Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipients Catalog No.
FHWA/TX-92+1239-3		

4. Title and Subtitle	5. Report Date / 6. Performing Organization Code
EVALUATION OF BRIDGE DECKS AS LATERAL BRACING FOR SUPPORTING STEEL STRINGERS	5. Report Date: May 1992 6. Performing Organization Code:
7. Author(s) Stuart T. Webb and Joseph A. Yura	**8. Performing Organization Report No.** Research Report 1239-3
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin Austin, Texas 78712-1075	**10. Work Unit No. (TRAIS)** **11. Contract or Grant No.** Research Study 3-5-90/1-1239
12. Sponsoring Agency Name and Address Texas Department of Transportation Transportation Planning Division P. O. Box 5051 Austin, Texas 78763-5051	**13. Type of Report and Period Covered** Interim **14. Sponsoring Agency Code**

15. Supplementary Notes

Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration
Research Study Title: "Bracing Effects of Bridge Decks"

16. Abstract

A full-size test on a wooden bridge deck showed that the deck nailers and the connections between the nailers and planks were the main contributors to the lateral stiffness of the deck. The measured coefficient of friction between the steel stringers and the wooden deck was 0.25. Design methods are presented for the bracing requirements for steel stringers including the required coefficient of friction.

Various typical bridge decks were evaluated, and it was found that the decks had sufficient stiffness to force the supporting stringers to yield before buckling. The required coefficient of friction for stringers with span-to-depth ratios = 40 is 0.08. Less friction is required for smaller span-to-depth ratios.

17. Key Words	18. Distribution Statement
bridge decks, lateral bracing, steel stringers, support, nailers, planks, connections, lateral stiffness, coefficient of friction, buckling	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19 Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	80	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

EVALUATION OF BRIDGE DECKS AS LATERAL BRACING FOR SUPPORTING STEEL STRINGERS

by

Stuart T. Webb

and

Joseph A. Yura

Research Report No. 1239-3

Research Project 3-5-90-1239
"Bracing Effects of Bridge Decks"

Conducted for

Texas Department of Transportation

In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

May 1992

NOT INTENDED FOR CONSTRUCTION,
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PREFACE

In this report the lateral stiffness of wooden bridge decks is evaluated for the purpose of providing bracing to the supporting steel stringers. A full-size bridge deck was tested along with its various components to provide guidance for engineers evaluating the bracing effect of similar systems. The timber deck was not attached to the steel stringers.

The work reported here in is one phase of Research Project 3-5-90-1239, "Bracing Effects of Bridge Decks." The studies described were conducted at the Phil M. Ferguson Structural Engineering Laboratory as part of the overall research program of the Center for Transportation Research of The University of Texas at Austin. The work was sponsored jointly by the Texas Department of Highways and Public Transportation and the Federal Highway Administration under an agreement with The University of Texas at Austin and the Texas Department of Highways and Public Transportation. Technical contact and support by the Bridge Division was provided by Mark Bloschock.

SUMMARY

A full-size test on a wooden bridge deck showed that the deck nailers and the connections between the nailers and planks were the main contributors to the lateral stiffness of the deck. The measured coefficient of friction between the steel stringers and the wooden deck was 0.25. Design methods are presented for the bracing requirements for steel stringers including the required coefficient of friction.

Various typical bridge decks were evaluated and it was found that the decks had sufficient stiffness to force the supporting stringers to yield before buckling. The required coefficient of friction for stringers with span-to-depth ratios = 40 is 0.08. Less friction is required for smaller span-to-depth ratios.

IMPLEMENTATION

Friction can be relied on to mobilize the bracing effect of the bridge deck. A simple method has been developed to determine the lateral stiffness of the deck which can then be compared to the bracing requirements given in another phase of this project. In all the practical cases considered herein, yielding of the stringers would occur before buckling because of the bracing effect of the bridge deck. The Bridge Rating Manual should adopt the procedures herein for evaluating the bracing effect of wooden bridge decks.

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

There are many older rural off-system short span steel bridges in Texas which must be periodically rated for capacity and overall condition. Typical construction consists of a timber or concrete deck supported by steel stringers. Depending upon the details of construction, the capacity of a stringer may be limited by lateral torsional buckling considerations to a value less than the yield strength of the material. Bracing is frequently provided to increase the buckling strength. This can take the form of cross frames between the stringers at discrete points located intermittently along the span or by continuous bracing provided by composite action between the stringer and deck. A common problem with short span off-system bridges is that no intermediate bracing between stringers or positive connection of the deck to the stringers has been provided, thus apparently rendering the compression flange unsupported over the full span. An engineer charged with the responsibility of evaluating the capacity of such a bridge system faces a difficult and challenging task. An assumption that the steel stringers are laterally unbraced over the full span often results in a calculated capacity which is well below the actual service loads supportable by the bridge. The deck, though not positively attached, increases the lateral buckling strength of the steel stringers by providing some degree of lateral restraint. An understanding of the bracing characteristics provided by the deck to the supporting stringers is necessary for an engineer to properly evaluate the capacity of the overall bridge system. Figure 1.1 is a photograph of a typical off-system bridge with a timber deck. For this bridge, the detail of the timber deck and supporting stringer in Figure 1.2 shows that no connection between the deck and stringer has been provided.

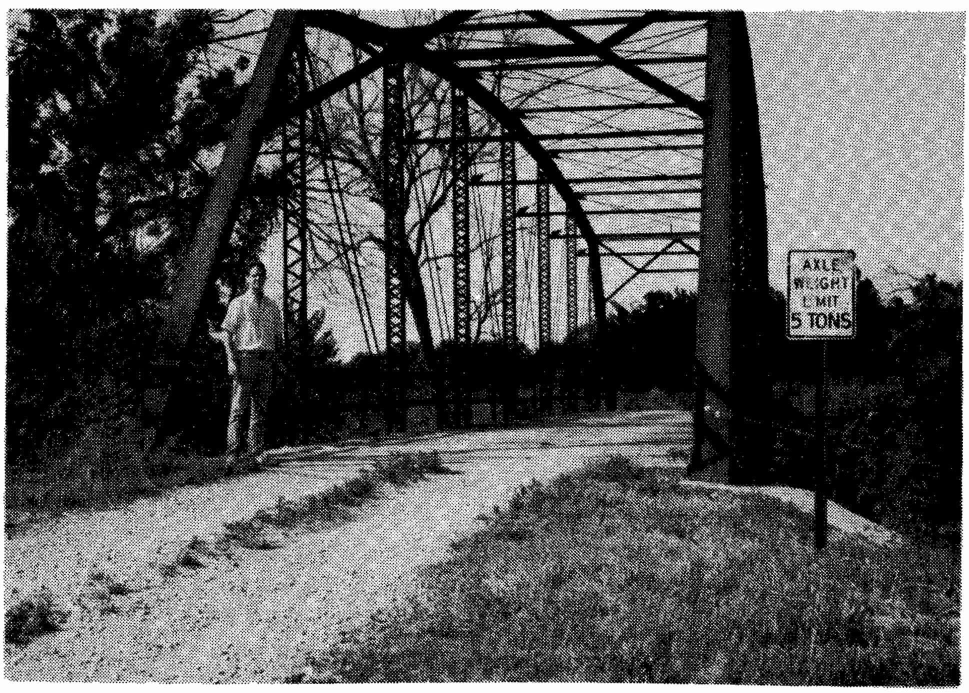


Figure 1.1 Typical off-system bridge with timber deck

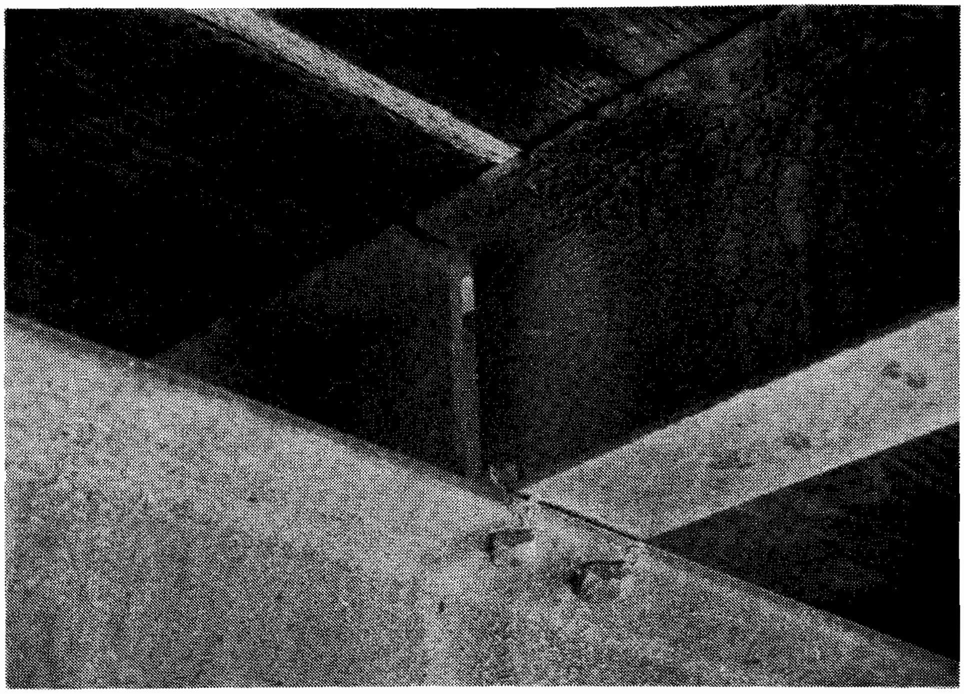


Figure 1.2 Typical timber deck/stringer connection

1.2 Objective/Scope

The investigation reported herein is one phase of a research project, sponsored by the Texas Department of Transportation. The purpose of the project is to define the bracing requirements for steel beams and to determine the bracing contribution of typical bridge decks. Phases of the research project which have been completed include: a study of the bracing requirements of individual elastic steel beams based on theoretical analysis and experimentation by Phillips (1991) and a full scale five stringer bridge tested to failure as reported by Vegnesa (1991). The objective of this investigation was to study the performance of a typical bridge deck as a lateral brace for the supporting steel stringers and to provide guidance for engineers evaluating similar bridge systems.

A timber plank deck with steel stringers was chosen as a worst case system. No positive connection was provided between the wood deck and the stringers. Four spliced 2x6 nailers were used to fasten the 4x8 planks together forming the deck. The full bridge system and its components are presented in Figure 1.3. The lateral stiffness of the individual components as well as the full system was determined experimentally. The stringer/deck lateral stiffness was tested both before and after the full bridge system was loaded to failure by Vegnesa (1991). The lateral stiffness provided by the deck was determined as the difference between the stiffness of the full bridge system and the stiffness contributions of other major components. Guidance for the evaluation of similar bridges was formulated accirding to the results of this investigation and equations developed as part of other phases of the overall research project.

1.3 Related Research

The most closely related study of the bracing effects of bridge decks was completed by Kissane (1985) for the New York State Department of Transportation. The objective of that research was to determine the effectiveness of a non-composite concrete bridge deck as a lateral brace for the compression flange of the supporting steel stringers without any positive shear connection between the two. Steps were taken to eliminate any physical or chemical bond between the concrete deck and the supporting S12x31.8 steel stringers. The steel stringers which

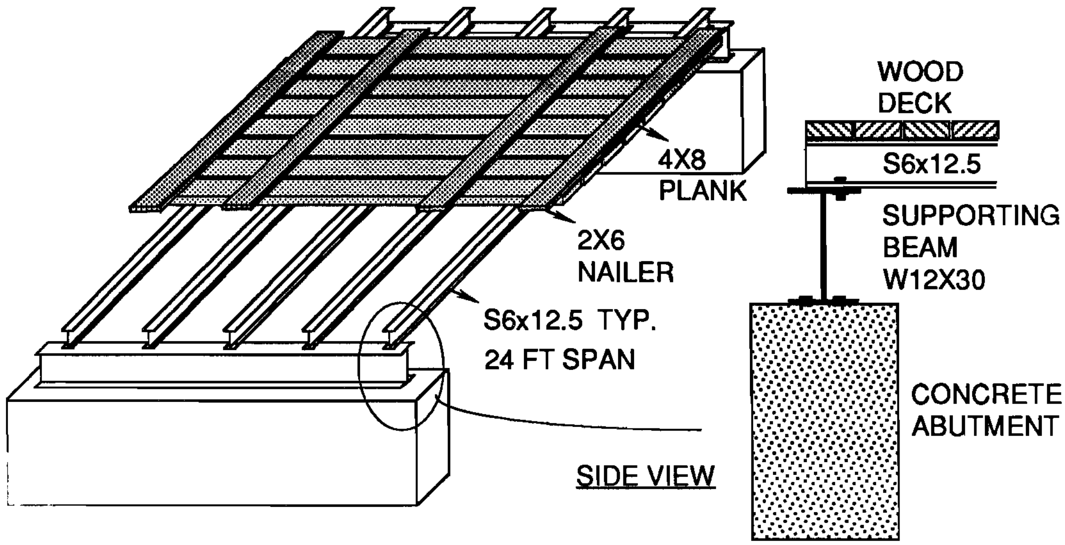


Figure 1.3 Full bridge system

spanned 21 feet were loaded until flange yield was observed. Kissane concluded that the friction resistance between the concrete deck and stringers was sufficient to mobilize the deck as a brace and allow the stringers to reach their full in-plane bending capacity without buckling laterally. Other factors which may have increased the buckling capacity of the stringers were not discussed. Such factors would include any restraint provided by the connections between the stringers and supporting transverse girders. Also, torsional restraint may have been provided if the deck prevented the top flange of the stringer from twisting. As part of this research (Kissane, 1985), a related field test was conducted on a similarly constructed bridge system. Test results indicated that the steel stringer resisted less than 15 percent of the applied load. The majority of the load was carried directly by the continuous concrete deck in bending. No conclusions could be drawn from the field test concerning the bracing effects of the bridge deck, since the load carried by the stringers was well below that required to cause buckling.

CHAPTER 2

BACKGROUND

2.1 General

The strong axis flexural capacity of a beam may be limited to a value less than that provided by its material strength due to the stability phenomenon known as lateral torsional buckling. In general, lateral torsional buckling involves two interdependent deformations. When the bending moment applied to a beam reaches the critical buckling value, M_{cr}, the member will experience out-of-plane displacement and twisting of the cross section, as shown in Figure 2.1.

Whether the capacity of a beam is limited by material strength or lateral torsional buckling depends on the properties of the cross section and the unbraced length of the member. When a laterally unbraced beam is bent about the weak axis moment of inertia, I_y, the capacity of the member will be limited by material strength. When bending occurs about the strong axis moment of inertia, I_x, lateral torsional buckling must be considered. If sufficient bracing is provided such that the beam is forced to buckle between intermediate brace points, the buckling moment may be increased to a high enough level that the capacity is limited by material strength. In order to design or evaluate a member properly, it is important to understand the factors contributing to lateral torsional instability. The following sections will discuss lateral torsional buckling equations and bracing as it relates to the stability of beams.

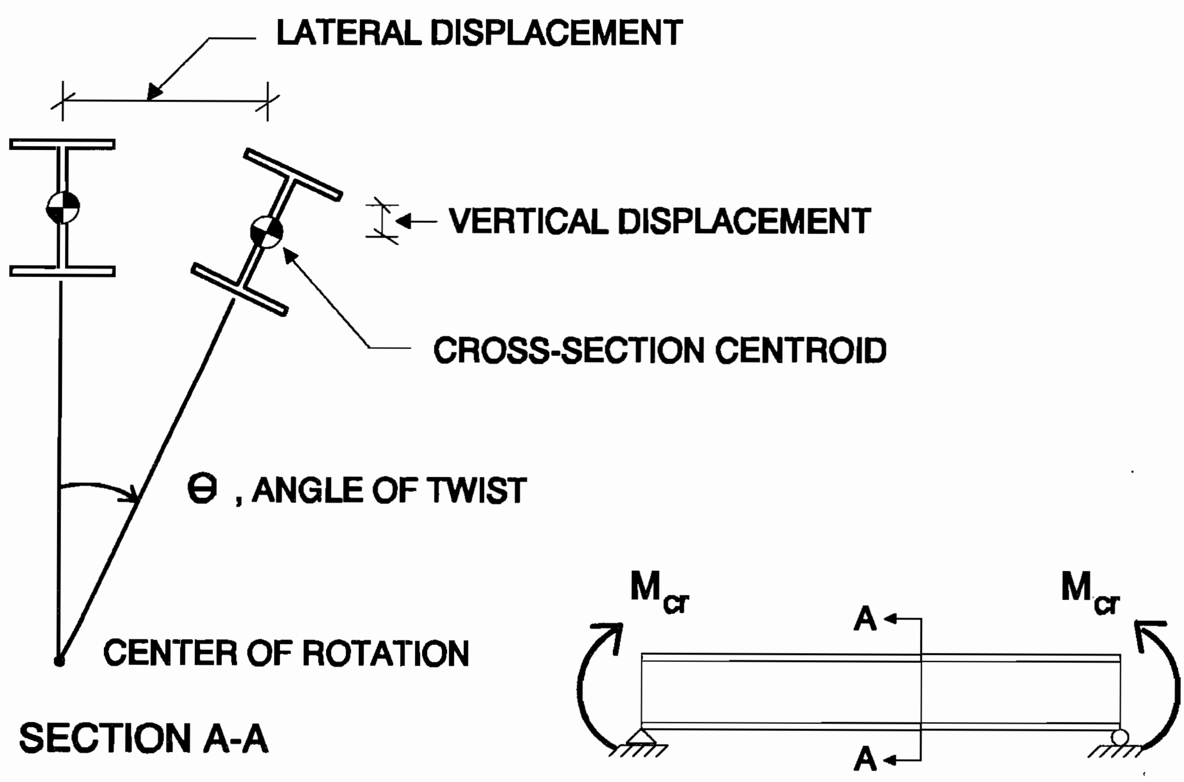


Figure 2.1 Lateral torsional buckling of simply supported beams

2.2 Theory

Lateral torsional buckling equations for beams have evolved from those developed for columns. The major difference between the two is due to the twist of the beam cross section caused by the applied moment which exerts a component of torque about the laterally deflected longitudinal axis. Equation 2.1 was developed by Timoshenko (1960) for the critical buckling moment, M_{cr}, of a doubly-symmetric elastic beam subjected to a uniform moment with twist and lateral displacement prevented at the supports.

$$M_{cr} = \frac{\pi}{L}\sqrt{EI_yGJ + \frac{\pi^2E^2I_y^2h^2}{4L^2}} \qquad (2.1)$$

For Equation 2.1, L = unbraced length, E = modulus of elasticity, G = shear modulus, J = St. Venant's torsional constant, I_y = weak axis moment of inertia, and h = distance between top and bottom flange centroids. The first term under the radical is related to the torsional rigidity, GJ, of the beam. The second term under the radical is related to the warping rigidity, EC_w.

$$C_w = \frac{I_yh^2}{4} \qquad (2.2)$$

Equation 2.1 was developed for the uniform moment load case. When other loading conditions are considered, three factors will affect the uniform moment solution. These factors are moment gradient, the point of load application with respect to the centroid of the cross section and any lateral bending or warping restraint provided at the supports.

A modified version of Equation 2.1 appears in the 1990 AASHTO Bridge Specification. In the Strength Design method, the moment capacity of the beam, M_r, in lb-in. units is

$$M_r = 91x10^6C_b\left(\frac{I_{yc}}{L}\right)\sqrt{0.772\left(\frac{J}{I_{yc}}\right)+9.87\left(\frac{d}{L}\right)^2} \leq M_y \qquad (2.3)$$

where C_b = factor to account for moment gradient between brace points, I_{yc} = weak axis moment of inertia of the compression flange, d = depth of beam, and M_y = yield moment. In the Service Load Design method, the allowable moment is based on a formula similar to Equation 2.3 with the coefficient 91 changed to 50 and limited to $0.55M_y$ corresponding to a design safety factor of 1.8.

The 1990 AASHTO Bridge Specification version (Equation 2.3) is in very good agreement with Equation 2.1. However, lateral torsional buckling equations published in the AASHTO Bridge Specification prior to 1989 result in lower calculated capacities with negative values possible for large unbraced lengths. For example, given an S12x31.8 stringer with an unbraced length of 30 ft, the critical buckling moment calculated using Equation 2.1 is 39.2 kip-ft, 37.8 kip-ft for Equation 2.3, and -1.6 kip-ft for the 1983 AASHTO Bridge Specification equation. It is evident that the 1990 AASHTO is more accurate than the previous AASHTO equation.

The critical buckling moment may be significantly greater than that determined by Equation 2.1 for load cases other than uniform moment. A modifying factor, C_b, may be applied to account for portions of a beam which are subjected to a lower moment due to a moment gradient along the span. The 1990 AASHTO Bridge Specification C_b factor for moment gradient between brace points is calculated as follows:

$$C_b = 1.75 + 1.05(\frac{M_1}{M_2}) + 0.3(\frac{M_1}{M_2})^2 \leq 2.3 \tag{2.4}$$

where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beam. However, this equation is only applicable to cases with a linear moment gradient between brace points. The 1990 AASHTO requires a C_b factor of 1.0 when the maximum moment occurs between brace points, as is the case for a simply supported unbraced beam with a concentrated load at midspan. The more general moment diagram cases are better represented by the AISC-LRFD (second edition), which recommends that the C_b factor be calculated by

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_2 + 4M_{cl} + 3M_4} \tag{2.5}$$

where M_{max} = maximum moment along span, M_2 = moment at 1/4 span, M_{cl} = moment at midspan, and M_4 = moment at 3/4 span, with all moments taken as positive. Applying Equation 2.5 to a simply supported unbraced beam with a concentrated load at midspan results in a C_b factor of 1.3. This represents a 30% increase in allowable buckling moment over the 1990 AASHTO Bridge Specification. Additional information concerning the effects of moment gradient may be found in Kirby and Nethercot (1979).

Critical buckling moment may be affected by the location of the point of load application with respect to the centroid of the cross section. In general, buckling strength is significantly reduced when load is applied at a point above the centroid, due to an increase in twist caused by the eccentricity; buckling strength is significantly increased when load is applied at a point below the centroid, due to a reduction in twist caused by the righting action of the eccentric load. Adjustment factors which account for load position may be found in the SSRC Guide (1988). When the first term under the radical of Equation 2.1 dominates, load position will have only a small effect on the critical buckling moment. If the second term under the radical dominates, load position will have significant effect on the buckling moment. If the load point is also a full lateral brace point, load position will have no effect since the load does not move laterally during buckling.

The pinned end boundary conditions used in the development of Equation 2.1 require that no lateral bending or warping restraint occur at the end supports. However, end conditions may exist in which either lateral bending and warping or both are restrained. In Equation 2.6, which has been modified from Kirby and Nethercot (1979), K factors account for these two possible types of end fixity,

$$M_{cr} = \frac{\pi}{K_1 L}\sqrt{EI_y GJ + \frac{\pi^2 E^2 I_y^2 h^2}{4K_2^2 L^2}} \tag{2.6}$$

where K_1 = lateral bending restraint factor, K_2 = warping restraint factor, and all other terms are as defined for Equation 2.1. Values for various end restraint conditions have been published (Vlasov, 1959). However, accurate assessment of the degree to which actual support connections provide restraint is difficult. Improper interpretation for K values may result in unconservative buckling strengths. It is conservative to assume simple supports and use the corresponding value of 1.0 for both K_1 and K_2.

2.3 Bracing

The critical buckling moment for a beam may be greatly increased when sufficient bracing exists to reduce the unbraced length. Such bracing can be accomplished in a number of ways. Bracing may be continuous along the span of the beam or located at discrete points. An example of continuous beam bracing would be that provided by a metal or composite concrete deck. Discrete beam bracing may be established through secondary members such as purlins, stringers, or cross beams.

When a beam experiences lateral torsional buckling there is a relative lateral movement between the top and bottom flanges of the beam, as shown in Figure 2.1. This relative lateral movement is termed twist of the cross section. The effectiveness of a brace is defined by its ability to prevent this twist. As a beam buckles, the lateral deflection of the tension flange is very small when compared with that of the compression flange. A simply supported beam is considered to be braced at a point when lateral displacement of the compression flange is prevented since twist of the cross section is also restricted. In the case of twin beams with a diaphragm or cross frame between the members, lateral displacement of the system is permitted at the cross frame. This location, while able to displace laterally, is still considered a brace point because twist is prevented. The most important consideration in the design of an ideal brace is not the prevention of lateral displacement but twisting of the cross-section.

In general, bracing may be divided into two main categories, lateral and torsional bracing. Lateral bracing restrains lateral displacement as its name implies. The effectiveness of a lateral brace is directly related to the degree that twist of the cross section is restrained. For the uniform moment case illustrated in Figure 2.1, the center of twist is located at a point near or outside of the tension flange. A lateral brace is most efficient in restricting twist when it is located at the compression flange. If applied at the centroid, the bracing requirements will be about seven times greater than that required for application at the top flange when cross-section distortion is prevented with a stiffener plate and about thirty times with distortion allowed (Yura, 1990). Lateral bracing applied at the bottom flange of a simply supported beam is almost totally ineffective. A torsional brace can be differentiated from a lateral brace in that twist of the cross section is restrained directly, as in the case of cross frames or diaphragms located between adjacent stringers. The location of the torsional brace with respect to the cross section is not a factor if cross-section distortion is prevented by a stiffener plate.

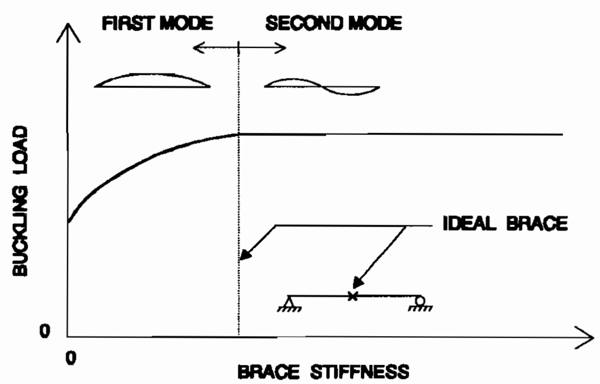


Figure 2.2 Buckling strength/brace stiffness relationship

An ideal brace must possess both minimum strength and minimum stiffness to prevent twisting of the cross section. If a brace has the required strength and stiffness the beam will be forced to buckle between full brace points, increasing the buckling strength of the member. Figure 2.2 illustrates the relationship between buckling strength and brace stiffness for a perfectly straight simply supported beam with a single brace located at midspan. When the brace has a stiffness less than the ideal value, the beam will buckle in the first mode as exemplified by a half sine curve. When the brace has a stiffness greater than or equal to the ideal stiffness, the beam is forced to buckle between brace points in the second mode as denoted by a full sine curve or "S" shape.

All members have some degree of initial out-of-straightness. This initial imperfection results in the lateral displacement of a beam prior to buckling. The force applied to the brace is related to the degree of initial imperfection. It is common practice to use a value for the brace force equal to 2% of the axial force in the compression zone of the cross section. Designs based on this 2% force provide sufficient strength in the brace.

Later in this report, it will be shown that a brace force of 0.8% is adequate for most cases. However, designing for strength alone does not guarantee that the brace will have sufficient stiffness to increase the critical buckling moment to the desired level.

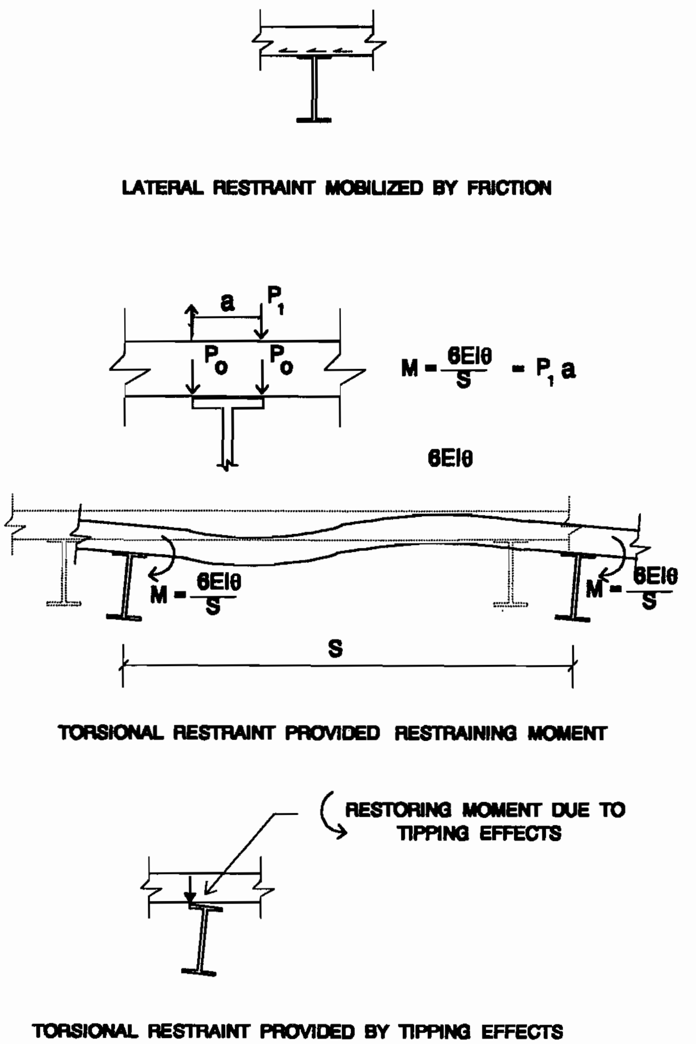


Figure 2.3 Lateral and torsional restraint provided by bridge deck

This investigation focused on the bracing characteristics provided by a bridge deck to the supporting steel stringers. A timber plank deck may have no positive connection to the stringers. The lateral bracing characteristic of the deck is mobilized through friction between the planks forming the deck and the supporting steel stringers. The deck is also effective as a direct torsional restraint, as shown in Figure 2.3. Torsional restraint is provided initially through the flexural stiffness of the timber planks. The restraining moment, M, provided by each plank is

$$M = \frac{6EI}{S}\theta \qquad (2.7)$$

where E = modulus of elasticity of the plank, I = weak axis moment of inertia of the plank, S = stringer spacing, and θ = angle of twist of the cross-section.

Wheel loads, P_o, are transferred to the supporting stringers through contact with one or two planks, as shown in Figure 2.3. When the angle of twist is small, P_1 is less than P_o, the plank remains in contact with the top flange of the stringer, and a restraining moment is provided by the loaded planks. At some critical angle of twist, P_1 becomes greater than P_o, and the planks partially lose contact, bearing on only one edge of the top flange. At this point, the restraining moment provided by the flexural stiffness of the planks is lost, but the deck continues to provide torsional restraint through a restoring moment due to "tipping effects," as shown in Figure 2.3. As the plank bears on the edge of the top flange, a restoring moment is applied to the cross section tending to prevent twist.

The bridge deck utilized in this investigation was composed of 4x8 planks tied together by four spliced 2x6 nailers. The dead weight of the deck was transferred to the supporting steel stringers continuously along the span. Truck wheel loads were applied and transferred by only one to three planks to the stringers below. These conditions resulted in the deck providing lateral and torsional restraint in various combinations of the modes illustrated in Figure 2.3.

CHAPTER 3

EXPERIMENTAL PROGRAM AND TEST RESULTS

3.1 General

The full bridge system consisted of a typical timber deck supported on five S6x12.5 steel stringers spaced 3-ft on center with a span of 24-ft. The deck was constructed with 16-ft 4x8 planks spanning over the stringers and fastened together with four 24-ft spliced 2x6 nailers. Figure 3.1 shows the full system bridge deck, nailer numbering system, and a detail of the spliced nailers. Figure 3.2 shows the configuration of the steel stringers and the supports used for the full bridge system. The spliced nailers were fastened to each plank with two 10d screw shank nails. The specifications for the screw shank nails with a detail of the nailer to plank connection are provided in Figure 3.3. All of the wood used to construct the bridge deck was No. 2 Southern Yellow Pine. No positive connection between the deck and supporting steel stringers was provided. Both lateral and axial translation of the deck was restrained at the North support while only lateral translation was restrained at the South support. This allowed for the deck to respond to lateral loads in accordance with a classic pin/roller model. The stringers were fastened to the supports with four 3/8- inch-diameter bolts (two at each end), tightened to 200 ft-lb. Wedge washers were used to compensate for the tapered flanges of the "S" shape stringers.

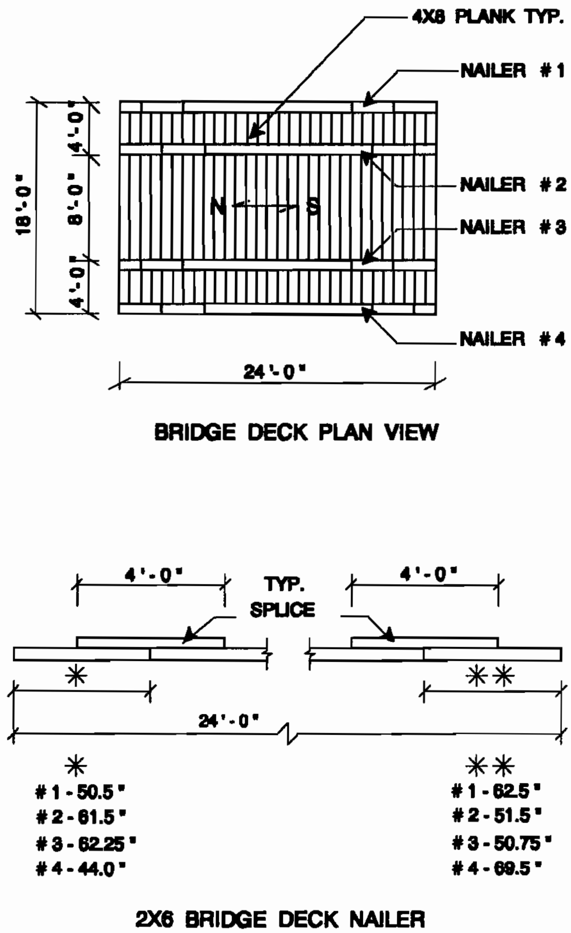


Figure 3.1 Full system bridge deck.

The experimental program was designed to determine the relative contribution of the planks, nailers, fasteners, steel stringers, and contact pressure to the lateral stiffness of the full bridge system. The test program was divided into two phases. A preliminary phase concentrated on the evaluation of the lateral stiffness contributions of each component of the stringer/deck system. Testing was conducted on individual components as well as combinations of components at various stages of construction. The full system phase concentrated on the evaluation of the lateral stiffness of the full bridge system. Testing was conducted before and after the bridge was loaded to failure (Vegnesa, 1991).

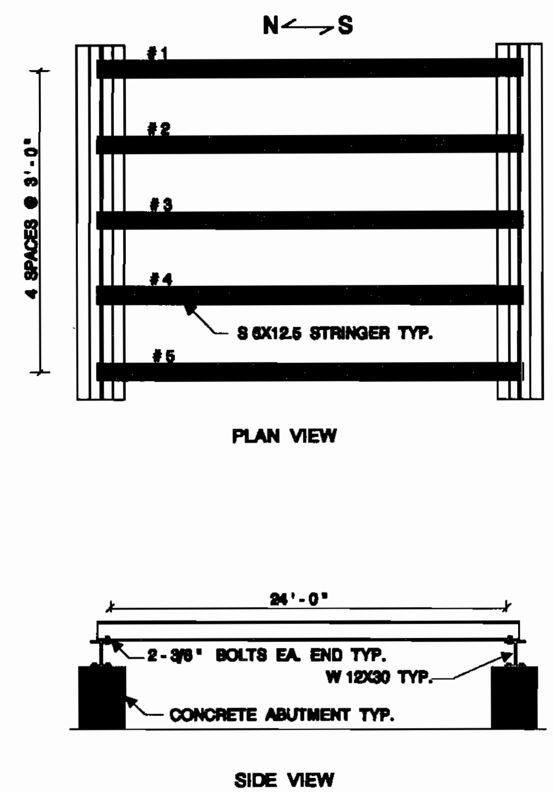


Figure 3.2 Full system stringers and supports.

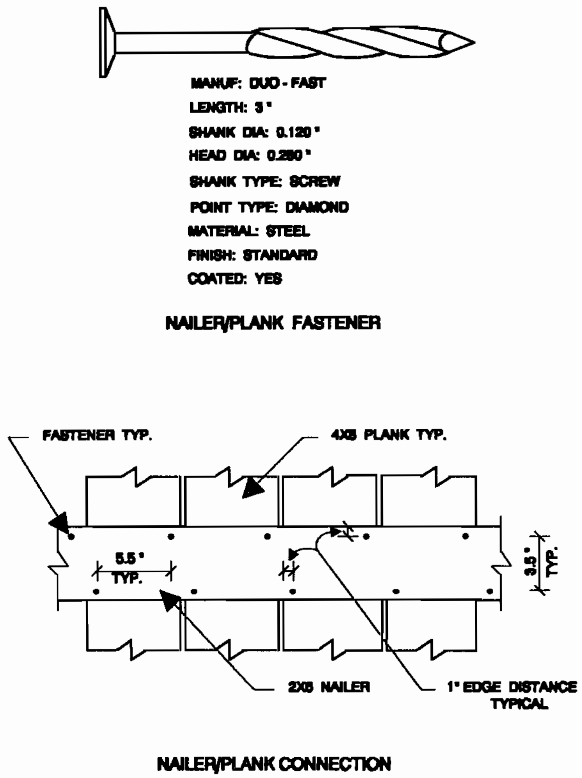


Figure 3.3 Bridge deck nailer/plank connection.

3.2 Preliminary Tests

3.2.1 Overview. Preliminary tests were conducted on each major component of the full bridge system. Testing at various stages of construction was also performed in order to detect significant contributions to the lateral stiffness of the full system. Dial gages with an accuracy of 0.001 were used for all tests. A summary and brief description of the preliminary phase tests is provided in Table 3.1. Preliminary tests LAT1 through LAT6 were conducted on stringers without the deck installed. The stringers were fastened to the full system supports with two 3/8-inch-diameter bolts at each end tightened to 200 ft-lbs.

3.2.2 Deck Nailers. The spliced 2x6 nailers used to construct the full system deck were tested to determine lateral stiffness. Figure 3.4 shows the test setup and orientation of the nailers. Section A-A is a detail of the setup supports. The 2x4 support blocks prevent lateral rotation of the nailers. Successive dead weights were applied at midspan; midspan deflections were recorded for each load increment using a dial gage. The orientation of the applied load with respect to the North and South ends of the nailers was identical with that of the full system tests. The same test setup, modified for a 13'-6" span, was used to determine the material properties and lateral stiffness of the 14-ft continuous 2x6 members used to construct the 24-ft spliced deck nailers.

Table 3.1 Summary of Preliminary Phase Tests

Preliminary Test	Description of Test
Spliced 2x6 Deck Nailers	Lateral stiffness test for each of the four spliced 2x6 nailers used to construct the full system bridge deck.
Continuous 2x6	Test to determine the material properties and lateral stiffness of continuous 2x6 members.
Nailer/Plank Connection	Test to determine the rotational restraint provided by the nails connecting the nailers to the planks.
WAX	Weak axis (lateral) stiffness test of the five S6x12.5 stringers with ideal simply supported end conditions
*LAT1	Lateral stiffness test for a single S6x12.5 stringer with midspan lateral centroid loading.
*LAT2	Lateral stiffness test for a single S6x12.5 stringer with midspan lateral top flange loading.
*LAT3	Lateral stiffness test for twin S6x12.5 stringers with top flanges connected at midspan by a thin cable and midspan lateral top flange loading.
LAT4	Test to determine the coefficient of friction between the 4x8 planks and the s6x12.5 stringers
*LAT5	Lateral stiffness test for twin S6x12.5 stringers with top flanges connected at midspan by a loaded plank with 2x4 shear blocks and midspan lateral top flange loading.
*LAT6	Lateral stiffness test for twin S6x12.5 stringers with top flanges connected at midspan by an unloaded plank with 2x4 shear blocks and midspan lateral top flange loading.

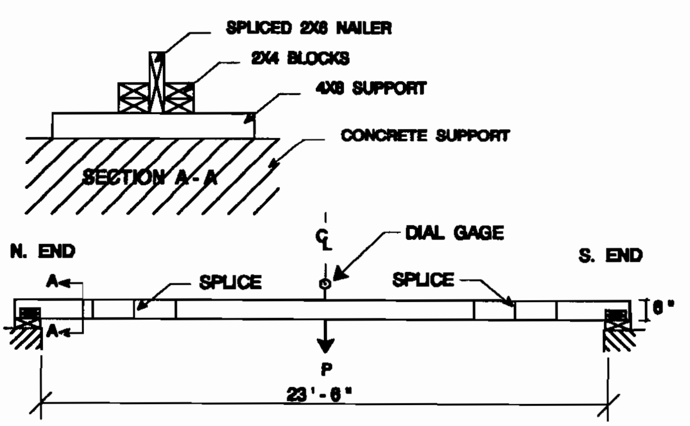


Figure 3.4 Lateral stiffness test setup for 2x6 nailers.

The test setup used to determine the lateral stiffness of the spliced nailers had a span of 23'-6". The recorded deflection values were factored to reflect the 24'-0" span of the subsequent full system tests. Figure 3.5 presents the factored results for each spliced nailer along with an average nailer stiffness curve. The stiffness of nailers #1 through #3 was fairly uniform. The lower stiffness of nailer #4 may be regarded as insignificant with respect to the relative position of the average nailer curve.

The modulus of elasticity was determined to be 1,760 ksi by the continuous 2x6 member test. The National Design Specification (1986) recommended value of 1,600 ksi for the modulus of elasticity is 10% lower than the tested value. Figure 3.6 compares the stiffness of the average spliced nailer and the continuous 2x6 nailer. This figure

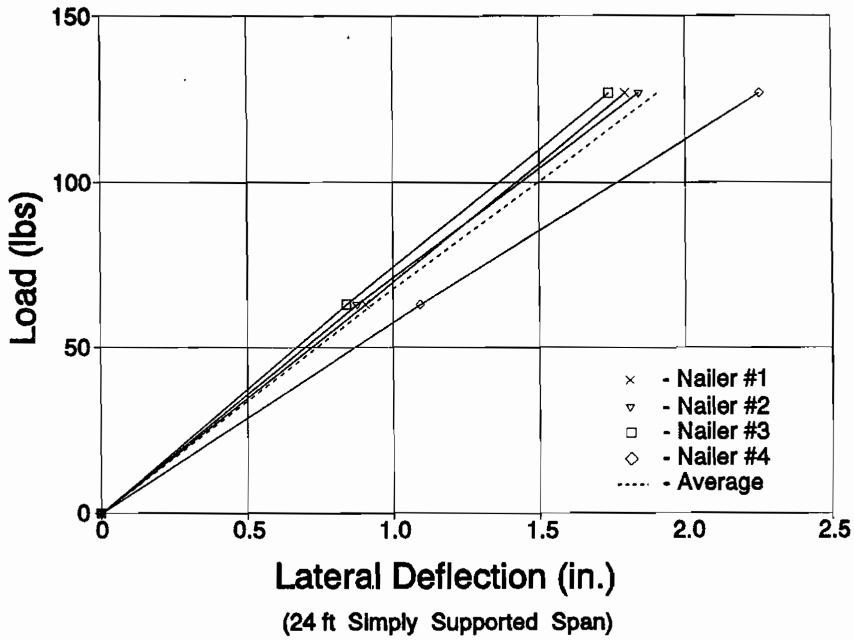


Figure 3.5 Lateral stiffness of spliced 2x6 nailers.

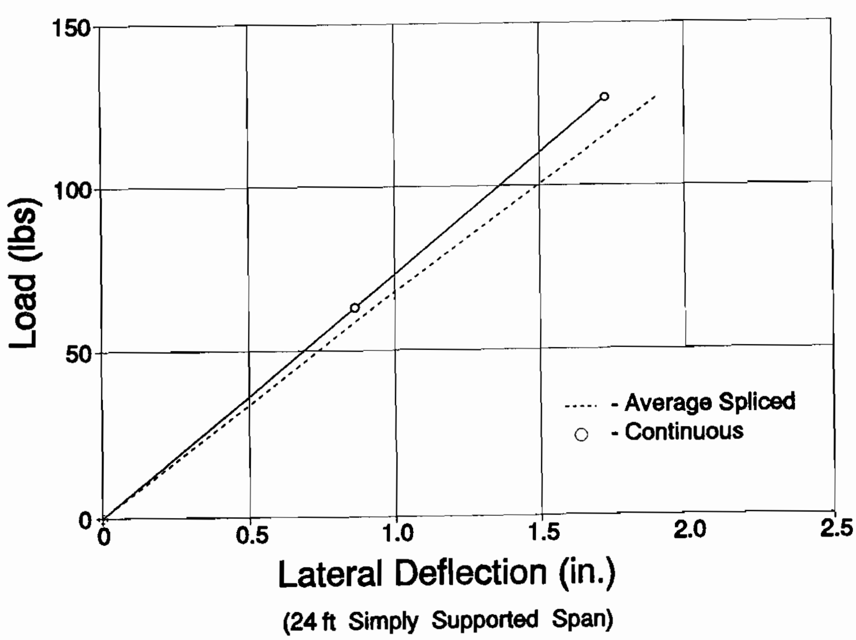


Figure 3.6 Lateral stiffness of continuous 2x6 nailer.

indicates that the splices slightly reduced the nailer stiffness. Table 3.2 lists stiffness values for each of the curves in Figures 3.5 and 3.6; as determined through linear regression analysis. A stiffness of 66.9 lb/in was calculated using a moment of inertia based on nominal cross-section dimensions and the published value of 1,600 ksi for the modulus of elasticity. This stiffness is within 1% of the tested average. It can be concluded that given an adequate splice a good approximation for the lateral stiffness of the nailers can be obtained by using nominal cross-section dimensions, published material properties, and an unspliced model for the member.

Table 3.2 Lateral stiffness of 2x6 nailers.

Member	Spliced Nailer Number				Avg. Nailer	Tested Cont. 2x6	Calc. Cont. 2x6
	#1	#2	#3	#4			
Lateral Stiffness (lbs/in.)	70.5	69.5	73.4	56.6	67.5	73.7	66.9

3.2.3 Nailer/Plank Connection. All of the 4x8 planks used to construct the full system bridge deck were significantly warped. Because of this warping, the 2x6 deck nailers were not in full contact with the 4x8 planks. The nailers were fastened to each plank with two 10d screw shank nails as detailed in Figure 3.3. The rotational restraint provided by this connection was determined experimentally for various degrees of contact between the members. Figure 3.7 shows the test setup consisting of a 4-ft 2x6 nailer fastened to a 5-ft length of 4x8 plank with various imposed gaps as detailed by Section A-A. The 4x8 plank was secured to a support to prevent rotation. Successive dead weights were applied to the nailer at a distance of 33.75 inches from the centroid of the nail group. Dial gages were located 6 inches from each side of the nail group centroid; readings of nailer rotation were recorded for each increment of applied dead load moment. Readings were taken after all substantial creep had occurred.

The results of the nailer/plank connection rotational restraint test are presented in Figure 3.8. The "No Gap" curve represents full contact between the 2x6 nailer and 4x8 plank. The 1/8, 1/4, and 5/16 "Gap" curves represent tests in which gaps between the members were imposed, as illustrated by Section A-A of Figure 3.7. Two to three specimens were tested for each degree of contact. Dial gage readings for successive tests on each specimen were averaged. The averaged dial gage readings of successive specimen tests with the same degree of contact were averaged. From these reduced data the left and right dial gage values were averaged and divided by the six-inch distance between the nail group centroid and each dial gage. Thus, the average rotation per applied dead load moment was obtained. The rotational stiffness of the full contact curve is due to both friction between the wood members and the lateral capacity of the nails. The frictional component was insignificant without full contact between the members. Figure 3.8 indicates that the rotational stiffness for the members with imposed gaps was fairly uniform, with the stiffness decreasing as the gap increased. As expected, the full contact stiffness was much greater.

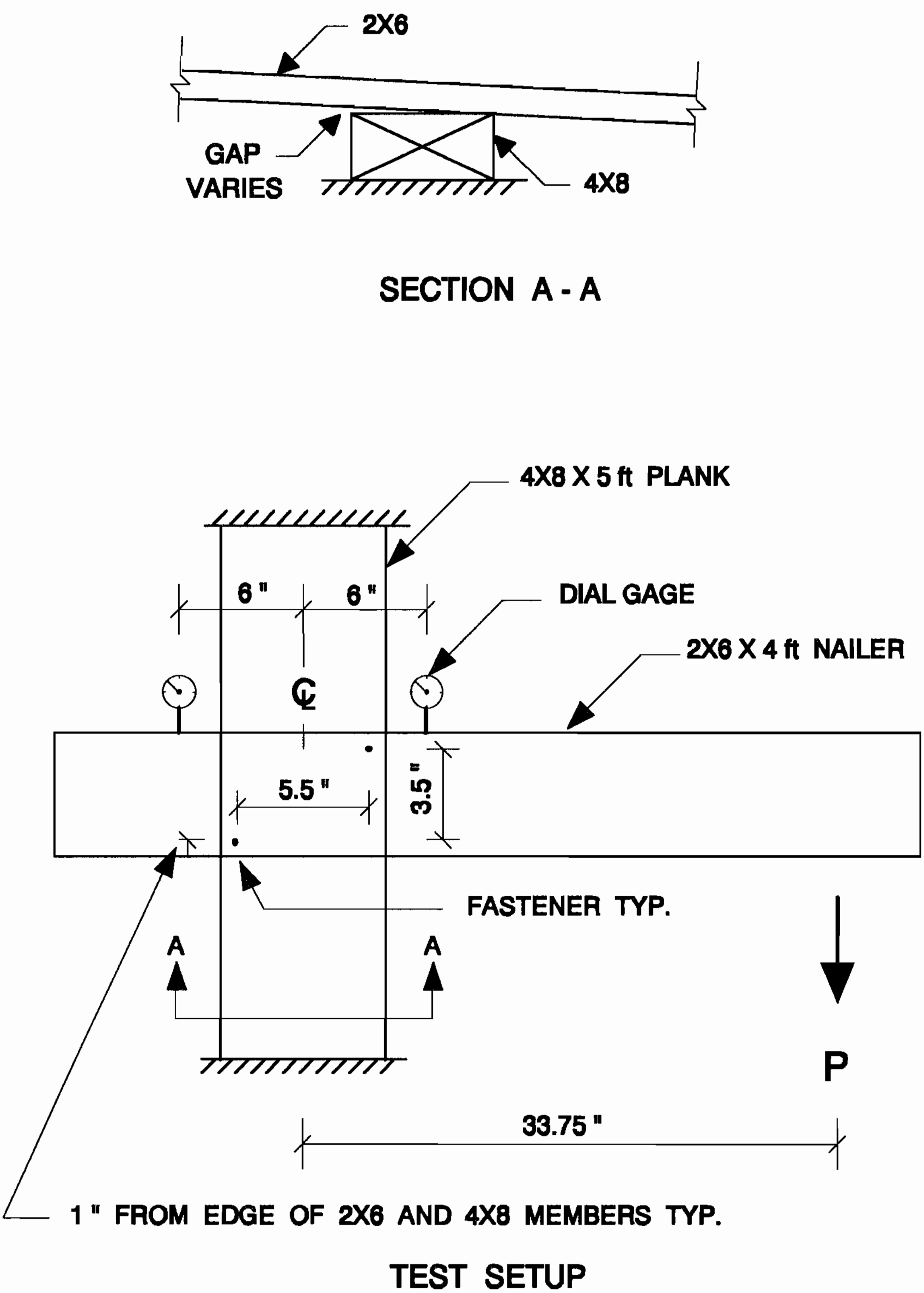


Figure 3.7 Nailer/plank connection rotational restraint test setup.

Table 3.3 Summary of Rotational Stiffness Values.

Degree of Contact	Full Contact	1/8-in. Gap	1/4-in. Gap	5/16-in. Gap
Rotational Stiffness (kip-in./rad)	861	150	140	119

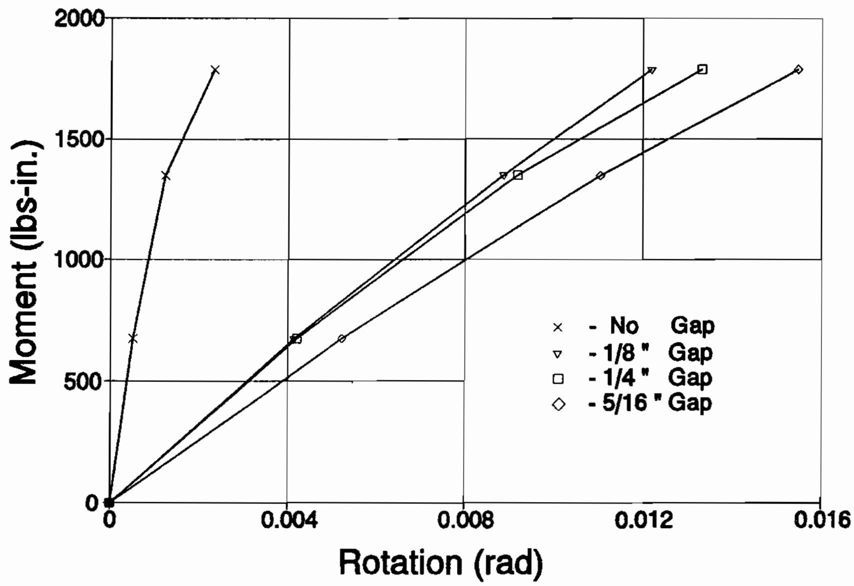


Figure 3.8 Rotational restraint provided by nailer/plank connection.

The rotational stiffness values, as determined by linear regression analysis for each degree of contact, are listed in Table 3.3. It can be seen that the loss of friction reduces the rotational stiffness significantly. The stiffness with a 5/16 gap is only 14% of that with full contact. Gaps between the nailers and planks of the full system deck were not measured. The average rotational stiffness for the nailer/plank connection was postulated as 119 kip-in./rad for subsequent analysis of the full bridge system because the 5/16 inch gap most closely approximated average actual conditions.

3.2.4 Simply Supported Weak Axis (WAX) Stiffness. The WAX test determined the weak axis (lateral) stiffness of the five S6x12.5 stringers used to construct the full scale bridge. Each stringer was placed in the test setup shown in Figure 3.9. The end conditions of this setup represent ideal simple supports. Successive dead weights were applied at midspan and a dial gage was used to measure midspan deflection for each increment of applied dead load.

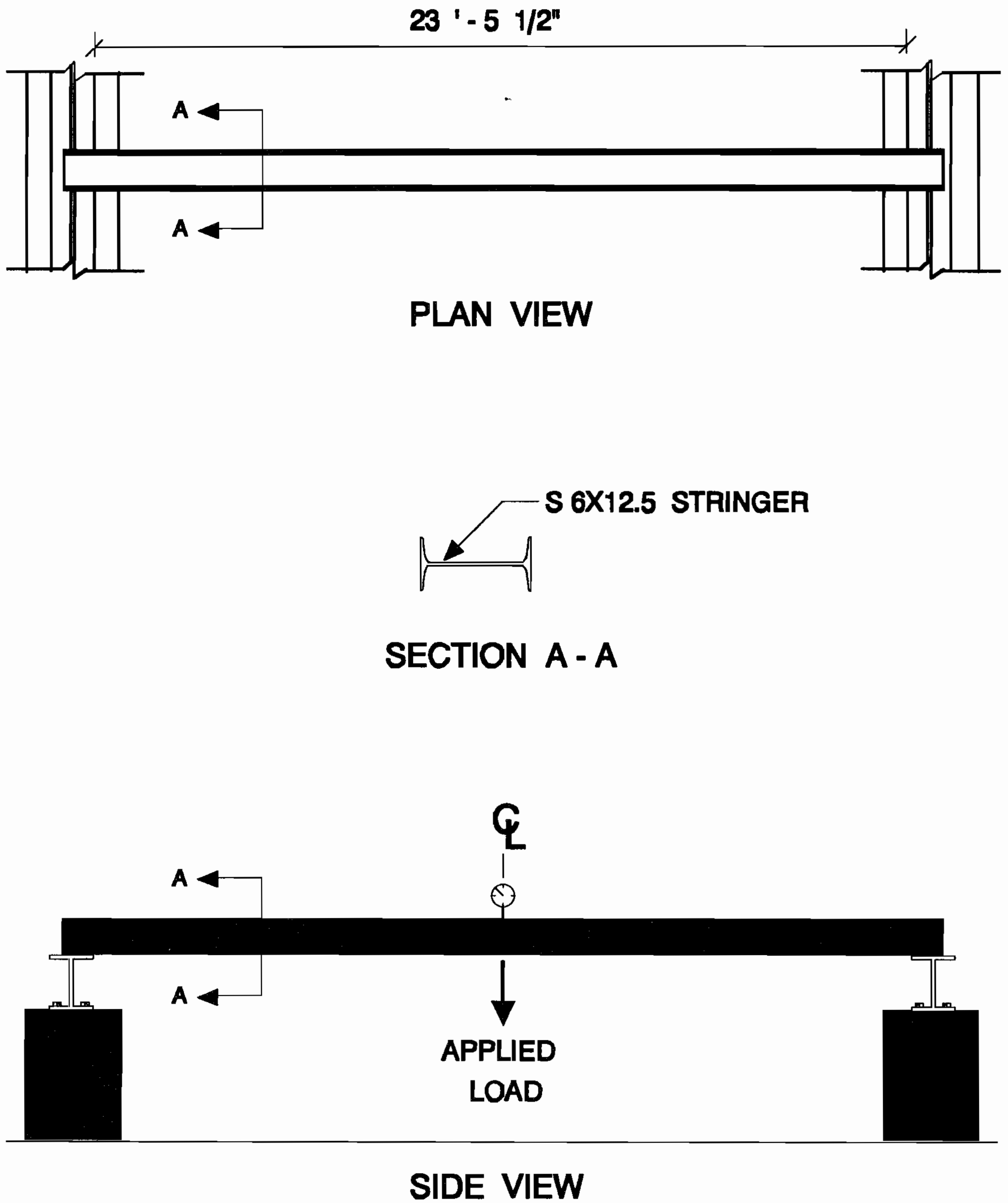


Figure 3.9 Stringer simply supported weak axis stiffness test setup.

The WAX test setup had a span of 23'-5 1/2". Recorded midspan deflection values were factored to reflect the 24-ft span of subsequent tests. Table 3.4 presents the lateral stiffness values for each of the five stringers as determined by linear regression analysis. A calculated stiffness based on the weak axis moment of inertia provided by the AISC Manual for a S6x12.5 and an assumed value of 29,000 ksi for the modulus of elasticity has also been listed in the table. The lateral stiffness of the five stringers was nearly constant. The calculated stiffness was within 2% of the average tested stiffness for the stringers.

Table 3.4 Summary of simply supported weak axis stiffness values.

Member	S5x12.5 Stringer Number					Avg. Stringer	AISC Value
	#1	#2	#3	#4	#5		
Lateral Stiffness (lbs/in.)	107	108	108	109	109	108	106

3.2.5 *Lateral Stiffness of the Stringers Without the Deck.*

3.2.5.1 General. After the five S6x12.5 stringers were installed in the full bridge system shown in Figure 3.2, several tests were conducted on the stringers prior to the installation of the deck. Two 3/8-inch-diameter bolts with wedge type washers were used to fasten each end of each stringer to the W12x30 supports to represent typical field connections. Bolts were tightened to 200 ft-lbs with a manual torque wrench to provide uniform end conditions for all five stringers. Dial gages were used to measure lateral displacements. Inclinometers were used to measure any lateral rotation at the supports. Lateral rotations for all of the stringer tests were very small and determined to have no significant effect on lateral stiffness.

The loading system used for the lateral stiffness tests on the stringers is presented in Figure 3.10. The loading frame was constructed with back to back channels and fastened to the floor with expansion type anchors. A roller was used to support a 1/4-inch-diameter steel cable in order to minimize the effects of friction. One end of the cable was fastened to the stringers and successive dead weights were attached to the other end. The system applied incremental lateral dead loads to the stringers at midspan.

3.2.5.2 Test LAT1. A schematic drawing of test LAT1 is presented in Figure 3.11. This test was conducted on stringer #5, as identified in Figure 3.2. Lateral loads using dead weights were applied through the centroid of the cross section at midspan. Dial gages were located at the top and bottom flanges of the stringer at midspan to measure the lateral deflection of the centroid and the rotation of the cross section. Inclinometers were attached to the top flange over each end support to measure any lateral twist of the cross section.

The results for the LAT1 test along with those of the averaged simply supported lateral stiffness (WAX) test are presented in Figure 3.12. Comparison of these curves indicates that the bolted end conditions of the LAT1 test increased the initial lateral stiffness over the ideal simple support end conditions used in the WAX tests. The slope of the LAT1 curve through the last three data points is slightly less than the slope of the WAX

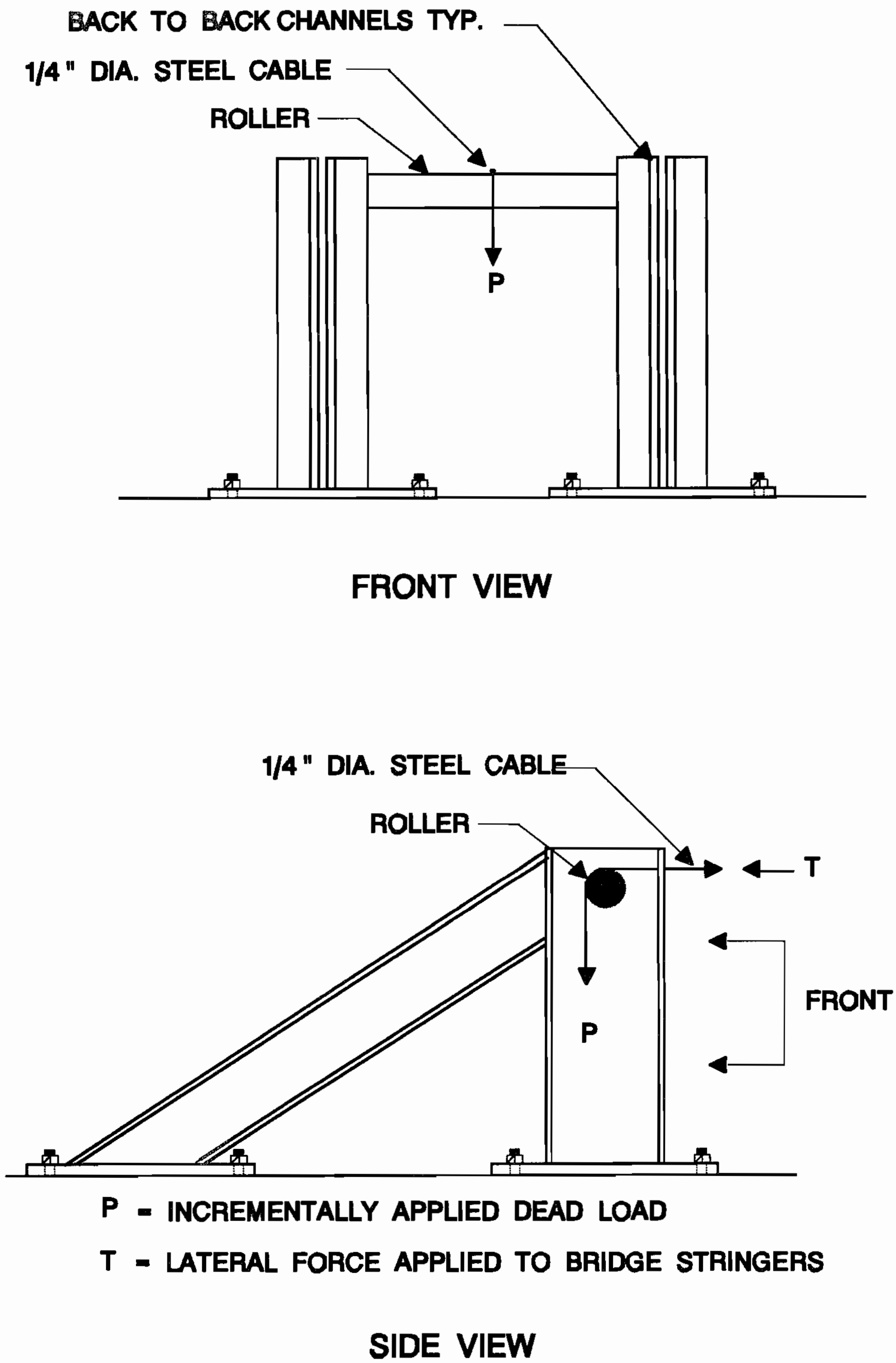


Figure 3.10 Loading system for preliminary stringer tests.

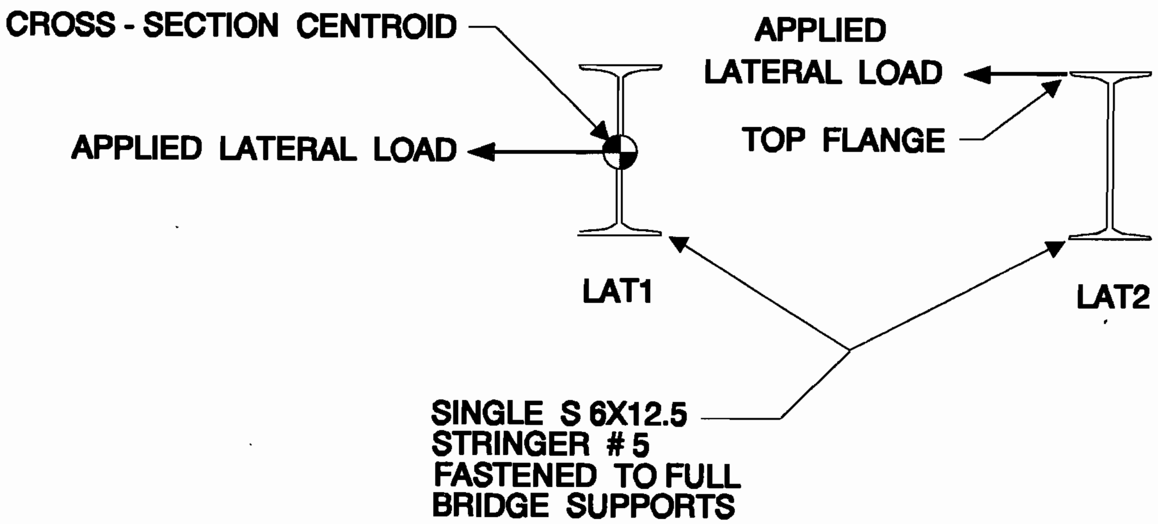


Figure 3.11 Test setups for LAT1 and LAT2.

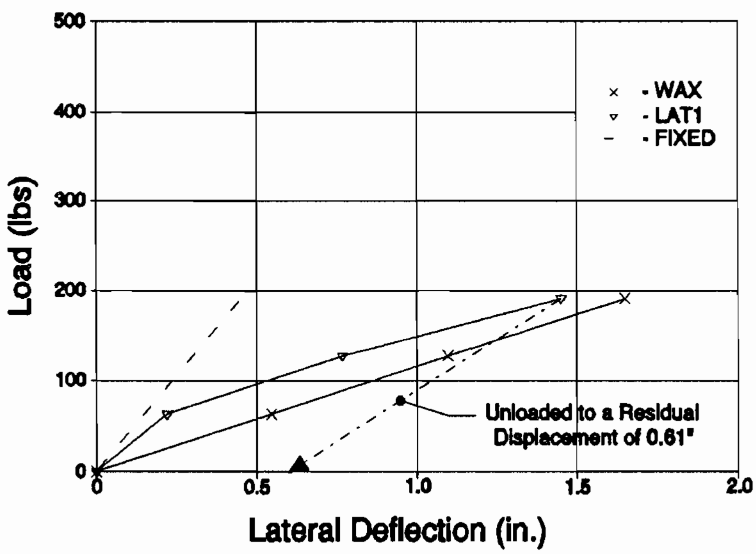


Figure 3.12 Lateral stiffness results for Test LAT1.

curve. The LAT1 stiffness falls between the fixed (dashed line) and simple (WAX) support conditions shown in the figure. The torqued bolts used to fasten the stringer to the supports provided significant end restraint until slip occurred. After slip, the load-deflection response was similar to the linear behavior of the ideal simple support (WAX) test data. After unloading, a residual midspan displacement of 0.61" was observed. Residual displacement was due the lateral rotational slip which occurred at the bolted end connections. Bolts were not retorqued for subsequent stringer tests. Residual displacement was used as the beginning point for measured deflection in subsequent lateral stiffness tests.

3.2.5.3 Test LAT2. The LAT2 test setup is shown in Figure 3.11. The test procedure was identical to that of LAT1, with the exception that the lateral loads were applied at the top flange of stringer #5 instead of through the centroid. The results for the LAT2 test along with those for the WAX and LAT1 tests are presented in Figure 3.13. The 0.61" residual displacement after unloading shown in Figure 3.12 has been omitted from Figure 3.13 for the purpose of comparison. The lateral stiffness of the stringer #5 was increased significantly for the LAT2 test. The stiffness curve is fairly linear. The slight decreases in stiffness between data points were probably due to slip occurring at the support connections. After unloading, an insignificant residual displacement of 0.016" was observed. There appears to be some correlation between increasing stiffness and increasing the distance from the bolted bottom flange support to the point of application for the applied lateral load. Further testing, to include deeper sections and loading applied at the bottom flange, would be required to fully describe any relationship which may exist.

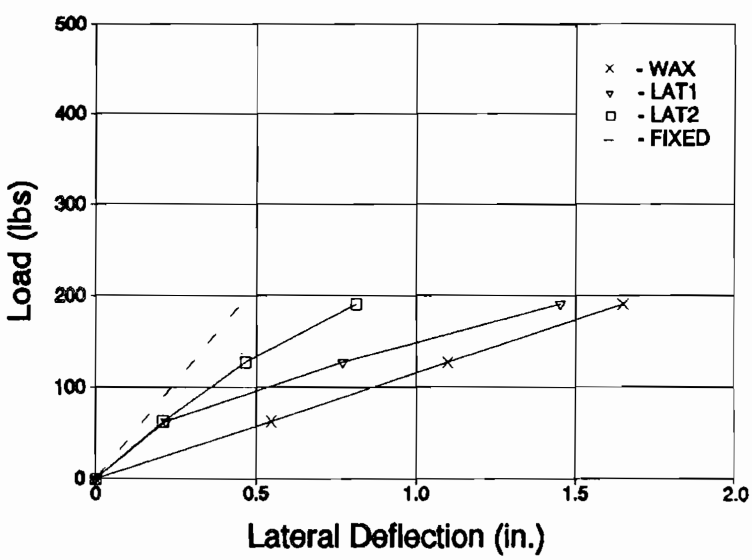


Figure 3.13 Lateral stiffness results for Test LAT2.

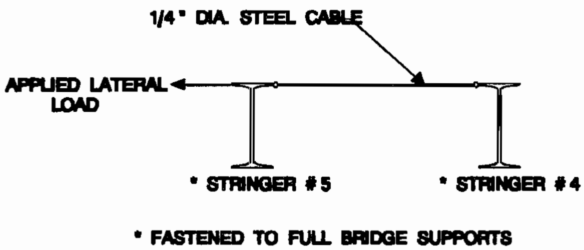


Figure 3.14 Test setup for LAT3.

3.2.5.4 Test LAT3. For the LAT3 test, stringers #5 and #4 were connected together at their top flanges at midspan using a 1/4 inch diameter steel cable and steel hooks, as shown schematically in Figure 3.14. The steel cable and hooks transferred the applied lateral load without providing any significant rotational restraint. The loading and instrumentation was identical with that of LAT1 and LAT2 except that midspan dial gages were provided for both stringers.

The lateral deflections of stringer #5 were 12% greater than those for stringer #4 because of movements in the connecting steel cable and hooks. For comparison purposes, the lateral deflections of both stringers were added together to reflect the response of a single stringer to equivalent applied lateral loads. The results of the LAT3 test, representing the equivalent response of a single stringer, are presented in Figure 3.15 with the results from the WAX, LAT1, and LAT2 tests. No appreciable difference between the LAT2 and LAT3 stiffness test results was observed.

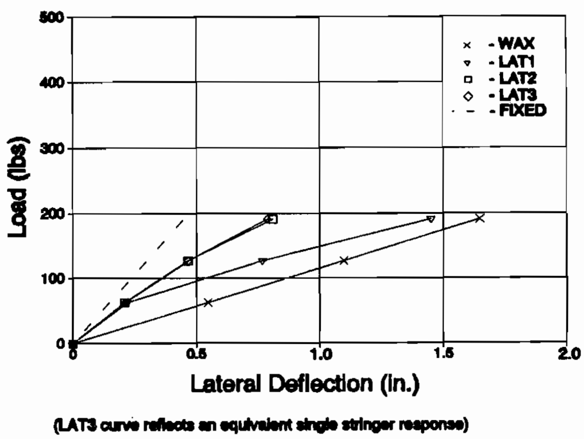


Figure 3.15 Lateral stiffness results for Test LAT3.

3.2.5.5 Test LAT4. The purpose of the LAT4 test was to determine the static coefficient of the friction between the wood planks and steel stringers. A single 6-ft 4x8 plank was located at midspan bearing on the top flange of stringers #5 and #4, as shown in Figure 3.16. A 485-lb. dead weight was applied to the plank at the midpoint between the two supporting stringers. Successive lateral dead loads were applied until the plank was observed to slip. The coefficient of friction was determined to be 0.25.

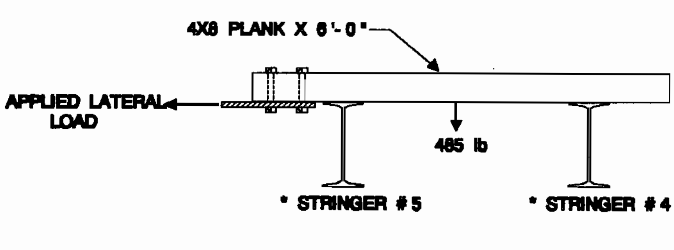


Figure 3.16 Test setup for LAT4.

3.2.5.6 Test LAT5. The purpose of the LAT5 and LAT6 tests was to determine the effect of in-plane loading on the lateral stiffness of the stringers. In-plane loading results in the tipping effects discussed in Chapter 2. While tipping effects provide increased lateral stiffness, there is also a corresponding loss in stiffness due to the applied load. As the applied bending moment reaches the value of the critical buckling moment, the lateral stiffness of the stringer is reduced to zero. When no bending moment is applied to the stringer, the lateral stiffness is at a maximum. The lateral stiffness of the stringer will follow a linear path between these two values for intermediate levels of in-plane bending moment. The intention of the LAT5 and LAT6 tests was to provide an understanding of the relationship between benefits due to tipping effects and decreased lateral stiffness associated with in-plane loading.

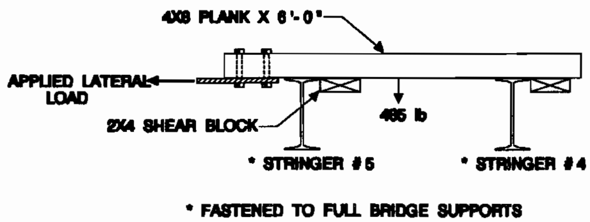


Figure 3.17 Test setup for LAT5.

For the LAT5 test, the 4x8 plank used in LAT4 was modified to include 2x4 shear blocks installed to bear against the top flange of both stringers #5 and #4, as shown in Figure 3.17. The shear blocks were necessary to transfer the lateral loads to the top flange of the stringers in the absence of sufficient friction. A 485-lb. dead weight was applied to the plank at the midpoint between the stringers. Dial gages were located at the top and bottom flanges of both stringers at midspan to measure the lateral deflection of the centroid and rotation of the cross section. A dial gage was located at the plank to measure the lateral deflection of the system. Inclinometers were placed over the support at the South end of stringer #4 and over the support at the North end of both stringers to measure any lateral rotation.

The lateral deflections of stringer #5 were 8% larger than those of stringer #4 because of movements in the plank and shear blocks. The lateral deflections of both stringers were added together to reflect the response of a single stringer to equivalent applied lateral loads. The results of the LAT5 test representing the equivalent response of a single stringer are presented in Figure 3.18 with the results from the previous preliminary tests. Only a slight increase in lateral stiffness from that of the LAT2 and LAT3 tests was observed.

3.2.5.7 Test LAT6. The LAT6 test, shown in Figure 3.19, was identical to the LAT5 test except that no dead load was applied to the plank. The loading and instrumentation were identical to the LAT5 test.

The lateral deflections of stringer #5 were 1% greater than those of stringer #4. The lateral deflections of both stringers were added to reflect the response of a single stringer to the equivalent applied lateral loads. These results are presented in Figure 3.20 with the results from all other lateral tests on stringers. No appreciable difference in lateral stiffness between LAT5 and LAT6 was observed. While both curves are fairly linear, the LAT6 curve is slightly more linear than LAT5. The increase in nonlinear behavior of the LAT5 test may be due in part to the P-Delta effect caused by the higher axial force in the compression flange from the applied plank loading.

While the LAT5 and LAT6 tests both demonstrated a small increase in lateral stiffness, it was not possible, based on the test results, to separate the benefits due to tipping effects from the losses due to in-plane loading. The stringers in LAT5 were expected to exhibit a decreased stiffness due to the vertical dead weight applied to the plank. It appears that the expected loss of stiffness due to the in-plane applied load was offset by the

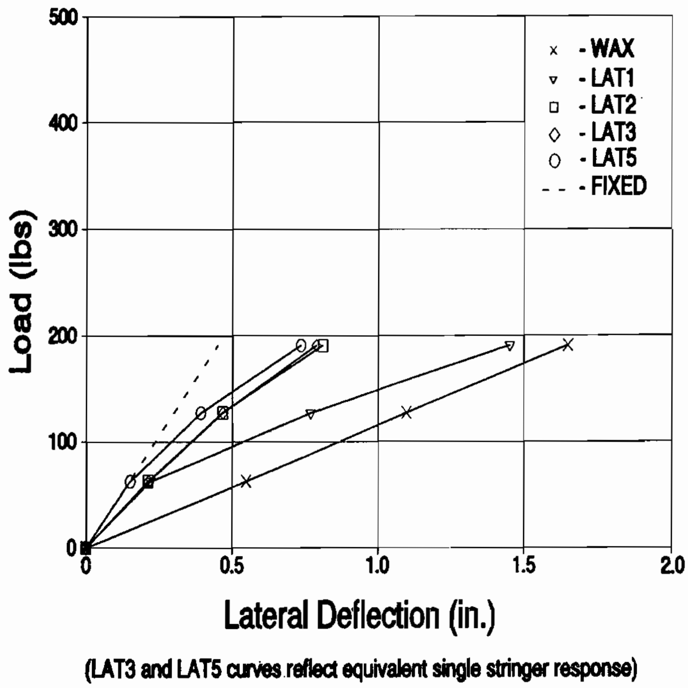


Figure 3.18 Lateral stiffness results for Test LAT5.

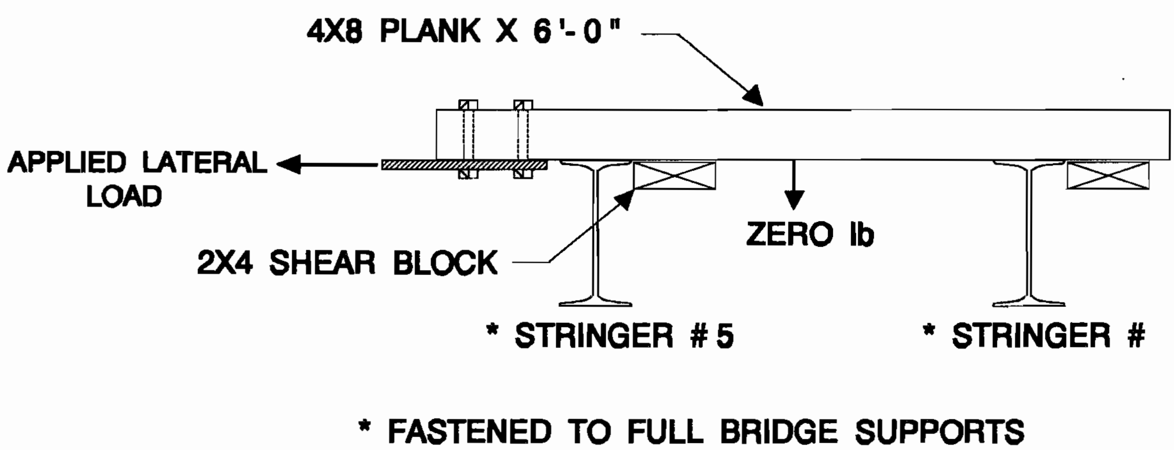


Figure 3.19 Test setup for LAT6.

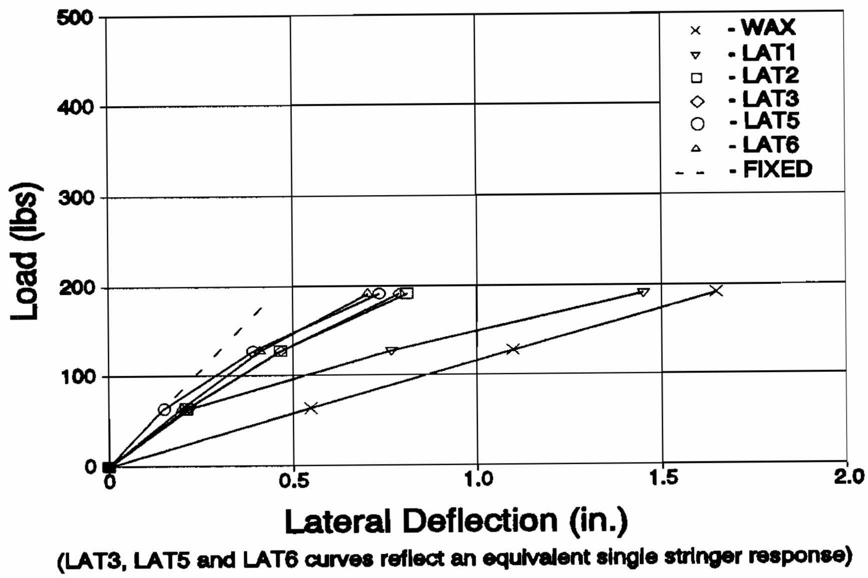


Figure 3.20 Lateral stiffness results for Test LAT6.

benefits from the tipping effects for the LAT5 test. This response may be specific to this test setup and the particular "S" shape used. Further testing would be required to describe the relationship between tipping effects and in-plane loading and to allow prediction of behavior with respect to lateral stiffness.

3.3 Full System Tests

3.3.1 Overview. After completion of the preliminary tests, the bridge deck was installed and testing of the full bridge system was begun. The purpose of this testing phase was to determine the lateral stiffness provided by the full bridge system. Figure 3.21 shows the spliced 2x6 nailers and 4x8 planks of the finished full system deck. Figure 3.22 shows the S6x12.5 stringers and full system supports. Additional information is provided in Figures 3.1 through 3.3. Tests were performed before and after the bridge was loaded to failure (Vegnesa, 1991). Tests LAT7 through LAT9 were completed prior to Vegnesa's tests and henceforth will be referred to as "Before Truck Loading" tests. LAT10 through LAT12 were conducted after the full system was loaded to failure. For clarity, the latter three tests are referred to as "After Truck Loading" in related figures. Table 3.5 presents a summary of the full system tests.

The loading system used for the preliminary tests was modified for the full system tests. The roller, cable and dead weights were replaced with an hydraulic ram and yoke. The accuracy of the hydraulic ram was +/- 25 psi, which translates approximately to 60 lbs of applied load. Photographs of the modified loading frame

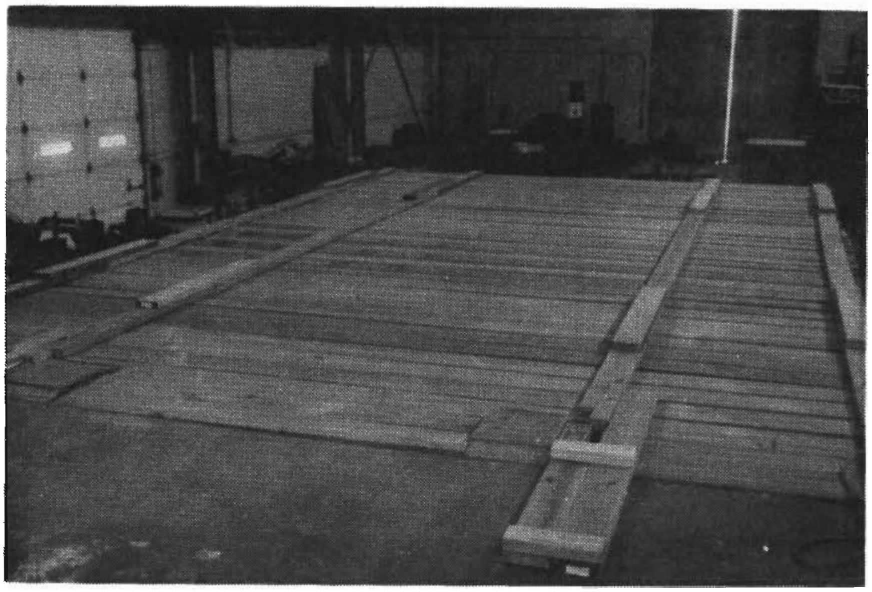


Figure 3.21 Full system bridge deck.

Figure 3.22 Full system stringers and supports.

Table 3.5 Summary of Full System Phase Tests

Full System Test	Test Description
LAT7	Lateral stiffness test with deck weight only. (Before Truck Loading)
LAT8	Lateral stiffness test with deck weight plus concentrated dead loads over each stringer at midspan (Before Truck Loading)
LAT9	Lateral stiffness test with deck weight plus concentrated dead loads over the three interior stringers at midspan. (Before Truck Loading)
LAT10	Lateral stiffness test with deck weight only. (After Truck Loading)
LAT11	Lateral stiffness test with deck weight plus concentrated dead loads over each stringer at midspan (After Truck Loading)
LAT12	Lateral stiffness test with deck weight plus concentrated dead loads over the three interior stringers at midspan. (After Truck Loading)

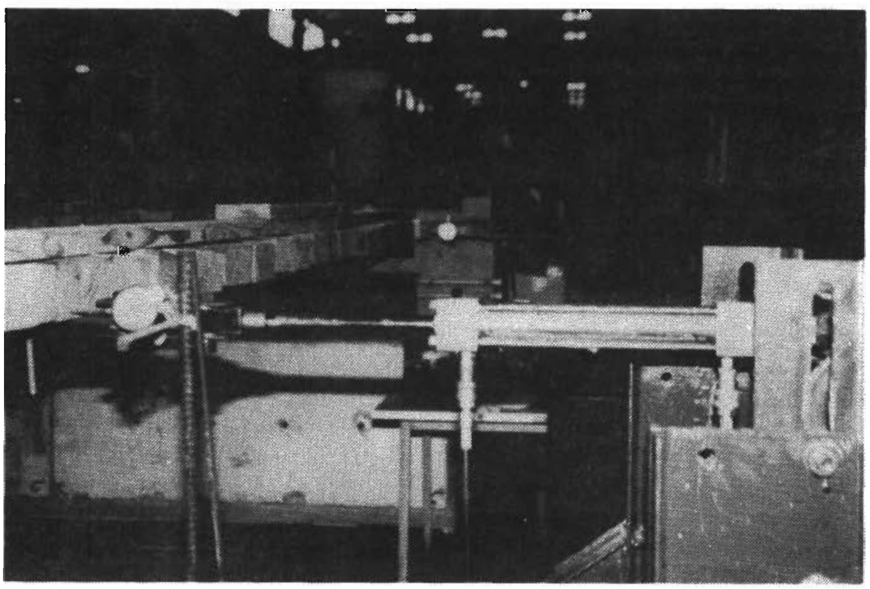


Figure 3.23 Full system loading frame and hydraulic ram.

with the ram are provided in Figure 3.23. Lateral loads were applied to the full system by means of a bolted connection to the midspan 4x8 plank and 2x6 deck nailer #4. The midspan plank and deck nailer were attached to the ram with the yoke shown in Figure 3.24. The ram and yoke system utilized pin connections to remove any rotational or translational restraint. Without positive mechanical attachment, the applied lateral load was transferred from the deck to the supporting stringers through friction alone.

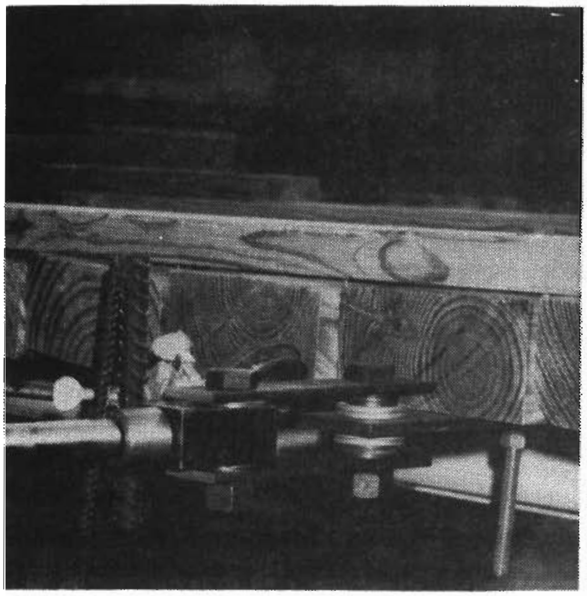


Figure 3.24 Full system loading yoke.

Dial gages were located against the deck at each support to determine any lateral rigid body motion. A dial gage was located against the midspan plank to monitor the lateral midspan deflection of the system. All dial gages had an accuracy of 0.001 in. Linear potentiometers with a similar accuracy were used to measure the relative slip between the deck and the top flange of all five stringers at quarter point and midspan locations.

3.3.2 Tests Conducted Before Truck Loading. Tests LAT7 through LAT9 were performed on the full bridge system before the full system was loaded to failure with a truck (Vegnesa, 1991). The purpose of these tests was to determine the lateral stiffness of the full system.

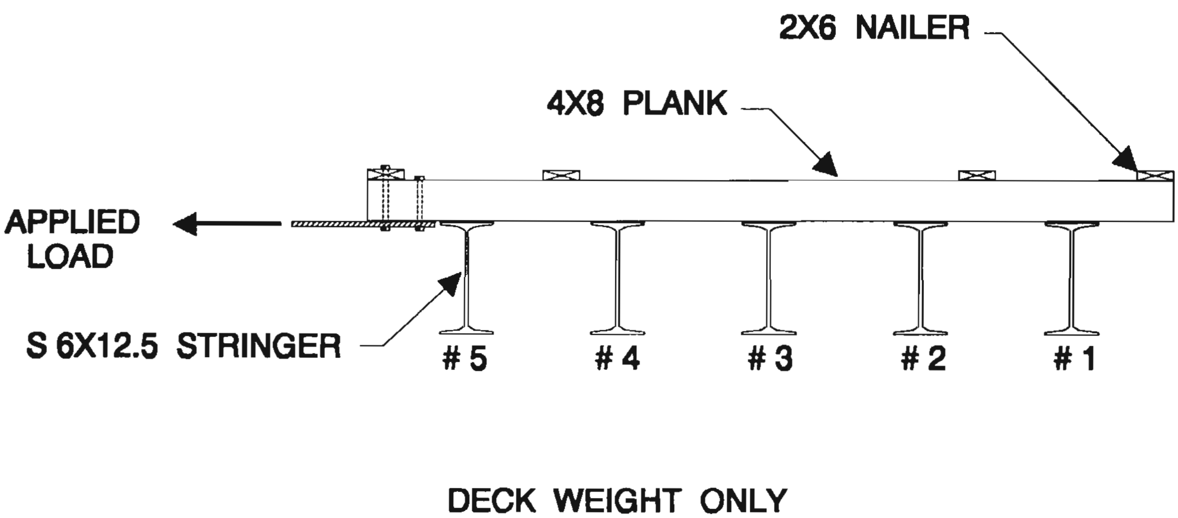
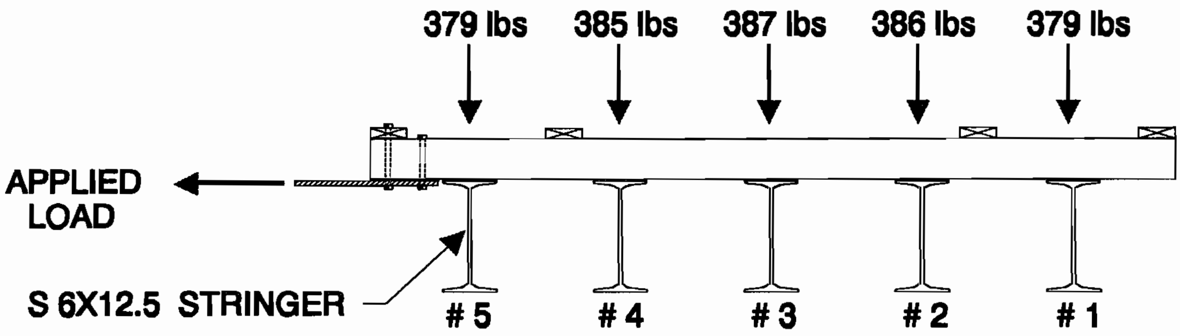


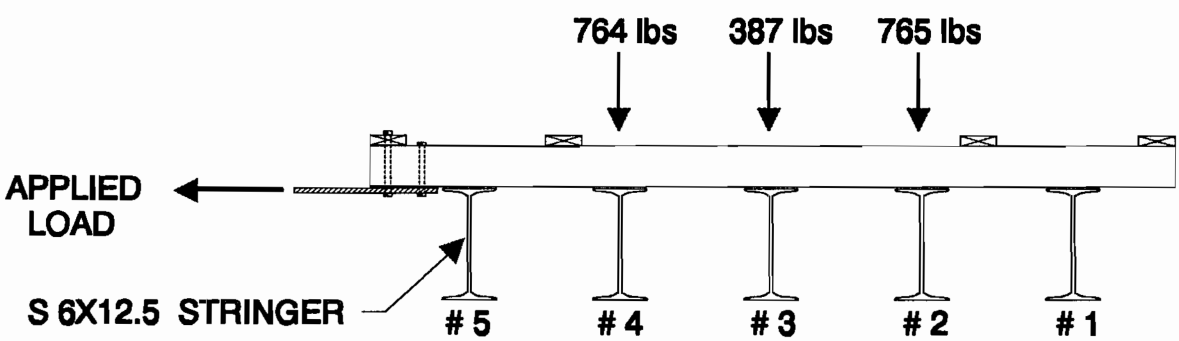
Figure 3.25 Test setup for LAT7

For the full system test LAT7 shown in Figure 3.25, only the dead weight of the deck was supported by the stringers. Lateral loads were applied to the deck at midspan and dial gages were used to measure the lateral deflection of the deck at each support and at midspan. Linear potentiometers measured the relative slip between the deck and top flange of each stringer at quarter point an midspan locations.



DECK WEIGHT PLUS MIDSPAN CONC. LOADS SHOWN

Figure 3.26 Test setup for LAT8.



DECK WEIGHT PLUS MIDSPAN CONC. LOADS SHOWN

Figure 3.27 Test setup for LAT9

Figures 3.26 and 3.27 present the test setups for full system tests LAT8 and LAT9. These tests were identical to the LAT7 test except that concentrated dead loads were applied at midspan as shown in the respective figures. The concrete blocks used to apply the concentrated loads contacted the three 4x8 planks at midspan of the stringers. These tests were conducted to determine any effect on lateral stiffness due to the additional in-plane bending moment applied to the stringers or the improved contact between the deck and stringers.

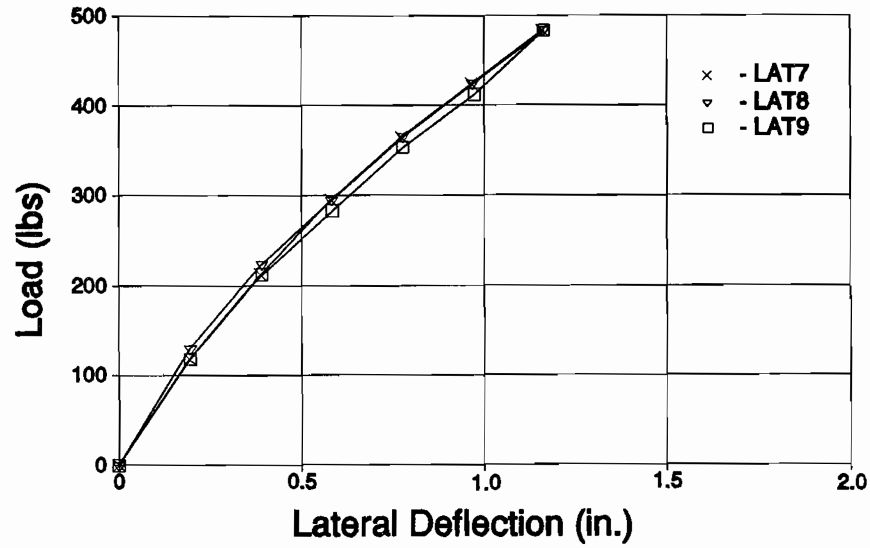


Figure 3.28 BTL full system stiffness (equiv. single stringer response).

The full system lateral stiffness results for LAT7 through LAT9 are presented in Figure 3.28. The recorded load values have been divided by five so that the stiffness curves reflect an equivalent single stringer response to the applied lateral loads for comparison. There was no significant difference in the full system results of these three tests, as shown by the figure. The slight nonlinearity of these curves is probably due to the inconsistent behavior among the individual nailer/plank connections. The small difference in load values for the LAT9 test compared with those of LAT7 and LAT8 is beyond the accuracy of the instrumentation.

The stiffness values for each curve as determined by linear regression analysis are presented in Table 3.6. From these tests, the lateral stiffness of the full bridge system before truck loading was 2,250 lb/in. or five times the equivalent single stringer response of 450 lb/in. for the LAT7 test. This represents an 80% increase over the single stringer fastened to the full system supports without the deck installed.

Table 3.6 Before truck loading stiffness summary.

Full System Test	LAT7	LAT8	LAT9
*Lateral Stiffness (lb/in.)	450	450	439

For each load step the relative slip between the stringers and deck was recorded. The relative midspan slip of all five stringers was determined for each increment of applied lateral load. The average

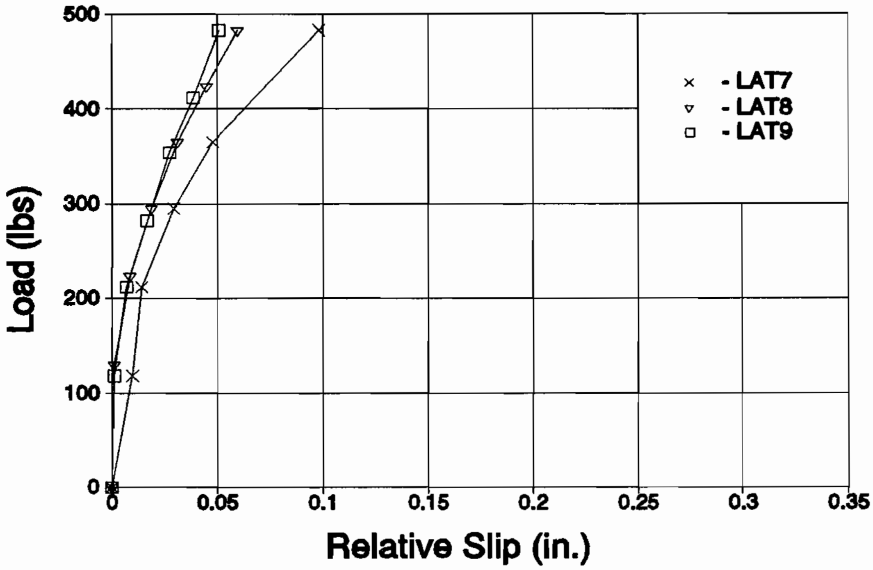


Figure 3.29 Average relative stringer slip for full system tests.

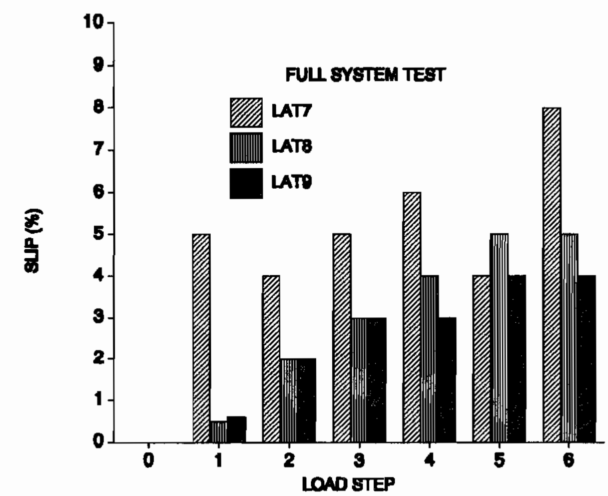


Figure 3.30 Average relative slip as a percent of system displacement.

relative midspan slip results for LAT7 through LAT9 are presented in Figure 3.29. The figure shows that the relative slip was reduced for LAT8 and LAT9. The improved behavior can be attributed to the additional friction between the deck and stringers due to the vertical concentrated loads applied at midspan. The total deck dead weight was 4,000 lbs (Vegnesa 1991). The uniformly applied deck weight is equivalent to a 400-lb concentrated midspan load per stringer. The vertical loads applied externally were approximately 380-lb, which was an 95% increase. The average lateral deflection of a single stringer expressed as a percentage of the lateral deflection of the full system is provided in Figure 3.30. This figure shows that the losses due to slip were greatest for the LAT7 test, which was prior to the addition of concentrated dead loads at midspan. At a lateral load of 483-lbs, the measured slip was approximately 0.1" for the LAT7 test. Since the full system lateral deflection was 1.2" at this load level, the slip constituted about 8% of the total displacement. A 1.2" lateral deflection corresponds to approximately L/240. LAT7 and LAT8 slip values were uniform over the full range of load increments. The difference in slip was not significant with an 8% maximum for LAT7, as compared to a 5% maximum for LAT8 and LAT9.

3.3.3 Tests Conducted After Truck Loading. Tests LAT10 through LAT12 were identical to the corresponding LAT7 through LAT9 tests with the exception that the full system had been loaded to failure (Vegnesa 1991). In Vegnesa's tests, successive loads were applied to the full bridge system through a truck until failure by lateral torsional buckling of the stringers was observed. The loading process damaged the deck by causing a separation, represented in Figure 3.31, between the interior two nailers and planks over half of the bridge span. The lateral loading and instrumentation was identical to that of the Before Truck Loading tests. The test setups for LAT11 and LAT12 are presented in Figures 3.32 and 3.33, respectively. The LAT10 test was identical to the LAT7 test shown in Figure 3.25.

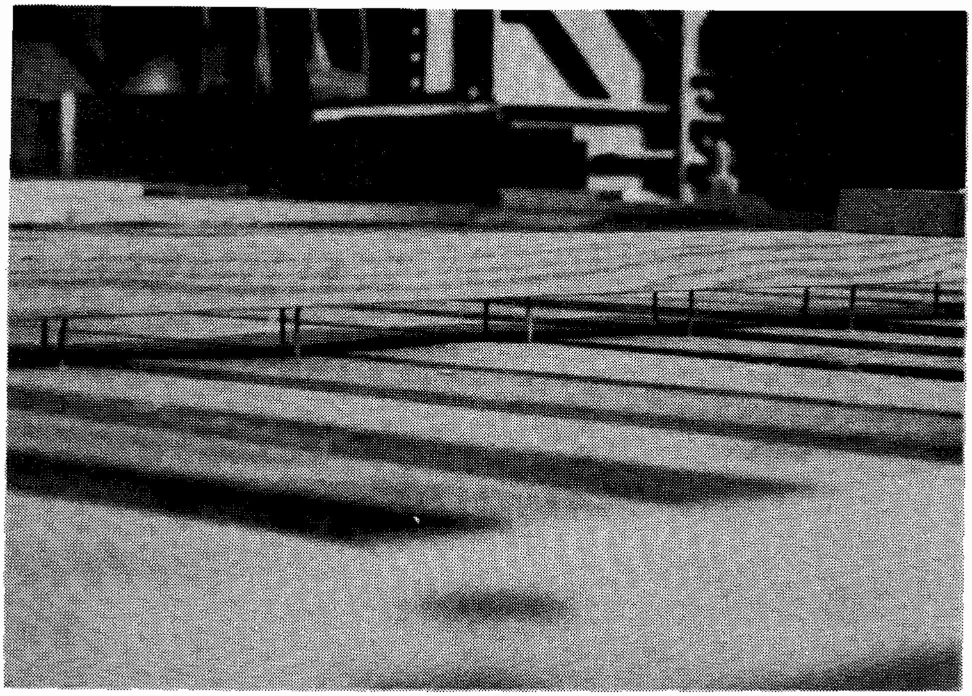


Figure 3.31 Deck damaged by truck loading process.

The full system lateral stiffness results for LAT10 through LAT12 are presented in Figure 3.34, along with an average curve representing LAT7 through LAT9. All curves reflect equivalent single stringer response to the applied lateral loads. As shown in the figure, no significant difference in stiffness between the After Truck Loading tests was observed. The dashed line represents the average of the Before Truck Loading tests. Although the deck sustained damage due to the truck loading process, the lateral stiffness was not reduced significantly.

The stiffness values for each curve as determined by linear regression analysis are presented in Table 3.7. Linear regression values are presented in order to give numerical approximation to the nonlinear curves. From these tests, the lateral stiffness of the full bridge system before truck loading was established to be 2,250 lb/in. or five times the equivalent single stringer response of the LAT7 test.

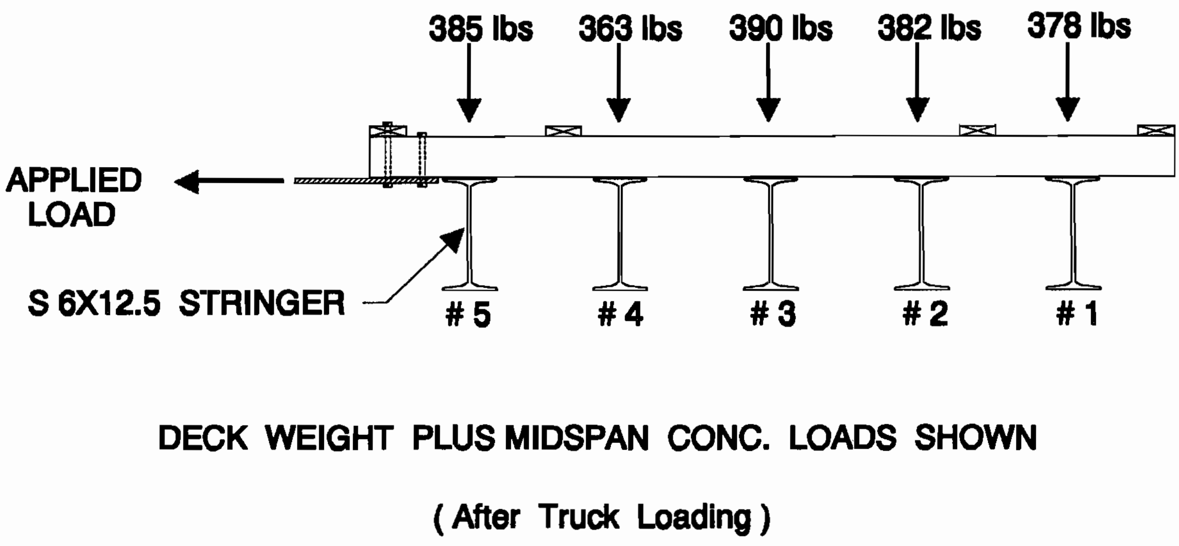


Figure 3.32 Test setup for LAT11.

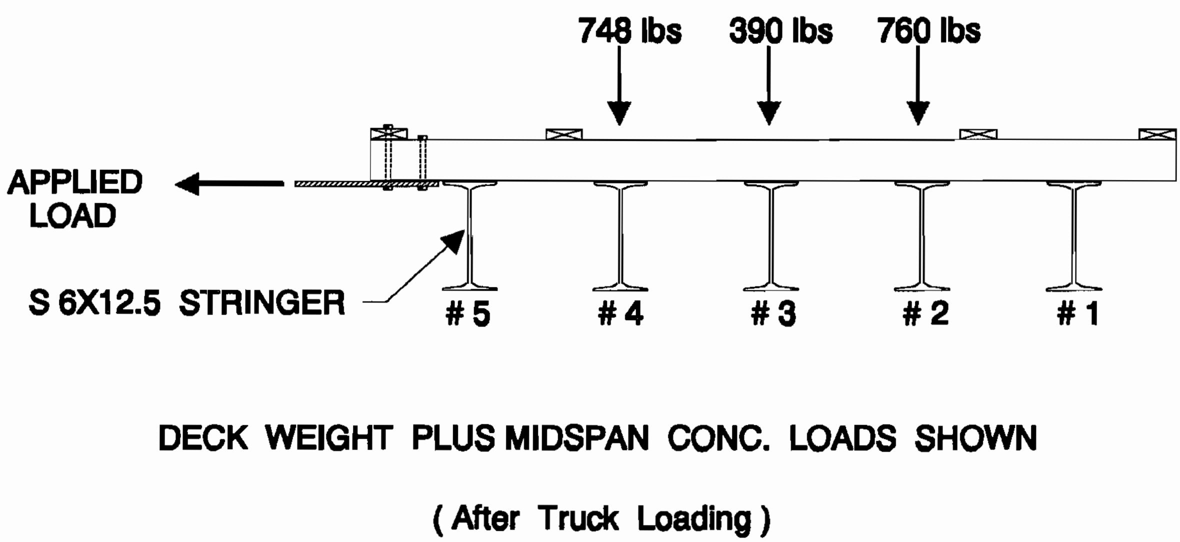
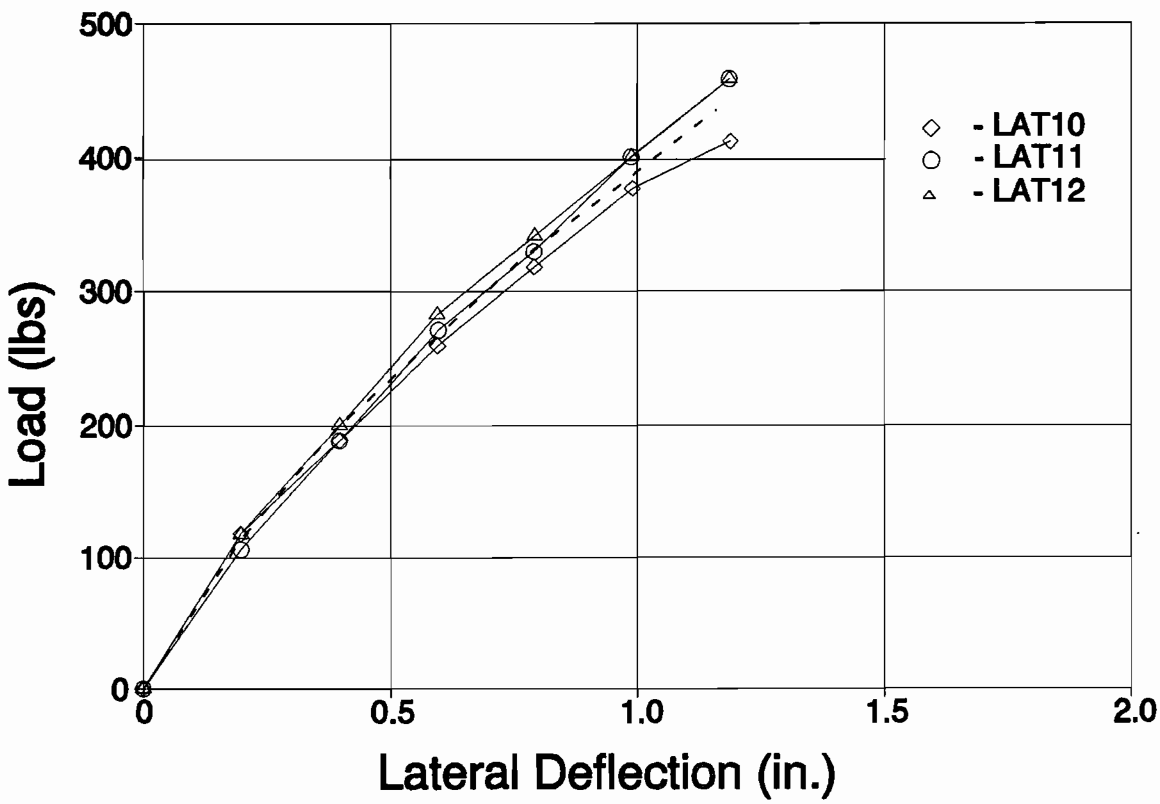


Figure 3.33 Test setup for LAT12.

Table 3.7 After Truck Loading Stiffness Summary

Full System Test	LAT10	LAT11	LAT12
*Lateral Stiffness (lb/in.)	383	410	416



Note: Dashed line represents the average of tests LAT7 through LAT9.

Figure 3.34 ATL full system stiffness (equiv. single stringer response).

The relative slip between the stringers and deck was recorded for each load step. The relative midspan slip of all five stringers was determined for each increment of applied lateral load. The average relative midspan slip results for After Truck Loading tests LAT10 through LAT12 are presented in Figure 3.35. The figure shows that the relative slip was significantly greater for LAT10. The slip for the LAT10 test was increased because of the damage sustained by the deck during truck loading, resulting in a loss of contact with the supporting stringers. The improved behavior of the LAT11 and LAT12 tests can be attributed to the improved contact between the deck and stringers provided by the vertical concentrated loads applied at midspan with concrete blocks. The average lateral deflection for a single stringer expressed as a percentage of the lateral deflection of the full system is provided in Figure 3.36. This figure shows that the losses due to slip were greatest for the LAT10 test. LAT11 and LAT12 slips were fairly uniform over the full range of load increments, with LAT11 having the least amount of slip in general.

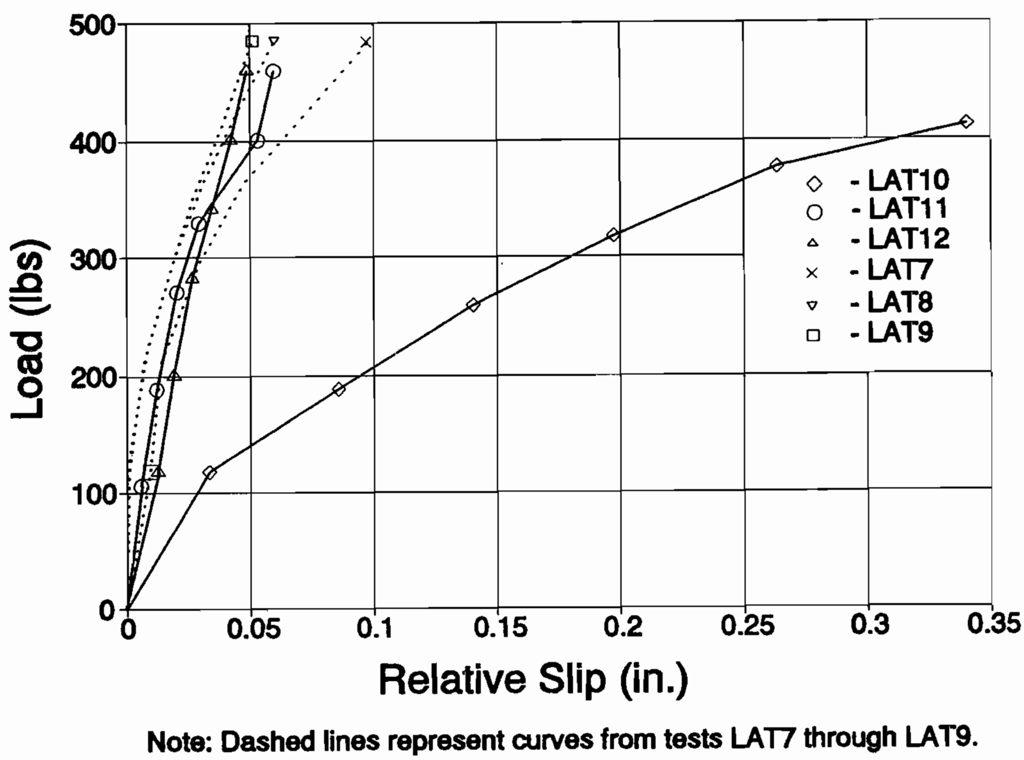
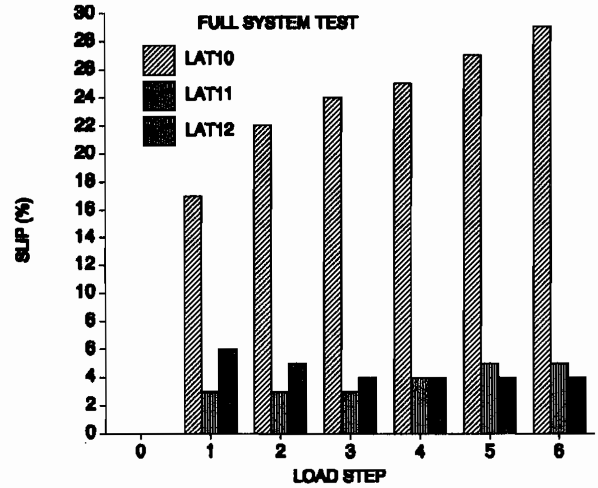


Figure 3.35 Average relative stringer slip (after truck loading).

The difference in slip was significant for the LAT10 test. When midspan loads were applied, which represents the actual loading situation, the slip was reduced to a level essentially equivalent to that of Before Truck Loading tests.



Figue 3.36 Average relative slip as a percent of system displacement.

3.4 Summary of Test Results

The purpose of the preliminary test phase was to establish values for the lateral stiffness of various components of the full bridge system such that the contribution of the bridge deck could be taken as the difference between those values and the lateral stiffness of the full system as determined through subsequent testing.

Table 3.8 lists the single stringer lateral stiffness values as determined through linear regression analysis for all lateral stiffness tests on the stringers without the deck installed. A calculated value corresponding to a fixed end condition has been provided for comparison. From a review of these values, the representative lateral stiffness of a single stringer is 250 lb/in. Table 3.9 lists the component stiffness values taken from the results of the preliminary tests and used in determining the lateral stiffness of the bridge deck itself.

Table 3.8 Summary of stringer lateral stiffness values.

Lateral Stiffness Test	WAX	LAT1	LAT2	*LAT3	*LAT5	*LAT6	FIXED END
Lateral Stiffness (lbs/in.)	108	142	248	253	279	284	424

*Values represent an equivalent single stringer response.

Table 3.9 Summary of Preliminary Test Values for Use in Deck Analysis

Component	Spliced 2x6 Deck Nailer	Nailer/Plank Connection	S6x12.5 Bridge Stringer
Stiffness	67.5 lbs/in.	119 kip-in./rad	250 lbs/in.

The purpose of the full system test phase was to establish the lateral stiffness of the full bridge system. Test LAT7 was determined to be representative and the system stiffness was established as 2,250 lb/in. through linear regression analysis. The lateral stiffness of the bridge deck itself is discussed in Chapter 4.

CHAPTER 4

LATERAL STIFFNESS OF TESTED BRIDGE DECK

4.1 Bridge Deck Stiffness Determined Experimentally

The lateral stiffness of the bridge deck was determined from the results of tests performed in the preliminary and full system test phases. From the preliminary test phase, the lateral stiffness of a single S6x12.5 stringer bolted to the full system supports was established. The lateral stiffness of the full bridge system in terms of an equivalent single stringer response was determined before and after truck loading by the full system test phase. Using slip data, the average relative lateral displacement of the stringers at each load step was determined. The force required to displace the stringers was calculated using the single stringer lateral stiffness value of 250 lb/in. previously established. The lateral load supported by the bridge deck was calculated as the difference between the total applied load and that required to displace the stringers. Figure 4.1 presents the

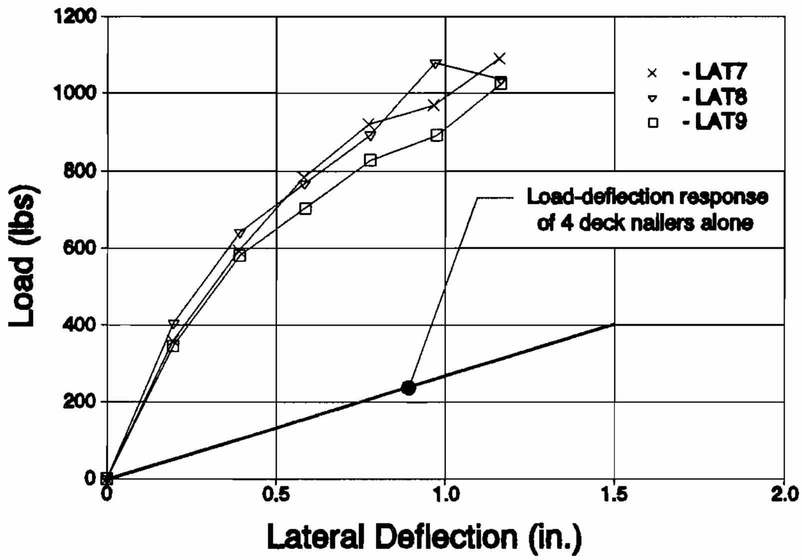


Figure 4.1 Lateral stiffness of bridge deck before truck loading.

lateral stiffness curves for the deck before truck loading (LAT7 through LAT9). The non-linear load-displacement response of the deck is probably due to the friction between the nailers and planks at the nailed connections. In addition, the rotational restraint provided by each nailer/plank connection varies with gap width, as discussed in Chapter 3. All three curves are fairly uniform through the first five data points and then exhibit increased scatter for the final two data points. The increased scatter is associated with the greater slip activity which occurred at the higher lateral loads.

Figure 4.2 presents the lateral stiffness curves for the deck after truck loading (LAT10 through LAT12). A dashed line representing the average of the LAT7 through LAT9 curves of Figure 4.1 is provided for

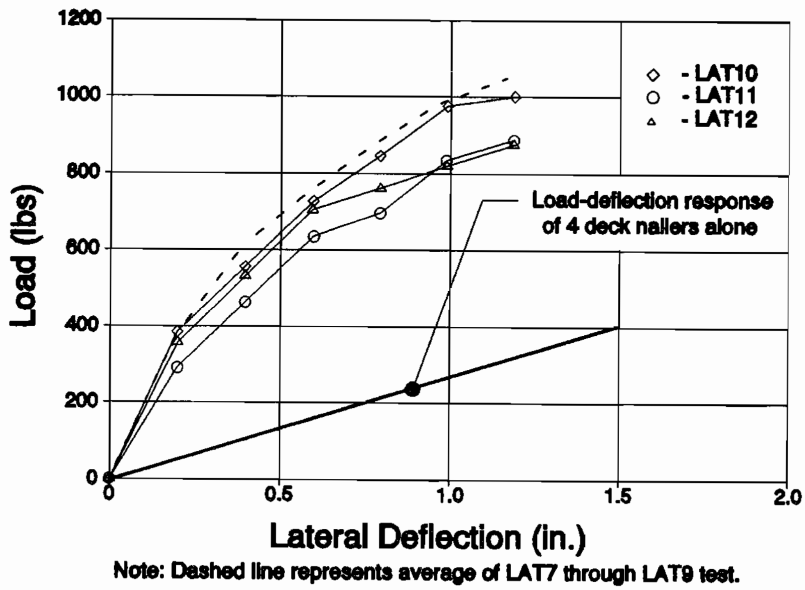


Figure 4.2 Lateral stiffness of bridge deck after truck loading.

comparison. The truck loading process damaged the bridge deck by separating the interior two nailers from the planks over approximately half the span. This damage resulted in reduced stiffness due to a loss of rotational restraint at the separated connections. A photograph of the damaged deck is presented in Figure 3.31. While the LAT10 curve closely follows the average of the tests before truck loading, the LAT11 and LAT12 curves indicate slightly lower stiffness.

For comparison, the load-deflection response for the sum of the four nailers as determined in the preliminary test phase is provided in Figures 4.1 and 4.2. The lateral stiffness of the bridge deck could be conservatively taken as that of the nailers alone without any consideration of the rotational restraint provided by the nailer/plank connections. The figures show that the actual lateral stiffness of the deck is approximately three times that provided by the four deck nailers alone, indicating that such an approach may be too conservative.

4.2 Bridge Deck Stiffness Determined by Computer Analysis

The full system bridge deck was modeled using a commercial finite element software package (SAP90). The 3-dimensional model of the deck consisted of thirty-seven 4x8 plank members connected together with four 2x6 nailers. The section properties of the planks and nailers were based on nominal cross-section dimensions (3.5" x 7.5" for the 4x8 plank; 1.5" x 5.5" for the 2x6 nailers). The modulus of elasticity of the planks was taken as the published value of 1600 ksi (National Design Specification, 1986). The nailer modulus of elasticity was adjusted to 1615 ksi in order to simulate the tested average lateral stiffness of 67.5 lb/in. established in the preliminary test phase. This adjustment results in a 1% increase in stiffness over a value calculated with a continuous nailer model and the published modulus of elasticity. The rotational restraint provided by the nails used to fasten the nailers to the planks was modeled by a member connecting the two with a torsional rigidity, (GJ/L), equal to the tested value of 119 kip-in./rad. Constraints were imposed on these torsional members to prevent relative translational displacements between the ends.

Two models were analyzed; one representing the deck before truck loading and another modeling the damaged deck after truck loading. The Before Truck Loading deck model is described in the previous paragraph. During the full scale bridge test (Vegnesa 1991), the two interior deck nailers pulled away from the planks from one end of the deck to about midspan. The After Truck Loading deck was modeled by adjustment of the Before Truck Loading model to account for the damaged deck. This modification was accomplished by setting the torsional rigidity to zero for 60% of the connecting members along the two interior nailers starting from one end of the deck.

Both models were loaded with a lateral force at midspan. In order to be consistent with the load-deflection response of the full system tests, the SAP90 deck stiffness was calculated for each model by dividing the applied load by the maximum midspan deflection. Figure 4.3 presents the SAP90 lateral stiffness results for

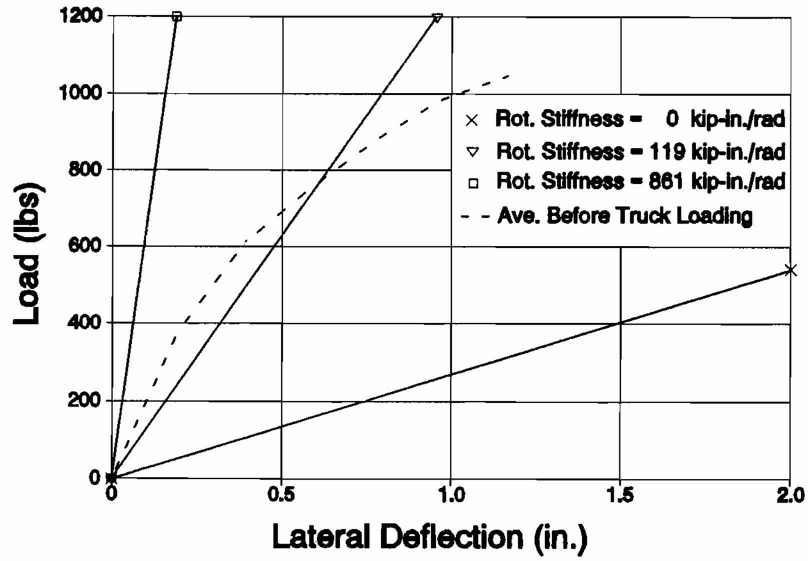


Figure 4.3 Lateral stiffness of bridge deck computed with SAP90.

the deck with various nailer/plank connection rotational stiffness values. The zero rotational stiffness curve represents the response of the deck without any connection between the deck nailers and the planks. This curve corresponds to a stiffness of 270 lb/in. which is equal to the sum of the lateral stiffnesses of the nailers alone, as expected. The 861 kip-in./rad curve represents the ideal condition of full contact between the nailers and planks, such that the rotational stiffness includes the contribution from friction between the members. The corresponding stiffness is 6,440 lb/in., which is an increase of 24 times the stiffness of the nailers alone. The 119 kip-in./rad curve reflects the response of the bridge deck with a rotational restraint provided by the nailer plank connection, which is most representative of actual conditions before truck loading. This corresponds to a stiffness of 1260 lb/in., which is about five times the stiffness of the four nailers alone. A comparison of the SAP90 stiffness representing actual conditions before truck loading with the average of the BTL full system tests (LAT7 through LAT9) shows that the SAP90 analysis provided a good indication of the initial lateral stiffness for the bridge deck. The SAP90 stiffness is conservative for lateral loads less than 800 lbs.

4.3 Discussion

The lateral stiffness of the bridge deck system as determined by computer analysis using SAP90 was 1,260 lb/in. for the deck as a whole and 252 lb/in. for a single stringer. The SAP90 stiffness must be interpreted with an understanding of the effect of the nailer/plank connection rotational restraint on the overall lateral stiffness of the deck. Figure 4.4 presents the relationship between the SAP90 lateral stiffness values for the deck and rotational stiffness of the nailer/plank connection. At a rotational stiffness of zero, the deck stiffness is equal to the sum of the lateral stiffness values for the four spliced 2x6 nailers. The curve is fairly linear with the deck stiffness increasing sharply from this lower bound for increasing rotational stiffness of the nailer/plank connection. The deck system is very sensitive to increases in the rotational stiffness of the connection. An increase of 10 kip-in./rad in rotational stiffness results in about a 75 lb/in. increase in lateral stiffness for the bridge deck.

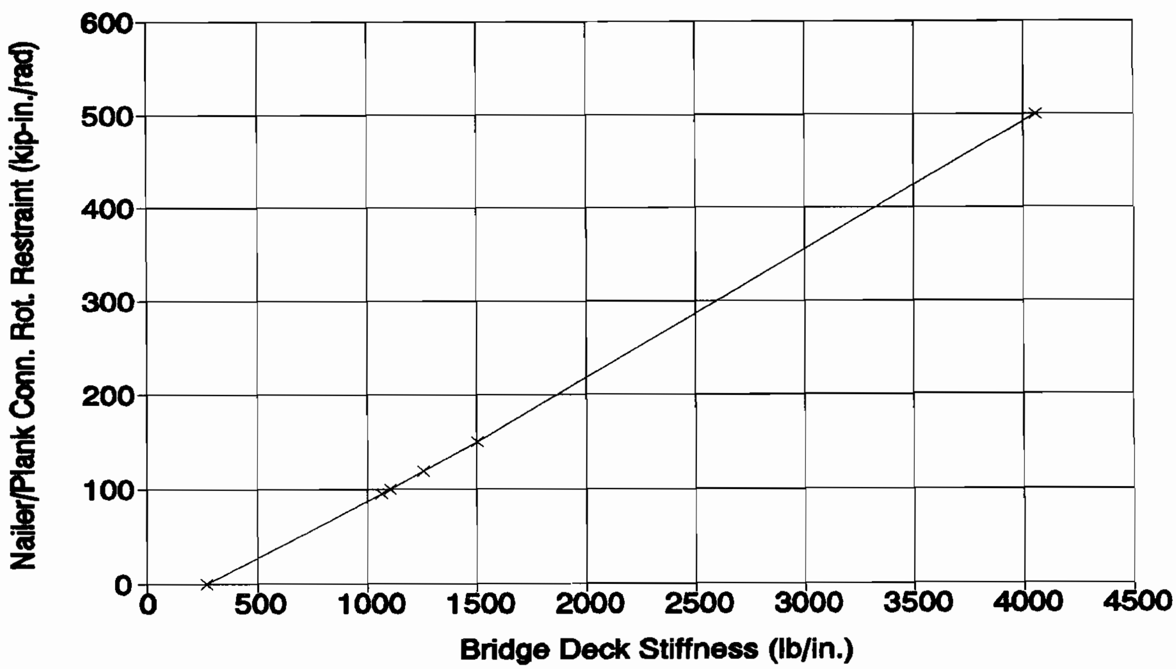


Figure 4.4 The effect of rotational restraint on SAP90 deck stiffness.

CHAPTER 5

EVALUATION OF BRACING PROVIDED BY BRIDGE DECKS

5.1 General

A bridge deck may function as a lateral, torsional, or combination lateral and torsional brace for supporting steel stringers. Deck bracing characteristics depend on the structural arrangement of the deck and its connection to the stringers. Many off-system short span bridges do not have any positive connection provided between the stringers and the deck. Without positive connection, the bridge system must rely on friction between the deck and the top flanges of the stringers to mobilize the lateral bracing characteristics of the deck. The torsional bracing effects provided by the flexural rigidity of the deck as a whole cannot be relied on without positive connection because the top flanges of the stringers may lose partial contact with the deck under loading, as shown by Vegnesa (1991). Only that portion of the deck in direct contact with wheel loads will maintain contact with the supporting stringers. Without full contact, only the torsional restraint provided by tipping effects remains.

This investigation focused on the evaluation of the lateral bracing characteristics of bridge decks. Lateral loads were applied to the deck of a full scale bridge and friction alone was relied on to transfer the applied loads to the supporting steel stringers. This process is actually reversed in the real situation. Stringers tend to buckle and displace laterally when the in-plane load level approaches the critical buckling load. As the stringers deflect laterally, the deck is engaged and resists the displacement through friction and or shear connection between the top flanges of the stringers and the deck. In the full scale tests, at loads approaching the buckling load, friction between the timber deck and the top flanges of the stringers at the location of the truck wheel loads was sufficient to prevent significant slip and to transfer the lateral loads induced by the stringers to the deck. This timber deck was designed to have a minimal lateral stiffness as a worst case system. Failure of the bridge occurred at a load of 18.0 kips as a result of lateral torsional buckling of all the S6x12.5 stringers. The actual failure load was more than twice the capacity of the stringers without any bracing provided. This indicated that there was sufficient friction to engage the deck as a lateral brace at the location of the wheel loads.

The bridge deck must possess some minimum strength and stiffness in order to function as a brace. In the following two sections, the strength and stiffness requirements for lateral braces will be summarized.

5.2 Strength Requirements for Lateral Braces

Winter (1960) presented a formulation for the required strength or brace force, F_{br}, for a column with a lateral brace at midheight:

$$F_{br} = \frac{2P_e}{L_b}(\Delta_o + \Delta) \tag{5.1}$$

where P_e = Euler buckling load between braces, L_b = spacing between braces, Δ_o = initial out-of-straightness, and Δ = additional deflection permitted before the column fails. Yura simplified this formulation and found that F_{br} = 0.008P was conservative for all cases, where P is the load applied to the column. This simplified formulation may be used to determine required brace strength for applied load levels less than that necessary to cause buckling in between brace points. Yura's development is presented in the study of structural steel design by Marcus (1977). While the simplified formula was developed for columns, it can be used for beams by replacing the load P with M/h, where M is the applied moment and h is the distance between the centroids of the top and bottom flanges. For a bridge, the maximum moment in the stringers is related to the axle load; the total lateral brace force may be calculated as follows,

$$TOTAL\ LATERAL\ BRACE\ FORCE,\ F_{br} = 0.008\frac{P_{axle}L}{4h} \tag{5.2}$$

The required coefficient of friction may be determined by dividing the total lateral brace force by the corresponding axle load. Stringer depth is usually within 5 percent of the magnitude of h. It follows that for a span to depth ratio (L/d) of 40, a friction coefficient of 0.08 is required. For a span to depth ratio of 20, a friction coefficient of 0.04 is required. These values are much less than the 0.25 coefficient of friction between the timber plank and steel stringer measured as part of the preliminary test phase.

Friction is much greater for concrete decks in contact with steel stringers. Chemical bonding and mechanical interlocking between the concrete and steel improve lateral load transfer. Kissane (1985) concluded from a full scale test of a concrete deck supported by steel stringers that friction alone was sufficient to engage the deck as a brace and allow the stringers to reach full in-plane bending capacity without buckling laterally. As shown above, friction requirements for short span bridges are very small. For this type of bridge, sufficient friction exists to mobilize the timber or concrete decks as lateral bracing without the provision of positive connections. If positive connections rather than friction are to be used, brace forces may be determined as outlined above.

The following sections will present useful lateral bracing equations and discuss their application to short span bridges with timber and concrete decks.

5.3 Stiffness Requirements for Lateral Braces

The equations for lateral bracing of beams presented in this section were developed by J. A. Yura (1990). The application of these equations to short span steel bridges must be based on the assumption that sufficient friction and/or shear connection exists to transfer lateral loads associated with stringer buckling to the bridge deck, allowing the deck to be engaged and function as a lateral brace.

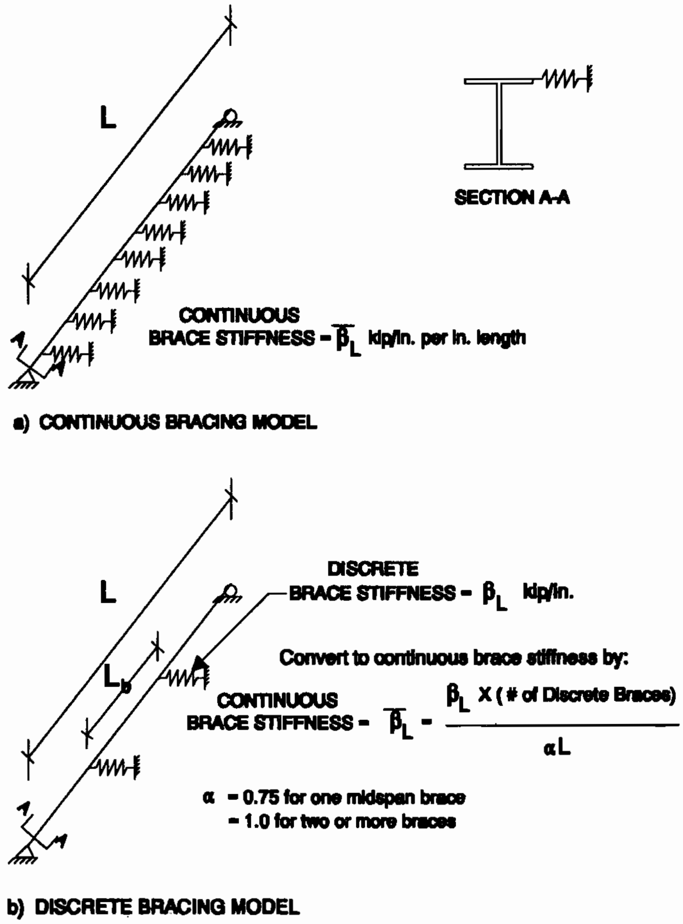


Figure 5.1 Bracing Models for Simply Supported Beams

In addition to serving as a lateral brace, the deck also serves to transfer lateral loads among the stringers, such that the most heavily loaded stringers are braced by those with lighter loading. The lateral stiffness of a short span bridge should be based on the stiffness of the total system, to include contributions from the deck and all of the stringers. Determination of deck stiffness will be discussed in subsequent sections. The stiffness for steel stringers can be obtained using commonly assumed material properties, AISC manual section properties, and loading models appropriate for the deck/stringer connection configuration.

There are two basic bracing models used for the design of lateral braces, the continuous bracing model and discrete bracing model. An example of the continuous bracing model would be a composite concrete deck and steel beam floor system that is in continuous contact along the full length of the beam. A discrete brace is effective only at a few locations, as in the bracing at an axle location provided by a bridge deck. Yura has converted the discrete bracing model into an equivalent continuous bracing model so that only the continuous model is needed for design. Figures 5.1a and 5.1b present the continuous and discrete bracing models for a simply supported beam laterally braced at the top (compression) flange. For the case of a steel stringer continuously braced by a bridge deck, represented in Figure 5.1a, the continuous brace stiffness, $\overline{\beta_L}$, would equal the lateral stiffness of the deck associated with a continuous loading model divided by the stringer span length. In the case of a stringer braced by a deck through attachment at third points, represented in Figure 5.1b, the continuous brace stiffness, $\overline{\beta_L}$, would equal two times the lateral stiffness of the deck associated with a third point loading model divided by the stringer span length. If the stringer is braced through attachment to a deck at midspan only, the equivalent continuous brace stiffness would equal the lateral stiffness of a midspan loaded model divided by 0.75L. The amount of lateral stiffness provided by the deck toward the bracing of each individual stringer is equal to the total deck stiffness divided by the number of supporting stringers.

The general bracing design recommendations developed by Yura are summarized in Appendix B of Yura and Phillips (1992). Those recommendations are applicable for any number of braces within the span and any

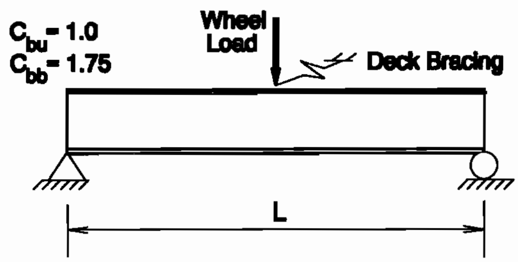


Figure 5.2 Loading Condition for Short Span Bridges

loading condition. For the design or rating of short span bridges, the governing loading condition for flexure is a wheel load at midspan as shown in Figure 5.2. To determine the lateral bracing effect of the deck at midspan, β_L, for this loading condition, the general continuous bracing equations can be simplified to the following for the Load Factor Design method in the AASHTO Specification:

$$M_{cr} = \sqrt{(M_o^2 + 0.766\,P_y^2 h^2 A)(1 + A)} \;\le M_s \;\; or \;\; M_y \tag{5.3}$$

where

$$M_o = \frac{91 \times 10^6}{L}\sqrt{0.772 I_{yc} J} \tag{5.4}$$

$$M_s = \frac{91\times10^6 C_b I_{yc}}{L_b}\sqrt{0.772\frac{J}{I_{yc}} + 9.87\left(\frac{d}{L_b}\right)^2} \quad with \;\; C_b = 1.75 \;,\; L_b = L/2 \tag{5.5}$$

$$M_y = F_y S_x \tag{5.6}$$

$$P_y = \frac{\pi^2 E I_y}{L^2} = \frac{286\times10^6 I_y}{L^2} \tag{5.7}$$

$$A = 0.45\sqrt{\frac{\beta_L L}{P_y}} \tag{5.8}$$

and L = span length, I_y = weak axis moment of inertia, I_{yc} = lateral moment of inertia of the compression flange = $I_y/2$, J = St. Venant's torsional constant, d = beam depth, h = distance between flange centroids, C_b = factor to account for moment gradient between brace points, P_y = Euler buckling load for columns (Eq. 5.7), A = bracing factor (Eq. 5.8), M_s = buckling capacity between brace points, and M_y = yield moment. M_o is the beam buckling strength assuming no bracing and top flange loading and is approximated by using the AASHTO C_b = 1.0 for the case when the moment within the unbraced segment is greater than the end moments and by neglecting the (d/L_b) term in Equation 5.5. The lateral bracing effect of the bridge deck is represented by β_L in Equation 5.8, which includes a reduction factor of 2 for initial out-of-straightness as described in Yura and Phillips (1992).

For a known deck stiffness, the buckling strength may be determined directly from Equation 5.3. If a certain capacity, M_{req}, is required for a given bridge system, the bracing factor, A_{req}, may be determined by substituting M_{req} for M_{cr} in Equation 5.3 and solving for A_{req} as follows:

$$A_{req}^2 + \left[1 + 1.3\left(\frac{M_o}{P_y h}\right)^2\right]A_{req} - \frac{1.3}{P_y^2 h^2}\left(M_{req}^2 - M_o^2\right) = 0 \qquad (5.9)$$

The required bracing effect may be found by solving for the appropriate root of the quadratic. From the required bracing factor, the corresponding midspan brace stiffness may be determined from Equation 5.8 as follows:

$$\beta_{Lreq} = A_{req}^2 \frac{5 P_y}{L} \qquad (5.10)$$

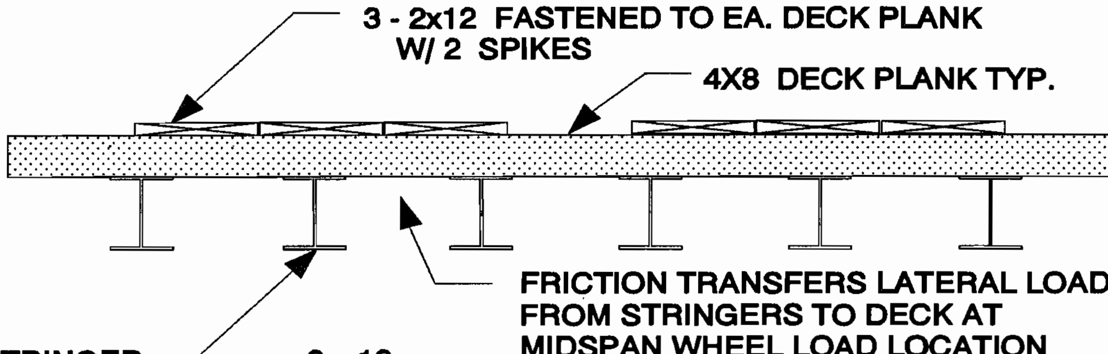
Two bracing example problems follow which the use of the bracing formulas.

5.4 Typical Timber Decks

Construction details among off-system short span bridges with timber decks vary considerably. Some bridges still in service were built in the early 1900's. Upgrading and renovation which has been implemented since the original completion of these bridges does not follow any uniform set of construction details. Variability in construction often makes evaluation a difficult task.

Timber decks have the potential to increase the capacity of a bridge system by functioning as a lateral brace as well as a connecting link between stringers, so that stringers with lighter loading are able to serve as braces for those supporting heavier loads. Lateral stiffness provided by a timber deck is dependent upon specific construction and details. Except for a laminated deck, which may be treated as a diaphragm for stiffness calculation, the lateral bracing contribution of a timber deck is derived from the stiffness of the deck nailers and the rotational restraint provided by nailer/plank connections. Adequate attachment must be provided between the deck nailers and the planks in order to utilize the lateral stiffness of the nailers. When this attachment is made with two or more spikes, additional lateral stiffness is available through rotational restraint. The deck acts as a connecting link between adjacent stringers, regardless of the level of lateral stiffness. The resulting interbracing of the stringers will increase buckling capacity, since wheel loads are not distributed evenly among

BRACING DESIGN EXAMPLE 1



STRINGER

S 10 x 25.4

$S_x = 24.7 \text{ in.}^3$

$I_y = 6.79 \text{ in.}^4$

$I_{yc} = 3.40 \text{ in.}^4$

$J = 0.60 \text{ in.}^4$

$F_y = 36000$ psi

$h = 9.51$ in.

2 x 12

$I_y = 178 \text{ in.}^4$

$E = 1600000$ psi

BRIDGE SPAN = 17 ' - 0 "

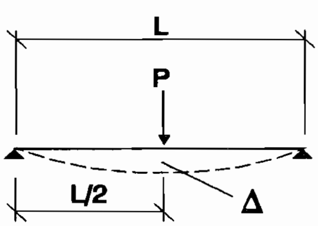
STIFFNESS OF TIMBER DECK:

With each 2x12 fastened to each 4x8 deck plank with 2 spikes and no splice within the middle two thirds of the span, the lateral stiffness of the deck can be conservatively taken as the sum of that for the six 2x12's.

$$I_y = 6\,(178) = 1068 \text{ in.}^4$$

Discrete Brace Stiffness, : β_L

From AISC Manual, Beam Diagram and Formula # 7 :



$$\beta_L = P / \Delta = 48\, E\, I_y / L^3$$

$$= 48\,(1.6\times10^6)(1068) / (204)^3 = 9660 \text{ lb/in.}$$

$$\beta_L \text{ per Stringer} = 9660 / 6 = 1610 \text{ lb/in.}$$

BRACING DESIGN EXAMPLE 1 (CONT.)

BUCKLING STRENGTH :

Unbraced Buckling Capacity,

$$M_o = \frac{91\times10^6}{204}\sqrt{772\,(3.40)\,(0.60)}$$

$$M_o = 559800 \text{ lb - in.}$$

Buckling Capacity With Bracing,

$$P_y = \frac{286\times10^6(6.79)}{(204)^2} = 46700 \text{ lb}$$

$$A = 0.45\sqrt{\frac{1610\,(204)}{46700}} = 1.21$$

$$M_{cr} = \sqrt{\left[(559800)^2 + 0.766\,(46700)^2(9.51)^2(1.21)\right]\,(1 + 1.21)}$$

$$= 1.046\times10^6 \text{ lb-in}$$

Check Capacity for Buckling Between Braces,

$$M_s = \frac{91\times10^6(1.75)(3.40)}{102}\sqrt{0.772\,\frac{0.60}{3.40} + 9.87\left(\frac{10.0}{102}\right)^2}$$

$M_s = 2.55\times10^6$ lb - in. $>$ M_{cr} No buckling between braces

Check Material Strength,

$M_y = 36000\,(24.7) = 889000$ lb - in. $<$ M_{cr} Yielding Controls

BRACING DESIGN EXAMPLE 1 (CONT.)

REQUIRED COEFFICIENT OF FRICTION :

Brace Force per Stringer,

$$F_{br} = 0.008 \; \frac{889000}{9.51} = 748 \text{ lb}$$

Maximum Wheel Load per Stringer,

$$P_{max} = \frac{4\,(889000)}{204} = 17400 \text{ lb}$$

Required Coefficient of Friction,

$$C = \frac{748}{17400} = 0.043 \ll 0.25 \;\; \text{OK}$$

FRICTION IS SUFFICIENT

BRACING DESIGN EXAMPLE 2

Given an S 7x15.3 stringer with a span of 15ft, determine the midspan discrete brace stiffness required to force the stringer to yield before buckling.

S 7 x 15.3 STRINGER PROPERTIES :

$S_x = 10.5 \text{ in.}^3$	$h = 6.61 \text{ in.}$
$I_y = 2.64 \text{ in.}^4$	$I_{y} = 1.32 \text{ in.}^4$
$J = 0.24 \text{ in.}^4$	$F_y = 36 \text{ ksi}$

YIELD MOMENT:

$$M_y = 36000\,(10.5) = 378000 \text{ lb - in.}$$

REQUIRED BRACE STIFFNESS:

Unbraced buckling capacity,

$$M_o = \frac{91 \times 10^6}{180} \sqrt{0.772(1.32)(0.24)}$$

$$= 250000 \text{ lb - in.}$$

Euler buckling load,

$$P_y = \frac{286 \times 10^6\,(2.64)}{(180)^2} = 23300 \text{ lb}$$

Bracing factor,

$$A_{req}^2 + \left[1 + 1.3\left(\frac{250000}{23300\,(6.61)}\right)^2\right] A_{req} - 1.3\,\frac{(378000)^2 - (250000)^2}{(23300)^2 (6.61)^2} = 0$$

$$A_{req} = 0.837$$

$$\beta_{L\,req} = \frac{5\,(.837)^2\,23300}{180} = 454 \text{ lb/in.}$$

BRACING DESIGN EXAMPLE 2 (CONT.)

DETERMINE DISCRETE BRACE FORCE :

$$F_{br} = 0.008\ \frac{378000}{6.61} = 500\ \text{lb}$$

CHECK REQUIRED COEFFICIENT OF FRICTION :

Beam load P corresponding to the yield moment,

$$P = \frac{4\,(378000)}{180} = 8400\ \text{lb}$$

Required coefficient of friction,

$$C_{req} = \frac{500}{8400} = .06 < 0.25\ (\text{Measured Value})\quad \text{OK}$$

the stringers. In summary, timber decks can provide lateral bracing to the supporting steel stringers, either through friction resulting from the applied wheel load or by direct stringer/deck connection.

Several short span timber decked bridges located in Bastrop County, Texas were examined as part of this investigation. A lack of positive attachment between the deck and supporting stringers was common. Most of the decks had some degree of lateral stiffness to offer to the system.

Figure 5.3 Typical J-Bolt Connection at Exterior Stringer

Connection details varied among the decks examined. The decks of the older bridges were attached to exterior stringers with J-bolts as shown in Figure 5.3, with no attachment provided between the deck and interior stringers. The decks of the newer bridges visited were attached to the top flanges of the supporting stringers with light gage galvanized metal clips, as shown for a deck constructed with 4x8 planks in Figure 5.4 and a laminated 2x4 deck in Figure 5.5. The clips were randomly spaced along the span and were typically provided on only one side of each stringer. This configuration restricted relative lateral displacement between the stringers and deck in only one direction. The deck was prevented from slipping away to either side of the bridge by alternating the attached side among the stringers. The clips engage the deck in one direction so that friction must be relied on to engage the deck and resist lateral buckling.

A bridge deck may function as a lateral and torsional brace when light gage metal clips are provided uniformly along both sides of a stringer, since rotation of the stringer relative to the deck will be resisted by the clips. It must be noted, however, that the demonstrated useful life of this type of bridge will far exceed that which can be reasonably expected for the light gage clips. Replacement of these clips is difficult because they are fastened with nails to the side of the deck planks and located in between the joints.

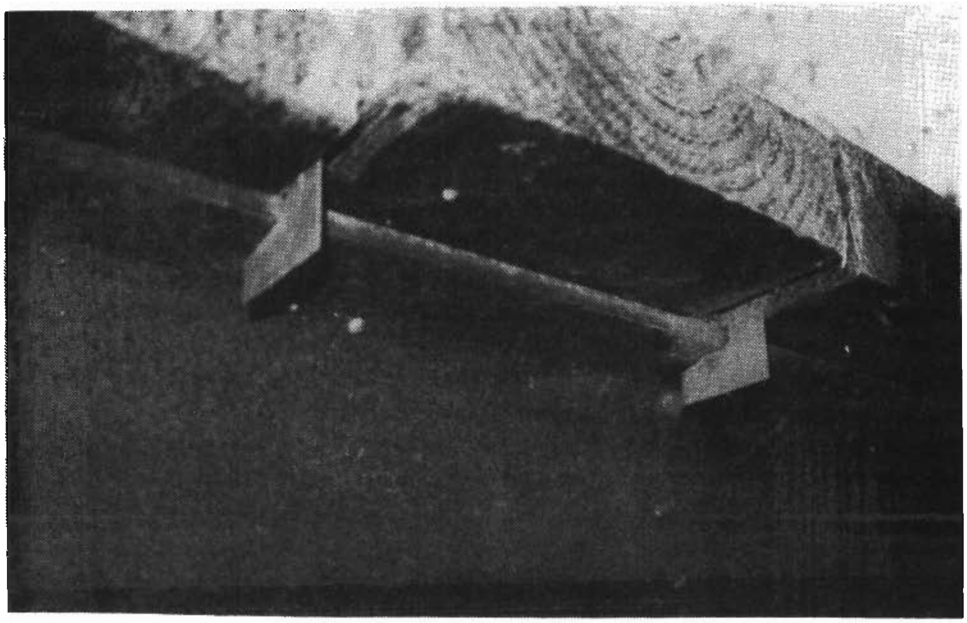


Figure 5.4 Light Gage Metal Clip for Deck/Stringer Connection

Figure 5.5 Laminated Deck Connected With Light Gage Metal Clips

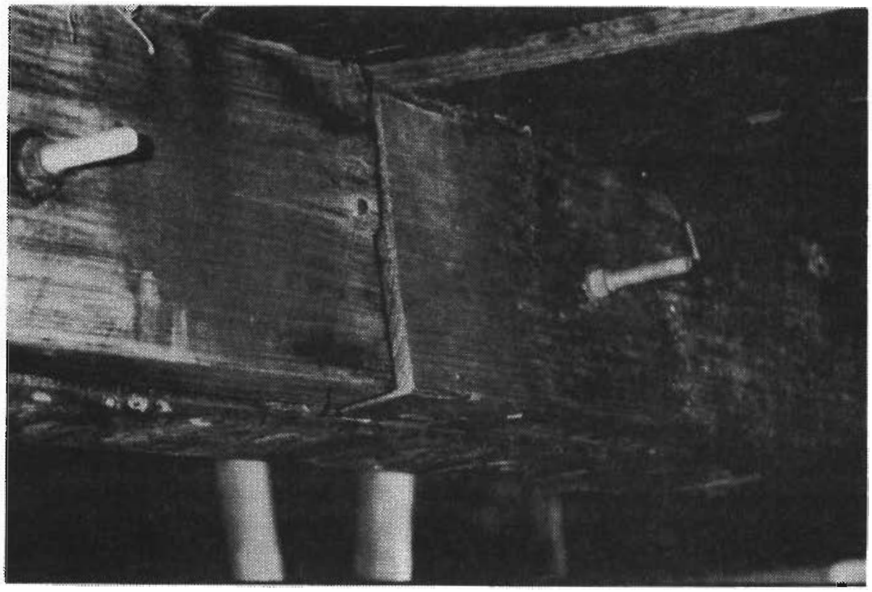


Figure 5.6 Deck Nailer Bolted to the Web of the Stringer

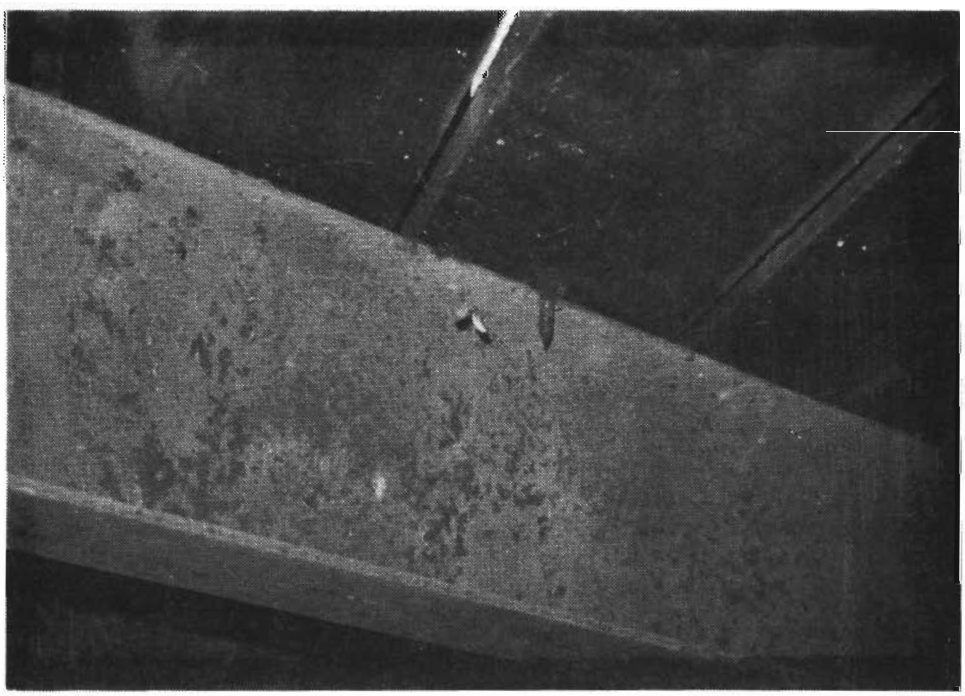


Figure 5.7 Spike Driven Through Deck for Lateral Shear Connection

Figure 5.8 Typical Laminated 2x4 Deck

Nailers bolted to the web, as shown in Figure 5.6, are another common connection detail. The width of the nailer extends out beyond the edge of the flange and the deck is connected to the nailer with spikes. AASHTO specifies that this type of nailer be a minimum of four inches thick and fastened to the web with 5/8 bolts spaced no greater than four feet apart. While this detail certainly increases the lateral stiffness of the stringer, its contribution is beyond the scope of this investigation. Another detail that was observed was the use of timber members for the exterior stringers. The deck planks were spiked directly to these timber stringers and reliance on friction is only necessary for the interior stringers which were not connected to the deck. Figure 5.7 shows a spike driven through the deck plank so that it rests against the top flange of the stringer. This detail, which provides lateral shear resistance, was observed on one of the bridges visited in Bastrop County. Figure 5.8 shows a typical laminated 2x4 deck. This type of deck may be treated as a diaphragm with respect to lateral stiffness considerations, resulting in much greater stiffness than provided by the nailers in timber plank decks.

Even though friction requirements are small, there may be some reluctance among engineers to depend upon friction resistance to transfer lateral loads and engage the deck as a lateral brace. Figure 5.9 presents an alternate remedial lateral shear connection detail which can be implemented on any of the bridges that were examined. The 2x6 members located adjacent to the stringers have been oriented to provide additional contact with the top flange and minimize end grain splitting which may otherwise result from interaction with the stringer. The connection to the deck for this detail should be designed for brace forces determined as discussed in the previous section on strength requirements for lateral braces.

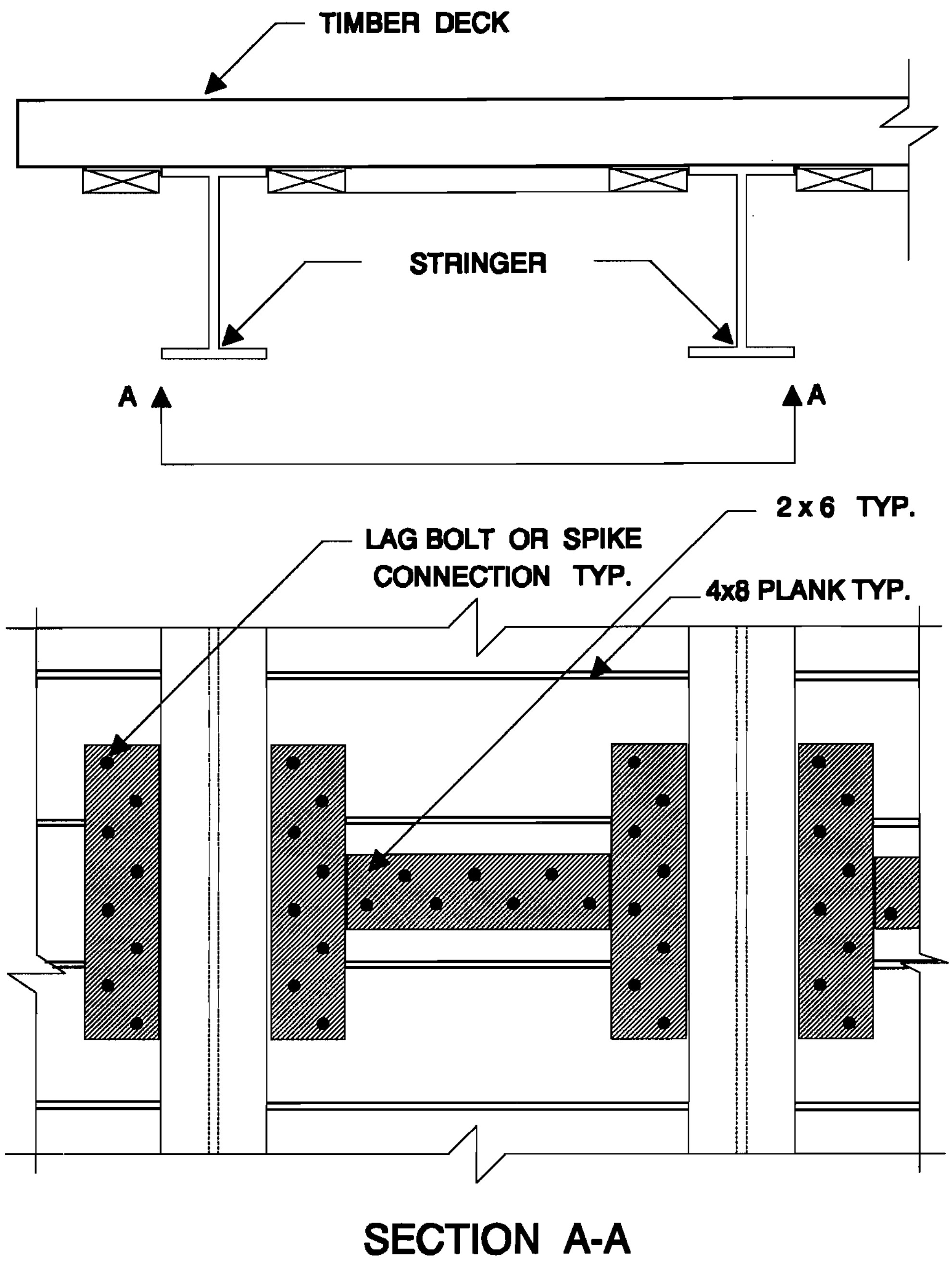


Figure 5.9 Alternate Remedial Lateral Shear Deck/Stringer Connection

5.5 Stiffness Evaluation of Timber Decks

Deck construction and details were not uniform for the bridges visited in Bastrop County. This wide variation is representative of the off-system bridges in Texas. Lateral stiffness of decks must be determined according to the merits of each individual bridge system. Two bridges are selected for discussion.

A photograph of the deck for Bastrop County bridge AA-0233-01 is presented in Figure 5.10. Two rows of three 2x12's are located in the center of this 34-ft one lane bridge composed of two 17-ft spans. The 2x12 overlay detail was found on most of the bridges visited. The 4x8 timber deck is supported by six continuous railroad rail stringers. 2x12's serve to tie the deck planks together. Lateral stiffness for this deck is derived from the six 2x12's and any rotational restraint provided by their connection to the planks. Typically, the 2x12's are fastened to the planks with only one spike and connections have not been provided at every plank. It is recommended that all of the 2x12's be fastened to each plank with two spikes. While it may be conservative to ignore the rotational restraint of the nailed connections, the use of two spikes will significantly improve the lateral stiffness of the bridge deck, as demonstrated in Chapter 4. With one connector, the lateral stiffness of this deck can be conservatively taken as that provided by the sum of the six 2x12's, since no splices occur within the middle two-thirds of the span. With friction engaging the deck as a lateral brace at midspan, the discrete brace stiffness, β_L, based on a midspan loading model is equal to 9.66 kip/in. or 1.61 kip/in. per stringer. The corresponding uniform brace stiffness per stringer is 0.0105 kip/in. per in. of length. The calculations for this deck stiffness are provided in detail as part of Design Example 1, given previously. For simplicity the bridge was conservatively modeled as two simple 17-ft spans.

The deck for Bastrop County bridge AA-0237-01 is shown in Figure 5.11. Two rows of three 2x12's are located in the center of this 45-ft one lane bridge composed of three 15-ft spans. This bridge had a posted axle or tandem weight limit of 7,500 lbs. In addition to the 2x12's, 4x8 planks are located along the outside edges of the deck. The 4x8 deck is supported by ten continuous railroad rail stringers. The 2x12's were not uniformly attached, with connections provided at about every other plank. It is recommended that the 4x8 side members and the 2x12's be fastened with two spikes to each 4x8 deck plank. A splice in one of the 2x12's and one of the 4x8 side members occurs in the middle two-thirds of both exterior spans. Assuming that the recommended connections have been provided, the lateral stiffness for this bridge deck can be conservatively taken as the sum of the individual stiffnesses for five of the six 2x12's and one of the two 4x8 side members due to the splices. Relying on friction to transfer loads at midspan, the corresponding discrete brace stiffness based on a midspan loading model is 13.3 kip/in. or 2.67 kip/in. per stringer. The continuous brace stiffness per stringer is 0.0198 kip/in. per in. of length. For an S7x15.3, this deck would provide sufficient lateral bracing to force yielding before buckling.

While no general conclusions can be drawn, the lateral bracing provided by the decks reported herein was capable of forcing yielding before buckling for the stringers considered.

Figure 5.10 Timber Deck for Bastrop County Bridge AA-0233-01

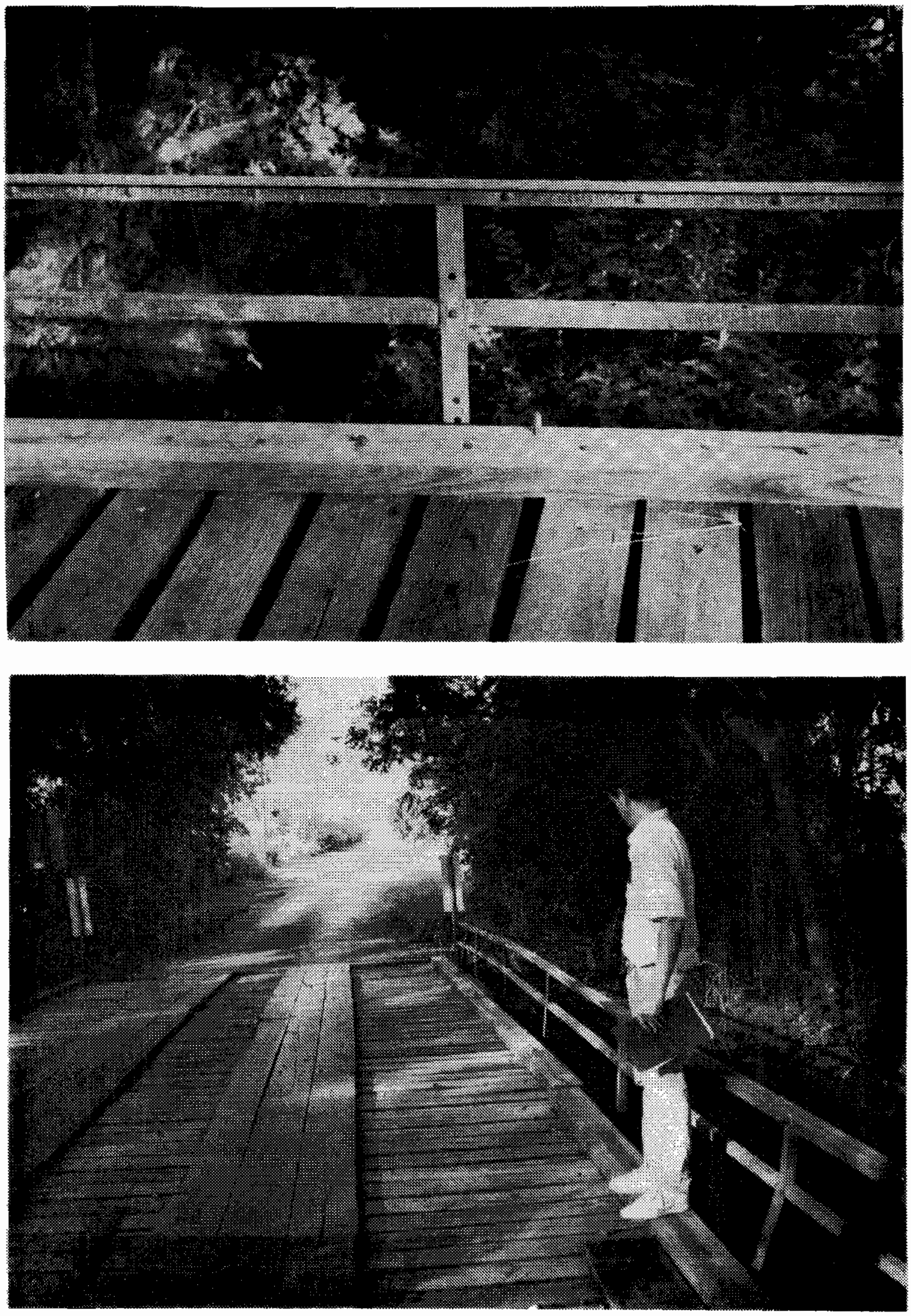


Figure 5.11 Timber Deck for Bastrop County Bridge AA-0237-01

5.6 Stiffness Evaluation of Concrete Decks

The lateral stiffness provided by concrete decks is significantly greater than that of timber decks. For a 5-in. thick, normal weight 3,000 psi concrete deck supported by 6 stringers, with a width of 16 ft and a span of 17 ft, the corresponding continuous brace stiffness per stringer, based on a single midspan discrete bracing model, is 68.0 kip/in. per in. of length. This is six thousand times greater than 0.0105 value for the same bridge with a timber deck (see County bridge AA-0233-01, discussed above). Concrete decks provide much greater lateral stiffness and have better friction resistance than timber decks. For such bridges, the axle location can be considered a brace point.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Summary

There are many off-system short span steel bridges in Texas which must be periodically rated for capacity and overall condition. The decks for this type of bridge possess some degree of strength and stiffness and can provide lateral bracing to the supporting steel stringers when engaged either through friction at the location of the wheel load or by direct shear connection between the deck and stringers. Equations have been presented which can be used to determine the strength and stiffness of both timber and concrete decks as lateral bracing for short span bridges with steel stringers. A procedure for calculating the required coefficient of friction between the deck and stringers has been included in the design examples of Chapter 5. Guidance has also been provided for the evaluation of components contributing to the lateral stiffness characteristics of timber and concrete decks.

6.2 Conclusions

The full scale lateral stiffness tests of a timber deck composed of planks fastened together with nailers showed that the nailers and the rotational restraint of the nailer/plank connections made with two nails provided most of the resistance. The magnitude of the rotational restraint furnished by the nailer/plank connections depends on the degree of contact between the members. Large gaps significantly reduce the level of restraint. However, even with large gaps between the nailers and planks, the connections significantly increase the lateral stiffness of the deck. The deck stiffness was five times greater than the stiffness of the nailers alone. The restraint provided by the specific nails used to construct the full scale bridge were determined experimentally and no conclusions can be drawn for other fasteners. Good approximations for the lateral stiffness of a bridge deck may be obtained using a standard finite element computer program. The experimental stiffness compared very well with theoretical calculations based on handbook properties but measured connection restraint. Such approximations may not be feasible for all bridge decks, given the uncertainty concerning the actual rotational restraint provided by the connections. It is conservative to ignore the benefit from the connections and to approximate the lateral stiffness of the deck solely as that provided by the nailers. Laminated timber decks and concrete decks may be treated as diaphragms for lateral stiffness calculations.

The decks evaluated as part of this investigation had sufficient stiffness, based solely on the contribution from the nailers, to force the supporting stringers to yield before buckling. The bracing equations presented in Chapter 5 indicate that for the bridges investigated, friction at the location of contact for wheel loads is sufficient to engage timber decks and provide lateral bracing. Further analysis of the brace force equation showed that a coefficient of friction of 0.08 is required to engage the deck as a lateral brace for a stringer with a span-to-depth ratio (L/d) equal to 40. Less friction is required for smaller span to depth ratios. The required value is significantly less than the 0.25 friction coefficient measured as part of this investigation.

A comprehensive conclusion applying to all bridges cannot be drawn from this investigation. Friction between the deck and stringers was more than sufficient to engage the deck as a lateral brace, forcing the beams to yield before buckling. The bracing provided by a deck can significantly increase the lateral buckling capacity of a bridge.

6.3 Recommendations

Lateral bracing effects associated with bridge decks should be considered in design and evaluation. Friction is sufficient to mobilize the deck as a lateral brace. For those engineers reluctant to depend on friction, a cost-effective lateral shear connection detail has been provided. Nailers should be adequately connected to the deck planks in order to mobilize their lateral stiffness as bracing. Further testing should be performed so that the benefits from tipping effects and the rotational restraint provided by the spiked connections may be accurately predicted and utilized in design. Finally, a standard set of construction details should be followed for these types of bridges so that corresponding allowable design values may be provided to the design engineer. Standardization will result in cost-effective bridges with efficient structural systems.

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