.6.1 General

A review of bridge load tests conducted in the United States, Canada and other countries (22, 26) indicate that some generalizations can be made regarding the behavior of existing bridges and their potential as candidates for nondestructive load testing. In general, lower stresses due to applied live loads when compared with calculations have been found during bridge load tests.

This section summarizes the reported behavior of various types of bridges when load tested and suggests appropriate nondestructive load tests. A detailed discussion of the specific factors which may contribute to the enhanced load-carrying capacity of bridges is contained in Chapter 3.

### 2.6.2 Slab Bridges

Experience has shown that load capacities of slab bridges, as determined by load tests, are generally several times higher than predicted by simplified analytical methods. Experience indicates that analytical approximations such as one-way slab action do not reflect the true behavior of many slab bridges. In cases where the bridge width is equal to or greater than span length then plate behavior of the slab should be taken into account. When as-built drawings with structural details are available, diagnostic load tests may be helpful in verifying assumptions used in the analytical rating. When this information is not available, proof load tests have been helpful in providing a realistic live load capacity for slab bridges.

### 2.6.3 Multi-Stringer Bridges

In stringer bridges the distribution of applied wheel loads to the supporting stringers is an important factor in computing the load rating. Distribution factors used for rating calculations in the AASHTO Specifications are mostly conservative and are intended primarily for design purposes. In addition, the two-way stiffness of deck slabs provides significant lateral and longitudinal distribution of wheel loads, a factor neglected during design. The actual live load stresses in the supporting members are often lower than their design values.

Composite action can greatly increase the stiffness of steel stringer-and-slab bridges. Experience has shown that composite action will exist in these bridges whether or not shear connections were provided by design. Service load stresses in the stringers of non-composite bridges are reduced due to the unintended composite action. At ultimate loads, however, the composite action can break down if shear connectors are not used, due to slip at the stringer-slab interface, and thus cannot be relied upon for increased ultimate capacity. This issue is discussed in more detail in Chapter 3.

In many instances, unintended end restraint at stringer and girder supports may reduce live load stresses in a span. End restraints are caused by friction at the bearings, rockers that are frozen or out of position, and by the butting of ends of beams against backwalls or against adjacent beams. Bearing restraints are difficult to predict without a load test. Even with a load test they are not always reliable as the bearings may be jarred loose by impact or be replaced in the future.

The presence of concrete parapets and "New Jersey" barriers acting integrally with the deck may significantly stiffen the outside girders, particularly if they are continuous, and result in increased resistance of the bridge cross section to live load.

Either diagnostic or proof load tests may be helpful in establishing a realistic service live load for multi-stringer bridges.

### 2.6.4 Two-Girder Bridges

The girders and the individual members of the floor system and their connections should be evaluated for the rating vehicle. In some cases the stringers, designed to act as simple spans with web connections at the floorbeams, may develop partial continuity at the ends due to the deck slab being made continuous over the transverse floorbeams. The continuous deck slab provides rotational stiffness at the ends of the stringers resulting in lower positive live load moments in the stringers. The ends of members designed as simple spans, at which continuity is present, should be evaluated.

Diagnostic and/or proof load tests may be helpful in establishing a realistic service live load for this type of bridge.

### 2.6.5 Truss Bridges

Trusses are usually analyzed as idealized two-dimensional structures with pin connected joints. Most of the trusses built in the U.S. after 1900 have riveted joints and no provisions for end rotation. These trusses are actually stiffer than pin connected trusses. Also, the load capacity of individual compression members is increased due to end fixity. Significant stiffening of the truss chords by the floor system and bracing has been demonstrated by load tests. Experience has indicated that load test results will not significantly change the rating of top chord or end diagonal members (see for example Reference 7).

Generally, a diagnostic load test would be used to determine truss performance relative to that predicted by computations. Proof testing may also be used, but only after the deck, stringers, floorbeams and connections are evaluated to determine their ability to carry the proposed proof load.

### 2.6.6 Arch Bridges

Older stone arches are of the voussoir type consisting of truncated wedge shaped stones placed with or without mortared joints. The design of these arches was based on rules-of-thumb or semi-empirical formulas. Proof load tests are useful in establishing the load capacity of such bridges.

### 2.6.7 Rigid Frame Bridges

Because of the nature of rigid frame design and construction, proof-load testing is the simplest approach to establishing a safe service load for this type of bridge.

### 2.6.8 Longspan Bridges

For longspan bridges, the live load is generally a small percentage of the total load carried by the bridge. Procedures for load testing such bridges are beyond the scope of this manual.

### 2.6.9 Timber Bridges

Very few load tests have been performed on timber bridges. The load-carrying capacity of timber bridges is time dependent, generally decreasing with age.

Proof-load testing is suggested for establishing a safe service load rating for timber bridges.

## 2.7 WHEN NOT TO LOAD TEST

Prior to load testing, a thorough evaluation of the physical condition of the bridge followed by load rating calculations should be carried out, and potential failure modes should be determined. If the failure could be sudden, without warning, proof testing should not be used. This problem is discussed further in Section 2.8.

Where a bridge suffers advanced deterioration, calculations may show the bridge to be unsafe for even a light test vehicle, and such a marginal bridge may fail under the test load.

Non-redundant steel bridges with corrosion damage will require inspection of their critical members and links prior to a load test to ensure safety during the load test. In many non-redundant truss bridges the condition of the pins, hangers, hinges and eyebar heads may be difficult to evaluate since closely packed truss joints often make it difficult to perform full visual and tactile inspections, and non-destructive tests of the components. It would not be appropriate to load test such bridges if the condition of critical components and connections cannot be evaluated.

In summary, the following bridge conditions may not be suitable for load tests:

- The cost of testing reaches or exceeds the cost of bridge rehabilitation.
- The bridge, according to calculations, cannot sustain even the lowest level of load.
- Calculations of weak components of the bridge indicate that a field test is unlikely to show the prospect of improvement in load-carrying capacity.
- In the case of concrete beam bridges, there is the possibility of sudden shear type of failure.
- The forces due to restrained volume changes from temperature induced stresses may not be accounted for by load tests. Note that significant strains and corresponding stresses induced by temperature changes could invalidate load test results especially when end bearings are frozen.
- There are frozen joints and bearing which could cause sudden release of energy during a load test.
- Load tests may be impractical because of inadequate access to the span.
- Soil and foundation conditions are suspect. The bridge has severely deteriorated piers and pier caps, especially at expansion joints where water and salt have caused severe corrosion of reinforcement.

## 2.8 BRIDGE SAFETY DURING LOAD TESTS

### 2.8.1 General

An element of risk is inherent in all load testing, especially in proof load testing of bridges whose load paths and behavior are not clearly identifiable beforehand. Also, bridges exhibiting advanced deterioration of critical structural elements and bridges where there is no prior information on material strengths or as-built details, can be considered as risk prone. The bridge owner and rating engineer must be aware of the risks and their consequences. In assessing the risks, consideration should be given to possible structural damage, safety of personnel, loss of equipment, traffic disruption, and possible load posting. The degree of risk involved depends upon the bridge type, its location, loading method, condition, amount of deterioration, and anticipated behavior. For example, the degree of risk involved due to failure of a secondary member or a floorbeam is not the same as that due to failure of a main member. The risks involved can be classified as follows:

**Minimum:** Bridge sustains superficial damage requiring minimum repairs. No equipment damage or loss of life.

**Medium:** Bridge sustains tolerable damage requiring minor repairs and traffic disruption. Possible equipment damage but no loss of life.

**Major:** Bridge sustains significant damage requiring major repairs and rerouting of traffic for an extended period. Possible loss of equipment and loss of life.

The risks can be minimized by judicious selection of test methods; for example, by applying a proof test load in smaller increments and monitoring the bridge response very closely for possible signs of distress.

In certain situations, safety shoring may be erected underneath the bridge to provide support in the event of a excessive deflection. The safety shoring should be independently supported and should not interfere with the bridge movements during testing.

### 2.8.2 Redundancy

Redundancy can generally be defined as the reserve strength available for preventing failure of the entire bridge upon failure of a single element thereof. Redundancy can also be defined as the degree and safety of alternate load paths, or redundancy mechanisms, available to support the bridge following failure of critical load-carrying members or components.

Redundancy cannot be determined by load testing, but must be determined analytically by considering damage to or removal of various bridge components. Redistribution of loads can be determined by load tests. The knowledge that a damaged bridge has the ability to redistribute forces and maintain load-carrying capacity is important in establishing a safe service load level for the bridge.

### 2.8.3 Fracture Critical Members

Prior to any load test, fracture critical steel bridge members should be identified and inspected to determine whether cracking exists. If cracks are found, proof load testing should not be done.

Fracture critical members and fatigue-prone details cannot be evaluated directly by means of load tests. Details suffering from notches, defective welds, improper fabrication, or lack fracture toughness of base-metal or weld-metal may cause reduced fatigue strength and corresponding reduced design life. In the determination of load rating, allowances should be made for fatigue and fracture considerations.

Bridge details prone to fatigue failure should always be evaluated during the rating process. It is possible to estimate the remaining life of a bridge by analyzing critical details in light of the number of stress cycles they have experienced in their lifetime. The theoretical remaining stress cycles are then used to estimate the remaining life. This process could be modified to incorporate findings from load tests to provide a more accurate value for the stresses induced in the member under consideration. Generally, static load tests would give lower stresses than those found by analytical methods. Fatigue may control load rating if details are susceptible to fatigue damage. A discussion of the fatigue life testing of steel bridges is provided in Appendix C.

# CHAPTER 3

# FACTORS WHICH INFLUENCE THE LOAD-CARRYING CAPACITY OF BRIDGES

## 3.1 INTRODUCTION

This section outlines several factors which affect the actual behavior of bridges. Many of these factors are not considered in the design and load rating of bridges, although they may provide enhancements to the bridge response to applied loads. However, such enhancements may not be present at the higher load levels.

In some cases, trying to quantify these potential enhancements may greatly expand the test data needed. For example, to determine how much of the test load is actually carried by secondary members, the parapets and deck would have to be instrumented in addition to the primary members. Alternatively, one can use the tests to obtain accurate lateral load distribution values but rely on section properties verified in the inspection for the actual computation of component rating. Caution is urged before taking the applied test load and extrapolating to a high load condition without considering the factors raised in this chapter.

A summary of the effects of several variables on the load capacity of a bridge is presented in Table 3-1. These and other variables are discussed in detail in this chapter.

TABLE 3-1 Factors Influencing Bridge Load Capacity
(Adapted from Ref. 26)

| Variable \ Bridge Type | Beam and Slab | Concrete Slab | Truss | Box Girder |
|---|---|---|---|---|
| Unintended composite action | P, I/T | N/A | S[1], I/T | P, I/T |
| Participation of parapets and railings | P, A | P, A | N/A | P,A |
| Differences between actual & assumed material properties | S, I/T | S, I/T | S, I/T | S, I/T |
| Participation of bracing and secondary members | S | N/A | S | S |
| Differing support characteristics and unintended continuity | S, I/T | S, I/T | S, I/T | S, I/T |
| Analysis/load distribution effects | P, A | P, A | P, A | P, A |
| Effects of skew | S, A | P[2] | N/A | S, A |

P = Primary factor
S = Secondary factor
A = Include in conventional analysis
I/T = Inspection and or testing needed to verify
N/A = Not applicable

In utilizing the results of a diagnostic load test, the extent to which the factors shown in Table 3-1 are reliable at load levels which are higher than those used during the load test is the key to establishing a safe service load level for the bridge (see Chapter 6). On the other hand, the results of proof testing may be used directly and with the confidence that the bridge actually carried a higher load than the safe service load level established for the bridge (see Chapter 7).

## 3.2 UNINTENDED COMPOSITE ACTION

Most bridges built before 1950 were designed without shear connectors between the main load carrying girders and the concrete deck. Nevertheless, field tests have shown that such a noncomposite deck can participate in composite action with the stringers. However, as the test loads are increased and approach the maximum capacity of the bridge, there have been cases where slippage took place, composite action was lost and a sudden increase in the stresses in the main members occurred. Thus, it is important in calculating the load rating of bridges that behavior and stress values taken at working loads and below are not arbitrarily extrapolated to higher load levels which approach the limiting strength of the member being evaluated. A method for extrapolating diagnostic load test results based on a limiting bond stress between the slab and the stringers will be proposed.

The limiting bond stress between the concrete deck slab and steel girders can be assumed to be 70 psi for concrete with $f'_c = 3{,}000$ psi, when the deck slab rests on top of the girder flanges. For girders with their flanges partially or fully embedded in the deck slab, a limiting bond strength of 100 psi is recommended. As long as the horizontal shear force is less than or equal to this limiting bond stress, composite action can be assumed to act in otherwise noncomposite girders. Application of this recommended procedure is illustrated in Example 9-1, Chapter 9. A detailed presentation of this method and additional background data are given in reference 43.

It should also be noted that load distribution in the girders is affected by whether the girders are composite or noncomposite. Thus, unintended composite action contributes to both the strength of a girder bridge and its ability to distribute loads transversely.

## 3.3 LOAD DISTRIBUTION EFFECTS

An important part of the rating equation concerns the distribution of the live loads to the main load-carrying members of the bridge, and to the individual components of a multi-component member. Typically, in design and rating, load distribution to main supporting members is based on the AASHTO Specifications distribution factors. However, this distribution is affected by several variables. A major aim of diagnostic testing is often to confirm the precise nature of the load distribution. Both lateral (transverse) and longitudinal distribution of loads in a statically indeterminate girder system are functions of relative stiffness.

Another important part of the load distribution is that the factors in the AASHTO Specifications assume a pattern of load which should envelope existing traffic conditions. Thus, the HS configuration simulates closely spaced heavy vehicles. Similarly, the transverse distribution factors are intended to represent side-by-side load occurrence. If the bridge test is performed with only a single test vehicle then some way must be found to simulate a multi-lane loading event. Usually this is done by assuming superposition, i.e., linear behavior and adding the bridge responses from vehicles in different lane positions. It is especially important in extrapolating the diagnostic test results that both the worst traffic position and the multi-lane load cases be investigated. Subsequently, the test results may be used to validate the analytical model, which is in turn then used to extrapolate the load effects in critical components to maximum service levels.

For built-up members, such as truss chords, the components may share the member force unequally. Test results may indicate the actual division of such forces. If the members are ductile, it is often correctly assumed in the rating that the loads are equally shared at failure level. However, a similar extrapolation for brittle members may not be justified.

## 3.4 PARTICIPATION OF PARAPETS, RAILINGS, CURBS AND UTILITIES

Deflections, stresses and load distribution may be affected by the stiffness contributed by nonstructural members such as railings, parapets and barriers, and to a lesser extent by the curbs and utilities on the bridge. Since the such components cannot be relied on at the ultimate load condition, it is important that their contributions be considered in comparing the bridge test-load response with the calculated response.

## 3.5 MATERIAL PROPERTIES DIFFERENCES

Prediction of bridge behavior under test loads requires knowledge of the actual material strength properties which are usually higher than those assumed in design.

Load ratings may be increased through computations which utilize the actual material properties of the bridge rather than those used in design. This rational step may be taken before the decision is made to load test the bridge. The determination of the actual material properties may be done in accordance with procedures described in the C/E Manual.

The cost and time required to obtain the actual material properties may be significant and should be considered in light of the benefits expected. If the steel is found to be significantly stronger than assumed in design, the calculated load rating based on the actual steel properties will be correspondingly higher. On the other hand, differences in the concrete properties will have little impact on the flexural strength of reinforced concrete members which meet the ductility requirements of the AASHTO Specifications. The load rating of timber members would also benefit from an increase in actual strength versus design values, but considerable effort may be required in establishing in-situ timber strength values.

## 3.6 UNINTENDED CONTINUITY

For simply-supported bridges it is assumed that the ends are supported on idealized rollers and do not carry any moment. However, tests have shown that there can be significant end moments attributable to the continuity provided by the deck slab as well as to frozen bearings. Similar end moments may develop at the connections of stringers to floorbeams and floorbeams to main supporting trusses or girders.

For rating purposes it may not be justified to extrapolate the results of a diagnostic test done at moderate load levels. It is quite possible that the enhanced behavior shown by the unintended continuity would not be present at extreme load levels. When such restrain is detected during the test, the test results should be compared with calculations on an analytical model which considers end rotational restraint.

## 3.7 PARTICIPATION OF SECONDARY MEMBERS

Secondary bridge members are those members which are not directly in the load path of the structure, and includes lateral bracing members, diaphragms and wind bracing. In some bridge types, secondary members enhance the load-carrying system by increasing the stiffness of the bridge. For example, rigid floor systems in a truss bridge may help carry portions of the load. Advantage can be taken of the effects of secondary members provided that it can be shown that they are effective at the designated service load level.

## 3.8 EFFECTS OF SKEW

The conventional AASHTO live load distribution factors may not be applicable to girder systems with large skews (20° or more). Jaeger and Bakht (40) have given methods for calculating such distribution factors. Such factors may be needed when using measured strains to obtain distribution factors to ensure that gages are properly located for finding the maximum moment effects.

## 3.9 EFFECTS OF DETERIORATION AND DAMAGE TO STRUCTURAL MEMBERS

In general, common forms of minor deterioration have no significant influence on the load rating of a bridge. However, extensive loss of concrete and/or steel or timber cross-sectional area must be considered. Prior to load tests, it is imperative to perform a thorough overall condition assessment of the bridge to evaluate the observed deterioration. It is often difficult to analyze the effect of observed deterioration on the load-carrying capacity of the bridge, and in such cases load testing can be justified. There are many cases where the load capacity of deteriorated bridges, especially short-span concrete and timber bridges, has been found to be greater than predicted, so that posting or replacement was not required.

Damage to steel, timber or concrete members may also limit the range of linear behavior. Stability is also of concern when there is extensive deterioration in the webs and flanges of steel members.

## 3.10 PORTION OF LOAD CARRIED BY DECK

Depending on the bridge span and the thickness of the deck, there may be a portion of the load carried directly by the deck slab spanning between end supports of the bridge. The deck may, however, not be able to carry significant amounts of load at higher load levels so that any portion carried during the diagnostic test should be determined and transferred back, if necessary, into the main load carrying members.

## 3.11 UNINTENDED ARCHING ACTION DUE TO FROZEN BEARINGS

The effects of unintended arching action are similar to those discussed in connection with unintended continuity. In one test done by Bakht (21), the results showed that even in the presence of elastomeric bearings, the girders may develop enough bearing restraint force to reduce the applied bending moments at midspan by a significant margin. Identification of such contributions to stiffness at the load levels in the test may be necessary to avoid an unjustified extrapolation to higher service load levels. Field load test results should be compared with calculations based on an analytical model which considers these end effects, when such effects are detected during the test.

# CHAPTER 4

# GENERAL LOAD TESTING PROCEDURES

## 4.1 INTRODUCTION

General load testing procedures are given in this chapter, and are intended as a guide. Because of the varying nature of bridge types, structural systems, materials, loadings, and extent of deterioration, the procedures used for any specific bridge would need to be developed based on conditions at the bridge site.

The steps required for rating of bridges through load testing include the following:

Step 1—Preliminary inspection and theoretical rating

Step 2—Development of load test program

Step 3—Planning and preparation for load test

Step 4—Execution of load test

Step 5—Evaluation of load test results

Step 6—Determination of final load rating

Step 7—Reporting

Each of these steps is described in detail below.

## 4.2 PRELIMINARY INSPECTION AND THEORETICAL RATING

### 4.2.1 Preliminary Inspection

The results of a recent field condition inspection of the bridge to be tested, conducted in accordance with the AASHTO C/E Manual, are necessary for use as the base condition for planning and conducting the load test. In some cases, such as where there is loss of bearing at supports or undermining of the substructure load testing may be inapplicable.

The condition inspection should include measurements to determine such factors as displacements, crack widths, misalignments and movements at joints and bearings. Measurements should be made to determine the actual dead loads including additional layers of pavement and other modifications. In addition, the condition of expansion joints, unusual thermal movements, the condition of approaches, and other factors which may effect load testing should be determined.

### 4.2.2 Preliminary Rating

If feasible, the bridge should be load rated by calculations in accordance with the AASHTO C/E Manual and the rating practices of the bridge owner. The analytical model developed at this stage will also be used in evaluating the results of the load test and in establishing the final load rating for the bridge.

Data obtained from field investigations and review of records should be used to calculate the approximate load capacity of a bridge as a whole; to identify critical structural elements, including connection details, and their load capacities; and to evaluate the presence of conditions which may enhance the response of the bridge to applied loads. In addition, alternate load paths and conditions not suitable for load testing should be evaluated. At this point a determination should be made as to whether load testing is a feasible alternative to establishing the load rating of the bridge. If load testing is a realistic option, the above information should be used to select a test method, plan general strategy for evaluation and determination of load rating, and to control the intensity and position of loading during the actual load test.

Calculations should be performed to predict, as far as possible, the response of the bridge to applied loadings before the tests are conducted. This procedure will establish the approximate amount of loading required and the magnitude of expected deflections and strains to be measured in the field. **The procedure to interpret the test results should be determined before the tests are commenced so that the instrumentation can be arranged to provide the relevant data necessary for the calculations.** If calculations to predict test results are based upon design specifications, then the material strengths and stiffness should be adjusted to actual rather than minimum values. The results of tests of in-situ material strengths may be used in the calculations (see Section 3.5).

## 4.3 DEVELOPMENT OF LOAD TEST PROGRAM

### 4.3.1 General

A program for the field load testing of a bridge should be developed based on the results of the preliminary inspection and rating phase described in Section 4.2. A test program should be prepared prior to commencing with a load test, and should include the test objectives, the type of test(s) to be performed and related criteria.

### 4.3.2 Establish Test Objectives

The objectives of the load testing program should be clearly defined in order to effectively select the type of test to be conducted and its related criteria. For example, if the objective of the load test is to confirm assumptions made regarding lateral load distribution, then a diagnostic test is needed.

The field measurements required are also a function of the test objectives. If one of the test objectives is to establish the extent of restraint at the bearings, then rotations at the bearings will need to be measured as loads are applied.

### 4.3.3 Select Type of Test

The choice of load test method depends on several factors including type of bridge, availability of design and as-built details, bridge condition, results of preliminary inspection and rating, reasons for load posting (if any), availability of equipment and funds, and test objectives.

In general, diagnostic tests are recommended if sufficient data and information on as-built bridge details, dimensions, and materials, are available. Diagnostic tests are performed to verify assumptions used in load rating calculations and to establish the extent to which the load-carrying capacity of the bridge has been enhanced when compared with design values. Diagnostic tests are not appropriate when the magnitude of dead load stress or other permanent stresses can not be estimated reliably.

Proof load tests may be performed if as-built details are not available and/or effects of deterioration and other factors cannot be otherwise evaluated.

Another factor to be considered is the level of risk. Generally, diagnostic load tests are conducted at or near the appropriate service load levels, with little associated risk. On the other hand, it should be recognized that, if not properly conducted, proof loading a bridge has a higher risk of failure of one or more bridge components than does diagnostic testing.

## 4.4 PLANNING AND PREPARATION FOR LOAD TEST

### 4.4.1 General

Careful planning and preparation of test activities are required to ensure that test objectives are realized. At this point, the load effects to be measured in the field during the test are identified, instrumentation is selected, personnel requirements are established, and target loadings are defined, all with due regard to safety considerations.

### 4.4.2 Load Effect Measurements

The load effect(s) to be measured during the load test must be selected consistent with the objectives of the load test. Displacements, rotations, strains, crack widths and joint movements are typical load effects which could be directly monitored during the load test. Bending and axial stresses can be determined from strain measurements.

### 4.4.3 Equipment Selection

Instrumentation should be selected consistent with the test load objectives and the load effects to be measured and the availability of equipment. Chapter 5 provides a guide to the selection of load test equipment.

Measurements may be recorded manually or automatically depending on factors such as the size and type of bridge, the location of the instrumentation, the number of readings, and the type of loading used. The number and location of measurement positions should be based on the preliminary rating computations and analysis described in Section 4.2.2.

Instrumentation generally should be limited to the minimum that will provide adequate and accurate information for the proper interpretation of results. The instrumentation used in proof tests may be as simple as the use of deflection gages to monitor bridge deflections as loads are applied.

### 4.4.4 Personnel Requirements

A qualified bridge engineer should be responsible for the planning and execution of the load test. Experience in testing and instrumentation, field investigations and knowledge of bridge structural behavior are required.

Adequate staff should be available to perform the load test, to provide traffic control during the test and to assist in evaluating the results.

### 4.4.5 Loading Requirements

#### 4.4.5.1 General

The magnitude, configuration and position of the test load will vary based on the type of bridge and the type of test conducted. For diagnostic and proof load tests, such information is presented in Chapters 6 and 7, respectively. Some general guidelines are presented below.

#### 4.4.5.2 Magnitude of Load(s)

The test load should stress all critical elements of the bridge (see also Section 4.4.5.3). The test load may consist of stationary dead weights or a fully-loaded vehicle with known weight and axle configuration, or may be applied by hydraulic jacks.

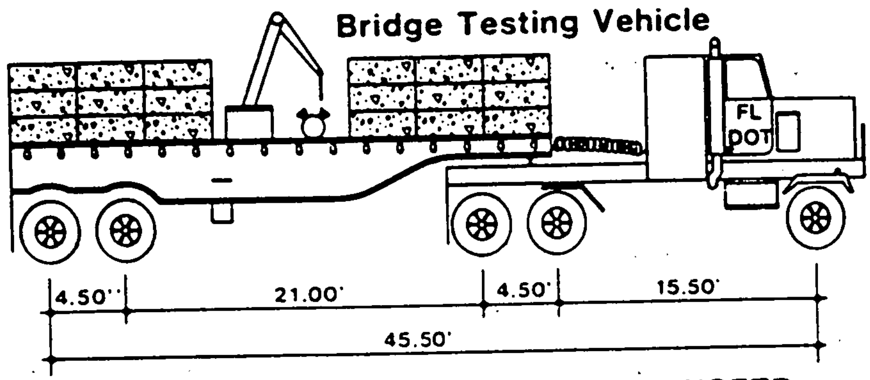
For diagnostic tests, the load is generally low enough such that one or two loaded dump trucks are adequate. The load required for proof testing is considerably higher and may not be available to every bridge owner.

Test vehicles representative of AASHTO legal and rating vehicles are seldom available. The test vehicle and axle loads should be selected to simulate the load effects of the rating vehicles. Some states and agencies may find it useful to develop special vehicle configurations for the purpose of load testing. The special purpose load testing vehicle used by the FLDOT is shown in Figure 4.1.

#### 4.4.5.3 Application of Load(s) and Loading Patterns

The test load(s) should be placed on the bridge at pre-selected locations to obtain the maximum load effect being studied. Alternatively, moving-vehicle loads may be used in various transverse positions on the bridge to produce the maximum load effect at each measurement point. Chapter 6 provides additional guidance for test loads used in diagnostic load tests. For proof load testing, it may be necessary to load multiple lanes simultaneously (see Chapter 7).

The test load(s) configuration and wheel loads should be measured prior to the start of the test, and may be determined by portable truck weight scales. At the completion of a proof load test, the total load placed on the bridge should be confirmed by weighing all components of the maximum proof load.




| WEIGHTS: | |
|---|---|
| 72 Ballast blocks | 154,800 lb. |
| Equipment | 8,200 lb. |
| Trailer | 24,000 lb. |
| Tractor | 17,000 lb. |
| Total | 204,000 lb. |

| LOAD TRANSFER: | |
|---|---|
| 5th wheel | 82,350 lb. |
| Steering axle | 15,630 lb. |
| Drive tandem | 83,720 lb. |
| Trailer tandem | 104,650 lb. |

Note: All weights and dimensions are approximate and for information only.

FIGURE 4.1: Special Purpose Load Testing Vehicle–Florida DOT

#### 4.4.5.4 Provisions for Impact

For diagnostic load tests, the AASHTO Specifications impact factor is used in the load rating calculations. In accordance with the AASHTO C/E Manual, some agencies may wish to establish the dynamic impact factor based on bridge site conditions. A suggested procedure is contained in Appendix B.

Minimum proof load levels must incorporate an allowance for impact (see Chapter 7).

### 4.4.6 Safety and Traffic Control

The safety of test personnel, equipment and the bridge is paramount during the performance of a bridge load test. Precautions should be taken to control and regulate traffic and pedestrians during the test. Generally, public vehicles and pedestrians should not be allowed on or under the bridge during testing.

## 4.5 EXECUTION OF LOAD TEST

### 4.5.1 General

The first step in executing the load test is to install and check out the instrumentation to be used. The time required to install the instrumentation depends on the number of measurement positions, the type of instruments, the accessibility to the measurement positions and the weather conditions. Generally, the instrumentation can be installed without closing the bridge to traffic.

After the instrumentation has been installed, the actual bridge load test may begin. The preferred method of conducting the load test is to close the bridge to all vehicular and pedestrian traffic during the test period, usually 1 to 4 hours. The loads should be applied to the bridge in several increments while observing structural behavior. The bridge may be re-opened to traffic between successive load increments or positions, if necessary, but the instrumentation must be rezeroed before and after each such event.

Because of the costs associated with the load test and the closing of the bridge, precautions should be taken to ensure that accurate and reliable data is obtained during the test. Thus, it is important to monitor the behavior of the bridge, assess the response of the bridge to repeated load positions and to account for temperature changes during the performance of the load test. These are discussed in more detail below.

### 4.5.2 Monitoring Bridge Behavior

Measurements of displacements, strains, rotations, and crack widths should be taken at the start of the bridge load test and at the end of each load increment. A sufficient number of measurements should be made at all possible critical locations to fully evaluate the structural response under each load increment. During the test, measured responses should be compared with predicted response (based upon preliminary calculations) to detect unusual behavior which may warrant changes in the test procedures. Load-deformation response and deflection recovery at critical locations, after each load increment, should be monitored very closely to determine the onset of nonlinear behavior. Once significant nonlinear behavior is observed, the bridge should be unloaded immediately and measurements for the unloaded bridge should be recorded. Temperature and weather conditions should also be recorded during the test.

### 4.5.3 Repeatability of Results

To eliminate secondary effects such as slippage in the connections of multiple component members, critical test load cases should be repeated a minimum of two times or until correlation between each repetition is obtained. The load effects for the repeated load positions should be compared and any deviations explained. Good agreement between the results for repeated load positions generally indicates elastic behavior of the bridge and also provides assurance that the test instrumentation is performing correctly.

### 4.5.4 Temperature Changes

The influence of temperature variations during the load test and other environmental changes such as weather conditions should be accounted for in the load test measurements. It is necessary to compensate for both temperature effects on the instrumentation employed as well as the temperature-induced effects in the structural members. The latter effects are minimized if the duration of the load test is short and the temperature steady.

The use of frequent "no load" cases, where the test load is removed from the structure, is one approach to assessing the impact of temperature changes. These "no load" readings, when connected by straight lines, provide the baseline for the load case readings. It should be noted that the use of temperature compensating gages when strains are measured eliminates the temperature effects on the instrumentation only.

## 4.6 EVALUATION OF LOAD TEST RESULTS

### 4.6.1 General

At the completion of the field load test and prior to using the load test results in establishing a load rating for the bridge, the reliability of the load test results should be evaluated. **Also, it is important to understand any differences between measured load effects and those anticipated or based on standard design practices.** This evaluation is generally performed in the office after completion of the field load test.

### 4.6.2 Reliability

Factors which contribute to the reliability of the load test results are the experience of the test team members, the type and extent of instrumentation used during the load test, including the use of any redundant measuring devices, the repeatability of the results for the same load case, the temperature conditions and the compatibility of the measured effects with those predicted by theory, if available (see Section 4.6.3).

These factors should be considered in evaluating the overall acceptability of the test results.

### 4.6.3 Differences Between Measured and Computed Values

**To fully utilize the results of the load test, it is important to be able to explain why the bridge behaves differently from the analytical model used in the preliminary rating computations.**

Factors which may explain all, or part of, the differences between observed and theoretical load effects are described in Chapter 3.

## 4.7 LOAD RATING

The determination of a revised load rating based on field testing should be done in accordance with Chapter 6 for Diagnostic Tests and Chapter 7 for Proof Tests. The rating established should be consistent with good engineering judgment and the structural behavior observed during the load test.

## 4.8 REPORTING

A comprehensive report should be prepared describing the results of field investigations, testing procedures, type and location of instrumentation, description of test load, and the final rating calculations. The report should include the final assessment of the bridge according to the results of load testing and rating calculations. The report may also contain recommendations for the repair and/or strengthening and periodic maintenance.

The load test should be documented in a report containing the following information:

**Identification of Bridge Structure** - This should include the name of bridge, location, size of bridge including length, width, number of spans, number of lanes, description of span tested including type of superstructure, material and other pertinent information.

**Purpose of Load Test** - A statement regarding the reasons for testing and the test objectives.

**Condition Inspection** - Field inspection findings, including the condition of structural components and overall condition of the structure. Include any measurements used in calculating existing dead loads, member section properties, or establishing a baseline for crack widths and other such parameters.

**Preliminary Load Rating/Analysis** - Description of the load rating and analysis made prior to the load test including assumptions made, type of analytical model, and rating results.

**Instrumentation** - Locations and types of instrumentation and approximate range of measurements expected under the test loads.

**Test Load** - Description of test loads including whether bridge was closed or open to traffic during the test, type of loads, loading increments, weight and axle configuration of load, direction, speed of vehicle (if applicable), and position on structure. No-load and repeated load cases should be indicated.

**Load Effect Measurements** - Location of instrumentation, actual measurements for each load case, and comparison of measured versus computed values due to the test load(s).

**Test Observations** - Summary of observations made during load placement including crack propagation, lateral deflection, rotation, noise, temperature and weather conditions, and other relevant observations.

**Final Load Rating Calculations** - A complete set of calculations performed in accordance with Chapter 6 or 7 should be provided including assumptions. For diagnostic tests, calculations should show differences between measured and calculated stresses due to the applied test load along with the reasons for the differences.

**Findings** - A statement describing the results of the test, the load rating, recommendations for repairs or strengthening, if any, and follow-up actions including the need for future load testing. This information may also be helpful to the bridge owner in making posting and permit decisions.

# CHAPTER 5

# LOAD TEST EQUIPMENT AND MEASUREMENTS

## 5.1 INTRODUCTION

Load test instrumentation is used to measure the following: (1) strains (stresses) in bridge components, (2) relative or absolute displacements of bridge components, and (3) relative and absolute rotation of bridge components.

Prior to conducting a field load test, the engineer must determine the goals of the test and the types and magnitude of the measurements to be made (see Chapter 4). Preliminary calculations may be needed to estimate the range of the measurements as well as the best locations for the instrumentation.

## 5.2 TYPICAL MEASUREMENTS

### 5.2.1 General

Strain, displacement and rotation measurements on bridges are performed under a wide range of environmental, loading, and response conditions. Care is needed in every step of the load test, from initial planning to installation and data acquisition and interpretation. The equipment used to make these measurements is described in Section 5.3, which is intended only as a guide.

### 5.2.2 Strains

Strain data may be needed at several locations consistent with the needs of a diagnostic and or a proof load test. Strain sensors are usually attached on critical members to monitor response. Also, locations are selected so that the analytical model can be validated. This is done by placing sensors on several main load-carrying members and monitoring simultaneously. Subsequently, the measured responses can be compared to the predicted values from the model. Finally, attachment details can be studied by placing strain sensors so as to obtain stress concentrations.

Data should be monitored in the field to ensure proper operation of equipment and to prevent damage to the structure. For a typical installation, data will be taken with each increment of loading as well as at every new position of load application.

### 5.2.3 Displacements

Displacement measurements are often an important part of the load testing program, especially for proof loading. These help to determine linear behavior while the test loads are being incremented and also to determine whether the displacments are recoverable when the test loads are removed. Typically, only a few locations need be monitored during a test. Vertical deflections are usually required only at midspan of the structure.

The measurement of relative vertical displacements between the top and bottom flanges of a girder can establish the integrity of the section, particularly if extensive deterioration is present. In some cases, such as at bearings, the measurement of horizontal displacements may be helpful in determining whether a bearing is functioning as designed.

### 5.2.4 Rotations and Other Measurements

The measurement of end rotations can establish the extent of end restraint which exists at bearings. The elastic curve for a bending member can be developed by measuring rotations along the length of the member.

Depending on the test objectives, other data may be useful, such as temperature and wind speed. The position of the test vehicle, both transversely and longitudinally should be recorded. Also, if data is being recorded under random traffic, it may be necessary to monitor the traffic with counters or WIM devices. Other measurements may be needed such as for crack openings, slippage, and rigid body motion.

## 5.3 TYPICAL EQUIPMENT

### 5.3.1 Strain Measurements

#### 5.3.1.1 General

The most common devices for field measurement of strains are electrical resistance gages (bonded resistance strain gages), mechanical strain indicators, and transducers. Strain transducers are calibrated in the laboratory and are easy to install in the field, even in adverse weather conditions.

The equipment necessary for the use of strain gages includes: (1) strain sensors, (2) signal conditioners to power the sensors and amplify and filter the signals, and (3) recording instruments such as oscilloscopes, analog recorders or digital computers.

Since strain measurements on bridges are performed under varying environmental conditions, selection and installation of strain sensors may affect the quality and reliability of the data. Four common types of strain sensors are bondable gages, weldable gages, strain transducers, and vibrating wire gages. The first three types use electrical resistance strain gages with thin metal foils or wire, and the fourth type utilizes a thin wire filament.

Gages are attached by adhesive, welding or mechanical means to bridge members at selected points and orientation and are incorporated into a Wheatstone bridge circuit as illustrated in Figure 5-1. Strain measurement is based on the change in electrical resistance of the gage caused by its change in length when the member to which it is attached undergoes strain.

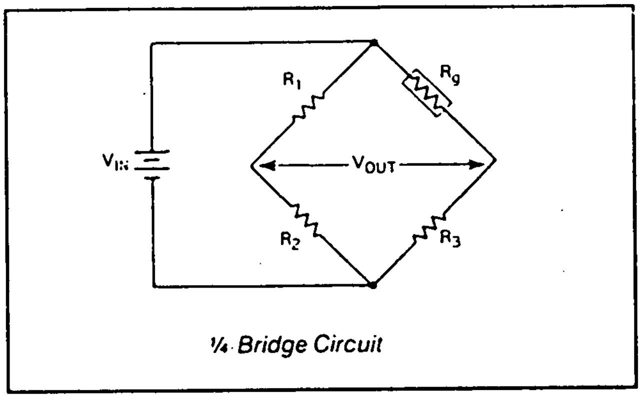



Figure 5-1: Wheatstone Bridge Circuit

The type of sensor used will depend on a number of factors including the following:

1. Strain magnitude—Often the strain levels in bridge components will be low, in the range of only 50 to 150 microstrains, corresponding to 2 to 4 ksi in the steel. Strain gages with high impedance values (1000 or 350 ohms versus 120 ohms) will provide a better signal-to-noise ratio. Strain transducers with mechanical amplification also improve the signal-to-noise ratio.

2. Strain gradient—Since a gage produces output proportional to the strain over the gage length, the gage length should be considered in selecting the type of gage for the response to be measured. For example, at locations where stress concentrations at an attachment are being measured, the gage length must be short and bonded gages rather than transducers are appropriate. Longer gages should be used with concrete and timber members.

3. Environmental conditions—Temperature and moisture affect the installation of strain gages, particularly in the case of bonded gages. Adhesives used to install gages require temperatures of 65 degrees or higher. Moisture is a common cause of gage failure, but its effect can be mitigated by applying a waterproof coating immediately after gage installation.

4. Accuracy—Bonded gages are the most accurate. Weldable gages are somewhat less accurate but have better long-term stability.

5. Electromagnetic noise—Such noise can be a problem especially if the site is located near power lines or radio transmitters. The use of high quality, shielded cable for lead wires provides an effective noise barrier. The Wheatstone bridge mentioned above should be installed as close as possible to the gage.

6. Measurement period—Most load tests involve strain measurements over a short time period, usually one or two days. Strain measurements over a long time period require special precautions due to changes in environmental conditions, weatherability, and in some cases, the potential for vandalism. Long-term tests may require the use of vibrating wire gages which have proven to be stable over long time periods.

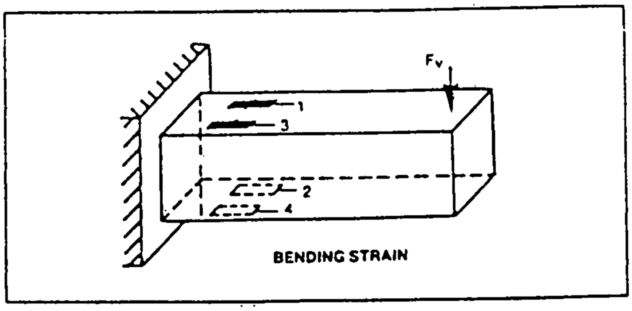
5.3.1.2 Bonded Strain Gages

Depending on the purposes of the measurement, different types of bonded strain gages can be used. For uni-directional strain measurement on steel members, a single common strain gage with a quarter Wheatstone bridge could be sufficient. For strain measurements in two directions at a point, strain rosettes with two linear strain gages in set directions are used. Special purpose gages which contain long measuring foils are commonly used on concrete elements to measure strains to avoid any local fluctuations at the interface of aggregate particles or at the location of micro-cracks.

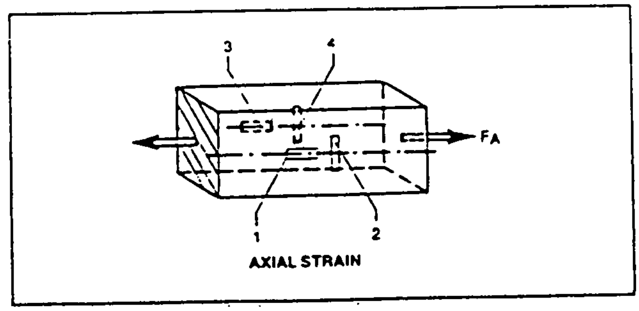
Pure bending and pure shear can be measured by using a combination of gages with half or full Wheatstone bridge circuits. Temperature variations can be conpensated for by incorporating a temperature compensating gage in one leg of the Wheatstone bridge. Table 5-1 and Figure 5-2 show the arrangement of gages for various measurements.

TABLE 5-1 Strain Gage Arrangements

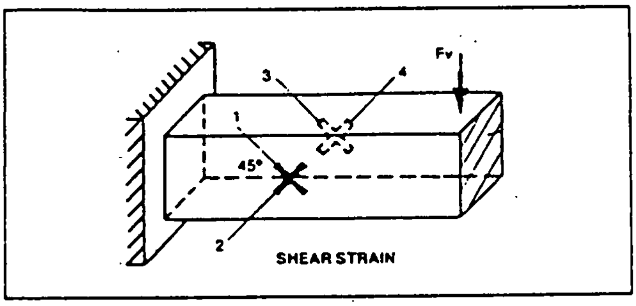
| TYPE OF STRAIN | WHEATSTONE BRIDGE TYPE | POSITION OF GAGES | TEMP. COMPENSATION | STRAIN COMPENSATION |
|---|---|---|---|---|
| Bending | 1/4 | Fig. 5-2(a), 1 | No | None |
| | 1/2 | Fig. 5-2(a), 1,2 | Yes | Axial |
| | Full | Fig. 5-2(a), All | Yes | Axial |
| Axial | 1/4 | Fig. 5-2(b), 1 | No | None |
| | 1/2 | Fig. 5-2(b), 1,2 | Yes | None |
| | 1/2 | Fig. 5-2(b), 1,3 | No | Bending |
| | Full | Fig. 5-2(b), All | Yes | Bending |
| Shear and Torsional | 1/2 | Fig. 5-2(c), 1,2 | Yes | Axial & Bending |
| | Full | Fig. 5-2(d), All | Yes | Axial & Bending |



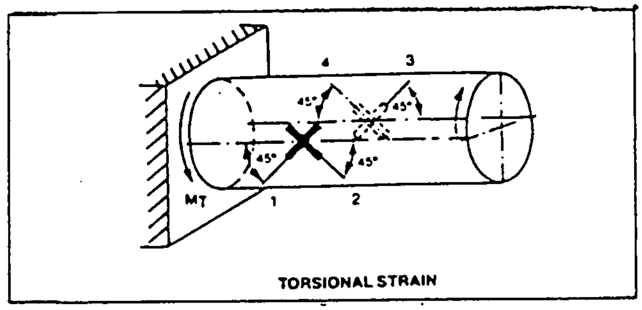

(a) Bending Strain




(b) Axial Strain




(c) Shear Strain




(d) Torsional Strain

FIGURE 5-2: Strain Gage Installations

Many types of gages are available in various sizes, grid patterns, sensitivities and materials. Similarly, many types of bonding agents are available having a variety of curing rates, long-term stability, temperature properties and moisture protection. Bonded gages (see Fig. 5-3) usually take the longest time to install. Reference 41 provides more in-depth data on bonded strain gages and strain measurements.

### 5.3.1.3 Weldable Strain Gages

Weldable strain gages provide an acceptable alternative to bonded strain gages when weather conditions do not permit curing or when installation time is short. These gages (see Fig. 5-4) are more costly than foil gages and require a larger contact area for installation.

### 5.3.1.4 Strain Transducers

Strain transducers must be assembled and calibrated in the laboratory. Bondable gages are mounted to a metal alloy frame and, together with the lead wires, sealed for environmental protection. Transducers have the advantage of easy installation on timber, steel or concrete members, and are reusable although initial cost is high, however, their large size usually does not allow measurements in areas of high strain gradients. Strain transducers (see Fig. 5-5) must be installed with C-clamps or adhesives, by drilling, or by setting one or more anchors.

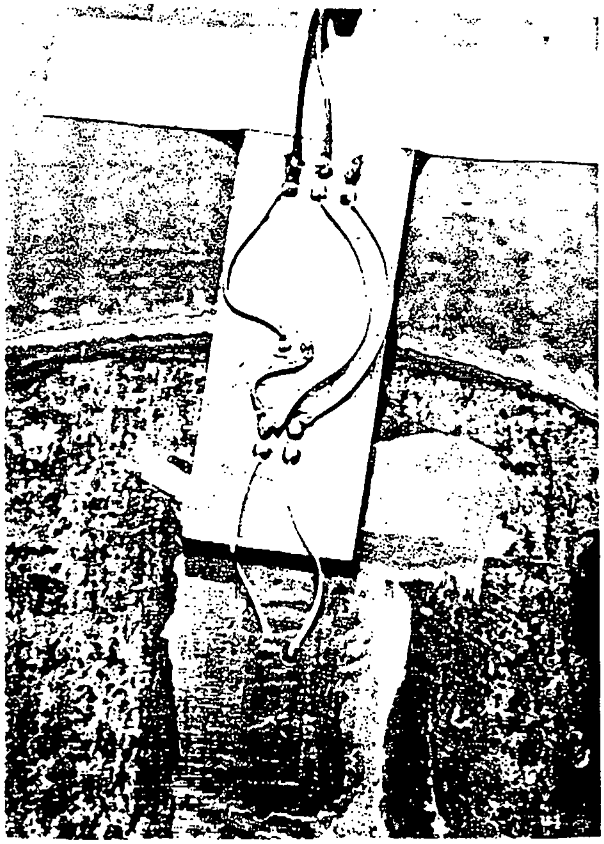


FIGURE 5-3: Electrical resistance gage with dummy gage for temperature compensation.

FIGURE 5-4: Four weldable gages used to measure strains in steel eyebars of an old truss bridge.

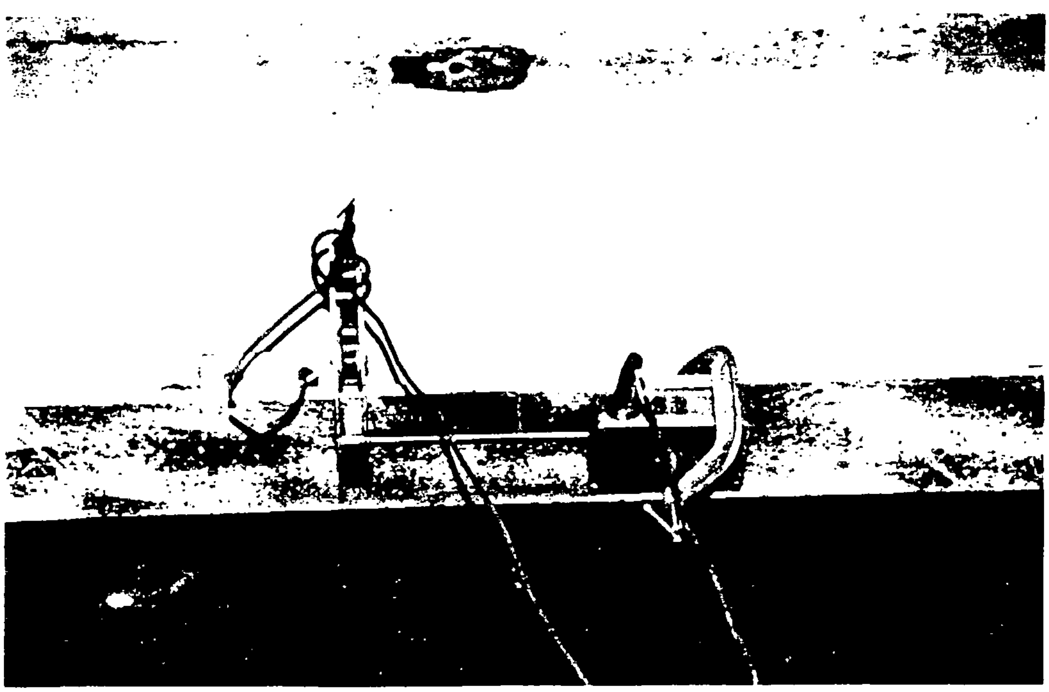


FIGURE 5-5: Demountable strain gage installed on bottom flange of steel beam using "C" clamps. Note weldable gage attached to top flange.

5.3.1.5 Vibrating Wire Gages

Vibrating wire gages (see Fig. 5-6) measure strains by means of a wire in tension. The vibration frequency of the wire is a function of the tension in the wire. As the member undergoes strain, the wire tension changes and the corresponding change in frequency is a measure of the change of wire strain. This gage is ideal for long-term measurements requiring stable initial conditions, but it is limited in usefulness for measuring strains induced by moving or rapidly applied loads.

5.3.2 Displacement Measurements

5.3.2.1 General

Measurement of displacements usually requires a fixed reference point. The most commonly used displacement-measuring instruments are dial gages and electrical transucers. Mechanical instruments and water leveling techniques are also applicable.

FIGURE 5-6: Vibrating wire gage used to monitor crack width during bridge load test.

5.3.2.2 Electrical Transducers

Linear Variable Differential Transformers (LVDT) (see Fig. 5-7), and potentiometers, transform displacement to a proportional change of electrical voltage in a circuit. Both static and dynamic displacements can be monitored. Electrical strain gages mounted on small metal pieces can also serve as accurate displacement instruments. Such metal pieces can be positioned as a cantilever beam or column, and horizontal or vertical displacements can be measured. Electrical displacement transducers must be calibrated to establish the relationship between voltage and displacement.

### 5.3.2.3 Mechanical Instruments

Dial gages (see Fig. 5-8) are the most commonly-used type for measuring displacements due to static loads. Dial gages are easy to set up and their accuracy is usually sufficient for load tests. Laser methods and other surveying tools can be used when higher accuracy is required. Where a fixed reference point is difficult to obtain, a water-leveling instrument can be used for measuring vertical movements. Such instruments consist of two vessels, one attached to the structure and the other to a fixed reference point. The vessels are connected by a flexible hose. Changes in the heights of water in the vessels can be precisely measured to give the relative displacements. These are especially useful for long-term slow movements and settlements. Tilt meters can be used to measure rotations.

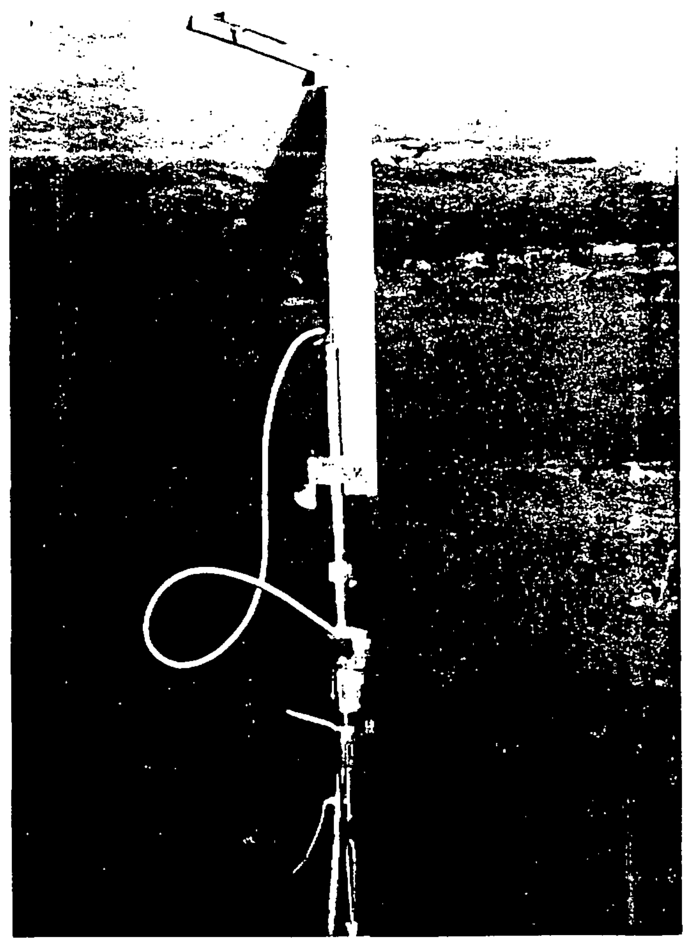


FIGURE 5-7: Close-up view of steel frame and an LVDT

5.3.3 Data Acquisition Instrumentation

The data acquisition system includes signal conditioner, analog to digital (A/D) converter, and a data recording system. Figure 5-9 illustrates the system used by Florida DOT.

A signal conditioner provides excitation to the strain gage, compares the plus and minus signals from the Wheatstone bridge circuit, and amplifies the signal; and also provides balancing capabilities for the Wheatstone bridge circuit. For extended periods of testing, self-balancing capabilities are desirable since the Wheatstone bridge zero relationships will drift, as a result of the inherent nature of the bridge along with temperature effects. In some reported cases, temperature effects were found to be an order of magnitude greater than the strains due to applied live loads.

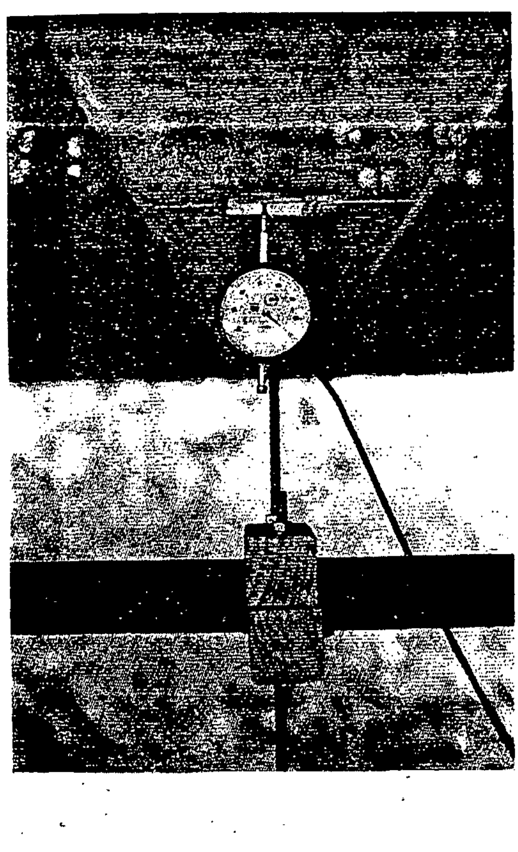


FIGURE 5-8: Dial gages positioned to measure vertical deflection of concrete girder

The amplifications of the signal conditioner are governed by the range of the A/D converter or the analog recorder. Typically, gains of 500 to 5000 may be used. Finally, the signal conditioner should filter the signals to reduce high frequency noise. It is important that the filter band not distort the measured strains since dynamic responses of highway bridges may be up to 12 Hz for main members and higher for some components. Higher frequencies are sometimes found for bridges carrying railroad or transit lines.

Selection of the A/D converter will be governed by the accuracy of the sensors, the range of the output, and the sampling rate. The latter depends on the nature of the test, i.e., slow crawl vs. normal-speed run, and the frequency of the recorded strains. If a low sampling rate is used, there may be some lag of data in one channel with respect to another.

Strain recorders are needed for processing of the strain information. In many early bridge tests, strains were recorded on oscillograph paper, on analog tape, or manually. Currently, most strains are recorded on data acquisition systems which have portable computers for both data recording and processing. The number of channels which can be recorded is high and the speed of data collection is not sacrificed. Software is also available for processing of the load test data. It is important in bridge testing that the recording system be capable of displaying the bridge response as the load test is conducted in the field. Such monitoring of bridge performance as loads are applied is important to the integrity and safety of the load test.

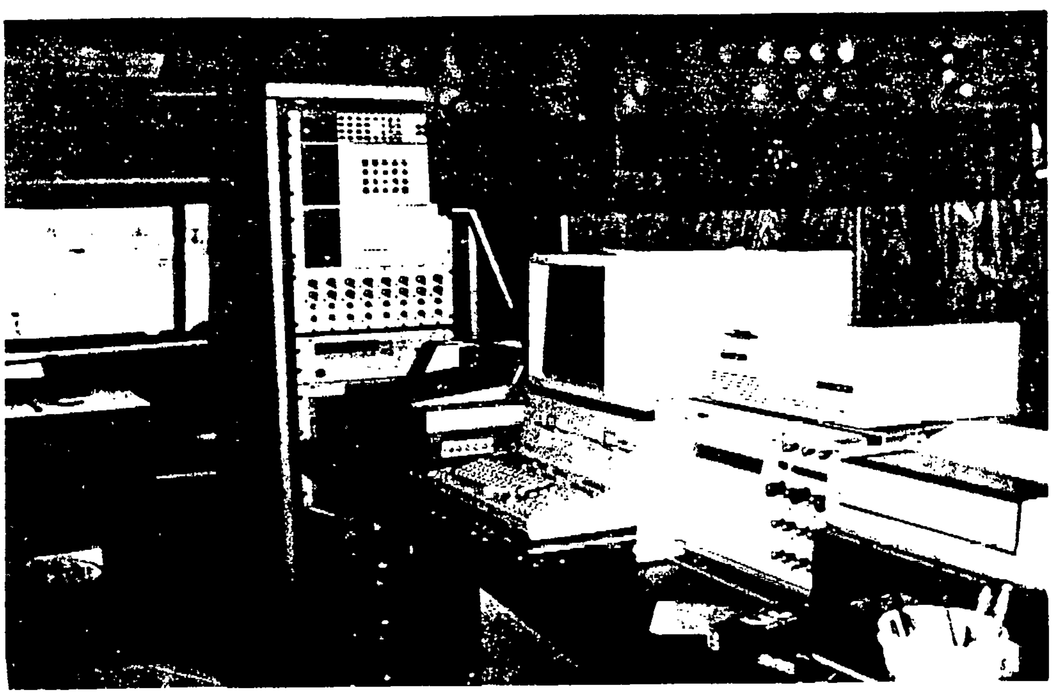


FIGURE 5-9: Computer equipment and automatic data acquisition system used by Florida DOT for load testing of bridges.

# CHAPTER 6

# DIAGNOSTIC LOAD TESTS

## 6.1 INTRODUCTION

Diagnostic nondestructive load tests have been employed by many bridge owners to improve their understanding of the behavior of the bridges being tested. Diagnostic type tests will reduce the uncertainties related to material properties, boundary conditions, cross-section contributions, effectiveness of repair, influence of damage and deterioration not easily detected, and other similar parameters. The intent of this diagnostic testing is to provide methods and procedures for the implementation of the field test results in the load rating process.

Prior to initiating a diagnostic load test, the bridge should be rated analytically. The AASHTO Condition Evaluation Manual (35) and the AASHTO Guide Specifications (37) contain the requirements for the load rating of highway bridges. The bridge owner may select the methods and level for rating by calculation consistent with established policy. The procedures outlined in this chapter will enable the bridge owner to re-examine the theoretical values and adjust these ratings to reflect the actual performance of the bridge obtained from the diagnostic test results. Once the adjusted load ratings have been set, the bridge owner may select the appropriate level for posting the bridge or issuing permits for overloads based on the bridge owner's policies, as discussed in Chapter 8.

## 6.2 GENERAL PROVISIONS

Translating the results of bridge load tests into bridge load ratings depends on the type of diagnostic test performed, the analytical method employed and the structural characteristics of the bridge tested. A load rating equation is presented in Section 6.5 which recognizes the diversity in load test applications.

In general, diagnostic testing may be more elaborate than proof testing since both an analytical model and more stringent field measurements are required. A major part of the engineer's responsibility is the interpretation of test results. **Often this means deciding how much of the load-carrying capacity observed in the test as compared to the values predicted analytically should actually be utilized in establishing the bridge load rating.** Some of the observed load capacity may be counted on at service load levels. Factors which should be considered in evaluating the usable enhacements are discussed in greater detail in Chapter 3.

As a result of a successful bridge load test, the engineer achieves greater confidence in the analytical model used to predict the live load effects on the bridge. Such higher confidence may be utilized by the engineer in the posting of the bridge at load levels higher than what was deemed appropriate prior to the load test.

## 6.3 APPROACH

As long as a bridge exhibits linear behavior, a diagnostic load test can be used to validate the live load model. If the behavior is further assumed to be linear until an allowable maximum load limit is reached, then the test results can be extrapolated to provide a safe load rating level. It is thus important that the test load be placed at various positions on the bridge to determine the response in all critical bridge members. Further, the magnitude of the test load must be sufficiently high so that there is little likelihood of non-linear behavior at the anticipated service load levels. This means that the engineer must monitor the response of the bridge to the applied load, check for linear behavior during the test, and compare the test results with those predicted by the analytical model. If the engineer is satisfied that the model is valid, then an extrapolation to load levels higher than those placed on the bridge during the test may be feasible.

**There are risks associated with extrapolating diagnostic test results to load effect levels which are higher than those placed on the bridge during the test.** Care must be exercised when extrapolating diagnostic test results to ensure safe bridge performance at the extrapolated load level. Section 6.5 presents a method for extrapolating the results of a diagnostic load test.

## 6.4 CANDIDATE BRIDGES FOR DIAGNOSTIC LOAD TESTS

Bridges which have been load rated analytically in accordance with the AASHTO C/E Manual or the AASHTO Guide Specifications, but whose load rating is less than HS20, are candidates for diagnostic load testing. Thus, candidate bridges are limited to those bridges for which an analytical load rating model can be developed.

Furthermore, in selecting candidate bridges, the appropriateness of extrapolating the diagnostic test results to load levels higher than those utilized during the test should be considered. Redundant structures such as multigirder bridges in steel, reinforced concrete, prestressed concrete or timber are good candidates from this point of view. Two-girder systems, two-truss systems and other such non-redundant structures require greater care in extrapolating diagnostic test results to higher load levels. Computations should be performed to determine whether stringers, floorbeams, and connections can safely support the loads established by extrapolating the results of a diagnostic load test.

## 6.5 APPLICATION OF DIAGNOSTIC TEST RESULTS

### 6.5.1 General

In a diagnostic load test, load effects in critical bridge members are measured and then compared with values predicted by an analytical model. A major part of diagnostic testing is the assessment of the differences between predicted and measured responses for subsequent use in determining the load rating of the bridge.

This section provides guidelines for modifying the analytical load rating for a bridge based on the results of a diagnostic load test.

### 6.5.2 Rating Equation

The proposed rating equation to be used following a diagnostic load test is:

$$RF_T = RF_c \times K \qquad (6\text{-}1)$$

where:

$RF_T$ = The load-rating factor for the liveload capacity based on the test load results.

$RF_c$ = The rating factor from Eq. 2-1 based on the calculations prior to incorporating test results.

K = Adjustment factor resulting from the comparison of measured test behavior with the analytical model.

The rating factor multiplied by the rating vehicle weight in tons gives the rating of the structure. Eq. 6-1 separates the computations used in determinating the $RF_c$ value based on Eq. 2-1 from the benefits of the field load test represented by the factor "K". Each of these two components is discussed in detail below.

### 6.5.3 Calculating $RF_C$

Eq. 2-1 can be written in generic form as follows:

$$RF_c = \frac{\text{(Capacity)-(Factored Dead Load Effect)}}{\text{(Factored Live Load Effects Plus Impact)}} \qquad (6\text{-}2)$$

"Capacity" depends on the rating method and rating level selected by the engineer in accordance with the AASHTO C/E Manual. Section 6.6 describes the various rating methods and Section 6.7 discusses the load rating levels.

The appropriate section factor (area, section modulus) to be used in determining $RF_c$ should be determined after evaluation of the load test results including observations made during the placement of the test vehicle on the bridge. For composite structures with shear connectors, the full composite section as defined by AASHTO Specifications should be used unless observations during the test indicate slippage at the deck-girder interface. Non-composite structures which show no evidence of composite action under the test load should be evaluated based on non-composite section factors.

The enhancement to the section factor resulting from unintended composite action needs to be critically evaluated. For example, some researchers recommend using 50% of the equivalent additional composite action from a non-composite deck. Other researchers have suggested that composite action without positive shear connectors is not dependable at high moment levels.

The degree of effective composite action may be a function of the extent of encasement of the girders. Studies of slab-on-girder bridges without mechanical shear connectors have shown that composite action exists up to certain load levels due to the bond between the deck slab and the girders (Ref 43). A method for determining the load level beyond which unintended composite action cannot be counted on is given in Section 3.2.

While $RF_c$ is usually based on standard procedures for determining the section properties, bridge owners may want to re-evaluate the section properties used in determining $RF_c$ based on the results of the load test, using the method described in Section 3.2.

### 6.5.4 Determining K

The Adjustment Factor (K) is given by:

$$K = 1 + K_a \times K_b \tag{6-3}$$

where

$K_a$ accounts for both the benefit derived from the load test, if any, and consideration of the section factor resisting the applied test load.

$K_b$ accounts for the understanding of the load test results when compared with those predicted by theory, the type and frequency of follow-up inspections, and the presence or absence of special features such as non-redundant framing and fatigue-prone details.

Without a load test, K=1. If the load test results agree exactly with the theory, then K=1 also. Generally, after a load test K is not equal to one. If K>1, then response of the bridge is more favorable than predicted by theory and the bridge load capacity may be enhanced. On the other hand, if K<1, then actual response of the bridge is more severe than that predicted and the theoretical bridge load capacity may have to be reduced.

The following general expression should be used in determining $K_a$:

$$K_a = \frac{\varepsilon_c}{\varepsilon_T} - 1 \tag{6-4}$$

where:

$\varepsilon_T$ = maximum member strain measured during load test.

$\varepsilon_c$ = corresponding theoretical strain due to the test vehicle and its position on the bridge which produced $\varepsilon_T$.

In general:

$$\varepsilon_c = \frac{L_T}{(SF)\,E} \tag{6-5}$$

where:

$L_T$ = calculated theoretical load effect in member corresponding to the measured strain $\varepsilon_T$.

SF = member appropriate section factor (area, section modulus, etc.).

E = member modulus of elasticity.

For those members which were designed as non-composite sections, and where there is no possibility for composite action during the load test, the section factor will be the same as that used in determining $RF_c$.

Typically for girders, the section modulus may include the effect of some composite action regardless of whether any composite action was intended in design. The theoretical strain resulting from the test load should be calculated using a section factor which most closely approximates the *member's actual resistance* during the test.

Thus the factor $K_a$ represents the test benefit without the effect of unintended composite action, the most common source of enhancement. To illustrate the concept of $K_a$, consider the following diagnostic load test situation as adapted from Ref. 1:

- Multigirder, simple span bridge with non-composite concrete deck.
- During diagnostic load test, measured strain in member was 197 microinches, corresponding to a measured stress $\sigma_M = 5.7^{ksi}$. The weight, configuration and position of the test vehicle which produced this strain were also noted.
- Computations indicated:

  (1) Maximum theoretical moment $M_T$ produced in member was calculated as 5225 k-in.

  (2) Non-composite girder section modulus $SF_{NC}$ was calculated as 395 $in^3$ and the corresponding maximum girder bending stress under $M_T$ would be 5225/395 = 13.2 ksi.

  (3) Composite section modulus $SF_C$ assuming maximum contribution of concrete deck (per AASHTO Specifications) was found to be 537 $in^3$ and the corresponding stress under $M_T$ would be 5225/537 = 9.7 ksi.

  (4) Effective section modulus based on theoretical moment $M_T$ divided by measured stress $\sigma_M$ is 5225/5.7 = 920 $in^3$.

The above data is shown graphically in Figure 6.1.

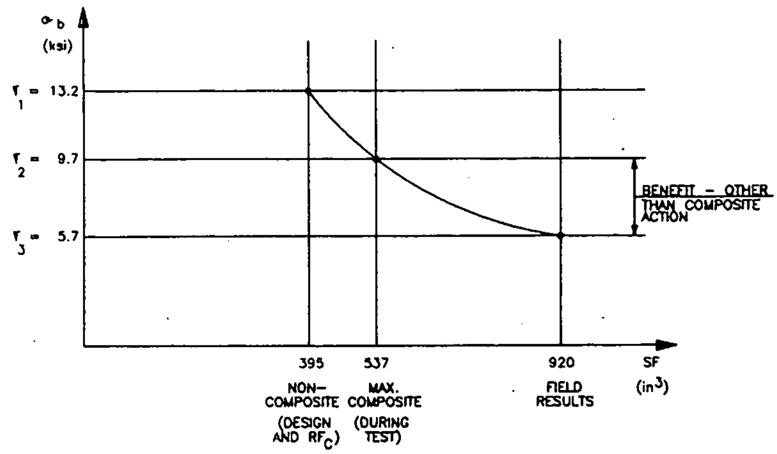



FIGURE 6.1: Stress vs. Section Modulus

The computations indicate that a maximum girder bending stress of 13.2 ksi would be produced in the non-composite member by the test load, whereas the corresponding measured stress was only 5.7 ksi. On this basis, the apparent test benefit would be 13.2/5.7 = 2.32, i.e., the measured stress was 2.32 times smaller than the calculated stress.

During the application of the test load, no slippage between the concrete deck and the top flange of the stringers was noted indicating that composite action did indeed exist under the applied test load. Thus, the measured stress of 5.7 ksi was based on the much larger resistance of the composite section. The computations indicated that a stress of 9.7 ksi would be produced in the composite member under the test load. On this basis, the apparent test benefit would be 9.7/5.7 = 1.70, i.e. the measured stress was 1.70 times smaller than the calculated stress.

Using the field results and assuming non-composite behavior during the test will result in the greatest test benefit (2.32) and, hence the highest rating; but it is the most optimistic, least conservative approach and, in this case, an inaccurate assessment of the test benefit. For this example, the test benefit should be based on the composite section resisting the test load (see also Section 6.5.3).

It should be noted that the difference between the field results and maximum composite represents the difference between the actual behavior of the bridge and the revised analytical model, specifically the assumptions made concerning such factors as lateral and longitudinal load distribution and the participation of other members. These are taken into consideration by the factor $K_b$, defined as follows:

$$K_b = K_{b1} \times K_{b2} \times K_{b3} \tag{6-6}$$

where:

$K_{b1}$ takes into account the analysis performed by the load test team and their understanding and explanations of the possible enhancements to the load capacity observed during the test. Items to be considered in understanding and explaining the test results are discussed in Chapter 3.

In particular, the load test team should consider the items below and reduce $K_{b1}$ to account for those contributions that cannot be depended on at the rating load level. Linear behavior during the load test as well as the magnitude of the test load compared to the rating load should be considered by the team in selecting the factor $K_{b1}$.

The factor $K_{b1}$ should be assigned a value between 0 and 1.0 to indicate the level of the test benefit that is expected at the rating load level. $K_{b1} = 0$ reflects the inability of the test team to explain the test behavior or validate the test results, whereas $K_{b1} = 1$ means that the test measurements can be directly extrapolated to performance at higher loads corresponding to the rating levels.

It is not possible to provide strict guidelines for deciding on the degree to which of the measured enhancements found in a diagnostic test that can be relied on at the rating load level. Table 6-1 provides guidance based on the anticipated behavior of the bridge members at the rating load level, and the relationship between the test vehicle effect (T) and the gross rating load effect (W).

TABLE 6-1
Values for $K_{b1}$

| Can member behavior be extrapolated to 1.33W? | | Magnitude of test load | | | $K_{b1}$ |
|---|---|---|---|---|---|
| Yes | No | $\frac{T}{W} < 0.4$ | $0.4 \leq \frac{T}{W} \leq 0.7$ | $\frac{T}{W} > 0.7$ | |
| √ | | √ | | | 0 |
| √ | | | √ | | 0.8 |
| √ | | | | √ | 1.0 |
| | √ | √ | | | 0 |
| | √ | | √ | | 0 |
| | √ | | | √ | 0.5 |

The intent of "Can member behavior be extrapolated to 1.33W?" in Table 6-1 is to provide some assurance that the structure has adequate reserve capacity beyond its rating load level (W). Normally this would be established by calculation but proof testing would also be acceptable.

Examples of typical calculations which could be performed to check this criterion include:

(1) Load the analytical model with 1.33W and determine whether there is linear behavior of the components of the structure. The model could be based on the AASHTO Design Specifications or a three-dimensional computer model (Ref 6).

(2) Using the procedures of Section 3.2, determine whether there is composite action at 1.33W where none was intended.

Some of the load-carrying enhancement of a bridge at test load levels is a result of structural features which increase the overall resistance of the bridge system when compared to its individual component members. Such enhancement is permanent, not subject to temperature variations, and would be present at the higher rating load levels. It is this aspect of member behavior which should be evaluated in determining an appropriate value for $K_{b1}$ using the Table 6-1.

$K_{b2}$ takes into account the ability of the inspection team to find problems in time to prevent any changes of bridge condition from invalidating the test results, and will depend on the type and frequency of inspection. Values for $K_{b2}$ are given in Table 6-2.

Table 6-2
Values for $K_{b2}$

| INSPECTION | | $K_{b2}$ |
|---|---|---|
| Type | Frequency | |
| Routine | between 1 & 2 years | 0.8 |
| Routine | less than 1 year | 0.9 |
| In-Depth | between 1 & 2 years | 0.9 |
| In-Depth | less than year | 1.0 |

$K_{b3}$ takes into consideration the presence of critical structural features which cannot be determined in a diagnostic test and which could contribute to the sudden fatigue, fracture or instability failure of the bridge. Typical values for $K_{b3}$ are given in Table 6-3.

Table 6-3
Values of $K_{b3}$

| Fatigue Controls? | | Redundancy | | $K_{b3}$ |
|---|---|---|---|---|
| No | Yes | No | Yes | |
| | x | x | | 0.7 |
| | x | | x | 0.8 |
| x | | x | | 0.9 |
| x | | | x | 1.0 |

$K_{b3}$ is introduced here to make certain that test engineers consider the various types of failure modes (ductile or brittle) and the presence of clear indications of possible distress which may provide warnings prior to failure. In Table 6-3, to establish whether or not fatigue controls, an analysis would be performed in accordance with the AASHTO Specifications and the AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members. The column entitled "Redundancy" refers to those situations where the failure of critical structural components would not result in the bridge collapse.

It is important that the intent of factor $K_{b3}$, which is dependent on the structure type, not be considered twice in the ratings. Some agencies now use these same characteristics in selecting a load level for posting, such as the use of the lower inventory ratings for non-redundant bridge types. If this is done, then the agency should use $K_{b3} = 1$ to avoid double consideration of these terms.

Engineering judgment based on observations made during the diagnostic load test must be used in establishing values for $K_{b1}$, $K_{b2}$ and $K_{b3}$. The values recommended for these parameters are based on experience and have been selected to provide a "level of comfort" in extrapolating the diagnostic test results to a realistic rating load. They should be considered as maximum values; smaller values may be selected by the engineer as deemed appropriate.

## 6.6 LOAD RATING METHODS

### 6.6.1 General

Diagnostic test results can be incorporated into any of the standard rating methods which follow. Such rating methods are in the format which compares applied load effects with the capacity following Eq. 6-2. The different rating methods refer to use of the safety margins on loads, strengths or both. The diagnostic test serves to provide a more accurate relationship of load with load effect.

### 6.6.2 Allowable Stress Method

In this method rating is found from the expression:

$$RF_c = \frac{\text{Allowable maximum stress — Nominal dead load stress}}{\text{Nominal stress plus impact from rating vehicles}} \qquad (6\text{-}7)$$

Following the diagnostic test, the theoretical rating vehicle effects are modified by the term K (see Eq. 6-3) which includes the benefit of both the test results as well as the adjustment factors.

### 6.6.3 Load Factor Method

In this method the bridge rating is found from the expression:

$$RF_c = \frac{\text{Component strength — Factored dead load component effects}}{\text{Factored live load component effects plus impact from rating vehicles}} \qquad (6\text{-}8)$$

Following the diagnostic test, the theoretical rating vehicle effects are modified by the term K (see Eq. 6-3) which includes both the benefits of the test results as well as the adjustment factor.

### 6.6.4 Load and Resistance Factor Rating Method (LRFR)

In this method rating is found from the expression:

$$RF_c = \frac{\text{Factored strength — Factored dead load effects}}{\text{Factored live load effect plus impact from rating vehicles}} \qquad (6\text{-}9)$$

Following the diagnostic test, the theoretical rating vehicle effects are modified by the term K (see Eq. 6-3) which includes both the benefits of the test results as well as the adjustment factor. LRFR factors are based on target safety indices and corresponding load and capacity uncertainties. Hence, a load test reduces the uncertainties about behavior as well as establishes the actual behavior. Criteria for adjusting the live load factor are further explained in the Guide Specifications.

## 6.7 LOAD RATING LEVELS

### 6.7.1 General

Agencies traditionally report load capacities at both inventory level and operating level as defined in the AASHTO C/E Manual.

### 6.7.2 Inventory

The inventory level corresponds to design levels of safety recommended by the AASHTO Specifications. The greater confidence in the behavior resulting from a load test should allow some reduction in the factors of safety, but there is no recognized method in design for incorporating such improved analysis.

### 6.7.3 Operating

The operating level corresponds to the upper bound of allowable safety permitted by the C/E Manual. Caution is urged in utilizing the operating levels in all posting and permit decisions if a diagnostic load test is used to justify confidence in the analytical computations. The diagnostic tests cannot always indicate the possible behavior which may be nonlinear in nature which could occur above the response level used in the diagnostic test. Such higher loads may in some instances lead to instability or component fractures.

# CHAPTER 7

# PROOF LOAD TESTS

## 7.1 INTRODUCTION

Proof load testing provides an alternative to analytically computing the load rating of a bridge. A proof test "proves" the ability of the bridge to carry its full dead load plus some "magnified" live load. A larger load than the live load the bridge is expected to carry is placed on the bridge. This is done to provide a margin of safety in the event of an occasional overload during the normal operation of the bridge. The calibration of these values is described in a separate Technical Report (Ref. 43).

At the conclusion of the proof test, the maximum applied load is a bound on the live load capacity of the bridge. This capacity of the bridge needs to be adjusted by a safety factor to obtain an Operating level equivalent of the bridge's capacity. This safety factor does not need to be the same as that used in design or rating since the capacity has been "proved". Once an Operating level capacity has been established from the proof test results, an Inventory level capacity may be calculated by adjusting the Operating level capacity.

## 7.2 GENERAL PROCEDURES.

During a proof load test, the loads must be incremented and the response measured until the desired load is reached or until the test is stopped for reasons cited below. Loads must also be moved to different positions to properly check all load path components. Upon load removal the structure should again be inspected to see that damage has not occurred and that there are no residual movements or distress.

Usually, the loads are applied in steps so that the response of the bridge under each load increment can be monitored for linear elastic behavior and to limit distress due to cracking or other physical damage. The proof load test is usually terminated when either of the following occurs:

1. The desired live load plus the appropriate margin of safety is reached.
2. The bridge response exhibits the start of nonlinear behavior or other visible signs of distress, such as buckle patterns appearing in compressive zones in steel, or cracking in concrete.

Prior to the proof load test, the structure is supporting its dead load. The live load is simulated during the test by methods described in Chapter 2. Usually, when proof-load testing is used, it is sufficient to consider the magnitude of the proof load rather than the proof-load effect on individual components of the bridge, provided the test load configuration is similar to the vehicle used in rating the bridge. However, an analytical model may be used to describe the proof-load effects at critical sections in terms of the applied test loads. These proof-load effects could include bending moments, shears and/or axial forces. In proof-load testing, when an analytical model is used and strains are measured in the field for comparison with calculated strains or stresses, the load test combines elements of both diagnostic and proof testing. Such "mixed" testing can be beneficial, but requires greater emphasis on computations and more extensive field measurements than that normally required for a basic proof-load test, one in which only the applied loads and resulting displacements are monitored. The proof-load procedures which follow are based on the applied proof load in tons or kips. If a "mixed" proof test is conducted, the procedures which follow may be used by simply substituting "load effect" for "load" as appropriate.

For girder type structures, the test load should envelope the AASHTO rating and posting vehicles or appropriate legal vehicles. If the test vehicle differs markedly from the rating vehicle, it may be necessary to perform additional calculations to determine whether the proper load effects are being achieved during the tests.

The test loads must provide for both the rating vehicles, including the AASHTO Specifications for impact, and a load factor for the required margins of safety. The load factor may be as described in Section 7.4 or as specified by the bridge agency.

The proof loads provide a lower bound on the true strength capacity of the components and hence the lower bound on the load rating capacity. Since a satisfactory proof load test usually provides higher confidence in the load capacity than a calculated capacity, the Engineer is perhaps more likely to depend on the higher operating safety levels for permit and posting decisions if verified by proof testing. The Engineer should, however, give consideration to several site-related factors in establishing permit and posting values as discussed in Chapter 8.

## 7.3 CANDIDATE BRIDGES FOR PROOF LOAD TESTS

Bridge candidates for proof load testing may be separated into two groups. The first group consists of those bridges whose make-up is known and which can be load rated analytically. Proof-load testing of "known" bridges is called for when the calculated load ratings are low and the field testing may provide more realistic results and higher ratings. Bridges with large dead loads compared to the live loads are also sometimes typical candidates for proof load testing.

Higher observed capacities than those that are analytically calculated for such bridges may be due to several factors as discussed in Chapter 3. In some cases, such as bridges with high skew or nonprismatic cross-sectional properties, a proof test may be both more economical and more accurate than developing an elaborate finite element analysis model for use in load rating the bridge.

The second group consists of "hidden" bridges, those bridges which cannot be load rated by computations because of insufficient information on their internal details and configuration. Many older reinforced concrete and prestressed concrete beam and slab bridges whose construction and/or design plans are not available need proof testing to determine a realistic live load capacity. Bridges that are difficult to model analytically because of uncertainties associated with their construction and the effectiveness of repairs are also potential candidates and beneficiaries of proof-load testing.

Caution must be exercised in applying the results of proof tests for this second group, where structural details are not known. This concern is for cases where structure deterioration over time may also be hidden from the inspection. In such cases, greater margins of safety should be used especially when there will be long intervals between subsequent bridge evaluations.

## 7.4 TARGET PROOF LOADS

### 7.4.1 Selection of Live Load Factor, $X_p$, and Its Adjustment

A proof test provides information about the bridge capacity including dead load effect, live load distributions and component strengths. However, other uncertainties, in particular the possibility of bridge overloads during normal operations as well as the impact allowance, are not measured during the test. These remaining uncertainties should be considered in establishing a target proof load.

$X_p$ represents the live load factor which is needed to bring the bridge to an operating rating factor of 1.0. If the test safely reaches this level of load, namely the legal rating plus impact allowance magnified by the factor $X_p$, then the rating factor is 1.0.

Higher proof loads may also be warranted to incorporate ratings for permit vehicles, and in this instance the permit load vehicle plus impact should be magnified by $X_p$.

The AASHTO C/E Manual suggests several areas where site conditions may have an influence on the capacity rating. These factors are included herein by making adjustments to $X_p$ to account for such conditions. Each of these adjustment quantities is presented below. After $X_{pA}$, the adjusted $X_p$, is obtained, this value is multiplied by the rating load plus impact to get the proof-load magnitude that is needed to reach an operating level rating factor of 1.0.

The recommended base value for $X_p$ before any adjustments are applied is 1.40. This value was calibrated to give the same overall reliability as the level inherent in the operating capacity. This value is consistent with the 1.30 load factor used in load factor rating (operating) and the 1.33 margin used in working stress (operating). Since these factors or safety margins are applied in calculated ratings to both dead and live load, the nominal strength, $R_n$, after a test may appear lower than a nominal strength based on the rating calculations (both ratings assumed to be 1.0). That is:

$$R_n = 1.40\,(L+I) + D, \text{ for strength based on test} \qquad (7.1a)$$
$$R_n = 1.3(L+I) + 1.3D, \text{ for bstrength based on calculation} \qquad (7.1b)$$

The reliability levels associated with equations 7-1a and 7-1b are equivalent because the strength value obtained from a proof test is more reliable than that obtained solely by analytical methods.

The following are some of the adjustments to $X_p$ that should be considered in selecting a liveload test magnitude to achieve a rating factor of 1.0. Any of these adjustments may be neglected, however, if the posting and permit policies of the agency already include allowances for these factors.

1. The intent is that for most situations the liveload factor applies to a test with loads in two lanes, without any lane reduction coefficient. This situation controls most structures. If one lane load controls response, however, then increase $X_p$ by 15%. This increase is consistent with possible overload statistics generated for the AASHTO LRFD code now under development. This adjustment is needed for one-lane structures or for other spans in which the single-lane loading augmented by an

additional 15% would govern. In all loading cases, loads shall be placed to produce maximum load effects in a component.

2. For spans with fracture critical details, the live load factor $X_p$ shall be increased by 10%, in order to raise the reliability level to a safer level. A similar increase in test load shall be considered for any structure without redundant load paths.

3. For structures in which routine inspections conducted in accordance with the AASHTO C/E Manual are to be performed less often than 2-year frequency, then increase $X_p$ by 10%. This increase reflects greater chance of overloads, as well as possible undetected deterioration. Further increases in the test load factor are warranted for poorly maintained bridge.

4. If the structure is ratable, that is, there are no hidden details, and if the calculated rating factor exceeds 1.0, $X_p$ can be reduced by 5%. The test in this instance is performed to confirm calculations.

5. If there are observed signs of distress prior to reaching the target live load factor $X_p$, and the test must be stopped, then the full $X_p$ cannot be used in calculating the ratings; instead, it must be reduced by 12% as shown below be means of the factor $k_o$. This reduction is consistent with observations that show that nominal material properties used in calculating ratings are typically 12% below observed material properties from coupons. In the event the test must be stopped because material failure is imminent, it is clear that the mean rather than nominal capacity has been reached.

6. Additional factors including traffic intensity and bridge condition may also be incorporated in the selection of the live load factor $X_p$. Adjustments in $X_p$ may be made using methods outlined in Ref. 44 which concerns evaluation using LRFD procedures. Due account must be given to maintaining the desired reliability level following a proof test consistent with the standards described therein and in Ref. 44.

The adjustments described above should be considered as minimum values; larger values may be selected by the Engineer as deemed appropriate. Further descriptions of the basis for the magnitudes of these adjustments are given in Ref. 43.

### 7.4.2 Application of Target Live Load Factor, $X_{PA}$

Applying the adjustments recommended above leads to the target live load factor, $X_{PA}$. The net percent increase in $X_p$ ($\Sigma\%$) is found by summing the appropriate adjustments given above. Then

$$X_{PA} = X_p \left(1 + \frac{\Sigma\%}{100}\right) \tag{7.2}$$

The target proof load ($L_T$) is then:

$$L_T = X_{PA} L_R (1+I) \tag{7-3}$$

where

$L_R$ = the comparable live load due to the rating vehicle for the lanes loaded

I = the AASHTO Specifications impact allowance

$X_{PA}$ = the target live load factor

In no case should a proof test load be applied that does not envelop the rating vehicle plus impact. For multiple-lane bridges a minimum of two lanes should be loaded concurrently.

$X_{PA}$ should not be less than 1.3 nor more than 2.2.

The target proof load $L_T$ should be placed on the bridge in stages, with the response of the bridge to the applied loads carefully monitored. The first-stage loading should not exceed 0.25 $L_T$ and the second stage loading should not exceed 0.5 $L_T$. Smaller increments of loading between load stages may be warranted, particularly when the applied proof load approaches the target load.

## 7.5 LOAD CAPACITY AND RATING

### 7.5.1 Operating Level Load Capacity and Rating

At the conclusion of the proof-load test, the actual maximum proof live load $L_P$ applied to the bridge is known. The Operating level capacity OP is found as follows:

$$OP = \frac{k_O L_P}{X_{PA}} \tag{7-4}$$

where

$X_{PA}$ = the target live load factor resulting from the adjustments described in Section 7.4.

$k_O$ = is a factor which takes into consideration how the proof-load test was terminated and is found from the Table 7.1.

TABLE 7.1
Values for $k_O$

| Terminated | $k_O$ |
|---|---|
| Reached Target Load | 1.00 |
| Reached Distress Level | 0.88 |

If the test is terminated prior to reaching the target load, the load $L_P$ to be used in Eq. 7-4 should be the load just prior to reaching the load causing the distress which resulted in the termination of the test.

The rating factor at the Operating level ($RF_O$) is:

$$RF_O = \frac{OP}{L_R(1+I)} \tag{7-5}$$

The operating capacity, in tons, is the rating factor times the rating vehicle weight in tons.

### 7.5.2 Inventory Level Load Capacity and Rating

As explained above, the emphasis herein has been to obtain the operating ratings by use of proof test response. It is believed most agencies will base their permit, posting and replacement schedules on these operating values. Agencies may also want to refine the inventory level for the bridge, and this value can be approximated by multiplying the operating ratings just determined by a typical ratio of inventory to operating safety factors, i.e., $\frac{1.33}{1.82} = 0.73$. Thus, the Inventory level capacity (IN) is found from:

$$IN = K_{IN} \, OP \tag{7-6}$$

where the recommended value for $K_{IN}$ is 0.73.

It should be noted that the Inventory level capacity (IN) is based on an Operating level capacity (OP) which has itself been reduced from the actual proof load placed on the bridge. There is no single path in going from "OP" to "IN". The value for $K_{IN}$ recommended above should be used in conjunction with engineering judgment and the practices of the Bridge Owner.

The rating factor at the Inventory level is:

$$RF_I = \frac{IN}{L_R(1 + I)} \tag{7-7}$$

The proof load concepts discussed in Sections 7.5.1 and 7.5.2 are illustrated in Fig. 7-1.

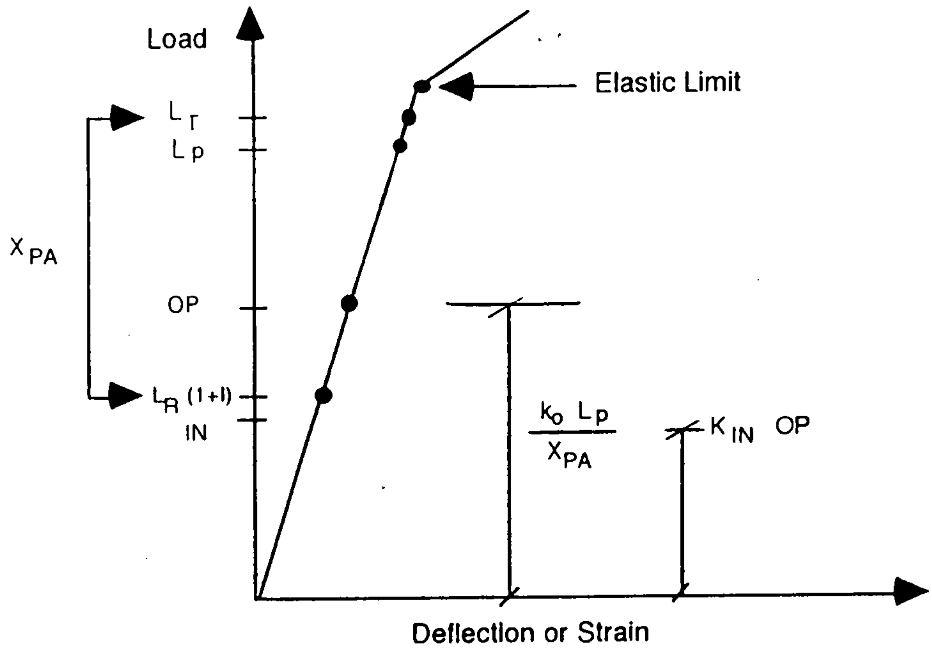



FIGURE 7-1: Proof Load Concept

Note: The position of OP and IN illustrates the situation where a base load factor $X_P$ = 1.92 was used and the applied test load ($L_P$) is slightly less than the target load ($L_T$).

where:

| | | |
|---|---|---|
| $L_R(1+I)$ | = | Rating Vehicle load plus Impact |
| $L_T$ | = | Target Proof load |
| $L_P$ | = | Actual maximum Proof load placed on bridge |
| OP | = | Operating load |
| IN | = | Inventory load |
| $X_{PA}$ | = | Target Load factor as discussed in Art. 7.4 |
| $k_o$ | = | Adjustment factor as described in Art. 7.5.1 |
| $k_{IN}$ | = | Inventory level adjustment factor as described in Art. 7.5.2 |

### 7.5.3 Target Proof Load to Ensure a Rating Factor of 1.0 at the Inventory Level

The basic load factor $X_P$ = 1.40, as adjusted in accordance with Section 7.4, provides a Rating Factor of 1.0 at the Operating level. If a Rating Factor of 1.0 at the Inventory level is required, the base load factor $X_P$ should be 1.40/.73 = 1.92, and this value should be adjusted in accordance with Section 7.4.

Then:

$$IN = \frac{k_o L_p}{X_{PA}} \qquad (7\text{-}8)$$

and

$$OP = \frac{IN}{K_{IN}} \qquad (7\text{-}9)$$

The rating factors at the Inventory and Operating levels would be computed in accordance with equations 7-7 and 7-5, respectively.

## 7.6 LOAD AND RESISTANCE FACTOR METHOD

As an alternative to the above procedure which leads to operating and inventory ratings, the Load and Resistance Factor Rating method (LRFR) may be used as modified below. LRFR does not distinguish between operating and inventory levels. Rather, a single rating is found which considers the factors appropriate to the site loading, methods of inspection and the condition of the bridge and type of maintenance. The dead and live load factors, as well as the resistance or capacity factors, are given in the Guide Specifications.

To incorporate the proof test result, the live load factor may be based on the values given in the Guide Specification. In addition, the resistance factor may be increased by 0.05 to recognize the smaller levels of capacity uncertainty that exists following a proof test. The value to be used for resistance, R, in the LRFR equation should be the value $k_o L_p$.

# CHAPTER 8

# POSTING AND PERMIT CONSIDERATIONS

## 8.1 USE OF LOAD TEST RESULTS IN RATING ANALYSIS

Agencies are required to rate bridges using either allowable stress or load factor methods. Usually, both inventory and operating load levels are reported. For purposes of comparison, these ratings must be uniformly interpreted and have the same calculation basis. The selection of an appropriate rating level for site-specific decisions about rehabilitation, posting and permit load reviews are discussed in this chapter.

The application herein of diagnostic testing is to allow adjustments to the rating calculation made prior to the load test of some portion of any enhanced load capacity observed during the test. As explained in Chapter 6, the full enhancement observed in a test may not always be allowed since the rating refers to a full legal vehicle loading and the diagnostic test is usually performed with smaller vehicle loads. To determine to what extent the measured enhancements may be used, the procedures given in Chapter 6 should be followed.

The remainder of the rating calculation is a scaling of the diagnostic test performance to reach the limits prescribed by the agency's rating method, e.g., allowable stress, load factor or load and resistance factors. The safety levels expressed in the rating correspond to the values associated with the rating level, e.g., Operating, Inventory or LRFR. No attempt is made in calculating these ratings to utilize any subjective benefits in terms of greater confidence in the capacity following a test compared, say, to a rating without benefit of a test. This aspect of interpretation is given in the next sections where posting and permit applications are discussed. By separating the application decisions from the actual ratings reported, it is possible to maintain the uniformity of the reporting system.

In a similar manner, if a proof test is done, then a safe strength capacity is established. Load rating calculation may again be done as explained in Chapter 7. Again, both Inventory and Operating levels can be identified and reported as needed. Uniformity of reporting ratings is maintained by this process.

## 8.2 USE OF LOAD TEST RESULTS IN POSTING ANALYSIS

The use of ratings for posting is based partly on the overall safety assessment of the structure. According to the C/E Manual, this assessment should include the confidence of the rater in such variables as traffic enforcement, inspection, and maintenance. In fact, agencies may select as a rating basis for posting, the values between Inventory and Operating levels or even the LRFR value. Each Bridge Owner should develop a uniform approach to posting based on load test results which is consistent with their past practices.

## 8.3 USE OF LOAD TEST RESULTS IN PERMIT DECISIONS

Load tests may be used to describe capacity for purposes of reviewing special permit loads which exceed the normal legal levels. These tests should be carried out using a load pattern similar to the effects of the permit vehicle. Special consideration should be given in the interpretation of the tests and the review of the permit load calculations to the following: (1) Will other traffic be permitted on the bridge when the permit load crosses the structure?; (2) Will the load path of the vehicle crossing the bridge be known in advance, and can it be assured?; (3) Will the speed of the vehicle be controlled to limit dynamic impact?; and (4) Will the bridge be inspected after the movement to ensure that the bridge is structurally sound?

Based on these considerations the results of the bridge load test, whether diagnostic or proof, can be extrapolated to provide a basis for the review of requests for permit vehicles. If a diagnostic test has been performed, then test results should be used to predict the response of the bridge to the permit vehicle. The same modifications and reduced use of any enhancements in capacity observed during the test shall apply to the permit evaluation in the same way as discussed with the rating computation. Similarly, if the test is a proof load, it is necessary that the load effects of the test vehicles exceed the permit effects. A safety margin will also be needed to account for variations in weight of the permit trucks, the position of the loading, and possible dynamic effects.

## 8.4 RETESTING

When the load capacity of a bridge has been established through load testing, the bridge may require retesting in the future. Retesting should be performed at the same intervals used by the Bridge Owner for recalculating load ratings for bridges which have not been load tested. However, whenever conditions at the bridge site change materially from those which existed at the time the bridge was last tested, the bridge should be load tested again. Where a bridge has been load tested more than once, the results from all tests should be compared and analyzed.

## CHAPTER 9

## ILLUSTRATIVE EXAMPLES

### 9.1 APPLICATION OF DIAGNOSTIC LOAD TEST PROCEDURES

The procedures described in Chapter 6 will be illustrated by applying them to typical highway bridges.

In the examples which follow, "Manual" refers to the proposed manual for "Bridge Rating Through Load Testing," "AASHTO" refers to the AASHTO "Standard Specifications for Highway Bridges" (36), "C/E" refers to the "Manual for Condition Evaluation of Bridges" (35), and "Guide" refers to the AASHTO "Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges" (37).

#### 9.1.1 Multi-Girder Steel Composite Bridge

Given: A 65' long, simple span highway bridge as shown below.

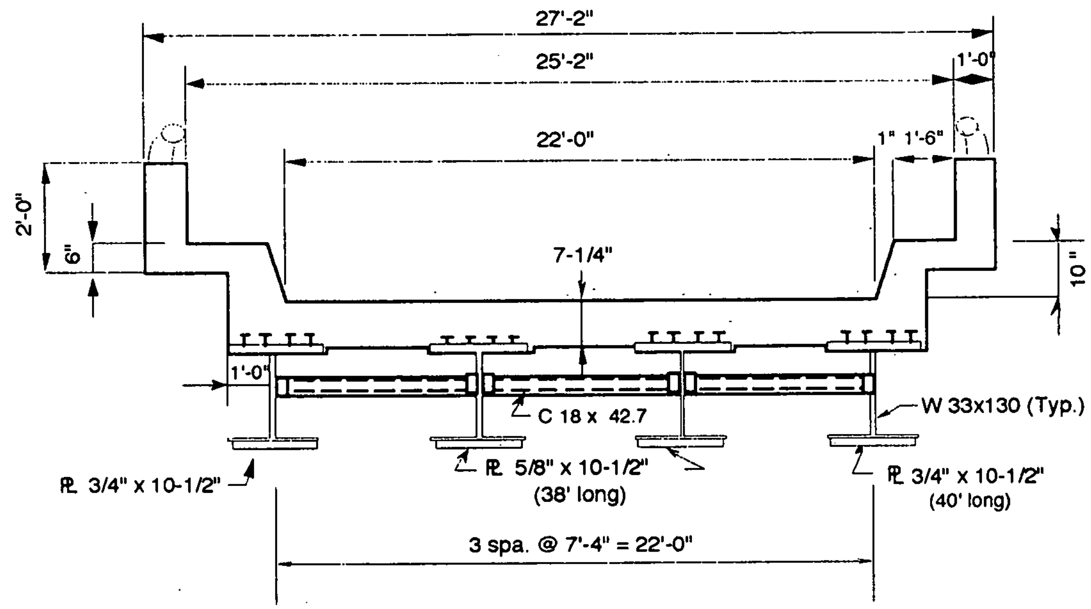



FIGURE 9-1: Section (No Scale)

| | | |
|---|---|---|
| Materials: | A36 Steel - $F_y$ = 36 ksi | Year built: 1964 |
| | $f'_c$ = 3,000 psi | Redundant (multi-stringer) |

Conditions at Site of Bridge:

ADTT > 1000 with little likelihood of overloads, i.e. good enforcement.
Maintenance is good and no deterioration was noted.
The approaches and wearing surface are smooth and in good condition.
Inspections are routinely performed.

The bridge was rated analytically using the three methods described in the proposed AASHTO C/E Manual. Table 9-1 summarizes the analytical ratings.

TABLE 9-1 ANALYTICAL RATING RESULTS

| | HS20 Truck | | HS 20 Truck | |
|---|---|---|---|---|
| | $RF_c$ | R (tons) | $RF_c$ | R (tons) |
| Allowable Stress: | | | | |
| Inventory | 0.74 | 26.7 | 1.05 | 21 |
| Operating | 1.35 | 48.7 | 1.93 | 38.3 |
| Load Factor: | | | | |
| Inventory | 1.00 | 36 | 1.42 | 28.4 |
| Operating | 1.67 | 60 | 2.37 | 47.4 |
| Load and Resistance Factor | 1.45 | 52 | 2.06 | 41.2 |

The rating factor is less than 1 only when evaluated at Inventory level using the Allowable Stress method. However, for convenience, this bridge will be used to illustrate the application of diagnostic load testing since it is the same bridge evaluated in Appendix B of the C/E Manual.

The analytical rating was based on AASHTO design distribution and impact factors. A typical interior stringer was idealized as a simply-supported beam and basic statics were used to find maximum moments due to dead and live loads. The stresses produced by these moments may be found by applying the appropriate section modulus. The data available from the analytical rating of a typical interior stringer, which is pertinent to this diagnostic test, are the following:

- Non-composite section modulus to bottom of steel at maximum moment section — $SF_{nc} = 564 \text{ in}^3$
- Composite section modulus to bottom of steel at maximum moment section — $SF_c = 788 \text{ in}^3$
- Maximum live load moment plus impact due to rating vehicle — $L_R(1+I) = 751'^k$ (stringer moment including AASHTO design distribution)
- Maximum dead load moment = $(439'^k + 129'^k) = 568'^k$
- AASHTO factors — $I = 0.26$; $DF = 1.33$

A diagnostic test was designed to verify the composite behavior of the bridge *system* and the AASHTO design distribution factor. It was decided to place strain gages on the bottom flange of each steel stringer near the maximum moment point (near midspan). The test truck was then placed in various longitudinal and transverse positions across the bridge deck, first with the cab facing in one direction and then with the cab facing in the other direction. For each position of the test truck the strains in each stringer were recorded. The strains were also monitored during the test to ensure elastic behavior as the truck moved closer to midspan and to check that there was no permanent strain after the test truck was removed from the bridge. The test vehicle used during the test is shown in Fig. 9-2.

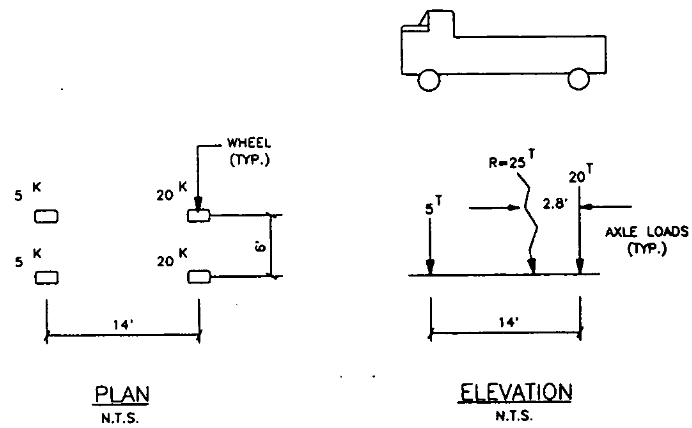



FIGURE 9-2: Test Vehicle

The maximum strain recorded was 130 microinches and occurred in stringer S2 when the truck was positioned with one wheel line directly over the stringer as shown in Figure 9-3:

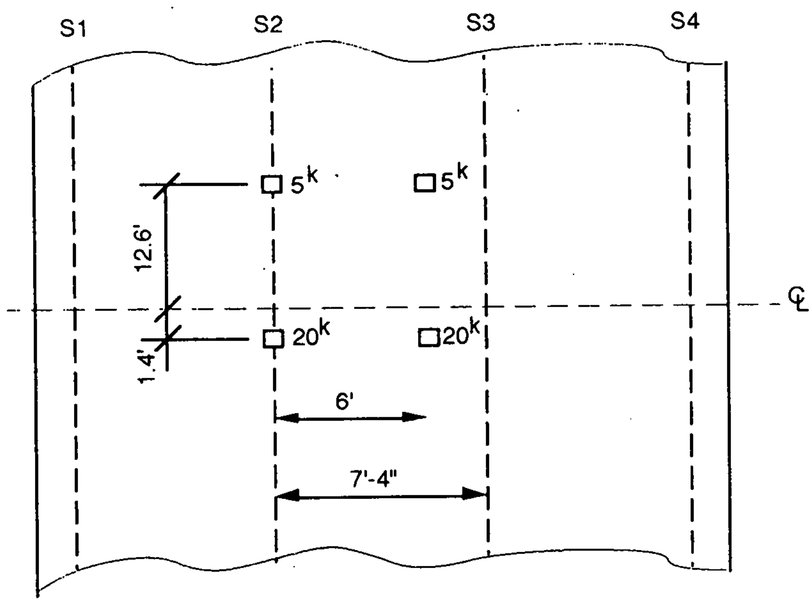
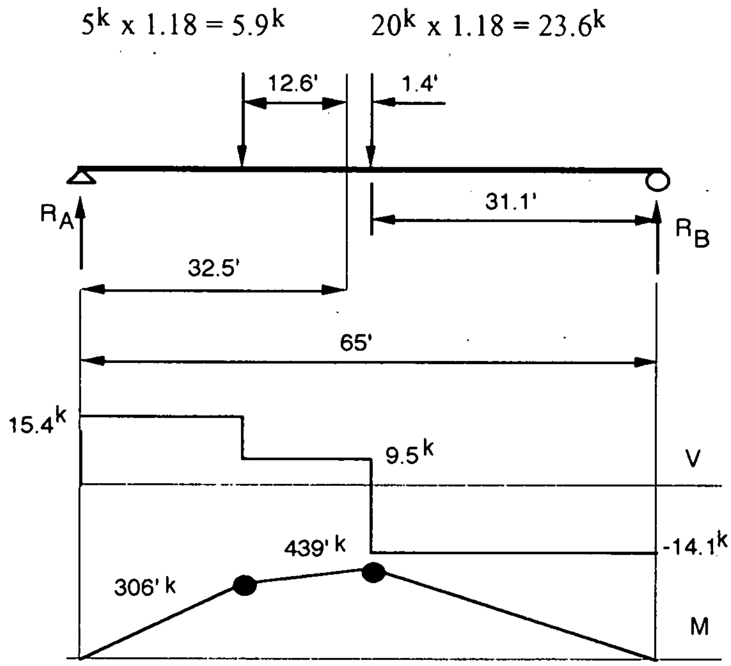



FIGURE 9-3: Position of Truck Which Resulted in Maximum Strain in S2

Based on statics and a simple beam distribution factor for the wheels not directly on the stringer of (1 + 1.33'/7.33) = 1.18, the idealized stringer S2 under the test truck load is shown in Figure 9-4:




$$R_B = \frac{5.9(19.9) + 23.6(33.9)}{65}$$

$$R_B = 14.1^k$$

$$R_A = 15.4^k$$

FIGURE 9-4

The maximum theoretical moment produced by the test truck in stringer S2 is:

$$L_T = 439'^k$$

Applying Eqn. 6-5, the theoretical bottom-flange maximum strain resulting is the maximum moment divided by the composite section modulus:

$$\varepsilon_c = \frac{L_T}{(SF)\ E} = \frac{439'^k \times 12''/FT}{788^{in3} \times 29 \times 10^3 \text{ ksi}} = 231 \times 10^{-6} \text{ in/in}$$

If there was slippage between the steel stringer and the concrete deck, the resulting strain would be more appropriately measured by the noncomposite section modulus ($439 \times \frac{12}{(565 \times 29 \times 10^3)} = 322 \times 10^{-6}$ in/in).

The measured strain in the bottom flange at the maximum moment section resulting from this position of the truck is:

$$\varepsilon_T = 130 \times 10^{-6} \text{ in/in} = 130 \text{ microinches}$$

Using the measured strain and the corresponding moment results in an apparent section modulus ($SF_A$) of:

$$SF_A = \frac{439'^k \times 12''/FT}{(130 \times 10^{-6})(29 \times 10^3 \text{ ksi})} = 1397^{in3}$$

The composite section modulus in accordance with AASHTO criteria and the rating calculations is 788 $in^3$, and will be used in this example as the section modulus resisting the applied test truck loads. The reasons for the difference between the AASHTO composite section modulus and the apparent section modulus are related to differences between the actual and assumed transverse and longitudinal distributions, as well as additional composite action beyond that specified by AASHTO.

The portion of the test benefit to be used, i.e. the difference between a measured strain of 130 microinches and a calculated strain of 231 microinches, will now be found in accordance with Section 6.5. The analytical rating $RF_c$ will be adjusted by a factor K to obtain a new rating $RF_T$ based on the diagnostic test results.

The adjustment coefficient includes two factors which must be evaluated.

First:

$$k_a = \frac{\varepsilon_c}{\varepsilon_T} - 1$$

$$k_a = \frac{231}{130} - 1 = 0.77$$

and second: $$k_b = k_{b1} \times k_{b2} \times k_{b3}:$$

$k_{b1}$ depends on ratio of T/W or their effects:

$$\frac{T}{W} = \frac{L_T}{L_R(1+I)} = \frac{439'k}{751'k} = 0.58$$

From Table 6-1, with T/W > 0.4 but < 0.7, and member behavior at 1.33W expected to be similar to that observed during test $\rightarrow k_{b1} = 0.8$.

$k_{b2}$ is found from Table 6-2 based on routine biennial inspections $\rightarrow k_{b2} = 0.8$

$k_{b3}$ is found from Table 6-3 based on fatigue details (welded cover plate) and redundant system $\rightarrow k_{b3} = 0.8$

Thus:

$$k_b = 0.8 \times 0.8 \times 0.8 = 0.51$$

then using equation 6-3:

$$K = 1 + k_a k_b$$

$$K = 1 + (0.77)(0.51) = 1.39$$

Finally,

$$RF_T = RF_c \times K$$

At the Inventory level:

$$RF_T = 0.74\ (1.39) = 1.03$$

The diagnostic test has resulted in an adjustment factor of 1.39 which could be applied to any rating level. The Inventory level was selected above to illustrate the application of K and to see if the test results provided sufficient improvement to obtain an Inventory rating factor of 1 or more.

The HS20 ratings for this bridge before and after the load test are summarized in Table 9-2. The after test values are obtained by multiplying the before test values by 1.39.

TABLE 9-2 HS20 RATINGS

| | Before Test | | After Test | |
|---|---|---|---|---|
| | $RF_c$ | R (tons) | $RF_T$ | R (tons) |
| Allowable Stress: | | | | |
| Inventory | 0.74 | 26.7 | 1.03 | 37.1 |
| Operating | 1.35 | 48.7 | 1.88 | 67.6 |
| Load Factor: | | | | |
| Inventory | 1.00 | 36 | 1.39 | 50.0 |
| Operating | 1.67 | 60 | 2.32 | 83.6 |
| Load and Resistance Factor | 1.45 | 52 | 2.02 | 72.6 |

9.1.2 Multi-Girder Non-Composite Bridge

The bridge is the same as defined in Section 9.1.1 except that for this example non-composite action between the deck and steel girders will be assumed. On this basis, the bridge was rated analytically using the Load Factor Method. Table 9-3 below summarizes the load ratings.

TALBE 9-3 ANALYTICAL RATING RESULTS

| | HS20 Truck | | H20 Truck | |
|---|---|---|---|---|
| | $RF_c$ | R (tons) | $RF_c$ | R (tons) |
| Load Factor: | | | | |
| Inventory | 0.46 | 16.6 | 0.65 | 13.0 |
| Operating | 0.76 | 27.4 | 1.08 | 21.6 |

The analytical rating was based on AASHTO design distribution and impact factors. The non-composite, steel only, section properties for a typical interior girder were used in this analysis.

A diagnostic test was conducted, as described in Section 9.1.1. During the test, no slippage between the concrete deck and the top flange of the steel girder was noted as the test load was applied. This indicates that the deck and girders were acting compositely during the test.

The results of the diagnostic test are given in Section 9.1.1 and are summarized below for convenience:

- The applied test load resulted in a maximum moment of $L_T = 439$ k-ft
- Based on this maximum moment and the composite section which resisted the test load, the theoretical strain is $\varepsilon_c = 231$ microinches.
- The measured strain due to the test load is $\varepsilon_T = 130$ microinches.
- The apparent section modulus based on the measured strain is $SF_A = 1397$ in$^3$.
- The adjustment coefficient (k) is evaluated in Section 9.1.1 and k = 1.39. Note there is no difference in the evaluation of this coefficient in terms of whether or not composite action was intended by the designers.

Hence, the rating factor at the Inventory level becomes:

$$RF_T = RF_C \times k$$

$$RF_T = 0.46(1.39)$$

$$RF_T = 0.64$$

This results in only a slight improvement in the load rating for this bridge. However, the engineer may also elect, based on the test results, to recompute the value of $RF_c$ considering some composite action available to resist the applied loads. For example, if the full composite section was acting, then the rating factors would be the same as for the example in Section 9.1.1. Thus, at Inventory level:

$$RF_T = 1.03 \quad \text{say } 1.0$$

and the rating load (W) is

$$W = 1.0\ (36\text{ tons}) = 36\text{ tons or } 72\text{ kips}$$

To show that composite action can be relied on at 1.33W, the limiting bond stress criteria method will be used. For an HS20 truck, W = 72 k and 1.33W = 96 k. The total dead load stress in the bottom of the steel is:

$$f_{DL} = \frac{M_{DL}}{SF_{nc}} = \frac{568\text{ ft-k} \times 12\text{ in/ft}}{564\text{ in}^3} = 12.1\text{ ksi}$$

The maximum stress available for live load, assuming loading can continue until first yielding, is:

$$f_{LL} = F_y - f_{DL} = 36 \text{ ksi} - 12.1 \text{ ksi} = 23.9 \text{ ksi}$$

and the corresponding live load strain is

$$\varepsilon_{LL} = \frac{f_{LL}}{E} = \frac{23.9 \text{ ksi}}{29000 \text{ ksi}} = 824 \times 10^{-6} \text{ in/in}$$

The maximum live load strain due to the test truck ($T=50^k$) was $\varepsilon_T = 130 \times 10^{-6}$ in/in.

Then, if full mechanical connection between the slab and steel stringer was present, designed in accordance with AASHTO, the maximum truck weight which could be placed on the bridge with composite action controlling is:

$$T_{max} = \frac{\varepsilon_{LL}}{\varepsilon_T} \times T = \frac{824}{130} \times 50 = 317\text{k}$$

However, in the absence of such connections, the maximum truck weight will be based on the limiting bond stress between the slab and steel flange. From classical strength of materials, the horizontal shear stress, $p_h$, is given by:

$$p_h = \frac{V b_1 d_1 (\bar{y} - .5\, d_1)}{n I_c}$$

where:

$V$ = the vertical shear force (kips) due to the test truck.

$b_1$ = the effective width (in.) of the concrete slab per AASHTO.

$d_1$ = the depth (in.) of the concrete slab.

$\bar{y}$ = the distance (in.) from the top of the concrete slab to the neutral axis of the composite section for live loads.

$I_c$ = the moment of inertia ($in.^4$) of the composite section for live loads.

$n$ = modular ratio, $\frac{E_s}{E_c}$.

For this example:

$V = 15.4^k$
$b_1 = 87"$
$d_1 = 7.25"$
$\bar{y} = 13.035"$
$I_c = 22{,}007 \text{ in}^4$
$n = 10$

Then:

$$p_h = \frac{(15400 \text{ lbs})(87")(7.25")\left(13.035" - \frac{7.25"}{2}\right)}{(10)(22007 \text{ in}^4)}$$

$$p_h = 415 \text{ lb/in}$$

and the interface shear stress across the width of the top steel flange (bf) is:

$$\rho_h = \frac{p_h}{b_f} = \frac{415 \text{ lb / in}}{11.51 \text{ in}} = 36 \text{ psi}$$

This interface shear stress is limited by the bond stress criteria presented in Section 3.2. For $f'_c = 3{,}000$, the bond stress is 70 psi. Since $\rho_h = 36$ psi is less than 70 psi, the bond stress has not been exceeded, and there is a reserve capacity.

The truck weight which would result in reaching a shear (bond) stress of 70 psi is:

$$T_{max\ B} = \left(\frac{70 \text{ psi}}{36 \text{ psi}}\right) \cdot \left(50 \text{ k}\right) = 97 \text{ k}$$

Since this is greater than 1.33W, we can count on the composite action up through at least this load level.

The HS20 rating for this bridge before and after the load test are summarized in Table 9-4. The after test values are based on using a composite section and an adjustment factor of 1.39.

TABLE 9-4
HS20 RATINGS

| | Before Test | | After Test | |
|---|---|---|---|---|
| | $RF_C$ | R (tons) | $RF_T$ | R (tons) |
| Load Factor: | | | | |
| Inventory | 0.46 | 16.6 | 1.39 | 50.0 |
| Operating | 0.76 | 27.4 | 2.32 | 83.6 |

### 9.1.3 Simple Span Steel Truss Bridge

Given: A simple-span truss bridge, as shown in Figure 9-5, has been load rated analytically for AASHTO H20 loading. The load capacity of the bridge is limited by member $L_2L_3$, a tension member in the bottom chord of the main supporting truss. Since this is a truss, the load effect selected for evaluation was axial force. All dead loads were based on field measurements and the bridge was in good condition with little deterioration noted. The bridge carries less than 1000 ADTT, with a limited potential for overloads due to restrictive site conditions. The bridge deck is in good condition and is maintained on a regular schedule. The bridge was built in 1922 and the trusses are nonredundant. Routine inspections of the bridge are performed every two years since fatigue is not a problem.

Based on the analytical rating, the following is known for member $L_2L_3$:

| | | |
|---|---|---|
| Net area | $A_n$ | = 7.375 sq. in. |
| Yield stress of steel | $F_y$ | = 30 ksi (by coupon tests) |
| Dead load force | D | = 76.5 kips |
| Live load force due to Rating load (H20) with distribution | $L_R$ | = 53.8 kips |
| Impact | I | = 0.224 (AASHTO) |

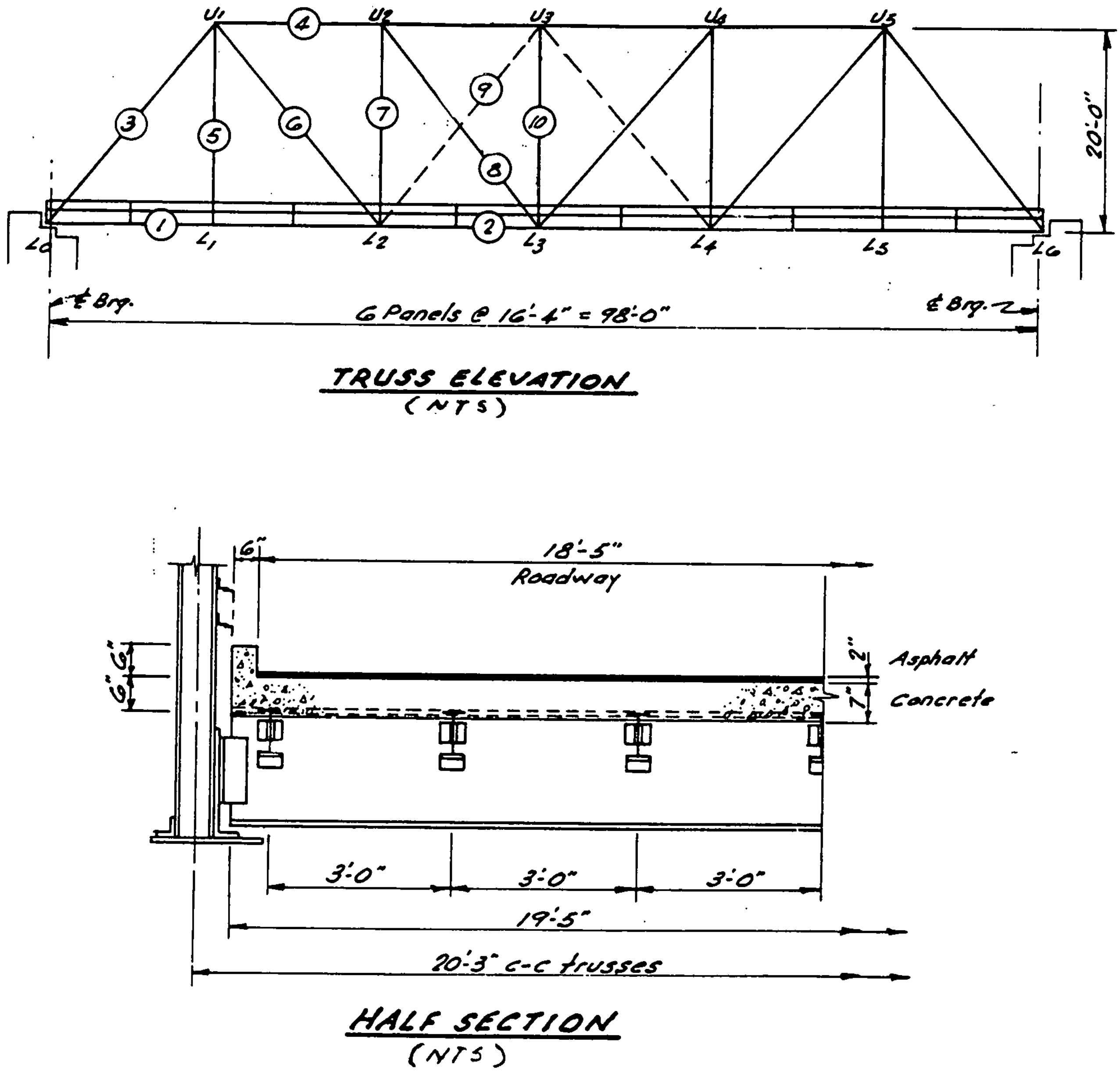



FIGURE 9-5: Truss Bridge

The analytical rating factors without the benefit of the load test for member $L_2L_3$ are:

<u>Allowable Stress Method</u>

@ Inventory Level

$$RF_C = \frac{(7.375)(0.55)(30)-76.5}{53.8(1+0.224)}$$

$$RF_C = 0.69$$

@ Operating Level

$$RF_C = \frac{(7.375)(0.75)(30)-76.5}{53.8(1+0.224)}$$

$$RF_C = 1.36$$

<u>Load Factor Method</u>

@ Inventory Level

$$RF_C = \frac{(7.375)(30)-1.3(76.5)}{2.17(53.8)(1+0.224)}$$

$$RF_C = 0.85$$

@ Operating Level

$$RF_C = \frac{(7.375)(30)-1.3(76.5)}{1.3(53.8)(1+0.224)}$$

$$RF_C = 1.42$$

Since the rating factor is less than 1.0 at Inventory level, a diagnostic load test was performed. The purpose of the test was to confirm the model used in the analytical rating and to determine if a higher load rating was possible. The results of this load test indicated that the maximum axial strain measured in member $L_2L_3$ was $\varepsilon_T = 51$ microinches when a 12-ton, two-axle dump truck was placed 2'-5" from the truss centerline with the heavy axle at panel point $L_2$. Using the influence line for $L_2L_3$ and the axial load distribution for the truck, the theoretical axial force in member $L_2L_3$ due to this position of the test truck is $L_T = 18.4$ kips, which corresponds to a theoretical strain of $\varepsilon_c = \frac{18.4}{(7.375 \text{x} 29 \text{x} 10^3)} = 86$ microinches.

All critical truss members, i.e. those carrying the greatest forces and those in the worst condition, were monitored during the load test using strain gages. The instrumentation was installed and monitored by experienced test engineers. The dump truck was frequently removed from the bridge to check the "zero" or no-load condition. Certain load positions were tested more than once to ensure consistent and repeatable results.

The difference between the measured axial strain (51 microinches) for member $L_2L_3$ and that predicted by theory (86 microinches) could be explained in part by the longitudinal distribution of the truck weight through the deck, stringer, and floor beam system.

Applying the proposed rating procedure (Section 6.5) for member $L_2L_3$, the load test results are as follows:

The analytical rating $RF_C$ will be adjusted by the factor K (Eqn. 6-3), to obtain a new rating $RF_T$ based on the diagnostic test results.

First:

$$k_a = \frac{\varepsilon_c}{\varepsilon_T} - 1$$

$$k_a = \frac{86}{51} - 1 = 0.69$$

and

$$k_b = k_{b1} \times k_{b2} \times k_{b3}$$

$k_{b1}$ depends on ratio of T/W or their effects. Using gross vehicle weights,

$$\frac{T}{W} = \frac{12 \text{ tons}}{20 \text{ tons}} = 0.6$$

From Table 6-1, with $T/W > 0.4$ but $< 0.7$ and member behavior at 1.33W expected to be similar to that observed during test $\rightarrow k_{b1} = 0.8$.

$k_{b2}$ is found from Table 6-2 based on routine biennial inspections $\rightarrow k_{b1} = 0.8$.

$k_{b3}$ is found from Table 6-3 based on no fatigue problems and a nonredundant system $\rightarrow k_{b3} = 0.9$.

Thus:

$$k_b = 0.8 \times 0.8 \times 0.9 = 0.58$$

Then using equation 6-3:

$$k = 1 + k_a k_b$$

$$k = 1 + (0.69)(0.58) = 1.40$$

Finally, the rating factors resulting from the results of this diagnostic load test are:

<u>Allowable Stress Method</u>

| | | | | | |
|---|---|---|---|---|---|
| @ Inventory Level | $RF_T$ | = | 1.40(0.69) | = | 0.97 or 19.4 tons |
| @ Operating Level | $RF_T$ | = | 1.40(1.36) | = | 1.90 or 38 tons |

<u>Load Factor Method</u>

| | | | | | |
|---|---|---|---|---|---|
| @ Inventory Level | $RF_T$ | = | 1.40(0.85) | = | 1.19 or 23.8 tons |
| @ Operating Level | $RF_T$ | = | 1.40(1.42) | = | 1.99 or 39.8 tons |

Thus, some improvement in the load rating has been gained. More importantly, the diagnostic load test results provide additional confidence in the rating methods, assumptions used and ability of bridge to carry a given load.

## 9.2 APPLICATION OF PROOF LOAD TEST PROCEDURES

The procedures described in Chapter 7 will be illustrated by applying them to typical highway bridges.

### 9.2.1 Multi-Girder Steel Composite Bridge

The bridge is the same as defined in Section 9.1.1. It is in good condition with an ADTT > 1000. Routine inspections are performed annually with special emphasis on the fatigue prone areas near the ends of the welded cover plates.

The bridge was proof tested in order to obtain an Inventory level rating of 1.0 or more. Some Agencies may find it unnecessary to "proof" test this bridge since it "rates" using load factor methods.

Since the objective of this proof test is to obtain a RF=1 at the Inventory level, a target proof load effect will be determined in accordance with Section 7.5.3. Thus:

$$X_p = 1.92$$

The adjustments to the basic load factor per Section 7.4.1 are found as follows:

1. More than one lane to be loaded during the test, no increase required.
2. No fracture critical details, no increase required.
3. Routine inspection plus fatigue prone area observations every year, no increase required.
4. Bridge load rating computed, decrease by 5%.

From the above the net *decrease* in $X_p$ is 5% or .05. Then:

$$X_{pA} = (1 - .05)\, X_p = .95\,(1.92) = 1.82$$

Based on an HS20 rating live load and AASHTO Specification Impact (I), the target proof load is found from equation 7-3 as follows:

$$L_T = X_{PA}\, L_R\,(1+I)$$

$$L_T = 1.82\text{ (36 tons)}(1+.26) = 82.6\text{ tons each lane}$$

The proof load consisted of four vehicles similar to the test vehicle illustrated in Figure 9-2. Prior to applying the proof load to the bridge, deflection gages were installed at the midspan of all stringers to monitor their vertical deflection during the proof loading. The proof load was applied one vehicle at a time, and the deflections were recorded and compared with the previous loading stage to ensure linear elastic behavior.

The proof test was terminated when all four vehicles were on the bridge in the position shown in Figure 9-6. This loading position resulted in the largest deflection in stringer S2. Additional loads were added to each truck at the test site and the final axle loads for each truck were 8 tons on the front axle and 32 tons on the rear axles.

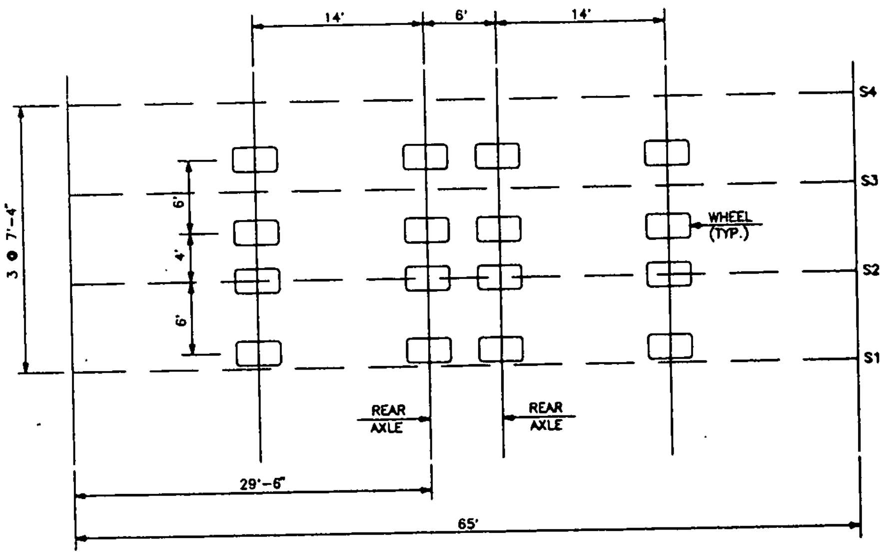



FIGURE 9-6: Plan

The final total proof load of 4 trucks times 40 tons each was 160 tons ($L_P$) which is slightly smaller than the target proof load 2 lanes time 82.6 tons or 165 tons. The test was terminated at this point since no further loading blocks were available at the bridge site. Since the load test was terminated when the target proof load was reached, $K_O = 1.0$ per Section 7.5.1.

By equation 7-8, the Inventory level capacity (IN) is:

$$IN = \frac{k_O L_P}{X_{PA}}$$

$$IN = \frac{1.0(160 \text{ tons}/2 \text{ lanes})}{1.82} = 44 \text{ tons}$$

and the Inventory level rating factor ($RF_I$) is:

$$RF_I = \frac{IN}{L_R(1+I)} = \frac{44 \text{ tons}}{36(1+.26)} = 0.97$$

Similarly,

$$OP = \frac{IN}{k_{IN}} = \frac{44 \text{ tons}}{0.73} = 60.3 \text{ tons}$$

and

$$RF_O = \frac{OP}{L_R(1+I)} = \frac{60.3 \text{ tons}}{36(1+0.26)} = 1.33$$

The inventory rating factor is slightly less than 1.0 due to loading limitations of the trucks used. It should also be noted that the proof load vehicles were placed in other transverse and longitudinal positions on the bridge to check all possible supporting members and load placement.

### 9.2.2 Multi-Girder Prestressed Concrete Composite Bridge

Florida's turnpike bridge over I-595 and the North New River Canal is a twin structure with five spans of 130-151-99-99-99 feet, respectively.

The bridge was originally designed with spans 1 and 2 (130' and 151') to be built as a continuous unit from post-tensioned segmental AASHTO Type V girders. The 99' spans were to be simply supported AASHTO Type IV girders. The contractor's engineer submitted a proposal to eliminate the post-tensioning and build spans 1 and 2 with simply-supported Type V girders at a savings of $104,000.

In order for the Type V girders to span 151' and not crack under the dead weight of the freshly poured slabs, temporary shoring of the girders at midspan was required. After the deck was poured and the concrete had gained adequate strength to act compositely with the girders, the shoring could be removed. The southbound structure was constructed in this manner and was completed in July, 1988.

In June, 1989 as the temporary shoring was being removed from the northbound structure, the west facia beam was struck by the pile cap of the temporary support. Apparently, when the eastern pile of the temporary support was demolished at ground level, the cap rotated toward the bridge and collided with the facia beam. This caused a 10-foot crack in the bottom flange of the girder at the centerline of the span. Concrete spalled off the girder at the point of impact and along the crack exposing several prestressing strands. The exposed strands showed no signs of damage and no other cracks could be found in the girder.

The stresses in the damaged girder were recalculated assuming that 7 strands were debonded in the damaged area. The calculations showed that the section is adequate for the ultimate AASHTO moment, but the reserve capacity was reduced by about 10%. Since the girder was still adequate, the damaged area was patched with epoxy, however, some concern about the adequacy of the girder and the effectiveness of repairs still existed.

Due to this innovative design and construction approach, along with the damaged girder in span 2, Florida's turnpike bridge over I-595 was identified for proof-load testing. The objectives of the load test were to check the validity of the design assumptions, establish the true strength and load rating and also, to evaluate the performance of the damaged girder.

Test Span

The northbound and southbound bridges are identical and consist of 5 simply-supported spans. Span 1 is 130-feet long and consists of 9 Type V girders spaced at 8'-1½". Span 2 consists of 12 Type V girders spaced at 5'-11" and is 152-feet long. Spans 3, 4 and 5 are 99-feet long and consist of 9 Type IV girders spaced at 8'-1½". The bridges are 68' wide from curb to curb and carry four, 12-feet lanes and two, 10-feet shoulders with typical crash barriers on either side. The slab is 7 inches thick and the bridges are skewed 20 degrees.

Span 2 of the northbound bridge was chosen as the test span because of its length and because of the temporary shoring used during construction. It is also the span that was damaged when the temporary shoring was removed. Span 2, shown in FIgure 9-7 consists of 12 Type V girders spaced at 5'-11". AASHTO specification impact for this span is I=0.18.

Computer Modeling

The bridge analysis was performed with a finite element analysis computer program. The program uses a finite element approach to solve for deflections and stresses in bridges and bridge components. The program allows the user to model many types of bridges and to load the bridges with a number of standard or user-defined vehicles. The finite element model is based on linearly elastic material.

This model uses standard beam elements to model the girders and shell elements to model the slab. The shell elements allow both bending and stretching of the slab, which takes into account the in-plane stresses developed in the slab. Lateral beam elements are used in this model to stiffen the slab over the girders.

Theoretical girder deflections were obtained from the program by loading the bridge model with the Florida DOT bridge testing vehicle. The results of these analyses were plotted against the actual deflections obtained during load testing.

Bridge Instrumentation

Instrumentation for measuring strain and deflections were installed at the centerline and at quarter point of each girder as shown in Figure 9-8. Instruments for measuring deflection were also placed at the bearings of girders 1, 3, 5, 6 and 7. Vertical deflections were measured with linear variable displacement transducers (LVDT's).

Testing Procedure

The plan for testing this bridge included positioning the two test vehicles in three (3) different positions on the span as shown in Figure 9-9. The load positions corresponded to the 12-foot-wide travel lanes on the bridge. Strain and deflection measurements were taken with the trucks in all three load positions. Load position 1 induced maximum shear in the girders, while load positions 2 and 3 induced maximum moments in various girders of the cross section.

After span 2 of the northbound bridge was instrumented as described above, the bridge was ready for testing. Each testing vehicle was loaded to an initial weight of 101 kips. Initial readings of all instrumentation were recorded with no loads on the structure. The trucks were then driven and placed on the three critical load positions on the bridge. Strain and deflection readings were measured and recorded at each load position. The trucks were then driven off the bridge.

The measured data was immediately analyzed, displayed and compared to the theoretical prediction. This process took approximately ten minutes. It was determined that all strains and deflections were within acceptable limits and that the loads could be safely increased. The loads on each truck were increased by 26 kips and the trucks were then driven back onto the bridge and readings were again taken with the trucks in all three load positions. This procedure was repeated until the trucks weighed 204 kips each.

Results and Discussion

The measured deflections at 1/4 and midspan for load positions 2 and 3 are shown in Figures 9-10 and 9-11. The measured deflections for load position 1 are similar, yet much lower and re not discussed in this section to avoid repetition. Figure 9-12 shows a comparison between the analytical and the maximum measured center-line deflections of the two intermediate girders for load position 2. It can be seen from the figure that the measured deflections are linear and well below the analytical prediction. The maximum measured deflection at centerline of span was 1" which is approximately 66 percent of the analytical prediction.

Comparisons between the analytical prediction and maximum measured deflection at midspan are shown in Figures 9-13 and 9-14 for the load positions 2 and 3.

Figure 9-15 shows the measured centerline strains of all girders under incrementally applied moment up to maximum ultimate live load for position 2. It can be seen from the figure that the measured strains in all the girders remained linear and fairly low throughout the whole range of loading. Assuming an $E_c$ value of 4000 ksi, the maximum measured stress at full ultimate live load was approximately 700 psi. The maximum measured stress was only 65% of the analytical predictions. No cracks or any other distress signs were observed under maximum applied loads.

The measured deflections and stresses in the damaged girder were low and did not represent any cause for alarm. The test results show that the damaged girder is adequate and the overall behavior of the bridge is excellent.

Load Rating Based on Proof Test Results

Since the objective of this proof test is to obtain an RF=1 at the Inventory level, the base value for $X_p$ is based on Section 7.5.3:

$$X_p = 1.92$$

The adjustments to the basic load factor per Section 7.4.1 are found as follows:

1. More than one lane loaded, no increase required.
2. No fracture critical details, no increase required.
3. Routine inspections every 2 years, no increase required. However, due to reported damage, increase $X_p$ by 5%.
4. Bridge can be rated analytically, but due to unknowns mentioned under background above, do not decrease $X_p$.

From the above, the net increase in $X_p$ is 5% or 0.05. Then:

$$X_{PA} = (1+0.05)\, X_p = 1.05(1.92)$$

$$X_{PA} = 2.02$$

Based on an HS20 rating live load, the target proof load is found from equation 7-3 as follows:

$$L_T = X_{PA}\, L_R(1+I)$$

$$L_T = 2.02(36^{tons})(1+0.18)$$

$$L_T = 86^{tons} \text{ each lane}$$

The actual load placed on the bridge was $204^k$ in each of two lanes.

Thus:

$$L_p = \frac{2 \text{ lanes x } 204^k}{2 \text{ kips/ton x 2 lanes}} = 102^T$$

Thus $k_o$ = 1.0 since more than the target proof load was placed on the bridge.

For 150 ft span length, the FLDOT test truck, when fully loaded, produces a maximum lane moment of 6,217 ft-k. Based on Appendix A of the AASHTO Specifications with a load factor of 2.82, the corresponding HS20 lane moment is 6,980 ft-k. Thus, the ratio of test load effect to rating load effect is:

$$\frac{6217 \text{ ft-k}}{6980 \text{ ft-k}} = 0.89$$

and

$$L_p = 102 \text{ tons x } 0.89 = 91 \text{ tons}$$

By equation 7-8, the Inventory level capacity (IN) is:

$$IN = \frac{k_o\, L_p}{X_{PA}}$$

$$IN = \frac{1.0(91 \text{ tons})}{2.02}$$

$$IN = 45 \text{ tons}$$

and the Inventory level rating factor $RF_I$ is:

$$RF_I = \frac{IN}{L_R(1+I)} = \frac{45 \text{ tons}}{36(1+0.18)} = 1.06$$

Similarly,

$$OP = \frac{IN}{k_{IN}} = \frac{45 \text{ tons}}{0.73} = 61.6 \text{ tons}$$

and

$$RF_o = \frac{OP}{L_R(1+I)} = \frac{61.6}{36(1+0.18)} = 1.45$$

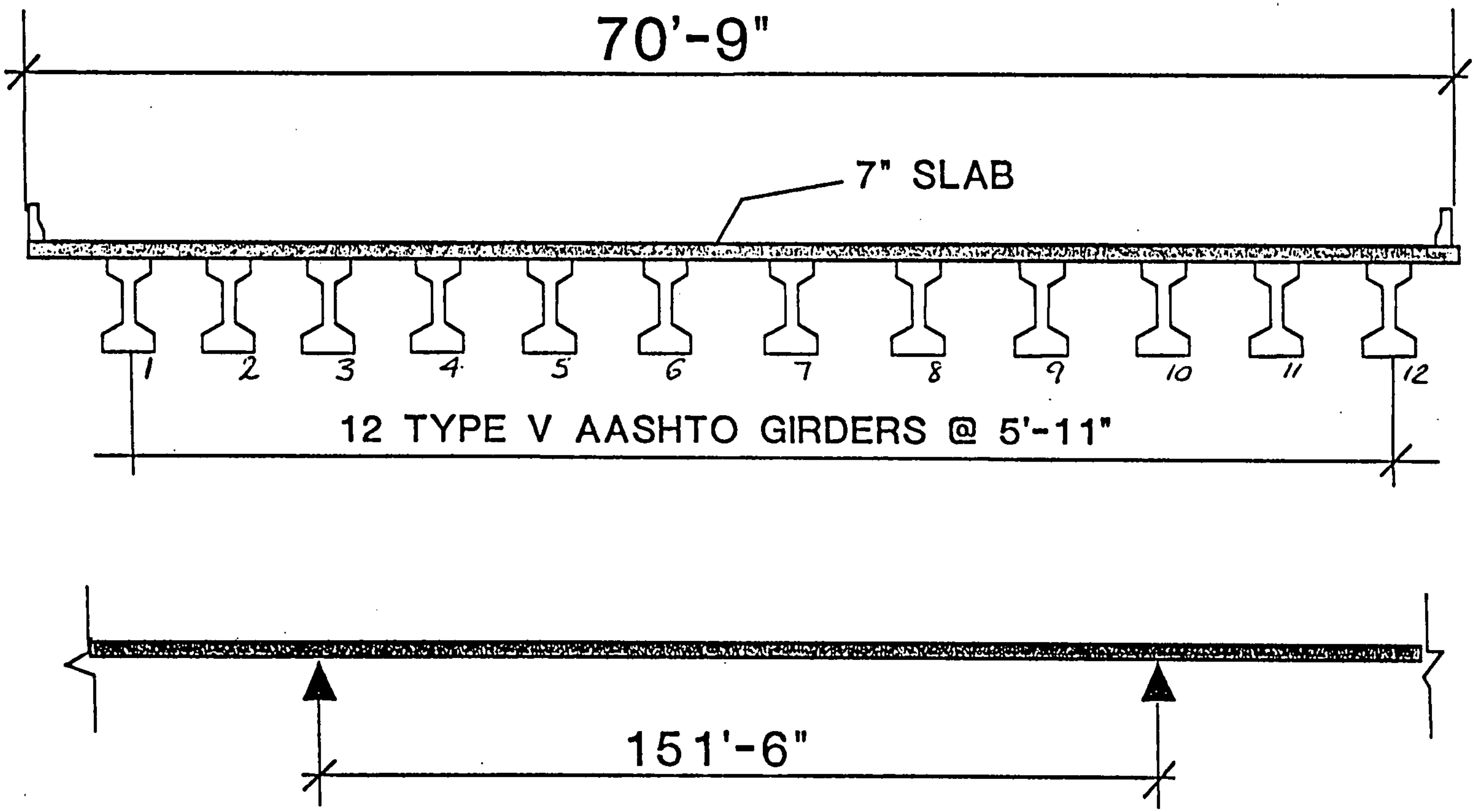



FIGURE 9-7: Dimensions and Cross Section of Test Span

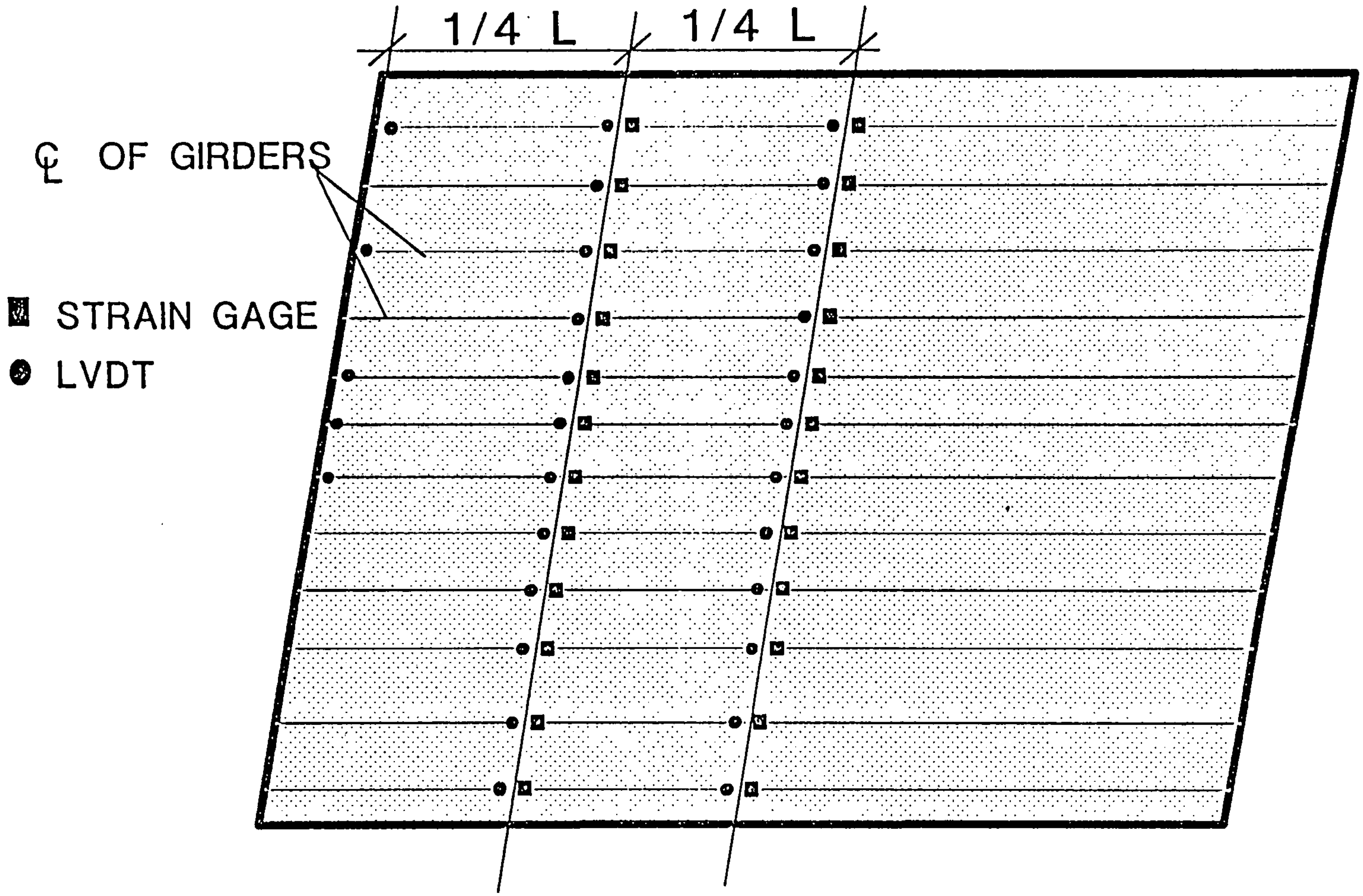



FIGURE 9-8: Locations of Strain and Deflection Gages

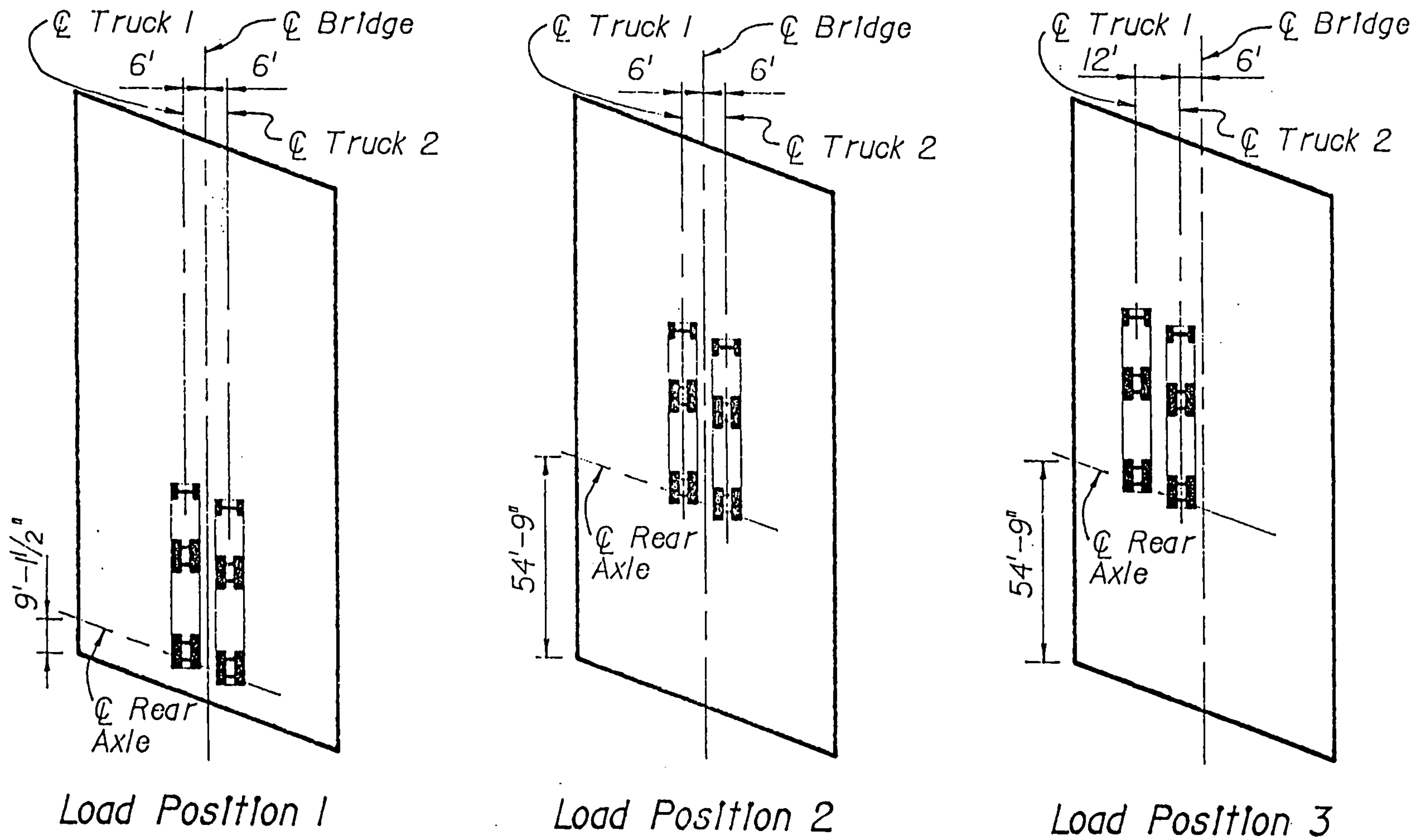



**FIGURE 9-9: Truck Positions on Bridge**

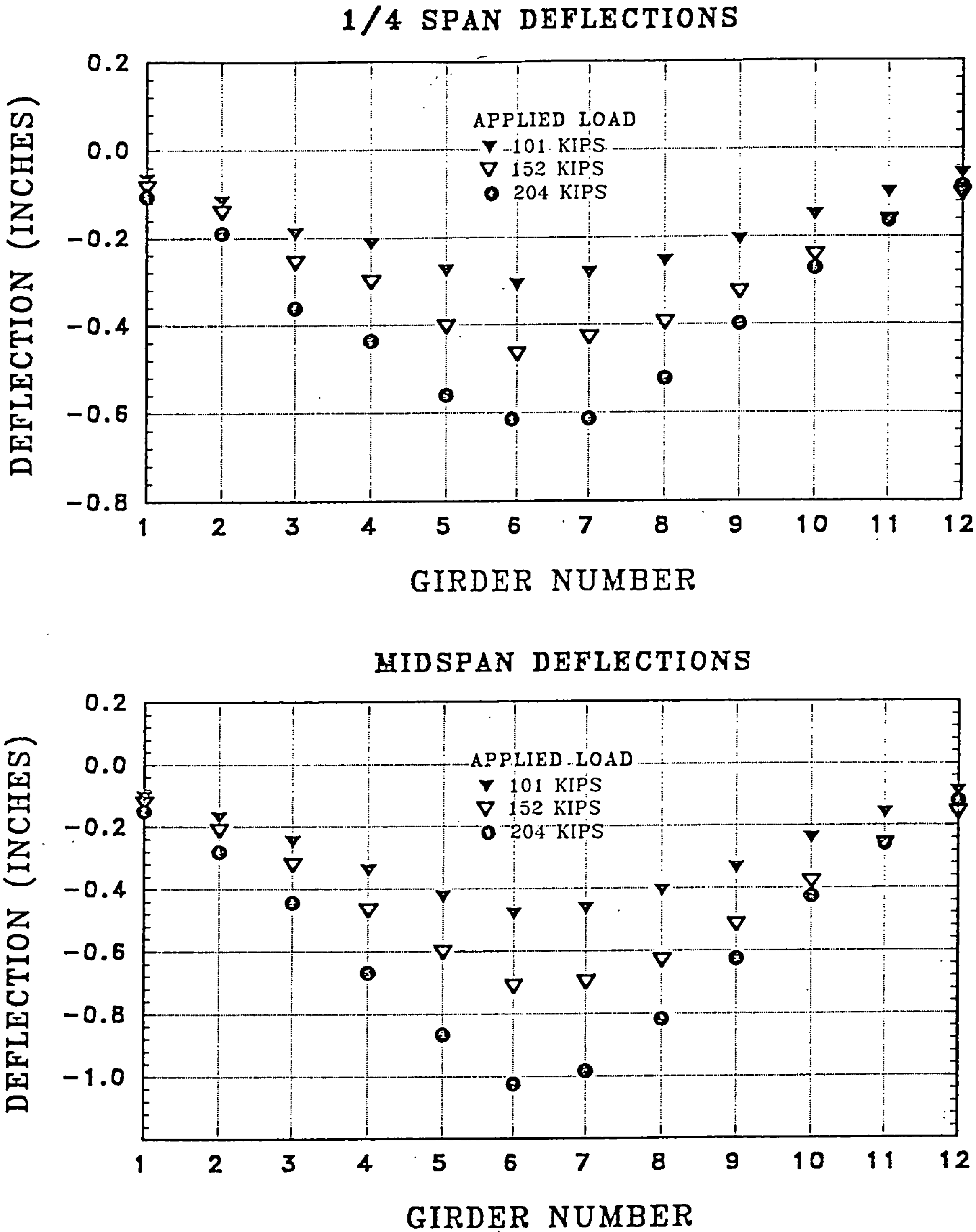
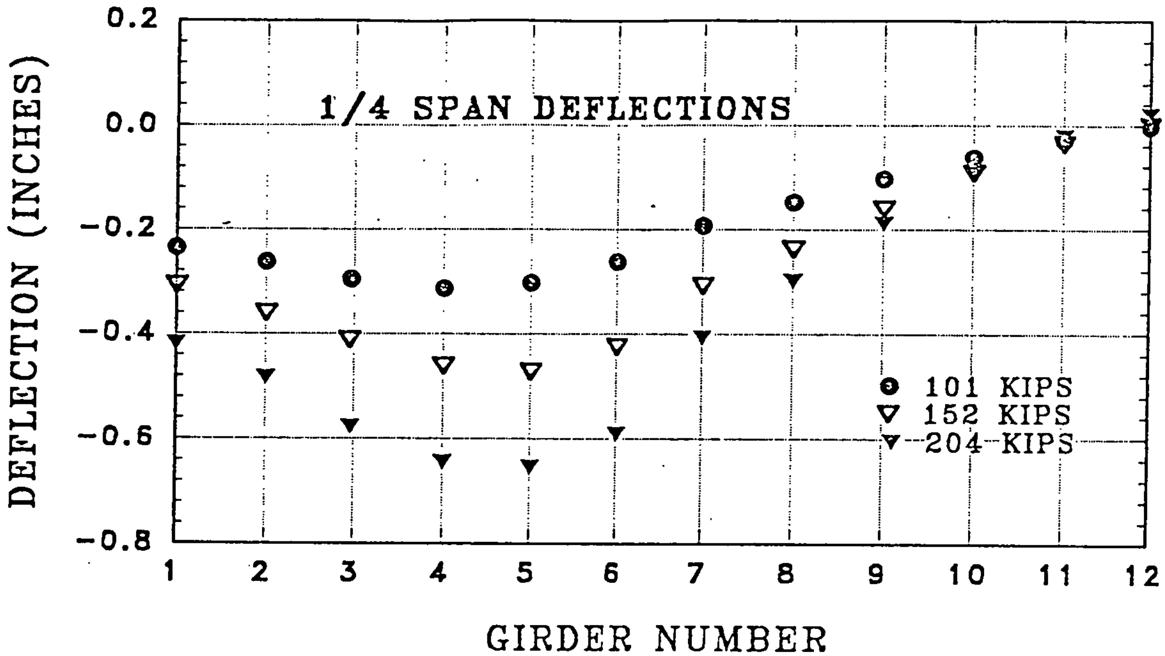



FIGURE 9-10: Measured Deflections for Load Position 2



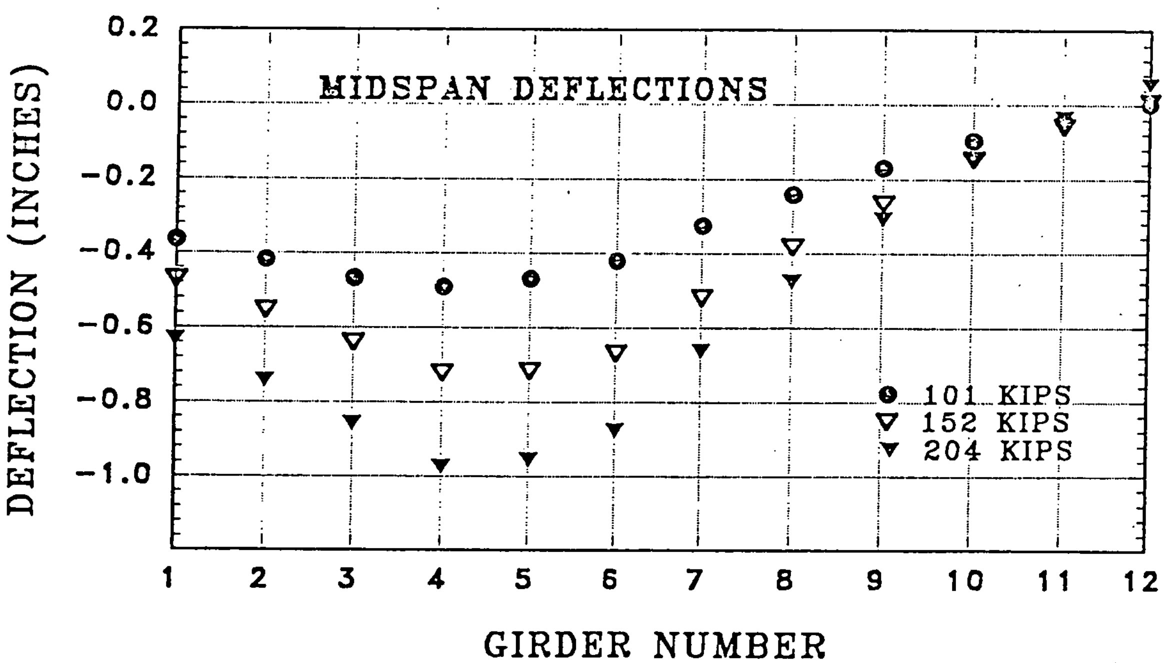




FIGURE 9-11: Measured Deflections for Load Position 3

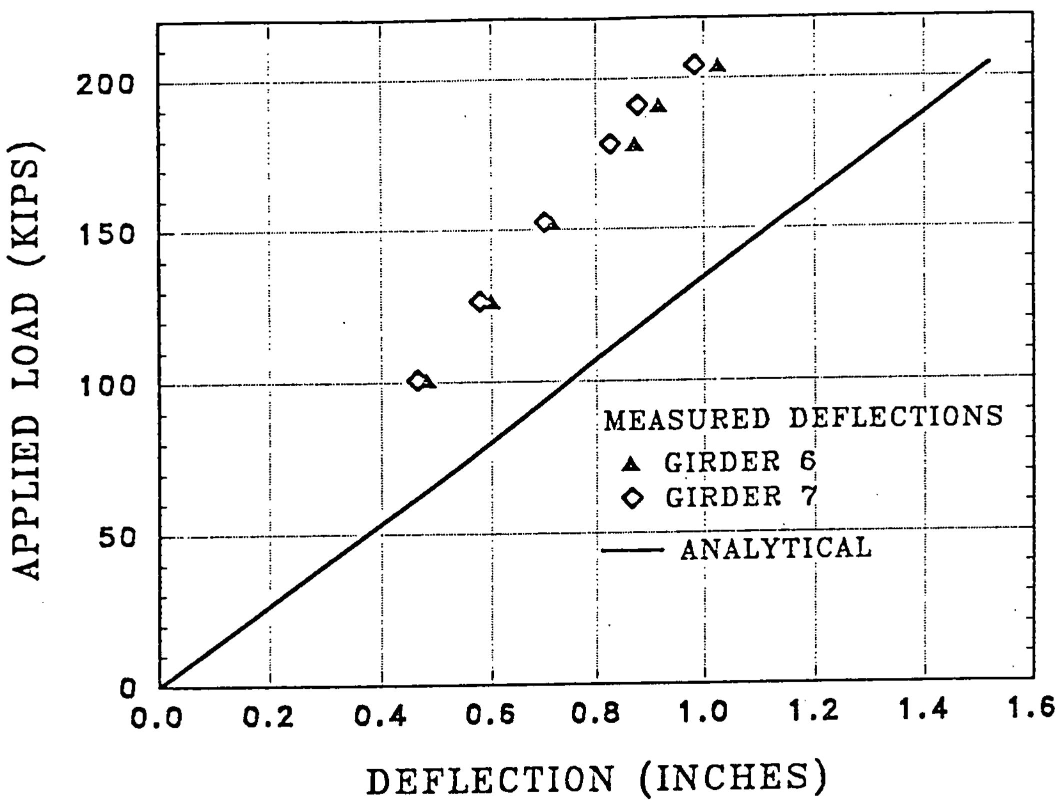



FIGURE 9-12: Applied Load vs. Midspan Deflections for Girders 6 and 7
(Load Position 2)

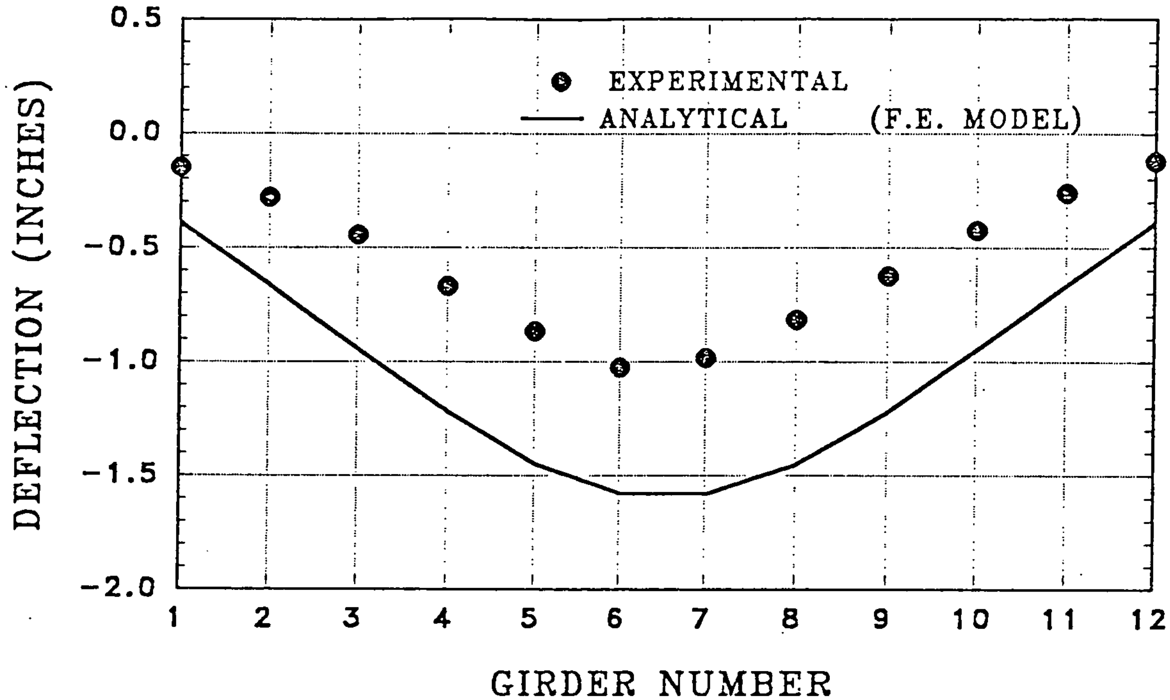



FIGURE 9-13: Midspan Deflections at 204 kips for Load Position 2

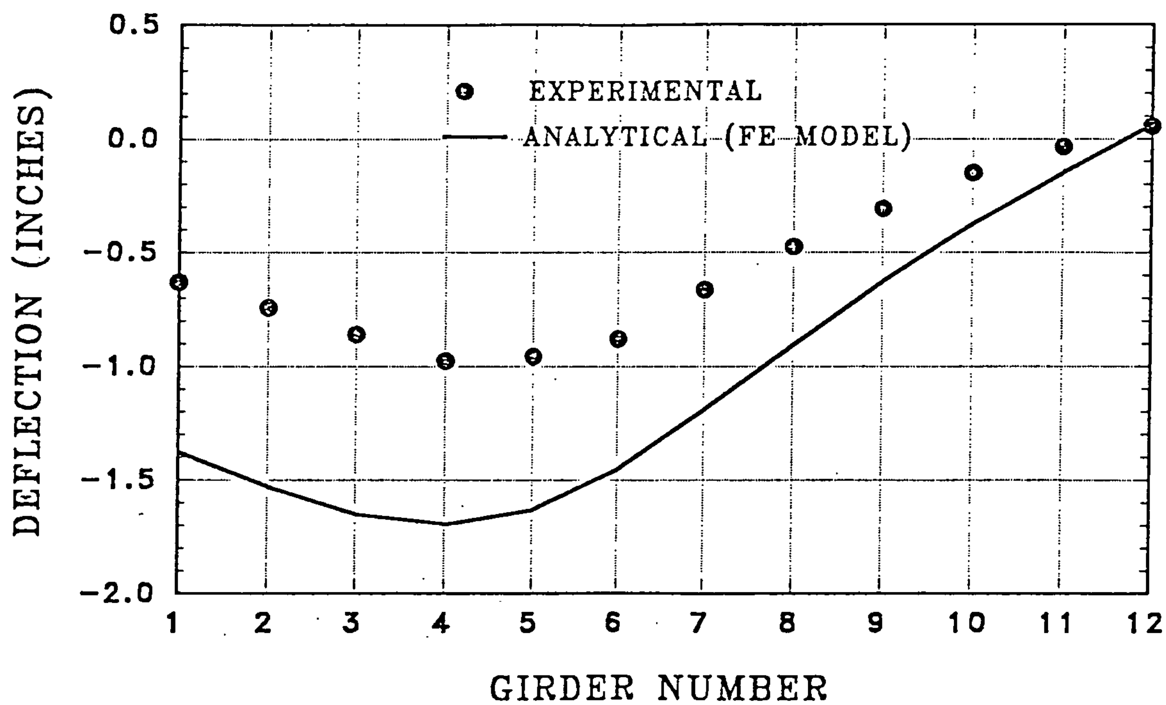



FIGURE 9-14: Midspan Deflections at 204 kips for Load Position 3

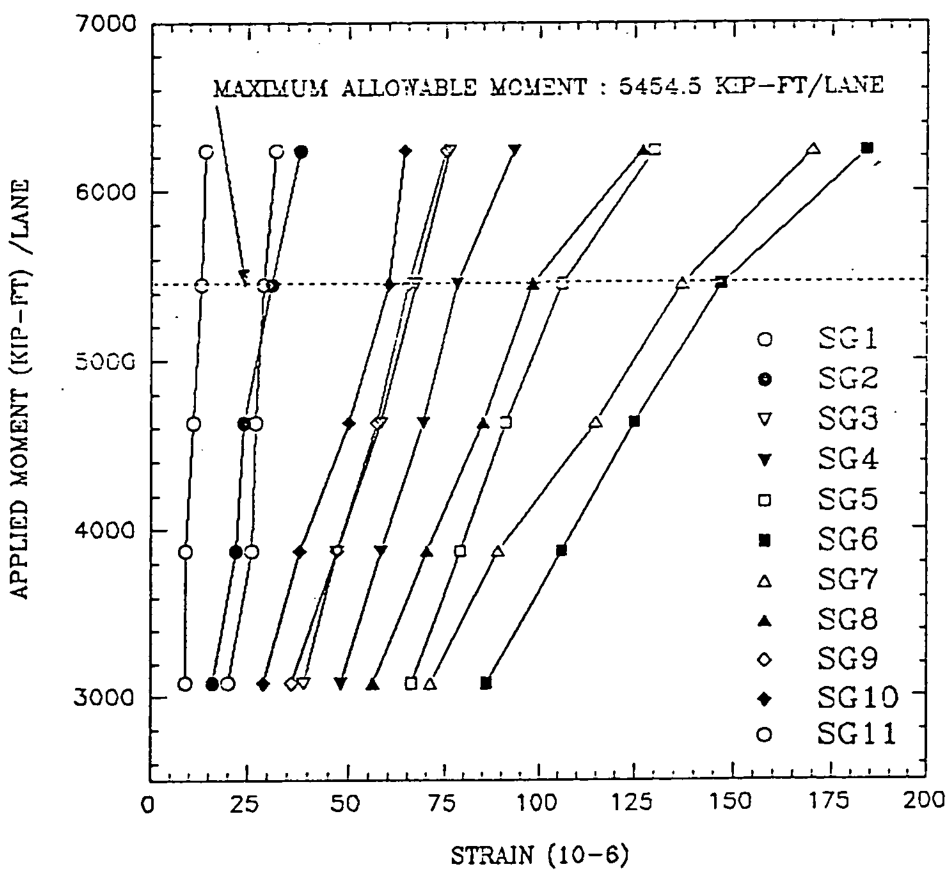



FIGURE 9-15: Turnpike Over I-595 Load Position 2—Moment vs. Strain
Span Length: 152.81 Feet

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# APPENDIX A

## REVIEW OF BRIDGE LOAD TESTS

This section summarizes the results of a review of various bridges which have been load tested. The purpose of the review was to determine how the load test results were utilized in the load rating of each bridge. Data was also obtained on the instrumentation used during the load test. All information was gathered from the specific bridge reports.

It should be noted that Burdette and Goodpasture (26) have previously studied and reported on the "Correlation of Bridge Load Capacity Estimates with Test Data". Their report identifies the effects of a number of variables on the load carrying capacity of a bridge and should be useful in explaining differences between observed and predicted behavior of bridges during load tests.

Both diagnostic and proof load tests of bridges were reviewed. For each category of test, brief descriptions of a representative sample of the bridges reviewed and a summary table are presented below.

### DIAGNOSTIC TESTING

**A. Rt 28 Bridge Over Cedar River**
**Indian Lake, Hamilton Co., NY, U.S.A. (1)**

The bridge is a simple span through truss structure with a span length of 120'-0". The bridge was built in 1930 and should be posted for 15 Tons based upon the level two H20 load rating. The controlling members were floorbeams. A diagnostic load testing was conducted by the Engineering Research and Development Bureau of the New York State Department of Transportation in 1988. Dump trucks were used to apply the load. The induced strains were monitored with a multi-channel micro-processor controlled data acquisition system. The results showed that the load testing rating was higher than the analytical rating and posting was not necessary.

**B. Rt 30 Over Sacandaga River**
**Wells, Hamilton Co., NY, U.S.A. (1)**

The bridge is a simple span through-truss structure with a span length of 100'-0". The bridge was built in 1929 and should be posted for 22 Tons based upon the level two H20 load rating. The controlling members were floorbeams. A diagnostic load testing was conducted by the Engineering Research and Development Bureau of the New York State Department of Transportation in 1988. Dump trucks were used to apply the load. The induced strains were monitored with a multi-channel micro-processor data acquisition system. The results showed that the weight limit is not necessary.

C. **Rt 13 Bridge over W. Br. Fish Creek Camden, Oneida Co., New York, U.S.A. (1)**

The bridge is a simple span through girder structure with a span length of 70'-0". The bridge was built in 1931 and should be posted for 18 Tons based upon the level two H20 load rating. The controlling members were main girders. A diagnostic load testing was conducted by the Engineering Research and Development Bureau of the New York State Department of Transportation in 1988. Dump trucks were used to apply the load. The induced strains were monitored with a multi-channel micro-processor data acquisition system. The results showed that the weight limit could be raised to 25 Tons based upon the working stress method. The weight limit could be eliminated if the load factor method was used.

D. **Rt. 30 Over Kennyetto Creek, Mayfield, Fulton Co., NY, U.S.A. (1)**

The bridge is a simple span through girder structure with a span length of 50'-0". The bridge was built in 1935 and should be posted for 17 Tons based upon the H20 load rating. The controlling members were main girders. A diagnostic test was conducted by the Engineering Research and Development Bureau of the New York State Department of Transportation in 1988. Dump trucks were used to apply the load. The induced strains were monitored with a multi-channel micro-processor data acquisition system. The results showed that the weight limit was not necessary.

E. **Rt 233 Over Deans Creek, Westmoreland, Oneida Co., NY, U.S.A. (1)**

The bridge is a simple span through girder structure with a span length of 60'-6_". The bridge was built in 1926 and should be posted for 17 Tons based upon the H20 load rating. The controlling member was the right girder. A diagnostic test was conducted by the Engineering Research and Development Bureau of the New York State Department of Transportation in 1988. Dump trucks were used to apply the load. The induced strains were monitored with a multi-channel micro-processor data acquisition system. The results showed that the weight limit was not necessary.

F. **Rt 5S Bridge Over Schoharie Creek, NY, U.S.A. (2)**

The bridge was built in 1928 and consists of three 175'-0" through truss structures. The bridge had been closed to local traffic prior to being rushed back into service as a temporary detour, as a result of the collapse of the New York State Thruway bridge over the Schoharie creek in 1987. The floorbeams were repaired before opening the detour because of the severe deterioration to the webs. An analytical load rating of the structure indicated that the repaired floorbeams were the controlling members. The HS20 ratings for the interior floorbeams were 0.78 inventory and 1.31 operating.

A controlled load test was conducted by the Engineering Research and Development Bureau of the New York State Department of Transportation in 1987. Two dump trucks were used as the applied loads. Tee-rosettes electrical resistance strain gages were bonded to critical members at points of interest. Strain gages oriented at 45 degree to the horizontal were placed on the web of the floor beam near the truss connection to measure the shearing stress on a vertical plane. The induced static strains were monitored with a multi-channel micro-processor controlled data acquisition system. The load test justified an increase of inventory rating in all bridge members. The floor beam inventory rating factor was increased from 0.78 to 1.17.

The dynamic response of the bridge under service traffic was also monitored after the detour had been opened to traffic. The data collection system for the in-service monitoring consists of eight signal conditioning units, a multi-channel analog to digital converter, and a portable computer. The dynamic response was acquired at a sampling rate of 25 Hz and the data were stored on floppy disks. The results of the service load monitoring of the structure showed that the stress ranges induced by the traffic condition were generally less than those produced in the static testing. This also suggested that the impact factors used in the analysis were higher than those experienced in-service.

Based upon the results of this testing program, the in-depth inspection, and the load rating, it was concluded that this bridge can be used as a temporary detour structure without concern for its structural integrity.

## G. Morton Creek Bridge, Kingston, Ontario, Canada (3)

The bridge is a three-span reinforced concrete deck-girder type structure with a central drop-in span. The bridge is 36.1 m long and the central suspended span is 8.2 m. The structural evaluation of the bridge indicated that only the middle section of the drop-in span has marginally lower strength (6% lower) than required by the Ontario Highway Bridge Design Code (OHBDC). All the other bridge superstructure components had sufficient strength to carry OHBDC live loads.

The bridge was load tested by The Research and Development Branch of the Ontario Ministry of Transportation and Communications in 1984. Two trucks were loaded with concrete dead weights, with a total maximum load of 1440 KN. The weight of the vehicle was gradually increased and for each load level the trucks were placed either individually or together at predetermined locations to give the critical effects. Strains and deflections were measured by using forty-four (44) uniaxial strain gages (weldable and demountable), one (1) 45 degree rosette and twenty-two (22) deflection transducers. The weldable strain gages were used around bottom chord locations where the concrete has either spalled or appeared to have a weak bound to the steel angles. The 45 degree rosette strain gage was used to monitor the shear key behavior. The measured responses were recorded through a computer based multi-channel data acquisition system. The underside of the concrete deck was also checked for any undue cracking by initially marking the cracks at selected locations and monitoring the crack growth after application of loads, particularly at higher load levels.

The tests indicated that the superstructure behaved linear elastically under the applied loads. The post-test analysis with the strain and deflection measurements of the critical members, showed that the deck elements had considerable reserve strength to carry the legal weights of Ontario without posting. It was concluded that the bridge need not be strengthened or replaced due to structural considerations.

## H. Hubby Bridge over Des Moines River, Boone Co., Iowa, U.S.A. (4)

The bridge consists of four spans of Parker-type high truss structures, each with a span length of 165'-0". The deck of the bridge was made of timber stringers, timber cross-beams and timber floor planks supported by steel floor beams. Only the tension eyebars were wrought iron. The bridge was built in 1909 and was scheduled to be removed as a result of construction of a dam and reservoir. Service load and ultimate load tests were conducted by an Iowa State University research team in 1974 to relate design and rating procedures to the field behavior of the bridge.

The deck test load was applied by four hydraulic jacks to simulate an AASHTO H15 truck. Load was applied in increments of 10 kips to start and then reduced to 5 kip increments at the high load levels. Dial gages were used to measure deflections placed across the panel midspan between panel points. Strain gages with temperature compensation for steel were attached to truss members. A total of nine deflection indicators and 108 strain gages were used.

Load rating was computed by using the load test results. The field inspection results were also sent to three agencies for independent analytical rating per AASHTO. Three independent agencies rated the bridge at H11.4, H12.7 and H11.9 compared to load test results of H66.5 at operating stress level. The result supports the fact that not only is the AASHTO code conservative, but it allows a large margin in rating results due to interpretation of field data and code specifications. However, it should be noted that concern over the presence of poor fatigue details governed the loads which this bridge could safely carry!

## I. Mead Avenue Bridge, Meadville, Pennsylvania, U.S.A. (5)

The bridge was originally constructed in 1871 and is a two-span, thru truss structure which consists of a 1937 Baltimore truss constructed outside the original 1871 wrought iron double intersection Whipple truss. Each span is 130'-4" for an overall length of 264'-0". Both trusses support floorbeams from hangers at panel points. The deck is comprised of a 5" deep open steel grid deck that is welded directly to the stringer top flanges. Many repairs have been made throughout the trusses. Design drawings and repair plans are not available for this structure. The bridge was closed to traffic because of severe deterioration to the Baltimore truss bottom chords.

A load testing was conducted by A. G. Lichtenstein and Associates, Inc. in 1990. The purpose of the test was to study the feasibility of repairing the structure for car traffic and the emergency vehicle. A dump truck loaded to 25.1 kips was used as the applied load. Forty-two (42) electrical resistance strain gages and strain indicators were used to monitor strains on both trusses in both spans. The test results were used to fine-tune the three-dimensional finite element models. The models were then studied to provide a repair scheme so that the bridge could be opened to car traffic.

The results showed that even though a number of Baltimore truss bottom chords exhibited severe section losses, the Baltimore trusses shared more than 40% of the load under the test truck. By replacing the deteriorated Baltimore truss bottom chords, the bridge can be re-opened to car traffic and the emergency vehicle. Because of the load sharing between the Whipple and

Baltimore trusses, temporary supports are not necessary during replacement of the bottom chords.

### J. Arikaree Creek Bridge, Lincoln County, Colorado, U.S.A. (6)

The bridge is a five span continuous structure consisting of eight steel girders and a concrete slab. The exterior spans are approximately 31'-0". Interior spans are approximately 39'-0". No design plans are available. The bridge carries a two lane county highway servicing primarily traffic from farm vehicles and equipment.

Current limits restrict truck weights to be approximately _ of the legal Colorado load limits. The load limit based upon the HS-20 loading is 7 Tons at operating stress level. The controlling component was the negative moment capacity of the interior girders over the interior supports.

A diagnostic test was conducted by the University of Colorado research team. Re-usable strain transducers and a digital data acquisition system were used to monitor and record the strain response. The test results were used to fine-tune the finite element model of the structure and the final rating was based on the modified model.

The results showed that the operating rating for HS-20 loading can be raised from 7 Tons to 46 Tons. The load limits based upon other legal loads were not required.

### K. Rattlesnake Creek Bridge, Clinton County, Ohio, U.S.A. (7)

The bridge is a three-span continuous reinforced concrete (RC) slab bridge, built in 1965. The bridge was selected by a University of Cincinnati research team as a test specimen to try the non-destructive test and evaluation procedure.

The dynamic characteristics of the bridge were measured by impact testing. The impact was applied by an instrumented sledge-hammer with a special rubber tip so that the force due to impact may be measured. The vertical accelerations were measured at 85 locations, and the bridge modal parameters were identified for the first eleven modes. A 3-dimensional finite element model of the bridge including the piles and soil-pile connectivity was developed and calibrated until its dynamic characteristics properly correlated with those which were measured.

Accuracy of the analytical model to correctly simulate the bridge was verified by static load testing. The static load was applied by positioning three loaded dump trucks and the vertical deflections were measured by using sixteen (16) displacement transducers. The calibrated finite element model was then used for bridge rating. The rating factor was 1.16 due to the HS-20 loading, 60% higher than that computed by the conventional AASHTO procedures.

### L. Wurtz Street Bridge, Kingston, New York, U.S.A. (30)

Built in 1920, the bridge has a 700 ft. main span crossing the Rondout Creek in Kingston, N.Y. It was the first suspension bridge built with continuous stiffening trusses. Corrosion of the eyebars in the anchorage and problems with the stiffening truss at its connection to the north tower indicated a low live-load capacity.

Diagnostic load testing measured the strains in critical stiffening truss members, determined the load distribution in the eyebars at the anchorage and gauged the bending in the towers. If measured values were less than predicted, the load-carrying capacity could be increased and anchorage rehabilitation would not be that critical.

Unfortunately, load-test data were generally consistent with theoretical predictions and did not support a higher load capacity for the bridge. But, just as importantly, the test results clearly established the need to rehabilitate the anchorages.

### M. Washington Street Bridge, Boonton, New Jersey, U.S.A. (33)

This bridge carries U.S. Route 202 over the Jersey City Reservoir in Boonton, N.J. The 25 ft. wide roadway rests on steel deck trusses with a maximum span length of 133 ft. The deck truss was built in 1895, with a midchord added in 1909. The lower chord and diagonals consist of eyebars.

Extensive repairs were made to the bridge over the years, but significant deterioration raised questions about safety and the proper bridge rating. Diagnostic load tests were conducted to determine a suitable live-load distribution factor. The tests showed the simple beam analogy for transverse distribution for wheel loads to the truss was appropriate. Also, the midchord and repairs actually had little effect on load capacity. This was established by comparing the percentage of the applied load carried by each truss, as determined by the strains in the bottom chord vs. those in the end diagonals. The load tests have cleared up questions about the bridge rating and have led to a cost-effective rehabilitation program.

### N. Calhoun Street Bridge, Trenton, New Jersey, U.S.A. (31)

Built in 1884, this bridge consists of seven identical wrought-iron through trusses, each 180 ft. long, spanning the Delaware River at Trenton, N.J. The bridge's "Phoenix sections"—built-up members consisting of four curved plates, forming a circular cross section for the upper chord, and intermediate verticals and end posts of each truss—are an unusual feature.

In 1986, an automobile crashed on span one, destroying the vertical at panel point six and damaging other members. By all rights the span should have collapsed. It didn't, and subsequent emergency reconstruction led to an interesting application of diagnostic load-test procedures. A two-part testing program was planned to assist in the unloading of the damaged truss prior to its repair, followed by normal load testing after reconstruction to ensure that both trusses were load sharing.

The tests produced important results. First, the release of loads from the south truss to a temporary supporting girder was confirmed by monitoring changes in tension member strain vs. incremental changes in jacking pressure. When a large change in strain was noted, the member was relieved of all tension. After reconstruction, load tests indicated both elastic behavior and load sharing of the north and south trusses. Measured and predicted influence ordinates were plotted for principal members to facilitate comparison. Tests showed that the measured axial strains in members of both trusses were at least 10% less than the computed values, confirming that the reconstructed trusses were indeed load sharing.

### O. Northampton Street Bridge, Easton, Pennsylvania, U.S.A. (32)

Built in 1895, this bridge connects Easton, PA and Phillipsburg, N.J. Damage from floodwater led to major reconstruction in 1957. The superstructure is a steel, pin-connected, cantilever, 550 ft. long. The upper chord members and many diagonals consist of flat eyebars.

Load-rating computations considered deterioration and damage to structural members and the bridge received a 4 ton posting . To enforce this limitation, officers manned the bridge 24 hours a day to turn away over-sized vehicles. But high manpower costs ultimately led to a diagnostic load-test program for the bridge.

The test data showed excellent correlation between measured axial strains (and corresponding forces) and computed values, thus validating the analytical model. All influence line values were in close agreement with those predicted by theory, except when the load was on the suspended span. This was explained in part by the contribution of a dummy top chord member at each end of the suspended span and the failure to adjust the bridge profile during the 1957 reconstruction. Regrettably, the load capacity of the bridge could not be increased and round-the-clock enforcement continued.

### P. Kellam-Stalker Bridge, Kellam, New York, U.S.A. (29)

The bridge, built in 1900, spans the Delaware River near Kellam, N.Y. The superstructure is a one-lane, open-deck, steel suspension bridge with a 388 ft. main span. The suspension system consists of the original cables (two 2-1/8 in. diameter cables on each side) and a 1-5/8 in. diameter cable on each side, installed in 1936. A light-weight "stiffening" through truss completes the main support system.

Load rating studies indicated that the load capacity of the bridge was limited by members of the stiffening truss. A load test program was developed to verify the response of the cables and critical truss members to a known load. The data from the test program confirmed that all cables were working together to carry the applied load, and that the axial forces (from field strain readings) measured in the top and bottom chords of the stiffening truss agreed with predicted values. But tests also showed the forces measured in the diagonals of the stiffening truss and all cables were 35% less than predicted by analysis. Once again, the load test provided a check of the model results and increased the level of confidence in the final determination of the load rating for the bridge.

### Q. Walnut Street Bridge, Chattanooga, Tennessee, U.S.A. (28)

The main spans of the Walnut Street Bridge consist of six simple truss spans varying in length from 210 ft. to 320 ft. The bridge was closed to all traffic as a result of the presence of potential fracture critical features in the bottom chord tension eyebar members.

The bridge was load tested in 1989 as part of a comprehensive inspection and testing program to determine the feasibility of rehabilitating the bridge and opening it to traffic. Re-usable strain transducers were attached to eleven critical members in spans 2 and 5 and the bridge was load tested using 14.24 ton truck provided by the City of Chattanooga. The truck travelled across the bridge at a crawl speed to eliminate impact effects and strains were read as the rear truck axle rolled over certain panel points. At least two repetitions of the truck load were made for each member tested in order to establish the repeatability of the load-strain measurements.

The measured and computed strains for each member tested were plotted, for the different truck wheel locations. The findings from the load test were:

- The measured strains in the lower chords were about 20% less than the computed strains.

- The measured strains in the end posts were almost 50% lower than the computed strains.

- The measured strains in the vertical members were about 30% higher than the computed strains.

- The measured strains in the diagonals and top chord (span 5) had values both lower and higher than the computed values, depending on the location of the truck. The average of strain readings for all truck locations indicates the measured strains to be higher than the computed strains by about 15%.

- The repair bands are working under live loads. The strains in the bands were significantly lower than strains in the eyebar pair.

The diagnostic tests, together with other material and sonic testing and a comprehensive physical inspection established the feasibility of repairing the bridge.

## R. Ohio Tests

The following tests were done in Ohio of a diagnostic nature. In almost all the cases the results were for rating for permit load purposes following the ODOT practice to allow permit vehicles up to 120 kips provided the operating rating is above 150%. The test consisted of measuring both distribution factors and impacts and using these in the rating instead of the AASHTO values. Although the tests also showed much lower stresses than computed by AASHTO methods due to greater composite action than reflected in the analysis and other unintended stiffening effects, these reduced stresses were not included in the rating.

The reason was that these additional stiffness could not be relied on at ultimate loads.

Details on the test measurements which also included weigh-in-motion studies to observe the site traffic characteristics are presented in the following reports:

"Weigh In Motion Applied to Bridge Evaluation," by Fred Moses, Michel Ghosn and John Gobieski, Final Report to Ohio DOT and FHWA, Case Western Reserve University, Sept. 1985.

"Evaluation of Steel Bridges Using In-Service Testing," by Michel Ghosn, Fred Moses, and John Gobieski, Transportation Research Record 1072, TRB, Washington, D.C. 1986.

The summary of the test results is as follows:

Description - Site 1: Heavily travelled interstate, 18% skew, six girders, designed noncomposite at 8' spacings. Span 49, 81 and 49' length.

Site 2: County bridge, posted at 19 tons (due to slab checks), three-span steel girders, lengths of 40, 50 and 40'. Five girders spaced at 5.75', noncomposite, 20 degree skew.

Site 3: Five girders, no skew, spans 56, 70 and 56', noncomposite, 7.33' spacings.

Site 4: High traffic interstate, noncomposite, 6 girders, 8' spacings, no skew, spans continuous 51, 73, and 51'.

Site 5: Composite design, 7 girders at 8.5' spacing, spans 68, 85 and 68' lengths, urban traffic.

TEST DATA

| Site | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| AASHTO Distributions | .72 | .52 | .67 | .72 | .78 |
| Measured Distributions | .66 | .64 | .56 | .54 | .56 |
| AASHTO Impact | .25 | .30 | .28 | .28 | .23 |
| Measured Impact | .10 | .12 | .10 | .10 | .18 |
| AASHTO Rating | 1.42 | 1.59 | 1.57 | 1.28 | 2.34 |
| Measured Rating | 1.75 | 1.50 | 2.06 | 1.94 | 3.46 |

Ratings shown here are operating values using 75% of capacity.

Thus, in four of the five cases, the ratings are improved by the tests using only measured girder distributions and measured impacts derived from both normal traffic and test trucks. No attempt is made here to increase ratings due to measured increases in section stiffness due to additional composite action and other unintended effects such as boundary restraints.

## TABLE A-1: Diagnostic Test Summary

| Bridge Location | Bridge Type & Year Built | Bridge Condition or Analytical Rating | Controlling Member | Testing Instru-mentation | Bridge Rating Based on Load Test | Result of Load Testing | Comments |
|---|---|---|---|---|---|---|---|
| Rt. 28 over Cedar River<br>Indian Lake, NY, U.S.A. | Simple thru truss (120'-0")<br>1930 | Posted for 15 tons | Floor beam | STA<br>LT<br>STRG | RF > 1.0<br>(H20) | Posting eliminated | |
| Rt. 30 over Sacandaga River<br>Wells, NY, U.S.A. | Simple thru truss (100'-0")<br>1929 | Posted for 22 tons | Floor beam | STA<br>LT<br>STRG | RF > 1.0 | Posting eliminated | |
| Rt. 13 over W. Br. Fish Creek<br>Camden, NY, U.S.A. | Simple thru girder (70'-0")<br>1931 | Posted for 18 tons | Girders | STA<br>LT<br>STRG | Rating=25 tons | Posting limit raised | |
| Rt. 30 over Kennyetto Creek<br>Mayfield, NY, U.S.A. | Simple thru girder (50'-0")<br>1935 | Posted for 17 tons | Girders | STA<br>LT<br>STRG | RF > 1.0 | Posting eliminated | |
| Rt. 233 over Deans Creek<br>Westmoreland, NY, U.S.A. | Simple thru girder (60'-6½")<br>1926 | Posted for 17 tons | Right girder | STA<br>LT<br>STR | RF > 1.0 | Posting eliminated | |
| Rt. 55 over Schoharie Creek<br>NY, U.S.A. | Simple thru truss (175'-0")<br>1928 | Closed to traffic<br>INV RF=0.78 (HS20) | Floor beams | STA<br>DYN<br>LT<br>STRG | INV RF-1.17<br>(HS20) | The bridge can be used as a temporary detour | |

## TABLE A-1: Diagnostic Test Summary Cont.

| Bridge Location | Bridge Type & Year Built | Bridge Condition or Analytical Rating | Controlling Member | Testing Instru-mentation | Bridge Rating Based on Load Test | Result of Load Testing | Comments |
|---|---|---|---|---|---|---|---|
| Morton Creek Bridge Kingston, Ontario, Canada | Three-span R.C. deck girder with a central drop-in span (36.1 m) | Middle section of the drop-in span has 6% lower strength than required by the codes | Drop-in span | STA<br>LT<br>STRG<br>STRT<br>DEF | The super-structure behaved linear elas-tically under the applied load. The deck ele-ments had consider-able reserve strength to carry legal load. H66 | No strength-ening or replacement of the bridge was required. | |
| Hubby Bridge over Des Moines River Iowa, U.S.A. | Simple thru truss (165'-0") 1909 | Bridge to be removed as a result of construction of a dam & reservoir. H11,H11,H12 | | HJ<br>DIAL<br>STRG<br>DEFL | | Load testing allows a large margin in rating results due to interpretation of field data and code specifica-tions. | |
| Mead Ave. Bridge Meadville, PA, U.S.A. | Dual Whipple-Baltimore thru trusses (130'-4") 1871, 1937 | Closed to traffic due to severe sec-tion losses to the Baltimore truss bottom chord | Truss bottom chords | STA<br>LT<br>STRG | | After replacement of the Baltimore truss bottom chords, the bridge can be opened to car traffic & emer-gency vehicles. | The load test was used to determine the behavior of the whole structure so that a repair scheme can be proposed for opening the bridge to car traffic. |

## TABLE A-1: Diagnostic Test Summary Cont.

| Bridge Location | Bridge Type & Year Built | Bridge Condition or Analytical Rating | Controlling Member | Testing Instru-mentation | Bridge Rating Based on Load Test | Result of Load Testing | Comments |
|---|---|---|---|---|---|---|---|
| Arikaree Creek Bridge Colorado, U.S.A. | Five-span continuous RC deck-steel girder (179'-0") | Load limit =¼ Colorado legal load OPR: 7 tons (HS-20) | Negative moment at interior supports | STA STRT | OPR: 46 tons (HS-20) | Load limits were not required | |
| Rattlesnake Creek Bridge Ohio, U.S.A. | Three-span continuous RC slab (105'-0") 1965 | INV RF=0.72 (HS-20) | Slab | DYN STA HAM ACC LT DEF | INV, RF=1.16 | | The bridge was selected as a test specimen to try the non-destructive test and evaluation procedure. |
| Wurtz Street Kingston, NY, U.S.A. | Suspension (700') 1920 | Concern over condition of Anchorage eyebars | Anchorage eyebars | STA STRG | RF < 1.0 | Anchorage rehabilitation required. | |
| Washington Street Boonton, NJ, U.S.A. | Simple Deck Truss (133') 1895 | Closed to traffic | Lower chord and verticals | STA STRG | RF < 1.0 | Confirmed need for overall rehabilitation program. | |
| Calhoun Street Trenton, NJ, U.S.A. | Seven-span simple thru truss (180') 1884 | Closed to traffic due to damage | Vertical and diagonals | STA STRG | | Assisted during reconstruction of bridge. | |
| Northampton Street Easton, PA, U.S.A. | Cantilever truss (150') 1895 | Posted for 4 tons | Eyebars | STA STRG | RF < 1.0 | Continued Posting | |
| Kellam-Stalker Kellam, NY, U.S.A. | Suspension (388') 1900 | Posted | Stiffening truss | STA STRG | RF < 1.0 | Confirmed model | |

## TABLE A-1: Diagnostic Test Summary Cont.

| Bridge Location | Bridge Type & Year Built | Bridge Condition or Analytical Rating | Controlling Member | Testing Instru-mentation | Bridge Rating Based on Load Test | Result of Load Testing | Comments |
|---|---|---|---|---|---|---|---|
| Walnut Street<br>Chattanooga, TN, U.S.A. | Six-span simple thru truss (210'-320')<br>1891 | Closed to traffic | Potential fracture critical features | STA<br>STRG | RF < 1.0 | Confirmed model. FCM not as critical as suspected. | |

Loading:
- HJ = Hydraulic Jacks
- CB = Concrete Blocks
- LT = Loaded Trucks
- HAM = Impact Hammer

Type of Testing:
- STA = Static Test
- DYN = Dynamic Test

Response Measurement:
- STRG = Strain Gages
- STRT = Strain Transducers
- DIAL = Dial Gages
- LVDT = LVDT Displacement Transducers
- ACC = Acceleration Transducers
- DEF = Deflection Transducers

## PROOF TESTING

### S. Flack Bridge Over Mitchell's Creek Wellington Co., Ontario, Canada (8)

The bridge is a single span steel pony truss structure with a span length of 21.3 m. The bridge was built in 1954. The structure evaluation showed that the floorbeams, truss bottom chords and top chords have less strength than required. A proof load test was conducted by the Research and Development Branch of the Ontario Ministry of Transportation and Communications in 1984. The test truck loads were designed to produce force effects reaching the ultimate load levels described in Ontario Highway Bridge Design Code (OHBDC) (18). Two trucks were used either individually or together for truck loads of up to approximately 780 KN each. A single truck was used for loads of up to 890 KN. The weight of the vehicle was gradually increased. The strains and deflections were measured by strain gages and deflection transducers. The data for each loading were recorded using a computer-based data acquisition system. The results showed that the floor system and the truss members behaved linear elastically under the applied truck loads and the bridge has adequate strength as a system to sustain OHBDC loads without posting .

### T. Finney Bridge, Township of Charlottenburg Ontario, Canada (9)

The bridge is a one-lane structure with pin-connected steel trusses spanning a distance of 51.21 m. The top chord is continuous, but the remaining chord members are all pin-connected. The bridge was built at the turn of the century and was posted for 9 Tons. The floor system was the governing element. A proof test was conducted by the Research and Development Branch of the Ontario Ministry of Transportation and Communications in 1984. Strains in truss members, stringers and floor beams, and deflections of one truss were monitored. Strains were measured through demountable strain transducers and an electrical resistance strain gage. A vehicle with the gross weight of 34 Tons was first brought on the bridge. The measured response of the structure indicated that a higher load level might induce a permanent deformation in some components of the structure. It was decided not to proceed with the test at higher load levels. The results required either posting the bridge with a three level posting of 9, 16, 22 Tons or, for single level posting of 9 Tons.

### U. Stephenson Townline Bridge, Township of Bracebridge Ontario, Canada (9)

The bridge is a one-lane structure with pin-connected steel trusses spanning a distance of 40.0 m. The top chord is continuous but the remaining truss members are all pin-connected. The trusses were fabricated in 1892 and relocated to their present location circa 1922. The bridge was posted for triple posting of 9, 14 and 18 Tons. The bottom chords of the truss were the governing members. A proof testing was conducted by the Research and Development Branch of Ontario Ministry of Transportation and Communications in 1984. Strains in truss members, stringers and floorbeams were measured by strain transducers and electrical resistance strain gages. A number of deflection transducers were attached to one truss to measure its vertical deflection and the horizontal movement of the supports. The results showed that the existing three level posting can be marginally improved to 9, 16 and 21 tons.

### V. Waubaushene Bridge, Ontario, Canada (10)

The bridge is a single-lane steel pony truss with concrete deck over steel stringers and floorbeams. The bridge has three simply-supported spans of 64, 74 and 74 feet. The bridge was posted for 11 tons at the time of the test.

A proof test was conducted by the Structures Research Office of the Ontario Ministry of Transportation and Communications in 1987. From the consideration of the stability of the top chords of the trusses the proof load was limited to a gross vehicle of 44 tons. Strains at critical locations were measured in the trusses of the first two spans by means of strain transducers. The deflections and the horizontal movement of the trusses at supports were measured by means of deflection transducers. The lateral movements of the top chords of the trusses were measured by a theodolite.

On the basis of the proof test, the posting load was recommended to be lowered to 9 tons. This test highlights the difficulty in ascertaining a safe limit of the posting load when the load-carrying capacity of a bridge is governed by a component that is predominantly in compression.

### W. Malone Bridge, Ontario, Canada (11)

The bridge is a reinforced concrete slab-on girder bridge with the rigid frame type of construction, and a right single span of 40 feet. The width of the bridge is 18.5 feet. The concrete is badly cracked and spalled in the deck slab and girders with the reinforcing bars exposed at several locations. The bridge was posted for 16.5 tons.

A proof test was conducted by the Structures Research Office of the Ontario Ministry of Transportation and Communications. The maximum test load applied to the bridge was the test vehicle brought on to the bridge at crawling speed with a maximum gross weight of 62 tons. Strains of the girders at critical locations were measured by strain transducers and the girder deflections by deflection transducers.

The testing was stopped at the weight level of maximum test load when it was observed that, at that level, the load-deflection curve was becoming markedly nonlinear. This test represents one of the few occasions when the posting limit was recommended to be lowered as a result of the test. The posting limit of 16.5 tons was recommended to be lowered to 11 tons.

### X. Pakowhai Bridge, New Zealand (12)

The bridge is a nine-span reinforced concrete structure built in 1939. It is composed of four reinforced concrete tee-beams with a concrete deck slab. The span lengths are 75 feet except for the two exterior slabs which are 62.5 feet.

The total length is 650 feet. There is a skew between the piers and the bridge. A proof load test was conducted by the Central Laboratories of New Zealand Ministry of Works and Development in 1982. Proof loads were applied by two vehicles which traveled the length of the bridge. Testing was to be stopped when any of the control points indicated 20 percent non-linearity or when a load 20 percent greater than design was reached. Target proof load was calculated with a safety factor of 1.2 times 85 percent of the HN truck load, times 1.3 impact factor. Displacement transducers were placed in 24 locations to monitor vehicle deflection of the four main beams and five were used to measure horizontal movements. Two dial gages were used to measure settlement of the north pier. Vibrating wire strain gages were used to monitor crack widening at 20 locations. Vibrating wire gages were recorded manually while the displacement transducers were recorded by a computer.

The result showed many cracks did not show appreciable movement until 80 percent of the maximum load. All cracks returned to original size after the target load was removed. After testing, no new cracks were found in the areas that were monitored. The bridge behaved linearly up to maximum target test load with no signs of distress. The result suggests that the bridge be open to Class I traffic with a speed restriction of 30 Km/h being applied to heavy vehicles.

## Y. Upper High Street Bridge, Blenheim, New Zealand (13)

The bridge is a five span continuous concrete bridge constructed in 1913. The bridge is 150 feet long total; each span is 30 feet. Five haunched concrete tee-beams are built integrally with the abutments at both ends without expansion joints between. The interior supports are five octagonal concrete piles with a deep diaphragm beam on top. A proof load test was conducted by the Central Laboratories of the New Zealand Ministry of Works and Development in 1984.

Proof loading was accomplished by loading the beams and piers with two trucks and testing the deck slab with hydraulic jacks. The trucks were driven onto the bridge and stopped at various locations while instrumentation was read. Coils of prestressing wire were used to add weight to both the flat-bed and dump truck. A maximum of three times the design load was loaded to the bridge. Deflection at midspan of the beams was monitored by linear potentiometers mounted under each beam. Vertical displacement of the piles and abutment walls was also monitored by linear potentiometers. A data acquisition system was used to collect and plot the data.

The results showed that the beam response was elastic with no permanent set throughout the proof tests. At maximum load no sign of cracking was evident. This allowed the load restriction to be removed for this bridge.

## Z. Sunshine State Parkway Bridge No. 37, Florida, U.S.A. (14)

The bridge on the northbound lanes of the Sunshine State Parkway is a three-span, reinforced concrete bridge constructed in 1955. The end spans have span length of 16'-6" each and the middle span consists of concrete tee-beams spanning 48'-0". An identical southbound bridge was rated to carry HS-20 loading. Both bridges were designed to carry HS-20 loading, however, as-built drawings were not available for the northbound bridge. A load test was conducted on the northbound bridge by the Florida DOT to confirm HS-20 load rating. Both bridges currently carry the normal truck traffic.

Loads were applied to the bridge using the specially built FDOT test vehicle which has a rear-mounted crane for loading concrete blocks on the trailer. A total load of 92 kips, on the two rear axles of the test vehicle, was the maximum target test load. The maximum test load was applied in three increments using two fully loaded vehicles placed side by side on the bridge. Each concrete block weighed approximately 2,100 pounds. The instrumentation consisted of eight LVDT's to measure slab and girder deflections. An automatic data acquisition system was used to record data at the end of each load increment.

The results indicated that the bridge did not show any visible cracking, and displacements were within the limits predicated by engineers. The displacement recovery, after unloading, was almost 100%. The bridge was considered safe to support HS-20 truck traffic.

**TABLE A-2: Proof Test Summary**

| Bridge Location | Bridge Type & Year Built | Bridge Condition or Analytical Rating | Controlling Member | Testing Instru-mentation | Result of Load Testing | Comments |
|---|---|---|---|---|---|---|
| Flack Bridge over Mitchell's Creek Ontario, Canada | Simple span pony truss (21.3 m) 1954 | Floor beams, truss top chords, truss bottom chords have less strength than required. | Floor beams, truss top chords & bottom chords | STA STRG DEF LT | The floor beams & truss members behaved linear elastically, the bridge has adequate strength under the legal loads. | |
| Finney Bridge Ontario, Canada | Simple span thru truss (51.21 m) ≈ 1900 | Posted for 9 tons | Floor beam | STA STRT STRG DEF LT | Post the bridge with three level posting of 9 tons. | The test truck was loaded up to 34 tons. A heavier load might induce a permanent deformation in some components. |
| Stephenson Townline Bridge Ontario, Canada | Simple-span thru truss (40.0 m) 1892 | Posted for triple posting of 9, 14 & 18 tons | Truss bottom chords | STA STRT STRG DEF CT | The existing three-level posting can be marginally improved to 9, 16 & 21 tons. | |
| Waubaushene Bridge Ontario, Canada | Simple pony trusses (64', 74', 74') | Posted for 11 tons | Truss top chords | STA STRT DEF LT | Posting load to be lowered to 9 tons. | The test truck was up to a gross weight of 44 tons. Load-carrying capacity was governed by the stability of the truss top chord. |
| Malone Bridge Ontario, Canada | Simple RC deck-girder (40'-0") | Posted for 15 tons | Deck & girder | STA STRT DEF LT | Posting limit of 16.5 tons was to be lowered to 11 tons. | The test truck was loaded up to 62 tons. Nonlinear behavior was observed at this load level. |

## TABLE A-2: Proof Test Summary Cont.

| Bridge Location | Bridge Type & Year Built | Bridge Condition or Analytical Rating | Controlling Member | Testing Instru-mentation | Result of Load Testing | Comments |
|---|---|---|---|---|---|---|
| Pakowhai Bridge<br>New Zealand | Nine-span RC T-beam RC deck<br>(650'-0") | | | STA<br>DEF<br>LT<br>DIAL<br>VW | The bridge can be opened to Class I with a speed limit of 30 km/h. | The target proof load was computed by multiplying 85% of the legal load by a safety factor of 1.2 times 1.3 impact factor. The bridge behaved linearly up to maximum target test load with no signs of distress. |
| Upper High Street Bridge<br>New Zealand | Five-span continuous RC T-beam slab.<br>(150'-0")<br>1913 | Bridge was posted. | RC beams & slab | STA<br>LT<br>HJ<br>DEF | Load restrictions to be removed. | The target proof load was computed as three times of the design load. |
| Sunshine State Parkway Bridge No. 37<br>Florida, U.S.A. | Three-span RC deck T-beam<br>(81'-0)<br>1955 | Bridge was opened to all traffic. | | STA<br>LT<br>LVDT | The bridge was considered safe to support HS-20 truck traffic. | The target proof load was 96 kips. As-built drawings are not available. |

Loading:
- HJ = Hydraulic Jacks
- CB = Concrete Blocks
- LT = Loaded Trucks
- HAM = Impact Hammer

Type of Testing:
- STA = Static Test
- DYN = Dynamic Test

Response Measurement:
- STRG = Strain Gages
- STRT = Strain Transducers
- DIAL = Dial Gages
- LVDT = LVDT Displacement Transducers
- ACC = Acceleration Transducers
- DEF = Deflection Transducers

# APPENDIX B

# PROCEDURES FOR FIELD EVALUATION OF LIVE LOAD IMPACT

## BACKGROUND

Dynamic impact values for highway bridges have been a concern for years and there have been numerous analytical and experimental studies trying to sort out the variables influencing these responses. Several bridge evaluation studies have used or recommended that measured impact in the field be used in place of code values in the bridge rating calculations.

The AASHTO impact value (50/125+span(feet) < 0.30), is generally believed to be an upper bound for most spans, although larger values than 0.30 have sometimes been reported. The AASHTO impact depends on the span and decreases with longer spans. The Ontario Code used an impact formula which depends on the bridge natural frequency, although in the most recent code edition Ontario is replacing this impact formula with a fixed percentage applicable to all bridges regardless of frequency.

In general, it is also not clear what consequence is being expressed by the impact value. Clearly, serviceability and fatigue are influenced by impact responses. Strength capacity, however, may not be influenced by impact. Since dynamic load is applied rapidly, one would expect that the upper bound yield stress for steel would come into play and more than compensate for the added stresses due to impact. Further, since impact is reduced by any damping, steel yielding will in effect cause the impact response to be self-limiting.

Impact value, however, although hard to quantify may still represent a significant portion of the safety factor, especially for short spans. The current load factor in operating ratings of 1.30, by itself, may be insufficient. That is, it is likely that an overloaded vehicle could exceed 30% above the legal rating load especially for short span bridges with large live to dead load ratios. (See NCHRP 12-33 load statistics). It is also likely that such an overloaded vehicle could occur simultaneously in both lanes. However, with the additional 30% for impact in the rating equation, the actual safety factor is 1.69 for the static load, which would seem satisfactory against an overload situation. (The 1.69 is based on 1.3 increased by 30%).

## BRIDGE IMPACT ANALYSIS

The major variables affecting impact, as observed from field studies, include the following.

1. The bump or roadway roughness is very significant. It is believed that the major excitation causing bridge dynamics is the roadway surface roughness which causes the truck oscillations and in turn excites the bridge.

2. Impact falls off dramatically with increased weight. This partly explains some of the high reported impact values obtained with relatively light vehicles. Overload of the bridge, except for very low bridge ratings, will

be associated with heavy trucks whose impact values are lower than for light vehicles.

3. Simultaneous presence of vehicles in two lanes usually governs strength capacity. The probability of both vehicles also simultaneously causing large dynamic impacts will be low and reduce the expected impact for the overload situation.

4. Truck characteristics such as vehicle suspension frequency and damping do play an important part on the impact and have been modeled. Response is magnified when vehicle frequency resonates with bridge frequency.

5. Bridge characteristics such as frequency and damping can also affect peak dynamic response.

Clearly because of the many variables present in the bridge dynamics problem it is expected that the AASHTO code value should be an envelope to cover worst case scenarios. Some recognition of this is provided in the recent AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges. This guideline allows for reduction in the impact when computing the rating factor for bridges with smooth approaches and surface. The text further allows for the use of measured impacts which can be developed by the methods described in the following paragraphs.

## MEASURED IMPACTS

Measured impacts can be estimated from either continuous displacement or strain records. A typical strain record due to a moving truck appears as in Figure B.1. The response builds up with an oscillating portion superimposed over an apparently static response and then declines to zero except for residual vibrations which eventually disappear. Several researchers have defined impact percentage by taking the oscillating portion (one half the peak-to-peak value) near the peak response and dividing by the peak static response. Although this calculation sounds simple in principle, it is difficult to apply in practice. In particular for short span bridges, the *static* portion of the bridge response oscillates due to the axle spacings. Hence, if one tries to estimate the dynamic oscillations directly from the records these impact values could easily be thrown off by the static oscillations, especially if the analysis is done by computer processing. The latter may be necessary to gain enough statistics of impacts to cover all the variables (truck weight, suspensions, etc.) cited above. It is recommended that the impact value to be used for the rating equation should be the average impact plus one standard deviation.

The following procedures are recommended for accurately assessing the impact values.

1. Compare maximum dynamic response, such as stress or displacement, obtained from a moving vehicle with a static or slow speed crawl run. It is important that the path of the vehicle coincide in both the moving and crawl passages. Otherwise, small differences in a vehicle's transverse position could be a factor in giving the wrong impact value.

2. Approximate the dynamic amplification by examining the residual vibrations after the truck has left the bridge. Although this may not be a

precise indication of dynamic amplification at the peak stress level, especially for longer spans, the results may still be reasonably accurate.

3. Assess the impact by comparing the measured peak response with a static computation of the maximum response. The static response requires a knowledge of the vehicle axle weights, speed and bridge influence line. Ghosn has shown in a recent paper how the established bridge weigh-in-motion techniques can be extended to carry out such analyses for the impact values. The results appear promising for providing consistent measures of the impact value. The advantage is that it can be easily carried out with a large population of a random truck spectra and impact results can be correlated to vehicle type, weight and speed.

In all methods of experimentally finding the impact values, the analyst should examine a variety of vehicle types, weights, speeds and vehicle positions in estimating the appropriate impact factor. Repetitive trials should also be considered with a selection based on a mean plus one sigma as suggested above.

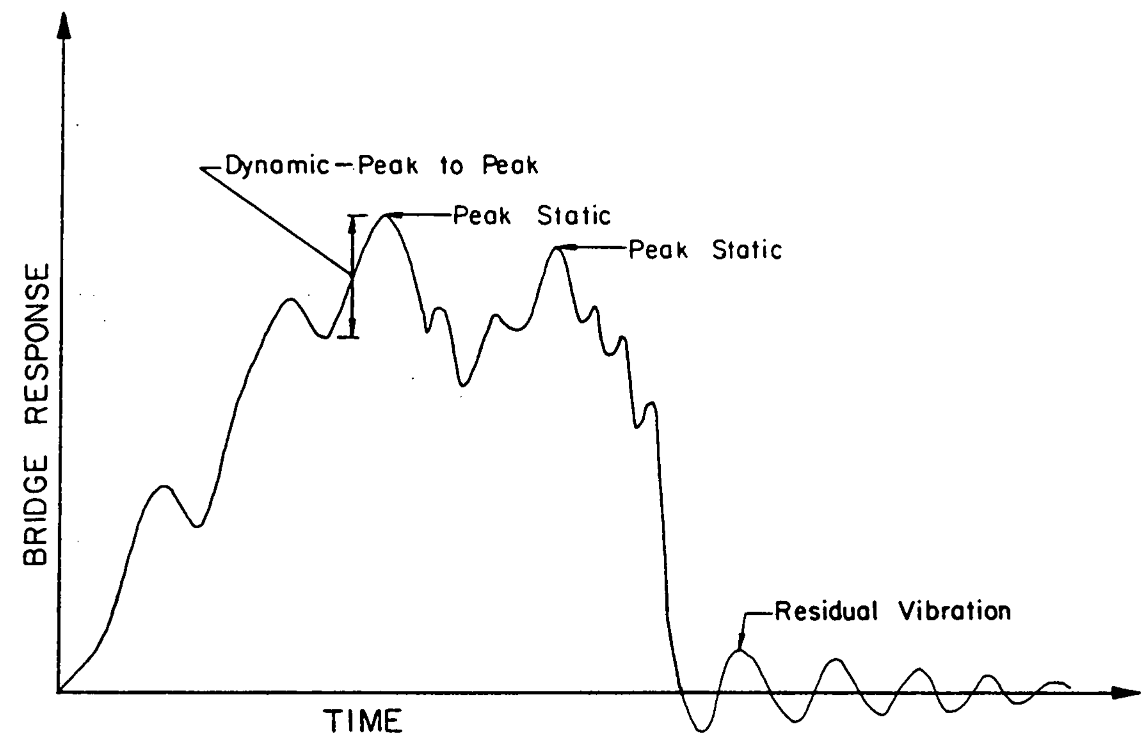


FIGURE B.1: Typical Strain Record

# APPENDIX C

# FATIGUE LIFE TESTING FOR STEEL BRIDGES

## SUMMARY

This section is based on recent research on fatigue life evaluation procedures and is presented in detail in NCHRP Report 299. This report gives information and references on the fatigue behavior of both uncracked steel members subject to primary stresses and also the possibility of fatigue due to secondary stresses that are not normally calculated. These procedures have also been summarized in the AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges (1990). The general procedures therein do not apply, however, to members that have sustained severe corrosion or mechanical damage or that have been repaired after sustaining fatigue cracking. Research at Lehigh University and elsewhere should be consulted for these latter problems.

## METHODOLOGY

Attention should be given in bridge evaluation to the possibility of fatigue occurring in steel members and attachments. A number of events in which serious fatigue cracking and even bridge failure has been observed especially on heavily traveled routes. The reason for this situation is that many bridges, especially those built just after welding became common practice, were in fact never reviewed in their original design for the possibility of fatigue occurring. AASHTO adopted its first fatigue provisions in the 1960s. Thus, details which today are being checked for adequate fatigue life under long duration loadings were never considered during their original design. This is the case in many of the bridges which are 25 to 50 years of age. In addition, both the numbers and magnitudes of truck loads have increased in recent years which may further lead to shortening of fatigue lives.

Furthermore, in the last 20 years, there has been increased understanding of the factors influencing fatigue including loading, materials and fabrication details. In particular, it has been observed that fatigue most commonly occurs in attachment details which have inherently high stress concentrations induced by loading behavior which in fact is not checked in usual design analysis. Such distortion induced loadings are very difficult to model and often must be checked in the field by direct measurements under traffic.

Consequently, field tests have become common in recent years both to ascertain existing primary stress ranges as well as secondary or distortion induced stresses. Testing equipment requires strain gages, signal conditioning and recording and processing as discussed in Chapter 5. The requirements are usually that strain data be taken under ordinary moving traffic rather than under statically applied loads, although data from static loads may help to further understand the mechanism of loading especially for distortion or secondary stresses.

## FATIGUE LIFE ANALYSIS

Present AASHTO bridge design procedures do not reflect actual fatigue conditions in bridges; instead they combine an artificially high stress range with an artificially low number of stress cycles. Furthermore, the AASHTO design rules do not easily permit the calculation of remaining bridge life which may be necessary in bridge management decisions regarding posting , permit loading, repair of strengthening, and future replacement. It is especially important that the recent remaining life procedures outlined in NCHRP Report 299 or the AASHTO Guide Specification for Fatigue Evaluation of Existing Steel Bridges (1990) be used in its entirety to provide consistent estimates. Frequently, it has been reported that measured stresses are much lower than those computed in the AASHTO design specifications. These lower observed stresses do not necessarily imply adequate life since the number of cycles as well as the safety margins needed for either redundant or nonredundant spans must be included in the analyses.

The AASHTO Guide Specification does refer to the use of field measurements which can then be used to assess remaining safe life of the attachment. This is done while the bridge is under normal traffic. Stress-range histograms are taken for critical details. (The stress range is the difference between maximum stress (tensile) and the minimum stress recorded while a vehicle crosses the structure.) It is important to also have the vehicle count, normally the truck traffic, excluding panel, pickup and other 2-axle/4-wheel trucks. These counts permit the stress ranges to be related directly to truck traffic and extrapolated to other periods of the bridge life. The equivalent number of simple cycles for a given stress-plot of a truck passage may be calculated by procedures given in NCHRP Report 299. This provides the parameter, C, the cycles per truck passage discussed below.

The effective stress range for each histogram ($S_r$) may be found from the equation:

$$S_r = \left(S\, f_i\, S_{ri}^{3}\right)^{1/3}$$

where:

$f_i$ = fraction of stress ranges within an interval

$S_{ri}$ = midwidth stress range for the interval

The remaining safe life of the details can be calculated from procedures, including safety factors given in the AASHTO Guide Specification. These depend on $S_r$, the estimated volume of trucks, and the stress cycles per truck passage. If records of truck volume are available over the bridge history, these may be factored into the equation for remaining life. Projected growth rates in truck traffic may also be used.

## SECONDARY STRESSES

The procedures described are directly applicable to measurements taken on primary stress details such as coverplates or stiffeners. The strains should be nominal values taken several inches away from any possible stress riser due to the weld or other attachment. In the case of secondary or displacement induced stresses, maximum stresses may occur in gaps in the neighborhood of the attachment such as at the end of cut-off attachment plates. In such cases, there may be high-strain gradients that have to be extrapolated to identify the maximum stress ranges. In contrast to measurements on primary members, where maximum stress ranges rarely exceed 5 ksi, the peak stresses in the neighborhood of attachment plates and stiffener gaps could exceed 20 ksi.

Careful efforts may be needed in such measurements near high-stress gradients. Because of the large gradient, the gage length selected should be small. Consideration should be given when needed to using strip gages which are a series of gages in a line making simultaneous measurements. These gages have small lengths and may allow extrapolation to the root of the stress concentration. Further review of distortion induced stress studies such as at Lehigh University should be consulted for interpreting these stresses along with the procedures used in the AASHTO Guide Specification.

# BRIDGE RATING THROUGH NONDESTRUCTIVE LOAD TESTING

## NCHRP 12-28(13)A

## TECHNICAL REPORT

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June, 1993

# CHAPTER 1

# INTRODUCTION

It is well known that a large percentage of our nation's bridges are structurally deficient and in need of posting and/or rehabilitation. Also, a significant number of bridges have hidden structural components and cannot be load rated analytically with any degree of confidence.

The results of bridge load tests have generally shown that many structures have greater load-carrying capacity than that predicted by calculations. Aside from the conservative approach used in design, the actual response of the structure under live load may be different due to the magnitude and distribution of loads, the interaction of structural (and to a lesser degree, non-structural) components, and the impact of deterioration and repairs.

The potential reduction in the number of bridges considered to be structurally deficient through the use of load testing was recognized in 1987 with the initiation of NCHRP Project 12-28(13), "Nondestructive Load Testing for Bridge Evaluation and Rating" (8). This project was completed in 1990. However, additional research was needed to develop a detailed procedure for integrating the results of load tests with rating calculations to establish the safe load capacity of bridges.

The primary goal of NCHRP Project 12-28(13)A, Bridge Rating Through Nondestructive Load Testing, was to develop a manual of procedures and techniques for incorporating bridge load test results into the bridge load rating process. Other objectives of this research included the development, presentation and refinement of a two-day workshop on bridge rating through load testing. To accomplish these objectives, a working plan consisting of eight tasks was developed and approved by TRB.

The major product of this project was the development of a "Manual for Bridge Rating Through Load Testing" which provides guidelines for integrating the load testing of bridges with their load rating.

The Manual includes recommendations based on the experience of the project team members in the load testing of bridges, published data on load tests and instrumentation and technical research conducted by project team members.

This report presents detailed data on two major technical areas: evaluating unintended composite action and establishing target proof load levels. The information contained in the next two chapters of this report was used as the basis for the guidelines and recommendations in the Manual. Each of these two chapters is independent of the other and stands alone, complete with equations and figures.

The material presented by Baidar Bakht in Chapter 2 is based on his own research, including field load tests, and has not been presented in its current form in any technical publication or conference. There are, however, other papers on unintended composite action which have been available to bridge engineers for review and comment (e.g. Ref. 7). The recommendations made by Bakht in Chapter 2 appear conservative with respect to findings reported by others (e.g. Ref. 7). The bridge owner should decide based on his own judgment and experience the applicability of the material presented in Chapter 2.

# CHAPTER 2

# EVALUATING COMPOSITE ACTION IN SLAB-ON-GIRDER BRIDGES WITHOUT MECHANICAL SHEAR CONNECTION BY BAIDAR BAKHT

## 2.1 INTRODUCTION

There are a great many of slab-on-girder bridges with concrete deck slabs and steel girders, or stringers, in North America which do not have any mechanical shear connection between the deck slab and the beams. It has been found through many field tests that despite the absence of mechanical shear connection, some composite action exists. between the deck slab and the beams in most of these bridges. The composite action, which is believed to exist because of friction and bond between the steel and the concrete, however, is known to deteriorate with increase in load level. Bakht and Jaeger (2) have recommended that such composite action should be ignored completely in the strength calculation of the ultimate limit state.

It is emphasized that this recommendation of Bakht and Jaeger applies for only analytical evaluations. If full or partial composite action is confirmed by a proof test, then, of course, the composite action should be implicitly included in the evaluation of bridge strength. There is uncertainty, however, about the reliability of composite action found by a diagnostic test. The question is, can this composite action be assumed to exist at load levels higher than those of the diagnostic test?

The purpose of this chapter is to explore systematically the composite action in a slab and girder combination without mechanical shear connection, and to determine a procedure for extrapolating the results of diagnostic tests to give a bridge rating.

## 2.2 DETERIORATION OF COMPOSITE ACTION (WHEN NONE WAS INTENDED BY DESIGN)

Bakht and Jaeger (2) have provided evidence of the deterioration of the composite action with increasing load in structures which were designed as noncomposite. An example of the shifting of the neutral axis, indicating the changing degree of composite action, is presented in Fig. 1, which shows the strains at the top and bottom flanges of a stringer, plotted against the load level. The stringer belongs to the floor system of a truss bridge in which the floor beams are spaced at 14 ft centers, and in which there is no mechanical connection between the concrete deck slab and the stringers. The data for Fig. 1 have been taken from a report by Mahue and Agarwal (4).

In a slab-on-girder bridge, the neutral axis of the partially composite beams usually maintain their positions during the early stages of increasing loads. The neutral axis tends to move down at higher load levels, thus indicating the deterioration of the composite action. For the case shown in Fig. 1 the loss of the composite action was almost directly proportional to the load level.

It is important to note that the load-strain diagram shown in Fig. 1 was repeatable, i.e. unloading the bridge and then reloading resulted in similar strain patterns. This indicates that transfer of the horizontal shear from the stringer to the deck slab was an elastic phenomenon.

## 2.3 ELEMENTARY ANALYSIS

To study the mechanics of composite action, a simply supported beam carrying a uniformly distributed load with unit length is considered. The beam has a rectangular flange and a rectangular web both of concrete as shown in Fig. 2. To simplify calculations it is further assumed that the neutral axis of the composite beam lies in the web. This can be verified by using equation 9 below.

By considering a longitudinal segment of the beam of length dx (Fig. 2b), the shear force Q is obtained, as usual in terms of moment M.

$$Q = \frac{dM}{dx} \tag{1}$$

Similarly:

$$w = -\frac{dQ}{dx} = -\frac{d^2M}{dx^2} \tag{2}$$

## 2.4 INTERFACE VERTICAL SHEAR STRESS

Using the notation shown in Fig. 2(a) and denoting I as the moment of inertia of the composite beam, the vertical shear stress $\tau$ in the web at a distance z from the bottom (Fig. 3(a)) is given by:

$$\tau = \frac{Q(b_2 Z)(d_1 + d_2 - \bar{y} - 0.5Z)}{b_2 I} \tag{3}$$

The total shear force F taken by the web is obtained from

$$F = \frac{Q\, b_2}{I} \int_0^{d_2} z(d_1 + d_2 - \bar{y} - 0.5z)\,dz$$

or

$$F = \frac{Qb_2}{I}\left\{(d_1 + d_2 - \bar{y})\frac{d_2^2}{2} - \frac{d_2^3}{6}\right\} \tag{4}$$

The vertical interaction force per unit length between the flange and the web is denoted as $p_v$, as illustrated in Fig. 3(b). It is obvious that

$$p_v = -\frac{dF}{dx}$$

or using Eq. (4)

$$p_v = -\frac{b_2}{I}\left\{(d_1 + d_2 - y)\frac{d_2^2}{2} - \frac{d_2^3}{6}\right\}\frac{dQ}{dx}$$

Using Eq. (2), the above equation becomes

$$p_v = \frac{wb_2}{I}\left\{\left(d_1 + d_2 - \bar{y}\right)\frac{d_2^2}{2} - \frac{d_2^3}{6}\right\} \quad (5)$$

## 2.5 INTERFACE HORIZONTAL SHEAR STRESS

The horizontal interactive shear force per unit length between the flange and the web is denoted as $p_h$. Using the familiar elementary theory, it can be shown that for any loading

$$p_h = \frac{Qb_1d_1\left(\bar{y} - 0.5\,d_1\right)}{I} \quad (6)$$

## 2.6 COMPOSITE ACTION THROUGH ONLY FRICTION

If the composite action takes place only through friction between the web and the flange, then it is obvious that

$$p_h \leq \mu\, p_v \quad (7)$$

Where $\mu$ is the coefficient of Coulomb friction. Using Eqs. (5) and (6), relationship (7) can be written as:

$$Q\,b_1d_1\left(\bar{y} - 0.5d_1\right) \leq \mu\, w\, b_2\left\{\left(d_1 + d_2 - \bar{y}\right)\frac{d_2^2}{2} - \frac{d_2^3}{6}\right\} \quad (8)$$

It is recalled that $\bar{y}$ is given by:

$$\bar{y} = \frac{b_1 d_1^2 + b_2 d_2^2 + 2 b_2 d_1 d_2}{2\left(b_1 d_1 + b_2 d_2\right)} \tag{9}$$

For a simply supported beam of span L and carrying a uniformly distributed load, w, per unit length, Q = 0.5 wL. By substituting this expression for Q in inequality (8), it can be appreciated the w occurring on both sides is self canceling. Consequently, for a given cross section, L and m are the only variables which determine whether the full composite action can be developed by friction alone. The limiting value of L obtained from inequality (8) is given as follows:

$$L \le \frac{2\mu b_2 \left\{\left(d_1 + d_2 - \bar{y}\right)\frac{d_2^2}{2} - \frac{d_2^3}{6}\right\}}{b_1 d_1\left(\bar{y} - 0.5 d_1\right)} \tag{10A}$$

Figure 4 shows the cross section of an equivalent beam in which for ease of calculation, both the flange and the web are assumed to be of the same material. For this cross section using $\mu = 1.0$, the limiting value of L is found to be 49.1 inches. Clearly, this very small limiting span length indicates that friction alone is not very effective in generating the composite action.

## 2.7 EXAMPLE

To explore qualitatively the effect of friction on the composite action, an example is presented. The example is that of the most heavily loaded girder of the slab-on-girder bridge tested to failure by Bakht and Jaeger (2). The cross section of the girder and the associated portion of the deck slab is shown in Fig. 5(a). For a modular ratio of 10, the effective moment of inertia of the fully composite section is found to be 5336 $in^4$ in steel units. The effective applied loading on one girder is shown in Fig. 5(b). As shown by Jaeger and Bakht (3), this partial uniformly distributed load can be represented approximately by a sinusoidally distributed load of intensity $p_x$ which is given by:

$$p_x = \frac{2P}{\pi u} \sin\frac{\pi u}{L} \sin\frac{\pi x}{L} \tag{10B}$$

Where u is half the length of the centrally-placed load P, x is the distance along the beam from the left support, and L is the span. It can be shown that for the loading shown in Fig. 5(b), $p_x$ is given by:

$$p_x = 0.05625 \sin\frac{\pi x}{540} \tag{11}$$

In this expression, x is in inches and $p_x$ in kips/in. (See Fig. 5(c)).

To obtain the maximum benefit of friction, it is assumed that (a) $\mu = 1.0$; (b) all the applied loading is transferred through the deck slab and girder interface as $p_v$; and (c) the dead load of the deck slab, etc. is of the same order of magnitude as the live load. In this case, the friction force $p_u = 2 \times \mu \times p_x$ or:

$$p_u = 0.1125 \sin \frac{\pi x}{540} \text{ (kips/in)} \tag{12}$$

It can be shown that shear $Q_x$ along the span is given by:

$$Q_x = \frac{2PL}{\pi^2 u} \sin \frac{\pi u}{L} \cos \frac{\pi x}{L} \tag{13}$$

Replacing Q in Eq. (6) by $Q_x$, the expression for $p_h$ becomes:

$$p_h = 3.529 \cos \frac{\pi x}{540} \text{ (kips/in)} \tag{14}$$

The quantitative comparison of $p_u$ and $p_h$ thus obtained is presented in Fig. 6 for the entire length of the beam. It can be seen in this figure that (a) the resistance that can be generated by friction is very small compared to the horizontal interface shear; and (b) the patterns of the interface horizontal shear and the corresponding frictional resistance are not compatible to each other, so that, for example at the supports the former attains the highest value and the latter the lowest. It is also worth noting that in practice, $p_u$ is likely to be smaller than the values given by Eq. (12), in which case the contribution of friction to the composite action would be even smaller.

## 2.8 CONCLUSION WITH RESPECT TO FRICTION

From the above discussion, it is obvious that friction is not significantly effective in generating the composite action between a beam and the deck slab. Any composite action observed in the absence of mechanical shear connection should be attributed to factors other than friction. If friction had a significant influence, the composite action would have been observed in all girders, and this is clearly not the case.

## 2.9 COMPOSITE ACTION THROUGH ONLY BOND

The term "bond" is used herein for the chemical bond between the concrete of the deck slab and the flange of the steel girder; it is also used for the resistance that may be generated due to aggregate interlocking between the concrete of the deck slab and a delaminated strip of concrete that has detached from the deck slab but is still bonded to the flange of the girder. Unlike resistance due to friction, both these kinds of resistance are believed to be relatively free from the normal pressure at the interface.

Unfortunately, except for a field test, there is no practical way of ascertaining if bond exists between the deck slab and girders. It has been observed that when the top flange of girder is partially embedded in the deck slab, the bond resistance is very effective in promoting the composite action. However, even this generally true statement is not free from exceptions. In the bridge tested by Bakht and Jaeger (2) to failure, it was observed that despite their top flanges being partially embedded in the deck slab, the two outer girders had practically no composite action even at low levels of loads.

If in the absence of mechanical shear connection, the presence of the composite action is confirmed by a proof test, then clearly it can be relied upon for the nominal ultimate evaluation load. Such reliance on the composite action, however, may not be axiomatic if its presence is established only at low level loads of a diagnostic test.

To explore the degree of composite action beyond the level of the test load, the realistic case of the composite beam shown in Fig. 5(a) is considered. The moment of inertia of the non-composite slab and beam combination is 2169 $in^4$ (steel units) and that of the fully composite beam 5336 $in^4$ (also in steel units). The beam is subjected to gradually increasing load.

It is assumed that during the initial stages of loading the bond between the concrete deck slab and the steel beam remains intact thereby offering full composite action. The load-deflection curve for the fully composite beam is shown schematically in Fig. 7 by line 0A. If the bond between the concrete and steel breaks completely at load level A, the deflection of the beam will suddenly increase by a factor of 5336/2169 (=2.46), in which case the deflection will increase from A to B. For higher loads, the load deflection curve would follow BC. If the breakage of the bond is permanent, the load-deflection curve for subsequent loading will be similar to OBC.

In practice, the deflections of slab-on-girder bridges without mechanical shear connection do not suddenly increase under gradually increasing loads. In most cases, the load-deflection curves of these bridges are linear in the initial stages of the load, thus following the path OA in Fig. 7. The curves become nonlinear similar to path AD under heavier loads. Unless the steel of the girder has yielded, upon removal of the load all the deflections are recovered with the load-deflection curve following path DEO.

The typical observed load-deflection curve presented in Fig. 7 confirms that

a. the bond strength, while deteriorating with increasing load, does not suddenly drop to zero;

b. the deterioration of the bond strength under high load levels is not permanent, i.e. the bond strength can be relied upon even if the limit of linearity is exceeded.

In the light of the above discussion, it seems feasible to divide the load-deflection behavior of the beam and slab combination into two linear segments, OA and AF shown in Fig. 7. In segment OA, the deck slab acts compositely with the steel beam. The upper limit of this segment is defined by the load which causes the interface horizontal shear to reach the limiting, and pre-specified, bond stress.

Segment AF represents the behavior when the slab and the beam flex about their own respective neutral axes, i.e. when the section acts non-compositely.

Figures 8(a) and (b) show a schematic representation of the process of extrapolating the level of proof load from the results of diagnostic testing. The former figure shows the case in which the load of the diagnostic test causes smaller interface horizontal shear stress than the limiting bond stress. In such a case, the diagnostic test is useful in only establishing the presence of bond between the deck slab and the girders. The finding of the proof load by extrapolation is done by using the bilinear load deflection behavior described above.

When the diagnostic test loads are high enough to cause higher interface horizontal shear than the limiting bond stress, then as shown in Fig. 8(b), the test load should be regarded as the limiting load beyond which the section ceases to act compositely.

## 2.10 SUGGESTED BOND STRENGTHS

Agarwal and Selvadurai (1) have suggested that a conservative value of bond strength between the concrete deck slab and steel girders can be assumed to be $0.1\sqrt{f'_c}$, where the compressive strength of concrete, $f'_c$, is in MPa. For 3000 psi concrete, this bond strength is about 70 psi.

It is suggested that in the absence of more reliable information, this bond strength be used for those bridges in which the deck slab rests above the girder flanges. Where the top flanges of the girders are partially, or fully, embedded in the deck slab, the bond strength is expected to be much higher; 100 psi is recommended.

It is emphasized that these values of bond stresses should be used only after the girder strains obtained during the diagnostic test have confirmed that the neutral axis of the section is high enough to justify the assumption of composite action.

The values of bond strengths recommended above are on the conservative side. Bond strengths of up to 145 psi have been observed (Agarwal and Selvadurai (1)).

## 2.11 ILLUSTRATIVE EXAMPLE

To illustrate the proposed technique, a slightly modified form of the bridge tested to failure by Bakht and Jaeger (2) is selected as an example with one layer of blocks being regarded as the diagnostic loading. The cross section of the girder and the associated portion of the deck slab receiving the maximum share of the test load are shown in Fig. 5(a). The following are assumed:

a. The girder attracts about one third of the total test load of 48 kips; i.e. 16 kips, is shown in Fig. 5(b).

b. The yield stress of the girder steel is 30 ksi and the maximum dead load stress in the girder is 7.2 ksi leaving 22.8 ksi stress or $760 \times 10^{-6}$ in/in strain available for the test load ($E = 30 \times 10^6$ psi).

c. The extrapolated proof test load, will be the calculated load which causes a maximum strain of $760 \times 10^{-6}$ in/in in the girder under consideration.

d. The bearing restraint offers negligible resistance to the movement of the girders.

e. Under diagnostic loading, the girder at the mid-span was found to have -21 and 200 x $10^{-6}$ in/in strains in the top and bottom flanges, respectively.

If the composite action between the deck slab and the girder is ensured by adequate mechanical shear connectors, the extrapolated proof load would simply be (760/200) x 48 = 182 kips. In the absence of such shear connectors, the steps in Sections 2.12 through 2.15 would have to be taken to obtain the extrapolated proof load.

## 2.12 CALCULATION OF INTERFACE HORIZONTAL SHEAR

The maximum shear due to the diagnostic load is 8 kips. Therefore, the maximum interface horizontal shear, $p_h$ is given by Eqn. 6:

$$p_h = \frac{(8000)48(7)\left(9.3 - 0.5(7)\right)}{(10)(5336)}$$

$$p_h = 292 \text{ psi}$$

Since the width of the girder flange is 9 in., the interface horizontal shear stress = 292/9 = 32 psi.

## 2.13 CALCULATION OF LOAD CAUSING LIMITING INTERFACE SHEAR

The permissible bond strength for the embedded flange, as given earlier, is 100 psi. Consequently, the load causing the limiting interface horizontal shear stress = (100/32) x 48 = 150 kips. The maximum tensile strain caused by this load is (150/48) x 200 = 625 x $10^{-6}$ in/in, so that strain of (760-625) = 135 x $10^{-6}$ in/in is available for further loading under which the girder will be assumed to be noncomposite.

## 2.14 CALCULATION OF REMAINING LOAD CAPACITY

As discussed earlier, after the limiting bond stress has been exceeded, the girder will be assumed to be acting noncompositely. For simplicity, it is further assumed that the girder will sustain all the load.

The moment of inertia of the 24-in-deep steel girder is 2032 $in^4$. The maximum moment due to the 16 kip load on the beam (Fig. 5b), or the total test load of 48 kips, is 1776 kip-in. The maximum stress caused by this moment in the naked steel girder = 1776 x 12/2032 = 10.5 ksi. This stress is equivalent to 350 x $10^{-6}$ in/in strain. The total load causing the maximum strain of 135 x $10^{-6}$ in/in = (135/350) x 48 = 18.5 kips.

## 2.15 EXTRAPOLATED PROOF LOAD

From the above calculations, the total extrapolated proof load = 150+18.5 = 168.5 kips, which is only about 7% less than the proof load which is obtained by assuming that the degree of composite action found in the diagnostic test will hold at the level of the proof load.

If the bond strength was assumed to be effective only up to the level of the diagnostic test, the proof load would have dropped to about 142 kips based on noncomposite action for loading beyond the level of the test loads.

## 2.16 CONCLUSIONS

It has been demonstrated that any composite action that might exist in slab-on-girder bridges without mechanical shear connections, is predominantly due to bond between the deck slab and the girders. An analytical method has been provided for obtaining the value of the proof loads from the results of a diagnostic test on a slab-on-girder bridge.

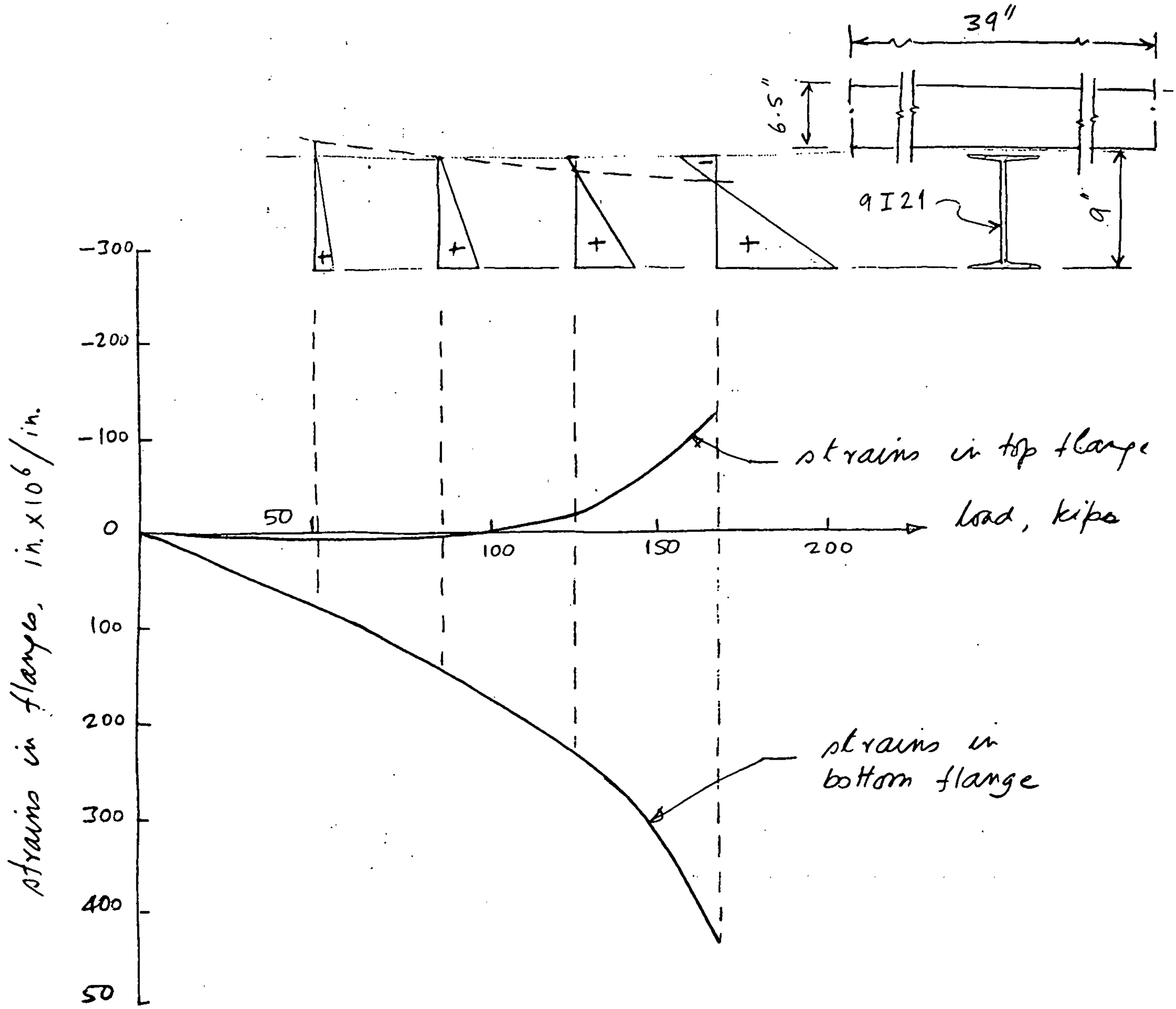
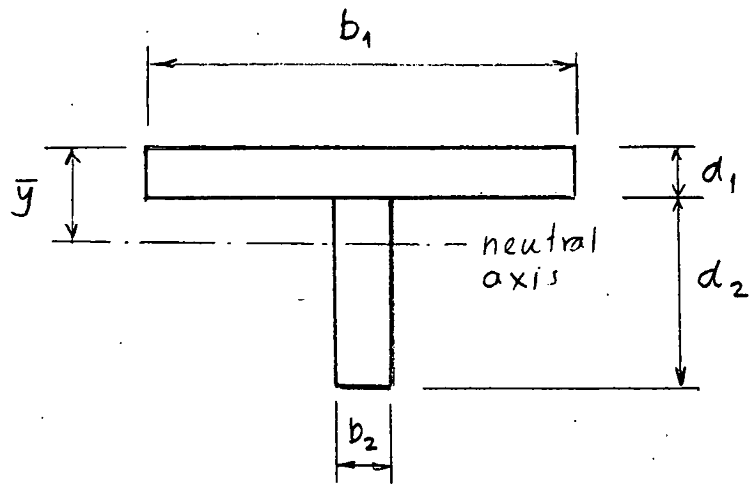
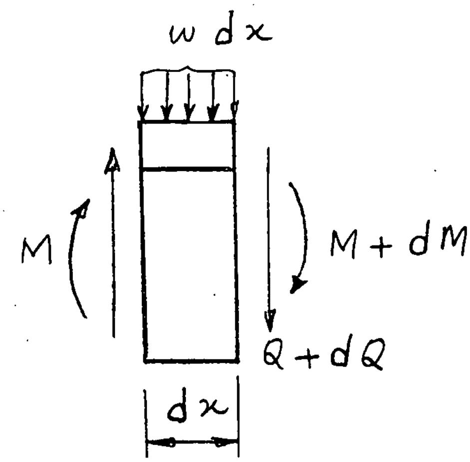



FIGURE 1: Observed strains in stringer of truss bridge

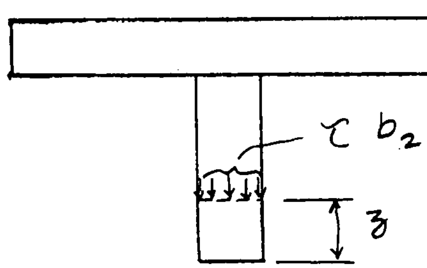



(a) Cross-section

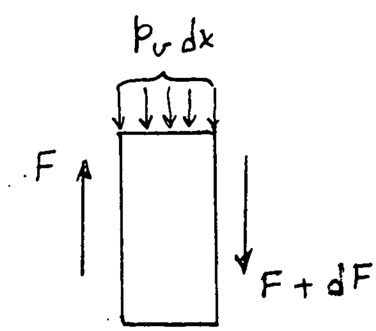



(b) Longitudinal Segment

FIGURE 2: Concrete Composite Beam




(a) Cross-Section




(b) Longitudinal Segment of Girder Only

FIGURE 3: Shear Stress

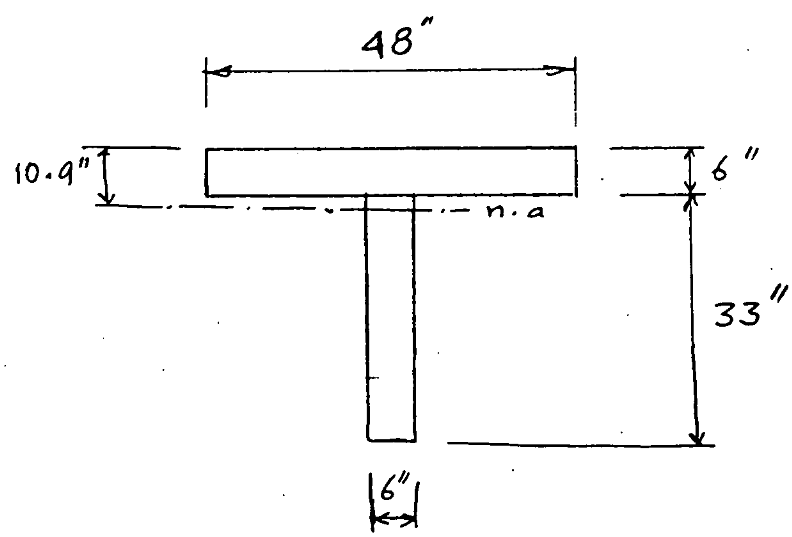
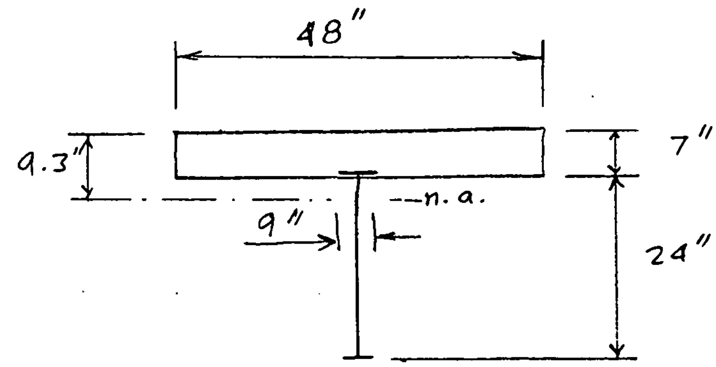
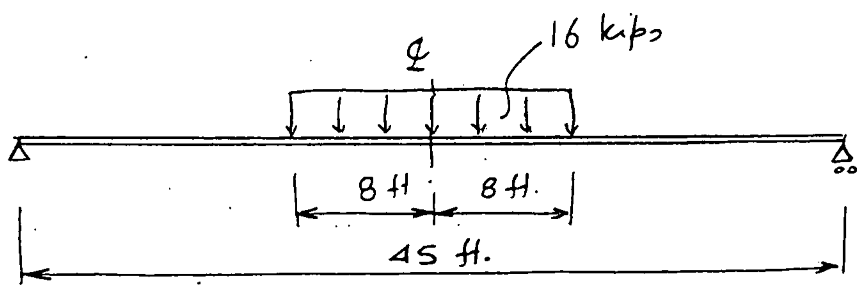


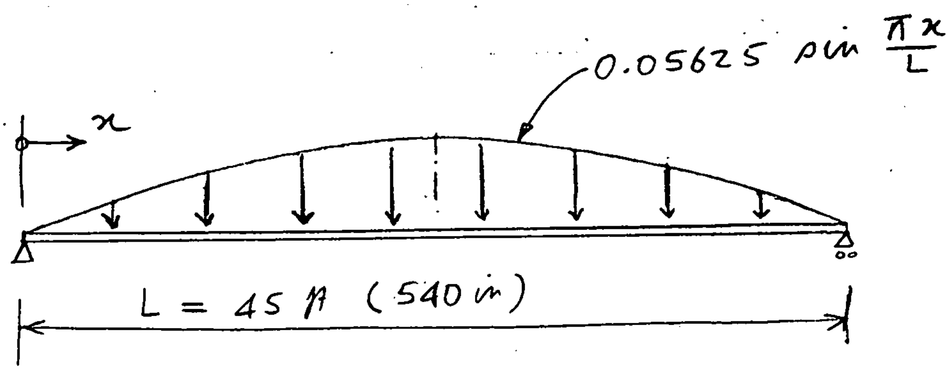

FIGURE 4: Cross Section of Concrete Composite Beam




(a) Cross Section of Steel-Concrete Composite Beam




(b) Loading on Beam




(c) Equivalent Loading

FIGURE 5: Composite Beam and Loading

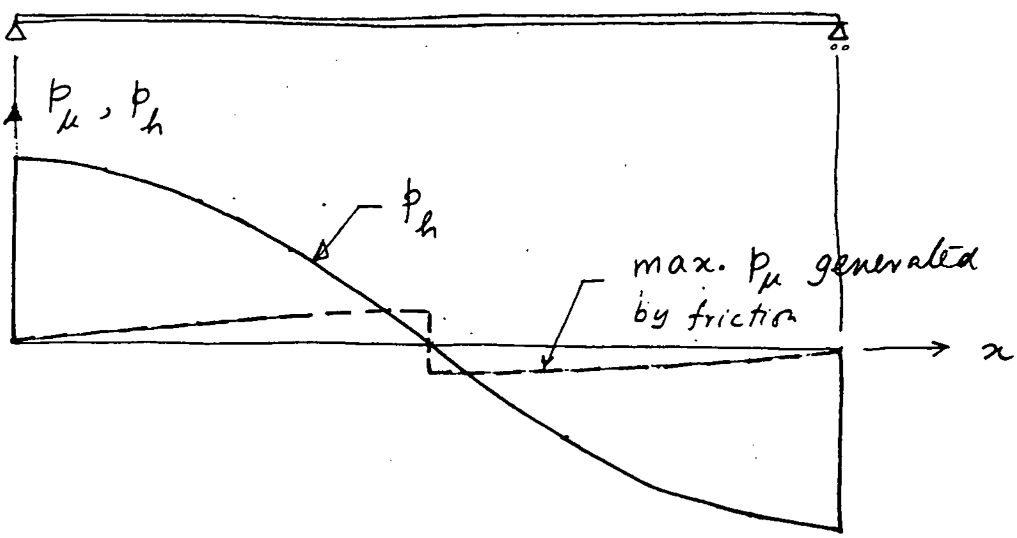



FIGURE 6: Comparison of Interface Horizontal Shear ($p_h$)
and Resistance Due to Friction ($p_\mu$)

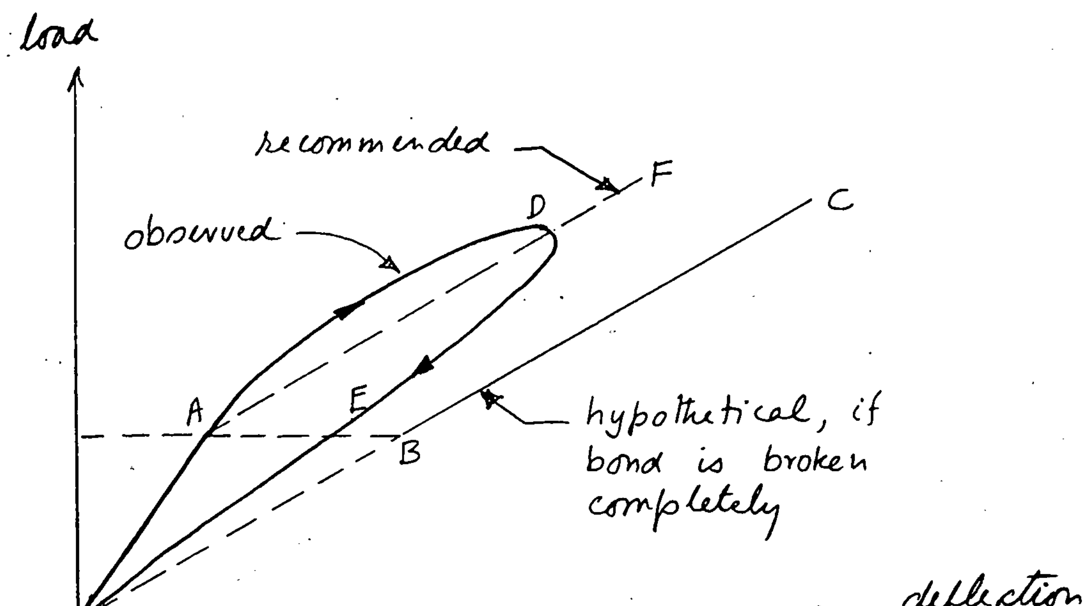
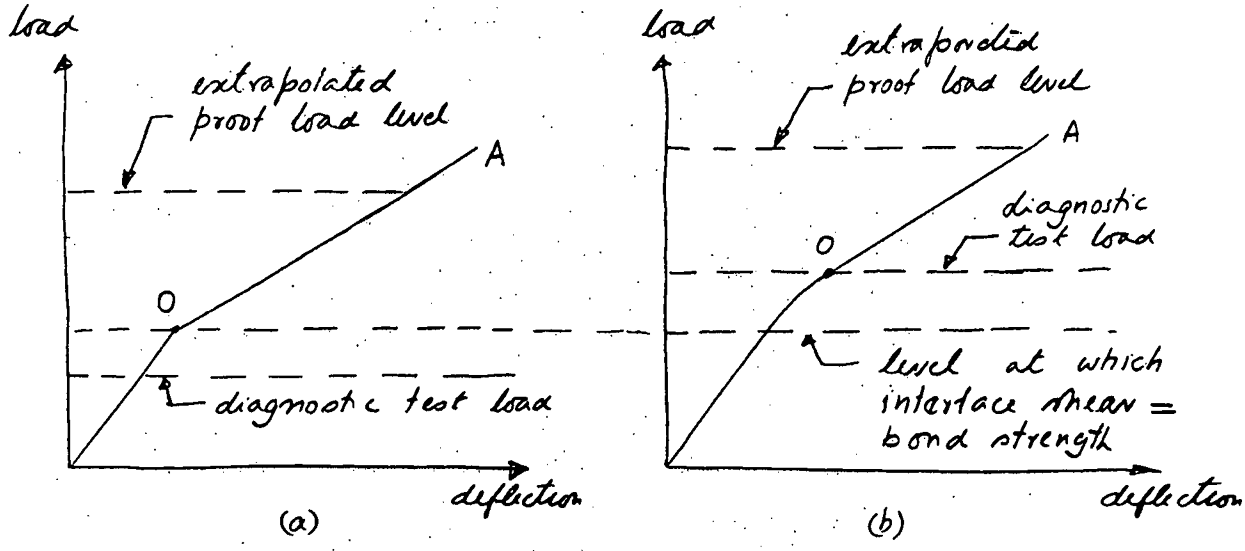



FIGURE 7: Load-Deflection Curves for Beam in which Composite Action Exists Due Only to Bond




NOTE: For OA segment no composite action is assumed

FIGURE 8: Schematic Representation of Determining Extrapolated Proof Load Level

# CHAPTER 3

## DERIVATION OF PROOF LOADING FACTORS BY FRED MOSES

### 3.1 SUMMARY

This chapter presents the basis for the recommended proof loading factors as outlined in Chapter 7 of the "Manual for Bridge Rating Through Load Testing." In particular, the different adjustments that may be needed to determine a target proof load are described herein. This material is presented as a background document to outline the methodology. It is not intended that individuals applying the recommendations in the Manual will have to consult this material in selecting a proof load factor. Rather, this chapter shows the technical basis for the recommended factors. As greater use is made of load and resistance factors following future adoption of AASHTO LRFD design and evaluation procedures, further review may be needed to provide more uniformity in safety between the various design, evaluation and testing procedures.

### 3.2 BACKGROUND

The derivation of proof-test load factors is based on several premises related to bridge safety. Following a proof test, a bridge which is opened to normal traffic should provide the same level of safety as a bridge which is checked by conventional analysis and rating methods. The higher confidence in the performance that results from a proof test will be reflected in permitting lower safety margins and/or less conservative assumptions about behavior and strength compared to those values used in a "paper" verification of bridge capacity.

Among the major uncertainties given in the AASHTO Manual for Condition Evaluation of Bridges (C/E Manual), which requires concern for safety, is the possibility of future overloading of the bridge and the possible development of further deterioration beyond that observed during the inspection. These uncertainties remain regardless of the fact that a proof test was carried out and are incorporated in the derivation of the proof load factors.

### 3.3 SAFETY MODELING

At present, the AASHTO C/E Manual provides a range of safety factors for capacity rating which can lead at their extreme limits to operating and inventory levels. States are then free to select either extreme value or some other level for the determination of posting loads. Different capacities for inventory and operating will also depend on whether working stress or load factor methods of checking are used. The Inventory levels are comparable to the current design safety factor and the Operating level was arbitrarily arrived at to ensure that typical H15 bridges will still be acceptable with current loadings.

The newly developed AASHTO LRFD specifications are different from the present design or inventory safety checks. The safety factors in the LRFD were derived to better correspond to modern truck loads and traffic and to provide more uniform safety over the full range of bridge spans, geometries and materials. Safety in the LRFD format is controlled in terms of a safety index ($\beta$), which accounts for various uncertainties in material properties, bridge behavior, loads and load analysis.

The LRFD framework is suitable for deriving the load factors needed for a proof test. This derivation can incorporate the level of proof testing and the added information obtained from a successful test. Further, the LRFD procedure is used to give the adjustments in the load factor for various circumstances described in the test manual, such as fracture critical members, site loading and inspection intervals.

One limitation in the LRFD procedures is the limited database for assigning statistical parameters needed to calculate the safety index. Further, there is the important issue of the target safety index that should be present after a bridge capacity assessment. These difficulties are alleviated by calibration of the proof-load factors to the safety targets implicit in the new AASHTO LRFD design and evaluation specifications. Also, the statistical data used in those derivations are assumed to be applicable to the present analysis as well as the implicit target betas. It can be seen from a sensitivity study that the derived proof-load test values are not very sensitive to any fluctuations in these parameters as long as the full calibration process is considered. That is, if a given proof load provides the target safety index comparable to the LRFD specifications with corresponding database, then the same proof load will be adequate if a change is made in the statistical database.

## 3.4 CALIBRATION OF SAFETY INDICES

For simplicity, the safety index will be shown in the following so-called "normal" format. That is, assuming that load and resistance are normal distributions, the safety index $\beta$ can be expressed as:

$$\beta=\frac{\text{Mean margin of safety}}{\text{Standard deviation of safety margin}} \tag{1}$$

An exact expression for the probability of failure can be had by using a normal probability table, available in any statistics book. Thus, $\beta = 3$ corresponds to a failure probability of about $10^{-3}$. Similarly, risks can be found for other values of $\beta$. To avoid dealing explicitly with risk numbers, however, structural codes usually express risk directly in terms of $\beta$. For example, in the AISC code, betas of 3.5 for main members and 4.5 for connections are given as target values. If load and resistance follow standard normal distributions, then these risk values are precise; otherwise, these are only approximate.

Letting the margin of safety be written as g, we have:

$$g = \text{Resistance (strength)} - \text{Load Effect} \tag{2}$$

or in typical terminology,

$$g = R - S \tag{3}$$

where the Mean is

$$\bar{g} = \bar{R} - \bar{S} \tag{4}$$

and

$$\sigma_g = \sqrt{\sigma_R^2 + \sigma_S^2} \tag{5}$$

Alternatively, the influence of the safety factor can be seen by rewriting Eqn. 4 as:

$$g' \frac{\bar{R}}{\bar{S}} - 1 = n - 1 \tag{6}$$

where n is the safety factor and g' is the margin of safety, referenced to the mean load and mean strength rather than the nominal values usually considered.

The final definitions relate to the expressions of statistical scatter in terms of the nondimensional coefficient of variation COV, (or $V_x$) which is defined as:

$$COV = \frac{\text{Standard deviation}}{\text{Mean value}} \tag{7}$$

Since engineers usually use conservative values for their variables in code checking, a bias is introduced which is defined as:

$$BIAS = \frac{\text{Mean value}}{\text{Nominal code value}} \tag{8}$$

The remaining part of this section introduces the database appropriate for calculating the safety indices. In general, these are values assigned by the various code committees and do not involve the designers. The latter see only the end product which is the safety factors calibrated by the code committees to achieve the appropriate safety indices.

Resistance—For many materials and representative component limit states the COV of resistance ranges close to 0.10. In addition, due to material specifications, the mean value of material strength is about 12% above the nominal value, i.e. $BIAS_R$ = 1.12. For example, the mean yield strength of A36 steel is about 42 ksi.

Load—The total load effect Q can be written as:

$$Q = D + L + I \tag{9}$$

where, D, L, and I are the dead, live and impact load effects, respectively.

The mean and sigma of the total load are then:

$$\bar{Q} = \bar{D} + \bar{L} + \bar{I} \tag{10}$$

$$\sigma_Q = \sqrt{\sigma_D^2 + \sigma_L^2 + \sigma_I^2} \tag{11}$$

Dead Load—This quantity will vary depending on whether asphalt overlay makes up a significant part of the dead load or whether most of the dead load is the steel and concrete of the beams and deck. Typically, the following parameters apply:

$$BIAS_D = 1.0, \text{ and } V_D = 0.10$$

Live Load—The data for live loads is taken from material developed by Nowak for the AASHTO LRFD bridge design specifications. Although the database for live load is limited by the sites available to that study, the reasoning is consistent and their use here should lead to proof-load factors consistent with the new LRFD safety criteria. For typical span ranges, the Nowak study shows a mean maximum load per lane over a lifetime (75 year) exposure of 1.79 x AASHTO HS20 loading. That is, it is expected that on the average the lane load will reach 1.79 HS20 vehicles in a single lane. For the combined two-lane loading case, the load is reduced to 0.85 x the one-lane situation. That is, simultaneously, once per 75 years both lanes are expected to see, on the average, 0.85 x 1.79 AASHTO loads in each lane. The COV for this live load is given by Nowak as 0.18. These COV cover both the uncertainty of heavy truck occurrences and the uncertainty associated with estimating the effects of these trucks on particular members of the structure (analysis COV). If only the truck load uncertainty and not the analysis is considered, the 0.18 COV should be reduced to about 0.14 based on Nowak's report.

Impact—Dynamic allowances are represented as a percentage of the live load, independent of span length. Nowak reports the mean impact as 10% with a large scatter represented by a COV of 0.80.

### 3.5 CALIBRATION TARGETS

An important aspect of selection of safety margins in the LRFD format is the target safety index. Typically, these are selected by examining existing designs having satisfactory performance. Beta values are extracted from these designs to provide safety levels for the future code changes. Specifications are changed when it is observed that there are significantly varying safety indices for different designs, which need realignment by modifying the LRFD safety factors. The averages of many designs are typically used in selecting the target betas.

From the AASHTO LRFD studies, it has been determined that the target betas should be about 3.5 corresponding to inventory design levels. For the AASHTO Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges (Guide Specifications), the target beta of about 2.3 was found comparable to the operating levels of rating. These lower betas for rating are justified since they reflect past rating practices at the operating levels. For the purpose of posting levels, however, system considerations and member failure consequences are also introduced in the AASHTO Guide Specifications. For example, non-redundant members are assigned lower allowables to bring their safety index to design levels, namely 3.5. Similarly, members with significant deterioration, or for which poor maintenance and infrequent inspections are evident are also assigned lower allowables. These latter situations reflect greater uncertainties in estimating nominal strength or performance variables.

In the new AASHTO C/E Manual, the operating levels require a factor of 1.3 on the combined dead plus live load and impact in the load factor method and a factor of 1.33 in the working stress format. These factors cover all the uncertainties described above. During the proof test, the structure supports both the existing dead load plus whatever live load is applied. A successful proof test should achieve, not the same total load effect as the 1.3 or 1.33 evaluation factor just mentioned, but the same target level of reliability.

Following the proof test, uncertainties are eliminated on the dead load and the strength capacity to support an appropriate pattern of the live load effect. The major uncertainties still remaining after the proof test are the magnitude of future live loads (which may exceed the rating and/or the legal load) and possible future deterioration.

## 3.6 EXAMPLES

### 3.6.1 General

To illustrate the calculation of safety indices, an example will be given. Cases will be described of a design which satisfies the AASHTO Standard Specifications for Highway Bridges (Design Code) and for which no proof test has been performed. Also, results will be shown for cases where proof test have been done to improve ratings. Subsequently, a general presentation of the proof-load factors will be given.

### 3.6.2 Example 1 - No Test Has Been Performed

60 ft. simple span - two lanes

HS20 L.L. - 807 k-ft/Lane

Code Impact, $I = \frac{50}{60+125} = .27$ (=218 k-ft)

For this example assume D.L., same as AASHTO L.L., = 807 k-ft/lane

Inventory level strength (working stress method): The nominal resistance capacity, $R_n$, required per lane after considering lateral load distribution is:

$$R_n = 1.82\,(807+807+218) = 3334 \text{ k-ft}$$

Note: $1.82 = \frac{1}{0.55}$ where 0.55 is the Inventory level allowable stress factor.

Using Eqn. 8 gives for the resistance:

$$\text{Mean, } \overline{R} = 1.12\, R_n = 3734 \text{ k-ft}$$

$$\text{Standard Deviation } \sigma_R = V_R\, \overline{R} = 0.10(3734) = 373 \text{ kip-ft}$$

$$\text{Dead Load, Mean, } \overline{D} = \text{Nominal} = 807$$

$$\sigma_D = \overline{D} V_D = 807(0.10) = 80.7$$

From data given above for the Live Load statistics, the expected maximum load (average load per lane):

$$\overline{L} = 0.85\ (1.79)(807) = 1228 \text{ k-ft}$$

where 0.85 accounts for 2 lanes and 1.79 gives mean largest load for 75-year projection.

$$V_L = 0.18 \text{ (includes analysis uncertainty)}$$

$$\sigma_L = .18(1228) = 221$$

Impact, Mean $\bar{I} = 0.1\ (1228) = 122.8$ (i.e. mean impact value is only 10%)

$$\sigma_I = .8\ (122.8) = 98$$

from Eqn. 9 the Total Load Q = D+L+I

Mean, $\overline{Q} = 807 + 1228 + 122.8 = 2157.8$

Standard Deviation, $\sigma_Q = \left[80.7^2 + 221^2 + 98^2\right]^{\frac{1}{2}} = 255$

Safety index

$$\beta = \frac{\overline{R} - \overline{Q}}{\sqrt{\sigma_R{}^2 + \sigma_Q{}^2}} = \frac{3734 - 2157.8}{\sqrt{373^2 + 255^2}} = 3.49$$

This $\beta$ value (3.49) is comparable to a design level safety target, - 3.5.

*Operating* level strength (working stress method)

In this case, $R_N = 1.33(807 + 807 + 218) = 2436$ k-ft

NOTE: $1.33 = \frac{1}{0.75}$ where 0.75 is the Operating level allowable stress factor.

Mean,

$$\bar{R} = 1.12(2436) = 2729$$

$$\sigma_R = (.10)(2729) = 273$$

Loads same as previous example,

$$\beta = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^{\,2} + \sigma_Q^{\,2}}} = \frac{2729 - 2157.8}{\sqrt{273^2 + 255^2}} = 1.53$$

NOTE: This value is too low; the AASHTO Guide Specification used 2.3 as a target for the operating level—the explanation is the relatively larger 75-year return period loading used in the AASHTO design LRFD calibration compared to evaluation.

For a typical live load used for evaluation—ref. 5 (NCHRP 301), used a mean load factor 1.50 which is appropriate for two-year intervals instead of 1.79 (75-year maximum design value). This leads to:

$$\bar{Q} = 807 + \left(\frac{1.50}{1.79}\right)(1228 + 122.8) = 1939$$

and

$$\sigma_Q = \left[80.7^2 + (1029 \times 0.18)^2 + (103 \times 0.8)^2\right]^{1/2} = 218.2$$

$$\beta = \frac{2729 - 1939}{\sqrt{273^2 + 218.2^2}} = 2.26$$

This $\beta$ value is similar to the acceptable target for evaluation at operating levels.

### 3.6.3 Derivation of Proof-Load Factors

Subsequent to a proof load, the following changes in data should be apparent.

Dead load—The bridge is known to carry whatever dead load is present. Therefore, $V_D = 0$.

Resistance—The test load capacity has been verified. So, corresponding to the level of load placed on the structure, the resistance uncertainty is zero, i.e., $V_R = 0$. A further question is whether the strength bias, i.e., the ratio of true mean to the nominal used in the strength equations, should be used. Typically, this bias ranges about 1.12. It is assumed herein that if the load test is stopped while behavior is still linear and no sign of initial distress has been observed, then the bias $B_R = 1.12$ can still be applied. If the test is stopped because the engineer feels the true strength has been reached, e.g., there is onset of nonlinear response or the appearance of distress, then the strength bias should be taken as 1.0. The effect of these differences will be seen in selection of the rating levels following the test.

Live Load—The future live loads are not known with any greater certainty following the test. However, a satisfactory performance during the test indicates that we should have no concern for any analysis uncertainty. That is, the loads will be carried in the future in the same distribution pattern in which the structure carried the proof loads. Therefore, the analysis portion of the load effect uncertainty should be removed. As cited above, eliminating analysis uncertainty reduces $V_L$ from 0.18 to 0.14.

Impact—There is no change in the assessment of the dynamic load allowance based on the AASHTO Design Code unless a moving load test is performed to investigate the impact.

### 3.6.4 Example 2—Test Has Been Performed

The safety indices will be shown for several levels of test magnitude.

1. Assume a successful load test has been carried out and the load has reached the nominal design strength moment (3334 k-ft) and no distress was observed.

   Using the database given in Example 1 and the parameters appropriate *after* a test, we obtain:

   Resistance, $\bar{R} = 1.12(3334) = 3734$; $\sigma_R = 0$

   Dead Load, $\bar{D} = 807$, $\sigma_D = 0$

   Live Load, $\bar{L} = 1228$, $\sigma_L = 0.14(1228) = 172$

   Impact, $\bar{I} = 122.8$, $\sigma_I = 98$ (same as before)

   $$\beta = \frac{3734 - (807 + 1228 + 122.8)}{\sqrt{0 + \left(0^2 + 172^2 + 98^2\right)}} = 7.96 \text{ (very high)}$$

2. Assume test reaches the nominal operating loading moment (2436 k-ft) with no distress. Then:

   $\bar{R} = 1.12(2436) = 2729$, $\sigma_R = 0$

   and: $\bar{D} = 807$, $\sigma_D = 0$; $\bar{L} = 1228$, $\sigma_L = 172$; $\bar{I} = 122.8$, $\sigma_I = 98$

   $$\beta = \frac{2729 - (807 + 1228 + 122.8)}{\sqrt{0 + \left(0^2 + 172^2 + 98^2\right)}} = 2.88$$

Thus, the proof test raises the calculated operating level safety index, $\beta$, from 1.53 (unacceptable) found in Example 1 to 2.88. As cited above, the acceptable target beta for operating level is given as 2.3.

## 3.7 TARGET PROOF-LOAD FACTORS

The next step is to select the load factors appropriate to a proof-test situation. The basis for the calibration model will be as follows:

Inspection Interval—2 years. This interval is appropriate to a rating level and is needed to select the statistical parameters of the load distribution. From Nowak's data, for a 2-year interval, the mean expected maximum load level for two lanes simultaneously loaded is 0.85 x 1.65 x the HS20 load effect.

Traffic Intensity—The load data just cited were developed from sites with heavy traffic volumes and overloaded vehicles.

Bridge Type—It will be assumed that the operating level load factor will be the reference level for calibration. The bridge will be assumed as a redundant structure without fracture critical details, for which most states would accept operating levels as their target safety requirements. Based on NCHRP Report 301 and the corresponding AASHTO Guide Specification that came from that project, a target safety index of about 2.3 was associated with operating levels.

Proof Test—It is assumed that the performance during the test is acceptable and the full load is applied without signs of distress. Further, it is assumed that the test loadings fully envelop all the limit state conditions that need to be considered in the analysis. For convenience, the loadings will be represented in terms of HS20 levels.

Let $X_p$—Proof load test factor

If the test is stopped prior to any visible distress, then the nominal resistance, $R_n$, is:

$$R_n = X_p + D$$

since the dead load is also being supported. The expression for safety index becomes:

$$\beta = \frac{1.12\left[X_p L_{HS20}\left(1 + I_{AASHTO}\right) + \bar{D}\right] - \left[\bar{D} + \bar{L} + \bar{I}\right]}{\sqrt{\sigma_R^2 + \sigma_D^2 + \sigma_L^2 + \sigma_I^2}}$$

in which the terms are as defined above. Using the same data as in the above example with a two-year interval on mean live load (i.e. 1.65 x 0.85 x HS20 or 1132 k-ft) gives:

$$\beta = \frac{1.12X_P(807)(1.27) - [1132 + 113.2] + 0.12(807)}{\sqrt{0.14^2(1132)^2 + 0.8^2(113.2)^2}}$$

$$\beta = \frac{X_P(1148) - 1245 + 97}{182}$$

which results in the following table:

| $X_P$ | β |
|---|---|
| 1.2 | 1.26 |
| 1.3 | 1.89 |
| 1.4 | 2.57 |
| 1.5 | 3.15 |
| 1.6 | 3.78 |

Rounding off to the nearest 0.1, it appears that a factor of $X_P$ = 1.4 would be consistent with the target safety index. This value is not surprising since the load factor is 1.3 in LFD and 1.33 in working stress method. The factor needs to be slightly higher because only the live load is factored during the proof test. The dead load is assumed to be the mean value. In calculating a rating using the working stress method, the strength must be 1.33 x the sum of dead and live load effects. In the proof test the total applied load is now 1.4 x live load plus 1.0 x dead load. However, the test reduces some of the uncertainty which allows a lower overall proof-load capacity than the 1.33 x (dead plus live) implied in the nominal rating calculations.

The 1.4 factor was derived above for the specific example of a 60-ft span. However, it does provide adequate safety for other spans. For example, for a very short span the impact is 1.3 and the dead load may be neglected. The above equations then give for beta, for a proof-load factor of 1.4:

$$\beta = \frac{1.12\, X_p\, L_{AASHTO}(1 + 0.3) - 1.65 \times 0.85 \times L_{AASHTO}(1.1)}{L_{AASHTO}\left(0.14^2 + 0.8^2 + 0.1^2\right)^{1/2}}$$

$$= 3.07$$

Similarly, for a long span, the impact factor drops off and the dead load quantity increases. This leads to a smaller value for beta, for a D/L value of 3.0:

$$\beta = \frac{1.12 X_p L_{AASHTO}(1.0) - 1.65 \times 0.85 \times L_{AASHTO}(1.1) + 0.12 D}{L_{AASHTO}(0.14^2 + 0.8^2 + 0.1^2)^{1/2}}$$

$$= 2.39$$

Thus, the selected value of 1.4 provides acceptable levels of beta over the full range of application. The lack of a uniform beta, i.e., higher values for shorter spans than for longer spans, may be offset somewhat by the fact that shorter spans are likely to have higher load biases (compared to HS20) than longer spans. These differences are not reflected in the data above.

In summary, the suggested proof-test load factor is 1.4 for the reference case described above. Adjustments of this factor are discussed in the next section to account for situations which differ from the base case.

## 3.8 ADJUSTMENTS IN PROOF-TEST LOAD FACTOR

This section describes the adjustments in proof-load factor needed to account for a variety of situations. The adjustments are reflected in the values given in Section 7 of the proposed Manual.

1. Observed Distress During the Test

   The resistance bias described above should not be used if the test is stopped due to observed distress prior to full application of the proof-test load. To account for this situation, the required proof-test factor $X_p$ should be increased by the factor of 1.12. The influence of this change will be seen in the calculated operating rating factor which is given in Eqn. 7-2 of the proposed Manual. The observed distress indicates the true resistance has been reached and should be used for the rating calculation. When no distress is observed, the final observed resisance is taken as a nominal resistance which typically is a safe conservative value and is exceeded by the true capacity by at least 12%.

2. One-Lane Controls

   One purpose of the proof-test load factor is to envelop the future extreme bridge loads. The data above showed that for a typical two-lane bridge the expected maximum load in each lane was 0.85 times the expected maximum single-lane value. An adjustment in $X_p$ is needed if we have either: a one-lane structure or some component of a multi-lane structure which due to its distribution factors shows greater load effects from a single-lane loading than from the two-lane loadings which are reduced by 0.85. In this case, the proof-test should be performed with a factor $X_p$ of 1.4 on two lanes loaded, and a factor of $X_p$ of 1.4 x 1.15 = 1.61 on a single-lane load.

3. Infrequent Inspections

   If inspections are expected to be infrequent, then the live loading to be enveloped by $X_p$ should correspond to a period longer than two years. The difference in expected maximum loading between two years and a full lifetime is given above as 1.65 vs. 1.79 about a 10% difference. Since infrequent inspection may imply that corrective maintenance will also not be undertaken, then it is prudent to increase the recommended proof-load factors by 10%.

4. Non-Redundant Structures

   The target beta of 2.3 is associated with operating levels and is deemed acceptable for redundant spans with reserve strength to provide greater safety against collapse. For non-redundant spans, a target beta of 3.5, corresponding to inventory or design levels is needed. From the table above, to increase beta by 1.0 requires a 10% increase in $X_p$. A double penalty, such as applying this additional 10% and also using the lower inventory level for posting, is not warranted.

5. Additional Factors

   Further adjustments in $X_p$ may be made using the material in Reference 6. The variables discussed therein include traffic intensity and quality of maintenance and data are provided for further adjustments in $X_p$.

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