**Technical Report Standard Page**

| 1. Report No. **FHWA/LA.06/418** | 2. Government Accession No. | 3. Recipient's Catalog No. |
|---|---|---|
| 4.Title and Subtitle<br>**Monitoring System to Determine the Impact of Sugarcane Truckloads on Non-Interstate Bridges** | | 5.Report Date **December 2008**<br>6.Performing Organization Code |
| 7.Author(s)<br>**Dr. Aziz Saber, P.E.**<br>**Dr. Freddy Roberts, P.E.** | | 8.Performing Organization Report No. |
| 9.Performing Organization Name and Address<br>Civil Engineering Program<br>Louisiana Tech University<br>P.O. Box 10348<br>Ruston, LA 71272 | | 10. Work Unit No.<br>11. Contract or Grant No.<br>**LTRC Project No. 03-2ST**<br>**State Project No. 736-99-1133** |
| Sponsoring Agency Name and Address<br>**Louisiana Department of Transportation and Development**<br>**Louisiana Transportation Research Center**<br>**4101 Gourrier Avenue**<br>**Bâton Rouge, LA 70808** | | 13. Type of Report and Period Covered<br>**Final Report**<br>14.Sponsoring Agency Code |

15.Supplementary Notes
Conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration


16.Abstract

The study included in this report assessed the strength, serviceability, and economic impact caused by overweight trucks hauling sugar cane on Louisiana bridges. Researchers identified the highway routes and bridges being used to haul this commodity and statistically chose samples to use in the analysis. Approximately 84 bridges were involved in this study. Four different scenarios of load configuration were examined:

1. GVW = 100,000 lb., with a maximum tandem load of 48,000 lb.,
2. GVW = 100,000 lb., with a maximum tridem load of 60,000 lb.,
3. Uniformly distributed tandem and tridem loads, and
4. GVW = 120,000 lb., with maximum tandem of 48,000 lb., and maximum tridem of 60,000 lb.

It is to be noted that a GVW of 120,000 lb. for sugarcane haulers was the highest level currently considered in this investigation.

The methodology used to evaluate the fatigue cost of bridges was based on the following procedures: 1) determine the shear, moment, and deflection induced on each bridge type and span, and 2) develop a fatigue cost for each truck crossing with a) a maximum GVW of 120,000 lb., and b) a GVW of 100,000 lb. with a uniformly distributed load.

Through the use of a field calibrated finite element model, Structure 03234240405451 was analyzed and load rated for loading vehicles HS-20, 3S2 and 3S3 (sugar cane loading cases 1 thru 4). The structure had adequate strength to resist both bending and shear forces for all six loading vehicles. It should be noted that all of the rating factors were acceptable for all 17 spans as long as the construction and the structural condition of each span were the same.

Results indicate that among the four cases of loading configurations, Case 4, which was a GVW=120,000 lb. with maximum tandem and tridem loads, generated the worst strength and serviceability conditions in bridges. Therefore, Case 4 is the loading configuration that controls the strength analysis and evaluation of fatigue cost for bridge girders. Based on the controlling load configuration, Case 4 with a GVW = 120, 000 lb., the estimated fatigue cost is $11.75 per trip per bridge. In Case 3, which was a GVW = 100,000 lb. uniformly distributed load; the estimated cost is $0.90 per trip per bridge.

The results from the bridge deck analyses indicate that the bridge deck is under a stable stress state, whether the stresses are in the tension zone or the compression zone. Moreover, the decks of bridges with spans longer than 30 ft. may experience cracks in the longitudinal direction under 3S3 trucks. Such cracks will require additional inspections along with early and frequent maintenance.

Based on the results of the studies presented in this report, it is recommended that truck configuration 3S3 be used to haul sugar cane with a GVW of 100,000 lb. uniformly distributed. This will result in the lowest fatigue cost on the network. It is recommended that truck configuration 3S3 not be used to haul sugar cane with GVW of 120,000 lb. This will result in high fatigue cost on the network and could cause failure in bridge girders and bridge decks.


| 17.Key Words | 18.Distribution Statement |
|---|---|
| Monitoring system, instrumentation, impact, heavy loads, deflections, analysis, overweight trucks, fatigue, sugar cane, stress, strain, bridge rating, live load test | Unrestricted. This document is available through the National Technical Information Service, Springfield, Va 21161 |

| 19.Security Classif.(of this report) | 20.Security Classif.(of this page) | 21.No. of Pages | 22.Price |
|---|---|---|---|
| | | 231 | |

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## Project Review Committee

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LTRC appreciates the dedication of the following Project Review Committee members in guiding this research study to fruition.

### *LTRC Manager*

**Walid R. Alaywan**
**Senior Structures Research Engineer**

### *Members*

**Gill Gautreau, LADOTD Bridge Maintenance**
**Paul Fossier, LADOTD Bridge Design**
**Mike Boudreaux, LTRC**
**Arturo Aguirre, FHWA**

### *Directorate Implementation Sponsor*

**William T. Temple, DOTD Chief Engineer**

# Monitoring System to Determine the Impact of Sugarcane Truckloads on Non-Interstate Bridges


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LTRC Project No. 03-2ST
State Project No. 736-99-1133

conducted for
Louisiana Department of Transportation and Development
Louisiana Transportation Research Center





December 2008

# ABSTRACT

The study included in this report assessed the strength, serviceability, and economic impact caused by overweight trucks hauling sugar cane on Louisiana bridges. Researchers identified the highway routes and bridges being used to haul this commodity and statistically chose samples to use in the analysis. Approximately 84 bridges were involved in this study. Four different scenarios of load configuration were examined:

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decks of bridges with spans longer than 30 ft. may experience cracks in the longitudinal direction under 3S3 trucks. Such cracks will require additional inspections along with early and frequent maintenance.

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# ACKNOWLEDGMENTS

This research project was sponsored by the Louisiana Transportation Research Center (LTRC) of the Louisiana Department of Transportation and Development (LA DOTD) under research project number 03-2ST and state project number 736-99-1133.

The researchers want to express their gratitude to the Project Review Committee, many of whom provided direct assistance to the project team as they developed information needed to complete the study. During the course of this research project, the research team at Louisiana Tech University received valuable and much appreciated support and guidance from LTRC staff and engineers, especially Harold "Skip" Paul and Walid Alaywan.

This study could not have been completed without the assistance of personnel from District 03. Personnel from district administration, construction engineering, maintenance, materials, and traffic all contributed to the successful completion of the project.

The assistance and support of the staff and employees at Bridge Diagnosis Incorporation is much appreciated.

# IMPLEMENTATION STATEMENT

The results of the study will be implemented in the maintenance, design, and construction of slab-girder bridges that are located in Louisiana and could be extended to other states. This research could also lead to a cost model for bridge maintenance in Louisiana and the entire nation.

The results from this project can be immediately implemented by the Louisiana legislature. It should be noted that none of the fatigue cost is currently being recovered through permit fees. In analyzing the effect of the current GVW defined by Louisiana statutes, the project staff determined the following:

- At the current 100,000 lb. GVW prescribed for 3S3 Type sugar cane trucks, the fatigue cost will be $0.90 per bridge per trip, **IF** the law requires that the load is uniformly distributed over the axles.

- Increasing the GVW to 120,000 lb. for 3S3 Type sugar cane trucks, will increase the fatigue cost to $11.75 per bridge per trip.

- Therefore, the project staff recommends that no consideration be given to increasing the GVW from current levels to 120,000 lb.

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# INTRODUCTION

The 1998 Transportation Equity Act for the 21st Century (TEA21) allows heavier sugarcane loads to be hauled on Louisiana interstate highways. These heavier loads are currently being applied to state and parish roads through trucks traveling from and to processing plants. TEA 21 also provides Federal funding to enable Louisiana to study the effects of increasing the allowable permitted loads for transporting sugarcane.

Generally, commercial vehicle weights and dimension laws are enforced by highway agencies to ensure that excessive damage (and subsequent loss of pavement life) is not imposed on the highway infrastructure. The axle load and the total load of heavy trucks, which are considered primarily responsible for decreasing the service life of bridges, are significant parameters of highway traffic. Currently in Louisiana, gross vehicle weight (GVW) on interstate routes has typically been restricted to 80,000 lb. for five axle semi-trailer (LA type 6) vehicles with a maximum tandem axle weight of 34,000 lb. For many years, permitted loads on the type 6 vehicle during harvest season have been allowed for up to 83,400 lb. GVW and 35,200 lb. GVW on tandem axles. TEA 21 now extends the GVW to 100,000 lb., with tandem axle weights increasing to 48,000 lb. for interstate travel. Prior to TEA 21, the Louisiana legislature has allowed sugarcane haul loads up to 100,000 lb. with a nominal permit fee. Because highways have traditionally been designed for the legal load of 80,000 lb., permitted trucks of 100,000 lb. or more decrease the expected service life of the infrastructure. This results in increased transportation costs due to high maintenance and the need for early rehabilitation.

The performance and design requirements of highway bridges are affected by the maximum allowable GVW that operates on the system. The Federal Bridge Formula limits the demands on bridges based on the regulated axle spacing, axle weights, and maximum GVWs of vehicles that operate on the highway system. Although the maximum allowable axle loads are in compliance with existing regulations, bridges are sensitive to the magnitude and spacing of the axle loads they can carry. Furthermore, the span length of the bridge and the support conditions (simple or continuous) affect the allowable combinations of axle load and spacing. The impact aspects of increasing the maximum allowable truck loads on bridge performance are safety, serviceability, and durability. While compromises can be made with respect to serviceability and durability in the interest of transportation efficiency, the fundamental safety of the existing bridge system must always be maintained. Safety is a concern for all bridges that are traversed by 100,000 lb. GVW sugarcane trucks, whether on the interstate or not.

In March 2005, the PI for this study completed a study for the Louisiana Department of Transportation, LA DOTD, and the Louisiana Transportation Research Center, LTRC. The study was to assess the effects of trucks hauling timber, coal, and lignite on Louisiana highways and bridges [1]. During that time, the Project Review Committee, PRC, for this study decided that the loads for sugarcane trucks should be investigated based on a GVW of 120,000 lb. Since loads of such magnitude will result in reduced service life of the Louisiana bridges, this study will evaluate the short-term and long-term behavior of bridges under these overweight vehicles. The solutions, which may include alternative vehicle axle configurations, reduced haul loads, or accepting more frequent rehabilitation of the bridges, will be investigated. Also, bridge costs will be generated for the 120,000 lb. GVW scenario plus the load factors included in the method of design in Load Resistance Factor Design (LRFD) [2].

# OBJECTIVE

Increasing the maximum allowable sugarcane truckload to 120,000 lb. will affect bridge safety, serviceability, and durability. While compromises can be made with respect to serviceability and durability in the interest of transportation efficiency, the fundamental safety of the existing bridge system must always be maintained. The objectives of this research were (1) to investigate impact of the load increase on bridge strength and safety, (2) to monitor a selected bridge based on current and future sugarcane overloads, and (3) to determine the economic impact of overweight trucks hauling sugar cane on Louisiana bridges.

# SCOPE

The investigation of the impact of sugarcane truckloads (GVW of 100,000 lb. and 120,000 lb.) on non-interstate bridges encompassed:

1. studying  the effects of sugarcane truckloads on distribution of forces and moments on slab-girder bridges, and
2. developing a long-term monitoring system that will help in assessing the impact of sugarcane truckloads on the safety, serviceability, and durability of non-interstate bridges.

The parameters that may affect the load distribution of a bridge can be divided into the following main categories:

**Type of loading on the bridge**

The effects of sugarcane truckloads will be evaluated based on Louisiana State laws for maximum axle, tandem, and tridem loads. Four different load combinations are considered:

1) GVW 100,000 lb., with maximum tandem load of 48,000 lb.
2) GVW 100,000 lb., with maximum tridem load of 60,000 lb.
3) GVW 100,000 lb., with uniform distributed tandem and tridem loads.
4) GVW=120,000 lb. with maximum tandem of 48,000 lb., and maximum tridem load of 60,000 lb.

**Geometry of the bridge**

(a) Type of girder, girder spacing, length of the span, and number of spans.

(b) Relative dimensions of the girders and slabs.

(c) Simply supported and continuous bridge conditions.

# METHODOLOGY

The roads that are highly traveled by sugarcane trucks were identified. The state bridge inventory was used to locate the state bridges on these roads.

The range of bridge parameters under investigation were used such that they adequately covered the range of the bridges traveled by sugarcane trucks.

The variation of forces and moments in bridge girders as a function of sugarcane truckloads was studied.

A bridge monitoring system was developed to monitor in-service conditions and assess adverse loading conditions.

The results of the analyses and experimental work were compared to the recommended design procedure stated in AASHTO specifications and the LA DOTD Bridge Manual [3], [4].

Cost estimate for bridges crossed by sugarcane trucks were determined.

Recommendations for sugarcane truckloads on bridges were made based on the results of this investigation.

Research on the long-term monitoring system was carried out in two phases. In the first phase, researchers selected a bridge for instrumentation and developed the instrumentation plan that measured and recorded the strains at critical locations in the bridge. The monitoring system and instrumentation plans were reviewed and approved by the Project Review Committee.

In the second phase, researchers acquired and installed the monitoring system in the field. They also set up testing procedures, signal processing procedures for identification of critical changes in the resistance, and selection criteria for proof load level(s).

This study utilized the state-of-the-art knowledge and technology regarding, structural behavior, and bridge loads, as demonstrated in Saber et al. [5]. Emphasis was placed on long-term monitoring techniques for damage caused to used bridges by overloaded trucks and the cost incurred in maintaining and repairing such bridges.

Louisiana bridge inventory was used to optimize the range of bridge and truck load parameters. A bridge load carrying capacity diagnostic system was developed and installed on the bridge selected by LTRC and the PRC. The structural responses (strength and serviceability) were correlated with applied loads. The results of the combined analytical and field procedures were compared to the recommended design procedures stated in AASHTO specifications and the LA DOTD Bridge Manual [3], [4]. The permanent structural health monitoring system was turned over to LTRC with a training manual for the operation of the system.

## Analysis Overview

The methodology used in the analysis phase evaluated the effect of the heavy loads on the bridges from the trucks transporting sugarcane products, based on LRFD and LFD design recommendations [1], [2]. The demand on the bridge girders due to the heavy truck loads was calculated based on bridge girder type, span type, and the bridge geometry. The effects of sugarcane truck loads on state bridges were determined by comparing the stresses in the deck, girders, and the vertical deflection of the girders to allowable stresses and deflection. The AASHTO Line Girder Analysis approach, detailed analysis using finite element models created by the GTSTRUDL software were used to achieve the objectives of this study [3], [6].

The short-and long-term effects of sugarcane truck loads were determined based on the ratio of the strength and serviceability for each bridge in the sample to those of an HS20 truck on the same bridge. The truck loads for hauling sugarcane were based on the 3S3 truck configuration shown in figure 1, with a maximum tandem load of 48,000 lb., a maximum tridem load of 60,000 lb., and a steering axle of 12,000 lb.

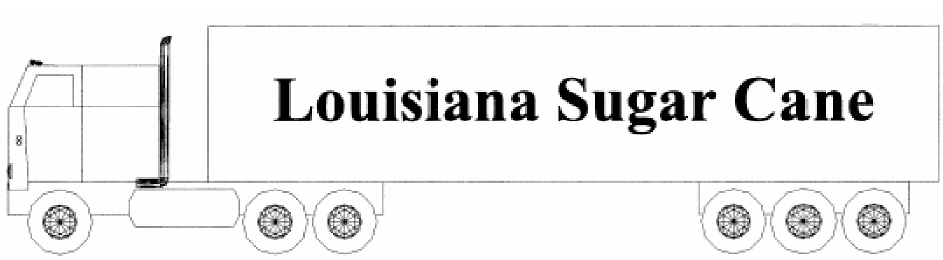



**Figure 1**
**Truck 3S3 hauling sugarcane on Louisiana bridges**

The study considered the following four different truck load configurations.

**Case 1**: GVW=100 Kip with Max Tandem Load 48 Kip and Tridem Load 40 Kip, as shown in figure 2.

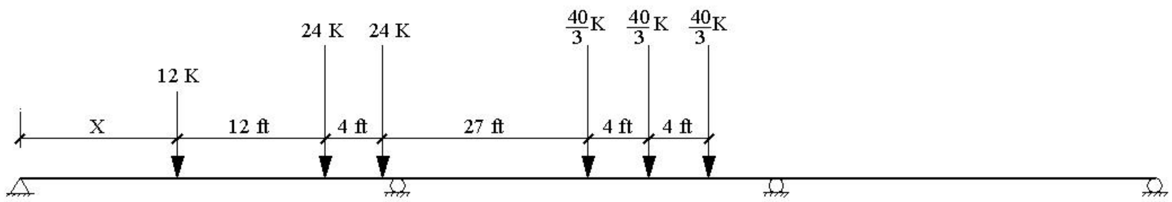



**Figure 2**
**Case 1 GVW=100 kip (max tandem load 48 kip, tridem load 40 kip)**

**Case 2:** GVW=100 Kip with Max Tandem Load 28 Kip and Max Tridem Load 60 Kip, as shown in figure 3.

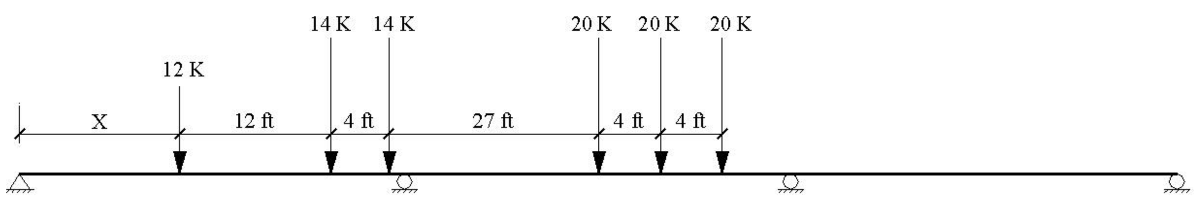



**Figure 3**
**Case 2 GVW=100 kip (max tandem load 28 kip, max tridem load 60 kip)**

**Case 3:** GVW=100 Kip with Uniformly Distributed Tandem and Tridem Loads, as shown in figure 4.

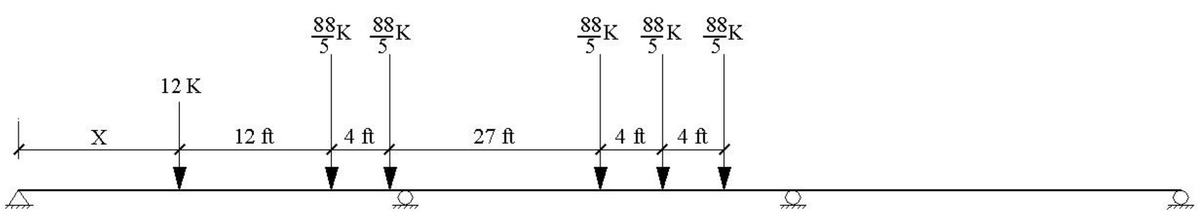



**Figure 4**
**Case 3 GVW=100 kip (uniformly distributed tandem and tridem loads)**

**Case 4:** GVW=120 Kip with Max Tandem Load 48 Kip and Max Tridem Load 60 Kip, as shown in figure 5.

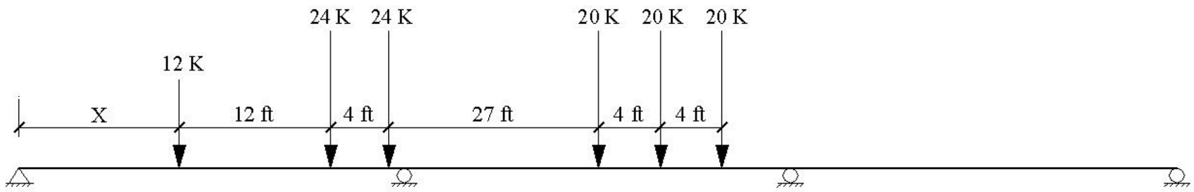



**Figure 5**
**Case 4 GVW=120 kip (max tandem load 48 kip max tridem load 60 kip)**

Bridge costs were generated for Case load 4 and Case load 3 (GVW 120,000 lb. and 100,000 lb.) including the load factors based on the method of design in Load Resistance Factor Design (LRFD) [1].

The first step in the analysis used the influence line procedures to determine the critical locations of the trucks on the bridges that resulted in maximum moment and shear forces. Based on the results from the influence line analyses, the effects of the loads on the bridge girders and bridge decks were determined. Also, the magnitude of the maximum moment and shear forces were calculated. Next, the ratios of the results for the 3S3 truck and the design truck (HS20-44) for flexural, shear forces and stresses were calculated. The serviceability criteria are evaluated for simply supported girders based on their deflections.

## Identification of Critical Bridges for Study

The critical bridges for this study were considered to be those located on the roads most traveled by the sugarcane truck. The roads considered were Louisiana State Highways, U.S. Numbered Roads, and Interstate Highways. The review and selection processes were based on two factors: (1) the amount of sugar cane produced in each parish; and (2) the parish's geographic location. The results of the review were submitted to the PRC and listed in table 1.

The bridges located on these highways were grouped into four different categories based on their structural type. These categories were (1) simple beam, (2) continuous beam, (3) culvert, and (4) others, as listed in Appendix A.

The analyses for bridges in the "simple beam" category were performed using spreadsheets to calculate the maximum moment along with shear and deflection for all the spans in the sample for this study. The ratios for the flexural, shear, and deflection due to the design load and the 3S3 truck load were calculated. All calculations pertaining to this category are included in the appendices of this report.

The analysis for bridges in the “continuous beam” category was performed using GTSTRUDL to develop the influence lines for moment (positive and negative) and shear forces [6]. These results were used in spreadsheets to determine the critical location for the design truck and the 3S3 truck. Then, the maximum moments and shear forces were calculated.

**Table 1**
**Highways with the heaviest sugarcane traffic**

| Highway | Control Section | Tons of Cane | Length (mi.) | Dir. | Between | Parish |
|---|---|---|---|---|---|---|
| LA 14 | 55-6 | 706373 | 1.5 | E | North Rd./LA 339 | Vermillion |
| US 90 | 424-4 | 712606 | 0.5 | SE | St. Martin Parish/Captain Cade Rd. | Iberia |
| LA 416 | 224-1 | 716943 | 1 | E | Oakland Rd./Alma Plantation Rd. | Pointe Coupee |
| LA 1 | 52-2 | 785378 | 4 | E | LA 420/LA 3131 | Pointe Coupee |
| LA 3131 | 839-24 | 785378 | 2 | SE | LA 1/LA 1 | Pointe Coupee |
| LA 1 | 52-1 | 785378 | 3 | S | LA 3131/Curet Rd. | Pointe Coupee |
| LA 14 | 55-6 | 794800 | 2 | E | LA 339/Iberia Parish | Vermillion |
| US 90 | 424-4 | 795170 | 1 | SE | Captain Cade Rd./LA 88 | Iberia |
| LA 1 | 52-1 | 804803 | 2 | S | Curet Rd./LA 78 | Pointe Coupee |
| LA 1 | 52-1 | 822193 | 1.5 | SE | LA 78/LA 978 | Pointe Coupee |
| LA 1 | 52-1 | 835093 | 2.5 | SE | LA 978/LA 413 | Pointe Coupee |
| US 90 | 424-4 | 835560 | 4.5 | SE | LA 88/LA 675 | Iberia |
| US 90 | 424-3 | 839730 | 2 | S | Iberia Parish/Lafayette Parish | St. Martin |
| LA 1 | 50-6 | 851359 | 2 | SE | Augusta Rd./Aloysia Rd. | Iberville |
| LA 14 | 55-7 | 874492 | 8.5 | NE | Vermillion Parish/US 90 | Iberia |
| I 10 | 450-7 | 876665 | 2 | E | LA 415/LA 1 | W. B. Rouge |
| LA 1 | 50-7 | 885265 | 4 | SW | I 10/LA 989-2 | W. B. Rouge |
| LA 1 | 50-6 | 919720 | 1.5 | SE | Aloysia Rd./LA 69 | Iberville |
| US 90 | 424-2 | 920688 | 13 | S | I 10/St. Martin Parish | Lafayette |
| US 90 | 424-4 | 1101664 | 2 | SE | LA 675/LA 14 | Iberia |
| US 90 | 424-4 | 1110054 | 5 | SE | LA 14/freyou Rd. | Iberia |
| US 90 | 424-4 | 1121554 | 1.5 | SE | Freyou Rd./Darnell Rd. | Iberia |
| US 90 | 424-4 | 1141554 | 3 | SE | Darnell Rd./LA 85 | Iberia |
| LA 85 | 236-2 | 1430715 | 0.5 | SW | US 90/mill | Iberia |

## Analysis of Bridge Girders

**Influence line analysis.** When the truck loads, performed as concentrated loads, were placed on the bridge deck, an influence surface was generated. Instead of using the influence surfaces to find the critical moments, shear, and deflection under certain load conditions, the influence line was used. The bending moments and shears for which the influence line was to be determined were computed as a unit load placed at different positions over the length and the width of the bridge. The maximum deflection was computed by superposition.

In this study, HS20-44 truck loads and 3S3 truck loads were used in the analysis procedure. Both hand calculations and computer models in GTSTRUDL were used to determine the critical load location and the corresponding moment and shear forces. Associated deflections and stresses in the bridge girders and bridge decks were determined.

**Simple Span Bridges.** The influence line analysis for bridges with simple spans was performed using GTSTRUDL software and spread sheets. The standard truck configurations HS20-44, as provided in AASHTO Chapter 3, were used. Louisiana's moving from a 3S2 to a 3S3 configuration as shown in figure 1. This is the desired configuration that not all industries are using at this point. The span length for the bridge girders considered in this study varied between 19 ft. and 94 ft.

The loads were moved on the bridge girder in 1 ft. increments to calculate the absolute maximum moment and shear forces. The absolute maximum shear in simply supported bridge girders occurred next to the supports. Therefore, the loads were positioned so that the first wheel load in sequence was placed close to the support.

The absolute maximum moment in simply supported bridge girders occurred under one of the concentrated forces. This force was positioned on the beam so that it and the resultant force of the system were equidistant from the girder's centerline. The truck location on the bridge girder that caused the maximum absolute moment was used to determine the maximum deflection.

Tables 11, 12, and 13 in Appendix B summarize the results for the absolute maximum moment, shear, and deflection, for the HS20-44 and 3S3 truck configurations.

**Continuous Span Bridges.** The influence line analysis was performed using GTSTRUDL software. The bridge girder models were considered as being three equal spans. The first support for the girder was considered pin support and the remaining three supports were roller type. The span length for the bridge girders considered in this study was 90 ft. GTSTRUDL software was used to calculate the influence line of moment and shear at each joint along the length of the bridge girder. Due to the symmetry of the bridge, only the left half of the bridge girder was considered. The truck loads were applied in both directions of the bridge, from left to right and from right to left. The results were used in the following steps to calculate the moment and shear forces.

After generating the influence line for each joint, the position of the truck loads on the bridge girder that resulted in maximum positive moment, maximum negative moment, and maximum shear forces was determined. The results are summarized in table 14 of Appendix B.

The maximum positive moment, maximum negative moment, and shear forces due to the wheel loads were calculated by moving the truck loads along the bridge girders in one foot increments. The magnitude of the moment and shear were calculated by taking the sum of the ordinates multiplied by the magnitudes of the loads. Then the loads were placed at the point which produced the maximum value. The location of the truck load that caused the maximum positive moment occurred around 40 percent of the first span, while the location of the maximum negative moment occurred close to the first support of the bridge. The results are presented in Appendix B, tables 15 and 16.

The results of the analysis for the maximum positive moment, the maximum negative moment, and the maximum shear forces for HS20-44 and 3S3 trucks on continuous bridge girders are shown in Appendix B, table 17.

**Analysis of Bridge Decks**

All bridges considered for this study had concrete decks. According to the LADOTD Bridge Manual, concrete bridge decks are designed as a continuous span over the girders [4]. The bridge deck analyses for this study were performed using finite element models and GTSTRUDL software [6]. The finite element models for typical bridge decks were generated with a typical 30-ft. bridge-deck width and 8-inch thickness supported by five girders. The design load for the bridges included in this study and the loads from 3S3 truck configuration were applied to the deck. Only the “fatigue” load combination, as presented in AASHTO LRFD, was performed for these typical bridge deck models [1].

The finite element model used for bridge decks in this study simulated the behavior of continuous span bridges. The girders were modeled using Type-IPSL tridimensional elements available in GTSTRUDL. Type-SBCR plate elements were used for the bridge deck. Prismatic space truss members were used to model end diaphragms and the connection between the deck plate elements and the girder elements.

**Geometry of Bridge Deck.** The geometry of the bridge depends on the width of the roadway, girder type and quantity, number of spans, span length, girder spacing, the bridge skew angle, and the diaphragm skew angle. The span length was measured from the center of one support to the center of an adjacent support. The girder spacing was measured from the center of one girder to the center of an adjacent girder, which was identical and parallel to the previous girder. All the models considered in this study were non-skewed with end diaphragms. The structures analyzed in this study were 30 ft. wide with three equal spans. The girders were simply supported and the concrete deck was continuous over the girders. The girders were spaced at 8 ft. in the middle and 7 ft. on the outside. All models contained only five girders, as shown in Appendix C, figures 8 and 9.

**Boundary Conditions.** The restraints for all models consisted of four joints across the width of the base of the girder at the end, and intermediate supports. Also, the two joints that connected the plate elements to the rigid members at the end supports behaved as pins.

**AASHTO Loading.** A uniform volumetric dead load of 150 pcf was used for all concrete members when accounting for the self weight of the concrete. The truck loading on the bridge was represented by the HS20-44 and 3S2 truck loading with a 1.3 impact factor, based on AASHTO Chapter 3. In addition to the dead and truck loads, a future wearing surface loading of 12 psf, according to LA DOTD Bridge Manual, was placed on the deck to account for future overlays. The loading conditions used in this investigation were the fatigue loads (self weight and live loads with impact factor) as required by the AASHTO LRFD Bridge Design Manual [1].

**Finite Element Modeling of the Girder over Interior Supports.** Since the girders were simply supported and the deck was continuous over the girders, a space was created between the two girders, over the interior supports, during the construction of the bridge. Because the end diaphragm did not provide continuity in this case, the girder required a 2 in. gap between the girders, as shown in Appendix C, figures 10 and 11.

The bridge decks contained longitudinal reinforcing bars for the tensile stresses induced by the negative moment over the support. In construction, the combination of the deck and the bearing pad restrict the rotation of the girder over the support. Although the girders, when constructed with the end diaphragm, are not joined end to end, the girder is not completely free to act as a truly simply supported beam. In modeling the connection with a two-inch gap between adjoining girders, the girders were free to rotate and act as a simply supported beam because the beam was supported by points at the end of the girder and not resting on the pad.

Due to the restricted rotation of the girders, tensile and compressive stresses still existed at the girder ends.

**Influence Lines.** To determine the critical location of the truck on the bridge, an influence line analysis on the transverse direction was required. The width of the bridge was 30 ft., supported by 5 girders with simple supports. The space between the central 3 girders was 8 ft. and the space to the outer girders was 7 ft. Truck loads were placed on the deck as concentrated loads. GTSTRUDL was used to obtain the influence line for each joint of the deck, and Excel was used to analyze the data to get the critical location of the truck, as discussed previously.

**Bridge Deck Fatigue Evaluation.** The materials in bridges were subject to high cycle fatigue damage. This means that after many cycles of stresses, even stresses below the maximum permitting stress, enough damage may accumulate to eventually cause the failure of the bridge. This would particularly occur on those bridges that carry heavily loaded vehicles. In this study, the fatigue behavior of three equal span bridges was evaluated. The finite element analysis was performed using GTSTRUDL, and the load combination included the fatigue factor and impact factor to investigate the behavior of the bridge. According to the AASHTO specification, the fatigue factor 0.75 and the impact factor 1.3 were used. The span lengths of the bridges are ranged from 20 to 120 feet with simple support conditions. Truck loads for HS20-44 and 3S3 were applied at critical locations for maximum positive and negative moment in the bridge deck to determine the corresponding stresses. The maximum value of longitudinal, transverse, and shear stresses in the bridge deck were obtained and then grouped as the tensile stress and compressive stress. Appendix C, tables 1 through 4, summarizes the results for the maximum stress values of the top and bottom surfaces of the bridge deck, under both HS20-44 and 3S3 truck loads.

### Bridge Monitoring System

The PRC reviewed the list of the critical bridges for this study and selected the bridge located on US 90 and LA 3212 to be monitored for this study. The long-term monitoring system was installed on the span 14 of the structure (No. 03234240405451).

**Long-Term Monitoring System.** A long-term monitoring system was installed to collect data due to heavy loads on the bridge. The instrumentation plans were developed based on the results from the analyses of the critical bridges for this study and the parametric studies presented in Appendix P of this report.

The instrumentation on beam line 3 and 4 (BL3, BL4) located at 4 ft. from the start and the end of the girder was needed to measure the effects of shear forces.

The instrumentation on BL3 and BL4 at 26 ft. from the start of the girder was needed to measure the effects of positive moments.

The instrumentation on BL1 and BL2 at 26 ft. from the start of the girder was needed to initiate the monitoring system in case the sugar cane truck was traveling on lane 2 on the bridge.

The instrumentation on bridge deck at 26 ft. from the start of the girder was needed to measure the effects of the longitudinal and transverse forces in the deck.

The instrumentation on the interior diaphragm at 30 ft. from the start of the girder was needed to determine the redistribution of the forces between the bridge girders.

The PRC reviewed and approved the long-term monitoring system plans as shown in Appendix E.

**Live Load Test.** The plans for the live load test are shown in Appendix F. These plans were reviewed by the Project Manager of the study. The test objectives were to determine the stiffness, capacity, and rating of the bridge. The instrumentation was installed on the bridge girders, railing, and deck. This installation required access to the underside of the structure, power to run some of the equipment (appropriate height ladders and a generator), and traffic support. A loaded test vehicle, with known weight of the front, rear axles, and gross vehicle weight, was used. Similar approach was reported in a study performed by Aktan et al. [4].

**Load Rating.** The selected bridge (structure 03234240405451) consisted of 17 pre-stressed concrete spans and was built in 1966. According to the structure's Inventory and Appraisal sheet, this structure has not been retrofitted or rebuilt since the initial construction. The bridge drawings were partially unreadable; therefore, some assumptions had to be made concerning beam properties and were confirmed with LTRC.

**Table 2**
**Description of structure 03234240405451**

| | |
|---|---|
| Structure Identification | Structure 03234240405451 |
| Location | I-90, North of New Iberia, LA |
| Structure Type | PS/C T-beam bridge |
| Number of Spans | 17 |
| Span Lengths | 60' c.c. of piers / 59'-7" c.c. of beam bearings |
| Skew | 0 (Perpendicular) |
| Structure/Roadway Widths | 32 ft. / 28 ft. |
| Beams | 4 – pre-stress T-beams at 8'-8" on center |
| Deck | RC Deck 7.5". Possibly additional 2" of concrete overlay but none specified in plans. |
| Curbs and Parapets | Cast in place R/C Parapets on outside of exterior beams. |
| Visual condition | All superstructure elements appear to be in good condition with no visible shear or flexural cracks. |

Highway bridges can be rated at two different load levels. One of these levels is referred to as inventory rating and the other is operating rating. Span 14 was load rated based on LRFD and LFD load combination criteria.

Inventory rating determines the load level that a bridge can safely be utilized for an indefinite period of time. Using the allowable stress method, the inventory rating for steel is based on 55 percent of the yield stress.

Operating rating is higher than the inventory rating and it is the absolute maximum permissible load level to which a bridge may be subjected. In no cases can the load levels used be greater than those permitted by the operating rating. For steel, the allowable stress for operating rating is 75 percent of the yield stress.

# DISCUSSION OF RESULTS

## Short Term Effects on Simple and Continuous Span Bridges

In this study, the effects of 3S3 truck loads on these bridges were investigated by comparing the flexural, shear, and serviceability conditions. The effects of 3S3 trucks loads on bridges designed for HS20-44 truck loads were evaluated by normalizing the critical conditions for each bridge span to the design load. The results are presented in Appendix B, tables 7 through 10.

**Simple span bridges.** Based on the results presented in Appendix B, tables 17 and 18, the ratio of the absolute maximum moment varied between 0.89 and 1.42. The ratio of the shear forces varied between 0.92 and 1.40. The ratio for deflection caused by 3S3 truck loads as compared to HS20-44 truck loads varied between 0.89 and 1.62. Deflection is a serviceability criterion and high ratios, as reported in this study, will result in uncomfortable riding conditions for vehicles crossing the bridges.

Where the bridge span was similar to the length of the 3S3 truck, the ratios of the absolute maximum moment and shear were within 10 percent. This confirms the findings in the previous studies that focus on bridge formula. The studies increased the GVW and the truck length to minimize the impact on the stresses in the bridge girders. However, bridge girders with absolute maximum moment ratio or shear larger than 1.1 will be overstressed.

In this study the bridges with absolute maximum moment ratios and shear ratios that are greater than 1.1 experienced more cracking in the bridge girders. Such cracks will require additional inspections along with early and frequent maintenance.

**Continuous span bridges.** Based on the results presented in Appendix B, tables 19 and 20, the ratio of the maximum positive moment varied between 0.99 and 1.24. For the maximum negative moment, the ratio varied between 1.17 and 1.50. The ratio of the shear forces varied between 1.06 and 1.45. Where the bridge span was similar to the length of the 3S3 truck, the ratio of the maximum positive moment and shear forces were within the findings of the previous studies. These studies focused on bridge formula, GVW, and truck length to determine the stresses in the bridge girders and decks. However, bridge girders with a maximum positive moment ratio or shear larger than 1.10 will be overstressed.

The ratio for negative moment for spans between 60 ft. and 90 ft. was high and will increase the compressive stress in the bridge decks. These conditions can result in compression cracks in bridge decks. The bridges in this study with ratios that were greater than 1.1 can experience more cracking in the bridge girders and bridge decks. Such cracks will require additional inspections along with early and frequent maintenance.

### Bridge Decks

This part of the research focused on how the strength and serviceability of bridge decks are impacted by the heavy loads from the trucks transporting sugar cane. Finite element analysis was used for a typical deck and girder system to determine the effects of the trucks on the stresses in the transverse and longitudinal directions and the shear stress.

**Continuous bridge decks.** The effects of 3S3 truck loads on continuous bridge decks designed for HS20-44 truck loads are presented in Appendix C, tables 21 through 26. The stresses are computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface are grouped based on their classification as a tensile or compressive stress.

In the longitudinal direction, the ratio of maximum tensile and compressive stress varied between 0.569 and 1.775. In the transverse direction, the ratio of maximum tensile and compressive stress varied between 0.731 and 1.45. The ratio of shear stress varied between 0.90 and 1.56.

The locations of maximum stresses due to HS20-44 or 3S3 truck loads may differ from each other. The difference is what made the ratio of 3S3 to HS20-44 truck for some span lengths less than 1.

The results from the bridge deck analyses indicate that the bridge deck was under a stable stress state, whether the stresses were in the tension or the compression zone. Moreover, the decks of bridges with spans longer than 30 ft. may experience cracks in the longitudinal direction under 3S3 trucks. Such cracks will require additional inspections along with early and frequent maintenance.

## Long-Term Effects of Hauling Sugar Cane on Louisiana Bridges

The long term effects of heavy loads, such as sugar cane trucks, play an important role in bridge life evaluation. The bridges selected for this study were designed under AASHTO standard HS20-44 truck loads. Overloaded trucks traveling across these bridges will increase the cost of maintenance and rehabilitation. An accurate estimate for the fatigue cost is hard to obtain since fatigue in bridge girders may lead to many actions including repairs, testing, rehabilitations, and replacements.

Many studies have evaluated the remaining lives of bridge structures. These studies have been sponsored by federal committees such as AASHTO and NCHRP and by State DOTs. In March 2005, LADOTD and LTRC published report number 398, "Effects of Hauling Timber, Lignite Coal, and Coke Fuel on Louisiana Highways and Bridges." [7] The method used to determine the fatigue cost on the bridges in the LTRC published report number 398 was used in this study [7], [8], and [9].

Fatigue is an important performance criterion for bridges that are evaluated. Most of the bridges in Louisiana are designed for a 50-year fatigue life. Overloaded trucks will definitely shorten the life of the bridges. The bridges in this study were evaluated for fatigue cost based on the flexural and shear results of the analyses performed in previous tasks of this study. The bridge costs used in this study were based on projects completed by LADOTD during 2004. The average cost to replace concrete bridge girder and bridge deck was \$90 per square foot. The following equation was used to determine the percentage of the life of the bridge used when a truck crosses it.

$$\%\text{ of life} = \frac{(\text{Ratio from analysis})^3}{(2500 \text{ trucks per day} * 365 \text{ days per year} * 50 \text{ years})} * 100$$

The estimated cost per trip across the bridge was obtained by multiplying the percentage of the life of the bridge by the total cost of the bridge. In this study, the cost to replace concrete bridge girder and deck is considered to be \$90 per square foot. Since the trucks are operating on a broad route structure, the total damage cost was estimated on a per bridge basis. This applied to cases with no defined route for the vehicle. The weighted average over all spans lengths and number of spans was used.

The long term effects of 3S3 trucks hauling sugar cane on Louisiana state bridges with simple supports and design load HS20-44 were calculated based on flexural analyses performed in previous tasks. The results are presented in Appendix E, tables 1 and 2.

The estimated fatigue cost for sugar cane trucks with a GVW of 120,000 lb. (Case 4) is $11.75 per bridge per trip. For a GVW of 100,000 lb. (Case 3 pay load is uniformly distributed), the cost is $0.90 per bridge per trip.

## Bridge Load Rating

The bridge that is located on US 90 and LA 3212, structure number 03234240405451 span 14, was selected for load rating and long-term monitoring.

**Load Rating**. The live load tests were used in the analysis for bridge rating. Load rating factors were computed using the Load and Resistance Factor Rating (LRFR) methods specified in the 2003 AASHTO Condition Evaluation of Highway Bridges Manual. Rating values were obtained by applying the dead load and the live-loads to the bridge and comparing the responses to its capacity. Shear and moment capacities were computed using current AASHTO LRFD and 17th Edition- 2002 LFD specifications. Standard width trucks were rated assuming two-lane loading. Live-load envelopes were generated for each member and compared with their respective live-load capacities. As per the AASHTO LRFD and LFD specifications, impact factors of 33 and 30 percent were used for all cases. Table 3 contains the maximum moment and rating factors for the critical member and table 4 contains the same information for shear.

As defined by the AASHTO Manual for Condition Evaluation of Bridges, the inventory rating level corresponds to the design level of stresses and reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the inventory level allow for a determination of a live load that can safely utilize an existing structure for an indefinite period of time. Loadings based on the operating rating level describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at the operating level may shorten the life of the bridge. However, infrequent intervals at the operating limit would not have adverse effects on a structure’s life span.

**Table 3**
**Load rating (moment)**

| Truck | Live-load Moment (kip-inch) | Inventory Rating Factor (LRFD / LFD) | Operating Rating Factor (LRFD / LFD) |
|---|---|---|---|
| HS-20 (3 axle 72 kip) | 4763 | 3.1 / 2.48 | 4.02 / 4.13 |
| 3S2 (5 axle 73 kip) | 5018 | 2.94 / 2.35 | 3.81 / 3.92 |
| 3S3 Case 1 (5 axle 100kip) | 4902 | 3.03 / 2.42 | 3.92 / 4.04 |
| 3S3 Case 2 (5 axle 100kip) | 5283 | 3.06 / 2.26 | 3.67 / 3.78 |
| 3S3 Case 3 (5 axle 100kip) | 4735 | 3.16 / 2.53 | 4.09 / 4.22 |
| 3S3 Case 4 (5 axle 120kip) | 5446 | 2.74 / 2.20 | 3.56 / 3.67 |

**Table 4**
**Load rating (shear)**

| Truck | Live-load Moment (kip-inch) | Inventory Rating Factor (LRFD / LFD) | Operating Rating Factor (LRFD / LFD) |
|---|---|---|---|
| HS-20 (3 axle 72 kip) | 34.3 | 2.52 / 1.67 | 3.26 / 2.78 |
| 3S2 (5 axle 73 kip) | 38.3 | 2.43 / 1.55 | 3.15 / 2.59 |
| 3S3 Case 1 (5 axle 100kip) | 34.5 | 2.50 / 1.66 | 3.24 / 2.77 |
| 3S3 Case 2 (5 axle 100kip) | 41.7 | 2.23 / 1.43 | 2.9 / 2.39 |
| 3S3 Case 3 (5 axle 100kip) | 38.2 | 2.44 / 1.56 | 3.16 / 2.61 |
| 3S3 Case 4 (5 axle 120kip) | 44 | 2.10 / 1.34 | 2.72 / 2.24 |

As shown in the above tables, the controlling load case was the 3S3 (Case 4 Sugar Cane Vehicle with 120kip GVW) for both moment and shear ratings. The structure was shear critical with the critical section located at the first change in vertical stirrup spacing (s = 15"). Overall, the structure rated well for both moment and shear even with the increased vehicle weight of the sugar cane haulers.

Tables 3 and 4 shows that ratings obtained using the LFD method are considerably lower than those obtained using the LRFD method. The primary differences between the two methods are the load factors applied to the live load responses. At the inventory level, the LFD live-load load factor of 2.17 was 24 percent greater than the LRFD load factor of 1.75. The operating limit load factors for both rating methods were relatively close at 1.3 for LFD and 1.35 for LRFD. The impact factor or dynamic allowance was also greater for the LRFD method. Another significant difference between the two rating methods was in the shear capacity calculation. Whereas the computation of moment capacities were similar, the shear capacity calculation in the LRFD method was based on a “strut and tie” approach and resulted in significantly different shear capacities than the LFD method.

## Bridge Monitoring System

The long-term monitoring system was installed on the bridge located on US 90 and LA 3212, structure number 03234240405451, span 14. The monitoring system was calibrated during the months of January and February of 2006. A sample of events occurring at the bottom of the middle girders is presented in figure 6. The strain cycle at the bottom of bridge girders is presented in figure 7. For the strain gauge, when using the LATechv3.CR5 program, the lower limit of the strain cycle was set at 5 microstrain. This way the noise from the gauge was filtered out and not picked up by the data logger. The trigger limit was set at 32 microstrain. The strain cycle data were counted on an hourly basis, thus one record was one hour of data.

The long-term monitoring system was used to monitor deterioration of the structure over the system's scheduled life. Due to the rating being higher than expected, it is likely that the structure will perform fine with the heavier trucks, but if a significant amount of change is seen over a period of time, it is advisable to retest this structure and allow engineers to do further evaluations. A comparison of load test and model re-calibration results from a follow up test would provide a direct measure of any structural changes.

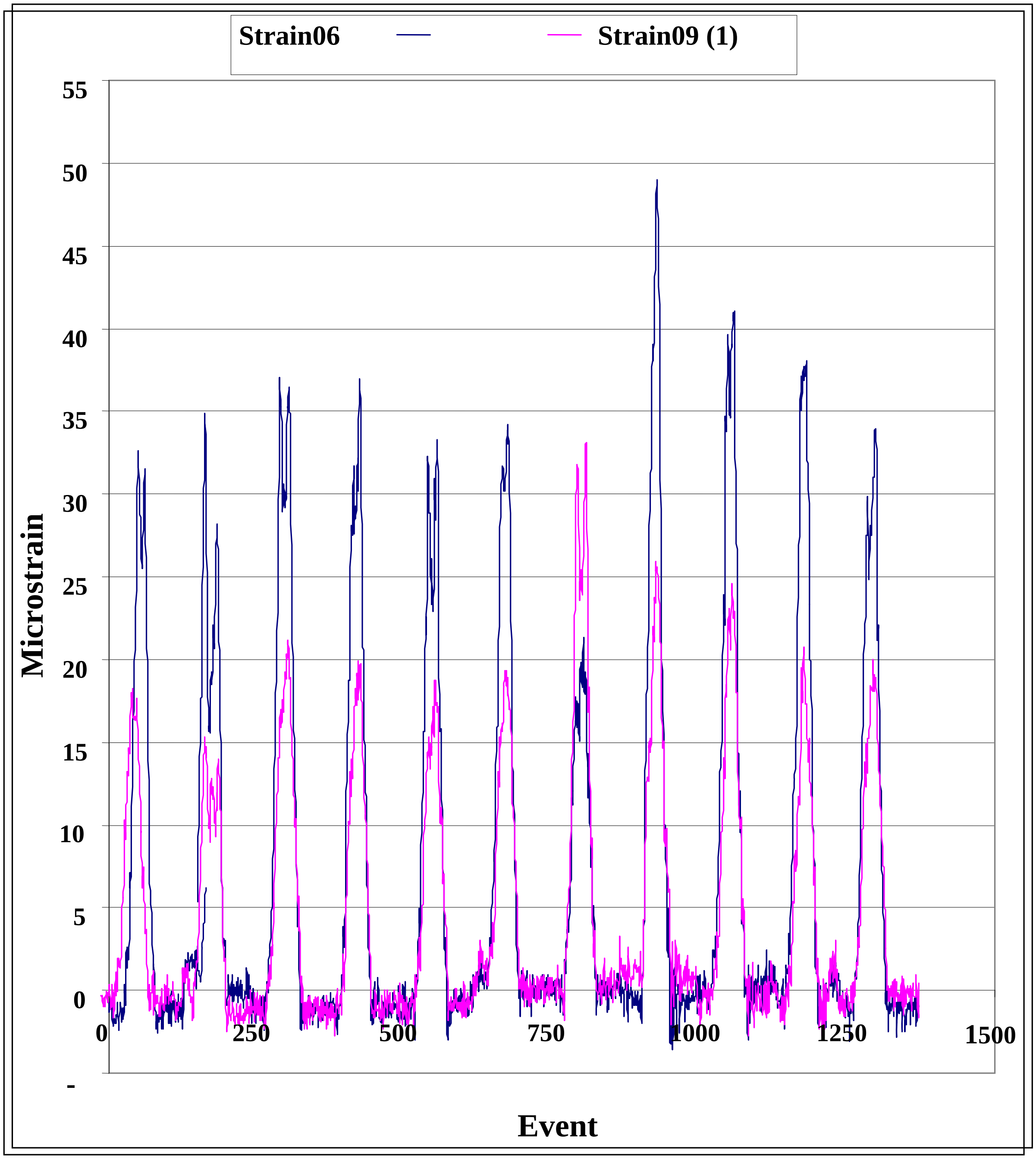



**Figure 6**
**Event chart at bottom of middle girders (structure no. 03234240405451)**

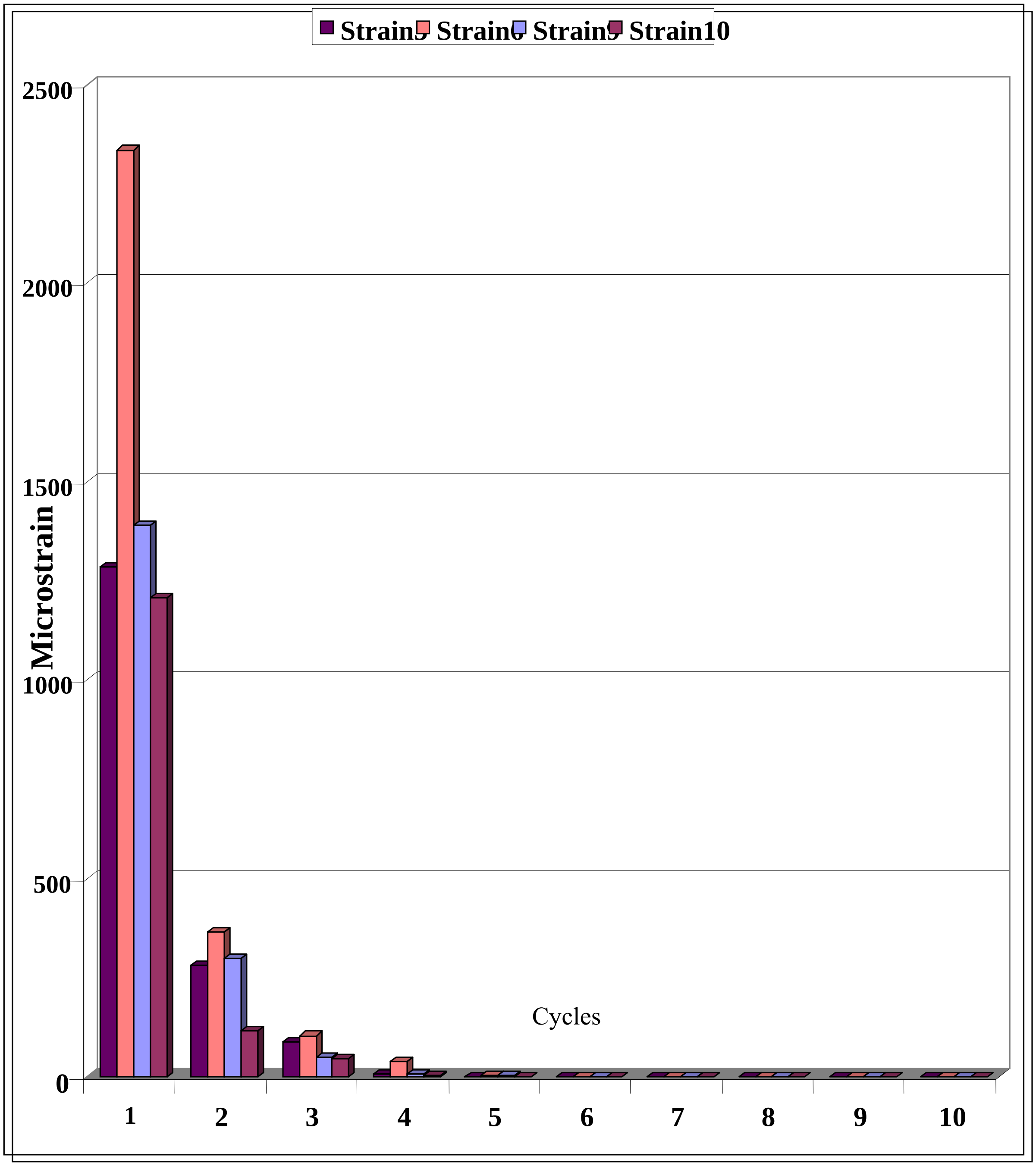



**Figure 7**
**Strain cycle at the bottom of bridge girders (structure no. 03234240405451)**

# CONCLUSIONS

The bridges in this study were evaluated for safety and reliability under 3S3 trucks hauling sugar cane with two GVWs of 120 kip and 100 kip in four different load cases.

Case 1: GVW=100 Kip with Max Tandem Load 48 Kip and Tridem Load 40 Kip, as shown in figure 2.

Case 2: GVW=100 Kip with Tandem Load 28 Kip and Max Tridem Load 60 Kip, as shown in figure 3.

Case 3: GVW=100 Kip with Uniformly Distributed Tandem and Tridem Loads, as shown in figure 4.

Case 4: GVW=120 Kip with Max Tandem Load 48 Kip and Max Tridem Load 60 Kip, as shown in figure 5.

The truck GVW considered at maximum 120,000 lb., and GVW of 100,000 lb., with the load lumped and uniformly distributed. Probability based method was used in this investigation and field experiments on a selected bridge were conducted to compare the theoretical results with the real response of the bridge.

The load test data indicated that the structure responded in a linear-elastic fashion and no damage was apparent. The structure 03234240405451 was analyzed and load rated for loading vehicles HS-20 and 3S3 (Sugar Cane loading cases 1 thru 4). The structure had adequate strength to resist both bending and shear forces for all six loading vehicles and was shear critical with a controlling load rating of 2.10 and 1.34 for inventory and 2.72 and 2.24 for operating for LRFD and LRF, respectively. The worst case was the 3S3 Case 4 for shear rating (Inventory RF: 2.10 / 1.34 Operating RF: 2.72 / 2.24), and for moment rating (Inventory RF: 2.74 / 2.20 Operating RF: 3.56 / 3.67).

Among the four cases of loading configurations, Case 4, which was GVW=120,000 lb., with maximum tandem and tridem loads, produced the largest moments and shear forces. Therefore, Case 4 is the load configuration that controls the strength, serviceability analyses, and evaluation of fatigue cost. Based on that load configuration, the estimated fatigue cost was $11.75 per trip per bridge.

It is important to note that for load Case 3, with GVW 100,000 lb., where the sugar cane load was uniformly distributed and the steering axle load was 12,000 lb., the fatigue cost was $0.90 per trip per bridge.

The results from the bridge deck analyses indicated that the bridge deck was under a stable stress state, whether the stresses were in the tension or the compression zone.  Moreover, the decks of bridges with spans longer than 30 ft. may experience cracks in the longitudinal direction under 3S3 trucks. Such cracks will require additional inspections along with early and frequent maintenance.

## RECOMMENDATIONS

Based on the results of the studies presented in this report, the following is recommended:

- It is recommended that truck configuration 3S3 be used to haul sugar cane with a GVW of 100,000 lb. uniformly distributed. This will result in the least fatigue cost on the network.

- It is not recommended that truck configuration 3S3 be used to haul sugar cane with a GVW of 120,000 lb. This will result in high fatigue cost on the network and could cause failure in bridge girders and bridge deck.

- It is recommended that the effects of different truck configurations hauling sugar cane on bridges be further evaluated.

- It is recommended that the data from the monitoring system be collected for more sugar cane seasons to develop a trend in the bridge performance.

Due to the high load limit for the sugar cane trucks, a long-term monitoring system was also installed on structure 03234240405451. It will be used to monitor deterioration of the structure over the system's scheduled life. Because the rating is higher than expected, it is likely that the structure will perform well with the heavier trucks. However, if a significant amount of change is seen over a period of time, it is advisable to retest this structure and allow engineers to do further evaluations. A comparison of load test and model re-calibration results from a follow-up test would provide a direct measure of any structural changes.

Off-system bridges are generally designed for lower loads than on-system bridges. As a result, the impact of trucks loaded to 100,000 lb. GVW and 120,000 lb. GVW can be very detrimental to the span life of these bridges, and requires further evaluation.

# ACRONYMS, ABBREVIATIONS, & SYMBOLS

3-S3............Truck with 3 axles on tractor and a semi-trailer with 3 axles
AASHTO....American Association of State Highway and Transportation Officials
ADT............Average Daily Traffic, vehicles/day
FHWA.........Federal Highway Administration
GVW...........Gross Vehicle Weight
LA DOTD...Louisiana Department of Transportation and Development
LRFD..........Load Resistance Factor Design
LTRC..........Louisiana Transportation Research Center
PRC.............Project Review Committee

# Structure Type Codes

**Fixed Girder Spans**

| | |
|---|---|
| stplgr | Steel Plate Girder |
| stcplg | Steel Plate Girder - Continuous |
| stcugr | Steel Curved Plate Girder |
| stbxgr | Steel Box Girder |
| stcubx | Steel Curved Box Girder |
| cntibm | Steel I-Beam (rolled) - Continuous |
| susibm | Steel I-Beam (rolled) - Suspended |
| susplg | Steel Plate Girder - Suspended |
| stcagr | Steel Box Girder - Cable Stayed |

**Concrete Spans**

| | |
|---|---|
| codekg | Concrete Deck Girder |
| ccovsl | Concrete Voided Slab - Continuous |
| coslab | Concrete Slab |
| covslb | Concrete Voided Slab |
| conibm | Concrete Deck & Bents W/Steel I-Beam (Rolled) |
| coribm | Concrete Deck & Bents W/Steel I-Beam W/Removable Span |
| conrch | Concrete Arch |
| copsgr | Concrete Prestressed Girders (AASHTO Type) |
| cpgccd | Concrete Prestressed Girders w/Continuity Diaphragms and w/Continuous Cast-in-Place Deck |
| copcss | Concrete Precast Slab Units |
| cntslb | Concrete Flat Slab - Continuous |
| copsch | Concrete Prestressed Channel Units (Welded) |
| corech | Concrete Precast Reinforced Channel Units (Bolted) |
| copvcd | Concrete Precast Voided Units W/Cast-in-Place deck |
| comwel | Concrete Deck W/Composite Welded I-Beams |
| cntwel | Concrete Deck W/Composite Welded I-Beams - Continuous |
| cobxgr | Concrete Box Girder |
| cobseg | Concrete Box Girder - Segmental |
| cpbxbm | Concrete Prestressed Box Beam |
| pcpssp | Concrete Prestressed Girders W/Precast Monolithic Deck |
| cntcdg | Concrete Deck Girder - Continuous |

# REFERENCES


1. *AASHTO Load Resistance Factor Design Bridge Design Specifications. Third Edition.* American Association of State Highway and Transportation Officials, Washington, D.C, 2004

2. *AASHTO Standard Specifications for Highway Bridges. Sixteenth Edition.* American Association of State Highway and Transportation Officials, Washington, D.C, 1996

3. ACI, ACI Building Code Requirements for Reinforced Concrete (ACI 318-05) and Commentary (ACI 318R -05), American Concrete Institute, Detroit, 2005

4. Aktan. A.E., Miller, R., Shahrooz. B., Zwick. M., Heckenmueller, M., Ho, I.. Hrinko. W., and Toksoy, T., *"Nondestructive and Destructive Testing of a Reinforced Concrete Slab Bridge and Associated Analytical Studies."* Report FHWA/OH-93/017, FHWA, U.S. Department of Transportation, Washington, D.C., December 1992.

5. Cook, R. A., Fagundo, F. E., Rozen, A. D., and Mayer, H., *"Service, Fatigue, and Ultimate Load Evaluation of a Continuous Prestressed Flat-Slab Bridge System."* Transportation Research Record, No.1393. National Academy Press, Washington. D.C., 1993, pp 104-111.

6. Fu, C. C., Elhelbawey, M., Sabin, M.A., and Schelling, D.R., *"Lateral Distribution Factor from Bridge Field Testing."* ASCE Journal of Structural Engineering. Vol. 122, No.9, 1996, pp 1106-1109.

7. Roberts, Saber, Ranadhir, Zhou, *"Effects of Hauling Timber, Lignite Coal, and Coke Fuel on Louisiana Highways and Bridges."* Louisiana Transportation Research Center Report No. 398, Baton Rouge, LA 2005.

8. Saber, A., Roberts, F., *"Cost of Higher Truck Loads on Remaining Safe Life of Louisiana Bridges",* Transportation Research Board 85$^{th}$ Annual Meeting, Washington D.C., January 2006.

9. Saber, A., Roberts, F., Alaywan, W., Zhou, X., *"Impact of Higher Truck Loads on Remaining Safe Life of Louisiana Concrete Bridge Girders"*, Accepted for publication and presentation at the 2006 Concrete Bridge Conference, May, Reno, Nevada.

10. Sanders, W. W. and El1eby, H. A., *"Distribution of Wheel Loads on Highway Bridges."* National Cooperative Highway Research Program Report No.83. Highway Research Board: Washington, D.C., 1970.

11. Sotelino, E. D., Liu, J., Chung, W. and Phuvoravan, K., *"Simplified Load Distribution Factor for Use in LRFD Design."* Report FHWA/IN/JTRP-2004/20, FHWA, U.S. Department of Transportation, Washington, D.C., September, 2004.

# APPENDIX A

# BRIDGES CONSIDERED FOR THIS STUDY

**Table 4**
**Categories for critical bridges**

| Categories | Number of bridges |
|---|---|
| Simple Beam | 59 |
| Continuous | 6 |
| Culvert | 12 |
| Others | 7 |
| Total | 84 |

**Table 5**
**Design load for critical bridges**

| Beam Categories | Design Load HS20-44 | Design Load < HS20-44 | Design Load Unknown | Total Number of bridges |
|---|---|---|---|---|
| Simple | 51 | 4 | 4 | 59 |
| Continuous | 6 | | | 6 |
| Culvert | 10 | 1 | 1 | 12 |
| Others | 4 | 3 | | 7 |
| Total | 71 | 8 | 5 | 84 |

**Table 6**
**List of critical bridges with simple supports**

| Structure No. | Control Section | Structure Type | Total length | Max Span length | No. of Main Spans | Design Load | State Route |
|---|---|---|---|---|---|---|---|
| 03230550707861 | 055-07 | COSLAB | 80 | 20 | 4 | 20 | LA0014 |
| 03230550707862 | 055-07 | COSLAB | 80 | 20 | 4 | 20 | LA0014 |
| 03234240409163 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US 90 |
| 03234240409164 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US 90 |
| 03234240409827 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US 90 |
| 03234240409823 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US90 |
| 61390520202221 | 052-02 | COSLAB | 100 | 20 | 5 | 20 | LA0001 |
| 03230550704151 | 055-07 | COSLAB | 100 | 20 | 5 | 20 | LA0014 |
| 03230550704152 | 055-07 | COSLAB | 100 | 20 | 5 | 20 | LA0014 |
| 03570550606411 | 055-06 | COSLAB | 100 | 20 | 5 | 20 | LA0014 |
| 61398392401031 | 839-24 | COSLAB | 100 | 20 | 5 | 20 | LA3131 |
| 03234240404701 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US0090 |
| 03234240404702 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US0090 |
| 03234240406661 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US0090 |
| 03234240406662 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US0090 |
| 03234240420407 | 424-04 | COSLAB | 100 | 20 | 5 | 20 | US0090 |
| 03234240409826 | 424-04 | COSLAB | 120 | 20 | 6 | 20 | US 90 |
| 03230550705261 | 055-07 | COSLAB | 160 | 20 | 8 | 20 | LA0014 |
| 03230550705262 | 055-07 | COSLAB | 160 | 20 | 8 | 20 | LA0014 |
| 03234240409161 | 424-04 | COSLAB | 100 | 25 | 4 | 20 | US0090 |
| 03234240409162 | 424-04 | COSLAB | 100 | 25 | 4 | 20 | US0090 |
| 61610500711951 | 050-07 | CONIBM | 1392 | 50 | 37 | 20 | LA0001 |
| 03234240405451 | 424-04 | COPSGR | 1022 | 60 | 17 | 20 | US0090 |
| 03234240405452 | 424-04 | COPSGR | 1022 | 60 | 17 | 20 | US0090 |
| 03284240203442 | 424-02 | COPSGR | 342 | 70 | 5 | 20 | US0090 |
| 03284240203441 | 424-02 | COPSGR | 407 | 70 | 6 | 20 | US0090 |
| 61610500711952 | 050-07 | COPSGR | 1393 | 70 | 21 | 20 | LA0001 |
| 03234240408431 | 424-04 | COPSGR | 1402 | 70 | 20 | 20 | US0090 |
| 03234240408432 | 424-04 | COPSGR | 1402 | 70 | 20 | 20 | US0090 |
| 03234240413911 | 424-04 | COPSGR | 1886 | 70 | 28 | 20 | US0090 |
| 03234240413912 | 424-04 | COPSGR | 1886 | 70 | 28 | 20 | US0090 |
| 03234240420272 | 424-04 | COPSGR | 1890 | 70 | 27 | 20 | US0090 |
| 03234240420271 | 424-04 | COPSGR | 1890 | 70 | 27 | 20 | US0090 |
| 61240500614251 | 050-06 | COPSGR | 1458 | 94 | 20 | 20 | LA0001 |

**COSLAB:** **Concrete Slab**
**CONIBM:** **Concrete Deck & Bents W/Steel I-Beam (Rolled)**
**COPSGR:** **Concrete Prestressed Girders (AASHTO Type)**

**Table 7**
**List of critical bridges with continuous supports**

| Structure No. | Control Section | Structure Type | Total length | Max Span length | No. of Main spans | Design Load | State Route |
|---|---|---|---|---|---|---|---|
| 03234240407911 | 424-04 | COMWEL | 320 | 90 | 4 | 20 | US0090 |
| 03234240407912 | 424-04 | COMWEL | 320 | 90 | 4 | 20 | US0090 |
| 03284240210401 | 424-02 | COMWEL | 827 | 90 | 18 | 20 | US0090 |
| 03284240210402 | 424-02 | COMWEL | 747 | 90 | 16 | 20 | US0090 |
| 61610500708321 | 050-07 | STCPLG | 2469 | 200 | 44 | 20 | LA0001 |
| 61610500708322 | 050-07 | STCPLG | 2469 | 200 | 44 | 20 | LA0001 |

**Table 8**
**Stress list of critical bridges with simple supports on I-10**

| Structure No. | Control Section | Structure Type | Support Condition | Total length | Max Span Length | No. of Main spans | Design Load |
|---|---|---|---|---|---|---|---|
| 61244500700001 | 450-07 | COPSGR | Simple | 739 | 70 | 10 | 20 |
| 61244500700651 | 450-07 | COPSGR | Simple | 40329 | 90 | 573 | 20 |
| 61244500700652 | 450-07 | COPSGR | Simple | 40329 | 90 | 573 | 20 |
| 61244500700725 | 450-07 | COPSGR | Simple | 253 | 66 | 4 | 20 |
| 61244500700846 | 450-07 | COPSGR | Simple | 126 | 66 | 2 | 20 |
| 61244500701017 | 450-07 | COPSGR | Simple | 191 | 63 | 3 | 20 |
| 61244500701048 | 450-07 | COPSGR | Simple | 191 | 62 | 3 | 20 |
| 61244500703921 | 450-07 | COPSGR | Simple | 129 | 43 | 3 | 20 |
| 61244500712131 | 450-07 | COPSGR | Simple | 2026 | 78 | 26 | 20 |
| 61244500712132 | 450-07 | COPSGR | Simple | 2026 | 78 | 26 | 20 |
| 61244500712891 | 450-07 | COPSGR | Simple | 1430 | 78 | 26 | 20 |
| 61244500712892 | 450-07 | COPSGR | Simple | 1430 | 78 | 26 | 20 |
| 61244500714231 | 450-07 | COPSGR | Simple | 147 | 48 | 3 | 20 |
| 61244500714232 | 450-07 | COPSGR | Simple | 147 | 48 | 3 | 20 |
| 61244500710251 | 450-07 | COSLAB | Simple | 140 | 28 | 5 | 20 |
| 61244500710252 | 450-07 | COSLAB | Simple | 140 | 28 | 5 | 20 |
| 61244500714233 | 450-07 | COSLAB | Simple | 144 | 29 | 5 | 20 |
| 61244500700141 | 450-07 | SUSPLG | Other | 2707 | 425 | 9 | 20 |

**COMWEL:** **Concrete Deck W/Composite Welded I-Beams**
**STCPLG:** **Steel Plate Girder - Continuous**
**SUSPLG:** **Steel Plate Girder - Suspended**
**COPSGR:** **Concrete Prestressed Girders (AASHTO Type)**

**Table 9**
**Stress list of critical bridges with simple supports design load less than HS20-44**

| Structure No. | Control Section | Structure Type | Total Length | max Span length | No. of Main Spans | Design Load | State Route |
|---|---|---|---|---|---|---|---|
| 03232360206641 | 236-02 | COPCSS | 59 | 19 | 3 | 15 | LA0085 |
| 03234240420404 | 424-04 | COPCSS | 57 | 19 | 3 | 15 | US0090 |
| 03234240409661 | 424-04 | COPSGR | 1452 | 138 | 17 | 18 | US0090 |
| 03234240409662 | 424-04 | COPSGR | 1452 | 138 | 17 | 18 | US0090 |
| 61240500609941 | 050-06 | COSLAB | 100 | 20 | 5 | | LA0001 |
| 61240500609942 | 050-06 | COSLAB | 120 | 20 | 6 | | LA0001 |
| 03234240420403 | 424-04 | COSLAB | 100 | 20 | 5 | | US0090 |
| 03234240420406 | 424-04 | COSLAB | 100 | 20 | 5 | | US0090 |

**Table 10**
**List of critical bridges culvert and other categories**

| Structure No. | Control Section | Structure Type | Support Condition | Total Length | Max Span length | No. Main Span | Post Load Limit | Design Load | State Route |
|---|---|---|---|---|---|---|---|---|---|
| 03230550702991 | 055-07 | CONBOX | Culvert | 32 | 5 | 4 | ----- | 15 | LA0014 |
| 03234240401371 | 424-04 | CONBOX | Culvert | 27 | 8 | 3 | ----- | 20 | US0090 |
| 03234240404931 | 424-04 | CONBOX | Culvert | 26 | 10 | 2 | ----- | 20 | US0090 |
| 03234240410911 | 424-04 | CONBOX | Culvert | 30 | 9 | 3 | ----- | 20 | US0090 |
| 03234240410912 | 424-04 | CONBOX | Culvert | 30 | 9 | 3 | ----- | 20 | US0090 |
| 03234240411331 | 424-04 | CONBOX | Culvert | 44 | 7 | 5 | ----- | 20 | US0090 |
| 03284240212431 | 424-02 | CONBOX | Culvert | 32 | 7 | 4 | ----- | 20 | US0090 |
| 03504240300731 | 424-03 | CONBOX | Culvert | 27 | 8 | 3 | ----- | 20 | US0090 |
| 03570550603201 | 055-06 | CONBOX | Culvert | 22 | 6 | 3 | ----- | 20 | LA00146 |
| 03570550605151 | 055-06 | CONBOX | Culvert | 30 | 6 | 4 | ----- | | LA0014 |
| 03230550707261 | 055-07 | CONPIP | Culvert | 23 | 6 | 3 | ----- | 20 | LA0014 |
| 03230550709141 | 055-07 | CONPIP | Culvert | 30 | 7 | 3 | ----- | 20 | LA0014 |
| 61244500712993 | 450-07 | CORIBM | Other | 200 | 50 | 6 | ----- | 20 | I0010 |
| 61244500712994 | 450-07 | CORIBM | Other | 200 | 50 | 7 | ----- | 20 | I0010 |
| 61240500614452 | 050-06 | STHITR | Other | 459 | 150 | 11 | ----- | 20 | LA0001 |
| 03230550700321 | 055-07 | STVERT | Other | 222 | 102 | 7 | ----- | 18 | LA0014 |
| 03570550600131 | 055-06 | STVERT | Other | 241 | 75 | 5 | 15-25 | 15 | LA0014 |
| 03232360202041 | 236-02 | TTTRES | Other | 40 | 19 | 2 | 20-35 | 10 | LA0085 |

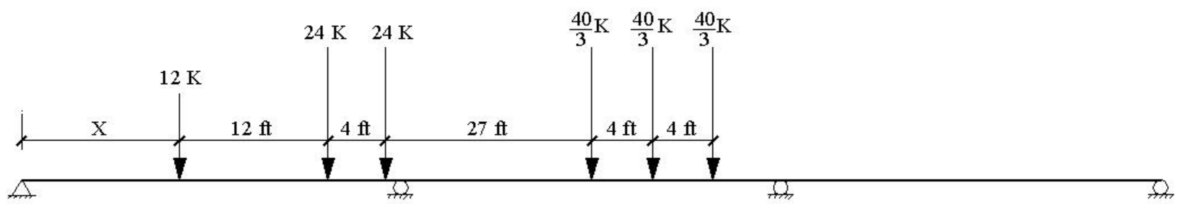
**COPCSS:** **Concrete Precast Slab Units**
**COPSGR** **Concrete Prestressed Girders (AASHTO Type)**
**COSLAB:** **Concrete Slab**
**CONBOX:** **Concrete Box Culvert(s) (over 20 ft total opening)**
**CONPIP:** **Concrete Pipe Culvert(s) (over 20 ft total opening)**
**CORIBM:** **Concrete Deck & Bents W/Steel I-Beam W/Removable Span**
**STVERT:** **Steel Vertical Lift Span**
**TTTRES:** **Untreated Timber Trestle**

# APPENDIX B

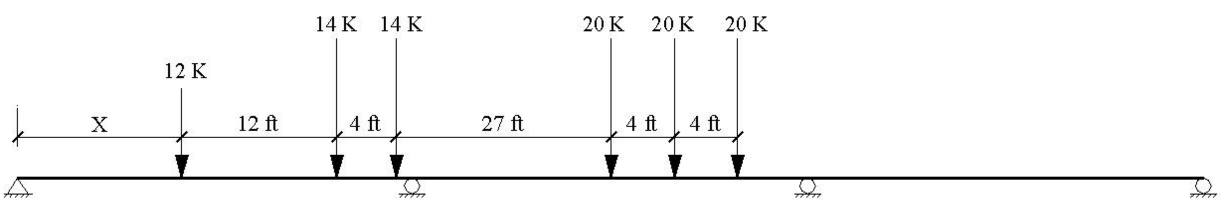
## BRIDGE GIRDER EVALUATION RESULTS

The following four scenarios of sugar cane truck load configurations are considered:

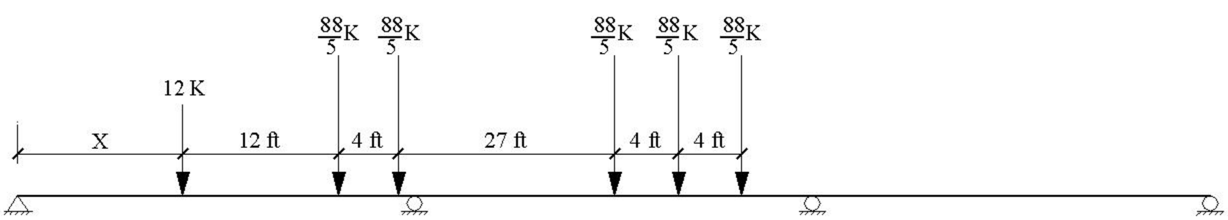
Case 1: GVW=100 k with max tandem load of 48 k and max tridem load of 40 k;
Case 2: GVW=100 k with max tandem load of 28 k and max tridem load of 60 k;
Case 3: GVW=100 k with uniformly distributed tandem and tridem loads;
Case 4: GVW=120 k with max tandem load of 48 k and max tridem load of 60 k.



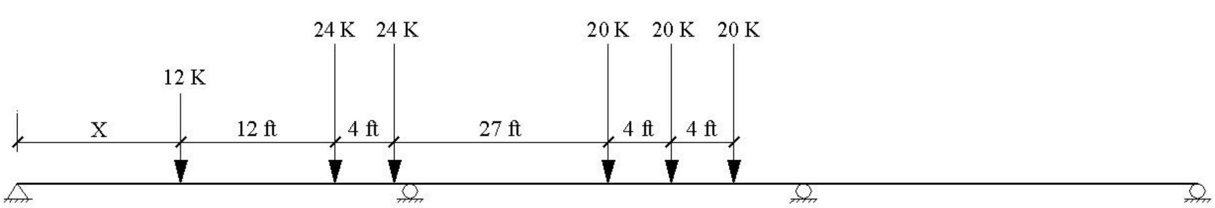

(a) Case 1 GVW=100 Kip with Max Tandem Load 48 Kip and Max Tridem Load 40 Kip




(b) Case 2 GVW=100 Kip with Max Tandem Load 28 Kip and Max Tridem Load 60 Kip




(c) Case 3 GVW=100 Kip with Uniformly Distributed Tandem and Tridem Loads.




(d) Case 4 GVW=120 Kip with Max Tandem Load 48 Kip and Max Tridem Load 60 Kip

**Table 11**
**Absolute maximum moment and shear for simple girders due to 3S3 truck**

| Span | Moment (K-ft) | | | | Shear (Kips) | | | |
|---|---|---|---|---|---|---|---|---|
| ( ft ) | Case 1 | Case 2 | Case 3 | Case 4 | Case 1 | Case 2 | Case 3 | Case 4 |
| 19 | 181.90 | 204.21 | 179.71 | 204.21 | 41.68 | 44.21 | 38.91 | 44.21 |
| 20 | 194.40 | 220.00 | 193.60 | 220.00 | 42.60 | 45.00 | 39.60 | 45.00 |
| 25 | 253.44 | 294.40 | 259.07 | 294.40 | 46.08 | 48.00 | 42.24 | 48.00 |
| 28 | 299.57 | 337.85 | 297.31 | 337.85 | 47.57 | 49.29 | 43.37 | 49.29 |
| 29 | 315.66 | 354.48 | 311.94 | 354.48 | 48.00 | 49.66 | 43.70 | 49.66 |
| 43 | 525.21 | 564.65 | 496.89 | 564.65 | 51.91 | 56.28 | 50.75 | 58.60 |
| 48 | 599.75 | 638.75 | 562.10 | 638.75 | 55.00 | 59.58 | 54.63 | 63.75 |
| 50 | 629.76 | 668.80 | 588.54 | 668.80 | 56.32 | 60.72 | 55.97 | 65.52 |
| 60 | 780.62 | 820.00 | 721.60 | 820.00 | 61.60 | 66.87 | 62.91 | 74.20 |
| 62 | 823.57 | 861.03 | 777.81 | 907.35 | 62.45 | 67.94 | 64.10 | 75.68 |
| 63 | 846.86 | 882.41 | 799.54 | 933.71 | 62.86 | 68.44 | 64.67 | 76.38 |
| 66 | 920.24 | 946.67 | 864.00 | 1012.36 | 64.00 | 69.88 | 66.28 | 78.36 |
| 70 | 1018.29 | 1032.57 | 950.40 | 1117.71 | 65.37 | 71.60 | 68.21 | 80.74 |
| 78 | 1214.97 | 1205.03 | 1132.14 | 1340.31 | 67.69 | 74.51 | 71.47 | 84.77 |
| 90 | 1511.11 | 1498.67 | 1424.66 | 1690.40 | 70.58 | 77.91 | 75.27 | 89.47 |
| 94 | 1610.04 | 1597.28 | 1522.59 | 1807.66 | 71.79 | 78.85 | 76.32 | 90.77 |

**Table 12**
**Maximum deflection for simple girders due to 3S3 truck**

| Span | Deflection*EI (Kips*ft$^3$) | | | |
|---|---|---|---|---|
| ( ft ) | Case 1 | Case 2 | Case 3 | Case 4 |
| 19 | -6.3983E+03 | -7.2239E+03 | -6.3570E+03 | -7.2239E+03 |
| 20 | -7.5520E+03 | -8.6133E+03 | -7.5797E+03 | -8.6133E+03 |
| 25 | -1.5008E+04 | -1.7685E+04 | -1.5563E+04 | -1.7685E+04 |
| 28 | -2.1864E+04 | -2.5245E+04 | -2.2216E+04 | -2.5245E+04 |
| 29 | -2.4684E+04 | -2.8310E+04 | -2.4913E+04 | -2.8310E+04 |
| 43 | -8.9921E+04 | -9.6051E+04 | -8.4525E+04 | -9.6051E+04 |
| 48 | -1.2728E+05 | -1.3430E+05 | -1.1818E+05 | -1.3430E+05 |
| 50 | -1.4476E+05 | -1.5213E+05 | -1.3387E+05 | -1.5213E+05 |
| 60 | -2.6024E+05 | -2.7201E+05 | -2.5489E+05 | -3.0604E+05 |
| 62 | -2.9876E+05 | -3.0740E+05 | -2.8983E+05 | -3.4877E+05 |
| 63 | -3.1889E+05 | -3.2618E+05 | -3.0829E+05 | -3.7170E+05 |
| 66 | -3.8440E+05 | -3.8644E+05 | -3.6802E+05 | -4.4955E+05 |
| 70 | -4.8323E+05 | -4.7727E+05 | -4.6540E+05 | -5.9796E+05 |
| 78 | -7.2369E+05 | -7.1027E+05 | -7.0343E+05 | -8.5521E+05 |
| 90 | -1.2037E+06 | -1.1863E+06 | -1.1799E+06 | -1.4301E+06 |
| 94 | -1.3986E+06 | -1.3800E+06 | -1.3736E+06 | -1.6634E+06 |

**Table 13**
**Absolute maximum moment and shear for simple girders due to HS20-44 Truck**

| Span | Moment | Shear | Deflection*EI |
|---|---|---|---|
| (ft) | (K-ft) | (Kips) | (Kips*ft$^3$) |
| 19 | 151.58 | 37.05 | -4.5474E+03 |
| 20 | 160.00 | 41.60 | -5.3333E+03 |
| 25 | 207.40 | 46.10 | -1.0921E+04 |
| 28 | 252.00 | 48.00 | -1.7493E+04 |
| 29 | 267.00 | 48.80 | -2.0117E+04 |
| 43 | 503.07 | 54.70 | -9.2263E+04 |
| 48 | 592.20 | 58.00 | -1.3100E+05 |
| 50 | 627.80 | 58.60 | -1.5079E+05 |
| 60 | 806.50 | 60.80 | -2.7816E+05 |
| 62 | 842.30 | 61.20 | -3.0983E+05 |
| 63 | 860.20 | 61.30 | -3.3237E+05 |
| 66 | 913.90 | 61.80 | -3.7993E+05 |
| 70 | 985.60 | 62.40 | -4.5953E+05 |
| 78 | 1128.92 | 63.40 | -6.5694E+05 |
| 90 | 1344.40 | 64.50 | -1.0203E+06 |
| 94 | 1434.10 | 64.90 | -1.2082E+06 |

**Table 14**
**Critical location for truck loads on continuous girders**

| Span Length | HS 20-44 | | | | 3S3 | | |
|---|---|---|---|---|---|---|---|
| (ft) | Truck Location X (ft) | | | | Truck Location X (ft) | | |
| | From Left Support to Front Tire | | | | From Left Support to Front Tire | | |
| | Max Positive Moment | Max Negative Moment | Max Absolute Shear | | Max Positive Moment | Max Negative Moment | Max Absolute Shear |
| 60 | 10 | 15 | 31 | Case 1 | 8RL | 28RL | 71LR |
| | | | | Case 2 | 69LR | 86LR | 8RL |
| | | | | Case 3 | 68LR | 87LR | 8RL |
| | | | | Case 4 | 68LR | 88LR | 8RL |
| 65 | 12 | 18 | 36 | Case 1 | 10RL | 33RL | 76RL |
| | | | | Case 2 | 71LR | 91LR | 13RL |
| | | | | Case 3 | 70LR | 92LR | 13RL |
| | | | | Case 4 | 70LR | 93LR | 13RL |
| 70 | 14 | 21 | 41 | Case 1 | 12RL | 37RL | 81LR |
| | | | | Case 2 | 73LR | 95LR | 18RL |
| | | | | Case 3 | 72LR | 96LR | 18RL |
| | | | | Case 4 | 69LR | 97LR | 18RL |
| 90 | 23 | 32 | 61 | Case 1 | 20RL | 74LR | 89LR |
| | | | | Case 2 | 81LR | 16RL | 38RL |
| | | | | Case 3 | 78LR | 19RL | 38RL |
| | | | | Case 4 | 77LR | 19RL | 38RL |

Note: LR--Truck moves from left to right; RL--Truck moves from right to left

**Table 15**
**Maximum moment for continuous girders due to 3S3 truck loads**

| Span (ft) | Maximum Positive Moment (Kip-ft) | | | | Maximum Negative Moment (Kip-ft) | | | |
|---|---|---|---|---|---|---|---|---|
| | Case 1 | Case 2 | Case 3 | Case 4 | Case 1 | Case 2 | Case 3 | Case 4 |
| 60 | 628.60 | 659.20 | 588.90 | 680.20 | -511.1 | -518.2 | -504.3 | -598.1 |
| 65 | 718.60 | 743.30 | 671.00 | 778.30 | -539.9 | -548.4 | -532.4 | -630.1 |
| 70 | 810.60 | 830.00 | 755.90 | 882.90 | -565.5 | -575.4 | -557.1 | -658.1 |
| 90 | 1191.60 | 1195.50 | 1124.00 | 1328.20 | -763.2 | -758.5 | -751.6 | -907.0 |

**Table 16**
**Maximum shear forces for continuous girders due to 3S3 truck loads**

| Span (ft) | Maximum Positive Shear (Kip) | | | | Maximum Negative Shear (Kip) | | | |
|---|---|---|---|---|---|---|---|---|
| | Case 1 | Case 2 | Case 3 | Case 4 | Case 1 | Case 2 | Case 3 | Case 4 |
| 60 | 62.71 | 67.08 | 62.84 | 74.01 | -67.02 | -71.86 | -68.24 | -80.88 |
| 65 | 64.91 | 69.73 | 65.84 | 77.76 | -69.16 | -74.66 | -71.41 | -84.79 |
| 70 | 66.82 | 72.06 | 68.49 | 81.06 | -70.95 | -77.01 | -74.07 | -88.08 |
| 90 | 72.29 | 78.97 | 76.32 | 90.78 | -77.72 | -83.46 | -81.37 | -97.07 |

**Table 17**
**Ratio of 3S3/HS20-44 truck of moment and shear for simple girders**

| **Span** | **Moment Ratio** | | | | **Shear Ratio** | | | |
|---|---|---|---|---|---|---|---|---|
| (ft) | Case 1 | Case 2 | Case 3 | Case 4 | Case 1 | Case 2 | Case 3 | Case 4 |
| 19 | 1.20 | 1.35 | 1.19 | 1.35 | 1.12 | 1.19 | 1.05 | 1.19 |
| 20 | 1.22 | 1.38 | 1.21 | 1.38 | 1.02 | 1.08 | 0.95 | 1.08 |
| 25 | 1.22 | 1.42 | 1.25 | 1.42 | 1.00 | 1.04 | 0.92 | 1.04 |
| 28 | 1.19 | 1.34 | 1.18 | 1.34 | 0.99 | 1.03 | 0.90 | 1.03 |
| 29 | 1.18 | 1.33 | 1.17 | 1.33 | 0.98 | 1.02 | 0.90 | 1.02 |
| 43 | 1.04 | 1.12 | 0.99 | 1.12 | 0.95 | 1.03 | 0.93 | 1.07 |
| 48 | 1.01 | 1.08 | 0.95 | 1.08 | 0.95 | 1.03 | 0.94 | 1.10 |
| 50 | 1.00 | 1.07 | 0.94 | 1.07 | 0.96 | 1.04 | 0.96 | 1.12 |
| 60 | 0.97 | 1.02 | 0.89 | 1.02 | 1.01 | 1.10 | 1.03 | 1.22 |
| 62 | 0.98 | 1.02 | 0.92 | 1.08 | 1.02 | 1.11 | 1.05 | 1.24 |
| 63 | 0.98 | 1.03 | 0.93 | 1.09 | 1.03 | 1.12 | 1.05 | 1.25 |
| 66 | 1.01 | 1.04 | 0.95 | 1.11 | 1.04 | 1.13 | 1.07 | 1.27 |
| 70 | 1.03 | 1.05 | 0.96 | 1.13 | 1.05 | 1.15 | 1.09 | 1.29 |
| 78 | 1.08 | 1.07 | 1.00 | 1.19 | 1.07 | 1.18 | 1.13 | 1.34 |
| 90 | 1.12 | 1.11 | 1.06 | 1.26 | 1.09 | 1.21 | 1.17 | 1.39 |
| 94 | 1.12 | 1.11 | 1.06 | 1.26 | 1.11 | 1.21 | 1.18 | 1.40 |

**Table 18**
**Ratio of 3S3/HS20-44 truck of deflection for simple girders**

| Span | Deflection Ratio | | | |
|---|---|---|---|---|
| (ft) | Case 1 | Case 2 | Case 3 | Case 4 |
| 19 | 1.41 | 1.59 | 1.40 | 1.59 |
| 20 | 1.42 | 1.62 | 1.42 | 1.62 |
| 25 | 1.37 | 1.62 | 1.43 | 1.62 |
| 28 | 1.25 | 1.44 | 1.27 | 1.44 |
| 29 | 1.23 | 1.41 | 1.24 | 1.41 |
| 43 | 0.97 | 1.04 | 0.92 | 1.04 |
| 48 | 0.97 | 1.03 | 0.90 | 1.03 |
| 50 | 0.96 | 1.01 | 0.89 | 1.01 |
| 60 | 0.94 | 0.98 | 0.92 | 1.10 |
| 62 | 0.96 | 0.99 | 0.94 | 1.13 |
| 63 | 0.96 | 0.98 | 0.93 | 1.12 |
| 66 | 1.01 | 1.02 | 0.97 | 1.18 |
| 70 | 1.05 | 1.04 | 1.01 | 1.30 |
| 78 | 1.10 | 1.08 | 1.07 | 1.30 |
| 90 | 1.18 | 1.16 | 1.16 | 1.40 |
| 94 | 1.16 | 1.14 | 1.14 | 1.38 |

**Table 19**
**Ratio of 3S3/HS20-44 truck of moment for continuous girders**

| Span | Maximum Positive Moment Ratio | | | | Maximum Negative Moment Ratio | | | |
|---|---|---|---|---|---|---|---|---|
| (ft) | Case 1 | Case 2 | Case 3 | Case 4 | Case 1 | Case 2 | Case 3 | Case 4 |
| 60 | 0.99 | 1.04 | 0.93 | 1.07 | 1.28 | 1.30 | 1.27 | 1.50 |
| 65 | 1.02 | 1.05 | 0.95 | 1.10 | 1.23 | 1.25 | 1.21 | 1.44 |
| 70 | 1.04 | 1.06 | 0.97 | 1.13 | 1.18 | 1.20 | 1.17 | 1.38 |
| 90 | 1.11 | 1.12 | 1.05 | 1.24 | 1.20 | 1.20 | 1.19 | 1.43 |

**Table 20**
**Ratio of 3S3/HS20-44 truck of shear for continuous girders**

| Span | Maximum Absolute Shear Ratio | | | |
|---|---|---|---|---|
| (ft) | Case 1 | Case 2 | Case 3 | Case 4 |
| 60 | 1.06 | 1.13 | 1.07 | 1.27 |
| 65 | 1.08 | 1.16 | 1.11 | 1.32 |
| 70 | 1.09 | 1.18 | 1.14 | 1.36 |
| 90 | 1.16 | 1.25 | 1.22 | 1.45 |

# APPENDIX C

# BRIDGE DECK EVALUATION RESULTS

**Table 21**
**Stresses at top surface of continuous bridge deck 3S3 truck load**

| Span (ft) | Max Tensile Stress (Ksi) | | | Max Compressive Stress (Ksi) | | |
|---|---|---|---|---|---|---|
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| 20 | 0.0571 | 0.1794 | 0.0543 | -0.2208 | -0.3108 | -0.0566 |
| 30 | 0.1214 | 0.1411 | 0.0564 | -0.2009 | -0.4028 | -0.0590 |
| 60 | 0.3266 | 0.1458 | 0.0675 | -0.2209 | -0.4943 | -0.0781 |
| 75 | 0.3425 | 0.2022 | 0.0921 | -0.2673 | -0.5610 | -0.0933 |
| 90 | 0.5172 | 0.2632 | 0.1059 | -0.4317 | -0.6135 | -0.1224 |
| 105 | 0.7233 | 0.3359 | 0.1288 | -0.3256 | -0.6516 | -0.1315 |

**Table 22**
**Stresses at bottom surface of continuous bridge deck 3S3 truck load**

| Span (ft) | Max Tensile Stress (Ksi) | | | Max Compressive Stress (Ksi) | | |
|---|---|---|---|---|---|---|
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| 20 | 0.2208 | 0.3179 | 0.0566 | -0.0571 | -0.1794 | -0.0543 |
| 30 | 0.2009 | 0.4028 | 0.0590 | -0.1214 | -0.1411 | -0.0564 |
| 60 | 0.2209 | 0.4943 | 0.0781 | -0.3266 | -0.1458 | -0.0675 |
| 75 | 0.2673 | 0.5610 | 0.0933 | -0.3425 | -0.2022 | -0.0921 |
| 90 | 0.4317 | 0.6135 | 0.1224 | -0.5172 | -0.2632 | -0.1060 |
| 105 | 0.3256 | 0.6516 | 0.1314 | -0.7233 | -0.3359 | -0.1288 |

**Table 23**
**Stresses at top surface of continuous bridge deck HS20-44 truck load**

| Span (ft) | Max Tensile Stress (Ksi) | | | Max Compressive Stress (Ksi) | | |
|---|---|---|---|---|---|---|
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| 20 | 0.0600 | 0.2160 | 0.0380 | -0.3090 | -0.3290 | -0.0380 |
| 30 | 0.0990 | 0.1930 | 0.0450 | -0.3530 | -0.4240 | -0.0570 |
| 60 | 0.1840 | 0.1270 | 0.0750 | -0.3370 | -0.4950 | -0.0650 |
| 75 | 0.2540 | 0.1460 | 0.0680 | -0.3520 | -0.5340 | -0.0660 |
| 90 | 0.3630 | 0.1910 | 0.0750 | -0.3630 | -0.5540 | -0.0860 |
| 105 | 0.4760 | 0.2310 | 0.0880 | -0.3730 | -0.5640 | -0.0840 |

**Table 24**
**Stresses at bottom surface of continuous bridge deck HS20-44 truck load**

| Span (ft) | Max Tensile Stress (Ksi) | | | Max Compressive Stress (Ksi) | | |
|---|---|---|---|---|---|---|
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| 20 | 0.3090 | 0.3290 | 0.0380 | -0.0600 | -0.2160 | -0.0380 |
| 30 | 0.3530 | 0.4240 | 0.0570 | -0.0990 | -0.1930 | -0.0450 |
| 60 | 0.3370 | 0.4950 | 0.0650 | -0.1840 | -0.1270 | -0.0750 |
| 75 | 0.3520 | 0.5340 | 0.0660 | -0.2540 | -0.1460 | -0.0680 |
| 90 | 0.3630 | 0.5540 | 0.0860 | -0.3630 | -0.1910 | -0.0750 |
| 105 | 0.3730 | 0.5640 | 0.0840 | -0.4760 | -0.2310 | -0.0880 |

**Table 25**
**Ratio of stresses at top surface of continuous bridge deck 3S3/HS20-44 truck**

| Span (ft) | Ratio of Max Tensile Stress | | | Ratio of Max Compressive Stress | | |
|---|---|---|---|---|---|---|
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| 20 | 0.9517 | 0.8306 | 1.4289 | 0.7146 | 0.9447 | 1.4895 |
| 30 | 1.2263 | 0.7311 | 1.2533 | 0.5691 | 0.9500 | 1.0351 |
| 60 | 1.7750 | 1.1480 | 0.9000 | 0.6555 | 0.9986 | 1.2015 |
| 75 | 1.3484 | 1.3849 | 1.3544 | 0.7594 | 1.0506 | 1.4136 |
| 90 | 1.4248 | 1.3780 | 1.4120 | 1.1893 | 1.1074 | 1.4233 |
| 105 | 1.5195 | 1.4541 | 1.4636 | 0.8729 | 1.1553 | 1.5655 |

**Table 26**
**Ratio of stresses at bottom surface of continuous bridge deck 3S3/HS20-44 truck**

| Span (ft) | Ratio of Max Tensile Stress | | | Ratio of Max Compressive Stress | | |
|---|---|---|---|---|---|---|
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| 20 | 0.7146 | 0.9663 | 1.4895 | 0.9517 | 0.8306 | 1.4289 |
| 30 | 0.5691 | 0.9500 | 1.0351 | 1.2263 | 0.7311 | 1.2533 |
| 60 | 0.6555 | 0.9986 | 1.2015 | 1.7750 | 1.1480 | 0.9000 |
| 75 | 0.7594 | 1.0506 | 1.4136 | 1.3484 | 1.3849 | 1.3544 |
| 90 | 1.1893 | 1.1074 | 1.4233 | 1.4248 | 1.3780 | 1.4133 |
| 105 | 0.8729 | 1.1553 | 1.5643 | 1.5195 | 1.4541 | 1.4636 |

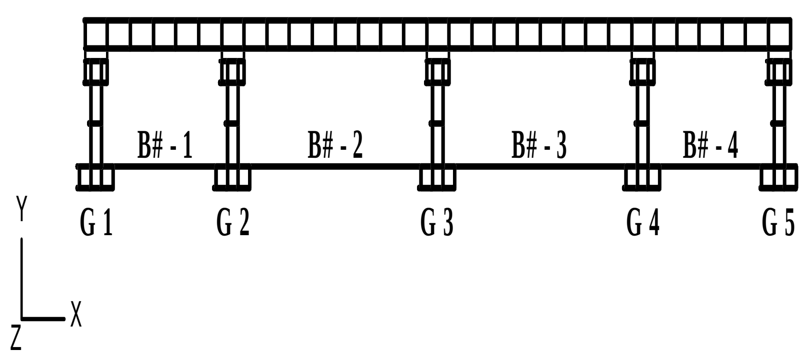



**Figure 8**
**Models used for bridge deck analysis**

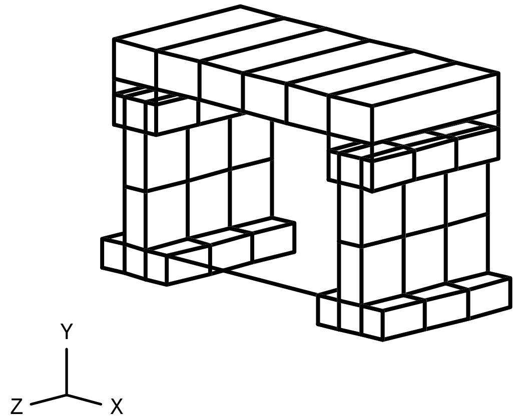



**Figure 9**
**Typical plate and girder elements**

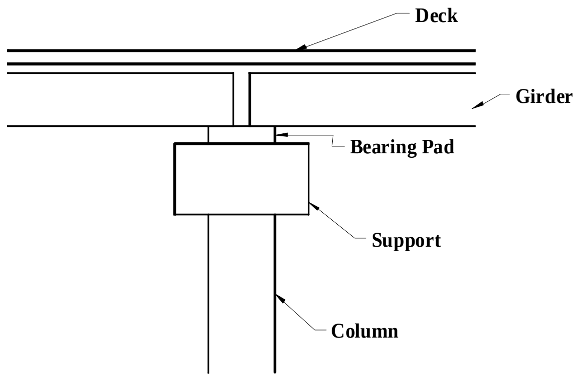



**Figure 10**
**Elevation view of girders over interior support**

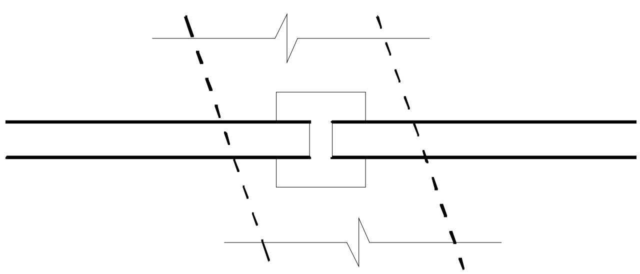


**Figure 11**
**Plan view of girders over interior support**

# APPENDIX D

# FATIGUE COST STUDY FOR NON-INTERSTATE BRIDGES

**Table 27**
**Results for fatigue cost study for state bridges based on flexural analysis**

| Load Case | Truck Weight | Fatigue Cost per Trip per Bridge |
|---|---|---|
| Case 4 | GVW 120 kip | $11.75 |
| Case 3 | GVW 100 kip Uniform | $0.90 |

**Table 28**
**Fatigue cost for simple girder bridges based on flexural analysis**

3S3 Trucks with GVW 120 kip, and $90/Ft$^2$

| Span Length | Number of Main Spans | Number of Bridges | Total Length | Total Length * # of Bridges | Ratio from Analysis | % of Life | Cost per Trip | Cost per Trip * # of Bridges * Total Length |
|---|---|---|---|---|---|---|---|---|
| (ft) | | | (ft) | | Moment | E-06 | $ | $ |
| 20 | 4 | 2 | 80 | 160 | 1.38 | 5.76 | 1.24 | 199.07 |
| 20 | 5 | 14 | 100 | 1400 | 1.38 | 5.76 | 1.56 | 2177.34 |
| 20 | 6 | 1 | 120 | 120 | 1.38 | 5.76 | 1.87 | 223.95 |
| 20 | 8 | 2 | 160 | 320 | 1.38 | 5.76 | 2.49 | 796.28 |
| 25 | 4 | 2 | 100 | 200 | 1.42 | 6.28 | 1.69 | 338.89 |
| 50 | 37 | 1 | 1392 | 1392 | 1.07 | 2.69 | 10.09 | 14047.23 |
| 60 | 17 | 2 | 1020 | 2040 | 1.02 | 2.33 | 6.41 | 13067.48 |
| 70 | 5 | 1 | 342 | 342 | 1.13 | 3.16 | 2.92 | 998.73 |
| 70 | 6 | 1 | 407 | 407 | 1.13 | 3.16 | 3.48 | 1414.44 |
| 70 | 21 | 1 | 1393 | 1393 | 1.13 | 3.16 | 11.89 | 16569.08 |
| 70 | 20 | 2 | 1402 | 2804 | 1.13 | 3.16 | 11.97 | 33567.75 |
| 70 | 28 | 2 | 1886 | 3772 | 1.13 | 3.16 | 16.10 | 60744.87 |
| 70 | 27 | 2 | 1890 | 3780 | 1.13 | 3.16 | 16.14 | 61002.81 |
| 94 | 20 | 1 | 1458 | 1458 | 1.26 | 4.38 | 17.26 | 25164.46 |
| Sum | | 34 | | 19588 | | | | 230312.3 |
| Weighted Average Cost per Trip per Bridge | | | | | | | | **$11.75** |

**Table 29**
**Fatigue cost for simple girder bridges based on shear analysis**

3S3 Trucks with GVW 120 kip, and $90/Ft$^2$

| Span Length | Number of Main Spans | Number of Bridges | Total Length | Total Length * # of Bridges | Ratio from Analysis | % of Life | Cost per Trip | Cost per Trip * # of Bridges * Total Length |
|---|---|---|---|---|---|---|---|---|
| (ft) | | | (ft) | | Shear | E-06 | $ | $ |
| 20 | 4 | 2 | 80 | 160 | 1.08 | 2.76 | 0.60 | 95.42 |
| 20 | 5 | 14 | 100 | 1400 | 1.08 | 2.76 | 0.75 | 1043.66 |
| 20 | 6 | 1 | 120 | 120 | 1.08 | 2.76 | 0.89 | 107.35 |
| 20 | 8 | 2 | 160 | 320 | 1.08 | 2.76 | 1.19 | 381.68 |
| 25 | 4 | 2 | 100 | 200 | 1.04 | 2.47 | 0.67 | 133.13 |
| 50 | 37 | 1 | 1392 | 1392 | 1.12 | 3.08 | 11.57 | 16109.92 |
| 60 | 17 | 2 | 1020 | 2040 | 1.22 | 3.98 | 10.96 | 22359.94 |
| 70 | 5 | 1 | 342 | 342 | 1.29 | 4.71 | 4.34 | 1485.87 |
| 70 | 6 | 1 | 407 | 407 | 1.29 | 4.71 | 5.17 | 2104.35 |
| 70 | 21 | 1 | 1393 | 1393 | 1.29 | 4.71 | 17.70 | 24650.87 |
| 70 | 20 | 2 | 1402 | 2804 | 1.29 | 4.71 | 17.81 | 49940.86 |
| 70 | 28 | 2 | 1886 | 3772 | 1.29 | 4.71 | 23.96 | 90373.98 |
| 70 | 27 | 2 | 1890 | 3780 | 1.29 | 4.71 | 24.01 | 90757.73 |
| 94 | 20 | 1 | 1458 | 1458 | 1.4 | 6.01 | 23.68 | 34519.15 |
| Sum | | 34 | | 19588 | | | | 334063.9 |
| Weighted Average Cost per Trip per Bridge | | | | | | | | **$17.05** |

**Table 30**
**Fatigue cost for simple girder bridges based on flexural analysis**

3S3 Trucks with GVW 100 kip Uniform, and $90/Ft$^2$

| Span Length | Number of Main Spans | Number of Bridges | Total Length | Total Length * # of Bridges | Ratio from Analysis | % of Life | Cost per Trip | Cost per Trip * # of Bridges * Total Length |
|---|---|---|---|---|---|---|---|---|
| (ft) | | | (ft) | | Moment | E-06 | $ | $ |
| 20 | 4 | 2 | 80 | 160 | 1.21 | 3.88 | 0.84 | 134.19 |
| 20 | 5 | 14 | 100 | 1400 | 1.21 | 3.88 | 1.05 | 1467.73 |
| 20 | 6 | 1 | 120 | 120 | 1.21 | 3.88 | 1.26 | 150.97 |
| 20 | 8 | 2 | 160 | 320 | 1.21 | 3.88 | 1.68 | 536.77 |
| 25 | 4 | 2 | 100 | 200 | 1.25 | 4.28 | 1.16 | 231.16 |
| 50 | 37 | 1 | 1392 | 1392 | 0.94 | 0 | 0.00 | 0.00 |
| 60 | 17 | 2 | 1020 | 2040 | 0.89 | 0 | 0.00 | 0.00 |
| 70 | 5 | 1 | 342 | 342 | 0.96 | 0 | 0.00 | 0.00 |
| 70 | 6 | 1 | 407 | 407 | 0.96 | 0 | 0.00 | 0.00 |
| 70 | 21 | 1 | 1393 | 1393 | 0.96 | 0 | 0.00 | 0.00 |
| 70 | 20 | 2 | 1402 | 2804 | 0.96 | 0 | 0.00 | 0.00 |
| 70 | 28 | 2 | 1886 | 3772 | 0.96 | 0 | 0.00 | 0.00 |
| 70 | 27 | 2 | 1890 | 3780 | 0.96 | 0 | 0.00 | 0.00 |
| 94 | 20 | 1 | 1458 | 1458 | 1.06 | 2.61 | 10.28 | 14982.82 |
| Sum | | 34 | | 19588 | | | | 17503.64 |
| Weighted Average Cost per Trip per Bridge | | | | | | | | **$0.90** |

**Table 31**
**Fatigue cost for simple girder bridges based on shear analysis**

3S3 Trucks with GVW 100 kip Uniform, and $90/Ft$^2$

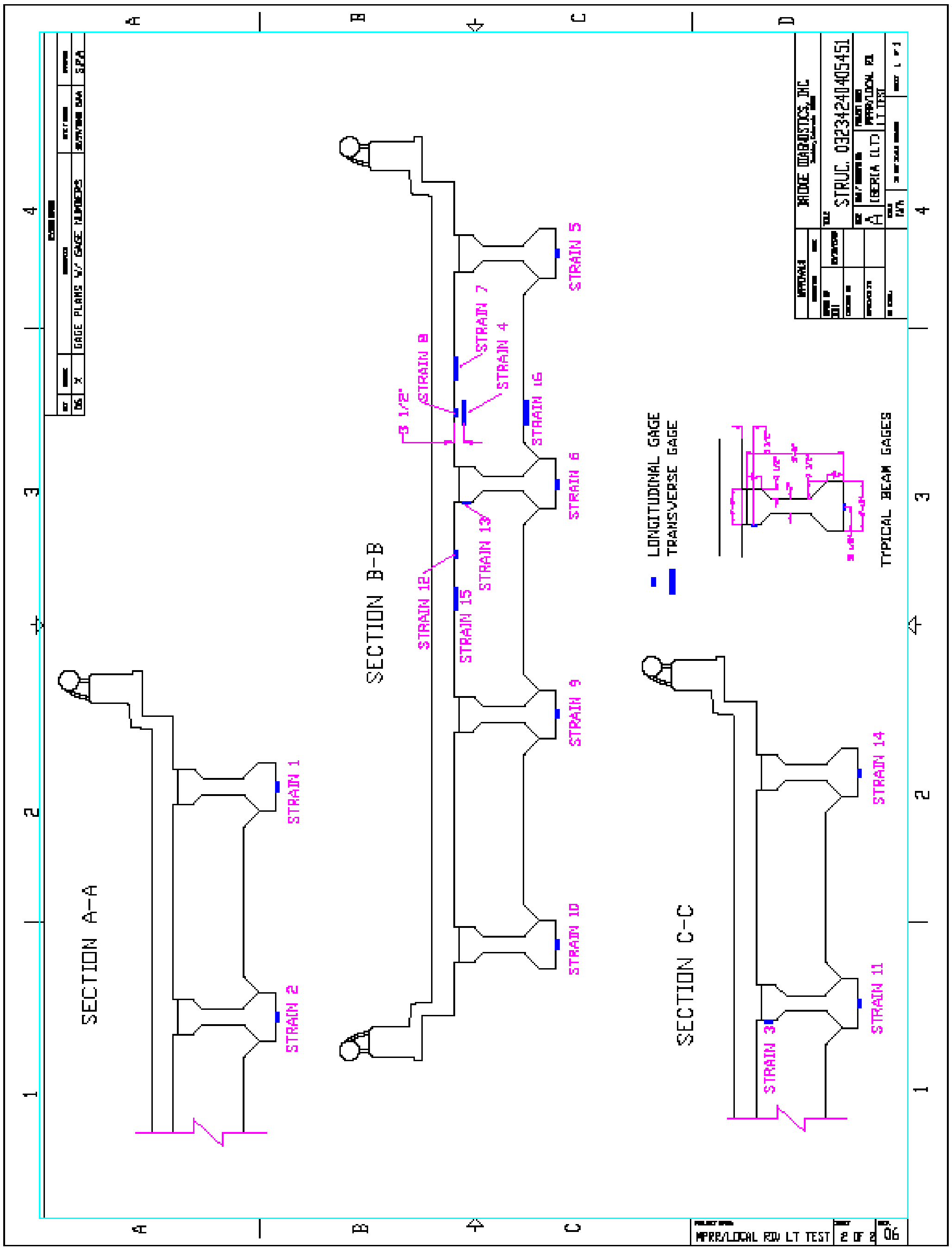
| Span Length | Number of Main Spans | Number of Bridges | Total Length | Total Length * # of Bridges | Ratio from Analysis | % of Life | Cost per Trip | Cost per Trip * # of Bridges * Total Length |
|---|---|---|---|---|---|---|---|---|
| (ft) | | | (ft) | | Shear | E-06 | $ | $ |
| 20 | 4 | 2 | 80 | 160 | 0.95 | 0 | 0 | 0 |
| 20 | 5 | 14 | 100 | 1400 | 0.95 | 0 | 0 | 0 |
| 20 | 6 | 1 | 120 | 120 | 0.95 | 0 | 0 | 0 |
| 20 | 8 | 2 | 160 | 320 | 0.95 | 0 | 0 | 0 |
| 25 | 4 | 2 | 100 | 200 | 0.92 | 0 | 0 | 0 |
| 50 | 37 | 1 | 1392 | 1392 | 0.96 | 0 | 0 | 0 |
| 60 | 17 | 2 | 1020 | 2040 | 1.03 | 2.4 | 6.60 | 13455.59 |
| 70 | 5 | 1 | 342 | 342 | 1.09 | 2.84 | 2.62 | 896.38 |
| 70 | 6 | 1 | 407 | 407 | 1.09 | 2.84 | 3.12 | 1269.49 |
| 70 | 21 | 1 | 1393 | 1393 | 1.09 | 2.84 | 10.68 | 14871.08 |
| 70 | 20 | 2 | 1402 | 2804 | 1.09 | 2.84 | 10.74 | 30127.73 |
| 70 | 28 | 2 | 1886 | 3772 | 1.09 | 2.84 | 14.45 | 54519.74 |
| 70 | 27 | 2 | 1890 | 3780 | 1.09 | 2.84 | 14.48 | 54751.24 |
| 94 | 20 | 1 | 1458 | 1458 | 1.18 | 3.6 | 14.18 | 20669.12 |
| Sum | | 34 | | 19588 | | | | 190560.4 |
| Weighted Average Cost per Trip per Bridge | | | | | | | | **$9.73** |

# APPENDIX E

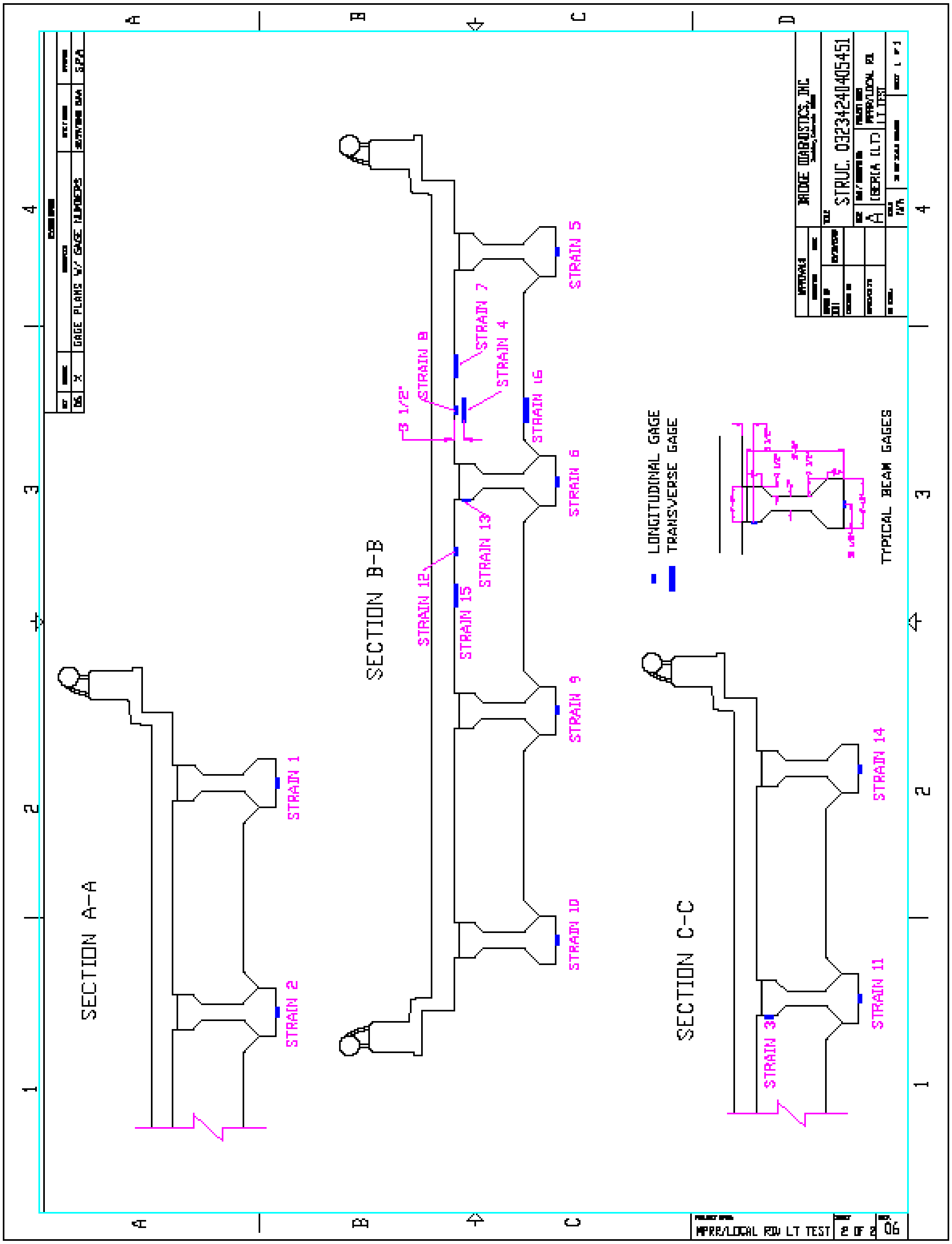
## INSTRUMENTATION PLANS FOR MONITORING SYSTEM

for

Bridge Structure Number 03234240405451 Span Number 14.

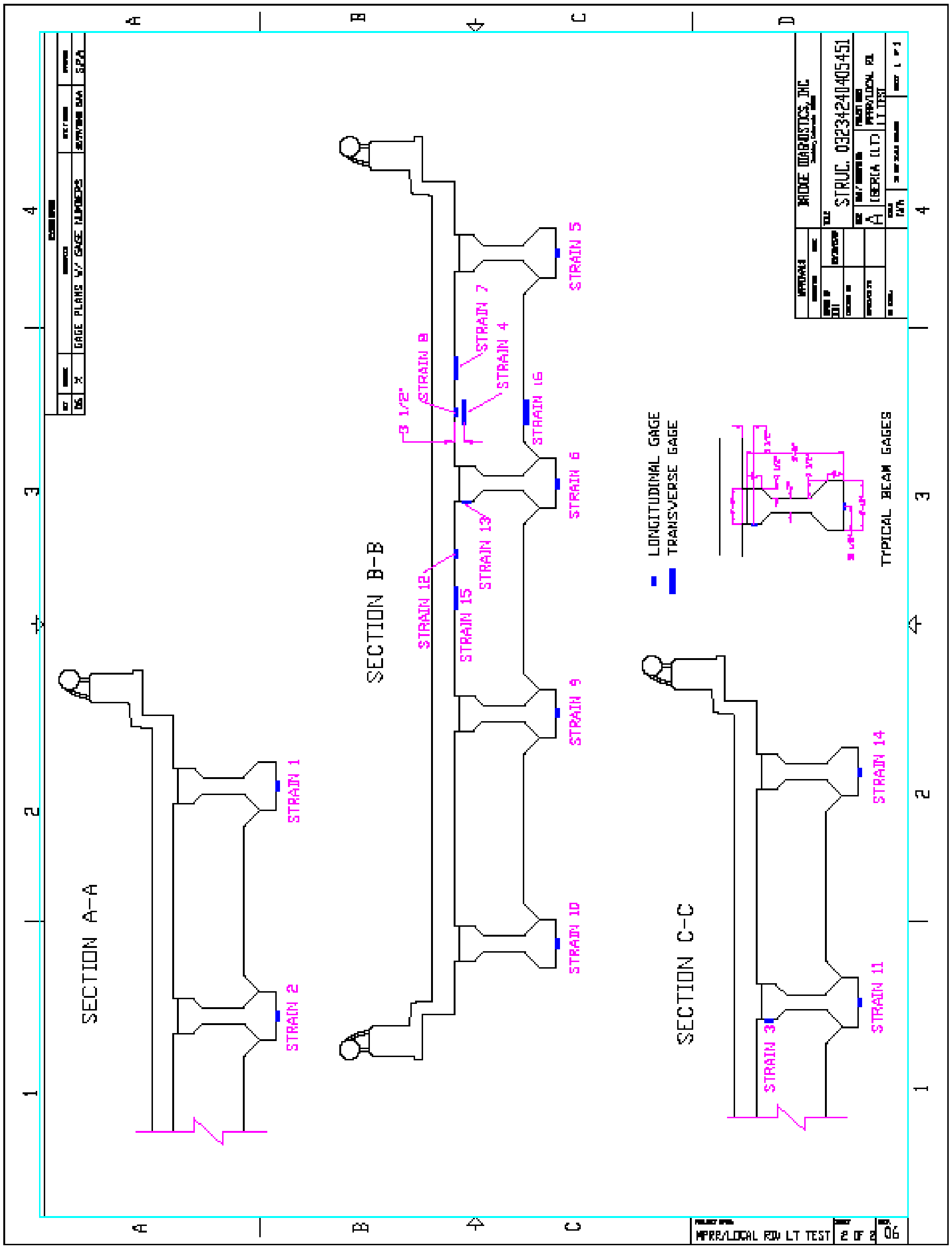


Instrumentation plans for monitoring system “Continued”



# APPENDIX F

**LIVE LOAD TEST**
for
Bridge Structure Number 03234240405451.



# APPENDIX G

## LIVE LOAD TESTING AND BRIDGE RATING

This appendix contains field work, live load testing, and bridge rating for Bridge Structure Number 03234240405451. (This appendix is the original appendix submitted by the company that ran the live load testing and conducted the bridge rating.)

## STRUCTURE DESCRIPTION

Structure 03234240405451 consists of 17 pre-stressed concrete spans and was built in 1966. According to the structure's Inventory and Appraisal sheet, this structure has not been retrofit or rebuilt since the initial construction. The available drawings are partially unreadable; therefore, some assumptions had to be made concerning beam properties and were confirmed with the Louisiana Transportation Research Center. Table 32 provides other information for Structure 03234240405451.

**Table 32**
**Description of structure**

| | |
|---|---|
| Structure Identification | Structure 03234240405451 |
| Location | I-90, North of New Iberia, LA |
| Structure Type | PS/C girder bridge |
| Number of Spans | 17 |
| Span Lengths | 60' c.c. of piers / 59'-7" c.c. of beam bearings |
| Skew | 0 (Perpendicular) |
| Structure/Roadway Widths | 32' / 28' |
| Beams | 4 – pre-stress AASHTO Type at 8'-8" on center |
| Deck | RC Deck 7.5" Possibly additional 2" of concrete overlay but none specified in plans. |
| Curbs and Parapets | Cast in place R/C Parapets on outside of exterior beams. |
| Visual condition | All superstructure elements appear to be in good condition with no visible shear or flexural cracks. |

## INSTRUMENTATION AND TESTING PROCEDURES

The primary objective of the instrumentation plan was to quantify the live load response behavior of the superstructure under normal service loads and the new heavy sugar cane trucks. The superstructure of the bridge was instrumented with 38 re-usable strain transducers as shown in Figure 13. Only one span was instrumented since all of the spans were the same length and in approximately the same condition. Selection of the span to instrument was based primarily on accessibility. As seen in the figure, Beams 1, 2, and 4 were instrumented with six gauges, two gages four feet from the bearings at each end and two at approximately midspan. Beam 3 had the same end gage set-up and the midspan was gauged with four gauges, two on the outside of the top flange and two on the bottom flange. A typical beam cross section can be seen in Figure 13.

Gages were also placed both longitudinally and transversely around Beam 3. The instrumentation mounted on the diaphragms was outfitted with transducer extensions to provide a total gage length of 12 in. These extensions are required for use on reinforced concrete structural elements to help account for possible cracking. Access to the superstructure was provided by the DOTD and consisted of a boom lift shown in Figure 14.

The load tests were performed by driving a 30-kip dump truck across the bridge at crawling speed along four different lateral paths as follows: passenger side wheels 2.5 ft. off the south curb, driver side wheel directly over Beam 3, center of truck directly down the center line of the bridge, and driver side wheel 2.5 ft. off north curb. The truck paths can be seen in Figure 13 and are referenced from the driver's side front wheel. Data was recorded continuously at 40Hz during each pass, and the truck position was monitored in order to record strain as a function of vehicle position. Typical vehicle speeds were approximately 3 to 5 mph to minimize dynamic responses and to facilitate monitoring of the vehicle position. Axle weights and spacing of the test truck are shown in Figure 15.

Tests were repeated for all four truck paths to ensure reproducibility in the procedures and in the structural response. All instrumentation was performed on Monday, December 12, and the testing procedures and instrumentation removal was completed the following morning. An outline of the test procedures is provided in Table 33. Also, please see Appendix A for further details on the basic field testing procedures.

**Table 33**
**Preliminary test procedures with dump truck**

| | |
|---|---|
| Date | December 13, 2005 |
| Structural Reference Point | X = 0, Y = 0 at the south-west corner, inside curb. |
| Test vehicle direction | South bound for all tests (Positive X direction). |
| Start of data recording | Data acquisition began with front axle at<br>X = -20.22' (-15' – ½ wheel rev.) |
| AutoClicker Position | Passenger side front wheel |
| Truck position | AutoClicker recorded truck position at each wheel revolution. Wheel rollout distance = 10.44' |
| Lateral truck path(s) | 4 truck paths were defined for the load test. The Y position refers to distance between driver's side front wheel and inside of east curb (Y= 0).<br>$Y_1$ = 9'-2", $Y_2$ = 9'-8", $Y_3$ = 17'-5", $Y_4$ = 25'-6" |
| Measurements | (38) removable strain transducers recorded at 40 Hz |
| Gage Placement | See fFigure 13 |
| Number of test cycles | Data was recorded while the test truck crossed the bridge at crawl speed (< 5 mph). Each truck path was run twice to check reproducibility. |

| Event | Data Files |
|---|---|
| Test truck traveling along Y1 | Iberia_1.dat |
| Test truck traveling along Y1 | Iberia_2.dat |
| Test truck traveling along Y2 | Iberia_3.dat |
| Test truck traveling along Y2 | Iberia_5.dat |
| Test truck traveling along Y3 | Iberia_6.dat |
| Test truck traveling along Y3 | Iberia_7.dat |
| Test truck traveling along Y4 | Iberia_8.dat |
| Test truck traveling along Y4 | Iberia_9.dat |

**Figure 12**
**Instrumentation near midspan**

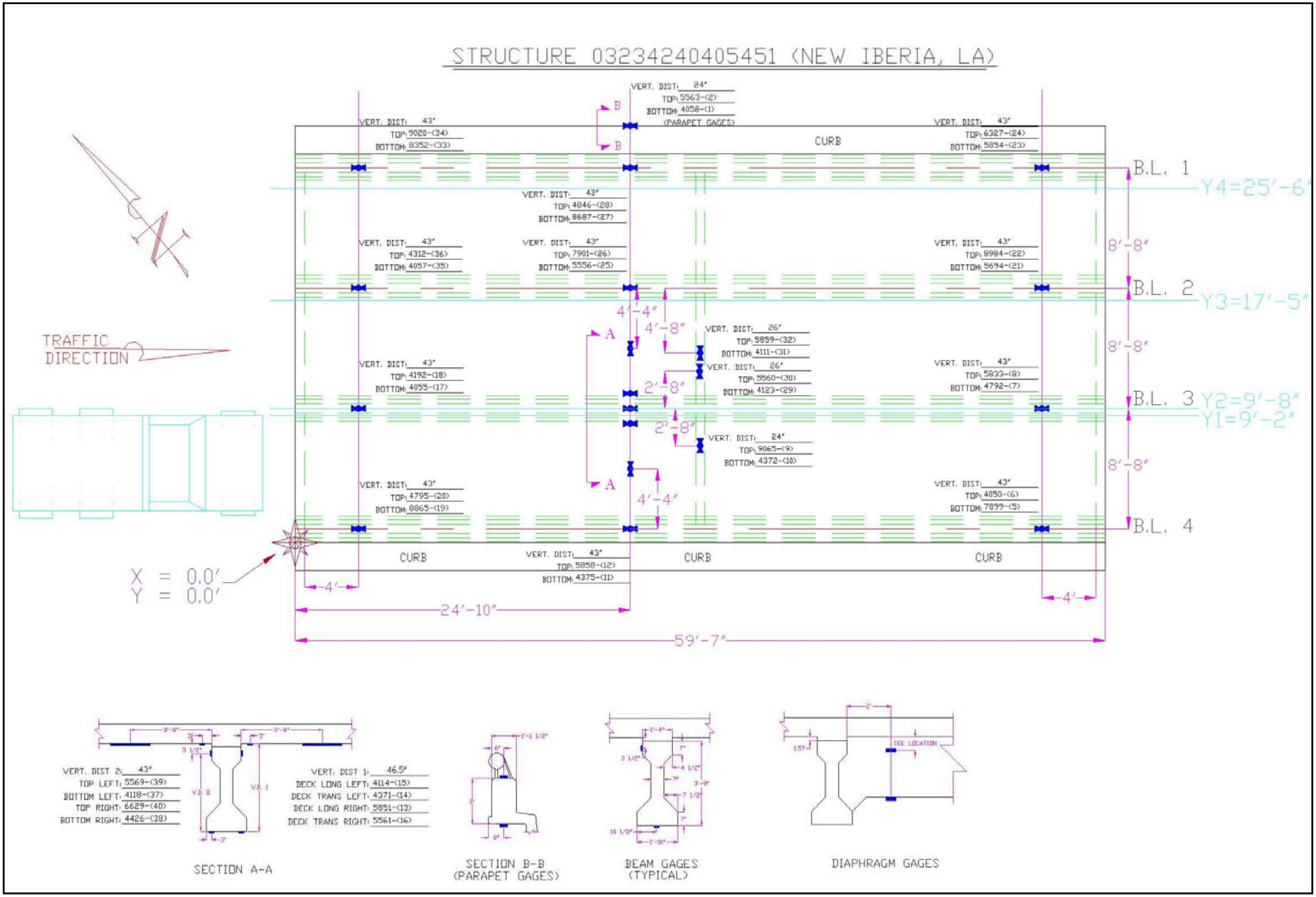



**Figure 13**
**Instrumentation plan**

**Figure 14**
**Instrumentation accessed by boom lift**

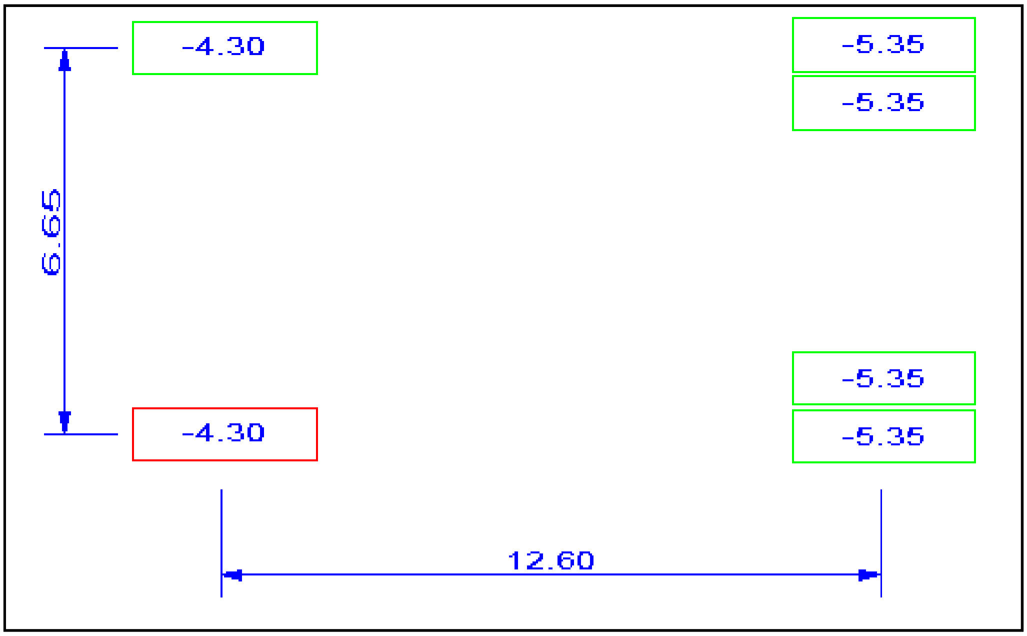



**Figure 15**
**Structure 03234240405451– load test vehicle configuration (ft., kips)**

# PRELIMINARY INVESTIGATION OF TEST RESULTS

The project personnel would like to thank the support crew and everyone else involved in this live load test.

## Live Load Test Data

Conclusions made directly from the field data:

Responses from identical truck paths were reproducible as shown in fFigure 16. However, it was found that gages on the diaphragms had a greater than normal variation because they were very sensitive to the lateral truck position. Note that due to the relatively light truck that was used for these tests, the strains magnitudes were small; therefore, the resolution of the strain measurement system (approximately ½ microstrain) can be seen in some of the strain histories.

Upper gages directly under the wheel lines all experienced tension spikes due to local deformation of the beam flange as the vertical wheel loads traveled across the gage location. This is a common phenomenon seen in most beam/slab type bridges and can be seen in fFigure 16.

From inspection of the upper and lower strain values, it was apparent that the upper gages experienced very little strain due to normal beam flexure. This was due to the combination of the loading vehicle being light and the top gage being relatively close to the neutral axis. This caused the resolution of the gages to affect the calculations for the neutral axes locations.

The average measured neutral axis location for the interior beams was 38 in. from the bottom of the beam, which corresponded closely to the theoretical neutral axes values. However, the measured neutral axis locations for the exterior beams were significantly higher than initially computed. Both exterior beam N.A. values were consistently found to be around 39 in., approximately 4 in. higher than expected. This confirmed that the parapets were adding a significant amount of stiffness to the exterior beams.

Although the top strains were relatively low in magnitude, it can be seen in Figure 17 that the structure was behaving compositely. The figure shows the responses from four gauges: two on the top flange of Beam 3 and two on the bottom of the deck directly adjacent to the top of the beam. The similar response histories show that the beams stayed composite with the deck during the entire loading cycle. Note, however, that the response recorded from gage 5851 (mounted on the bottom of the deck) shows that the strains corresponded well in the deck gages except when the rear axle was near the gage location. The cause of this difference was the width of dual real wheels. In this response history, the driver's side rear exterior tire was centered down Beam 3 while the interior tire was closer to the location of 5851. As a result, to understand what is happening globally, the visual analysis should be done while the rear axle of the loading vehicle is away from the location. The truck location also explains the higher wheel spike in 5851.

Based on the examination of mid-span strains across the bridge, there was good lateral load distribution (see Figure 18). This figure shows the peak mid-span strains recorded at the bottom of each beam during each of the four lateral truck locations. Note that the responses recorded when the truck was following Y1 was symmetric to the responses measured when the truck was following Y4. This type of symmetric response indicates not only excellent data quality, but that the structure is behaving in a linear fashion. If these responses were significantly different, it would indicate that the loads were not being distributed equally and possibly indicate damage or other unwanted behavior.

There was essentially no end restraint indicated by the response histories recorded near the supports.

Since no high speed tests were performed due to the extremely high volume of traffic, an experimental impact factor could not be calculated for this structure. Therefore, an AASHTO impact factor of 33 percent was used for load rating this structure.

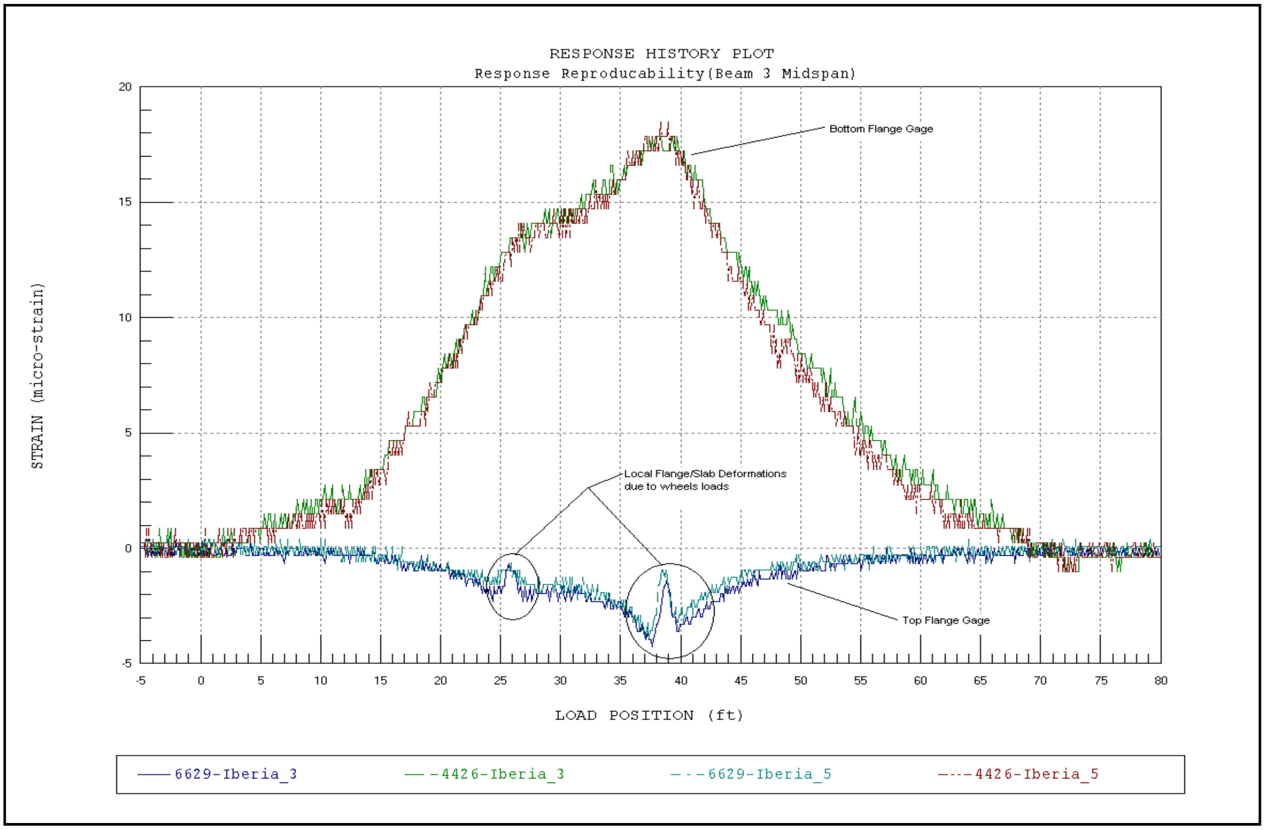



**Figure 16**
**Reproducibility of test results – gages directly below wheel line – Path 2**

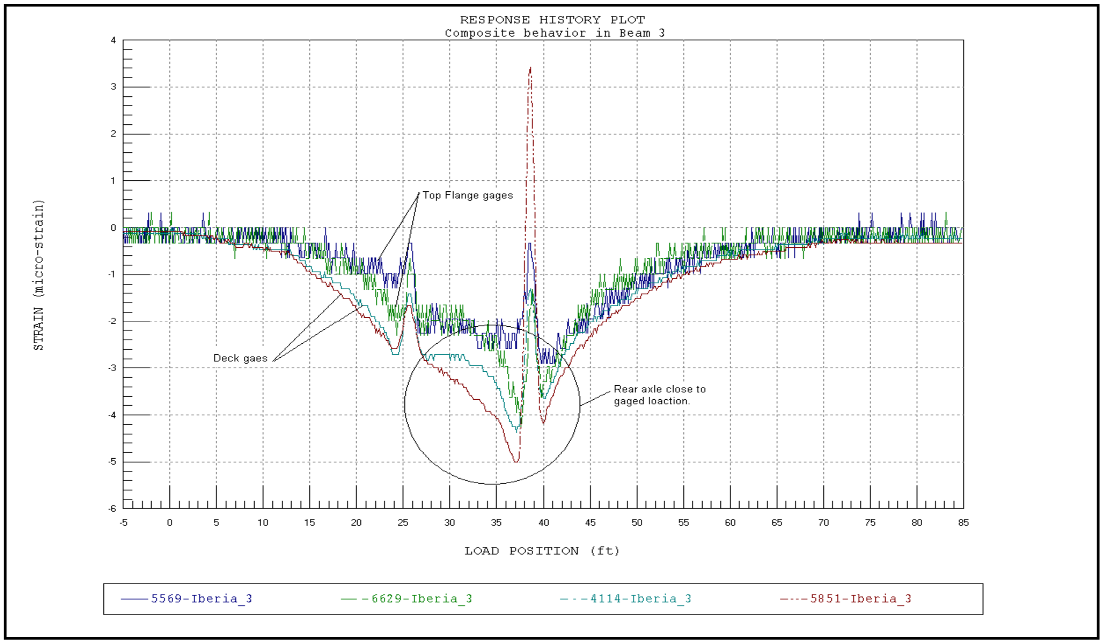



**Figure 17**
**Composite behavior between Beam 3 and deck**

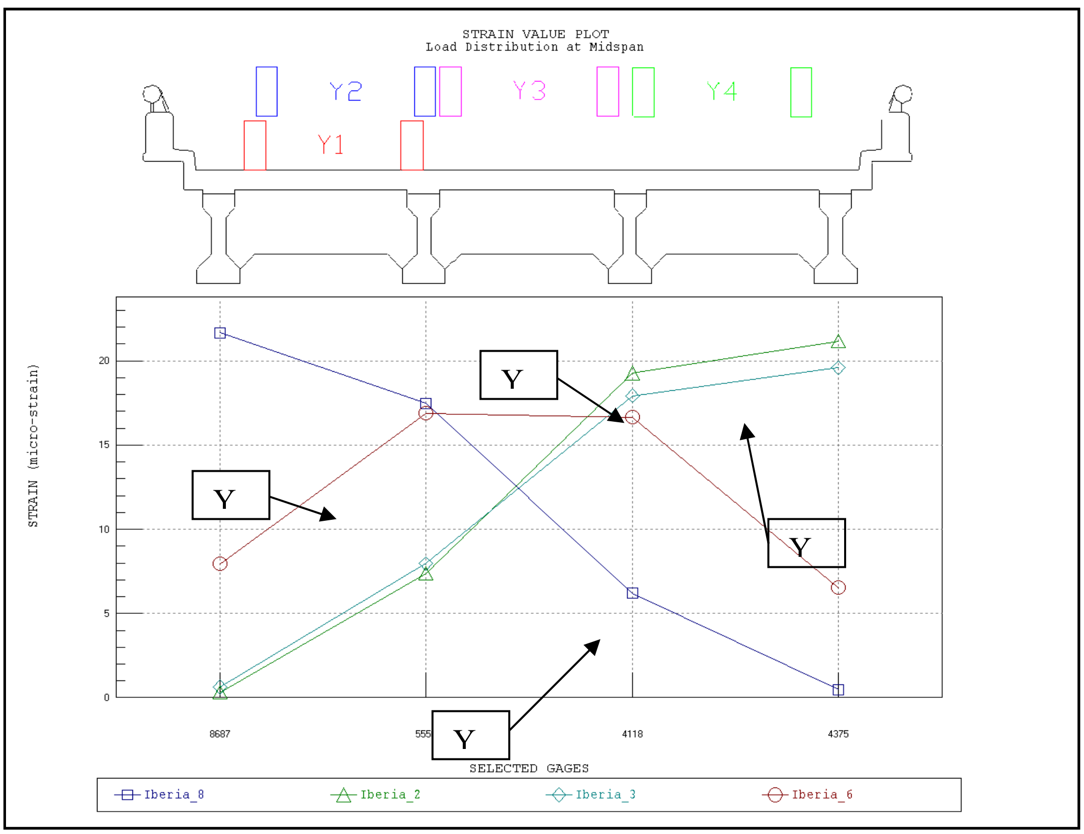



**Figure 18**
**Mid-span strains across the bridge at maximum moment – Path Y2**

## Modeling, Analysis, and Data Correlation

Up to this point, all discussion of the bridge behavior has been limited to examination of the load test data, illustrating that a qualitative understanding of the structure's response can be as important as the quantitative results. The next phase of the investigation was to verify the measured responses using structural analysis techniques. This was done by developing a two-dimensional model of the structure and making direct comparisons between the analytical results and the measured responses. The differences between the measured and computed data were then used as a means for model modification and improvement until a satisfactory correlation was made. The model calibration process was performed based on load test data with the legal load dump truck. This process was also used to verify linear behavior of the structure and verify that the model could be used to predict the structure's response to other load configurations.

The finite element model was initially developed as a two-dimensional grid consisting of beam lines at each I-beam and the centerline of the parapet. Shell elements were used to represent the cast-in-place deck that distributed the load to the beams. Linear springs eccentrically placed from the model plane were used as the supports to simulate any possible end-restraints and continuity of movement over the piers. It was also initially assumed that there was no continuity of movement over piers. Figure 19 shows the computer-generated display of the grid model. In order to make this 2-D model more representative of the structure's actual behavior, the parapet beams were modeled as trapezoidal cross-sections and given an eccentricity value of 36.85 in. from the deck. The beams were modeled as composite sections with a haunch of 1.6 in., and the deck thickness of the composite section was manipulated to make the model's neutral axis match the field data neutral axis. This was done for both the interior and exterior beams. Figure 20 shows the cross-section properties of the interior beams. Table 34 is a table of assumptions that were provided by Louisiana Tech University and were used for both modeling and rating of this structure. These assumptions had to be made due to the age of the plans making them virtually impossible to read much of the vital information.

Once the model was developed, the field load testing procedures could be reproduced on a PC with the BDI WinGen software. This process included placing gage locations on the model (using the same transducer IDs), generating a 2-D "footprint" of the test truck, and defining truck paths that were identical to those in the field. The analysis was run and strains were computed at each gage location for each load case consisting of the truck moving at 3-ft. intervals the length of the bridge.

After the first analysis run, the data was compared visually and various statistical measures of accuracy were computed. The primary differences between the measured and the initial calculated results indicated that the model did not have the proper load distribution and the beams were too flexible. As a result, the stiffness of the beams, end-restraints, and the deck were adjusted to improve the correlation. The stiffness variables were modified via the automated optimization process built into the WinSAC program. Results from the overall calibration process are shown in Table 35 where the initial and final variables and error values are listed. Additional discussion of the calibration process and definition of the error terms are provided in Appendix B.

**Table 34**
**Modeling assumptions**

| Assumed Properties and Dimensions | Assumed Value |
|---|---|
| Beam concrete strength (ksi) | 6 |
| Cast-in-place concrete strength (ksi) | 4.2 |
| Haunch (in.) | 1.6 |

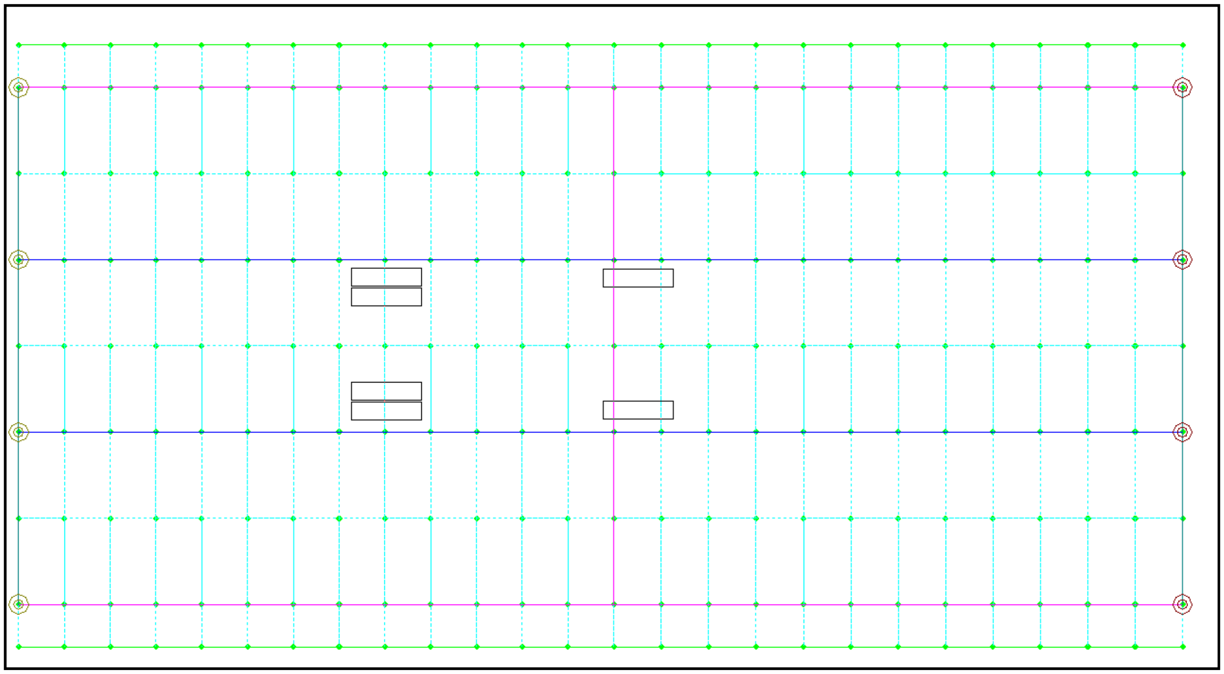


**Figure 19**
**Finite element model of tested span with computer generated tuck path Y3**

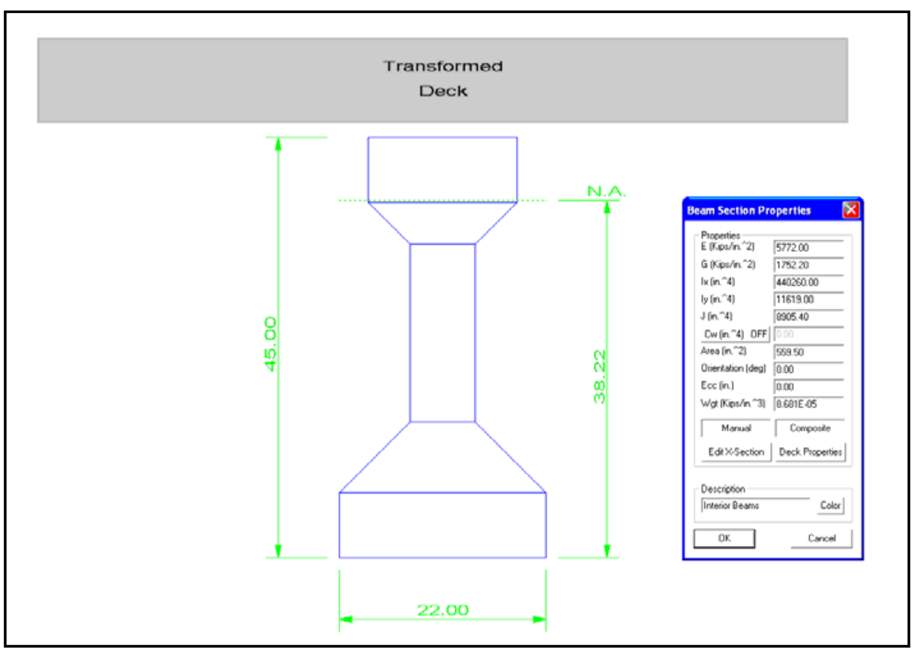



**Figure 20**
**Beam interior beam cross-section properties**

**Table 35**
**Model calibration and accuracy results**

| Stiffness Parameter | Original Value | Final Value |
|---|---|---|
| Beam Concrete Modulus ($E_c$-ksi) | 6000 | 5708 |
| Deck Concrete Modulus ($E_c$-ksi) | 4200 | 5600 |
| Mid-Span Diaphragm ($I_x$ - in$^4$) | 75441 | 90000 |
| End-restraint via axial springs-North ($F_x$ – kips-in) | 200 | 370.7 |
| End-restraint via axial springs-South ($F_x$ – kips-in) | 200 | 444.3 |
| | | |
| Error / Accuracy Term | Initial Model | Final Model |
| Absolute Error | 1864.1$\mu\varepsilon$ | 784.6$\mu\varepsilon$ |
| Percent Error | 7.5% | 1.7% |
| Scale Error | 7.5% | 3.7% |
| Correlation Coefficient | 0.9724 | 0.991 |

General conclusions that were made during the calibration process included:

- The "effective" concrete modulus was relatively high compared to the concrete modulus computed from the design concrete strength. It is important to note that this effective modulus value was not assumed to be a true concrete modulus because it included the effect of the pre-stressed and conventional reinforcement, and it compensated for variations between the design and actual member dimensions (i.e., additional fillets, contribution of additional wearing surface, etc.). Note that this same theory applies to the deck modulus as well. The "effective" modulus for the beams was sufficiently high to indicate a high-strength concrete with an estimated value for f'c being a minimum of 6 ksi and possibly as high as 8 ksi.
- The deck thickness was higher than expected by about 15 percent (9 in. compared to 7.5 in. indicated in the plans). The likely explanation for this is that an additional wearing surface was added to this structure that does not appear on the plans.
- The model calibration process confirmed that there was very little end-restraint and is shown in Figure 22.
- The parapet beams added a significant amount of stiffness to the exterior beams. This was confirmed in the initial data analysis through both the load distribution and the neutral axis locations of the exterior beams. The analysis confirmed this prediction when it matched the load distribution measured with the gages (see Figure 23).
- The load distribution was better than predicted by the initial model. As a result, the mid-span diaphragm was modeled as a composite section with the deck as its top flange. The diaphragm gages were used to predict the initial effective flange width by correlating the model neutral axis with the field data. The effective flange width was 30 in.
- The resulting final model based on the dump truck loading data was very accurate, indicating that the structure was behaving linearly elastic.

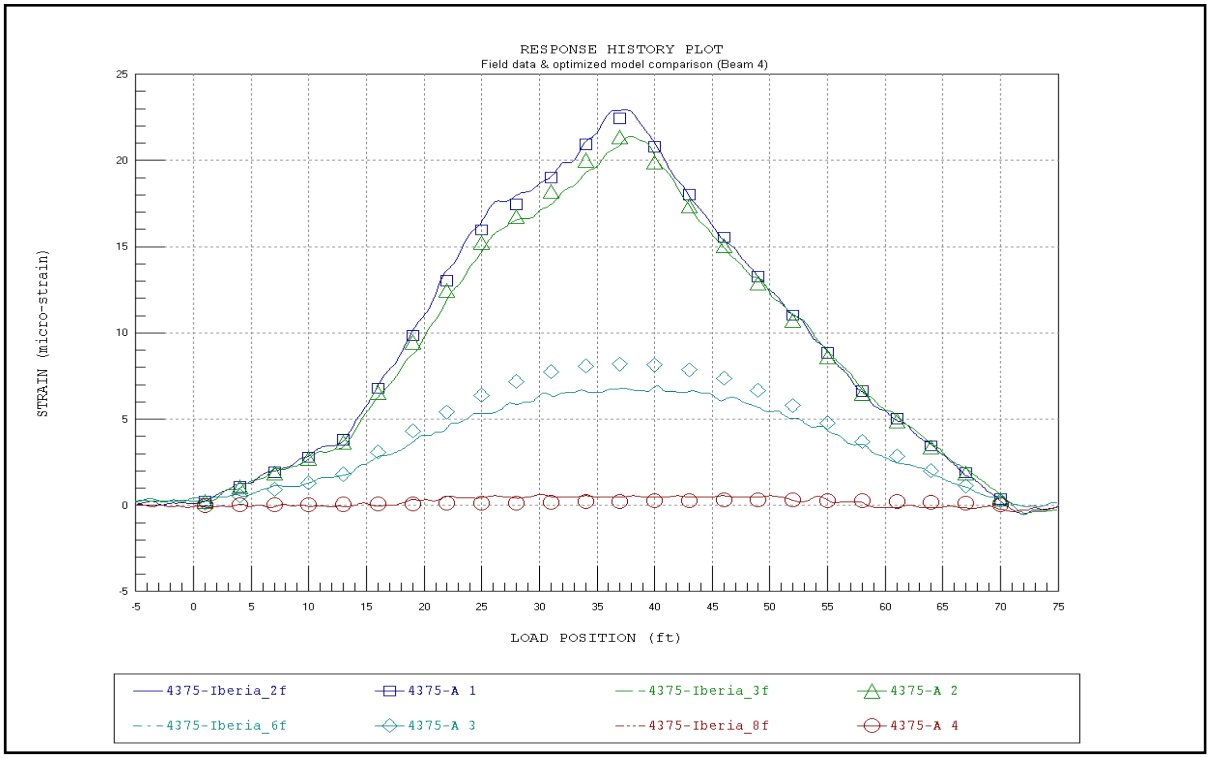



**Figure 21**
**Data comparison for Beam 4 (midspan response)**

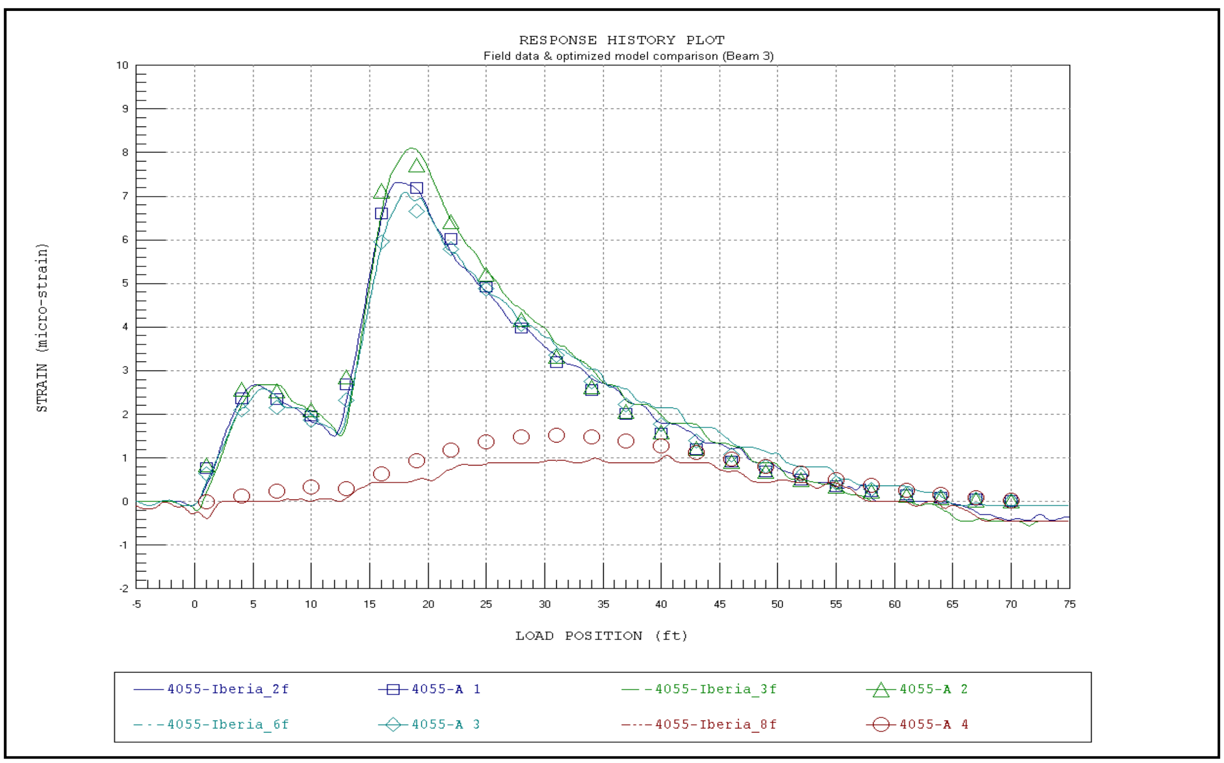



**Figure 22**
**Data comparison for Beam 3 (north abutment response)**

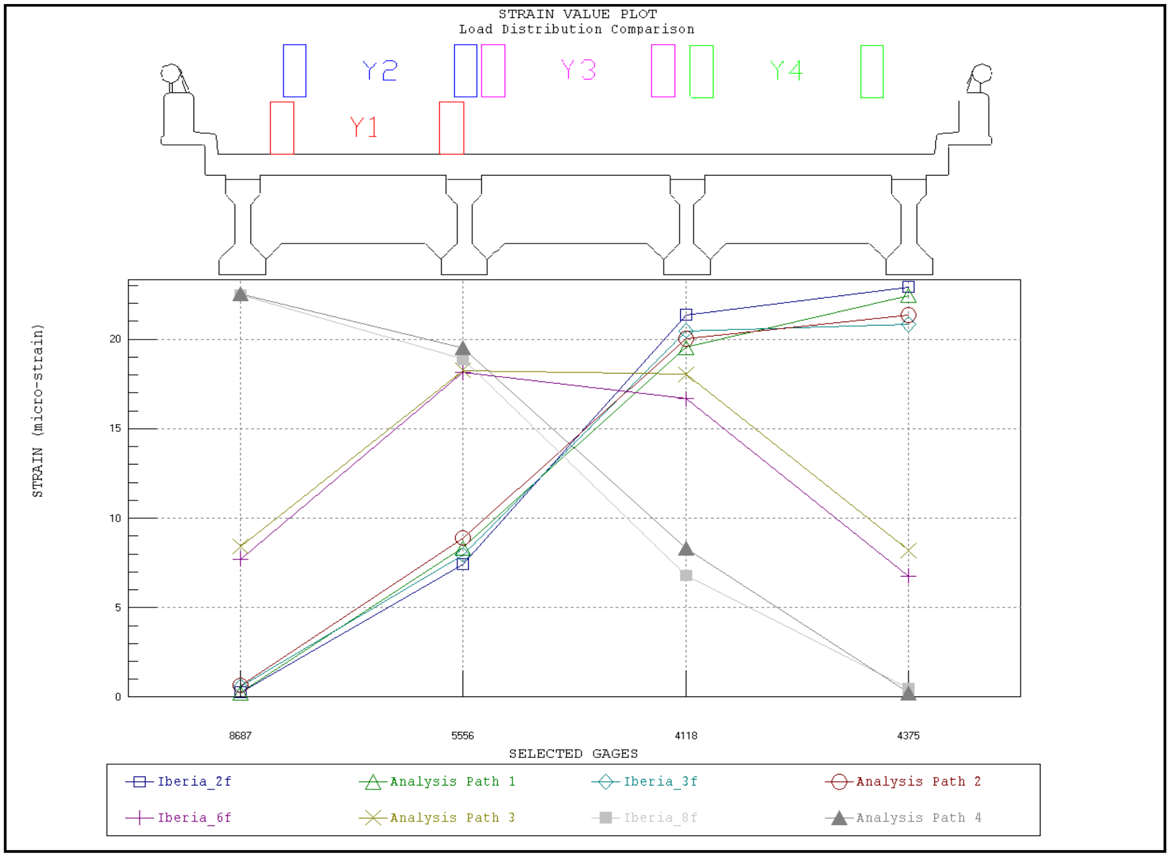



**Figure 23**
**Load distribution comparison at mid-span**

# LOAD RATING PROCEDURES AND RESULTS

The goal of producing an accurate model was to predict the structure's actual live load behavior when subjected to design and rating loads. The primary benefit of a "calibrated" model is that responses from the entire superstructure can be investigated rather than just the instrumented locations. This is important because in most cases, the instrumentation is not located at the critical location on the bridge. Since the load rating is based on an analysis, the approach is essentially identical to standard load rating procedures except that a "field verified" model is used instead of a typical beam analysis combined with load distribution factors.

Once the finite element model has been calibrated to field conditions, engineering judgment must be used to address any optimized parameter that may change over time or that may be unreliable with heavy loads. In this case, three optimized parameters that were not used for rating purposes were the end restraints at both bearings and the increased deck thickness. Due to bearing conditions changing over time, temperature, etc., a worst case scenario was taken for rating purposes and the end restraint was set to zero (i.e., simply supported span). As for the thicker deck, it was assumed that the extra concrete was some sort of wearing surface and may be removed at some point. As a result, the capacities have been calculated with a deck thickness of 7.5 in. as stated in the provided plans.

The load configurations used for developing the rating factors are provided in figures 24 and 25. Load rating factors were computed using the Load and Resistance Factor Rating (LRFR) methods specified in the *2003 AASHTO Condition Evaluation of Highway Bridges Manual.* Rating values were obtained by applying the dead load and the various live loads to the model and comparing the responses to the available capacity. Shear and moment capacities were computed using current AASHTO LRFD and 17$^{th}$ Edition-2002 LFD specifications. Important variables and computed values for LRFD and LFD capacity calculations can be seen below in Tables 36, 37, and 38. Much of the capacity information was unclear on the bridge drawings; therefore, this information was interpreted and confirmed by LTRC. The interpreted items have been denoted below in the capacity tables with a superscript "+."

Load rating factors were obtained by running each of the load configurations across the model. Standard width trucks were rated assuming two-lane loading. Live-load envelopes were generated for each member and compared with their respective live-load capacities. As per the AASHTO LRFD and LFD specifications, a dynamic allowance factor (impact factor) of 33 and 30 percent was used for all cases, respectively. Table 39 contains the maximum moment and rating factors for the critical member, and Table 40 contains the same information for shear.

As defined by the *AASHTO Manual for Condition Evaluation of Bridges*, Inventory Rating Level corresponds to the design level of stresses and reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the Inventory Level allow for a determination of a live load that can safely utilize an existing structure for an indefinite period of time. Loadings based on the Operating Rating Level describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at Operating Level may shorten the life of the bridge. However, infrequent intervals at the Operating limit would not have adverse effects on a structure's life span.

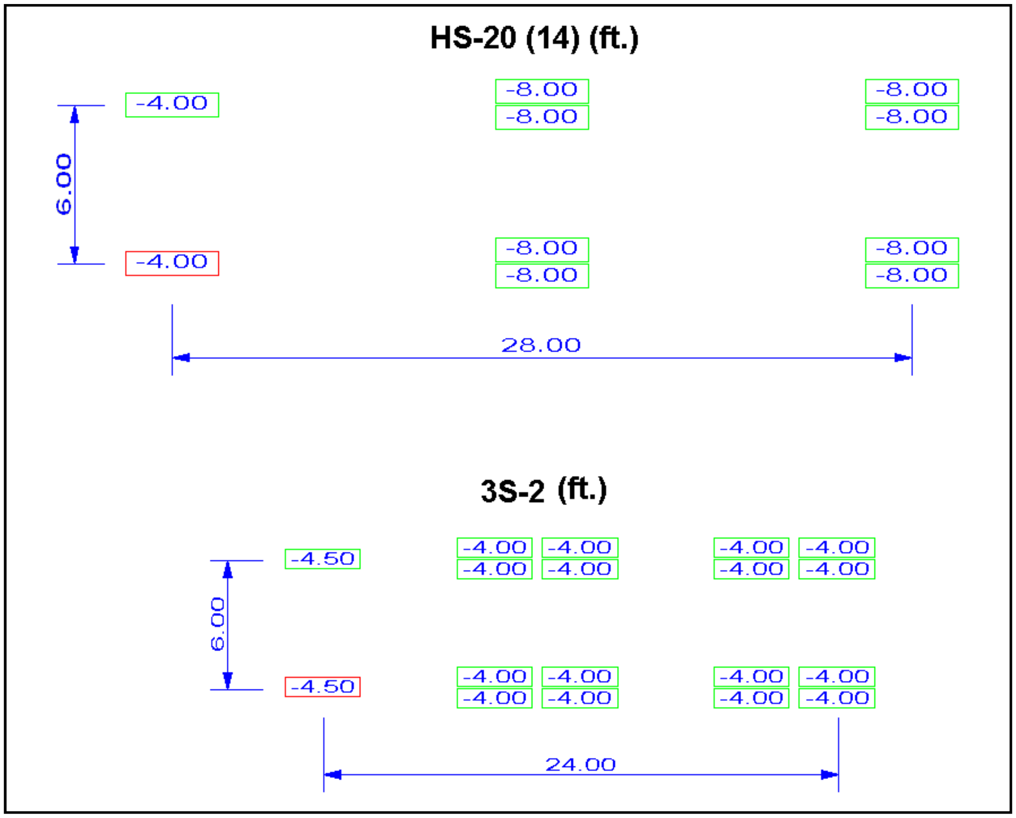



**Figure 24**
**HS-20 and 3S-2 load loading vehicles**

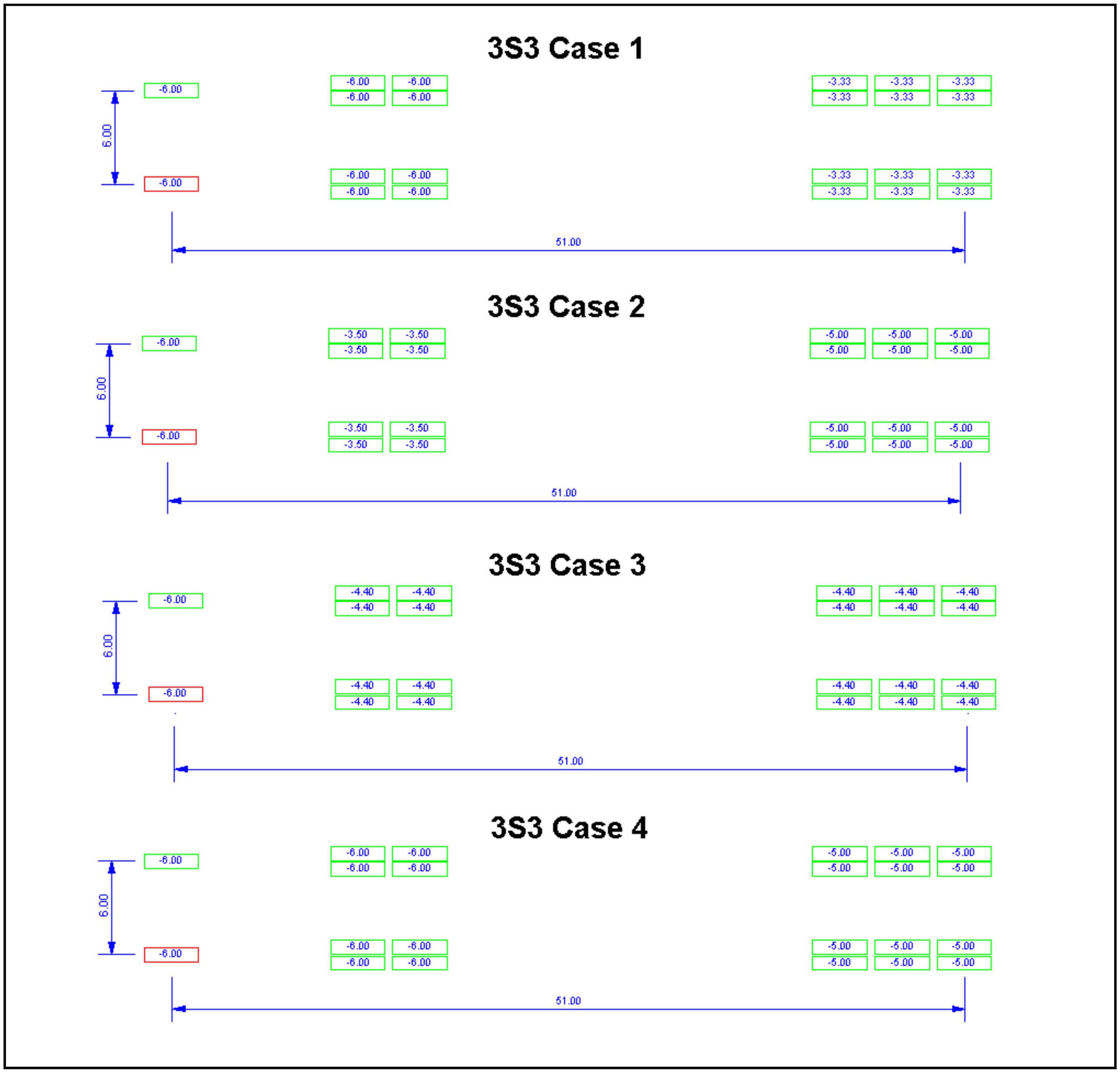



**Figure 25**
**3S3 Sugar cane loading vehicles**

**Table 36**
**LRFD live-load moment and shear capacity for exterior beams**

| Moment Capacity Calculations (Mid-span) | Variable | Value | Units |
|---|---|---|---|
| Concrete Strength (Beam/ Deck)+ | F'c | 6 / 4 | ksi |
| Width Concrete Flange | $b_{eff}$ | 71.5 | in |
| Pre-stressed Steel+ | $F_{pu}$ | 250 | ksi |
| Area Pre-stressed Steel/ Size | $A_{ps}$ / Dia | 3.67 / 0.4375 | $in^2$/in |
| Beam Depth | D | 45 | in |
| Haunch+ | — | 1.6 | in |
| Deck Thickness | $t_{deck}$ | 7.5 | in |
| Type of Pre-stress Steel+ | k | 0.38 | — |
| Dist. From bottom of beam to centriod of P.S. steel | — | 5.44 | in |
| Dist. btwn centroid of P.S. and extreme compression fiber | $d_p$ | 48.66 | in |
| Compression block | c | 4.29 | in |
| Theoretical compression block | a | 3.65 | in |
| Prestress stress at ultimate moment | $f_{ps}$ | 241.6 | ksi |
| Nominal Moment | $M_n$ | 41533 | kip-in |
| | | | |
| Reduction Factor | Φ | 1.0 | — |
| **Reduced Moment** | **$\Phi M_n$** | **41533** | **kip-in** |

| Shear Capacity Calculations | Variable | Value | Units |
|---|---|---|---|
| Concrete strength (beam)+ | F'c | 6 | ksi |
| Web thickness | t | 7 | in |
| Rebar strength (ASTM A305)+ | $F_y$ | 40 | ksi |
| Rebar size/ area (Abutment & Elsewhere)* | -/A | 5 / 0.306 & 4 / 0.196 | # / $in^2$ |
| Shear constant | β | 5.138/ 5.304/ 5.304/ 4.961 | — |
| Shear angle constant | θ | 19.91/ 19.5/ 19.55/ 20.08 | deg |
| Shear Length | $d_v$ | 46.8 | in |
| Stirrup spacing*+ | s | 4 / 12.5 / 15 / 22 | in |
| Pre-stressing force+ | | 18.9 | kips |
| Concrete shear capacity* | $V_c$ | N-A/ 135/ 135/ 126 | kips |
| Steel shear capacity* | $V_s$ | N-A / 166 / 137 / 91 | kips |
| Pre-stress steel shear capacity* | $V_p$ | 11 / 11 / 11 / 0 | kips |
| Nominal shear capacity* | $V_n$ | 500/ 311/ 284/ 217 | kips |
| | | | |
| Reduction factor | Φ | 0.9 | — |
| **Reduced Shear*** | **$\Phi V_n$** | **453/ 280 / 255 / 195** | **kip** |

*calculations were done for each section where the shear steel changed (Abutment/ Intermediate/Intermediate/Midspan).
+ Information interpreted and confirmed by LTRC.

**Table 37**
**LRFD live-load moment and shear capacity for interior beams**

| Moment Capacity Calculations (Mid-span) | Variable | Value | Units |
|---|---|---|---|
| Concrete Strength (Beam/ Deck)+ | F'c | 6 / 4 | ksi |
| Width Concrete Flange | $b_{eff}$ | 104 | in |
| Pre-stressed Steel+ | $F_{pu}$ | 250 | ksi |
| Area Pre-stressed Steel/ Size | $A_{ps}$ / Dia | 3.67 / 0.4375 | $in^2$/in |
| Beam Depth | D | 45 | in |
| Haunch+ | — | 1.6 | in |
| Deck Thickness | $t_{deck}$ | 7.5 | in |
| Type of Pre-stress Steel+ | k | 0.38 | — |
| Dist. From bottom of beam to centriod of P.S. steel | — | 5.44 | in |
| Dist. btwn centroid of P.S. and extreme compression fiber | $d_p$ | 48.66 | in |
| Compression block | c | 2.98 | in |
| Theoretical compression block | a | 2.54 | in |
| | $f_{ps}$ | 244.2 | ksi |
| Nominal Moment | $M_n$ | 42492 | kip-in |
| | | | |
| Reduction Factor | Φ | 1.0 | — |
| **Reduced Moment** | **$\Phi M_n$** | **42492** | **kip-in** |

| Shear Capacity Calculations | Variable | Value | Units |
|---|---|---|---|
| Concrete strength (beam)+ | F'c | 6 | ksi |
| Web thickness | t | 7 | in |
| Rebar strength (ASTM A305)+ | $F_y$ | 40 | ksi |
| Rebar size/ area (Abutment & Elsewhere)* | -/A | 5 / 0.306 & 4 / 0.196 | # / $in^2$ |
| Shear constant* | β | 4.727/ 4.665/ 4.831/ 4.202 | — |
| Shear angle constant* | θ | 20.42/ 20.48/ 20.28/ 20.91 | deg |
| Shear Length | $d_v$ | 47.4 | in |
| Stirrup spacing*+ | s | 4 / 12.5 / 15 / 22 | in |
| Pre-stressing force+ | | 18.9 | kips |
| Concrete shear capacity* | $V_c$ | N-A/ 120/ 124/ 113 | kips |
| Steel shear capacity* | $V_s$ | N-A/ 159/ 134/ 88 | kips |
| Pre-stress steel shear capacity* | $V_p$ | 11 / 11 / 11 / 0 | kips |
| Nominal shear capacity* | $V_n$ | 509/ 290/ 269/ 201 | kips |
| | | | |
| Reduction factor | Φ | 0.9 | — |
| **Reduced Shear*** | **$\Phi V_n$** | **458/ 261/ 242/ 181** | **kip** |

*calculations were done for each section where the shear steel changed (Abutment/ Intermediate/ Intermediate/ Midspan).
+ Information interpreted and confirmed by LTRC.

**Table 38**
**LFD live-load moment and shear capacity for all beams**

| Moment Capacity Calculations (Mid-Span) | Variable | Value | Units |
|---|---|---|---|
| Concrete Strength (Beam/ Deck)+ | F'c | 4-Jun | ksi |
| Width Concrete Flange | $b_{eff}$ | 104 | in |
| Pre-stressed Steel+ | $F_{pu}$ | 250 | ksi |
| Area Pre-stressed Steel/ Size | A* / Dia | 3.67 / 0.4375 | $in^2$/in |
| Beam Depth | D | 45 | in |
| Haunch+ | — | 1.6 | in |
| Deck Thickness | $t_{deck}$ | 7.5 | in |
| Type of Pre-stress Steel+ | k | 0.38 | — |
| Dist. btwn centroid of P.S. and extreme compression fiber | $d_p$ | 48.66 | in |
| Theoretical compression block | a | 2.54 | in |
| Avg. stress in pre-stressing steel at ult. Load | $f^*_{su}$ | 244.2 | ksi |
| Nominal Moment | $M_n$ | 42530 | kip-in |
| | | | |
| Reduction Factor | Φ | 1 | - |
| **Reduced Moment** | **$\Phi M_n$** | **42530** | **kip-in** |

| Shear Capacity Calculations | Variable | Value | Units |
|---|---|---|---|
| Concrete strength (beam)+ | F'c | 6 | ksi |
| Web thickness | t | 7 | in |
| Rebar strength (ASTM A305)+ | $F_y$ | 40 | ksi |
| Rebar size/ area (Abutment & Elsewhere)* | -/A | 5 / 0.306 & 4 / 0.196 | # / $in^2$ |
| Shear Length | d | 48.66 | in |
| Stirrup spacing*+ | s | 4 / 12.5 / 15 / 22 | in |
| Pre-stressing force+ | | 18.9 | kips |
| Concrete shear capacity | $V_c$ | 181 | kips |
| Steel shear capacity* | $V_s$ | 191/ 61/ 51/ 35 | kips |
| Pre-stress steel shear capacity* | $V_p$ | 11 / 11 / 11 / 0 | kips |
| Nominal shear capacity* | $V_n$ | 371/ 242/ 231/ 216 | kips |
| | | | |
| Reduction factor | Φ | 0.9 | — |
| **Reduced Shear*** | **$\Phi V_n$** | **334/ 218/ 208/ 194** | **kip** |

*calculations were done for each section where the shear steel changed (Abutment/ Intermediate/ Intermediate/ Midspan).

+ Information interpreted and confirmed by LTRC.

**Table 39**
**New Iberia load rating results (moment)**

| Truck | Live-load Moment (k-in) | Inventory Rating Factor LRFD / LFD | Operating Rating Factor LRFD / LFD |
|---|---|---|---|
| HS-20 (3 axle 72 kip) | 4763 | 3.1 / 2.48 | 4.02 / 4.13 |
| 3S2 (5 axle 73 kip) | 5018 | 2.94 / 2.35 | 3.81 / 3.92 |
| 3S3 Case 1 (5 axle 100kip) | 4902 | 3.03 / 2.42 | 3.92 / 4.04 |
| 3S3 Case 2 (5 axle 100kip) | 5283 | 3.06 / 2.26 | 3.67 / 3.78 |
| 3S3 Case 3 (5 axle 100kip) | 4735 | 3.16 / 2.53 | 4.09 / 4.22 |
| **3S3 Case 4 (5 axle 120kip)** | **5446** | **2.74 / 2.20** | **3.56 / 3.67** |

**Table 40**
**New Iberia load rating results (shear)**

| Truck | Live-load Shear (kips) | Inventory Rating Factor LRFD / LFD | Operating Rating Factor LRFD / LFD |
|---|---|---|---|
| HS-20 (3 axle 72 kip) | 34.3 | 2.52 / 1.67 | 3.26 / 2.78 |
| 3S2 (5 axle 73 kip) | 38.3 | 2.43 / 1.55 | 3.15 / 2.59 |
| 3S3 Case 1 (5 axle 100kip) | 34.5 | 2.50 / 1.66 | 3.24 / 2.77 |
| 3S3 Case 2 (5 axle 100kip) | 41.7 | 2.23 / 1.43 | 2.9 / 2.39 |
| 3S3 Case 3 (5 axle 100kip) | 38.2 | 2.44 / 1.56 | 3.16 / 2.61 |
| **3S3 Case 4 (5 axle 120 kip)** | **44** | **2.10 / 1.34** | **2.72 / 2.24** |

As seen in the above tables, the controlling load case was the 3S3 (Case 4 Sugar Cane Vehicle with 120 kip GVW) for both moment and shear ratings. The structure was shear critical with the critical section being the first change in shear stirrup spacing (s = 15 in.). Overall, the structure rates well for both moment and shear even with the increased vehicle weight of the sugar cane haulers.

It can be seen in Table 39 and Table 40 that the LFD ratings are considerably lower than the LRFD ratings. The primary differences between the two methods are the load factors applied to the live load responses. At the inventory level, the LFD live-load load factor of 2.17 is 24 percent greater than the LRFD load factor of 1.75. The operating limit load factors for both rating methods are relatively close at 1.3 for LFD and 1.35 for LRFD. The impact factor or dynamic allowance is also greater for the LRFD method (33 percent versus 27 percent). Another significant difference between the two rating methods is in the shear capacity calculations. Whereas the computation of moment capacities are nearly identical, the shear capacity calculation is in the LRFD method is based on a “strut and tie” approach and results in significantly different shear capacities than the LFD method.

# SUMMARY OF RESULTS

Conclusions made directly from the load test data were qualitative in nature and indicated that the structure responded in a linear-elastic fashion, and no damage was apparent. Measured neutral axis values were very close to the theoretical values, indicating the assumed beam section properties were valid. However, it was apparent that the 2 in. of concrete overlay were influencing the beam stiffness.

Through the use of a field calibrated finite element model, Structure 03234240405451 was analyzed and load rated for loading vehicles HS-20, 3S2, and 3S3 (sugarcane loading cases 1 through 4). The structure has adequate strength to resist both bending and shear forces for all six loading vehicles and is shear critical with a controlling load rating of 2.10 and 1.34 for inventory and 2.72 and 2.24 for operating for LRFD and LRF, respectively (see Table 39 and Table 40 for all other rating factors). The worst case loading vehicle was the 3S3 Case 4 and the critical shear location was at the first change in rebar spacing and size (2-No.4 at 1 ft.-3 in.). All of the components of the shear calculations were included in this rating, including the contribution from the draped pre-stressing steel, $V_p$. Loading vehicle 3S3 Case 4 also controls the moment rating as well (Inventory RF: 2.74 / 2.20 Operating RF: 3.56 / 3.67). Note that all of the rating factors are acceptable for all 17 spans as long as the construction and the structural condition of each span is the same.

Due to the new, higher load limit for the sugar cane trucks, a long-term monitoring system was also installed on this structure. This will be used to monitor deterioration of the structure over the system's scheduled life. Due to the rating being higher than expected, it is likely that the structure will perform well with the heavier trucks. However if a significant amount of change is seen over a period of time, it is advisable to retest this structure and allow engineers to do further evaluations.

# MODELING AND ANALYSIS: THE INTEGRATED APPROACH

## Introduction

The ultimate goal of the integrated approach is to obtain realistic rating values for highway bridges in a cost effective manner. This is accomplished by measuring the response behavior of the bridge due to a known load and determining the structural parameters that produce the measured responses. With the availability of field measurements, many structural parameters in the analytical model can be evaluated that are otherwise conservatively estimated or ignored entirely. Items that can be quantified through this procedure include the effects of structural geometry, effective beam stiffness, realistic support conditions, effects of parapets and other non-structural components, lateral load transfer capabilities of the deck and transverse members, and the effects of damage or deterioration. Often, bridges are rated poorly because of inaccurate representations of the structural geometry or because the material and/or cross-sectional properties of main structural elements are not well defined. A realistic rating can be obtained, however, when all of the relevant structural parameters are defined and implemented in the analysis process.

One of the most important phases of this approach is a qualitative evaluation of the raw field data. Much is learned during this step to aid in the rapid development of a representative model.

## Initial Data Evaluation

The first step in structural evaluation consists of a visual inspection of the data in the form of graphic response histories. Graphic software was developed to display the raw strain data in various forms. Strain histories can be viewed in terms of time or truck position. Since strain transducers are typically placed in pairs, neutral axis measurements, curvature responses, and strain averages can also be viewed. Linearity between the responses and load magnitude can be observed by the continuity in the strain histories. Consistency in the neutral axis measurements from beam to beam and as a function of load position provides great insight into the nature of the bridge condition. The direction and relative magnitudes of flexural responses along a beam line are useful in determining if end restraints play a significant role in the response behavior. In general, the initial data inspection provides the engineer with information concerning modeling requirements and can help locate damaged areas.

Having strain measurements at two depths on each beam cross-section, flexural curvature and the location of the neutral axis can be computed directly from the field data. Figure 26 illustrates how curvature and neutral axis values are computed from the strain measurements.

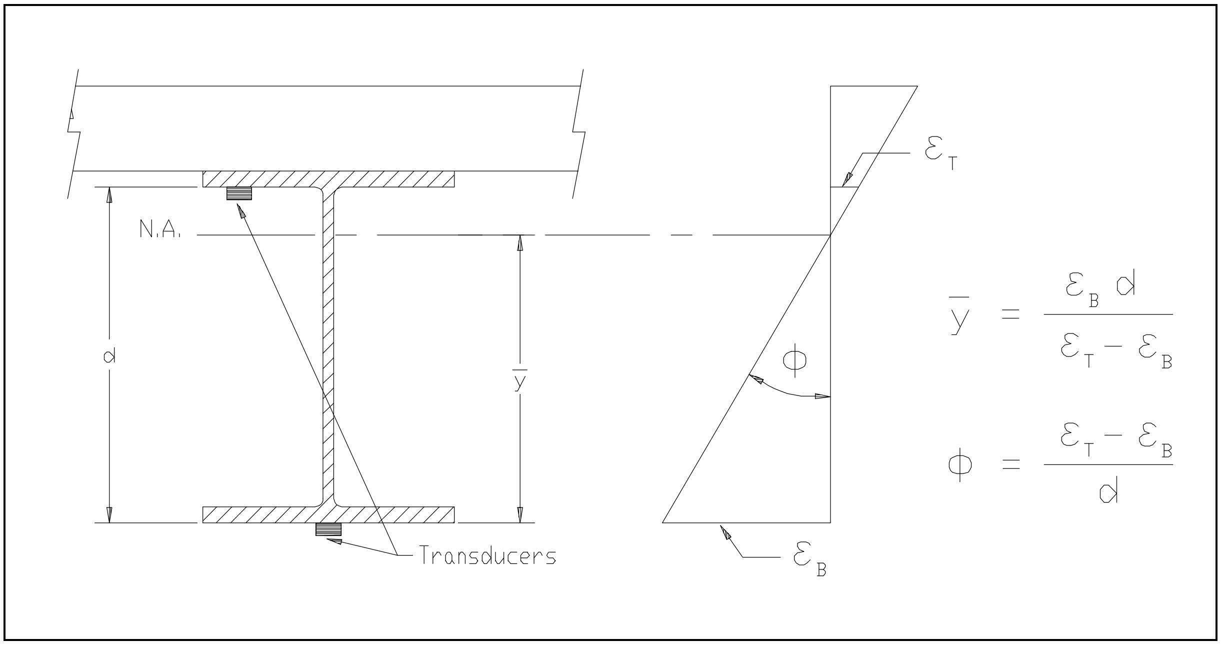



**Figure 26**
**Illustration of neutral axis and curvature calculations**

The consistency in the N.A. values between beams indicates the degree of consistency in beam stiffness. Also, the consistency of the N.A. measurement on a single beam as a function of truck position provides a good quality check for that beam. If for some reason a beam's stiffness changes with respect to the applied moment (i.e., loss of composite action or loss of effective flange width due to a deteriorated deck), it will be observed by a shift in the N.A. history.

Since strain values are translated from a function of time into a function of vehicle position on the structure and the data acquisition channel and the truck position tracked, a considerable amount of bookkeeping is required to perform the strain comparisons. In the past, this required manipulation of result files and spreadsheets, which was tedious and a major source of error. This process is now performed automatically by software, and all of the information can be verified visually.

## Finite Element Modeling and Analysis

The primary function of the load test data is to aid in the development of an accurate finite element model of the bridge. Finite element analysis is used because it provides the most general tool for evaluating various types of structures. Since a comparison of measured and computed responses is performed, it is necessary that the analysis can represent the actual response behavior. This requires that actual geometry and boundary conditions be realistically represented. In maintaining reasonable modeling efforts and computer run times, a certain amount of simplicity is also required, so a planar grid model is generated for most structures and linear-elastic responses are assumed. A grid of frame elements is assembled in the same geometry as the actual structure. Frame elements represent the longitudinal and transverse members of the bridge. The load transfer characteristics of the deck are provided by attaching plate elements to the grid. When end restraints are determined to be present, elastic spring elements having both translational and rotational stiffness terms are inserted at the support locations.

Loads are applied in a manner similar to the actual load test. A model of the test truck, defined by a two-dimensional group of point loads, is placed on the structure model at discrete locations along the same path that the test truck followed during the load test. Gage locations identical to those in the field are also defined on the structure model so that strains can be computed at the same locations under the same loading conditions.

## Evaluation of Rotational End Restraint

A common requirement in structural identification is the need to determine effective spring stiffness that best represents in-situ support conditions. Whereas it is simple to evaluate a spring constant in terms of moment per rotation, the value has little meaning to the engineer. A more conceptual approach is to evaluate the spring stiffness as a percentage of a fully restrained condition. For example: 0 percent being a pinned condition and 100 percent being fixed. This is best accomplished by examining the ratio of the beam or slab stiffness to the rotational stiffness of the support.

As an illustration, a point load is applied to a simple beam with elastic supports, see Figure **27**. By examining the moment diagram, it is apparent that the ratio of the end moment to the mid-span moment ($M_e/M_m$) equals 0.0 if the rotational stiffness ($K_r$) of the springs is equal to 0.0. Conversely, if the value of $K_r$ is set to infinity (rigid), the moment ratio will equal 1.0. If a fixity term is defined as the ratio ($M_e/M_m$), which ranges from 0 to 100 percent, a more conceptual measure of end restraint can be obtained.

The next step is to relate the fixity term to the actual spring stiffness ($K_r$). The degree to which the $K_r$ affects the fixity term depends on the beam or slab stiffness to which the spring is attached. Therefore, the fixity term must be related to the ratio of the beam/spring stiffness. Figure **28** contains a graphical representation of the end restraint effect on a simple beam. Using the graph, a conceptual measure of end-restraint can be defined after the beam and spring constants are evaluated through structural identification techniques.

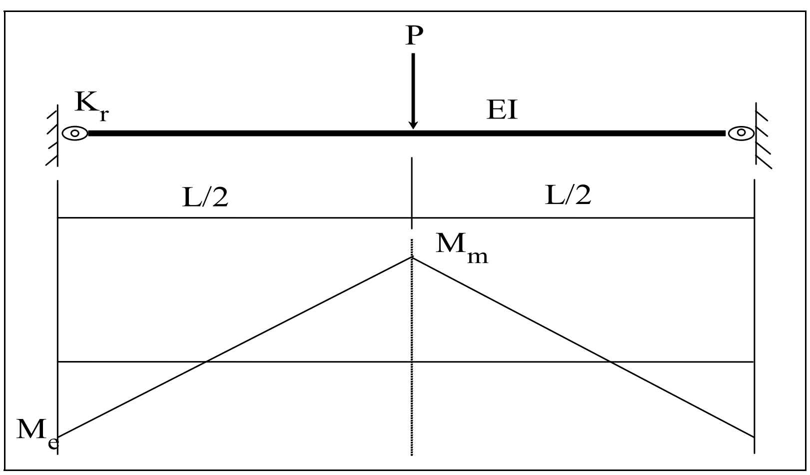



**Figure 27**
**Moment diagram of beam with rotational end restraint**

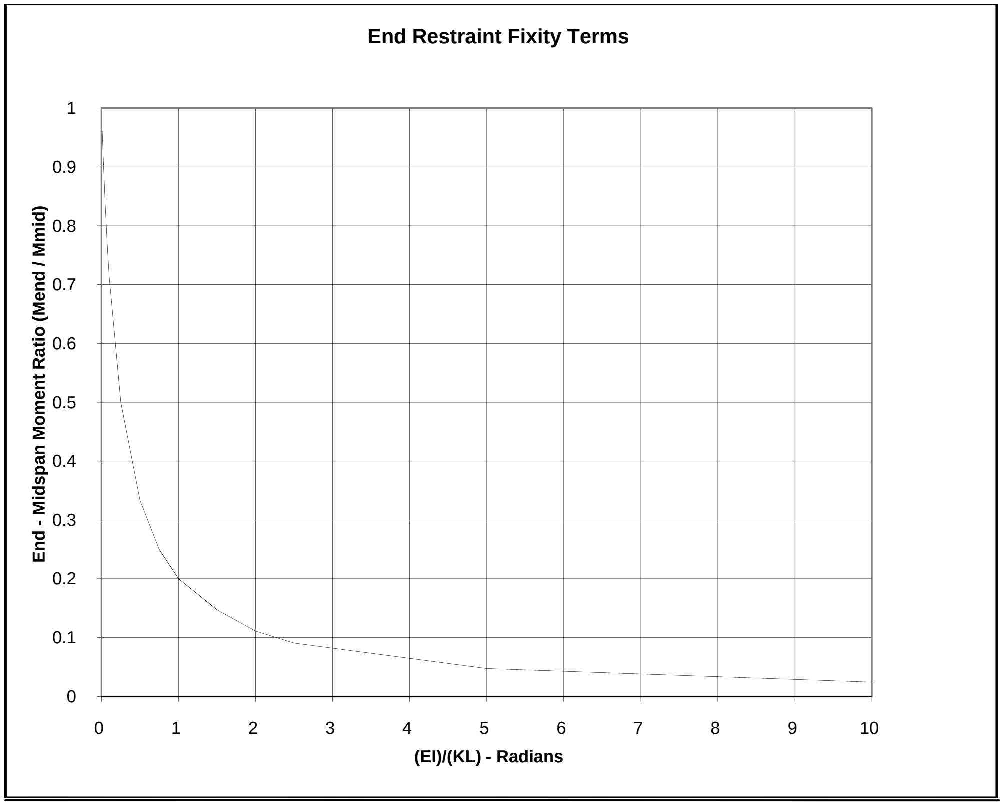



**Figure 28**
**Relationship between spring stiffness and fixity ratio**

## Model Correlation and Parameter Modification

The accuracy of the model is determined numerically by the analysis using several statistical relationships and through visual comparison of the strain histories. The numeric accuracy values are useful in evaluating the effect of any changes to the model, where as the graphical representations provide the engineer with the best perception for why the model is responding differently than the measurements indicate. Member properties that cannot be accurately defined by conventional methods or directly from the field data are evaluated by comparing the computed strains with the measured strains. These properties are defined as variables and are evaluated such that the best correlation between the two sets of data is obtained. It is the engineer's responsibility to determine which parameters need to be refined and to assign realistic upper and lower limits to each parameter. The evaluation of the member property is accomplished with the aid of a parameter identification process (optimizer) built into the analysis. In short, the process consists of an iterative procedure of analysis, data comparison, and parameter modification. It is important to note that the optimization process is merely a tool to help evaluate various modeling parameters. The process works best when the number of parameters is minimized and reasonable initial values are used.

During the optimization process, various error values are computed by the analysis program that provides a quantitative measure of the model accuracy and improvement. The error is quantified in four different ways, each providing a different perspective of the model's ability, to represent the actual structure: an absolute error, a percent error, a scale error, and a correlation coefficient.

The absolute error is computed from the absolute sum of the strain differences. Algebraic differences between the measured and theoretical strains are computed at each gage location for each truck position used in the analysis; therefore, several hundred strain comparisons are generally used in this calculation. This quantity is typically used to determine the relative accuracy from one model to the next and to evaluate the effect of various structural parameters. It is used by the optimization algorithm as the objective function to minimize. Because the absolute error is in terms of micro-strain (mε), the value can vary significantly depending on the magnitude of the strains, the number of gages, and the number of different loading scenarios. For this reason, it has little conceptual value except for determining the relative improvement of a particular model.

A percent error is calculated to provide a better qualitative measure of accuracy. It is computed as the sum of the strain differences squared divided by the sum of the measured strains squared. The terms are squared so that error values of different signs will not cancel each other out, and to put more emphasis on the areas with higher strain magnitudes. A model with acceptable accuracy will usually have a percent error of less than 10 percent.

The scale error is similar to the percent error except that it is based on the maximum error from each gage divided by the maximum strain value from each gage. This number is useful because it is based only on strain measurements recorded when the loading vehicle is in the vicinity of each gage. Depending on the geometry of the structure, the number of truck positions, and various other factors, many of the strain readings are essentially negligible. This error function uses only the most relevant measurement from each gage.

Another useful quantity is the correlation coefficient, which is a measure of the linearity between the measured and computed data. This value determines how well the shape of the computed response histories matches the measured responses. The correlation coefficient can have a value between 1.0 (indicating a perfect linear relationship) and -1.0 (exact opposite linear relationship). A good model will generally have a correlation coefficient greater than 0.90. A poor correlation coefficient is usually an indication that a major error in the modeling process has occurred. This is generally caused by poor representations of the boundary conditions or the loads were applied incorrectly (i.e. truck traveling in wrong direction).

The following table contains the equations used to compute each of the statistical error values:

**Table 41**
**Error functions**

| ERROR FUNCTION | EQUATION |
|---|---|
| **Absolute Error** | $\sum \lvert \varepsilon_m - \varepsilon_c \rvert$ |
| **Percent Error** | $\sum(\varepsilon_m - \varepsilon_c)^2 / \sum(\varepsilon_m)^2$ |
| **Scale Error** | $\dfrac{\sum \max \lvert \varepsilon_m - \varepsilon_c \rvert_{gage}}{\sum \max \lvert \varepsilon_m \rvert_{gage}}$ |
| **Correlation Coefficient** | $\dfrac{\sum(\varepsilon_m - \overline{\varepsilon_m})(\varepsilon_c - \overline{\varepsilon_c})}{\sum\sqrt{(\varepsilon_m - \overline{\varepsilon_m})^2(\varepsilon_c - \overline{\varepsilon_c})^2}}$ |

In addition to the numerical comparisons made by the program, periodic visual comparisons of the response histories are made to obtain a conceptual measure of accuracy. Again, engineering judgment is essential in determining which parameters should be adjusted so as to obtain the most accurate model. The selection of adjustable parameters is performed by determining what properties have a significant effect on the strain comparison and determining which values cannot be accurately estimated through conventional engineering procedures. Experience in examining the data comparisons is helpful; however, two general rules apply concerning model refinement. When the shapes of the computed response histories are similar to the measured strain records but the magnitudes are incorrect, this implies that member stiffness must be adjusted. When the shapes of the computed and measured response histories are not very similar then the boundary conditions or the structural geometry are not well represented and must be refined.

In some cases, an accurate model cannot be obtained, particularly when the responses are observed to be non-linear with load position. Even then, a great deal can be learned about the structure and intelligent evaluation decisions can be made.

# LOAD RATING PROCEDURE

For borderline bridges (those that calculations indicate a posting is required), the primary drawback to conventional bridge rating is an oversimplified procedure for estimating the load applied to a given beam (i.e., wheel load distribution factors) and a poor representation of the beam itself. Due to lack of information and the need for conservatism, material and cross-section properties are generally over-estimated and beam end supports are assumed to be simple when in fact even relatively simple beam bearings have a substantial effect on the mid-span moments. Inaccuracies associated with conservative assumptions are compounded with complex framing geometries. From an analysis standpoint, the goal here is to generate a model of the structure that is capable of reproducing the measured strains. Decisions concerning load rating are then based on the performance of the model once it is proven to be accurate.

The main purpose for obtaining an accurate model is to evaluate how the bridge will respond when standard design loads, rating vehicles, or permit loads are applied to the structure. Since load testing is generally not performed with all of the vehicles of interest, an analysis must be performed to determine load-rating factors for each truck type. Load rating is accomplished by applying the desired rating loads to the model and computing the stresses on the primary members. Rating factors are computed using the equation specified in the *AASHTO Manual for Condition Evaluation of Bridges*, see equation (1).

It is important to understand that diagnostic load testing and the integrated approach are most applicable to obtaining Inventory (service load) rating values. This is because it is assumed that all of the measured and computed responses are linear with respect to load. The integrated approach is an excellent method for estimating service load stress values, but it generally provides little additional information regarding the ultimate strength of particular structural members. Therefore, operating rating values must be computed using conventional assumptions regarding member capacity. This limitation of the integrated approach is not viewed as a serious concern, however, because load responses should never be permitted to reach the inelastic range.

Operating and/or Load Factor rating values must also be computed to ensure a factor of safety between the ultimate strength and the maximum allowed service loads. The safety to the public is of vital importance, but as long as load limits are imposed such that the structure is not damaged then safety is no longer an issue.

The following is an outline describing how field data is used to help in developing a load rating for the superstructure. These procedures will only complement the rating process, and must be used with due consideration to the substructure and inspection reports:

1. Preliminary investigation: Verify linear and elastic behavior through continuity of strain histories, locate neutral axis of flexural members, detect moment resistance at beam supports, and qualitatively evaluate behavior.

2. Develop representative model: Use graphic pre-processors to represent the actual geometry of the structure, including span lengths, girder spacing, skew, transverse members, and deck. Identify gage locations on model identical to those applied in the field.

3. Simulate load test on computer model: Generate two-dimensional model of test vehicle and apply to structure model at discrete positions along same paths defined during field tests. Perform analysis and compute strains at gage location for each truck position.

4. Compare measured and initial computed strain values: Compute and make visual comparisons with post-processor for various global and local error values at each gage location .

5. Evaluate modeling parameters: Improve model based on data comparisons. Engineering judgment and experience is required to determine which variables are to be modified. A combination of direct evaluation techniques and parameter optimization are used to obtain a realistic model. General rules have been defined to simplify this operation.

6. Model evaluation: In some cases, it is not desirable to rely on secondary stiffening effects if it is likely they will not be effective at higher load levels. It is beneficial, though, to quantify their effects on the structural response so that a representative computer model can be obtained. The stiffening effects that are deemed unreliable can be eliminated from the model prior to the computation of rating factors. For instance, if a non-composite bridge is exhibiting composite behavior, then it can conservatively be ignored for rating purposes. However, if it has been in service for 50 years and it is still behaving compositely, chances are that very heavy loads have crossed over it and any bond-breaking would have already occurred. Therefore, probably some level of composite behavior can be relied upon. When unintended composite action is allowed in the rating, additional load limits should be computed based on an allowable shear stress between the steel and concrete and an ultimate load of the non-composite structure.

7. Perform load rating: Apply HS-20 and/or other standard design, rating, and permit loads to the calibrated model. Rating and posting load configuration recommended by AASHTO are shown in
Figure 29 on the following page. The same rating equation specified by the *AASHTO - Manual for the Condition Evaluation of Bridges* is applied:

LRFR Equation

$$RF = \frac{C - \gamma_{DC}(DC) - \gamma_{DW}(DW) \pm \gamma_P(P)}{\gamma_L(LL + IM)} \tag{1}$$

where,

$RF$ = Rating Factor for individual member,

$C$ = Member Capacity,

$\gamma_{DC}$ = LRFD load factor for structural components and attachments,

$DC$ = Dead-load effect due to structural components,

$\gamma_{DW}$ = LRFD load factor for wearing surfaces and utilities,

$DW$ = Dead-load effect due to wearing surface and utilities,

$\gamma_P$ = LRFD load factor for permanent loads other than dead loads = 1.0,

$P$ = Permanent loads other than dead loads,

$LL$ = Live-load effect, and

*IM* = Impact effect, either AASHTO or measured.

The only difference between this rating technique and standard beam rating programs is that a more realistic model is used to determine the dead-load and live-load effects. Two-dimensional loading techniques are applied because wheel load distribution factors are not applicable to a planar model. Stress envelopes are generated for several truck paths, envelopes for paths separated by normal lane widths are combined to determine multiple lane loading effects.

8. Consider other factors: Other factors, such as the condition of the deck and/or substructure, traffic volume, and other information in the inspection report, should be taken into consideration and the rating factors adjusted accordingly.

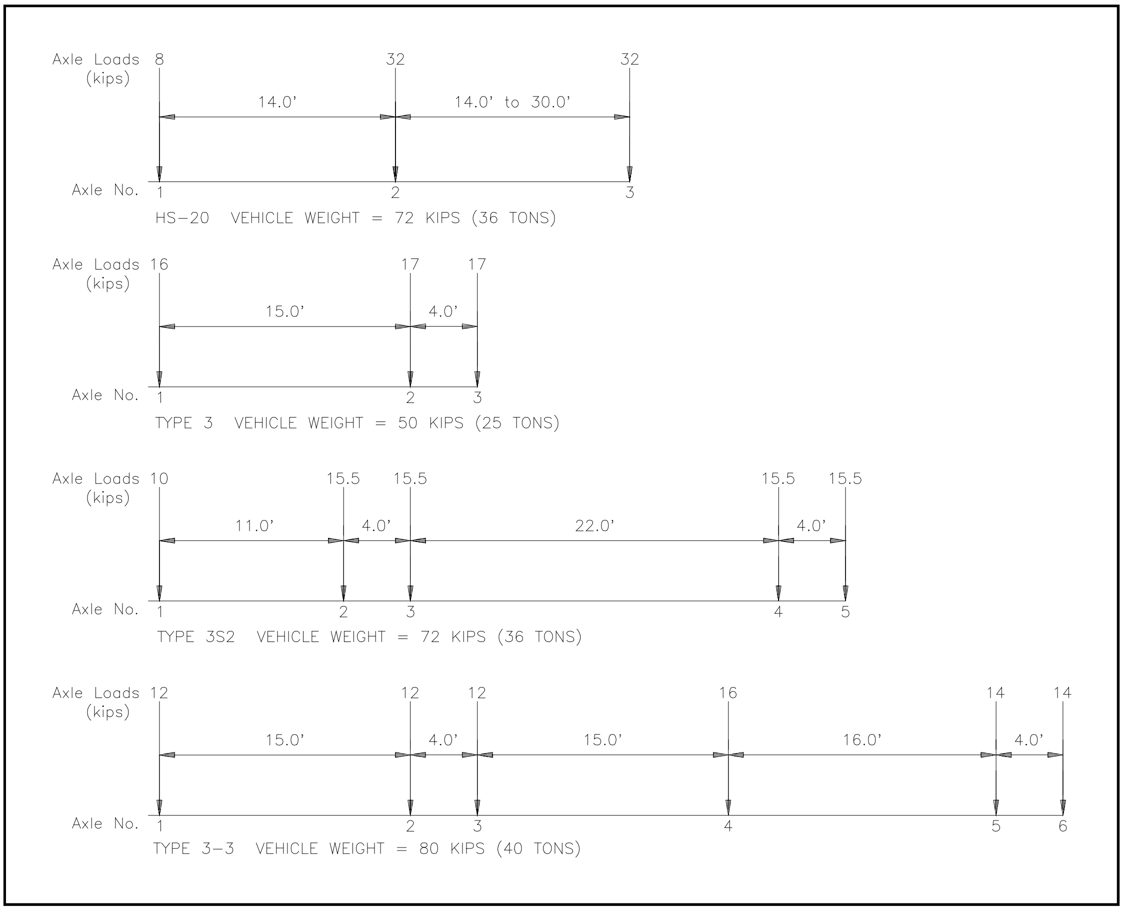



**Figure 29**
**AASHTO rating and posting load configurations**

# APPENDIX P

# PARAMETRIC STUDIES

# ANALYSIS OVERVIEW

The methodology used in the analysis phase evaluated the effect of the heavy loads on the bridges from the trucks transporting sugarcane products, based on LRFD and LFD design recommendations. The demand on the bridge girders due to the heavy truck loads was calculated based on bridge girder type, span type and the bridge geometry. Finite element analysis will be used in this task of the research.

The effects of sugarcane truck on Louisiana bridges were determined by comparing the stress of the longitudinal stress at the top and bottom surface of the girder, the vertical deflection of the girder, the stress state of the deck, and the axial force of the diaphragms of the bridges under their design load to the conditions under the Louisiana sugarcane truck configuration as shown in figure 1. A simplified method based on AASHTO design guidelines was determined to be the most prudent approach to meet the short and strict schedule for this study.

The short and long term effects of sugarcane truck loads were determined based on the ratio of the stress, force and deflection for each bridge in the sample. The AASHTO Line Girder Analysis approach, detailed analysis using finite element models, and GTSTRUDL Software were used. The design load HS20-44 for the bridge was used. The truck loads for hauling sugarcane were based on the 3S3 truck configuration, with maximum tridem load of 60,000 lb., maximum tandem load of 48,000 lb. and steering axle of 12, 000 lb., according to LADOTD Bridge Manual.

The first step in the analysis used the influence line procedures to determine the critical location of the trucks on the bridges that would result in maximum moment and shear forces. Based on the results from the influence line analyses, the further analysis of bridge girder, deck and diaphragm were applied, and the effects of the loads on the bridge girders and bridge decks were determined. Next, the ratios of the results for the 3S3 truck and the design truck (HS20-44) for stresses were calculated. The serviceability criteria were evaluated for simply supported girders based on their deflections.

## ANALYSIS VARIABLES

The span length was measured from the center of one support to the center of an adjacent support. The girder spacing was measured from the center of one girder to the center of an adjacent girder, which was identical and parallel to the previous girder. The model considered in this study was non-skewed with end diaphragms. Based on the girder numbers, the models were divided into two groups. The structures of both groups analyzed in this study were 30 feet wide. For group A, the girders were simply supported and spaced at 8 feet in the middle and 7 feet on the outside. The model contained five AASHTO Type IV, V, VI or Bulb-Tee 54, 63, 72 girders with the span length 90 feet and the slab thickness kept 8 inches as a constant; for group B, the girders were simply supported and spaced at 4.5 feet in the middle and 6 feet on the outside. The model contained seven AASHTO Type IV, V, VI or Bulb-Tee 54, 63, 72 girders with the slab thickness kept 8 inches as the constant; the span length of the model was identified with the maximum allowable displacement of the girders. The geometry of the bridge and its deck are shown in Figures 30, 31, and 32.

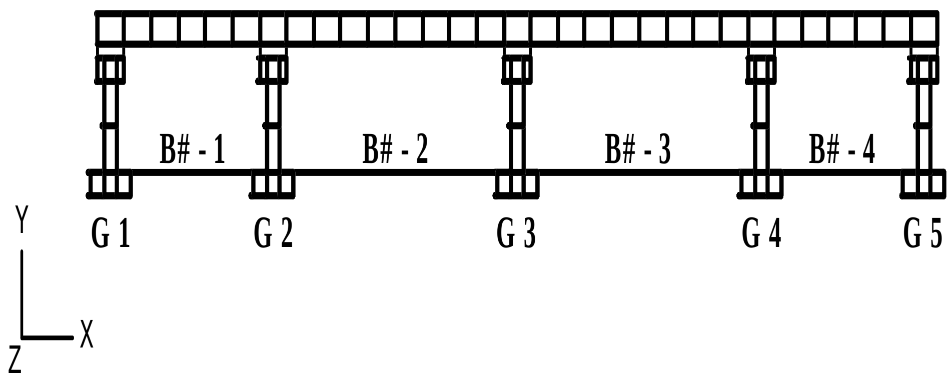



**Figure 30**
**Models used for bridge analysis – Five Girders' Model**

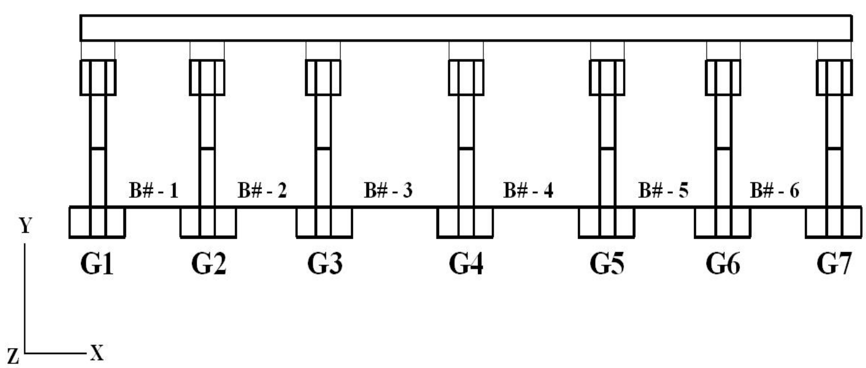



**Figure 31**
**Models used for bridge analysis – Seven Girders' Model**

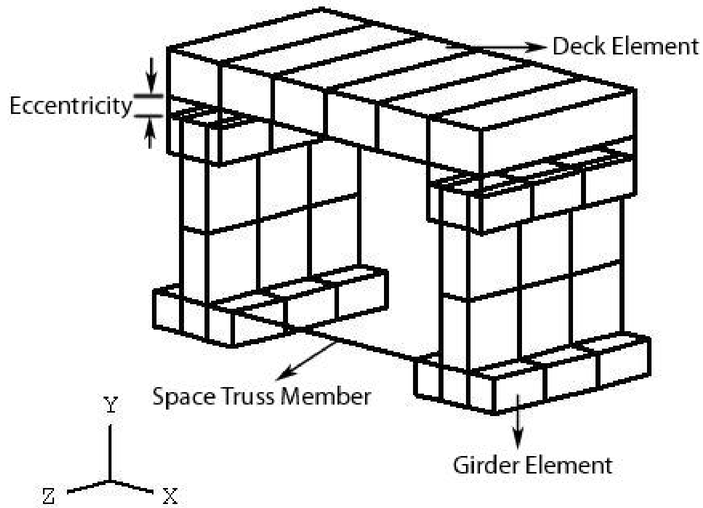



**Figure 32**
**Typical plate and girder elements**

The trucks that haul sugarcane in Louisiana were similar to the Type 3S3 truck configuration, as shown in Figure 33. The original design load HS20-44, was used. All truck loads were placed on the bridge as shown in Figure 33 and Figure 34.

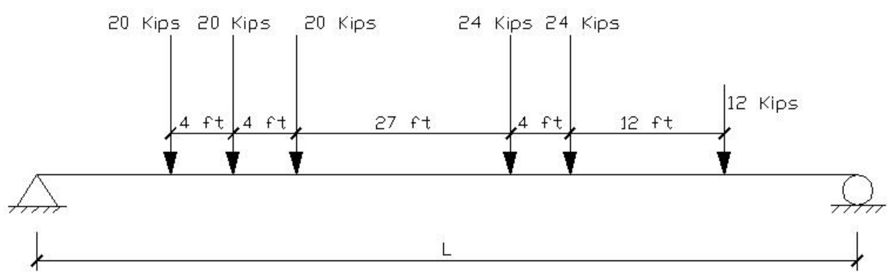



**Figure 33**
**Louisiana sugarcane truck loads on simple span bridge**

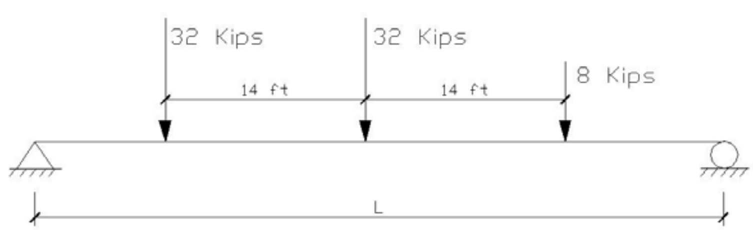



**Figure 34**
**HS20-44 truck loads on simple span bridge**

After the finite element analysis, the stress state and deflection along the bridge length of girders were considered as the key results to determine the effect of sugarcane truck load.

## Application Method

In this project, the finite element analysis of the bridge was performed by GTSTRUDL soft ware. The finite element model used for bridge in this study simulated the behavior of simple span bridges. The girders were modeled using Type-IPSL tridimensional elements available in GTSTRUDL. Type-SBCR plate elements were used for the bridge deck. Prismatic space truss members were used to model end diaphragms and the connection between the deck plate elements and the girder elements.

### Girder Element Type-IPSL

Properties of type tridimensional finite elements were explained in the GTSTRUDL user guide analysis. These were used to model the behavior of general three-dimensional solid bodies. Three translational degrees of freedom in the global X, Y, and Z directions were considered per node. Only force type loads could be applied to these tridimensional elements.

The Type-IPSL tridimensional finite element used was an eight-node element capable of carrying both joint loads and element loads. The joint loads could define concentrated loads or temperature changes, while the element loads could define edge loads, surface loads, or body loads. GTSTRUDL results included the output for stress, strain, and element forces for type-IPSL tridimensional elements at each node. The average stresses and average strains at each node were calculated. The details of the Type-IPSL element were shown in Table 42.

**Table 42**
**Detail properties of Type-IPSL tridimensional element**

| Element | | Output | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Name | Shape | List | | | | | Calculate Average | | | | |
| | Resultants | Stress | Strain | Principal Stresses | Principal Strain | Element Forces | Stresses | Strain | Principal Stresses | Principal Strain | von Mises |
| IPSL |  | N | N | | | N | X | X | X | X | X |

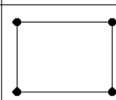
N - Output Element Nodes

## Plate Element Type-SBCR

Properties of type plate finite elements were explained in the GTSTRUDL User Guide Analysis. Type plate elements were used to model problems that involved both stretching and bending behavior. The element was a two-dimensional flat plate element commonly used to model thin-walled, curved structures. This type plate finite elements was formulated as a superposition of type plane stress and type plate bending finite elements. For flat plate structures, the stretching and bending behavior was uncoupled, but for structures where the elements did not lie in the same plane, the stretching and bending behavior was coupled.

The Type-SBCR plate finite element was a four-node element capable of carrying both joint loads and element loads. The joint loads could define concentrated loads, temperature change loads, or temperature gradients, while the element loads could define surface loads or body loads. GTSTRUDL provided the output for in-plane stresses at the centroid and moment resultants, the shear resultant, and element forces at each node for Type-SBCR plate elements. The average stresses, average principal stresses, and average resultants at each node were calculated. The details of the Type-SBCR plate element were shown in table 43.

**Table 43**
**Detail properties of type-SBCR plate element**

| Element | | Output | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Name | Shape | List | | | | Calculate Average | | | | | |
| | | Stress/Moment | Shear Resultants | Strain/Curvature | Element Forces | Stresses | Principal Stresses | Resultants | Principal Membrane Resultants | Principal Bending Resultants | von Mises |
| SBCR |  | * | N | | N | X | X | X | X | X | X |

N - Output Element Nodes
* - In Plane-Stress at Centroid, Moments Resultants at Nodes

## Prismatic Space Truss Members

Properties of space truss members were explained in the GTSTRUDL User Guide Analysis. Space truss members were used when a member experienced only axial forces and where the member was ideally pin connected to each joint. No force or moment loads could be applied to a space truss member. Only constant axial temperature changes or constant initial strain type loads could be applied. The self weight of these members was generated as joint loads which the member was incident upon.

When the prismatic member property option was used, the section properties were assumed to be constant over the entire length of the member. Up to 14 prismatic section properties could be directly specified or stored in tables. If not specified, the values could be assumed according to the material specified. All 14 member cross-section properties were assumed to be with respect to the member cross-section's principal axis (local y- and z- axes) which had their origin on the centroidal axis (local x- axis) of the member. Table 44 listed the detail properties of the prismatic space truss member.

**Table 44**
**Detail properties of prismatic space truss member**

| Member Type | Member Parallel To Globel Plane | Direction of Member Local x-axis | Beta Angle | Local Member Degree-of-Freedom | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Force | | | Moment | | |
| | | | | x | y | z | x | y | z |
| Space Truss | N/A | N/A | N/A | x | | | | | |

N/A - Not Applicatable

## Boundary Conditions

The restraints for all models consisted of four joints across the width of the base of the girder at the end and intermediate supports. Also, the two joints that connect the plate elements to the rigid members at the end supports behaved as pins.

## AASHTO Loading

A uniform volumetric dead load of 150 pcf was applied to all elements and all members to account for the self weight of the concrete. The truck loading on the bridge was represented by the HS20-44 and 3S3 truck loading. In addition to the dead and truck loads, a future wearing surface loading of 12 psf, according to LADOTD Bridge Manual, was placed on the deck to account for future overlays. Based on AASHTO Chapter 3, four kinds of load combinations were used in this study, and corresponding loading condition factors were applied to the model, as shown in Table 45. In the load combination "fatigue," the impact factor 1.3 was applied to all truck, as required by the AASHTO LRFD Bridge Design Manual, chapter 3.

**Table 45**
**AASHTO LRFD bridge design loading condition factors**

| Load Combination | Dead Load (DL) | Vehicular Live Load (LL) | Live Load Surcharge (LS) | Wind Load (WL) |
|---|---|---|---|---|
| Strength I Max | 1.25 | 1.75 | 1.75 | 0.00 |
| Strength II Max | 1.25 | 1.35 | 1.35 | 0.00 |
| Strength III Max | 1.25 | 0.00 | 0.00 | 1.40 |
| Strength V Max | 1.25 | 1.35 | 1.35 | 0.40 |
| Fatigue | | 0.75*1.3=0.975 | | |

## Bridge Model Analysis

### Influence Lines Analysis

When the truck loads, performed as the concentrated loads, were placed on the bridge deck, an influence surface could be generated. Instead of using the influence surfaces to find the critical moments, shear, and deflection under certain load conditions, the influence line was used. The bending moment and shear for which the influence line was to be determined was computed as a unit load placed at different positions over the length and the width of the bridge. The maximum deflection was computed by superposition.

In this study, HS20-44 truck loads, and Louisiana sugarcane truck loads were used in the analysis procedure. Both hand calculations and computer models in GTSTRUDL were used to determine the critical load location and the corresponding moment and shear forces. Also, associated deflections and stresses in the bridge girders and bridge decks were determined.

To determine the critical location of the truck on the bridge, an influence line analysis on the transverse direction was required. The width of the bridge was 30 feet, supported by 5 girders with simple supports. The space between the central 3 girders was 8 feet, and was 7 feet to the outer girders. Truck loads were placed on the deck as concentrated loads. GTSTRUDL was used to obtain the influence line for each joint of the deck, and Excel was used to analyze the data to get the critical location of the truck.

### Bridge Girder Evaluation

Bridge girder, is a straight, horizontal beam to span an opening and carry weight distributed from the bridge deck. By the difference of the shape of girder cross section, it can be divided into I section, Tee section, box section, and so on. The AASHTO type IV girder, type V girder, type VI girder, Bulb-Tee 54, Bulb-Tee 63 and Bulb-Tee 72 are typical I section girders and widely used in the United States. To evaluate this girder performance under the heavy truck loads, the displacement and the stress state of the girder must be determined. Both of the short term effect and the long term effect of the girder under the truck load must be evaluated.  Table 36 below lists the bridge models used in this study.

**Table 46**
**Bridge models and their specifications used in this study**

| Bridge Group | Bridge Model | Girder Type | # of Girders | Span Length | # of Spans | Support Condition | Applied Truck Load |
|---|---|---|---|---|---|---|---|
| Group A (5 Girders' Model) | 316 | AASHTO Type IV | 5 | 90 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 326 | AASHTO BT-54 | 5 | 90 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 336 | AASHTO BT-63 | 5 | 90 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 346 | AASHTO BT-72 | 5 | 90 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 356 | AASHTO Type V | 5 | 100 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 366 | AASHTO Type VI | 5 | 110 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| Group B (7 Girders' Model) | 376 | AASHTO Type V | 7 | 110 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 386 | AASHTO Type VI | 7 | 120 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 396 | AASHTO Type IV | 7 | 100 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 406 | AASHTO BT-54 | 7 | 100 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 416 | AASHTO BT-63 | 7 | 105 ft | 1 | Simply Supported | HS20-44 & Sugarcane |
| | 426 | AASHTO BT-72 | 7 | 120 ft | 1 | Simply Supported | HS20-44 & Sugarcane |

**Short term stress performances of AASHTO type IV girder bridges in group A.** In this study, four load combinations were used in the analysis. During these four load combinations, the Strength I max, Strength III max, and Strength V max were used to evaluate the short term performances of the bridge girders. By comparing the stress state of bridge girders under these load combinations, the load combination “Strength I max” lead the maximum stresses of the girder, as shown in tables 47 and 48, so we can determine that the “Strength I max” is the governing load combination, and all the analysis below are based on it.

**Table 47**
**Max. and min. stresses of AASHTO type IV girders in group A with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.5766 | -0.9478 | 0.4177 | -0.8595 | 0.4885 | -0.9368 |
| | SYY | 0.6289 | -1.1718 | 0.4888 | -1.0457 | 0.6021 | -1.1522 |
| | SZZ | 1.2236 | -4.4079 | 1.0604 | -4.0104 | 1.1986 | -4.3617 |
| | SXY | 0.2939 | -0.1965 | 0.1450 | -0.1327 | 0.2606 | -0.1804 |
| | SXZ | 0.4824 | -0.4953 | 0.5834 | -0.5834 | 0.5205 | -0.5304 |
| | SYZ | 1.8635 | -1.8662 | 1.6344 | -1.6344 | 1.7480 | -1.7484 |
| Girder 2 | SXX | 1.9388 | -2.6675 | 0.6353 | -1.0551 | 1.6371 | -2.3038 |
| | SYY | 0.5803 | -1.2613 | 0.4277 | -0.9975 | 0.5051 | -1.1367 |
| | SZZ | 1.2473 | -4.7307 | 1.0257 | -4.1160 | 1.1730 | -4.6345 |
| | SXY | 0.8422 | -0.3655 | 0.3350 | -0.1342 | 0.7277 | -0.3101 |
| | SXZ | 0.4987 | -0.5141 | 0.5851 | -0.5851 | 0.5335 | -0.5453 |
| | SYZ | 2.0526 | -2.0586 | 1.7008 | -1.7008 | 1.9097 | -1.9107 |
| Girder 3 | SXX | 2.2235 | -3.0516 | 0.6629 | -1.1083 | 1.8518 | -2.6150 |
| | SYY | 0.5895 | -1.3415 | 0.4526 | -1.0842 | 0.5575 | -1.2790 |
| | SZZ | 1.2959 | -5.0534 | 1.0030 | -4.1932 | 1.2410 | -4.9010 |
| | SXY | 0.8645 | -0.7544 | 0.3092 | -0.2855 | 0.7223 | -0.6420 |
| | SXZ | 0.4364 | -0.4406 | 0.5503 | -0.5503 | 0.4776 | -0.4808 |
| | SYZ | 2.2198 | -2.2253 | 1.7481 | -1.7481 | 2.0771 | -2.0808 |
| Girder 4 | SXX | 2.2147 | -3.0423 | 0.6386 | -1.0875 | 1.8508 | -2.6022 |
| | SYY | 0.6263 | -1.2979 | 0.4406 | -1.0658 | 0.5404 | -1.2537 |
| | SZZ | 1.3407 | -4.9406 | 1.0215 | -4.0410 | 1.2783 | -4.6489 |
| | SXY | 0.4044 | -0.9424 | 0.1699 | -0.3379 | 0.3424 | -0.8058 |
| | SXZ | 0.5039 | -0.4887 | 0.4936 | -0.4936 | 0.3863 | -0.3747 |
| | SYZ | 2.1307 | -2.1371 | 1.6952 | -1.6952 | 2.0038 | -2.0087 |
| Girder 5 | SXX | 0.7172 | -1.0155 | 0.2449 | -1.0188 | 0.5785 | -0.9959 |
| | SYY | 0.6496 | -1.2747 | 0.4930 | -1.2158 | 0.5817 | -1.1983 |
| | SZZ | 1.3247 | -4.7042 | 0.8388 | -4.7769 | 1.2133 | -4.6628 |
| | SXY | 0.2253 | -0.3594 | 0.2227 | -0.1738 | 0.2243 | -0.3194 |
| | SXZ | 0.5004 | -0.4844 | 0.7566 | -0.7566 | 0.4067 | -0.3947 |
| | SYZ | 1.9811 | -1.9855 | 2.0378 | -2.0378 | 1.9516 | -1.9506 |

**Table 48**
**Max. and min. stresses of AASHTO type IV girders in group A with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.7006 | -0.9940 | 0.4177 | -0.8595 | 0.5841 | -0.9724 |
| | SYY | 0.7000 | -1.2175 | 0.4888 | -1.0457 | 0.6569 | -1.1874 |
| | SZZ | 1.3118 | -4.6382 | 1.0604 | -4.0104 | 1.2666 | -4.5393 |
| | SXY | 0.3520 | -0.2005 | 0.1450 | -0.1327 | 0.3055 | -0.1810 |
| | SXZ | 0.5273 | -0.5490 | 0.5834 | -0.5834 | 0.5551 | -0.5719 |
| | SYZ | 1.9487 | -1.9527 | 1.6344 | -1.6344 | 1.8105 | -1.8103 |
| Girder 2 | SXX | 2.2842 | -3.1412 | 0.6353 | -1.0551 | 1.9035 | -2.6692 |
| | SYY | 0.6747 | -1.3497 | 0.4277 | -0.9975 | 0.5692 | -1.2049 |
| | SZZ | 1.3075 | -5.0748 | 1.0257 | -4.1160 | 1.2051 | -4.8999 |
| | SXY | 0.9841 | -0.4166 | 0.3350 | -0.1342 | 0.8372 | -0.3495 |
| | SXZ | 0.5595 | -0.5931 | 0.5851 | -0.5851 | 0.5804 | -0.6063 |
| | SYZ | 2.1946 | -2.2091 | 1.7008 | -1.7008 | 2.0098 | -2.0169 |
| Girder 3 | SXX | 2.5484 | -3.4847 | 0.6629 | -1.1083 | 1.8518 | -2.6150 |
| | SYY | 0.6525 | -1.4827 | 0.4526 | -1.0842 | 0.5575 | -1.2790 |
| | SZZ | 1.2186 | -5.6127 | 1.0030 | -4.1932 | 1.2410 | -4.9010 |
| | SXY | 0.9874 | -0.8830 | 0.3092 | -0.2855 | 0.7223 | -0.6420 |
| | SXZ | 0.4959 | -0.5102 | 0.5503 | -0.5503 | 0.4776 | -0.4808 |
| | SYZ | 2.4619 | -2.4877 | 1.7481 | -1.7481 | 2.0771 | -2.0808 |
| Girder 4 | SXX | 2.5377 | -3.4738 | 0.6386 | -1.0875 | 2.0999 | -2.9351 |
| | SYY | 0.7085 | -1.4172 | 0.4406 | -1.0658 | 0.5913 | -1.3458 |
| | SZZ | 1.3790 | -5.4159 | 1.0215 | -4.0410 | 1.3078 | -4.9763 |
| | SXY | 0.4531 | -1.0836 | 0.1699 | -0.3379 | 0.3800 | -0.9148 |
| | SXZ | 0.5951 | -0.5546 | 0.4936 | -0.4936 | 0.4567 | -0.4255 |
| | SYZ | 2.3290 | -2.3510 | 1.6952 | -1.6952 | 2.1568 | -2.1737 |
| Girder 5 | SXX | 0.9075 | -1.2686 | 0.2449 | -1.0188 | 0.7253 | -1.1006 |
| | SYY | 0.7282 | -1.3595 | 0.4930 | -1.2158 | 0.6303 | -1.2704 |
| | SZZ | 1.4490 | -5.0692 | 0.8388 | -4.7769 | 1.3080 | -4.8975 |
| | SXY | 0.2423 | -0.4445 | 0.2227 | -0.1738 | 0.2386 | -0.3851 |
| | SXZ | 0.5765 | -0.5401 | 0.7566 | -0.7566 | 0.4034 | -0.3805 |
| | SYZ | 2.1195 | -2.1294 | 2.0378 | -2.0378 | 2.0524 | -2.0524 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination “Strength I max” because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 3 and 4 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 3.27 ksi occurred at girder 4 at 51 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4. The maximum tensile stress at the bottom of the girder, 1.34 ksi occurred at girder 4 at 50 feet along the bridge, while at most other locations, there wasn't much difference between girder 3 and 4.

For the Sugarcane truck load, girders 3, 4 and 5 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 3.50 ksi occurred at girder 4 at 39 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than them in girder 4. At distance 42 feet to 50 feet, the stresses in girder 5 were governing. The maximum tensile stress at the bottom of the girder, 1.45 ksi occurred at girder 5 at 49 feet along the bridge, while at most other locations, there wasn't much difference among girder 3, 4 and 5.

**Short term stress performances of AASHTO type V girder bridges in group A.** For the bridges with AASHTO Type V girder, by comparing the stress state of bridge girders under all load combinations, the load combination "Strength I max" also lead to the maximum stresses of the girder, as shown in tables 49 and 50, so we can determine that the "Strength I max" is the governing load combination, and all the analysis below are based on it.

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination "Strength I max" because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface. For the truck load HS20-44, girder 3 and 4 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 2.32 ksi occurred at girder 3 at 48 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4. The maximum tensile stress at the bottom of the girder, was 1.14 ksi occurred at girder 4 at 58 feet along the bridge, while at most other locations, there wasn't much difference between girder 3 and 4.

For the Sugarcane truck load, girder 3 and 4 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 2.44 ksi occurred at girder 4 at 59 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4. The maximum tensile stress at the bottom of the girder was 1.18 ksi occurred at girder 4 at 58 feet along the bridge, while at most other locations, there wasn't much difference among girder 3 and 4.

**Table 49**
**Max. and min. Stresses of AASHTO Type V girders in group A with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4367 | -0.9202 | 0.4050 | -0.8655 | 0.4113 | -0.9164 |
| | SYY | 0.7562 | -1.1864 | 0.6188 | -1.0912 | 0.7287 | -1.1730 |
| | SZZ | 1.0156 | -4.2319 | 0.8600 | -3.9999 | 0.9863 | -4.2216 |
| | SXY | 0.2518 | -0.1958 | 0.1476 | -0.1092 | 0.2237 | -0.1640 |
| | SXZ | 0.4194 | -0.4343 | 0.5449 | -0.5449 | 0.4620 | -0.4735 |
| | SYZ | 1.5268 | -1.5283 | 1.4684 | -1.4684 | 1.5292 | -1.5303 |
| Girder 2 | SXX | 1.3436 | -0.9927 | 0.3760 | -0.8769 | 1.1186 | -0.9521 |
| | SYY | 0.7129 | -1.2924 | 0.5715 | -1.0939 | 0.6845 | -1.1997 |
| | SZZ | 1.0856 | -4.4467 | 0.8593 | -4.0645 | 1.0362 | -4.4019 |
| | SXY | 0.6703 | -0.1973 | 0.2563 | -0.1127 | 0.5773 | -0.1771 |
| | SXZ | 0.4245 | -0.4441 | 0.5452 | -0.5452 | 0.4662 | -0.4814 |
| | SYZ | 1.6282 | -1.6331 | 1.5091 | -1.5091 | 1.6165 | -1.6203 |
| Girder 3 | SXX | 1.4768 | -1.0416 | 0.3927 | -0.8912 | 1.2144 | -0.9831 |
| | SYY | 0.7434 | -1.3372 | 0.5534 | -1.1560 | 0.7038 | -1.2935 |
| | SZZ | 1.1004 | -4.5548 | 0.8329 | -4.0861 | 1.0457 | -4.4904 |
| | SXY | 0.6813 | -0.6110 | 0.2570 | -0.2393 | 0.5867 | -0.5274 |
| | SXZ | 0.3785 | -0.3829 | 0.5221 | -0.5221 | 0.4256 | -0.4289 |
| | SYZ | 1.7030 | -1.7044 | 1.5340 | -1.5340 | 1.6770 | -1.6793 |
| Girder 4 | SXX | 1.4656 | -1.0786 | 0.3606 | -0.8736 | 1.2083 | -0.9494 |
| | SYY | 0.7377 | -1.3032 | 0.5525 | -1.1451 | 0.6921 | -1.2756 |
| | SZZ | 1.1375 | -4.5374 | 0.8306 | -3.9940 | 1.0738 | -4.3133 |
| | SXY | 0.2283 | -0.7253 | 0.2134 | -0.2666 | 0.2115 | -0.6226 |
| | SXZ | 0.4363 | -0.4173 | 0.4891 | -0.4891 | 0.3593 | -0.3460 |
| | SYZ | 1.6699 | -1.6751 | 1.5077 | -1.5077 | 1.6268 | -1.6257 |
| Girder 5 | SXX | 0.5087 | -0.9548 | 0.2796 | -0.9615 | 0.4021 | -0.9353 |
| | SYY | 0.7813 | -1.2426 | 0.5940 | -1.2222 | 0.7277 | -1.2096 |
| | SZZ | 1.0617 | -4.3786 | 0.8809 | -4.4334 | 0.9728 | -4.2974 |
| | SXY | 0.2101 | -0.2895 | 0.2462 | -0.1648 | 0.2239 | -0.2641 |
| | SXZ | 0.4373 | -0.4190 | 0.6401 | -0.6401 | 0.3763 | -0.3626 |
| | SYZ | 1.5870 | -1.5899 | 1.6837 | -1.6837 | 1.6086 | -1.6072 |

**Table 50**
**Max. and min. stresses of AASHTO type V girders in group A with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.5409 | -0.9643 | 0.4050 | -0.8655 | 0.4478 | -0.9504 |
| | SYY | 0.8160 | -1.2265 | 0.6188 | -1.0912 | 0.7749 | -1.2040 |
| | SZZ | 1.0849 | -4.4546 | 0.8600 | -3.9999 | 1.0398 | -4.3934 |
| | SXY | 0.3088 | -0.2061 | 0.1476 | -0.1092 | 0.2676 | -0.1719 |
| | SXZ | 0.4532 | -0.4801 | 0.5449 | -0.5449 | 0.4881 | -0.5088 |
| | SYZ | 1.5912 | -1.5958 | 1.4684 | -1.4684 | 1.5788 | -1.5823 |
| Girder 2 | SXX | 1.6243 | -1.2002 | 0.3760 | -0.8769 | 1.3352 | -1.0141 |
| | SYY | 0.7686 | -1.3711 | 0.5715 | -1.0939 | 0.7274 | -1.2524 |
| | SZZ | 1.1532 | -4.7834 | 0.8593 | -4.0645 | 1.0643 | -4.6616 |
| | SXY | 0.8025 | -0.2400 | 0.2563 | -0.1127 | 0.6792 | -0.2100 |
| | SXZ | 0.4713 | -0.5181 | 0.5452 | -0.5452 | 0.5024 | -0.5385 |
| | SYZ | 1.7294 | -1.7463 | 1.5091 | -1.5091 | 1.6946 | -1.7076 |
| Girder 3 | SXX | 1.7791 | -1.2422 | 0.3927 | -0.8912 | 1.4476 | -1.0518 |
| | SYY | 0.8302 | -1.4505 | 0.5534 | -1.1560 | 0.7708 | -1.3805 |
| | SZZ | 1.0801 | -4.9684 | 0.8329 | -4.0861 | 1.0103 | -4.8094 |
| | SXY | 0.8063 | -0.7353 | 0.2570 | -0.2393 | 0.6831 | -0.6233 |
| | SXZ | 0.4181 | -0.4319 | 0.5221 | -0.5221 | 0.4561 | -0.4667 |
| | SYZ | 1.8505 | -1.8585 | 1.5340 | -1.5340 | 1.7912 | -1.8000 |
| Girder 4 | SXX | 1.7680 | -1.2942 | 0.3606 | -0.8736 | 1.4416 | -1.0873 |
| | SYY | 0.8063 | -1.3963 | 0.5525 | -1.1451 | 0.7449 | -1.3475 |
| | SZZ | 1.1813 | -4.9407 | 0.8306 | -3.9940 | 1.1075 | -4.5516 |
| | SXY | 0.2884 | -0.8590 | 0.2134 | -0.2666 | 0.2579 | -0.7257 |
| | SXZ | 0.5154 | -0.4638 | 0.4891 | -0.4891 | 0.3942 | -0.3544 |
| | SYZ | 1.7970 | -1.8158 | 1.5077 | -1.5077 | 1.7185 | -1.7114 |
| Girder 5 | SXX | 0.6580 | -1.0168 | 0.2796 | -0.9615 | 0.5172 | -0.9729 |
| | SYY | 0.8452 | -1.3072 | 0.5940 | -1.2222 | 0.7750 | -1.2633 |
| | SZZ | 1.1482 | -4.6822 | 0.8809 | -4.4334 | 1.0459 | -4.4683 |
| | SXY | 0.2256 | -0.3694 | 0.2462 | -0.1648 | 0.2359 | -0.3257 |
| | SXZ | 0.4956 | -0.4577 | 0.6401 | -0.6401 | 0.3758 | -0.3471 |
| | SYZ | 1.6797 | -1.6878 | 1.6837 | -1.6837 | 1.6737 | -1.6685 |

**Short term stress performances of AASHTO type VI girder bridges in group A.** For the bridges with AASHTO Type VI girder, by comparing the stress state of bridge girders under all load combinations, the load combination “Strength I max” resulted with the maximum stresses of the girder, as shown in tables 51 and 52, so we can determine that the “Strength I max” is the governing load combination, and all the analysis below are based on it.

**Table 51**
**Max. and min. stresses of AASHTO type VI girders in group A with HS20-44 truck load**

| Unit: Ksi | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4599 | -1.0373 | 0.5017 | -1.0459 | 0.4777 | -1.0523 |
| | SYY | 0.7727 | -1.3603 | 0.6626 | -1.3132 | 0.7525 | -1.3621 |
| | SZZ | 1.0242 | -4.7437 | 0.9772 | -4.8388 | 1.0143 | -4.8299 |
| | SXY | 0.2565 | -0.2132 | 0.1950 | -0.1315 | 0.2288 | -0.1693 |
| | SXZ | 0.4560 | -0.4672 | 0.7054 | -0.7054 | 0.5339 | -0.5426 |
| | SYZ | 1.7290 | -1.7313 | 1.7839 | -1.7839 | 1.7652 | -1.7669 |
| Girder 2 | SXX | 1.2985 | -1.0713 | 0.3466 | -1.0537 | 1.0755 | -1.0803 |
| | SYY | 0.7306 | -1.4307 | 0.6169 | -1.3009 | 0.7096 | -1.3683 |
| | SZZ | 1.1155 | -4.9293 | 0.9906 | -4.8974 | 1.0814 | -4.9862 |
| | SXY | 0.6450 | -0.2047 | 0.2628 | -0.1350 | 0.5598 | -0.1802 |
| | SXZ | 0.4577 | -0.4720 | 0.7072 | -0.7072 | 0.5360 | -0.5471 |
| | SYZ | 1.8199 | -1.8248 | 1.8222 | -1.8222 | 1.8438 | -1.8476 |
| Girder 3 | SXX | 1.4093 | -1.1021 | 0.3655 | -1.0671 | 1.1511 | -1.1072 |
| | SYY | 0.7585 | -1.4763 | 0.5982 | -1.3606 | 0.7268 | -1.4518 |
| | SZZ | 1.1503 | -5.0212 | 0.9854 | -4.9164 | 1.1020 | -5.0616 |
| | SXY | 0.6664 | -0.6070 | 0.2699 | -0.2448 | 0.5791 | -0.5261 |
| | SXZ | 0.4168 | -0.4199 | 0.6862 | -0.6862 | 0.4998 | -0.5022 |
| | SYZ | 1.8835 | -1.8846 | 1.8455 | -1.8455 | 1.8949 | -1.8966 |
| Girder 4 | SXX | 1.4037 | -1.0924 | 0.3253 | -1.0496 | 1.1506 | -1.0773 |
| | SYY | 0.7542 | -1.4406 | 0.5980 | -1.3485 | 0.7199 | -1.4323 |
| | SZZ | 1.1749 | -5.0150 | 0.9661 | -4.8258 | 1.0439 | -4.9067 |
| | SXY | 0.2306 | -0.7000 | 0.2646 | -0.2785 | 0.2360 | -0.6067 |
| | SXZ | 0.4634 | -0.4502 | 0.6548 | -0.6548 | 0.4428 | -0.4337 |
| | SYZ | 1.8597 | -1.8647 | 1.8187 | -1.8187 | 1.8492 | -1.8476 |
| Girder 5 | SXX | 0.5158 | -1.0716 | 0.3557 | -1.1442 | 0.3968 | -1.0701 |
| | SYY | 0.7974 | -1.4170 | 0.6430 | -1.4315 | 0.7579 | -1.3703 |
| | SZZ | 1.0660 | -4.8877 | 1.0494 | -5.2989 | 1.0385 | -4.9270 |
| | SXY | 0.2275 | -0.2895 | 0.3038 | -0.1878 | 0.2529 | -0.2706 |
| | SXZ | 0.4676 | -0.4546 | 0.8131 | -0.8131 | 0.4642 | -0.4548 |
| | SYZ | 1.7891 | -1.7928 | 2.0074 | -2.0074 | 1.8430 | -1.8413 |

**Table 52**
**Max. and min. stresses of AASHTO type VI girders in group A with sugarcane truck load**

| Unit: Ksi | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.5553 | -1.0881 | 0.5017 | -1.0459 | 0.4977 | -1.0915 |
| | SYY | 0.8262 | -1.4128 | 0.6626 | -1.3132 | 0.7938 | -1.4027 |
| | SZZ | 1.0891 | -4.9926 | 0.9772 | -4.8388 | 1.0588 | -5.0219 |
| | SXY | 0.3134 | -0.2258 | 0.1950 | -0.1315 | 0.2728 | -0.1789 |
| | SXZ | 0.4886 | -0.5117 | 0.7054 | -0.7054 | 0.5591 | -0.5769 |
| | SYZ | 1.8060 | -1.8113 | 1.7839 | -1.7839 | 1.8245 | -1.8286 |
| Girder 2 | SXX | 1.6312 | -1.2040 | 0.3466 | -1.0537 | 1.3322 | -1.1350 |
| | SYY | 0.7894 | -1.5114 | 0.6169 | -1.3009 | 0.7550 | -1.4266 |
| | SZZ | 1.1635 | -5.2712 | 0.9906 | -4.8974 | 1.1184 | -5.2500 |
| | SXY | 0.8026 | -0.2405 | 0.2628 | -0.1350 | 0.6814 | -0.2118 |
| | SXZ | 0.5000 | -0.5374 | 0.7072 | -0.7072 | 0.5687 | -0.5975 |
| | SYZ | 1.9273 | -1.9424 | 1.8222 | -1.8222 | 1.9267 | -1.9383 |
| Girder 3 | SXX | 1.7882 | -1.2539 | 0.3655 | -1.0671 | 1.4435 | -1.1736 |
| | SYY | 0.8472 | -1.5853 | 0.5982 | -1.3606 | 0.7952 | -1.5340 |
| | SZZ | 1.1563 | -5.4227 | 0.9854 | -4.9164 | 1.1282 | -5.3714 |
| | SXY | 0.8065 | -0.7226 | 0.2699 | -0.2448 | 0.6872 | -0.6153 |
| | SXZ | 0.4537 | -0.4643 | 0.6862 | -0.6862 | 0.5283 | -0.5364 |
| | SYZ | 2.0264 | -2.0306 | 1.8455 | -1.8455 | 2.0050 | -2.0119 |
| Girder 4 | SXX | 1.7791 | -1.3044 | 0.3253 | -1.0496 | 1.4402 | -1.1317 |
| | SYY | 0.8260 | -1.5330 | 0.5980 | -1.3485 | 0.7766 | -1.5036 |
| | SZZ | 1.1926 | -5.4170 | 0.9661 | -4.8258 | 1.1129 | -5.1582 |
| | SXY | 0.2822 | -0.8645 | 0.2646 | -0.2785 | 0.2591 | -0.7336 |
| | SXZ | 0.5296 | -0.4908 | 0.6548 | -0.6548 | 0.4537 | -0.4268 |
| | SYZ | 1.9922 | -2.0084 | 1.8187 | -1.8187 | 1.9426 | -1.9365 |
| Girder 5 | SXX | 0.6562 | -1.1416 | 0.3557 | -1.1442 | 0.5051 | -1.1141 |
| | SYY | 0.8627 | -1.4975 | 0.6430 | -1.4315 | 0.8087 | -1.4284 |
| | SZZ | 1.1494 | -5.2214 | 1.0494 | -5.2989 | 1.0848 | -5.1306 |
| | SXY | 0.2463 | -0.3660 | 0.3038 | -0.1878 | 0.2675 | -0.3297 |
| | SXZ | 0.5214 | -0.4900 | 0.8131 | -0.8131 | 0.4710 | -0.4476 |
| | SYZ | 1.8971 | -1.9063 | 2.0074 | -2.0074 | 1.9178 | -1.9124 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination “Strength I max” because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 3 and 4 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 2.42 ksi occurred at girder 3 at 54 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4. The maximum tensile stress at the bottom of the girder was 1.18 ksi occurred at girder 4 at 55 feet along the bridge, while at most other locations, there wasn’t much difference between girder 3 and 4.

For the Sugarcane truck load, girder 3 and 4 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 2.45 ksi occurred at girder 4 at 71 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4. The maximum tensile stress at the bottom of the girder was 1.19 ksi occurred at girder 4 at 52 feet along the bridge, while at most other locations, there wasn’t much difference among girder 3 and 4.

**Short term stress performances of AASHTO BT-54 girder bridges in group A.** For the bridges with AASHTO Bulb-Tee 54 girder, by comparing the stress state of bridge girders under all load combinations, the load combination “Strength I max” also leads the maximum stresses of the girder, as shown in tables 53 and 54, so we can determine that the “Strength I max” is the governing load combination, and all the analysis below are based on it.

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination “Strength I max” because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 3 and 4 were determined to be the  critical girders. The maximum compressive stress at the top of the girder was 2.82 ksi occurred at girder 4 at 50 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4. The maximum tensile stress at the bottom of the girder was 1.33 ksi occurred at girder 4 at 50 feet along the bridge, while at most other locations, there wasn’t much difference between girder 3 and 4.

For the Sugarcane truck load, girder 3, 4 and 5 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 3.01 ksi occurred at girder 4 at 50 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4, and at distance 42 feet to 49 feet, the stresses in girder 5 were governing in some points. The maximum tensile stress at the bottom of the girder was 1.42 ksi occurred at girder 5 at 50 feet along the bridge, while at most other locations, there wasn’t much difference among girder 3, 4 and 5.

**Table** 53
**Max. and min. stresses of AASHTO BT-54 girders in group A with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4891 | -0.9015 | 0.3349 | -0.7872 | 0.4075 | -0.8832 |
| | SYY | 0.7355 | -1.0850 | 0.5566 | -0.9326 | 0.6985 | -1.0577 |
| | SZZ | 1.1850 | -4.2196 | 0.9013 | -3.6982 | 1.0786 | -4.1390 |
| | SXY | 0.3201 | -0.1523 | 0.1365 | -0.1101 | 0.2788 | -0.1369 |
| | SXZ | 0.4497 | -0.4665 | 0.5283 | -0.5283 | 0.4811 | -0.4941 |
| | SYZ | 1.6900 | -1.6906 | 1.4731 | -1.4731 | 1.6279 | -1.6324 |
| Girder 2 | SXX | 1.3357 | -0.9694 | 0.3490 | -0.7943 | 1.1068 | -0.9176 |
| | SYY | 0.6825 | -1.1282 | 0.5033 | -0.9086 | 0.6456 | -1.0503 |
| | SZZ | 1.2656 | -4.4719 | 0.8864 | -3.7639 | 1.1172 | -4.3485 |
| | SXY | 0.7139 | -0.2247 | 0.2531 | -0.1161 | 0.6100 | -0.1889 |
| | SXZ | 0.4530 | -0.4717 | 0.5260 | -0.5260 | 0.4834 | -0.4978 |
| | SYZ | 1.8331 | -1.8372 | 1.5149 | -1.5149 | 1.7324 | -1.7375 |
| Girder 3 | SXX | 1.4601 | -0.9881 | 0.3705 | -0.8059 | 1.1971 | -0.9500 |
| | SYY | 0.7085 | -1.1914 | 0.4741 | -0.9600 | 0.6588 | -1.1392 |
| | SZZ | 1.2127 | -4.5974 | 0.8795 | -3.7804 | 1.1437 | -4.4492 |
| | SXY | 0.6826 | -0.6265 | 0.2446 | -0.2301 | 0.5845 | -0.5372 |
| | SXZ | 0.3887 | -0.3923 | 0.4981 | -0.4981 | 0.4275 | -0.4303 |
| | SYZ | 1.9452 | -1.9505 | 1.5324 | -1.5324 | 1.8188 | -1.8230 |
| Girder 4 | SXX | 1.4422 | -1.0360 | 0.3400 | -0.7849 | 1.1864 | -0.9071 |
| | SYY | 0.7134 | -1.1503 | 0.4809 | -0.9467 | 0.6569 | -1.1113 |
| | SZZ | 1.3276 | -4.5872 | 0.9082 | -3.6704 | 1.2390 | -4.2244 |
| | SXY | 0.2543 | -0.7508 | 0.1336 | -0.2625 | 0.2095 | -0.6411 |
| | SXZ | 0.4604 | -0.4424 | 0.4570 | -0.4570 | 0.3515 | -0.3376 |
| | SYZ | 1.8835 | -1.8884 | 1.4944 | -1.4944 | 1.7632 | -1.7665 |
| Girder 5 | SXX | 0.5611 | -0.9434 | 0.1954 | -0.8849 | 0.4462 | -0.8897 |
| | SYY | 0.7679 | -1.1468 | 0.5350 | -1.0407 | 0.7086 | -1.0558 |
| | SZZ | 1.2664 | -4.4044 | 0.7893 | -4.1645 | 1.1570 | -4.1780 |
| | SXY | 0.1687 | -0.3635 | 0.1531 | -0.1726 | 0.1634 | -0.3227 |
| | SXZ | 0.4687 | -0.4495 | 0.6334 | -0.6334 | 0.3500 | -0.3356 |
| | SYZ | 1.7704 | -1.7723 | 1.7133 | -1.7133 | 1.6966 | -1.6909 |

**Table 54**
**Max. and min. stresses of AASHTO BT-54 girders in group A with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.6077 | -0.9551 | 0.3349 | -0.7872 | 0.4990 | -0.9246 |
| | SYY | 0.8084 | -1.1355 | 0.5566 | -0.9326 | 0.7547 | -1.0967 |
| | SZZ | 1.2973 | -4.4863 | 0.9013 | -3.6982 | 1.1430 | -4.3447 |
| | SXY | 0.3907 | -0.1510 | 0.1365 | -0.1101 | 0.3333 | -0.1379 |
| | SXZ | 0.5010 | -0.5316 | 0.5283 | -0.5283 | 0.5207 | -0.5443 |
| | SYZ | 1.7659 | -1.7650 | 1.4731 | -1.4731 | 1.6978 | -1.7052 |
| Girder 2 | SXX | 1.5781 | -1.1447 | 0.3490 | -0.7943 | 1.2938 | -0.9810 |
| | SYY | 0.7412 | -1.2053 | 0.5033 | -0.9086 | 0.6908 | -1.1099 |
| | SZZ | 1.3193 | -4.8801 | 0.8864 | -3.7639 | 1.1587 | -4.6634 |
| | SXY | 0.8332 | -0.2743 | 0.2531 | -0.1161 | 0.7020 | -0.2272 |
| | SXZ | 0.5217 | -0.5681 | 0.5260 | -0.5260 | 0.5364 | -0.5721 |
| | SYZ | 1.9683 | -1.9801 | 1.5149 | -1.5149 | 1.8438 | -1.8560 |
| Girder 3 | SXX | 1.6993 | -1.1577 | 0.3705 | -0.8059 | 1.3816 | -1.0322 |
| | SYY | 0.7927 | -1.3109 | 0.4741 | -0.9600 | 0.7238 | -1.2308 |
| | SZZ | 1.1400 | -5.1056 | 0.8795 | -3.7804 | 1.0573 | -4.8412 |
| | SXY | 0.7792 | -0.7079 | 0.2446 | -0.2301 | 0.6591 | -0.5999 |
| | SXZ | 0.4401 | -0.4537 | 0.4981 | -0.4981 | 0.4672 | -0.4776 |
| | SYZ | 2.1531 | -2.1775 | 1.5324 | -1.5324 | 1.9788 | -1.9983 |
| Girder 4 | SXX | 1.6824 | -1.2099 | 0.3400 | -0.7849 | 1.3717 | -1.0114 |
| | SYY | 0.7847 | -1.2442 | 0.4809 | -0.9467 | 0.7110 | -1.1838 |
| | SZZ | 1.3721 | -5.0740 | 0.9082 | -3.6704 | 1.2733 | -4.4950 |
| | SXY | 0.3143 | -0.8690 | 0.1336 | -0.2625 | 0.2532 | -0.7323 |
| | SXZ | 0.5623 | -0.5116 | 0.4570 | -0.4570 | 0.4302 | -0.3910 |
| | SYZ | 2.0532 | -2.0697 | 1.4944 | -1.4944 | 1.8929 | -1.9049 |
| Girder 5 | SXX | 0.7080 | -1.0176 | 0.1954 | -0.8849 | 0.5595 | -0.9280 |
| | SYY | 0.8432 | -1.2223 | 0.5350 | -1.0407 | 0.7633 | -1.1083 |
| | SZZ | 1.4149 | -4.7683 | 0.7893 | -4.1645 | 1.2716 | -4.3533 |
| | SXY | 0.1723 | -0.4483 | 0.1531 | -0.1726 | 0.1662 | -0.3881 |
| | SXZ | 0.5507 | -0.5086 | 0.6334 | -0.6334 | 0.3638 | -0.3313 |
| | SYZ | 1.8812 | -1.8830 | 1.7133 | -1.7133 | 1.7704 | -1.7591 |

**Short term stress performances of AASHTO BT-63 girder bridges in group A.** For the bridges with AASHTO Bulb-Tee 63 girder, by comparing the stress state of bridge girders under all load combinations, the load combination “Strength I max” also lead the maximum stresses of the girder, as shown in tables 55 and 56, so we can determine that the “Strength I max” is the governing load combination, and all the analysis below are based on it.

**Table 55**
**Max. and min. stresses of AASHTO BT-63 girders in group A with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4285 | -0.7862 | 0.3154 | -0.7339 | 0.3554 | -0.7834 |
| | SYY | 0.4435 | -0.9511 | 0.3576 | -0.8670 | 0.4240 | -0.9407 |
| | SZZ | 0.9964 | -3.6739 | 0.7940 | -3.4503 | 0.8923 | -3.6673 |
| | SXY | 0.2874 | -0.1363 | 0.1285 | -0.1040 | 0.2519 | -0.1210 |
| | SXZ | 0.4050 | -0.4188 | 0.5386 | -0.5386 | 0.4510 | -0.4617 |
| | SYZ | 1.5273 | -1.5279 | 1.3858 | -1.3858 | 1.4600 | -1.4605 |
| Girder 2 | SXX | 1.1551 | -0.8477 | 0.2951 | -0.7441 | 0.9546 | -0.8221 |
| | SYY | 0.4508 | -0.9990 | 0.3701 | -0.8550 | 0.4342 | -0.9491 |
| | SZZ | 1.0761 | -3.9374 | 0.7848 | -3.5219 | 0.9388 | -3.8868 |
| | SXY | 0.6271 | -0.1915 | 0.2350 | -0.1095 | 0.5391 | -0.1599 |
| | SXZ | 0.4142 | -0.4309 | 0.5391 | -0.5391 | 0.4585 | -0.4714 |
| | SYZ | 1.6743 | -1.6791 | 1.4300 | -1.4300 | 1.5837 | -1.5874 |
| Girder 3 | SXX | 1.2610 | -0.8728 | 0.3186 | -0.7565 | 1.0296 | -0.8553 |
| | SYY | 0.4861 | -1.0708 | 0.3858 | -0.9088 | 0.4652 | -1.0390 |
| | SZZ | 1.0193 | -4.0680 | 0.7874 | -3.5412 | 0.9748 | -3.9921 |
| | SXY | 0.5976 | -0.5478 | 0.2292 | -0.2118 | 0.5159 | -0.4724 |
| | SXZ | 0.3542 | -0.3578 | 0.5136 | -0.5136 | 0.4065 | -0.4092 |
| | SYZ | 1.7923 | -1.7981 | 1.4487 | -1.4487 | 1.6821 | -1.6868 |
| Girder 4 | SXX | 1.2460 | -0.9039 | 0.2860 | -0.7348 | 1.0221 | -0.8065 |
| | SYY | 0.4647 | -1.0218 | 0.3773 | -0.8939 | 0.4460 | -1.0016 |
| | SZZ | 1.1321 | -4.0441 | 0.8152 | -3.4283 | 1.0681 | -3.7424 |
| | SXY | 0.2159 | -0.6573 | 0.1323 | -0.2466 | 0.1829 | -0.5657 |
| | SXZ | 0.4246 | -0.4081 | 0.4742 | -0.4742 | 0.3167 | -0.3050 |
| | SYZ | 1.7232 | -1.7286 | 1.4092 | -1.4092 | 1.6182 | -1.6214 |
| Girder 5 | SXX | 0.4882 | -0.8233 | 0.1935 | -0.8514 | 0.3823 | -0.7920 |
| | SYY | 0.4666 | -1.0034 | 0.4174 | -1.0086 | 0.4282 | -0.9491 |
| | SZZ | 1.0646 | -3.8393 | 0.6704 | -3.9994 | 0.9745 | -3.7098 |
| | SXY | 0.1493 | -0.3250 | 0.1407 | -0.1720 | 0.1462 | -0.2934 |
| | SXZ | 0.4259 | -0.4089 | 0.6760 | -0.6760 | 0.3389 | -0.3261 |
| | SYZ | 1.5990 | -1.6009 | 1.6628 | -1.6628 | 1.5225 | -1.5199 |

**Table 56**
**Max. and min. stresses of AASHTO BT-63 girders in group A with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.5287 | -0.8304 | 0.3154 | -0.7339 | 0.4328 | -0.8174 |
| | SYY | 0.4942 | -0.9936 | 0.3576 | -0.8670 | 0.4630 | -0.9735 |
| | SZZ | 1.0891 | -3.8916 | 0.7940 | -3.4503 | 0.9483 | -3.8352 |
| | SXY | 0.3493 | -0.1358 | 0.1285 | -0.1040 | 0.2997 | -0.1249 |
| | SXZ | 0.4470 | -0.4706 | 0.5386 | -0.5386 | 0.4834 | -0.5016 |
| | SYZ | 1.5898 | -1.5892 | 1.3858 | -1.3858 | 1.5132 | -1.5209 |
| Girder 2 | SXX | 1.3685 | -1.0038 | 0.2951 | -0.7441 | 1.1192 | -0.8776 |
| | SYY | 0.4837 | -1.0605 | 0.3701 | -0.8550 | 0.4605 | -1.0021 |
| | SZZ | 1.1311 | -4.2918 | 0.7848 | -3.5219 | 0.9812 | -4.1602 |
| | SXY | 0.7350 | -0.2336 | 0.2350 | -0.1095 | 0.6223 | -0.1923 |
| | SXZ | 0.4748 | -0.5149 | 0.5391 | -0.5391 | 0.5052 | -0.5362 |
| | SYZ | 1.7931 | -1.8061 | 1.4300 | -1.4300 | 1.6753 | -1.6854 |
| Girder 3 | SXX | 1.4750 | -1.0056 | 0.3186 | -0.7565 | 1.1947 | -0.9291 |
| | SYY | 0.5364 | -1.1771 | 0.3858 | -0.9088 | 0.5047 | -1.1221 |
| | SZZ | 0.9670 | -4.5232 | 0.7874 | -3.5412 | 0.8979 | -4.3433 |
| | SXY | 0.6760 | -0.6092 | 0.2292 | -0.2118 | 0.5764 | -0.5198 |
| | SXZ | 0.4013 | -0.4145 | 0.5136 | -0.5136 | 0.4428 | -0.4530 |
| | SYZ | 1.9850 | -2.0107 | 1.4487 | -1.4487 | 1.8309 | -1.8518 |
| Girder 4 | SXX | 1.4615 | -1.0609 | 0.2860 | -0.7348 | 1.1884 | -0.8924 |
| | SYY | 0.5077 | -1.1002 | 0.3773 | -0.8939 | 0.4748 | -1.0620 |
| | SZZ | 1.1735 | -4.4745 | 0.8152 | -3.4283 | 1.1000 | -3.9676 |
| | SXY | 0.2687 | -0.7654 | 0.1323 | -0.2466 | 0.2178 | -0.6491 |
| | SXZ | 0.5174 | -0.4714 | 0.4742 | -0.4742 | 0.3801 | -0.3445 |
| | SYZ | 1.8754 | -1.8926 | 1.4092 | -1.4092 | 1.7335 | -1.7447 |
| Girder 5 | SXX | 0.6171 | -0.8856 | 0.1935 | -0.8514 | 0.4817 | -0.8217 |
| | SYY | 0.5178 | -1.0669 | 0.4174 | -1.0086 | 0.4658 | -0.9883 |
| | SZZ | 1.1929 | -4.1430 | 0.6704 | -3.9994 | 1.0735 | -3.8453 |
| | SXY | 0.1524 | -0.4023 | 0.1407 | -0.1720 | 0.1486 | -0.3530 |
| | SXZ | 0.4943 | -0.4595 | 0.6760 | -0.6760 | 0.3272 | -0.3007 |
| | SYZ | 1.6917 | -1.6935 | 1.6628 | -1.6628 | 1.5887 | -1.5895 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination "Strength I max" because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 3 and 4 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 2.37 ksi occurred at girder 3 at 50 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4. The maximum tensile stress at the bottom of the girder was 1.13 ksi occurred at girder 4 at 50 feet along the bridge, while at most other locations, there wasn't much difference between girder 3 and 4.

For the Sugarcane truck load, girder 3, 4 and 5 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 2.53 ksi occurred at girder 4 at 50 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4, and at distance 45 feet, the stresses in girder 5 were governing. The maximum tensile stress at the bottom of the girder was 1.19 ksi occurred at girder 5 at 49 feet along the bridge, while at most other locations, there wasn't much difference among girder 3, 4 and 5.

**Short term stress performances of AASHTO BT-72 girder bridges in group A.** For the bridges with AASHTO Bulb-Tee 72 girder, by comparing the stress state of bridge girders under all load combinations, the load combination "Strength I max" also lead the maximum stresses of the girder, as shown in tables 57 and 58, so we can determine that the "Strength I max" is the governing load combination, and all the analysis below are based on it.

**Table 57**
**Max. and min. stresses of AASHTO BT-72 girders in group A with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.3751 | -0.7001 | 0.3021 | -0.6996 | 0.3092 | -0.7103 |
| | SYY | 0.3733 | -0.8525 | 0.3375 | -0.8252 | 0.3683 | -0.8565 |
| | SZZ | 0.8571 | -3.2657 | 0.7221 | -3.2902 | 0.7592 | -3.3217 |
| | SXY | 0.2589 | -0.1246 | 0.1217 | -0.0999 | 0.2285 | -0.1116 |
| | SXZ | 0.3719 | -0.3831 | 0.5546 | -0.5546 | 0.4310 | -0.4397 |
| | SYZ | 1.4081 | -1.4087 | 1.3327 | -1.3327 | 1.3545 | -1.3550 |
| Girder 2 | SXX | 1.0083 | -0.7511 | 0.2478 | -0.7123 | 0.8300 | -0.7525 |
| | SYY | 0.4098 | -0.9784 | 0.3518 | -0.8229 | 0.3997 | -0.9374 |
| | SZZ | 0.9379 | -3.5378 | 0.7178 | -3.3669 | 0.8070 | -3.5490 |
| | SXY | 0.5571 | -0.1642 | 0.2209 | -0.1050 | 0.4821 | -0.1360 |
| | SXZ | 0.3863 | -0.4012 | 0.5578 | -0.5578 | 0.4432 | -0.4547 |
| | SYZ | 1.5600 | -1.5650 | 1.3790 | -1.3790 | 1.4818 | -1.4857 |
| Girder 3 | SXX | 1.1005 | -0.7917 | 0.2735 | -0.7256 | 0.8936 | -0.7869 |
| | SYY | 0.4464 | -1.0389 | 0.3685 | -0.8792 | 0.4316 | -0.9942 |
| | SZZ | 0.8842 | -3.6756 | 0.7275 | -3.3888 | 0.8557 | -3.6605 |
| | SXY | 0.5307 | -0.4851 | 0.2182 | -0.1976 | 0.4621 | -0.4211 |
| | SXZ | 0.3298 | -0.3334 | 0.5340 | -0.5340 | 0.3943 | -0.3971 |
| | SYZ | 1.6829 | -1.6890 | 1.3987 | -1.3987 | 1.5851 | -1.5902 |
| Girder 4 | SXX | 1.0878 | -0.7966 | 0.2386 | -0.7030 | 0.8887 | -0.7327 |
| | SYY | 0.4235 | -0.9876 | 0.3595 | -0.8627 | 0.4093 | -0.9365 |
| | SZZ | 0.9889 | -3.6379 | 0.7538 | -3.2727 | 0.9448 | -3.3877 |
| | SXY | 0.1849 | -0.5825 | 0.1328 | -0.2348 | 0.1617 | -0.5057 |
| | SXZ | 0.3985 | -0.3833 | 0.4960 | -0.4960 | 0.3044 | -0.2938 |
| | SYZ | 1.6060 | -1.6117 | 1.3575 | -1.3575 | 1.5129 | -1.5159 |
| Girder 5 | SXX | 0.4259 | -0.7331 | 0.1954 | -0.8355 | 0.3267 | -0.7215 |
| | SYY | 0.3921 | -0.8977 | 0.4057 | -0.9966 | 0.3907 | -0.8728 |
| | SZZ | 0.9151 | -3.4149 | 0.5840 | -3.9183 | 0.8395 | -3.3714 |
| | SXY | 0.1351 | -0.2923 | 0.1718 | -0.1729 | 0.1337 | -0.2689 |
| | SXZ | 0.3934 | -0.3785 | 0.7222 | -0.7222 | 0.3356 | -0.3244 |
| | SYZ | 1.4725 | -1.4743 | 1.6442 | -1.6442 | 1.4079 | -1.4083 |

**Table 58**
**Max. and min. stresses of AASHTO BT-72 girders in group A with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4599 | -0.7372 | 0.3021 | -0.6996 | 0.3746 | -0.7389 |
| | SYY | 0.3899 | -0.8892 | 0.3375 | -0.8252 | 0.3811 | -0.8848 |
| | SZZ | 0.9340 | -3.4470 | 0.7221 | -3.2902 | 0.8023 | -3.4615 |
| | SXY | 0.3131 | -0.1247 | 0.1217 | -0.0999 | 0.2703 | -0.1157 |
| | SXZ | 0.4071 | -0.4253 | 0.5546 | -0.5546 | 0.4582 | -0.4723 |
| | SYZ | 1.4608 | -1.4603 | 1.3327 | -1.3327 | 1.3952 | -1.3948 |
| Girder 2 | SXX | 1.1951 | -0.8862 | 0.2478 | -0.7123 | 0.9741 | -0.8019 |
| | SYY | 0.4405 | -1.0712 | 0.3518 | -0.8229 | 0.4234 | -1.0090 |
| | SZZ | 0.9922 | -3.8515 | 0.7178 | -3.3669 | 0.8489 | -3.7910 |
| | SXY | 0.6541 | -0.1999 | 0.2209 | -0.1050 | 0.5570 | -0.1635 |
| | SXZ | 0.4409 | -0.4761 | 0.5578 | -0.5578 | 0.4854 | -0.5125 |
| | SYZ | 1.6662 | -1.6791 | 1.3790 | -1.3790 | 1.5638 | -1.5737 |
| Girder 3 | SXX | 1.2904 | -0.8897 | 0.2735 | -0.7256 | 1.0400 | -0.8543 |
| | SYY | 0.4933 | -1.2353 | 0.3685 | -0.8792 | 0.4680 | -1.1457 |
| | SZZ | 0.8446 | -4.0905 | 0.7275 | -3.3888 | 0.7867 | -3.9806 |
| | SXY | 0.5941 | -0.5312 | 0.2182 | -0.1976 | 0.5110 | -0.4566 |
| | SXZ | 0.3738 | -0.3867 | 0.5340 | -0.5340 | 0.4283 | -0.4382 |
| | SYZ | 1.8643 | -1.8925 | 1.3987 | -1.3987 | 1.7255 | -1.7475 |
| Girder 4 | SXX | 1.2795 | -0.9371 | 0.2386 | -0.7030 | 1.0366 | -0.7936 |
| | SYY | 0.4625 | -1.1171 | 0.3595 | -0.8627 | 0.4342 | -1.0281 |
| | SZZ | 1.0280 | -4.0254 | 0.7538 | -3.2727 | 0.9749 | -3.5785 |
| | SXY | 0.2313 | -0.6805 | 0.1328 | -0.2348 | 0.1892 | -0.5813 |
| | SXZ | 0.4845 | -0.4422 | 0.4960 | -0.4960 | 0.3409 | -0.3083 |
| | SYZ | 1.7447 | -1.7624 | 1.3575 | -1.3575 | 1.6171 | -1.6277 |
| Girder 5 | SXX | 0.5389 | -0.7862 | 0.1954 | -0.8355 | 0.4139 | -0.7449 |
| | SYY | 0.4167 | -0.9525 | 0.4057 | -0.9966 | 0.4053 | -0.9037 |
| | SZZ | 1.0259 | -3.6723 | 0.5840 | -3.9183 | 0.9250 | -3.4783 |
| | SXY | 0.1379 | -0.3624 | 0.1718 | -0.1729 | 0.1358 | -0.3229 |
| | SXZ | 0.4513 | -0.4226 | 0.7222 | -0.7222 | 0.3247 | -0.3027 |
| | SYZ | 1.5517 | -1.5533 | 1.6442 | -1.6442 | 1.4665 | -1.4665 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination "Strength I max" because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 3 and 4 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 2.04 ksi occurred at girder 3 at 50 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4. The maximum tensile stress at the bottom of the girder was 0.99 ksi occurred at girder 4 at 50 feet along the bridge, while at most other locations, there wasn't much difference between girder 3 and 4

For the Sugarcane truck load, girder 3, 4 and 5 were determined to be the critical girders. The maximum compressive stress at the top of the girder was 2.18 ksi occurred at girder 4 at 50 feet along the bridge, while at most other locations, the compressive stresses in girder 3 were slightly larger than those in girder 4, and at distance 45 feet, the stresses in girder 5 were governing. The maximum tensile stress at the bottom of the girder was 1.03 ksi occurred at girder 5 at 50 feet along the bridge, while at most other locations, there wasn't much difference among girder 3, 4 and 5.

**Short term deflection of AASHTO type IV girder bridges in group A.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated. The phrase "short term deflection" referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 3 was the critical girder, which had 2.31 inch maximum displacement with truck load HS20-44, and 2.47 inch maximum displacement with truck load sugarcane. Note that in this parametric study camber effects were not considered.

**Short term deflection of AASHTO type V girder bridges in group A.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase "short term deflection" referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 3 was the critical girder, which had 1.76 inch maximum displacement with truck load HS20-44, and 1.87 inch maximum displacement with truck load sugarcane.

**Short term deflection of AASHTO type VI girder bridges in group A.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase "short term deflection" referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 3 was the critical girder, which had 2.00 inch maximum displacement with truck load HS20-44, and 2.13 inch maximum displacement with truck load sugarcane.

**Short term deflection of AASHTO BT-54 girder bridges in group A.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 3 was the critical girder, which had 2.02 inch maximum displacement with truck load HS20-44, and 2.17 inch maximum displacement with truck load sugarcane.

**Short term deflection of AASHTO BT-63 girder bridges in group A.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 3 was the critical girder, which had 1.47 inch maximum displacement with truck load HS20-44, and 1.57 inch maximum displacement with Truck load sugarcane. Figures 35 and 36 show the deflection along the bridge with these truck loads.

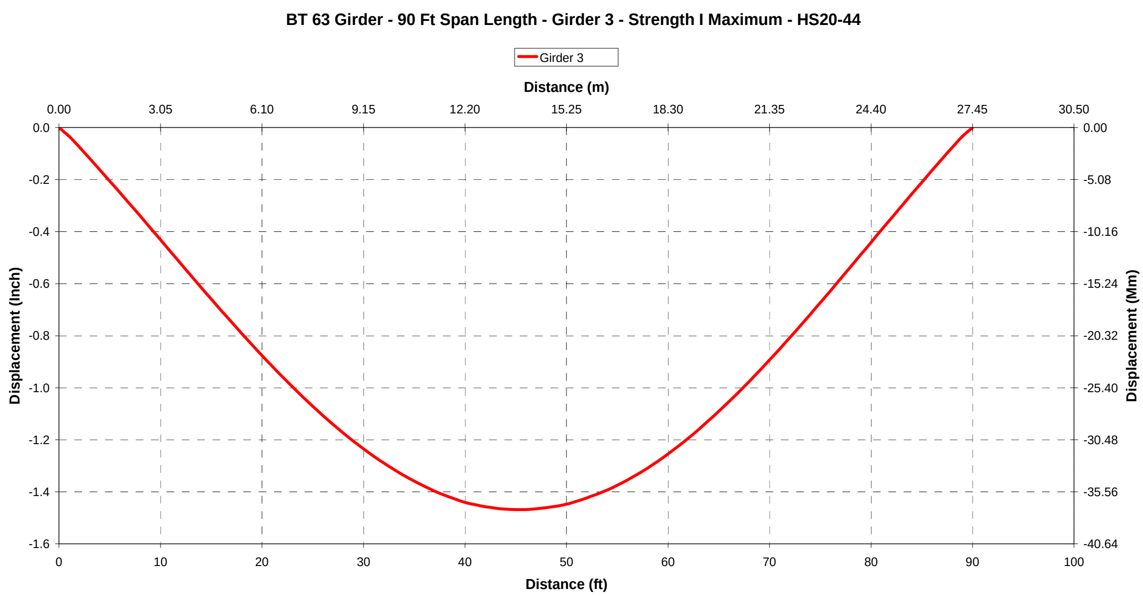



**Figure 35**
**Displacement in Girder 3 along the bridge - AASHTO BT-63 girder in group A - truck load HS20-44**

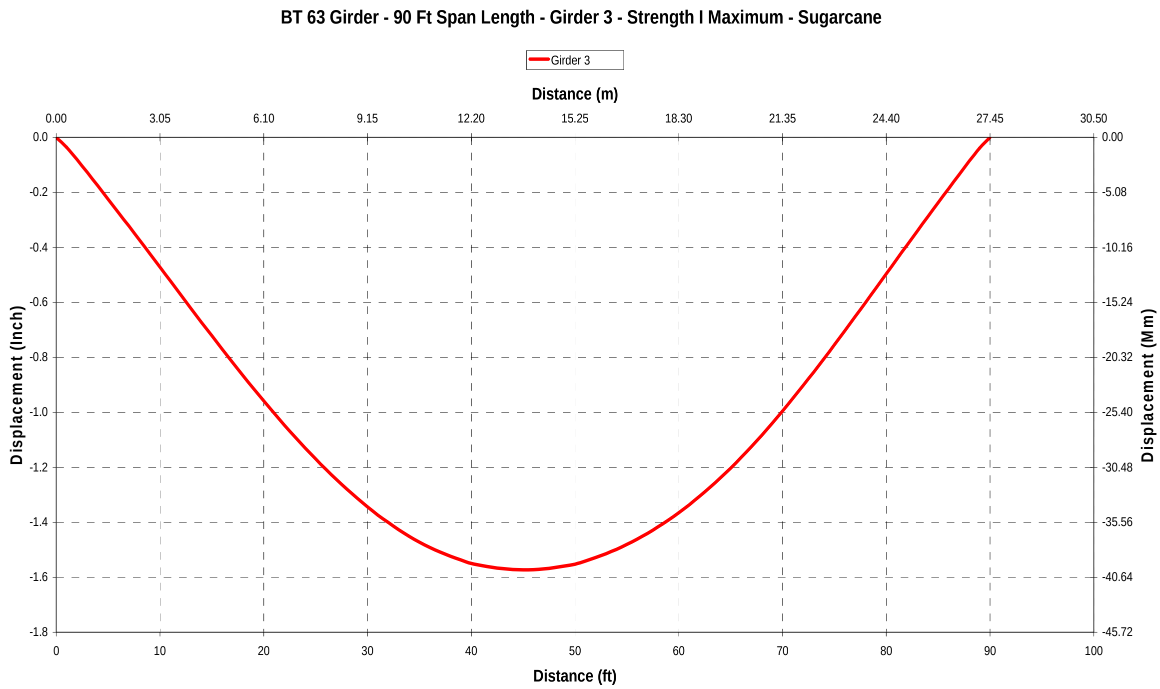



**Figure 36**
**Displacement in Girder 3 along the bridge - AASHTO BT-63 girder in group A - Truck load sugarcane**

**Short term deflection of AASHTO BT-72 girder bridges in group A.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 3 was the critical girder, which had 1.12 inch maximum displacement with truck load HS20-44, and 1.20 inch maximum displacement with Truck load sugarcane. Figure 37 and Figure 38 showed the deflection along the bridge with these truck loads.

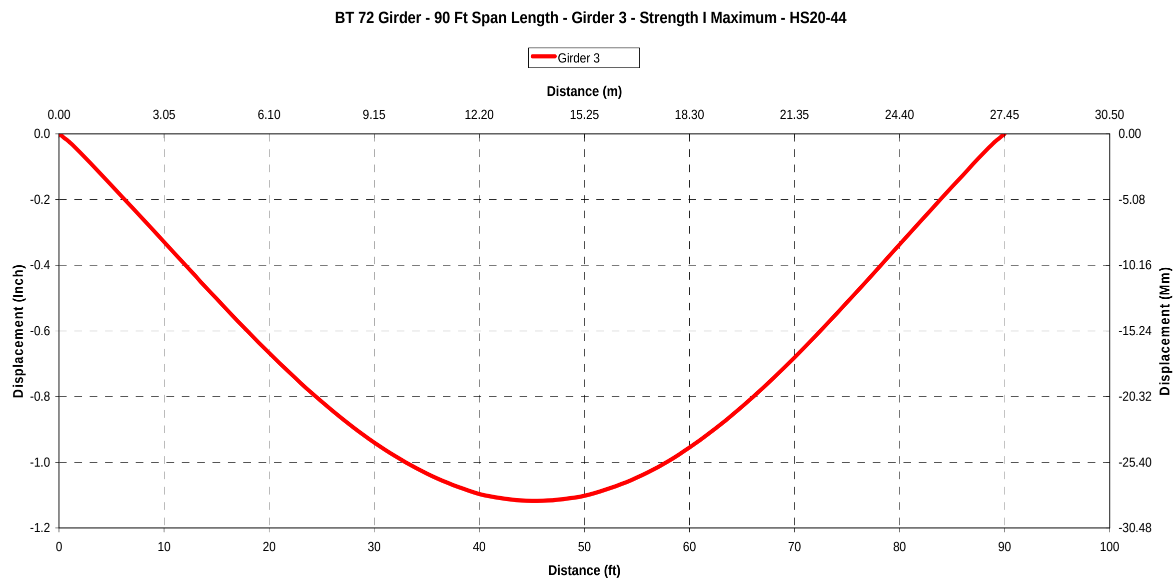



**Figure 37**
**Displacement in Girder 3 along the bridge - AASHTO BT-72 girder in group A - truck load HS20-44**

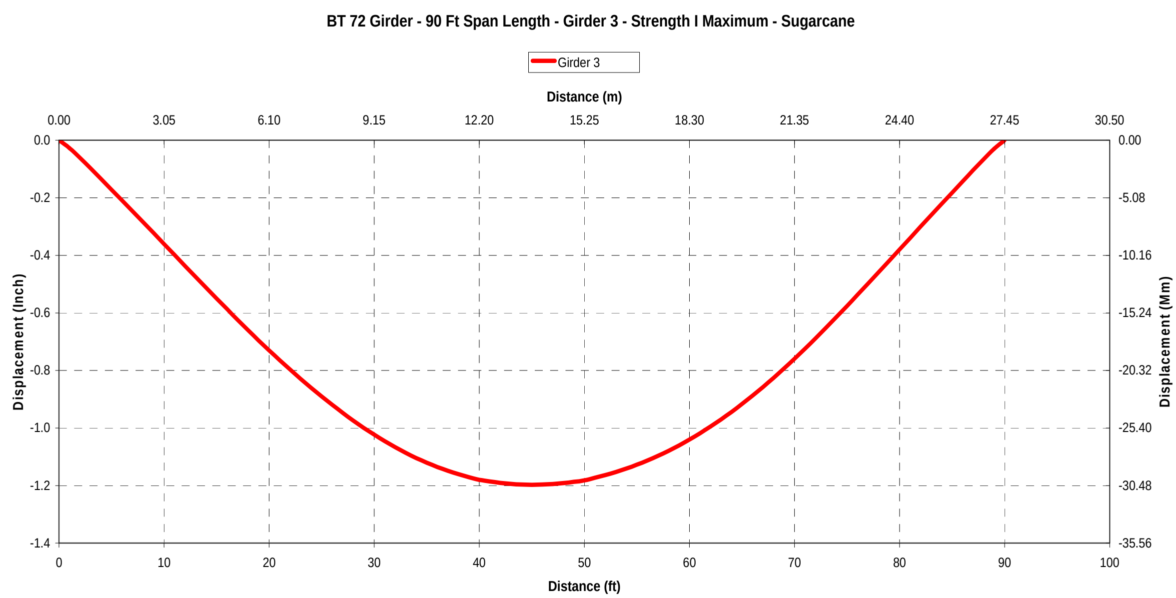



**Figure 38**
**Displacement in Girder 3 along the bridge - AASHTO BT-72 girder in group A - Truck load sugarcane**

**Long term stress performances of AASHTO type IV girder bridges in group A.** The long term effects of heavy trucks on bridges and bridge decks play an important role in the bridge life evaluation. The load combination "Fatigue" was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 3, 4 and 5. The maximum compressive stress was 0.41 ksi occurred at girder 3, at distance 39 feet, while in the distance range from 42 to 60 feet, girder 4 and 5 were the critical girders. The maximum tensile stress was 0.165 ksi occurred at girder 3, at distance 39 feet, while in the distance range from 40 to 61 feet, girder 4 and 5 were the critical girders.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.58 ksi was occurred at distance 39 feet, while the maximum tensile stress 0.25 ksi was occurred at distance 49 feet.

**Long term stress performances of AASHTO type V girder bridges in group A.** Similar to the analysis of AASHTO type IV girder, the load combination "Fatigue" was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 3 and 4. The maximum compressive stress, 0.27 ksi, was occurred at girder 3, at distance 45 feet, while in the distance range from 48 to 64 feet, girder 4 was the critical girders. The maximum tensile stress, 0.12 ksi, was occurred at girder 4, at distance 58 feet, while in the distance range from 70 to 87 feet, girder 3 was the critical girder.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.37 ksi was occurred at distance 59 feet, while the maximum tensile stress 0.19 ksi was occurred at distance 60 feet.

**Long term stress performances of AASHTO type VI girder bridges in group A.** Similar to the analysis of AASHTO type IV and V girder, the load combination "Fatigue" was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 3 and 4. The maximum compressive stress, 0.27 ksi, was occurred at girder 3, at distance 51 feet, while in the distance range from 53 to 73 feet, girder 4 was the critical girder. The maximum tensile stress, 0.13 ksi, was occurred at girder 4, at distance 70 feet, while in the distance range from 73 to 84 feet, girder 3 was the critical girder.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.37 ksi was occurred at distance 71 feet, while the maximum tensile stress 0.22 ksi was occurred at distance 70 feet.

**Long term stress performances of AASHTO BT-54 girder bridges in group A.** The long term effects of heavy trucks on bridges and bridge decks play an important role in the bridge life evaluation. The load combination “Fatigue” was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 3, 4 and 5. The maximum compressive stress, 0.35 ksi, was occurred at girder 3, at distance 39 feet, while in the distance range from 45 to 59 feet, girder 4 was the critical girders. The maximum tensile stress, 0.16 ksi, was occurred at girder 4, at distance 40 feet, while in the distance range from 41 to 62 feet, girder 4 and 5 were the critical girders.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.50 ksi was occurred at distance 50 feet, while the maximum tensile stress 0.26 ksi was occurred at distance 50 feet.

**Long term stress performances of AASHTO BT-63 girder bridges in group A.** Similar to the analysis of AASHTO type IV and Bulb-Tee 54 girders, the load combination “Fatigue” was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 3, 4 and 5. The maximum compressive stress, 0.31 ksi, was occurred at girder 3, at distance 39 feet, while in the distance range from 49 to 52 feet, girder 4 was the critical girders. The maximum tensile stress, 0.14 ksi, was occurred at girder 4, at distance 40 feet, while in the distance range from 41 to 62 feet, girder 4 and 5 were the critical girders.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.43 ksi was occurred at distance 50 feet, girder 4, while the maximum tensile stress 0.22 ksi was occurred at distance 50 feet, girder 5.

**Long term stress performances of AASHTO BT-72 girder bridges in group A.** Similar to the analysis of AASHTO type IV and other Bulb-Tee type girders, the load combination “Fatigue” was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 3, 4 and 5. The maximum compressive stress, 0.28 ksi, was occurred at girder 3, at distance 39 feet, while in the distance range from 49 to 52 feet, girder 4 was the critical girders. The maximum tensile stress, 0.12 ksi, was occurred at girder 4, at distance 40 feet, while in the distance range from 41 to 62 feet, girder 4 and 5 were the critical girders.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.37 ksi was occurred at distance 50 feet, girder 4, while the maximum tensile stress 0.19 ksi was occurred at distance 40 feet, girder 4.

**Long term deflection of AASHTO type IV girder bridges in group A.** The phrase "long term deflection" referred to the deflection calculated with load combination "Fatigue". By the same method used in evaluating the short term deflection, girder 3 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.22 inch and 0.36 inch under HS20-44 and Sugarcane truck, respectively, as shown in Figure 39 and Figure 40.

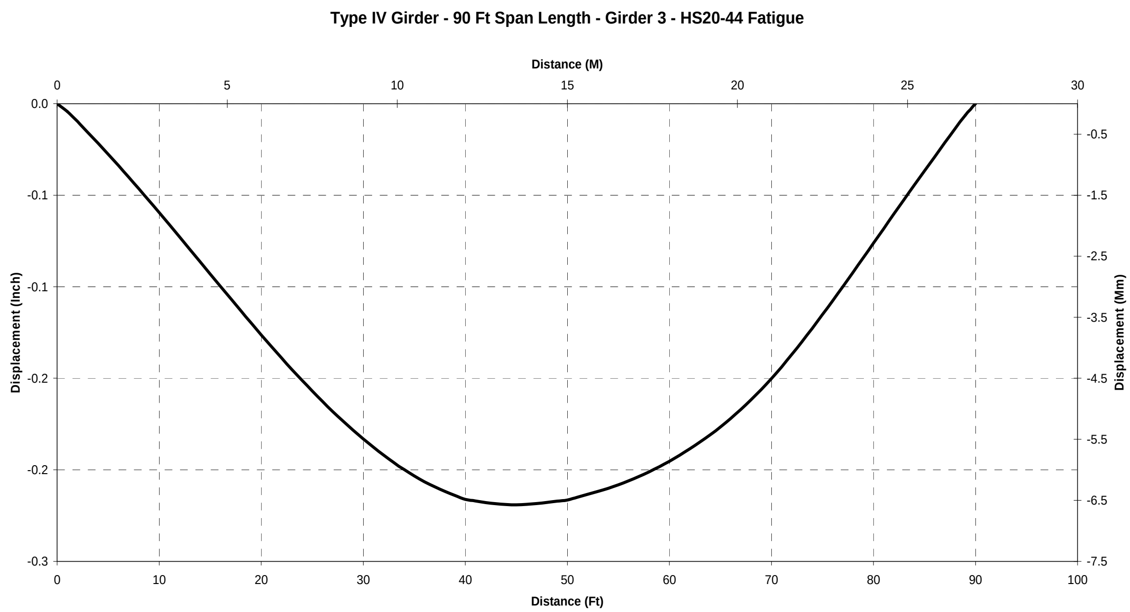



**Figure 39**
**Displacement in Girder 3 along the bridge - AASHTO type IV girder in group A - truck load HS20-44 fatigue**

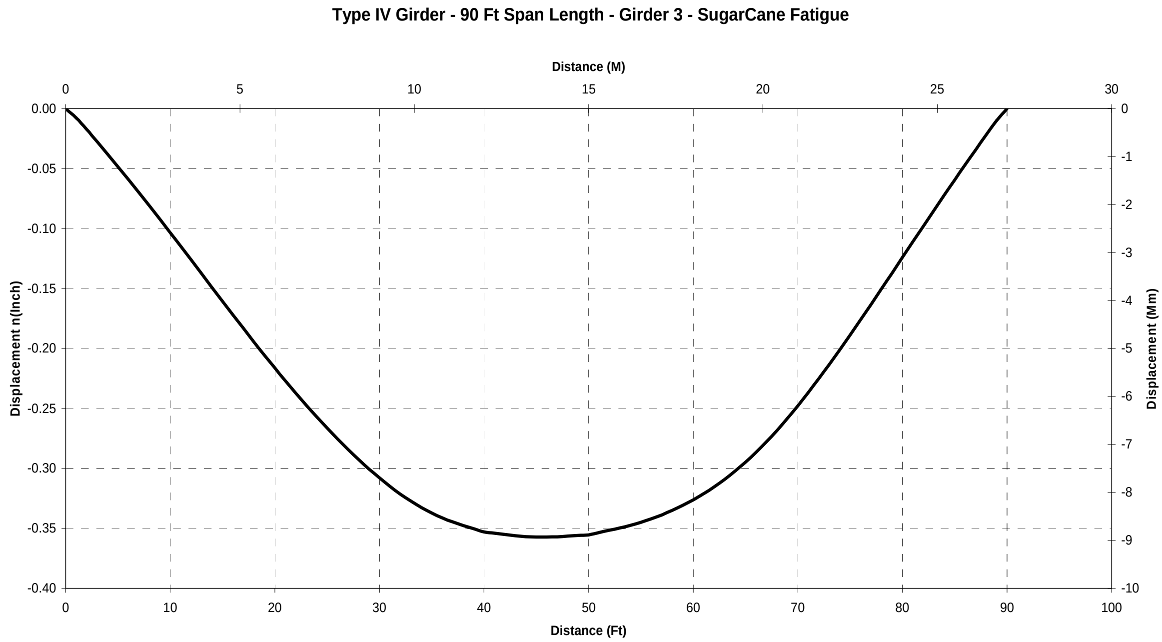



**Figure 40**
**Displacement in Girder 3 along the bridge - AASHTO type IV girder in group A - truck load sugarcane fatigue**

**Long term deflection of AASHTO type V girder bridges in group A.** By the same method used in evaluating the long term deflection of AASHTO type IV girder, girder 3 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.14 inch and 0.23 inch under HS20-44 and Sugarcane truck, respectively.

**Long term deflection of AASHTO type VI girder bridges in group A.** By the same method used in evaluating the long term deflection of AASHTO type IV and V girders, girder 3 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.15 inch and 0.24 inch under HS20-44 and Sugarcane truck, respectively.

**Long term deflection of AASHTO Bulb-Tee 54 girder bridges in group A.** The phrase "long term deflection" referred to  the deflection calculated with load combination "Fatigue". By the same method used in evaluating the short term deflection, girder 3 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.14 inch and 0.23 inch under HS20-44 and Sugarcane truck, respectively, as shown in Figure 41 and Figure 42.

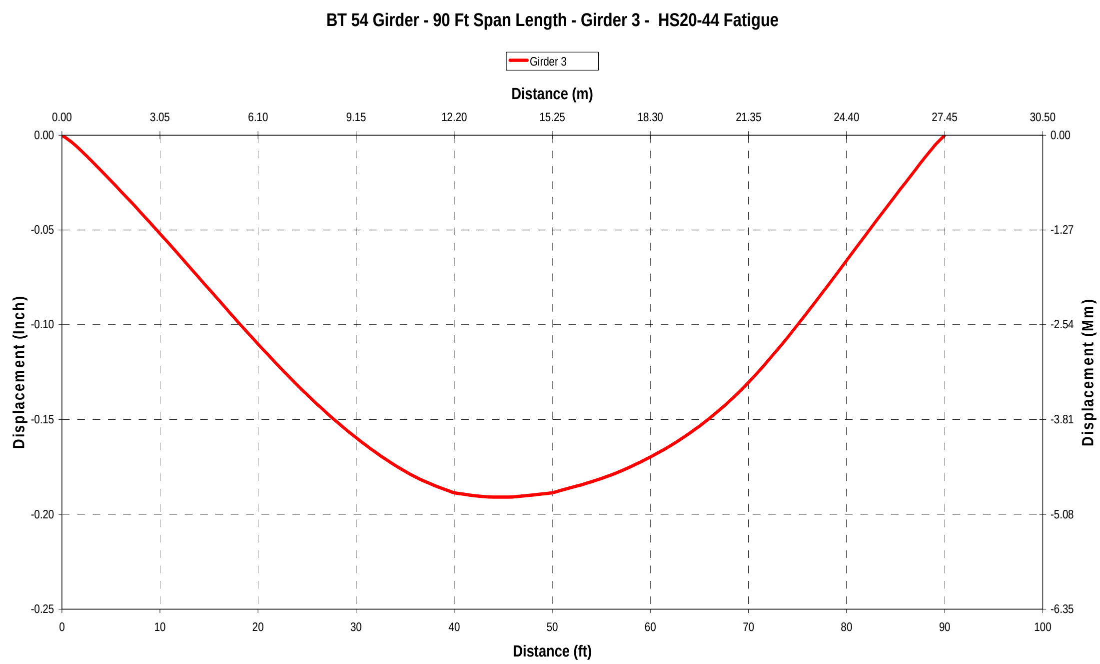



**Figure 41**
**Displacement in Girder 3 along the bridge - AASHTO BT-54 girder in group A - truck load HS20-44 fatigue**

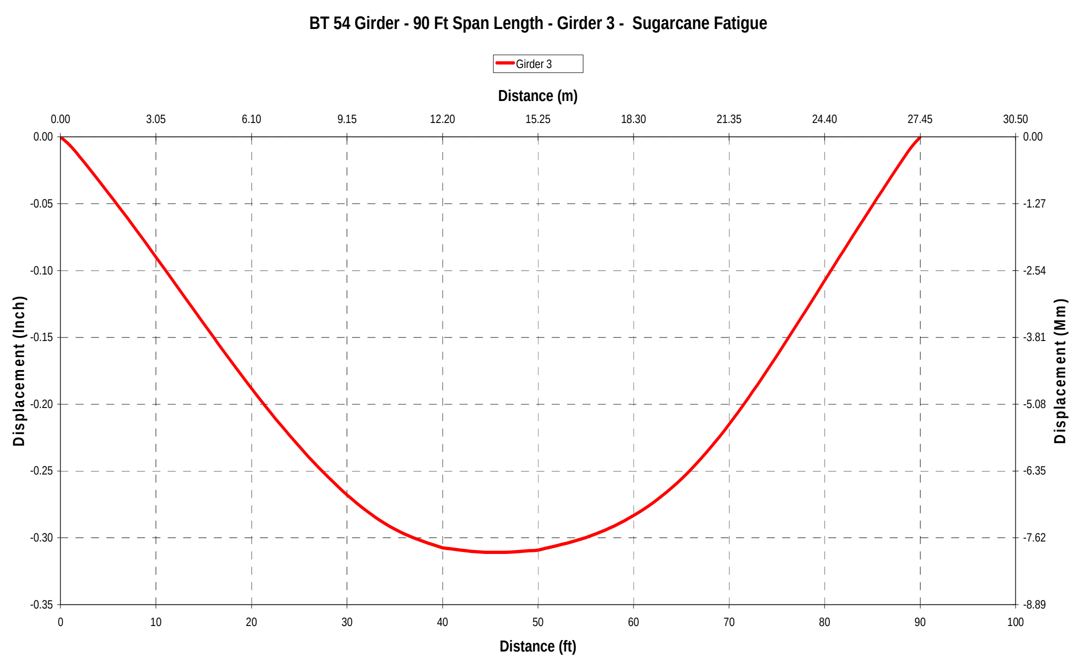



**Figure 42**
**Displacement in Girder 3 along the bridge - AASHTO BT-54 girder in group A - truck load sugarcane fatigue**

**Long term deflection of AASHTO Bulb-Tee 63 girder bridges in group A.** By the same method used in evaluating the long term deflection of AASHTO I-Beams and Bulb-Tee 54 girders, girder 3 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.14 inch and 0.23 inch under HS20-44 and Sugarcane truck, respectively.

**Long term deflection of AASHTO Bulb-Tee 72 girder bridges in group A.** By the same method used in evaluating the long term deflection of AASHTO type IV and Bulb-Tee girders, girder 3 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.11 inch and 0.17 inch under HS20-44 and Sugarcane truck, respectively.

**Short term stress performances of AASHTO type IV girder bridges in group B.** In this study, bridge models in group B contained seven girders with the shorter girder spacing. The span lengths of models in group B were determined by the maximum allowable girder displacement; for AASHTO type IV girder's model, the span length was identified as 99 ft; and as the same as group A, four load combinations were used in the analysis. The load combinations Strength I max, Strength III max, and Strength V max were used to evaluate the short term performances of the bridge girders. By comparing the stress state of bridge girders under these load combinations, the load combination "Strength I max" lead the maximum stresses of the girder, as shown in tables 59 and 60, so we can determine that the "Strength I max" is the governing load combination, and all the analysis below are based on it.

**Table 59**
**Max. and min. stresses of AASHTO type IV girders in group B with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| RESULT | (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4279 | -0.9462 | 0.4273 | -0.8934 | 0.4333 | -0.9424 |
| | SYY | 0.6709 | -1.1873 | 0.5533 | -1.1048 | 0.6487 | -1.1768 |
| | SZZ | 1.1055 | -4.3838 | 1.0096 | -4.1508 | 1.0930 | -4.3708 |
| | SXY | 0.1726 | -0.2285 | 0.1376 | -0.1565 | 0.1644 | -0.2016 |
| | SXZ | 0.4382 | -0.4509 | 0.5502 | -0.5502 | 0.4773 | -0.4870 |
| | SYZ | 1.8429 | -1.8458 | 1.6940 | -1.6940 | 1.7598 | -1.7580 |
| Girder 2 | SXX | 0.8075 | -1.1205 | 0.2541 | -0.8827 | 0.6777 | -0.9832 |
| | SYY | 0.5925 | -1.1688 | 0.4780 | -1.0193 | 0.5566 | -1.0943 |
| | SZZ | 1.1339 | -4.5229 | 1.0036 | -4.1729 | 1.1135 | -4.4829 |
| | SXY | 0.3666 | -0.2111 | 0.2267 | -0.1171 | 0.3221 | -0.1845 |
| | SXZ | 0.4283 | -0.4428 | 0.5382 | -0.5382 | 0.4670 | -0.4781 |
| | SYZ | 1.9268 | -1.9322 | 1.7246 | -1.7246 | 1.8314 | -1.8314 |
| Girder 3 | SXX | 1.5474 | -2.1268 | 0.3674 | -0.8910 | 1.2733 | -1.8115 |
| | SYY | 0.6110 | -1.2048 | 0.4653 | -1.0421 | 0.5700 | -1.1469 |
| | SZZ | 1.1945 | -4.6854 | 0.9959 | -4.1992 | 1.1585 | -4.6141 |
| | SXY | 0.6450 | -0.3310 | 0.2303 | -0.1345 | 0.5491 | -0.2871 |
| | SXZ | 0.4235 | -0.4379 | 0.5320 | -0.5320 | 0.4618 | -0.4729 |
| | SYZ | 2.0001 | -2.0067 | 1.7399 | -1.7399 | 1.8940 | -1.8955 |
| Girder 4 | SXX | 1.7322 | -2.3775 | 0.3854 | -0.8918 | 1.4109 | -2.0164 |
| | SYY | 0.6305 | -1.2256 | 0.4615 | -1.0459 | 0.5848 | -1.1737 |
| | SZZ | 1.2296 | -4.7710 | 0.9874 | -4.2002 | 1.1834 | -4.6802 |
| | SXY | 0.6769 | -0.6040 | 0.2282 | -0.1983 | 0.5630 | -0.5020 |
| | SXZ | 0.4077 | -0.4140 | 0.5240 | -0.5240 | 0.4478 | -0.4527 |
| | SYZ | 2.0427 | -2.0486 | 1.7427 | -1.7427 | 1.9333 | -1.9363 |
| Girder 5 | SXX | 1.7318 | -2.3757 | 0.3673 | -0.8877 | 1.4156 | -2.0112 |
| | SYY | 0.6360 | -1.2292 | 0.4570 | -1.0439 | 0.5892 | -1.1838 |
| | SZZ | 1.2563 | -4.8140 | 0.9768 | -4.1780 | 1.1778 | -4.6812 |
| | SXY | 0.4952 | -0.6853 | 0.2257 | -0.2201 | 0.4002 | -0.5808 |
| | SXZ | 0.4010 | -0.3944 | 0.5143 | -0.5143 | 0.4269 | -0.4227 |
| | SYZ | 2.0525 | -2.0566 | 1.7356 | -1.7356 | 1.9451 | -1.9495 |
| Girder 6 | SXX | 1.2675 | -1.7451 | 0.2707 | -0.8783 | 1.0206 | -1.4929 |
| | SYY | 0.6392 | -1.2105 | 0.4576 | -1.0361 | 0.5922 | -1.1786 |
| | SZZ | 1.2506 | -4.7976 | 0.9768 | -4.1304 | 1.1526 | -4.6206 |
| | SXY | 0.2517 | -0.5544 | 0.2250 | -0.1884 | 0.2489 | -0.4731 |
| | SXZ | 0.4286 | -0.4142 | 0.5009 | -0.5009 | 0.4024 | -0.3921 |
| | SYZ | 2.0269 | -2.0333 | 1.7177 | -1.7177 | 1.9261 | -1.9310 |
| Girder 7 | SXX | 0.4568 | -1.0313 | 0.2375 | -1.0596 | 0.3750 | -1.0205 |
| | SYY | 0.6986 | -1.3198 | 0.5189 | -1.2477 | 0.6227 | -1.1990 |
| | SZZ | 1.2249 | -4.7514 | 0.8865 | -4.9855 | 1.1229 | -4.8068 |
| | SXY | 0.2693 | -0.2191 | 0.2634 | -0.1697 | 0.2726 | -0.2000 |
| | SXZ | 0.4486 | -0.4337 | 0.7811 | -0.7811 | 0.4572 | -0.4462 |
| | SYZ | 1.9882 | -1.9916 | 2.1134 | -2.1134 | 1.9932 | -1.9917 |

**Table 60**
**Max. and min. stresses of AASHTO type IV girders in group B with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4494 | -0.9813 | 0.4273 | -0.8934 | 0.4499 | -0.9695 |
| | SYY | 0.7338 | -1.2208 | 0.5533 | -1.1048 | 0.6972 | -1.2027 |
| | SZZ | 1.1648 | -4.5611 | 1.0096 | -4.1508 | 1.1387 | -4.5076 |
| | SXY | 0.1759 | -0.2362 | 0.1376 | -0.1565 | 0.1669 | -0.2075 |
| | SXZ | 0.4725 | -0.4923 | 0.5502 | -0.5502 | 0.5037 | -0.5190 |
| | SYZ | 1.9089 | -1.9131 | 1.6940 | -1.6940 | 1.8049 | -1.8041 |
| Girder 2 | SXX | 1.0068 | -1.3938 | 0.2541 | -0.8827 | 0.8314 | -1.1940 |
| | SYY | 0.6292 | -1.2360 | 0.4780 | -1.0193 | 0.5901 | -1.1281 |
| | SZZ | 1.1936 | -4.7593 | 1.0036 | -4.1729 | 1.1595 | -4.6652 |
| | SXY | 0.4518 | -0.2179 | 0.2267 | -0.1171 | 0.3878 | -0.1898 |
| | SXZ | 0.4684 | -0.4945 | 0.5382 | -0.5382 | 0.4978 | -0.5180 |
| | SYZ | 2.0217 | -2.0320 | 1.7246 | -1.7246 | 1.8984 | -1.8973 |
| Girder 3 | SXX | 1.8634 | -2.5588 | 0.3674 | -0.8910 | 1.5171 | -2.1447 |
| | SYY | 0.6608 | -1.2926 | 0.4653 | -1.0421 | 0.6084 | -1.1993 |
| | SZZ | 1.2082 | -5.0128 | 0.9959 | -4.1992 | 1.1690 | -4.8666 |
| | SXY | 0.7634 | -0.4071 | 0.2303 | -0.1345 | 0.6405 | -0.3459 |
| | SXZ | 0.4714 | -0.5045 | 0.5320 | -0.5320 | 0.4988 | -0.5244 |
| | SYZ | 2.1340 | -2.1515 | 1.7399 | -1.7399 | 1.9896 | -1.9937 |
| Girder 4 | SXX | 2.1324 | -2.9277 | 0.3854 | -0.8918 | 1.7197 | -2.4408 |
| | SYY | 0.6926 | -1.3279 | 0.4615 | -1.0459 | 0.6327 | -1.2430 |
| | SZZ | 1.1973 | -5.1704 | 0.9874 | -4.2002 | 1.1516 | -4.9883 |
| | SXY | 0.8279 | -0.7207 | 0.2282 | -0.1983 | 0.6738 | -0.5920 |
| | SXZ | 0.4525 | -0.4701 | 0.5240 | -0.5240 | 0.4824 | -0.4960 |
| | SYZ | 2.2093 | -2.2274 | 1.7427 | -1.7427 | 2.0562 | -2.0661 |
| Girder 5 | SXX | 2.1322 | -2.9256 | 0.3673 | -0.8877 | 1.7244 | -2.4355 |
| | SYY | 0.7008 | -1.3341 | 0.4570 | -1.0439 | 0.6384 | -1.2597 |
| | SZZ | 1.2571 | -5.2340 | 0.9768 | -4.1780 | 1.1926 | -4.9981 |
| | SXY | 0.5903 | -0.8338 | 0.2257 | -0.2201 | 0.4752 | -0.6949 |
| | SXZ | 0.4465 | -0.4283 | 0.5143 | -0.5143 | 0.4517 | -0.4407 |
| | SYZ | 2.2259 | -2.2388 | 1.7356 | -1.7356 | 2.0770 | -2.0910 |
| Girder 6 | SXX | 1.5283 | -2.1019 | 0.2707 | -0.8783 | 1.2218 | -1.7681 |
| | SYY | 0.6982 | -1.3067 | 0.4576 | -1.0361 | 0.6377 | -1.2527 |
| | SZZ | 1.2867 | -5.1906 | 0.9768 | -4.1304 | 1.1939 | -4.9004 |
| | SXY | 0.2818 | -0.6547 | 0.2250 | -0.1884 | 0.2677 | -0.5505 |
| | SXZ | 0.4894 | -0.4547 | 0.5009 | -0.5009 | 0.4172 | -0.3926 |
| | SYZ | 2.1829 | -2.2005 | 1.7177 | -1.7177 | 2.0464 | -2.0600 |
| Girder 7 | SXX | 0.5434 | -1.1063 | 0.2375 | -1.0596 | 0.4049 | -1.0728 |
| | SYY | 0.7668 | -1.4128 | 0.5189 | -1.2477 | 0.6753 | -1.2663 |
| | SZZ | 1.3220 | -5.1084 | 0.8865 | -4.9855 | 1.1978 | -5.0529 |
| | SXY | 0.2934 | -0.2763 | 0.2634 | -0.1697 | 0.2912 | -0.2441 |
| | SXZ | 0.5058 | -0.4726 | 0.7811 | -0.7811 | 0.4673 | -0.4425 |
| | SYZ | 2.1234 | -2.1333 | 2.1134 | -2.1134 | 2.0945 | -2.0938 |

The study usedthe stress Szz (the flexural stress in the horizontal direction) under load combination “Strength I max” because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 5 and 6 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 3.04 ksi was occurred at girder 5 at 48 feet along the bridge, while at most other locations, there wasn’t much difference between girder 5 and 6. The maximum tensile stress at the bottom of the girder, 1.26 ksi was occurred at girder 5 at 50 feet along the bridge, while at most other locations, there wasn’t much difference between girder 5 and 6.

For the Sugarcane truck load, 5 and 6 were determined to the critical girders. The maximum compressive stress at the top of the girder, 3.17 ksi was occurred at girder 6 at 60 feet along the bridge; the compressive stresses in girder 6 was slightly larger than them in girder 5 in the middle range of the bridge, while at most other locations, there wasn’t too much difference between girder 5 and 6. The maximum tensile stress at the bottom of the girder, 1.29 ksi was occurred at girder 6 at 49 feet along the bridge, while at most other locations, there wasn’t much difference between girder 5 and 6.

**Short term stress performances of AASHTO type V girder bridges in group B.** For the bridges with AASHTO Type V girder, by comparing the stress state of bridge girders under all load combinations, the load combination “Strength I max” also lead the maximum stresses of the girder, as shown in tables 61 and 62, so we can determine that the “Strength I max” is the governing load combination; the span length of this model was identified as 111 feet; and all the analysis below are based on it.

**Table 61**
**Max. and min. stresses of AASHTO type V girders in group B with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4213 | -0.9559 | 0.4352 | -0.9370 | 0.4297 | -0.9598 |
| | SYY | 0.8655 | -1.2394 | 0.7554 | -1.1898 | 0.8443 | -1.2359 |
| | SZZ | 0.9666 | -4.3885 | 0.8916 | -4.3217 | 0.9498 | -4.4139 |
| | SXY | 0.1576 | -0.2404 | 0.1808 | -0.1281 | 0.1669 | -0.2106 |
| | SXZ | 0.4003 | -0.4090 | 0.5374 | -0.5374 | 0.4448 | -0.4515 |
| | SYZ | 1.5864 | -1.5860 | 1.5843 | -1.5843 | 1.6007 | -1.6005 |
| Girder 2 | SXX | 0.4481 | -0.9624 | 0.2570 | -0.9273 | 0.3577 | -0.9626 |
| | SYY | 0.8640 | -1.2340 | 0.7455 | -1.1396 | 0.8409 | -1.1954 |
| | SZZ | 0.9997 | -4.4631 | 0.8989 | -4.3151 | 0.9750 | -4.4698 |
| | SXY | 0.2426 | -0.2342 | 0.2557 | -0.1199 | 0.2498 | -0.2040 |
| | SXZ | 0.3874 | -0.3970 | 0.5276 | -0.5276 | 0.4329 | -0.4402 |
| | SYZ | 1.6255 | -1.6263 | 1.5890 | -1.5890 | 1.6313 | -1.6316 |
| Girder 3 | SXX | 0.8036 | -0.9775 | 0.2388 | -0.9259 | 0.6325 | -0.9739 |
| | SYY | 0.8909 | -1.2554 | 0.7461 | -1.1343 | 0.8617 | -1.2065 |
| | SZZ | 1.0672 | -4.5347 | 0.8995 | -4.3123 | 1.0266 | -4.5244 |
| | SXY | 0.4023 | -0.2444 | 0.2641 | -0.1195 | 0.3406 | -0.2117 |
| | SXZ | 0.3847 | -0.3931 | 0.5262 | -0.5262 | 0.4306 | -0.4371 |
| | SYZ | 1.6597 | -1.6606 | 1.5883 | -1.5883 | 1.6531 | -1.6538 |
| Girder 4 | SXX | 0.8309 | -0.9894 | 0.2363 | -0.9249 | 0.6465 | -0.9819 |
| | SYY | 0.9119 | -1.2672 | 0.7454 | -1.1326 | 0.8777 | -1.2187 |
| | SZZ | 1.0715 | -4.5787 | 0.8988 | -4.3082 | 1.0246 | -4.5575 |
| | SXY | 0.3944 | -0.3818 | 0.2644 | -0.1195 | 0.3375 | -0.3235 |
| | SXZ | 0.3790 | -0.3831 | 0.5248 | -0.5248 | 0.4259 | -0.4290 |
| | SYZ | 1.6851 | -1.6861 | 1.5869 | -1.5869 | 1.6682 | -1.6691 |
| Girder 5 | SXX | 0.8306 | -1.0001 | 0.2351 | -0.9242 | 0.6490 | -0.9856 |
| | SYY | 0.9209 | -1.2736 | 0.7432 | -1.1325 | 0.8839 | -1.2285 |
| | SZZ | 1.0792 | -4.6094 | 0.8977 | -4.3039 | 1.0344 | -4.5703 |
| | SXY | 0.3332 | -0.3997 | 0.2634 | -0.1329 | 0.2897 | -0.3412 |
| | SXZ | 0.3704 | -0.3682 | 0.5237 | -0.5237 | 0.4191 | -0.4173 |
| | SYZ | 1.7008 | -1.7025 | 1.5860 | -1.5860 | 1.6757 | -1.6761 |
| Girder 6 | SXX | 0.7025 | -1.0060 | 0.2345 | -0.9228 | 0.5349 | -0.9847 |
| | SYY | 0.9195 | -1.2810 | 0.7345 | -1.1329 | 0.8804 | -1.2318 |
| | SZZ | 1.0968 | -4.6375 | 0.8949 | -4.2951 | 0.9985 | -4.5637 |
| | SXY | 0.2631 | -0.3612 | 0.2597 | -0.1308 | 0.2662 | -0.3117 |
| | SXZ | 0.3792 | -0.3721 | 0.5208 | -0.5208 | 0.4114 | -0.4066 |
| | SYZ | 1.7068 | -1.7082 | 1.5848 | -1.5848 | 1.6751 | -1.6756 |
| Girder 7 | SXX | 0.4426 | -1.0190 | 0.3076 | -1.0281 | 0.3803 | -1.0063 |
| | SYY | 0.9258 | -1.3474 | 0.7478 | -1.2644 | 0.8885 | -1.2430 |
| | SZZ | 1.0485 | -4.6482 | 0.9747 | -4.7830 | 1.0068 | -4.6799 |
| | SXY | 0.2701 | -0.1720 | 0.2941 | -0.1387 | 0.2810 | -0.1641 |
| | SXZ | 0.4004 | -0.3924 | 0.6712 | -0.6712 | 0.4380 | -0.4324 |
| | SYZ | 1.6990 | -1.7001 | 1.7855 | -1.7855 | 1.7146 | -1.7148 |

**Table 62**
**Max. and min. stresses of AASHTO type V girders in group B with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4394 | -0.9869 | 0.4352 | -0.9370 | 0.4437 | -0.9837 |
| | SYY | 0.9156 | -1.2647 | 0.7554 | -1.1898 | 0.8829 | -1.2554 |
| | SZZ | 1.0094 | -4.5505 | 0.8916 | -4.3217 | 0.9828 | -4.5388 |
| | SXY | 0.1767 | -0.2481 | 0.1808 | -0.1281 | 0.1816 | -0.2166 |
| | SXZ | 0.4279 | -0.4443 | 0.5374 | -0.5374 | 0.4661 | -0.4788 |
| | SYZ | 1.6320 | -1.6311 | 1.5843 | -1.5843 | 1.6359 | -1.6353 |
| Girder 2 | SXX | 0.5943 | -1.0056 | 0.2570 | -0.9273 | 0.4704 | -0.9959 |
| | SYY | 0.9142 | -1.2989 | 0.7455 | -1.1396 | 0.8795 | -1.2243 |
| | SZZ | 1.0438 | -4.6827 | 0.8989 | -4.3151 | 1.0090 | -4.6392 |
| | SXY | 0.3111 | -0.2430 | 0.2557 | -0.1199 | 0.2641 | -0.2107 |
| | SXZ | 0.4194 | -0.4422 | 0.5276 | -0.5276 | 0.4576 | -0.4751 |
| | SYZ | 1.6948 | -1.6950 | 1.5890 | -1.5890 | 1.6815 | -1.6834 |
| Girder 3 | SXX | 1.0736 | -1.0339 | 0.2388 | -0.9259 | 0.8408 | -1.0174 |
| | SYY | 0.9554 | -1.3326 | 0.7461 | -1.1343 | 0.9114 | -1.2474 |
| | SZZ | 1.0699 | -4.8141 | 0.8995 | -4.3123 | 1.0261 | -4.7400 |
| | SXY | 0.5172 | -0.2684 | 0.2641 | -0.1195 | 0.4292 | -0.2295 |
| | SXZ | 0.4198 | -0.4438 | 0.5262 | -0.5262 | 0.4576 | -0.4762 |
| | SYZ | 1.7500 | -1.7494 | 1.5883 | -1.5883 | 1.7199 | -1.7241 |
| Girder 4 | SXX | 1.1342 | -1.0517 | 0.2363 | -0.9249 | 0.8805 | -1.0305 |
| | SYY | 0.9876 | -1.3525 | 0.7454 | -1.1326 | 0.9360 | -1.2710 |
| | SZZ | 1.0695 | -4.8813 | 0.8988 | -4.3082 | 1.0353 | -4.7909 |
| | SXY | 0.5204 | -0.4832 | 0.2644 | -0.1195 | 0.4346 | -0.4018 |
| | SXZ | 0.4112 | -0.4220 | 0.5248 | -0.5248 | 0.4507 | -0.4591 |
| | SYZ | 1.7927 | -1.7945 | 1.5869 | -1.5869 | 1.7473 | -1.7510 |
| Girder 5 | SXX | 1.1339 | -1.0680 | 0.2351 | -0.9242 | 0.8830 | -1.0358 |
| | SYY | 1.0007 | -1.3593 | 0.7432 | -1.1325 | 0.9455 | -1.2885 |
| | SZZ | 1.0845 | -4.9233 | 0.8977 | -4.3039 | 1.0395 | -4.8088 |
| | SXY | 0.4217 | -0.5271 | 0.2634 | -0.1329 | 0.3569 | -0.4395 |
| | SXZ | 0.3962 | -0.3903 | 0.5237 | -0.5237 | 0.4390 | -0.4344 |
| | SYZ | 1.8160 | -1.8208 | 1.5860 | -1.5860 | 1.7593 | -1.7608 |
| Girder 6 | SXX | 0.9423 | -1.0756 | 0.2345 | -0.9228 | 0.7199 | -1.0327 |
| | SYY | 0.9970 | -1.3623 | 0.7345 | -1.1329 | 0.9402 | -1.2925 |
| | SZZ | 1.1109 | -4.9671 | 0.8949 | -4.2951 | 1.0348 | -4.7895 |
| | SXY | 0.2864 | -0.4638 | 0.2597 | -0.1308 | 0.2842 | -0.3908 |
| | SXZ | 0.4222 | -0.3995 | 0.5208 | -0.5208 | 0.4262 | -0.4105 |
| | SYZ | 1.8197 | -1.8254 | 1.5848 | -1.5848 | 1.7557 | -1.7552 |
| Girder 7 | SXX | 0.4733 | -1.0855 | 0.3076 | -1.0281 | 0.4039 | -1.0503 |
| | SYY | 0.9987 | -1.4279 | 0.7478 | -1.2644 | 0.9447 | -1.2990 |
| | SZZ | 1.1159 | -4.9626 | 0.9747 | -4.7830 | 1.0544 | -4.8871 |
| | SXY | 0.2918 | -0.2275 | 0.2941 | -0.1387 | 0.2977 | -0.2069 |
| | SXZ | 0.4448 | -0.4203 | 0.6712 | -0.6712 | 0.4497 | -0.4318 |
| | SYZ | 1.8039 | -1.8083 | 1.7855 | -1.7855 | 1.7881 | -1.7867 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination “Strength I max” because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 5 and 6 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.27 ksi was occurred at girder 5 at 54 feet along the bridge, while at most other locations, there wasn’t too much difference between girder 5 and 6. The maximum tensile stress at the bottom of the girder, 1.10 ksi was occurred at girder 6 at 55 feet along the bridge, while at most other locations, there wasn’t much difference between girder 5 and 6.

For the Sugarcane truck load, girder 5, 6 and 7 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.27 ksi was occurred at girder 5 at 48 feet along the bridge, while at most other locations, there wasn’t too much difference among girder 5, 6 and 7. The maximum tensile stress at the bottom of the girder, 1.12 ksi was occurred at girder 7 at 56 feet along the bridge, while at most other locations, there wasn’t much difference among girder 5, 6 and 7.

**Short term stress performances of AASHTO type VI girder bridges in group B.** For the bridges with AASHTO Type VI girder, by comparing the stress state of bridge girders under all load combinations, the load combination “Strength I max” also lead the maximum stresses of the girder, as shown in tables 22 and 23, so we can determine that the “Strength I max” is the governing load combination; the span length of this model was identified as 120 feet; and all the analysis below are based on it.

**Table 63**
**Max. and min. stresses of AASHTO type VI girders in group B with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4490 | -1.0123 | 0.4972 | -1.0474 | 0.4672 | -1.0316 |
| | SYY | 0.8064 | -1.3331 | 0.7299 | -1.3283 | 0.7934 | -1.3428 |
| | SZZ | 0.9906 | -4.6255 | 0.9869 | -4.8327 | 0.9994 | -4.7287 |
| | SXY | 0.1515 | -0.2333 | 0.1962 | -0.1293 | 0.1578 | -0.1958 |
| | SXZ | 0.4179 | -0.4277 | 0.6466 | -0.6466 | 0.4882 | -0.4959 |
| | SYZ | 1.6869 | -1.6876 | 1.7804 | -1.7804 | 1.7286 | -1.7292 |
| Girder 2 | SXX | 0.4397 | -1.0158 | 0.3005 | -1.0361 | 0.3443 | -1.0317 |
| | SYY | 0.8065 | -1.3096 | 0.7208 | -1.2691 | 0.7915 | -1.2923 |
| | SZZ | 1.0159 | -4.6927 | 0.9954 | -4.8258 | 1.0208 | -4.7789 |
| | SXY | 0.2320 | -0.2268 | 0.2791 | -0.1305 | 0.2486 | -0.1890 |
| | SXZ | 0.4038 | -0.4147 | 0.6379 | -0.6379 | 0.4758 | -0.4842 |
| | SYZ | 1.7230 | -1.7247 | 1.7850 | -1.7850 | 1.7574 | -1.7587 |
| Girder 3 | SXX | 0.7549 | -1.0294 | 0.2799 | -1.0348 | 0.5854 | -1.0419 |
| | SYY | 0.8317 | -1.3301 | 0.7208 | -1.2634 | 0.8109 | -1.3010 |
| | SZZ | 1.0451 | -4.7582 | 0.9961 | -4.8237 | 1.0335 | -4.8290 |
| | SXY | 0.3647 | -0.2367 | 0.2882 | -0.1304 | 0.3133 | -0.1966 |
| | SXZ | 0.4015 | -0.4112 | 0.6373 | -0.6373 | 0.4740 | -0.4814 |
| | SYZ | 1.7538 | -1.7538 | 1.7849 | -1.7849 | 1.7772 | -1.7790 |
| Girder 4 | SXX | 0.7912 | -1.0400 | 0.2769 | -1.0340 | 0.6044 | -1.0490 |
| | SYY | 0.8517 | -1.3423 | 0.7189 | -1.2626 | 0.8257 | -1.3118 |
| | SZZ | 1.0546 | -4.7979 | 0.9952 | -4.8200 | 1.0413 | -4.8588 |
| | SXY | 0.3633 | -0.3407 | 0.2880 | -0.1301 | 0.3164 | -0.2933 |
| | SXZ | 0.3976 | -0.4022 | 0.6362 | -0.6362 | 0.4707 | -0.4743 |
| | SYZ | 1.7775 | -1.7787 | 1.7842 | -1.7842 | 1.7912 | -1.7925 |
| Girder 5 | SXX | 0.7910 | -1.0504 | 0.2752 | -1.0336 | 0.6076 | -1.0529 |
| | SYY | 0.8596 | -1.3502 | 0.7151 | -1.2637 | 0.8308 | -1.3211 |
| | SZZ | 1.0629 | -4.8287 | 0.9937 | -4.8163 | 1.0447 | -4.8736 |
| | SXY | 0.2940 | -0.3690 | 0.2864 | -0.1412 | 0.2689 | -0.3202 |
| | SXZ | 0.3917 | -0.3896 | 0.6353 | -0.6353 | 0.4661 | -0.4644 |
| | SYZ | 1.7930 | -1.7956 | 1.7843 | -1.7843 | 1.7990 | -1.7992 |
| Girder 6 | SXX | 0.6561 | -1.0572 | 0.2744 | -1.0323 | 0.4871 | -1.0531 |
| | SYY | 0.8571 | -1.3599 | 0.7054 | -1.2653 | 0.8261 | -1.3248 |
| | SZZ | 1.0707 | -4.8591 | 0.9904 | -4.8072 | 1.0440 | -4.8723 |
| | SXY | 0.2539 | -0.3252 | 0.2822 | -0.1448 | 0.2658 | -0.2882 |
| | SXZ | 0.3987 | -0.3909 | 0.6320 | -0.6320 | 0.4605 | -0.4555 |
| | SYZ | 1.8003 | -1.8034 | 1.7837 | -1.7837 | 1.8002 | -1.7998 |
| Girder 7 | SXX | 0.4698 | -1.0729 | 0.3514 | -1.1369 | 0.3977 | -1.0755 |
| | SYY | 0.8624 | -1.4361 | 0.7207 | -1.3951 | 0.8340 | -1.3355 |
| | SZZ | 1.0512 | -4.8751 | 1.0701 | -5.2924 | 1.0693 | -4.9940 |
| | SXY | 0.2613 | -0.1662 | 0.3186 | -0.1591 | 0.2815 | -0.1660 |
| | SXZ | 0.4200 | -0.4116 | 0.7798 | -0.7798 | 0.4871 | -0.4815 |
| | SYZ | 1.7945 | -1.7972 | 1.9831 | -1.9831 | 1.8409 | -1.8404 |

**Table 64**
**Max. and min. stresses of AASHTO type VI girders in group B with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4666 | -1.0433 | 0.4972 | -1.0474 | 0.4808 | -1.0556 |
| | SYY | 0.8482 | -1.3610 | 0.7299 | -1.3283 | 0.8257 | -1.3643 |
| | SZZ | 1.0298 | -4.7847 | 0.9869 | -4.8327 | 1.0296 | -4.8515 |
| | SXY | 0.1559 | -0.2410 | 0.1962 | -0.1293 | 0.1707 | -0.2018 |
| | SXZ | 0.4428 | -0.4604 | 0.6466 | -0.6466 | 0.5075 | -0.5211 |
| | SYZ | 1.7333 | -1.7340 | 1.7804 | -1.7804 | 1.7645 | -1.7650 |
| Girder 2 | SXX | 0.5778 | -1.0581 | 0.3005 | -1.0361 | 0.4509 | -1.0644 |
| | SYY | 0.8521 | -1.3709 | 0.7208 | -1.2691 | 0.8267 | -1.3221 |
| | SZZ | 1.0650 | -4.9047 | 0.9954 | -4.8258 | 1.0588 | -4.9425 |
| | SXY | 0.2993 | -0.2362 | 0.2791 | -0.1305 | 0.2620 | -0.1962 |
| | SXZ | 0.4322 | -0.4557 | 0.6379 | -0.6379 | 0.4976 | -0.5158 |
| | SYZ | 1.7895 | -1.7916 | 1.7850 | -1.7850 | 1.8069 | -1.8103 |
| Girder 3 | SXX | 1.0392 | -1.0834 | 0.2799 | -1.0348 | 0.8047 | -1.0836 |
| | SYY | 0.8913 | -1.4032 | 0.7208 | -1.2634 | 0.8569 | -1.3414 |
| | SZZ | 1.0920 | -5.0236 | 0.9961 | -4.8237 | 1.0797 | -5.0338 |
| | SXY | 0.5149 | -0.2633 | 0.2882 | -0.1304 | 0.4292 | -0.2264 |
| | SXZ | 0.4323 | -0.4565 | 0.6373 | -0.6373 | 0.4977 | -0.5164 |
| | SYZ | 1.8407 | -1.8393 | 1.7849 | -1.7849 | 1.8415 | -1.8471 |
| Girder 4 | SXX | 1.0799 | -1.0996 | 0.2769 | -1.0340 | 0.8271 | -1.0956 |
| | SYY | 0.9224 | -1.4238 | 0.7189 | -1.2626 | 0.8803 | -1.3631 |
| | SZZ | 1.1089 | -5.0867 | 0.9952 | -4.8200 | 1.0925 | -5.0816 |
| | SXY | 0.5042 | -0.4875 | 0.2880 | -0.1301 | 0.4251 | -0.4066 |
| | SXZ | 0.4272 | -0.4386 | 0.6362 | -0.6362 | 0.4936 | -0.5024 |
| | SYZ | 1.8802 | -1.8820 | 1.7842 | -1.7842 | 1.8668 | -1.8711 |
| Girder 5 | SXX | 1.0797 | -1.1158 | 0.2752 | -1.0336 | 0.8303 | -1.1012 |
| | SYY | 0.9346 | -1.4343 | 0.7151 | -1.2637 | 0.8886 | -1.3784 |
| | SZZ | 1.1162 | -5.1293 | 0.9937 | -4.8163 | 1.0978 | -5.1034 |
| | SXY | 0.4254 | -0.5115 | 0.2864 | -0.1412 | 0.3653 | -0.4302 |
| | SXZ | 0.4172 | -0.4117 | 0.6353 | -0.6353 | 0.4857 | -0.4815 |
| | SYZ | 1.9036 | -1.9098 | 1.7843 | -1.7843 | 1.8792 | -1.8802 |
| Girder 6 | SXX | 0.9114 | -1.1248 | 0.2744 | -1.0323 | 0.6840 | -1.1004 |
| | SYY | 0.9297 | -1.4408 | 0.7054 | -1.2653 | 0.8821 | -1.3831 |
| | SZZ | 1.1149 | -5.1754 | 0.9904 | -4.8072 | 1.0955 | -5.0957 |
| | SXY | 0.2775 | -0.4636 | 0.2822 | -0.1448 | 0.2840 | -0.3949 |
| | SXZ | 0.4364 | -0.4146 | 0.6320 | -0.6320 | 0.4773 | -0.4627 |
| | SYZ | 1.9102 | -1.9182 | 1.7837 | -1.7837 | 1.8791 | -1.8775 |
| Girder 7 | SXX | 0.4999 | -1.1391 | 0.3514 | -1.1369 | 0.4209 | -1.1202 |
| | SYY | 0.9299 | -1.5184 | 0.7207 | -1.3951 | 0.8861 | -1.3904 |
| | SZZ | 1.1143 | -5.1834 | 1.0701 | -5.2924 | 1.1181 | -5.2054 |
| | SXY | 0.2830 | -0.2171 | 0.3186 | -0.1591 | 0.2982 | -0.2053 |
| | SXZ | 0.4594 | -0.4359 | 0.7798 | -0.7798 | 0.5019 | -0.4852 |
| | SYZ | 1.8994 | -1.9063 | 1.9831 | -1.9831 | 1.9154 | -1.9128 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination "Strength I max" because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 5 and 6 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.25 ksi was occurred at girder 5 at 60 feet along the bridge, while at most other locations, there wasn't too much difference between girder 5 and 6. The maximum tensile stress at the bottom of the girder, 1.07 ksi was occurred at girder 6 at 60 feet along the bridge, while at most other locations, there wasn't much difference between girder 5 and 6.

For the Sugarcane truck load, girder 5 and 6 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.27 ksi was occurred at girder 5 at 54 feet along the bridge, while at most other locations, there wasn't too much difference between girder 5 and 6. The maximum tensile stress at the bottom of the girder, 1.10 ksi was occurred at girder 6 at 56 feet along the bridge, while at most other locations, there wasn't much difference among girder 5 and 6.

**Short term stress performances of AASHTO BT-54 girder bridges in group B.** For the bridges with AASHTO Bulb-Tee 54 girder, by comparing the stress state of bridge girders under all load combinations, the load combination "Strength I max" also leads the maximum stresses of the girder, as shown in tables 65 and 65, so we can determine that the "Strength I max" is the governing load combination; the span length of this model was identified as 99 feet; and all the analysis below are based on it.

**Table 65**
**Max. and min. stresses of AASHTO BT-54 girders in group B with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.3485 | -0.8750 | 0.3399 | -0.8086 | 0.3507 | -0.8669 |
| | SYY | 0.7781 | -1.0732 | 0.6384 | -0.9749 | 0.7499 | -1.0575 |
| | SZZ | 1.0350 | -4.0778 | 0.8765 | -3.7817 | 1.0043 | -4.0450 |
| | SXY | 0.1587 | -0.1642 | 0.1187 | -0.1128 | 0.1489 | -0.1508 |
| | SXZ | 0.3873 | -0.4015 | 0.4874 | -0.4874 | 0.4223 | -0.4333 |
| | SYZ | 1.6445 | -1.6441 | 1.5093 | -1.5093 | 1.5984 | -1.6002 |
| Girder 2 | SXX | 0.5119 | -0.8822 | 0.1648 | -0.7979 | 0.4103 | -0.8700 |
| | SYY | 0.7775 | -1.0419 | 0.6275 | -0.9213 | 0.7471 | -1.0073 |
| | SZZ | 1.0736 | -4.1702 | 0.8700 | -3.7725 | 1.0328 | -4.1141 |
| | SXY | 0.2896 | -0.1353 | 0.1458 | -0.1122 | 0.2479 | -0.1307 |
| | SXZ | 0.3754 | -0.3908 | 0.4763 | -0.4763 | 0.4108 | -0.4227 |
| | SYZ | 1.6967 | -1.7007 | 1.5175 | -1.5175 | 1.6410 | -1.6427 |
| Girder 3 | SXX | 0.8864 | -0.9020 | 0.1653 | -0.7970 | 0.7018 | -0.8850 |
| | SYY | 0.8096 | -1.0690 | 0.6267 | -0.9196 | 0.7717 | -1.0253 |
| | SZZ | 1.1458 | -4.2621 | 0.8652 | -3.7688 | 1.0874 | -4.1842 |
| | SXY | 0.4544 | -0.2357 | 0.1525 | -0.1125 | 0.3811 | -0.2045 |
| | SXZ | 0.3728 | -0.3863 | 0.4744 | -0.4744 | 0.4085 | -0.4189 |
| | SYZ | 1.7395 | -1.7447 | 1.5166 | -1.5166 | 1.6717 | -1.6732 |
| Girder 4 | SXX | 0.9297 | -0.9153 | 0.1653 | -0.7959 | 0.7293 | -0.8951 |
| | SYY | 0.8370 | -1.0889 | 0.6261 | -0.9181 | 0.7926 | -1.0401 |
| | SZZ | 1.1238 | -4.3173 | 0.8591 | -3.7639 | 1.0689 | -4.2257 |
| | SXY | 0.4479 | -0.4186 | 0.1525 | -0.1220 | 0.3781 | -0.3523 |
| | SXZ | 0.3653 | -0.3716 | 0.4725 | -0.4725 | 0.4023 | -0.4072 |
| | SYZ | 1.7710 | -1.7766 | 1.5147 | -1.5147 | 1.6926 | -1.6930 |
| Girder 5 | SXX | 0.9289 | -0.9274 | 0.1654 | -0.7947 | 0.7319 | -0.8995 |
| | SYY | 0.8490 | -1.1010 | 0.6228 | -0.9172 | 0.8009 | -1.0484 |
| | SZZ | 1.1468 | -4.3541 | 0.8535 | -3.7578 | 1.0702 | -4.2430 |
| | SXY | 0.3600 | -0.4565 | 0.1512 | -0.1344 | 0.3087 | -0.3850 |
| | SXZ | 0.3539 | -0.3499 | 0.4708 | -0.4708 | 0.3932 | -0.3901 |
| | SYZ | 1.7896 | -1.7949 | 1.5127 | -1.5127 | 1.7018 | -1.7000 |
| Girder 6 | SXX | 0.7701 | -0.9347 | 0.1655 | -0.7921 | 0.5948 | -0.8974 |
| | SYY | 0.8497 | -1.1073 | 0.6157 | -0.9152 | 0.7993 | -1.0482 |
| | SZZ | 1.1941 | -4.3945 | 0.8540 | -3.7443 | 1.0753 | -4.2308 |
| | SXY | 0.1595 | -0.4088 | 0.1485 | -0.1294 | 0.1597 | -0.3476 |
| | SXZ | 0.3670 | -0.3544 | 0.4665 | -0.4665 | 0.3822 | -0.3735 |
| | SYZ | 1.7955 | -1.7999 | 1.5083 | -1.5083 | 1.6982 | -1.6954 |
| Girder 7 | SXX | 0.3727 | -0.9519 | 0.1998 | -0.9208 | 0.3134 | -0.9251 |
| | SYY | 0.8637 | -1.1920 | 0.6336 | -1.0676 | 0.8143 | -1.0690 |
| | SZZ | 1.1595 | -4.4108 | 0.8044 | -4.3486 | 1.0619 | -4.3740 |
| | SXY | 0.1981 | -0.2163 | 0.1777 | -0.1388 | 0.1897 | -0.1964 |
| | SXZ | 0.3917 | -0.3771 | 0.6626 | -0.6626 | 0.4194 | -0.4089 |
| | SYZ | 1.7935 | -1.7964 | 1.7758 | -1.7758 | 1.7545 | -1.7515 |

**Table 66**
**Max. and min. stresses of AASHTO BT-54 girders in group B with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.3677 | -0.9114 | 0.3399 | -0.8086 | 0.3655 | -0.8950 |
| | SYY | 0.8381 | -1.1043 | 0.6384 | -0.9749 | 0.7962 | -1.0814 |
| | SZZ | 1.0971 | -4.2622 | 0.8765 | -3.7817 | 1.0522 | -4.1873 |
| | SXY | 0.1754 | -0.1618 | 0.1187 | -0.1128 | 0.1514 | -0.1489 |
| | SXZ | 0.4221 | -0.4480 | 0.4874 | -0.4874 | 0.4492 | -0.4692 |
| | SYZ | 1.6960 | -1.6954 | 1.5093 | -1.5093 | 1.6436 | -1.6483 |
| Girder 2 | SXX | 0.6728 | -0.9333 | 0.1648 | -0.7979 | 0.5345 | -0.9094 |
| | SYY | 0.8328 | -1.0999 | 0.6275 | -0.9213 | 0.7897 | -1.0411 |
| | SZZ | 1.1249 | -4.4286 | 0.8700 | -3.7725 | 1.0723 | -4.3134 |
| | SXY | 0.3709 | -0.1454 | 0.1458 | -0.1122 | 0.3106 | -0.1385 |
| | SXZ | 0.4166 | -0.4513 | 0.4763 | -0.4763 | 0.4426 | -0.4694 |
| | SYZ | 1.7824 | -1.7903 | 1.5175 | -1.5175 | 1.7075 | -1.7142 |
| Girder 3 | SXX | 1.1000 | -0.9695 | 0.1653 | -0.7970 | 0.8666 | -0.9371 |
| | SYY | 0.8815 | -1.1440 | 0.6267 | -0.9196 | 0.8271 | -1.0737 |
| | SZZ | 1.1257 | -4.5983 | 0.8652 | -3.7688 | 1.0714 | -4.4435 |
| | SXY | 0.5619 | -0.2996 | 0.1525 | -0.1125 | 0.4640 | -0.2538 |
| | SXZ | 0.4178 | -0.4545 | 0.4744 | -0.4744 | 0.4432 | -0.4715 |
| | SYZ | 1.8565 | -1.8687 | 1.5166 | -1.5166 | 1.7621 | -1.7701 |
| Girder 4 | SXX | 1.1966 | -0.9904 | 0.1653 | -0.7959 | 0.9352 | -0.9530 |
| | SYY | 0.9219 | -1.1771 | 0.6261 | -0.9181 | 0.8580 | -1.1010 |
| | SZZ | 1.1091 | -4.6793 | 0.8591 | -3.7639 | 1.0564 | -4.5049 |
| | SXY | 0.5602 | -0.5372 | 0.1525 | -0.1220 | 0.4648 | -0.4438 |
| | SXZ | 0.4064 | -0.4219 | 0.4725 | -0.4725 | 0.4340 | -0.4459 |
| | SYZ | 1.9115 | -1.9280 | 1.5147 | -1.5147 | 1.8009 | -1.8024 |
| Girder 5 | SXX | 1.1957 | -1.0070 | 0.1654 | -0.7947 | 0.9377 | -0.9589 |
| | SYY | 0.9385 | -1.1936 | 0.6228 | -0.9172 | 0.8699 | -1.1140 |
| | SZZ | 1.1444 | -4.7301 | 0.8535 | -3.7578 | 1.0742 | -4.5268 |
| | SXY | 0.4654 | -0.5717 | 0.1512 | -0.1344 | 0.3900 | -0.4738 |
| | SXZ | 0.3838 | -0.3746 | 0.4708 | -0.4708 | 0.4163 | -0.4091 |
| | SYZ | 1.9385 | -1.9550 | 1.5127 | -1.5127 | 1.8138 | -1.8139 |
| Girder 6 | SXX | 0.9442 | -1.0162 | 0.1655 | -0.7921 | 0.7291 | -0.9528 |
| | SYY | 0.9363 | -1.1946 | 0.6157 | -0.9152 | 0.8661 | -1.1117 |
| | SZZ | 1.1988 | -4.7896 | 0.8540 | -3.7443 | 1.1178 | -4.4929 |
| | SXY | 0.1960 | -0.5056 | 0.1485 | -0.1294 | 0.1751 | -0.4222 |
| | SXZ | 0.4274 | -0.3901 | 0.4665 | -0.4665 | 0.3969 | -0.3709 |
| | SYZ | 1.9361 | -1.9483 | 1.5083 | -1.5083 | 1.8032 | -1.8079 |
| Girder 7 | SXX | 0.4265 | -1.0293 | 0.1998 | -0.9208 | 0.3387 | -0.9732 |
| | SYY | 0.9453 | -1.2783 | 0.6336 | -1.0676 | 0.8772 | -1.1243 |
| | SZZ | 1.2470 | -4.7813 | 0.8044 | -4.3486 | 1.1293 | -4.6017 |
| | SXY | 0.2099 | -0.2717 | 0.1777 | -0.1388 | 0.1988 | -0.2392 |
| | SXZ | 0.4551 | -0.4139 | 0.6626 | -0.6626 | 0.4283 | -0.3977 |
| | SYZ | 1.9177 | -1.9234 | 1.7758 | -1.7758 | 1.8454 | -1.8336 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination “Strength I max” because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 5 and 6 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.57 ksi was occurred at girder 5 at 48 feet along the bridge, while at most other locations, there wasn’t much difference between girder 5 and 6. The maximum tensile stress at the bottom of the girder, 1.19 ksi was occurred at girder 6 at 49 feet along the bridge, while at most other locations, there wasn’t much difference between girder 5 and 6.

For the Sugarcane truck load, girder 5, 6 and 7 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.72 ksi was occurred at girder 6 at 59 feet along the bridge, while at distance 44 feet to 58 feet, the stresses in girder 7 were governing, and at most other locations, there wasn’t much difference among girder 5, 6 and 7. The maximum tensile stress at the bottom of the girder, 1.25 ksi was occurred at girder 7 at 50 feet along the bridge, while at most other locations, there wasn’t much difference among girder 5, 6 and 7.

**Short term stress performances of AASHTO BT-63 girder bridges in group B.** For the bridges with AASHTO Bulb-Tee 63 girder, by comparing the stress state of bridge girders under all load combinations, the load combination “Strength I max” also leads the maximum stresses of the girder, as shown in tables 67 and 68, so we can determine that the “Strength I max” is the governing load combination; the span length of this model was identified as 105 feet; and all the analysis below are based on it.

**Table 67**
**Max. and min. stresses of AASHTO BT-63 girders in group B with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.3460 | -0.8649 | 0.3671 | -0.8524 | 0.3564 | -0.8715 |
| | SYY | 0.6321 | -1.0684 | 0.5404 | -1.0211 | 0.6151 | -1.0667 |
| | SZZ | 0.9526 | -4.0204 | 0.8506 | -3.9930 | 0.9361 | -4.0609 |
| | SXY | 0.1612 | -0.1676 | 0.1188 | -0.1177 | 0.1507 | -0.1513 |
| | SXZ | 0.3919 | -0.3849 | 0.5643 | -0.5643 | 0.4476 | -0.4422 |
| | SYZ | 1.6671 | -1.6672 | 1.5996 | -1.5996 | 1.6167 | -1.6152 |
| Girder 2 | SXX | 0.4720 | -0.8706 | 0.1923 | -0.8415 | 0.3738 | -0.8733 |
| | SYY | 0.6282 | -1.0287 | 0.5301 | -0.9653 | 0.6100 | -1.0138 |
| | SZZ | 0.9946 | -4.1058 | 0.8469 | -3.9847 | 0.9678 | -4.1248 |
| | SXY | 0.2650 | -0.1364 | 0.1369 | -0.1195 | 0.2291 | -0.1334 |
| | SXZ | 0.3809 | -0.3724 | 0.5545 | -0.5545 | 0.4371 | -0.4306 |
| | SYZ | 1.7183 | -1.7161 | 1.6085 | -1.6085 | 1.6577 | -1.6560 |
| Girder 3 | SXX | 0.7868 | -0.8889 | 0.1924 | -0.8409 | 0.6167 | -0.8874 |
| | SYY | 0.6555 | -1.0551 | 0.5284 | -0.9641 | 0.6306 | -1.0306 |
| | SZZ | 1.0369 | -4.1917 | 0.8427 | -3.9822 | 0.9994 | -4.1904 |
| | SXY | 0.3964 | -0.2112 | 0.1436 | -0.1195 | 0.3370 | -0.1859 |
| | SXZ | 0.3778 | -0.3696 | 0.5533 | -0.5533 | 0.4346 | -0.4283 |
| | SYZ | 1.7592 | -1.7561 | 1.6085 | -1.6085 | 1.6862 | -1.6845 |
| Girder 4 | SXX | 0.8272 | -0.9011 | 0.1922 | -0.8401 | 0.6405 | -0.8966 |
| | SYY | 0.6798 | -1.0744 | 0.5263 | -0.9635 | 0.6488 | -1.0446 |
| | SZZ | 1.0417 | -4.2438 | 0.8366 | -3.9781 | 1.0018 | -4.2297 |
| | SXY | 0.3901 | -0.3620 | 0.1430 | -0.1230 | 0.3352 | -0.3092 |
| | SXZ | 0.3665 | -0.3627 | 0.5517 | -0.5517 | 0.4256 | -0.4226 |
| | SYZ | 1.7891 | -1.7852 | 1.6074 | -1.6074 | 1.7044 | -1.7040 |
| Girder 5 | SXX | 0.8268 | -0.9121 | 0.1919 | -0.8393 | 0.6442 | -0.9008 |
| | SYY | 0.6891 | -1.0859 | 0.5209 | -0.9637 | 0.6544 | -1.0527 |
| | SZZ | 1.0632 | -4.2793 | 0.8304 | -3.9731 | 0.9792 | -4.2463 |
| | SXY | 0.3067 | -0.3990 | 0.1407 | -0.1380 | 0.2704 | -0.3421 |
| | SXZ | 0.3503 | -0.3525 | 0.5502 | -0.5502 | 0.4128 | -0.4145 |
| | SYZ | 1.8071 | -1.8032 | 1.6065 | -1.6065 | 1.7145 | -1.7128 |
| Girder 6 | SXX | 0.6790 | -0.9193 | 0.1914 | -0.8367 | 0.5149 | -0.8993 |
| | SYY | 0.6873 | -1.0920 | 0.5113 | -0.9627 | 0.6502 | -1.0532 |
| | SZZ | 1.0795 | -4.3170 | 0.8304 | -3.9590 | 0.9624 | -4.2368 |
| | SXY | 0.1390 | -0.3547 | 0.1367 | -0.1379 | 0.1349 | -0.3086 |
| | SXZ | 0.3551 | -0.3629 | 0.5454 | -0.5454 | 0.3994 | -0.4047 |
| | SYZ | 1.8134 | -1.8102 | 1.6027 | -1.6027 | 1.7183 | -1.7159 |
| Girder 7 | SXX | 0.3697 | -0.9371 | 0.2268 | -0.9819 | 0.3028 | -0.9327 |
| | SYY | 0.7006 | -1.1781 | 0.5341 | -1.1367 | 0.6661 | -1.0802 |
| | SZZ | 1.0651 | -4.3353 | 0.8113 | -4.6390 | 0.9627 | -4.4062 |
| | SXY | 0.1986 | -0.1967 | 0.1804 | -0.1569 | 0.1931 | -0.1857 |
| | SXZ | 0.3774 | -0.3863 | 0.7644 | -0.7644 | 0.4430 | -0.4494 |
| | SYZ | 1.8119 | -1.8099 | 1.9043 | -1.9043 | 1.7758 | -1.7786 |

**Table 68**
**Max. and min. stresses of AASHTO BT-63 girders in group B with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.3626 | -0.8992 | 0.3671 | -0.8524 | 0.3692 | -0.8980 |
| | SYY | 0.6782 | -1.1007 | 0.5404 | -1.0211 | 0.6507 | -1.0915 |
| | SZZ | 1.0100 | -4.1891 | 0.8506 | -3.9930 | 0.9803 | -4.1910 |
| | SXY | 0.1641 | -0.1641 | 0.1188 | -0.1177 | 0.1529 | -0.1486 |
| | SXZ | 0.4252 | -0.4251 | 0.5643 | -0.5643 | 0.4733 | -0.4732 |
| | SYZ | 1.7200 | -1.7203 | 1.5996 | -1.5996 | 1.6633 | -1.6634 |
| Girder 2 | SXX | 0.5998 | -0.9182 | 0.1923 | -0.8415 | 0.4724 | -0.9101 |
| | SYY | 0.6681 | -1.0831 | 0.5301 | -0.9653 | 0.6407 | -1.0476 |
| | SZZ | 1.0462 | -4.3408 | 0.8469 | -3.9847 | 1.0077 | -4.3060 |
| | SXY | 0.3311 | -0.1446 | 0.1369 | -0.1195 | 0.2801 | -0.1398 |
| | SXZ | 0.4224 | -0.4209 | 0.5545 | -0.5545 | 0.4692 | -0.4680 |
| | SYZ | 1.8036 | -1.8031 | 1.6085 | -1.6085 | 1.7252 | -1.7248 |
| Girder 3 | SXX | 1.0267 | -0.9514 | 0.1924 | -0.8409 | 0.8018 | -0.9356 |
| | SYY | 0.7032 | -1.1252 | 0.5284 | -0.9641 | 0.6674 | -1.0776 |
| | SZZ | 1.0440 | -4.4972 | 0.8427 | -3.9822 | 1.0050 | -4.4261 |
| | SXY | 0.5409 | -0.2795 | 0.1436 | -0.1195 | 0.4485 | -0.2385 |
| | SXZ | 0.4248 | -0.4219 | 0.5533 | -0.5533 | 0.4709 | -0.4686 |
| | SYZ | 1.8742 | -1.8729 | 1.6085 | -1.6085 | 1.7760 | -1.7753 |
| Girder 4 | SXX | 1.0682 | -0.9727 | 0.1922 | -0.8401 | 0.8263 | -0.9518 |
| | SYY | 0.7333 | -1.1566 | 0.5263 | -0.9635 | 0.6900 | -1.1019 |
| | SZZ | 1.0214 | -4.5900 | 0.8366 | -3.9781 | 0.9861 | -4.4968 |
| | SXY | 0.5255 | -0.5093 | 0.1430 | -0.1230 | 0.4396 | -0.4228 |
| | SXZ | 0.4071 | -0.4063 | 0.5517 | -0.5517 | 0.4568 | -0.4563 |
| | SYZ | 1.9276 | -1.9254 | 1.6074 | -1.6074 | 1.8084 | -1.8084 |
| Girder 5 | SXX | 1.0677 | -0.9882 | 0.1919 | -0.8393 | 0.8301 | -0.9561 |
| | SYY | 0.7459 | -1.1736 | 0.5209 | -0.9637 | 0.6982 | -1.1142 |
| | SZZ | 1.0496 | -4.6396 | 0.8304 | -3.9731 | 0.9600 | -4.5096 |
| | SXY | 0.4384 | -0.5370 | 0.1407 | -0.1380 | 0.3721 | -0.4485 |
| | SXZ | 0.3788 | -0.3787 | 0.5502 | -0.5502 | 0.4347 | -0.4347 |
| | SYZ | 1.9549 | -1.9527 | 1.6065 | -1.6065 | 1.8268 | -1.8250 |
| Girder 6 | SXX | 0.8888 | -0.9962 | 0.1914 | -0.8367 | 0.6767 | -0.9504 |
| | SYY | 0.7469 | -1.1780 | 0.5113 | -0.9627 | 0.6962 | -1.1134 |
| | SZZ | 1.1033 | -4.6829 | 0.8304 | -3.9590 | 1.0289 | -4.4770 |
| | SXY | 0.1833 | -0.4873 | 0.1367 | -0.1379 | 0.1723 | -0.4109 |
| | SXZ | 0.3992 | -0.4024 | 0.5454 | -0.5454 | 0.4111 | -0.4132 |
| | SYZ | 1.9551 | -1.9538 | 1.6027 | -1.6027 | 1.8257 | -1.8247 |
| Girder 7 | SXX | 0.3999 | -1.0109 | 0.2268 | -0.9819 | 0.3261 | -0.9784 |
| | SYY | 0.7637 | -1.2665 | 0.5341 | -1.1367 | 0.7147 | -1.1350 |
| | SZZ | 1.1490 | -4.6801 | 0.8113 | -4.6390 | 1.0274 | -4.6206 |
| | SXY | 0.2088 | -0.2449 | 0.1804 | -0.1569 | 0.2010 | -0.2229 |
| | SXZ | 0.4243 | -0.4272 | 0.7644 | -0.7644 | 0.4494 | -0.4517 |
| | SYZ | 1.9401 | -1.9401 | 1.9043 | -1.9043 | 1.8650 | -1.8658 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination “Strength I max” because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 5 and 6 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.32 ksi was occurred at girder 5 at 45 feet along the bridge, while at most other locations, the compressive stresses in girder 6 were slightly larger than them in girder 5. The maximum tensile stress at the bottom of the girder, 1.08 ksi was occurred at girder 6 at 52 feet along the bridge, while at most other locations, there wasn’t much difference between girder 5 and 6.

For the Sugarcane truck load, girder 5, 6 and 7 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.50 ksi was occurred at girder 6 at 65 feet along the bridge, while at most other locations, there wasn’t much difference among girder 5, 6 and 7. The maximum tensile stress at the bottom of the girder, 1.15 ksi was occurred at girder 7 at 53 feet along the bridge, while at most other locations, there wasn’t much difference among girder 5, 6 and 7.

**Short term stress performances of AASHTO BT-72 girder bridges in group B.** For the bridges with AASHTO Bulb-Tee 72 girder, by comparing the stress state of bridge girders under all load combinations, the load combination “Strength I max” also leads the maximum stresses of the girder, as shown in tables 69 and 70, so we can determine that the “Strength I max” is the governing load combination; the span length of this model was identified as 120 feet; and all the analysis below are based on it.

**Table 69**
**Max. and min. stresses of AASHTO BT-72 girders in group B with HS20-44 truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4120 | -1.0178 | 0.4830 | -1.0831 | 0.4371 | -1.0477 |
| | SYY | 0.7122 | -1.2685 | 0.6377 | -1.2853 | 0.7003 | -1.2866 |
| | SZZ | 1.0139 | -4.7205 | 0.9623 | -5.0858 | 1.0110 | -4.8779 |
| | SXY | 0.1922 | -0.2060 | 0.1748 | -0.1480 | 0.1783 | -0.1810 |
| | SXZ | 0.4369 | -0.4493 | 0.7831 | -0.7831 | 0.5418 | -0.5514 |
| | SYZ | 1.9690 | -1.9701 | 2.0365 | -2.0365 | 1.9428 | -1.9457 |
| Girder 2 | SXX | 0.4861 | -1.0170 | 0.2504 | -1.0681 | 0.3763 | -1.0436 |
| | SYY | 0.7132 | -1.1987 | 0.6311 | -1.2083 | 0.7000 | -1.2098 |
| | SZZ | 1.0607 | -4.7927 | 0.9632 | -5.0748 | 1.0474 | -4.9311 |
| | SXY | 0.2699 | -0.1594 | 0.1978 | -0.1498 | 0.2354 | -0.1586 |
| | SXZ | 0.4212 | -0.4343 | 0.7742 | -0.7742 | 0.5281 | -0.5382 |
| | SYZ | 2.0106 | -2.0145 | 2.0455 | -2.0455 | 1.9816 | -1.9845 |
| Girder 3 | SXX | 0.7848 | -1.0331 | 0.2502 | -1.0672 | 0.6032 | -1.0558 |
| | SYY | 0.7420 | -1.2244 | 0.6295 | -1.2056 | 0.7218 | -1.2234 |
| | SZZ | 1.1381 | -4.8691 | 0.9609 | -5.0723 | 1.1067 | -4.9895 |
| | SXY | 0.3935 | -0.2139 | 0.2060 | -0.1496 | 0.3389 | -0.1906 |
| | SXZ | 0.4190 | -0.4304 | 0.7744 | -0.7744 | 0.5266 | -0.5354 |
| | SYZ | 2.0461 | -2.0511 | 2.0456 | -2.0456 | 2.0074 | -2.0097 |
| Girder 4 | SXX | 0.8256 | -1.0448 | 0.2498 | -1.0663 | 0.6243 | -1.0646 |
| | SYY | 0.7668 | -1.2434 | 0.6259 | -1.2050 | 0.7399 | -1.2362 |
| | SZZ | 1.1398 | -4.9185 | 0.9561 | -5.0676 | 1.1068 | -5.0265 |
| | SXY | 0.3868 | -0.3578 | 0.2050 | -0.1492 | 0.3386 | -0.3099 |
| | SXZ | 0.4152 | -0.4207 | 0.7731 | -0.7731 | 0.5235 | -0.5277 |
| | SYZ | 2.0730 | -2.0785 | 2.0445 | -2.0445 | 2.0260 | -2.0264 |
| Girder 5 | SXX | 0.8254 | -1.0562 | 0.2494 | -1.0655 | 0.6294 | -1.0699 |
| | SYY | 0.7764 | -1.2562 | 0.6188 | -1.2054 | 0.7453 | -1.2444 |
| | SZZ | 1.1545 | -4.9568 | 0.9513 | -5.0619 | 1.1131 | -5.0493 |
| | SXY | 0.3009 | -0.3967 | 0.2020 | -0.1584 | 0.2736 | -0.3461 |
| | SXZ | 0.4098 | -0.4077 | 0.7716 | -0.7716 | 0.5191 | -0.5174 |
| | SYZ | 2.0922 | -2.0981 | 2.0439 | -2.0439 | 2.0367 | -2.0345 |
| Girder 6 | SXX | 0.6757 | -1.0650 | 0.2482 | -1.0623 | 0.4964 | -1.0706 |
| | SYY | 0.7745 | -1.2663 | 0.6093 | -1.2043 | 0.7411 | -1.2464 |
| | SZZ | 1.1749 | -4.9994 | 0.9522 | -5.0445 | 1.0604 | -5.0512 |
| | SXY | 0.1619 | -0.3522 | 0.1978 | -0.1656 | 0.1601 | -0.3146 |
| | SXZ | 0.4142 | -0.4053 | 0.7654 | -0.7654 | 0.5128 | -0.5073 |
| | SYZ | 2.1046 | -2.1104 | 2.0392 | -2.0392 | 2.0386 | -2.0347 |
| Girder 7 | SXX | 0.4356 | -1.0900 | 0.3025 | -1.2009 | 0.3507 | -1.1042 |
| | SYY | 0.7868 | -1.3793 | 0.6368 | -1.3694 | 0.7575 | -1.2706 |
| | SZZ | 1.1133 | -5.0334 | 1.0500 | -5.6950 | 1.0491 | -5.2248 |
| | SXY | 0.2370 | -0.2102 | 0.2423 | -0.1904 | 0.2365 | -0.2016 |
| | SXZ | 0.4401 | -0.4302 | 0.9717 | -0.9717 | 0.5542 | -0.5478 |
| | SYZ | 2.1135 | -2.1187 | 2.3270 | -2.3270 | 2.1063 | -2.1023 |

**Table 70**
**Max. and min. stresses of AASHTO BT-72 girders in group B with sugarcane truck load**

| | | Strength I Max | | Strength III Max | | Strength V Max | |
|---|---|---|---|---|---|---|---|
| | RESULT (Unit: Ksi) | MAX. | MIN. | MAX. | MIN. | MAX. | MIN. |
| Girder 1 | SXX | 0.4317 | -1.0577 | 0.4830 | -1.0831 | 0.4523 | -1.0785 |
| | SYY | 0.7551 | -1.3085 | 0.6377 | -1.2853 | 0.7334 | -1.3174 |
| | SZZ | 1.0627 | -4.9170 | 0.9623 | -5.0858 | 1.0485 | -5.0295 |
| | SXY | 0.1971 | -0.2075 | 0.1748 | -0.1480 | 0.1820 | -0.1822 |
| | SXZ | 0.4667 | -0.4902 | 0.7831 | -0.7831 | 0.5648 | -0.5829 |
| | SYZ | 2.0309 | -2.0316 | 2.0365 | -2.0365 | 1.9945 | -2.0002 |
| Girder 2 | SXX | 0.6467 | -1.0687 | 0.2504 | -1.0681 | 0.5002 | -1.0835 |
| | SYY | 0.7646 | -1.2556 | 0.6311 | -1.2083 | 0.7396 | -1.2481 |
| | SZZ | 1.1127 | -5.0486 | 0.9632 | -5.0748 | 1.0875 | -5.1285 |
| | SXY | 0.3540 | -0.1688 | 0.1978 | -0.1498 | 0.3004 | -0.1659 |
| | SXZ | 0.4549 | -0.4838 | 0.7742 | -0.7742 | 0.5541 | -0.5764 |
| | SYZ | 2.0993 | -2.1073 | 2.0455 | -2.0455 | 2.0511 | -2.0581 |
| Girder 3 | SXX | 1.0823 | -1.0974 | 0.2502 | -1.0672 | 0.8327 | -1.1055 |
| | SYY | 0.8096 | -1.2970 | 0.6295 | -1.2056 | 0.7740 | -1.2734 |
| | SZZ | 1.1679 | -5.1845 | 0.9609 | -5.0723 | 1.1273 | -5.2328 |
| | SXY | 0.5662 | -0.2905 | 0.2060 | -0.1496 | 0.4722 | -0.2497 |
| | SXZ | 0.4557 | -0.4838 | 0.7744 | -0.7744 | 0.5549 | -0.5766 |
| | SYZ | 2.1592 | -2.1708 | 2.0456 | -2.0456 | 2.0958 | -2.1028 |
| Girder 4 | SXX | 1.1317 | -1.1162 | 0.2498 | -1.0663 | 0.8604 | -1.1197 |
| | SYY | 0.8479 | -1.3284 | 0.6259 | -1.2050 | 0.8025 | -1.2956 |
| | SZZ | 1.1652 | -5.2621 | 0.9561 | -5.0676 | 1.1259 | -5.2915 |
| | SXY | 0.5498 | -0.5253 | 0.2050 | -0.1492 | 0.4643 | -0.4391 |
| | SXZ | 0.4510 | -0.4650 | 0.7731 | -0.7731 | 0.5511 | -0.5619 |
| | SYZ | 2.2056 | -2.2203 | 2.0445 | -2.0445 | 2.1286 | -2.1305 |
| Girder 5 | SXX | 0.8254 | -1.0562 | 0.2494 | -1.0655 | 0.8656 | -1.1279 |
| | SYY | 0.7764 | -1.2562 | 0.6188 | -1.2054 | 0.8120 | -1.3084 |
| | SZZ | 1.1545 | -4.9568 | 0.9513 | -5.0619 | 1.1268 | -5.3263 |
| | SXY | 0.3009 | -0.3967 | 0.2020 | -0.1584 | 0.3894 | -0.4742 |
| | SXZ | 0.4098 | -0.4077 | 0.7716 | -0.7716 | 0.5435 | -0.5386 |
| | SYZ | 2.0922 | -2.0981 | 2.0439 | -2.0439 | 2.1451 | -2.1395 |
| Girder 6 | SXX | 0.9396 | -1.1455 | 0.2482 | -1.0623 | 0.7000 | -1.1278 |
| | SYY | 0.8584 | -1.3586 | 0.6093 | -1.2043 | 0.8058 | -1.3103 |
| | SZZ | 1.2193 | -5.3824 | 0.9522 | -5.0445 | 1.0971 | -5.3239 |
| | SXY | 0.1866 | -0.5120 | 0.1978 | -0.1656 | 0.1851 | -0.4380 |
| | SXZ | 0.4585 | -0.4330 | 0.7654 | -0.7654 | 0.5343 | -0.5176 |
| | SYZ | 2.2485 | -2.2618 | 2.0392 | -2.0392 | 2.1459 | -2.1354 |
| Girder 7 | SXX | 0.4692 | -1.1716 | 0.3025 | -1.2009 | 0.3765 | -1.1589 |
| | SYY | 0.8653 | -1.4786 | 0.6368 | -1.3694 | 0.8181 | -1.3309 |
| | SZZ | 1.1927 | -5.4149 | 1.0500 | -5.6950 | 1.1045 | -5.4863 |
| | SXY | 0.2544 | -0.2642 | 0.2423 | -0.1904 | 0.2499 | -0.2486 |
| | SXZ | 0.4882 | -0.4593 | 0.9717 | -0.9717 | 0.5735 | -0.5532 |
| | SYZ | 2.2542 | -2.2634 | 2.3270 | -2.3270 | 2.2088 | -2.1975 |

The study used the stress Szz (the flexural stress in the horizontal direction) under load combination “Strength I max” because it was the critical factor. The stresses were evaluated at both top and bottom surface of the girder. We looked for the maximum compressive stress at top surface, while the maximum tensile stress at bottom surface.

For the truck load HS20-44, girder 5 and 6 were determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.59 ksi was occurred at girder 5 at 60 feet along the bridge, while at most other locations, the compressive stresses in girder 6 were slightly larger than them in girder 5. The maximum tensile stress at the bottom of the girder, 1.18 ksi was occurred at girder 6 at 61 feet along the bridge, while at most other locations, there wasn’t much difference between girder 5 and 6.

For the Sugarcane truck load, girder 6 was determined to be the critical girders. The maximum compressive stress at the top of the girder, 2.64 ksi was occurred at 54 feet along the bridge. The maximum tensile stress at the bottom of the girder, 1.22 ksi was occurred at 55 feet along the bridge. The results of stresses was plotted and shown in Figure 14 and Figure 15.

**Short term deflection of AASHTO type IV girder bridges in group B.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to be the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 5 was the critical girder, which had 2.63 inch maximum displacement with truck load HS20-44, and 2.81 inch maximum displacement with truck load Sugarcane.

**Short term deflection of AASHTO type V girder bridges in group B.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to be the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 5 was the critical girder, which had 2.20 inch maximum displacement with truck load HS20-44, and 2.33 inch maximum displacement with truck load Sugarcane.

**Short term deflection of AASHTO type VI girder bridges in group B.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination

Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to be the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 5 was the critical girder, which had 2.22 inch maximum displacement with truck load HS20-44, and 2.34 inch maximum displacement with truck load Sugarcane.

**Short term deflection of AASHTO BT-54 girder bridges in group B.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination, and girder 5 was the critical girder, which had 2.29 inch maximum displacement with truck load HS20-44, and 2.45 inch maximum displacement with truck load Sugarcane.

**Short term deflection of AASHTO BT-63 girder bridges in group B.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination; and girder 5 or 6 were the critical girder, which had 2.05 inch maximum displacement with truck load HS20-44 at girder 5, and 2.19 inch maximum displacement with truck load Sugarcane at girder 6.

**Short term deflection of AASHTO BT-72 girder bridges in group B.** The deflection of the bridge girder was a serviceability criterion and needed to be investigated carefully. The phrase “short term deflection” referred to the girder deflection under load combination Strength I max, Strength III max, and Strength V max. By the similar analysis procedure to the stress analysis, the researcher determined the load combination Strength I max was the governing load combination; and girder 5 or 6 were the critical girder, which had 2.54 inch maximum displacement with truck load HS20-44 at girder 5, and 2.71 inch maximum displacement with truck load Sugarcane at girder 6.

**Long term stress performances of AASHTO type IV girder bridges in group B.** The long term effects of heavy trucks on bridges and bridge decks play an important role in the bridge life evaluation. The load combination “Fatigue” was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 5 and 6. The maximum compressive stress, 0.30 ksi, was occurred at girder 5, at distance 45 feet, while in the distance range from 46 to 67 feet, girder 6 was the critical girders. The maximum tensile stress, 0.13 ksi, was occurred at girder 5, at distance 45 feet, while in the distance range from 48 to 68 feet, girder 6 was the critical girders.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.48 ksi was occurred at distance 60 feet at girder 6, while the maximum tensile stress 0.20 ksi was occurred at distance 41 feet at girder 5.

**Long term stress performances of AASHTO type V girder bridges in group B.** Similar to the analysis of AASHTO type IV girder, the load combination "Fatigue" was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 5 and 6. The maximum compressive stress, 0.21 ksi, was occurred at girder 5, at distance 51 feet, while in the distance range from 53 to 74 feet, girder 6 was the critical girders. The maximum tensile stress, 0.10 ksi, was occurred at girder 6, at distance 51 feet, while at most other locations, there wasn't too much difference between girder 5 and 6.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.30 ksi was occurred at distance 71 feet at girder 6, while the maximum tensile stress 0.16 ksi was occurred at distance 70 feet at girder 6.

**Long term stress performances of AASHTO type VI girder bridges in group B.** Similar to the analysis of AASHTO type IV and V girder, the load combination "Fatigue" was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 5 and 6. The maximum compressive stress, 0.20 ksi, was occurred at girder 5, at distance 57 feet; while at most other location, girder 6 was the critical girders. The maximum tensile stress, 0.09 ksi, was occurred at girder 6, at distance 79 feet; while at most other locations, there wasn't too much difference between girder 5 and 6.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.28 ksi was occurred at distance 80 feet at girder 6, while the maximum tensile stress 0.15 ksi was occurred at distance 79 feet at girder 6.

**Long term stress performances of AASHTO BT-54 girder bridges in group B.** The long term effects of heavy trucks on bridges and bridge decks play an important role in the bridge life evaluation. The load combination “Fatigue” was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 5, 6 and 7. The maximum compressive stress, 0.27 ksi, was occurred at girder 5, at distance 45 feet; while in the distance range from 50 to 58 feet, girder 7 was the critical girders. The maximum tensile stress, 0.146 ksi, was occurred at girder 7, at distance 59 feet; while at most other locations, girder 5 and 6 were the critical girders.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.43 ksi was occurred at distance 59 feet at girder 6, while the maximum tensile stress 0.22 ksi was occurred at distance 59 feet at girder 7.

**Long term stress performances of AASHTO BT-63 girder bridges in group A.** Similar to the analysis of AASHTO I-Beams and Bulb-Tee 54 girders, the load combination “Fatigue” was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 5, 6 and 7. The maximum compressive stress, 0.24 ksi, was occurred at girder 6, at distance 42 and 65 feet; while in the distance range from 50 to 64 feet, girder 7 was the critical girders. The maximum tensile stress, 0.13 ksi, was occurred at girder 6, at distance 40 feet, while in the distance range from 48 to 64 feet, girder 7 was the critical girders.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.40 ksi was occurred at distance 65 feet at girder 6, while the maximum tensile stress 0.21 ksi was occurred at distance 65 feet at girder 6.

**Long term stress performances of AASHTO BT-72 girder bridges in group B.** Similar to the analysis of AASHTO I-Beams and other Bulb-Tee type girders, the load combination “Fatigue” was used to evaluate the long term stress performance of bridge girders, based on AASHTO chapter 3. Similar to the short term stress performance discussed before, the critical girders of bridge with truck load HS20-44 were determined as girder 5, 6 and 7. The maximum compressive stress, 0.25 ksi, was occurred at girder 5, at distance 57 feet, while in the distance range from 58 to 87 feet, girder 6 and 7 were the critical girders. The maximum tensile stress, 0.12 ksi, was occurred at girder 6, at distance 79 feet, while at most other locations, there wasn’t too much difference among girder 5, 6 and 7.

**The flexural stress along the bridge with Sugarcane truck load.** At top surface of the girders, the maximum compressive stress 0.39 ksi was occurred at distance 80 feet at girder 6, while the maximum tensile stress 0.21 ksi was occurred at distance 80 feet at girder 6.

**Long term deflection of AASHTO type IV girder bridges in group B.** The phrase “long term deflection” referred to the deflection calculated with load combination “Fatigue”. By the same method used in evaluating the short term deflection, girder 5 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.21 inch and 0.35 inch under HS20-44 and Sugarcane truck, respectively.

**Long term deflection of AASHTO type V girder bridges in group B.** By the same method used in evaluating the long term deflection of AASHTO type IV girder, girder 5 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.14 inch and 0.23 inch under HS20-44 and Sugarcane truck, respectively.

**Long term deflection of AASHTO type VI girder bridges in group B.** By the same method used in evaluating the long term deflection of AASHTO type IV and V girders, girder 5 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.13 inch and 0.22 inch under HS20-44 and Sugarcane truck, respectively.

**Long term deflection of AASHTO Bulb-Tee 54 girder bridges in group B.** The phrase “long term deflection” referred to the deflection calculated with load combination “Fatigue”. By the same method used in evaluating the short term deflection, girder 5 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.19 inch and 0.31 inch under HS20-44 and Sugarcane truck, respectively.

**Long term deflection of AASHTO Bulb-Tee 63 girder bridges in group B.** By the same method used in evaluating the long term deflection of AASHTO I-Beams and Bulb-Tee 54 girders, girder 6 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.16 inch and 0.27 inch under HS20-44 and Sugarcane truck, respectively.

**Long term deflection of AASHTO Bulb-Tee 72 girder bridges in group B.** By the same method used in evaluating the long term deflection of AASHTO type IV and Bulb-Tee girders, girder 6 was determined as the critical girder of the bridge with both truck load HS20-44 and Sugarcane. The maximum displacements were 0.18 inch and 0.29 inch under HS20-44 and Sugarcane truck, respectively, as shown in Figure 43 and Figure 44.

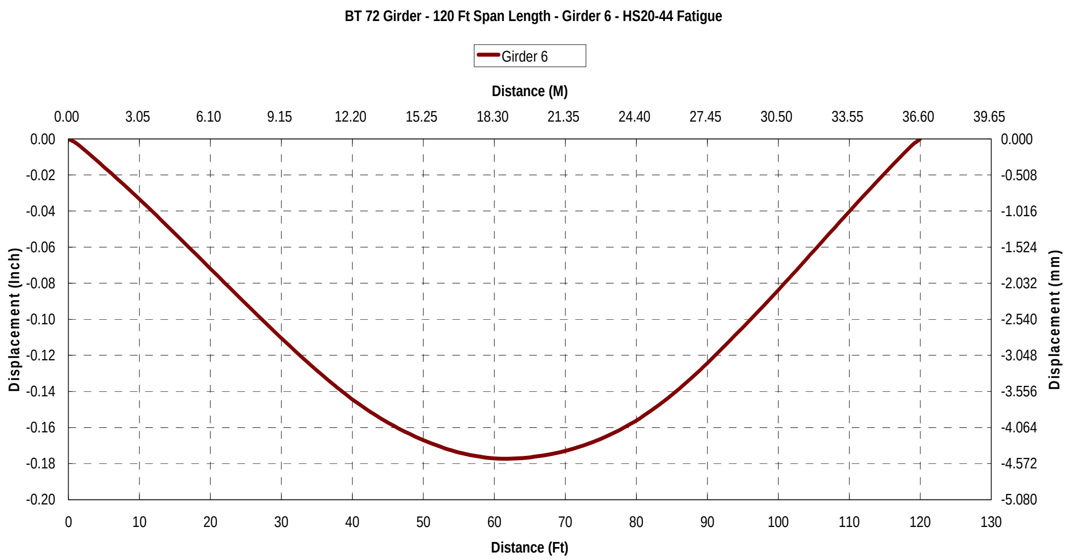



**Figure 43**
**Displacement in girder 6 along the bridge - AASHTO BT-72 girder in group B - truck load HS20-44 fatigue**

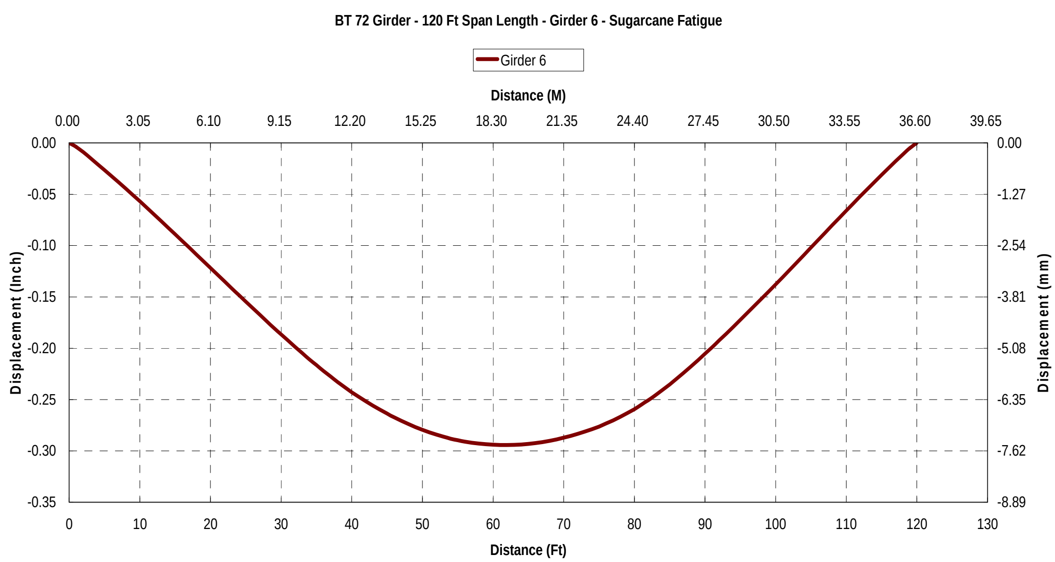



**Figure 44**
**Displacement in girder 6 along the bridge - AASHTO BT-72 girder in group B - truck load sugarcane fatigue**

## Bridge Deck Evaluation

This subtask focused on the strength and serviceability of bridge decks under the impact of the heavy loads from trucks. Finite element analysis was used for a typical deck and girder system to determine the effects of the trucks on the stresses in the transverse and longitudinal directions.

All bridges considered for this study had concrete decks. According to the LADOTD Bridge Manual, concrete bridge decks are designed as a continuous span over the girders. The bridge deck analyses for this study were performed using finite element models and GTSTRUDL software. The finite element models for typical bridge decks in group A were generated with a typical 30-ft. bridge-deck width and 8-inch thickness supported by 5 girders; while in group B the models were generated with a typical 30-ft. bridge-deck width and 8-inch thickness supported by 7 girders. The design load HS20-44 and the loads from Sugarcane truck configuration were applied to the deck. Load combination “Strength I max”, “Strength III max” and “Strength V max” were performed for the short term effect on bridge deck, the “Fatigue” load combination, as presented in AASHTO LRFD, was performed for long term effect on the deck.

The finite element model used for bridge decks in this study simulated the behavior of continuous span bridges. The girders were modeled using Type-IPSL tridimensional elements available in GTSTRUDL. Type-SBCR plate elements were used for the bridge deck. Prismatic space truss members were used to model end diaphragms and the connection between the deck plate elements and the girder elements, as discussed before.

**Bridge deck evaluation of models in group A.** Truck loads for HS20-44 and Sugarcane were applied at critical locations for maximum moment in the bridge deck to determine the corresponding stresses. The maximum value of longitudinal, transverse, and shear stresses in the bridge deck were obtained and then grouped as the tensile stress and compressive stress. The following tables summarize the results of the maximum stress values at the top and bottom surfaces of the bridge deck under both HS20-44 and 3S3 truck loads for all load combinations in group A, while tables 71 through 74 were the results of bridges with AASHTO Type IV girders; tables 75 through 78 were the results of bridges with AASHTO Type V girders; tables 79 through 82 were the results of bridges with AASHTO Type VI girders; tables 83 through 86 were the results of bridges with Bulb-Tee 54 girders; tables 87 through 90 were the results of bridges with Bulb-Tee 63 girders; and tables 91 through 94 were the results of bridges with Bulb-Tee 72 girders.

For short term effects, the load combination “Strength I max” was the governing load combination, while the "Fatigue" was the load combination considered for long term effects.

**Table 71**
**Stress at top surface of AASHTO type IV girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.175 | 0.537 | 0.100 | -0.820 | -0.913 | -0.111 |
| Strength III Max | 0.119 | 0.170 | 0.069 | -0.242 | -0.192 | -0.069 |
| Strength V Max | 0.162 | 0.431 | 0.088 | -0.681 | -0.739 | -0.097 |
| Fatigue | 0.016 | 0.215 | 0.041 | -0.300 | -0.394 | -0.033 |

**Table 72**
**The stress at top surface of AASHTO type IV girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.193 | 0.476 | 0.129 | -0.671 | -1.048 | -0.154 |
| Strength III Max | 0.119 | 0.170 | 0.069 | -0.242 | -0.192 | -0.069 |
| Strength V Max | 0.176 | 0.386 | 0.116 | -0.566 | -0.842 | -0.130 |
| Fatigue | 0.027 | 0.203 | 0.052 | -0.231 | -0.475 | -0.050 |

**Table 73**
**The stress at bottom surface of AASHTO type IV girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.820 | 0.913 | 0.111 | -0.175 | -0.537 | -0.100 |
| Strength III Max | 0.242 | 0.192 | 0.069 | -0.119 | -0.170 | -0.069 |
| Strength V Max | 0.681 | 0.739 | 0.097 | -0.162 | -0.431 | -0.088 |
| Fatigue | 0.300 | 0.394 | 0.033 | -0.016 | -0.215 | -0.041 |

**Table 74**
**Stress at bottom surface of AASHTO type IV girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.671 | 1.048 | 0.154 | -0.193 | -0.476 | -0.129 |
| Strength III Max | 0.242 | 0.192 | 0.069 | -0.119 | -0.170 | -0.069 |
| Strength V Max | 0.566 | 0.842 | 0.130 | -0.176 | -0.386 | -0.116 |
| Fatigue | 0.231 | 0.475 | 0.050 | -0.027 | -0.203 | -0.052 |

**Table 75**
**Stress at top surface of AASHTO type V girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.106 | 0.163 | 0.064 | -0.646 | -0.771 | -0.069 |
| Strength III Max | 0.080 | 0.092 | 0.047 | -0.171 | -0.108 | -0.047 |
| Strength V Max | 0.100 | 0.125 | 0.057 | -0.534 | -0.613 | -0.057 |
| Fatigue | 0.040 | 0.088 | 0.036 | -0.248 | -0.368 | -0.034 |

**Table 76**
**Stress at top surface of AASHTO type V girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.117 | 0.202 | 0.079 | -0.475 | -0.729 | -0.083 |
| Strength III Max | 0.080 | 0.092 | 0.047 | -0.171 | -0.108 | -0.047 |
| Strength V Max | 0.108 | 0.155 | 0.073 | -0.402 | -0.580 | -0.070 |
| Fatigue | 0.027 | 0.113 | 0.037 | -0.163 | -0.351 | -0.033 |

**Table 77**
**Stress at bottom surface of AASHTO type V girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.646 | 0.771 | 0.069 | -0.106 | -0.163 | -0.064 |
| Strength III Max | 0.171 | 0.108 | 0.047 | -0.080 | -0.092 | -0.047 |
| Strength V Max | 0.534 | 0.613 | 0.057 | -0.100 | -0.125 | -0.057 |
| Fatigue | 0.248 | 0.368 | 0.034 | -0.040 | -0.088 | -0.036 |

**Table 78**
**Stress at bottom surface of AASHTO type V girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.475 | 0.729 | 0.083 | -0.117 | -0.202 | -0.079 |
| Strength III Max | 0.171 | 0.108 | 0.047 | -0.080 | -0.092 | -0.047 |
| Strength V Max | 0.402 | 0.580 | 0.070 | -0.108 | -0.155 | -0.073 |
| Fatigue | 0.163 | 0.351 | 0.033 | -0.027 | -0.113 | -0.037 |

**Table 79**
**Stress at top surface of AASHTO type VI girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.092 | 0.184 | 0.069 | -0.632 | -0.797 | -0.069 |
| Strength III Max | 0.078 | 0.105 | 0.046 | -0.151 | -0.101 | -0.046 |
| Strength V Max | 0.088 | 0.143 | 0.054 | -0.522 | -0.634 | -0.053 |
| Fatigue | 0.043 | 0.076 | 0.033 | -0.249 | -0.364 | -0.035 |

**Table 80**
**Stress at top surface of AASHTO type VI girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.100 | 0.203 | 0.063 | -0.450 | -0.727 | -0.071 |
| Strength III Max | 0.078 | 0.105 | 0.046 | -0.151 | -0.101 | -0.046 |
| Strength V Max | 0.094 | 0.157 | 0.060 | -0.380 | -0.579 | -0.060 |
| Fatigue | 0.029 | 0.111 | 0.032 | -0.158 | -0.345 | -0.037 |

**Table 81**
**Stress at bottom surface of AASHTO type VI girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.632 | 0.797 | 0.069 | -0.092 | -0.184 | -0.069 |
| Strength III Max | 0.151 | 0.101 | 0.046 | -0.078 | -0.105 | -0.046 |
| Strength V Max | 0.522 | 0.634 | 0.053 | -0.088 | -0.143 | -0.054 |
| Fatigue | 0.249 | 0.364 | 0.035 | -0.043 | -0.076 | -0.033 |

**Table 82**
**Stress at bottom surface of AASHTO type VI girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.450 | 0.727 | 0.071 | -0.100 | -0.203 | -0.063 |
| Strength III Max | 0.151 | 0.101 | 0.046 | -0.078 | -0.105 | -0.046 |
| Strength V Max | 0.380 | 0.579 | 0.060 | -0.094 | -0.157 | -0.060 |
| Fatigue | 0.158 | 0.345 | 0.037 | -0.029 | -0.111 | -0.032 |

**Table 83**
**Stress at top surface of AASHTO Bulb-Tee 54 girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.118 | 0.174 | 0.087 | -0.760 | -0.862 | -0.097 |
| Strength III Max | 0.082 | 0.098 | 0.059 | -0.218 | -0.133 | -0.059 |
| Strength V Max | 0.110 | 0.144 | 0.081 | -0.632 | -0.688 | -0.083 |
| Fatigue | 0.025 | 0.066 | 0.044 | -0.286 | -0.392 | -0.040 |

**Table 84**
**Stress at top surface of AASHTO Bulb-Tee 54 girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.133 | 0.204 | 0.122 | -0.578 | -0.849 | -0.127 |
| Strength III Max | 0.082 | 0.098 | 0.059 | -0.218 | -0.133 | -0.059 |
| Strength V Max | 0.121 | 0.181 | 0.109 | -0.488 | -0.676 | -0.107 |
| Fatigue | 0.022 | 0.079 | 0.050 | -0.202 | -0.404 | -0.048 |

**Table 85**
**Stress at bottom surface of AASHTO Bulb-Tee 54 girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.760 | 0.862 | 0.097 | -0.118 | -0.174 | -0.087 |
| Strength III Max | 0.218 | 0.133 | 0.059 | -0.082 | -0.098 | -0.059 |
| Strength V Max | 0.632 | 0.688 | 0.083 | -0.110 | -0.144 | -0.081 |
| Fatigue | 0.286 | 0.392 | 0.040 | -0.025 | -0.066 | -0.044 |

**Table 86**
**Stress at bottom surface of AASHTO Bulb-Tee 54 girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.578 | 0.849 | 0.127 | -0.133 | -0.204 | -0.122 |
| Strength III Max | 0.218 | 0.133 | 0.059 | -0.082 | -0.098 | -0.059 |
| Strength V Max | 0.488 | 0.676 | 0.107 | -0.121 | -0.181 | -0.109 |
| Fatigue | 0.202 | 0.404 | 0.048 | -0.022 | -0.079 | -0.050 |

**Table 87**
**Stress at top surface of AASHTO Bulb-Tee 63 girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.092 | 0.190 | 0.078 | -0.674 | -0.818 | -0.082 |
| Strength III Max | 0.066 | 0.099 | 0.057 | -0.167 | -0.119 | -0.167 |
| Strength V Max | 0.085 | 0.157 | 0.072 | -0.553 | -0.652 | -0.070 |
| Fatigue | 0.033 | 0.072 | 0.043 | -0.275 | -0.380 | -0.037 |

**Table 88**
**Stress at top surface of AASHTO Bulb-Tee 63 girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.103 | 0.204 | 0.104 | -0.490 | -0.794 | -0.104 |
| Strength III Max | 0.066 | 0.099 | 0.057 | -0.167 | -0.119 | -0.167 |
| Strength V Max | 0.094 | 0.167 | 0.094 | -0.409 | -0.631 | -0.087 |
| Fatigue | 0.016 | 0.088 | 0.048 | -0.185 | -0.382 | -0.043 |

**Table 89**
**Stress at bottom surface of AASHTO Bulb-Tee 63 girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.674 | 0.818 | 0.082 | -0.092 | -0.190 | -0.078 |
| Strength III Max | 0.167 | 0.119 | 0.057 | -0.066 | -0.099 | -0.057 |
| Strength V Max | 0.553 | 0.652 | 0.070 | -0.085 | -0.157 | -0.072 |
| Fatigue | 0.275 | 0.380 | 0.037 | -0.033 | -0.072 | -0.043 |

**Table 90**
**Stress at bottom surface of AASHTO Bulb-Tee 63 girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.490 | 0.794 | 0.104 | -0.103 | -0.204 | -0.104 |
| Strength III Max | 0.167 | 0.119 | 0.057 | -0.066 | -0.099 | -0.057 |
| Strength V Max | 0.409 | 0.631 | 0.087 | -0.094 | -0.167 | -0.094 |
| Fatigue | 0.185 | 0.382 | 0.043 | -0.016 | -0.088 | -0.048 |

**Table 91**
**Stress at top surface of AASHTO Bulb-Tee 72 girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.076 | 0.196 | 0.077 | -0.618 | -0.785 | -0.075 |
| Strength III Max | 0.057 | 0.104 | 0.056 | -0.134 | -0.108 | -0.056 |
| Strength V Max | 0.071 | 0.163 | 0.066 | -0.503 | -0.624 | -0.061 |
| Fatigue | 0.039 | 0.076 | 0.042 | -0.267 | -0.370 | -0.036 |

**Table 92**
**Stress at top surface of AASHTO Bulb-Tee 72 girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.084 | 0.214 | 0.094 | -0.434 | -0.752 | -0.087 |
| Strength III Max | 0.057 | 0.104 | 0.056 | -0.134 | -0.108 | -0.056 |
| Strength V Max | 0.077 | 0.177 | 0.084 | -0.358 | -0.599 | -0.073 |
| Fatigue | 0.023 | 0.095 | 0.046 | -0.174 | -0.365 | -0.039 |

**Table 93**
**Stress at bottom surface of AASHTO Bulb-Tee 72 girder bridge deck in group A for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.618 | 0.785 | 0.075 | -0.076 | -0.196 | -0.077 |
| Strength III Max | 0.134 | 0.108 | 0.056 | -0.057 | -0.104 | -0.056 |
| Strength V Max | 0.503 | 0.624 | 0.061 | -0.071 | -0.163 | -0.066 |
| Fatigue | 0.267 | 0.370 | 0.036 | -0.039 | -0.076 | -0.042 |

**Table 94**
**Stress at bottom surface of AASHTO Bulb-Tee 72 girder bridge deck in group A for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.434 | 0.752 | 0.087 | -0.084 | -0.214 | -0.094 |
| Strength III Max | 0.134 | 0.108 | 0.056 | -0.057 | -0.104 | -0.056 |
| Strength V Max | 0.358 | 0.599 | 0.073 | -0.077 | -0.177 | -0.084 |
| Fatigue | 0.174 | 0.365 | 0.039 | -0.023 | -0.095 | -0.046 |

**Bridge deck evaluation of models in group B.** Truck loads for HS20-44 and Sugarcane were applied at critical locations for maximum moment in the bridge deck to determine the corresponding stresses. The maximum value of longitudinal, transverse, and shear stresses in the bridge deck were obtained and then grouped as the tensile stress and compressive stress. The following tables summarized the results of the maximum stress values at the top and bottom surfaces of the bridge deck under both HS20-44 and 3S3 truck loads for all load combinations in group B, while tables 95 through 98 were the results of bridges with AASHTO Type IV girders; tables 99 through 102 were the results of bridges with AASHTO Type V girders; tables 103 through 106 were the results of bridges with AASHTO Type VI girders;  tables 107 through 110 were the results of bridges with Bulb-Tee 54 girders; tables 111 through 114 were the results of bridges with Bulb-Tee 63 girders; and tables 115 through 118 were the results of bridges with Bulb-Tee 72 girders. Obviously we can see, for short term effects, the load combination “Strength I max” was the governing load combination, while the “Fatigue” was the load combination considered for long term effects.

**Table 95**
**Stress at top surface of AASHTO type IV girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.158 | 0.175 | 0.066 | -0.538 | -0.593 | -0.076 |
| Strength III Max | 0.121 | 0.116 | 0.051 | -0.230 | -0.109 | -0.051 |
| Strength V Max | 0.150 | 0.165 | 0.057 | -0.467 | -0.479 | -0.064 |
| Fatigue | 0.013 | 0.065 | 0.019 | -0.143 | -0.237 | -0.026 |

**Table 96**
**The stress at top surface of AASHTO type IV girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load Combination | Max Value of Stress (Unit: Ksi) | | | | | |
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.172 | 0.229 | 0.083 | -0.472 | -0.654 | -0.098 |
| Strength III Max | 0.121 | 0.116 | 0.051 | -0.230 | -0.109 | -0.051 |
| Strength V Max | 0.159 | 0.207 | 0.070 | -0.415 | -0.529 | -0.081 |
| Fatigue | 0.021 | 0.110 | 0.029 | -0.119 | -0.287 | -0.038 |

**Table 97**
**Stress at bottom surface of AASHTO type IV girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load Combination | Max Value of Stress (Unit: Ksi) | | | | | |
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.538 | 0.593 | 0.076 | -0.158 | -0.175 | -0.066 |
| Strength III Max | 0.230 | 0.109 | 0.051 | -0.121 | -0.116 | -0.051 |
| Strength V Max | 0.467 | 0.479 | 0.064 | -0.150 | -0.165 | -0.057 |
| Fatigue | 0.143 | 0.237 | 0.026 | -0.013 | -0.065 | -0.019 |

**Table 98**
**Stress at bottom surface of AASHTO type IV girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load Combination | Max Value of Stress (Unit: Ksi) | | | | | |
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.472 | 0.654 | 0.098 | -0.172 | -0.229 | -0.083 |
| Strength III Max | 0.230 | 0.109 | 0.051 | -0.121 | -0.116 | -0.051 |
| Strength V Max | 0.415 | 0.529 | 0.081 | -0.159 | -0.207 | -0.070 |
| Fatigue | 0.119 | 0.287 | 0.038 | -0.021 | -0.110 | -0.029 |

**Table 99**
**Stress at top surface of AASHTO type V girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load Combination | Max Value of Stress (Unit: Ksi) | | | | | |
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.103 | 0.066 | 0.039 | -0.462 | -0.485 | -0.045 |
| Strength III Max | 0.085 | 0.047 | 0.030 | -0.159 | -0.035 | -0.030 |
| Strength V Max | 0.098 | 0.063 | 0.033 | -0.393 | -0.380 | -0.037 |
| Fatigue | 0.026 | 0.067 | 0.021 | -0.153 | -0.237 | -0.025 |

**Table 100**
**Stress at top surface of AASHTO type V girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.110 | 0.091 | 0.048 | -0.354 | -0.449 | -0.059 |
| Strength III Max | 0.085 | 0.047 | 0.030 | -0.159 | -0.035 | -0.030 |
| Strength V Max | 0.103 | 0.083 | 0.040 | -0.308 | -0.353 | -0.048 |
| Fatigue | 0.012 | 0.041 | 0.023 | -0.100 | -0.231 | -0.025 |

**Table 101**
**Stress at bottom surface of AASHTO type V girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.462 | 0.485 | 0.045 | -0.103 | -0.066 | -0.039 |
| Strength III Max | 0.159 | 0.035 | 0.030 | -0.085 | -0.047 | -0.030 |
| Strength V Max | 0.393 | 0.380 | 0.037 | -0.098 | -0.063 | -0.033 |
| Fatigue | 0.153 | 0.237 | 0.025 | -0.026 | -0.067 | -0.021 |

**Table 102**
**Stress at bottom surface of AASHTO type V girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.354 | 0.449 | 0.059 | -0.110 | -0.091 | -0.048 |
| Strength III Max | 0.159 | 0.035 | 0.030 | -0.085 | -0.047 | -0.030 |
| Strength V Max | 0.308 | 0.353 | 0.048 | -0.103 | -0.083 | -0.040 |
| Fatigue | 0.100 | 0.231 | 0.025 | -0.012 | -0.041 | -0.023 |

**Table 103**
**Stress at top surface of AASHTO type VI girder Bridge Deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.094 | 0.075 | 0.036 | -0.432 | -0.475 | -0.043 |
| Strength III Max | 0.082 | 0.057 | 0.029 | -0.140 | -0.043 | -0.029 |
| Strength V Max | 0.090 | 0.062 | 0.028 | -0.365 | -0.372 | -0.033 |
| Fatigue | 0.033 | 0.069 | 0.020 | -0.149 | -0.235 | -0.023 |

**Table 104**
**Stress at top surface of AASHTO type VI girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.100 | 0.091 | 0.041 | -0.325 | -0.445 | -0.051 |
| Strength III Max | 0.082 | 0.057 | 0.029 | -0.140 | -0.043 | -0.029 |
| Strength V Max | 0.095 | 0.079 | 0.034 | -0.282 | -0.349 | -0.042 |
| Fatigue | 0.010 | 0.049 | 0.022 | -0.095 | -0.230 | -0.023 |

**Table 105**
**Stress at bottom surface of AASHTO type VI girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.432 | 0.475 | 0.043 | -0.094 | -0.075 | -0.036 |
| Strength III Max | 0.140 | 0.043 | 0.029 | -0.082 | -0.057 | -0.029 |
| Strength V Max | 0.365 | 0.372 | 0.033 | -0.090 | -0.062 | -0.028 |
| Fatigue | 0.149 | 0.235 | 0.023 | -0.033 | -0.069 | -0.020 |

**Table 106**
**Stress at bottom surface of AASHTO type VI girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.325 | 0.445 | 0.051 | -0.100 | -0.091 | -0.041 |
| Strength III Max | 0.140 | 0.043 | 0.029 | -0.082 | -0.057 | -0.029 |
| Strength V Max | 0.282 | 0.349 | 0.042 | -0.095 | -0.079 | -0.034 |
| Fatigue | 0.095 | 0.230 | 0.023 | -0.010 | -0.049 | -0.022 |

**Table 107**
**Stress at top surface of AASHTO Bulb-Tee 54 girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.112 | 0.088 | 0.052 | -0.595 | -0.559 | -0.062 |
| Strength III Max | 0.085 | 0.049 | 0.037 | -0.192 | -0.036 | -0.037 |
| Strength V Max | 0.105 | 0.080 | 0.044 | -0.502 | -0.440 | -0.051 |
| Fatigue | 0.030 | 0.066 | 0.026 | -0.198 | -0.272 | -0.033 |

**Table 108**
**Stress at top surface of AASHTO Bulb-Tee 54 girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.123 | 0.124 | 0.068 | -0.471 | -0.520 | -0.085 |
| Strength III Max | 0.085 | 0.049 | 0.037 | -0.192 | -0.036 | -0.037 |
| Strength V Max | 0.113 | 0.108 | 0.056 | -0.406 | -0.409 | -0.069 |
| Fatigue | 0.017 | 0.048 | 0.030 | -0.143 | -0.266 | -0.037 |

**Table 109**
**Stress at bottom surface of AASHTO Bulb-Tee 54 girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.595 | 0.559 | 0.062 | -0.112 | -0.088 | -0.052 |
| Strength III Max | 0.192 | 0.036 | 0.037 | -0.085 | -0.049 | -0.037 |
| Strength V Max | 0.502 | 0.440 | 0.051 | -0.105 | -0.080 | -0.044 |
| Fatigue | 0.198 | 0.272 | 0.033 | -0.030 | -0.066 | -0.026 |

**Table 110**
**Stress at bottom surface of AASHTO Bulb-Tee 54 girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.471 | 0.520 | 0.085 | -0.123 | -0.124 | -0.068 |
| Strength III Max | 0.192 | 0.036 | 0.037 | -0.085 | -0.049 | -0.037 |
| Strength V Max | 0.406 | 0.409 | 0.069 | -0.113 | -0.108 | -0.056 |
| Fatigue | 0.143 | 0.266 | 0.037 | -0.017 | -0.048 | -0.030 |

**Table 111**
**Stress at top surface of AASHTO Bulb-Tee 63 girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.091 | 0.075 | 0.050 | -0.522 | -0.534 | -0.051 |
| Strength III Max | 0.077 | 0.056 | 0.038 | -0.156 | -0.036 | -0.038 |
| Strength V Max | 0.087 | 0.072 | 0.042 | -0.436 | -0.418 | -0.039 |
| Fatigue | 0.035 | 0.073 | 0.026 | -0.189 | -0.263 | -0.028 |

**Table 112**
**Stress at top surface of AASHTO Bulb-Tee 63 girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.097 | 0.094 | 0.064 | -0.405 | -0.498 | -0.064 |
| Strength III Max | 0.077 | 0.056 | 0.038 | -0.156 | -0.036 | -0.038 |
| Strength V Max | 0.092 | 0.087 | 0.053 | -0.346 | -0.390 | -0.052 |
| Fatigue | 0.013 | 0.054 | 0.029 | -0.132 | -0.258 | -0.029 |

**Table 113**
**Stress at bottom surface of AASHTO Bulb-Tee 63 girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.522 | 0.534 | 0.051 | -0.091 | -0.075 | -0.050 |
| Strength III Max | 0.156 | 0.036 | 0.038 | -0.077 | -0.056 | -0.038 |
| Strength V Max | 0.436 | 0.418 | 0.039 | -0.087 | -0.072 | -0.042 |
| Fatigue | 0.189 | 0.263 | 0.028 | -0.035 | -0.073 | -0.026 |

**Table 114**
**Stress at bottom surface of AASHTO Bulb-Tee 63 girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.405 | 0.498 | 0.064 | -0.097 | -0.094 | -0.064 |
| Strength III Max | 0.156 | 0.036 | 0.038 | -0.077 | -0.056 | -0.038 |
| Strength V Max | 0.346 | 0.390 | 0.052 | -0.092 | -0.087 | -0.053 |
| Fatigue | 0.132 | 0.258 | 0.029 | -0.013 | -0.054 | -0.029 |

**Table 115**
**Stress at top surface of AASHTO Bulb-Tee 72 girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.096 | 0.071 | 0.044 | -0.524 | -0.542 | -0.053 |
| Strength III Max | 0.087 | 0.074 | 0.037 | -0.152 | -0.047 | -0.037 |
| Strength V Max | 0.093 | 0.074 | 0.034 | -0.439 | -0.425 | -0.041 |
| Fatigue | 0.038 | 0.073 | 0.025 | -0.190 | -0.268 | -0.029 |

**Table 116**
**Stress at top surface of AASHTO Bulb-Tee 72 girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.103 | 0.110 | 0.050 | -0.391 | -0.506 | -0.064 |
| Strength III Max | 0.087 | 0.074 | 0.037 | -0.152 | -0.047 | -0.037 |
| Strength V Max | 0.099 | 0.094 | 0.042 | -0.336 | -0.397 | -0.052 |
| Fatigue | 0.012 | 0.060 | 0.028 | -0.123 | -0.262 | -0.030 |

**Table 117**
**Stress at bottom surface of AASHTO Bulb-Tee 72 girder bridge deck in group B for HS20-44 truck load**

| HS20-44 | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.524 | 0.542 | 0.053 | -0.096 | -0.071 | -0.044 |
| Strength III Max | 0.152 | 0.047 | 0.037 | -0.087 | -0.074 | -0.037 |
| Strength V Max | 0.439 | 0.425 | 0.041 | -0.093 | -0.074 | -0.034 |
| Fatigue | 0.190 | 0.268 | 0.029 | -0.038 | -0.073 | -0.025 |

**Table 118**
**Stress at bottom surface of AASHTO Bulb-Tee 72 girder bridge deck in group B for sugarcane truck load**

| Sugarcane | | | | | | |
|---|---|---|---|---|---|---|
| Load | Max Value of Stress (Unit: Ksi) | | | | | |
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.391 | 0.506 | 0.064 | -0.103 | -0.110 | -0.050 |
| Strength III Max | 0.152 | 0.047 | 0.037 | -0.087 | -0.074 | -0.037 |
| Strength V Max | 0.336 | 0.397 | 0.052 | -0.099 | -0.094 | -0.042 |
| Fatigue | 0.123 | 0.262 | 0.030 | -0.012 | -0.060 | -0.028 |

## End and Intermediate Diaphragms Analysis

The diaphragm is defined to be a transverse stiffener, which is provided between girders in order to maintain section geometry. It has been thought to contribute to the overall distribution of live loads in bridges. In this study, the full-depth diaphragms were placed at both ends of the bridge supports, and two full-depth intermediate diaphragms were added at the distance of 40 feet to both of the bridge supports, as shown in Figure 45.

In the finite element analysis procedure, prismatic space truss members were used to model the end and intermediate diaphragms. These members were numbered and grouped as shown in Figure 45. The prismatic space truss member was limited to take only the axial forces. The load combination “Strength I Max” was the governing load combination that resulted in maximum axial forces in the diaphragms, while the load combination “Fatigue” was used to determine the long time effects of the truck load. The results indicated that compression and tension forces in all intermediate diaphragms are not critical.

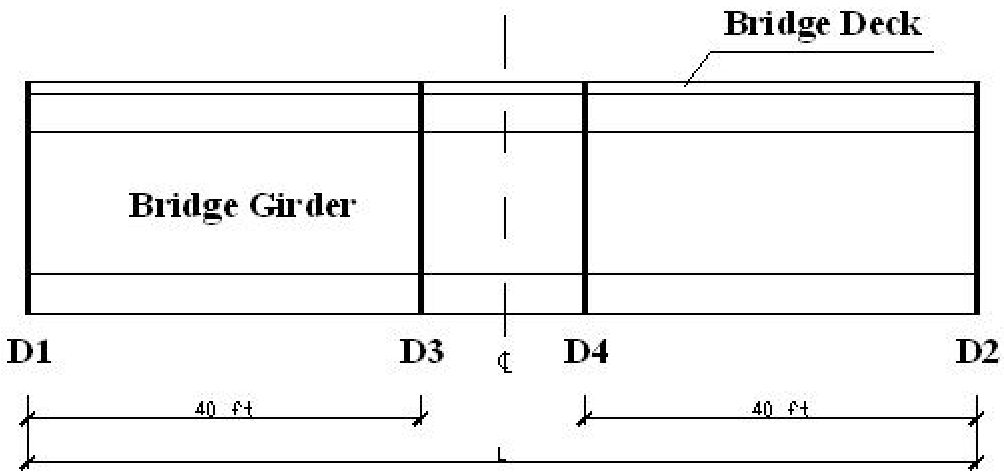



**Figure 45**
**Locations of end and intermediate diaphragms**

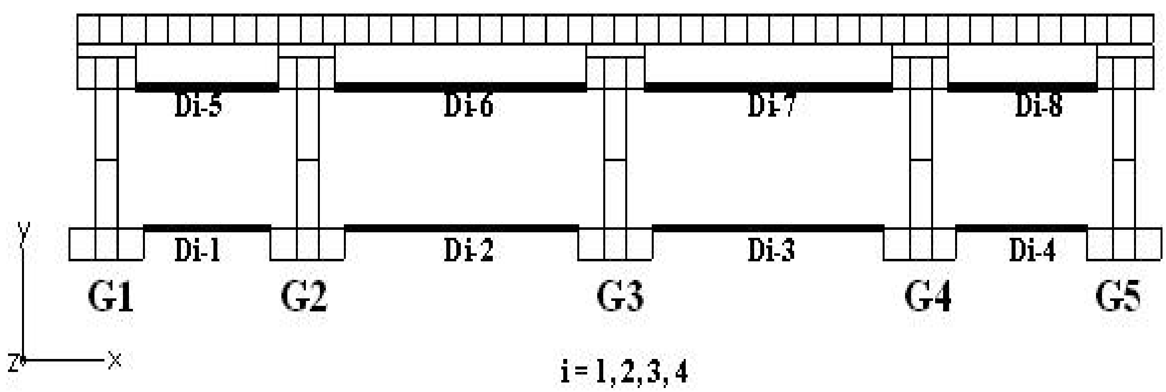



**Figure 46**
**Cross section of grouped diaphragms**

# DISCUSSION OF RESULTS

## Short Term Effects of Sugarcane Truck on Bridges

In this study, the short term effects of Sugarcane truck loads on simple span bridges designed for HS20-44 truck loads were evaluated by normalizing the maximum stress at both top and bottom surfaces of each girder, and the maximum deflections of each girder. By the discussion before, the "Strength I max" was considered as the critical load combination for the short term effects analysis.

**Short term effects on AASHTO type IV girder bridges in group A.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO type IV girders, designed for HS20-44 truck loads were summarized in table 119. The ratio of the maximum compressive stress at the top surface of girders varied between 1.04 and 1.09. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.94 and 1.09. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 119**
**Maximum short term stresses ratios AASHTO type IV bridge girders in group A**

| Strength I max | Ratio Sugarcane/HS20-44 | Ratio Sugarcane/HS20-44 |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.06 | 1.07 |
| Girder 2 | 1.08 | 1.05 |
| Girder 3 | 1.04 | 0.94 |
| Girder 4 | 1.07 | 1.03 |
| Girder 5 | 1.09 | 1.09 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.05 and 1.07, as summarized in table 120. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Type IV girders would not be significantly affected by the sugarcane truck load.

**Table 120**
**Maximum short term deflection ratios AASHTO type IV bridge girders in group A**

| Strength I Max | Deflection Ratio |
|---|---|
| Girder Number | Sugarcane/HS20-44 |
| Girder 1 | 1.05 |
| Girder 2 | 1.06 |
| Girder 3 | 1.07 |
| Girder 4 | 1.07 |
| Girder 5 | 1.07 |

**Short term effects on AASHTO type V girder bridges in group A.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO type V girders, designed for HS20-44 truck loads were summarized in table 121. The ratio of the maximum compressive stress at the top surface of girders varied between 1.02 and 1.08. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.96 and 1.08. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 121**
**Maximum short term stresses ratios AASHTO type V bridge girders in group A**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.05 | 1.07 |
| Girder 2 | 1.07 | 1.06 |
| Girder 3 | 1.02 | 0.96 |
| Girder 4 | 1.07 | 1.04 |
| Girder 5 | 1.08 | 1.08 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.05 and 1.06, as summarized in table 122. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Type V girders would not be significantly affected by the sugarcane truck load.

**Table 122**
**Maximum short term deflection ratios AASHTO type V bridge girders in group A**

| Strength I Max | Deflection Ratio |
|---|---|
| Girder Number | Sugarcane/HS20-44 |
| Girder 1 | 1.05 |
| Girder 2 | 1.06 |
| Girder 3 | 1.06 |
| Girder 4 | 1.06 |
| Girder 5 | 1.06 |

**Short term effects on AASHTO type VI girder bridges in group A.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO type VI girders, designed for HS20-44 truck loads were summarized in table 123. The ratio of the maximum compressive stress at the top surface of girders varied between 1.00 and 1.08. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.98 and 1.08. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 123**
**Maximum short term stresses ratios AASHTO type VI bridge girders in group A**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.05 | 1.06 |
| Girder 2 | 1.08 | 1.04 |
| Girder 3 | 1.00 | 0.98 |
| Girder 4 | 1.05 | 1.02 |
| Girder 5 | 1.07 | 1.08 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.05 and 1.06, as summarized in Table 83. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Type VI girders would not be significantly affected by the sugarcane truck load.

**Table 124**
**Maximum short term deflection ratios AASHTO type VI bridge girders in group A**

| Strength I Max | Deflection Ratio |
|---|---|
| Girder Number | Sugarcane/HS20-44 |
| Girder 1 | 1.05 |
| Girder 2 | 1.06 |
| Girder 3 | 1.06 |
| Girder 4 | 1.06 |
| Girder 5 | 1.06 |

**Short term effects on AASHTO Bulb-Tee 54 girder bridges in group A.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 54 girders, designed for HS20-44 truck loads are summarized in table 125. The ratio of the maximum compressive stress at the top surface of girders varied between 1.06 and 1.09. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.94 and 1.12. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 125**
**Maximum short term stresses ratios AASHTO Bulb-Tee 54 bridge girders group A**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.07 | 1.09 |
| Girder 2 | 1.07 | 1.04 |
| Girder 3 | 1.06 | 0.94 |
| Girder 4 | 1.07 | 1.03 |
| Girder 5 | 1.09 | 1.12 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.06 and 1.07, as summarized in table 126. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Bulb-Tee 54 Girders would not be significantly affected by the sugarcane truck load.

**Table 126**
**Maximum short term deflection ratios AASHTO Bulb-Tee 54 bridge girders in group A**

| Strength I Max | |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.06 |
| Girder 2 | 1.06 |
| Girder 3 | 1.07 |
| Girder 4 | 1.07 |
| Girder 5 | 1.07 |

**Short term effects on AASHTO Bulb-Tee 63 girder bridges in group A.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 63 girders, designed for HS20-44 truck loads are summarized in table 127. The ratio of the maximum compressive stress at the top surface of girders varied between 1.05 and 1.09. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.95 and 1.11. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 127**
**Maximum short term stresses ratios AASHTO Bulb-Tee 63 bridge girders group A**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.07 | 1.09 |
| Girder 2 | 1.08 | 1.05 |
| Girder 3 | 1.05 | 0.95 |
| Girder 4 | 1.07 | 1.04 |
| Girder 5 | 1.09 | 1.11 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.05 and 1.07, as summarized in table 128. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Bulb-Tee 63 Girders would not be significantly affected by the sugarcane truck load.

**Table 128**
**Maximum short term deflection ratios AASHTO Bulb-Tee 63 bridge girders in group A**

| Strength I Max | |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.05 |
| Girder 2 | 1.06 |
| Girder 3 | 1.07 |
| Girder 4 | 1.07 |
| Girder 5 | 1.07 |

**Short term effects on AASHTO Bulb-Tee 72 girder bridges in group A.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 72 girders, designed for HS20-44 truck loads are summarized in table 129. The ratio of the maximum compressive stress at the top surface of girders varied between 1.03 and 1.08. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.97 and 1.12. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 129**
**Maximum short term stresses ratios AASHTO Bulb-Tee 72 bridge girders group A**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.07 | 1.08 |
| Girder 2 | 1.03 | 1.05 |
| Girder 3 | 1.06 | 0.97 |
| Girder 4 | 1.07 | 1.04 |
| Girder 5 | 1.08 | 1.12 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.05 and 1.07, as summarized in table 130. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Bulb-Tee 72 Girders would not be significantly affected by the sugarcane truck load.

**Table 130**
**Maximum short term deflection ratios AASHTO Bulb-Tee 72 bridge girders in group A**

| Strength I Max | Deflection Ratios |
|---|---|
| Girder Number | Sugarcane/HS20-44 |
| Girder 1 | 1.05 |
| Girder 2 | 1.06 |
| Girder 3 | 1.07 |
| Girder 4 | 1.07 |
| Girder 5 | 1.06 |

**Short term effects on AASHTO type IV girder bridges in group B.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO type IV girders, designed for HS20-44 truck loads were summarized in table 131. The ratio of the maximum compressive stress at the top surface of girders varied between 1.00 and 1.07. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.97 and 1.08. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 131**
**Maximum short term stresses ratios AASHTO type IV bridge girders in group B**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.04 | 1.05 |
| Girder 2 | 1.05 | 1.05 |
| Girder 3 | 1.05 | 1.01 |
| Girder 4 | 1.00 | 0.97 |
| Girder 5 | 1.04 | 1.00 |
| Girder 6 | 1.07 | 1.03 |
| Girder 7 | 1.07 | 1.08 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.04 and 1.07, as summarized in table 132. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Type IV girders would not be significantly affected by the sugarcane truck load.

**Table 132**
**Maximum short term deflection ratios AASHTO type IV bridge girders in group B**

| Strength I Max | Deflection Ratios |
|---|---|
| Girder Number | Sugarcane/HS20-44 |
| Girder 1 | 1.04 |
| Girder 2 | 1.05 |
| Girder 3 | 1.06 |
| Girder 4 | 1.07 |
| Girder 5 | 1.07 |
| Girder 6 | 1.07 |
| Girder 7 | 1.07 |

**Short term effects on AASHTO type V girder bridges in group B.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO type V girders, designed for HS20-44 truck loads were summarized in table 133. The ratio of the maximum compressive stress at the top surface of girders varied between 1.00 and 1.06. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.00 and 1.06. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 133**
**Maximum short term stresses ratios AASHTO type V bridge girders in group B**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.04 | 1.04 |
| Girder 2 | 1.04 | 1.04 |
| Girder 3 | 1.02 | 1.00 |
| Girder 4 | 1.00 | 1.00 |
| Girder 5 | 1.00 | 1.01 |
| Girder 6 | 1.02 | 1.01 |
| Girder 7 | 1.06 | 1.06 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.04 and 1.06, as summarized in table 134. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Type V girders would not be significantly affected by the sugarcane truck load.

**Table 134**
**Maximum short term deflection ratios AASHTO type V bridge girders in group B**

| Strength I Max | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.04 |
| Girder 2 | 1.04 |
| Girder 3 | 1.05 |
| Girder 4 | 1.06 |
| Girder 5 | 1.06 |
| Girder 6 | 1.06 |
| Girder 7 | 1.06 |

**Short term effects on AASHTO type VI girder bridges in group B.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO type VI girders, designed for HS20-44 truck loads were summarized in table 135. The ratio of the maximum compressive stress at the top surface of girders varied between 1.01 and 1.06. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.01 and 1.06. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 135**
**Maximum short term stresses ratios AASHTO type VI bridge girders in group B**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.03 | 1.04 |
| Girder 2 | 1.04 | 1.04 |
| Girder 3 | 1.02 | 1.02 |
| Girder 4 | 1.01 | 1.01 |
| Girder 5 | 1.01 | 1.02 |
| Girder 6 | 1.03 | 1.03 |
| Girder 7 | 1.06 | 1.06 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.03 and 1.06, as summarized in table 136. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Type VI girders would not be significantly affected by the sugarcane truck load.

**Table 136**
**Maximum short term deflection ratios AASHTO type VI bridge girders in group B**

| Strength I Max | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.03 |
| Girder 2 | 1.04 |
| Girder 3 | 1.05 |
| Girder 4 | 1.05 |
| Girder 5 | 1.06 |
| Girder 6 | 1.06 |
| Girder 7 | 1.06 |

**Short term effects on AASHTO Bulb-Tee 54 girder bridges in group B.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 54 girders, designed for HS20-44 truck loads are summarized in table 137. The ratio of the maximum compressive stress at the top surface of girders varied between 1.02 and 1.08. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.98 and 1.08. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 137**
**Maximum short term stresses ratios AASHTO Bulb-Tee 54 bridge girders in group B**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.05 | 1.06 |
| Girder 2 | 1.06 | 1.05 |
| Girder 3 | 1.06 | 0.98 |
| Girder 4 | 1.02 | 0.99 |
| Girder 5 | 1.04 | 1.00 |
| Girder 6 | 1.08 | 1.00 |
| Girder 7 | 1.08 | 1.08 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.04 and 1.08, as summarized in table 138. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Bulb-Tee 54 Girders would not be significantly affected by the sugarcane truck load.

**Table 138**
**Maximum short term deflection ratios AASHTO Bulb-Tee 54 bridge girders in group B**

| Strength I Max | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.04 |
| Girder 2 | 1.05 |
| Girder 3 | 1.06 |
| Girder 4 | 1.07 |
| Girder 5 | 1.08 |
| Girder 6 | 1.07 |
| Girder 7 | 1.07 |

**Short term effects on AASHTO Bulb-Tee 63 girder bridges in group B.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 63 girders, designed for HS20-44 truck loads are summarized in table 139. The ratio of the maximum compressive stress at the top surface of girders varied between 1.05 and 1.09. The ratio of the maximum tensile stress at the bottom surface of girders varied between 0.95 and 1.11. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 139**
**Maximum short term stresses ratios AASHTO Bulb-Tee 63 bridge girders group B**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.05 | 1.06 |
| Girder 2 | 1.05 | 1.05 |
| Girder 3 | 1.04 | 1.01 |
| Girder 4 | 1.06 | 0.98 |
| Girder 5 | 1.06 | 0.99 |
| Girder 6 | 1.09 | 1.02 |
| Girder 7 | 1.07 | 1.08 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.04 and 1.08, as summarized in table 140. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Bulb-Tee 63 Girders would not be significantly affected by the sugarcane truck load.

**Table 140**
**Maximum short term deflection ratios AASHTO Bulb-Tee 63 bridge girders in group B**

| Strength I Max | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.04 |
| Girder 2 | 1.05 |
| Girder 3 | 1.06 |
| Girder 4 | 1.07 |
| Girder 5 | 1.07 |
| Girder 6 | 1.07 |
| Girder 7 | 1.08 |

**Short term effects on AASHTO Bulb-Tee 72 girder bridges in group B.** The short term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 72 girders, designed for HS20-44 truck loads are summarized in table 141. The ratio of the maximum compressive stress at the top surface of girders varied between 1.02 and 1.06. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.02 and 1.07. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 141**
**Maximum short term stresses ratios AASHTO Bulb-Tee 72 bridge girders group B**

| Strength I max | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.04 | 1.05 |
| Girder 2 | 1.05 | 1.05 |
| Girder 3 | 1.04 | 1.03 |
| Girder 4 | 1.02 | 1.02 |
| Girder 5 | 1.02 | 1.03 |
| Girder 6 | 1.05 | 1.04 |
| Girder 7 | 1.06 | 1.07 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.04 and 1.07, as summarized in table 142. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks. Since the maximum ratio in this study was not beyond the criterion 1.1, the serviceability of bridge which was built with AASHTO Bulb-Tee 72 Girders would not be significantly affected by the sugarcane truck load.

**Table 142**
**Maximum short term deflection ratios AASHTO Bulb-Tee 72 bridge girders in group B**

| Strength I Max | |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.04 |
| Girder 2 | 1.05 |
| Girder 3 | 1.06 |
| Girder 4 | 1.06 |
| Girder 5 | 1.07 |
| Girder 6 | 1.07 |
| Girder 7 | 1.07 |

## Long Term Effects of Sugarcane Truck on Bridges

In this study, the long term effects of Sugarcane truck loads on simple span bridges designed for HS20-44 truck loads were also evaluated by normalizing the maximum stress at both top and bottom surfaces of each girder, and the maximum deflections of each girder. "Fatigue" was considered as the critical load combination for the long term effects analysis.

Long term effects on AASHTO type IV girder bridges in group A. The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO type IV girders, designed for HS20-44 truck loads are summarized in table 143. The ratio of the maximum compressive stress at the top surface of girders varied between 1.34 and 1.88. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.29 and 1.70. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 143**
**Maximum long term stresses ratios AASHTO type IV bridge girders in group A**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.57 | 1.70 |
| Girder 2 | 1.88 | 1.58 |
| Girder 3 | 1.34 | 1.29 |
| Girder 4 | 1.49 | 1.44 |
| Girder 5 | 1.80 | 1.67 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.62 and 1.70, as summarized in table 144.

The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the sugarcane trucks.

**Table 144**
**Maximum long term deflection ratios AASHTO type IV bridge girders in group A**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.62 |
| Girder 2 | 1.63 |
| Girder 3 | 1.64 |
| Girder 4 | 1.70 |
| Girder 5 | 1.68 |

**Long term effects on AASHTO type V girder bridges in group A.** The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO type V girders, designed for HS20-44 truck loads are summarized in table 145. The ratio of the maximum compressive stress at the top surface of girders varied between 1.22 and 1.66. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.51 and 1.65. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 145**
**Maximum long term stresses ratios AASHTO type V bridge girders in group A**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.66 | 1.65 |
| Girder 2 | 1.65 | 1.61 |
| Girder 3 | 1.22 | 1.51 |
| Girder 4 | 1.65 | 1.63 |
| Girder 5 | 1.66 | 1.65 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.63 and 1.66, as summarized in table 146. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the sugarcane trucks.

**Table 146**
**Maximum long term deflection ratios AASHTO type V bridge girders in group A**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.66 |
| Girder 2 | 1.66 |
| Girder 3 | 1.63 |
| Girder 4 | 1.64 |
| Girder 5 | 1.66 |

**Long term effects on AASHTO type VI girder bridges in group A.** The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO type VI girders, designed for HS20-44 truck loads are summarized in table 147. The ratio of the maximum compressive stress at the top surface of girders varied between 1.25 and 1.65. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.40 and 2.03. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 147**
**Maximum long term stresses ratios AASHTO type VI bridge girders in group A**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.63 | 1.63 |
| Girder 2 | 1.64 | 2.03 |
| Girder 3 | 1.25 | 1.40 |
| Girder 4 | 1.65 | 1.67 |
| Girder 5 | 1.62 | 1.60 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.63 and 1.67, as summarized in table 148. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 148**
**Maximum long term deflection ratios AASHTO type VI bridge girders in group A**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.67 |
| Girder 2 | 1.65 |
| Girder 3 | 1.63 |
| Girder 4 | 1.65 |
| Girder 5 | 1.66 |

**Long term effects on AASHTO Bulb-Tee 54 girder bridges in group A.** The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 54 girders, designed for HS20-44 truck loads are summarized in table 149. The ratio of the maximum compressive stress at the top surface of girders varied between 1.37 and 1.79. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.47 and 1.67. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 149**
**Maximum long term stresses ratios AASHTO Bulb-Tee 54 bridge girders in group A**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.68 | 1.67 |
| Girder 2 | 1.63 | 1.57 |
| Girder 3 | 1.37 | 1.47 |
| Girder 4 | 1.56 | 1.56 |
| Girder 5 | 1.79 | 1.63 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.61 and 1.67, as summarized in table 150. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 150**
**Maximum long term deflection ratios AASHTO Bulb-Tee 54 bridge girders in group A**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Sugarcane/HS20-44 |
| Girder 1 | 1.67 |
| Girder 2 | 1.67 |
| Girder 3 | 1.63 |
| Girder 4 | 1.61 |
| Girder 5 | 1.63 |

Long term effects on AASHTO Bulb-Tee 63 girder bridges in group A. The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 63 girders, designed for HS20-44 truck loads are summarized in table 151. The ratio of the maximum compressive stress at the top surface of girders varied between 1.32 and 1.67. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.42 and 1.69. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 151**
**Maximum long term stresses ratios AASHTO Bulb-Tee 63 bridge girders group A**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.67 | 1.60 |
| Girder 2 | 1.61 | 1.58 |
| Girder 3 | 1.32 | 1.42 |
| Girder 4 | 1.54 | 1.50 |
| Girder 5 | 1.67 | 1.69 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.63 and 1.70, as summarized in table 152. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 152**
**Maximum long term deflection ratios AASHTO Bulb-Tee 63 bridge girders in group A**

| Fatigue | |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.63 |
| Girder 2 | 1.70 |
| Girder 3 | 1.64 |
| Girder 4 | 1.67 |
| Girder 5 | 1.64 |

Long term effects on AASHTO Bulb-Tee 72 girder bridges in group A. The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with AASHTO Bulb-Tee 72 girders, designed for HS20-44 truck loads are summarized in table 153. The ratio of the maximum compressive stress at the top surface of girders varied between 1.32 and 1.63. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.27 and 1.75. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 153**
**Maximum long term stresses ratios AASHTO Bulb-Tee 72 bridge girders group A**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.62 | 1.75 |
| Girder 2 | 1.63 | 1.70 |
| Girder 3 | 1.32 | 1.27 |
| Girder 4 | 1.54 | 1.58 |
| Girder 5 | 1.61 | 1.73 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.50 and 1.71, as summarized in table 154. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 154**
**Maximum long term deflection ratios AASHTO Bulb-Tee 72 bridge girders in group A**

| Fatigue | |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.50 |
| Girder 2 | 1.71 |
| Girder 3 | 1.55 |
| Girder 4 | 1.67 |
| Girder 5 | 1.50 |

**Long term effects on AASHTO type IV girder bridges in group B.** The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO type IV girders, designed for HS20-44 truck loads are summarized in table 155. The ratio of the maximum compressive stress at the top surface of girders varied between 1.43 and 1.65. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.48 and 1.70. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 155**
**Maximum long term stresses ratios AASHTO type IV bridge girders in group B**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.65 | 1.66 |
| Girder 2 | 1.64 | 1.70 |
| Girder 3 | 1.65 | 1.63 |
| Girder 4 | 1.43 | 1.48 |
| Girder 5 | 1.54 | 1.59 |
| Girder 6 | 1.65 | 1.63 |
| Girder 7 | 1.64 | 1.64 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.64 and 1.66, as summarized in table 156. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 156**
**Maximum long term deflection ratios AASHTO type IV bridge girders in group B**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.66 |
| Girder 2 | 1.66 |
| Girder 3 | 1.65 |
| Girder 4 | 1.65 |
| Girder 5 | 1.64 |
| Girder 6 | 1.65 |
| Girder 7 | 1.66 |

**Long term effects on AASHTO type V girder bridges in group B.** The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO type V girders, designed for HS20-44 truck loads are summarized in table 157. The ratio of the maximum compressive stress at the top surface of girders varied between 1.25 and 1.66. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.41 and 1.67. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 157**
**Maximum long term stresses ratios AASHTO type V bridge girders in group B**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.66 | 1.67 |
| Girder 2 | 1.64 | 1.60 |
| Girder 3 | 1.66 | 1.53 |
| Girder 4 | 1.25 | 1.41 |
| Girder 5 | 1.35 | 1.42 |
| Girder 6 | 1.65 | 1.61 |
| Girder 7 | 1.64 | 1.60 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.64 and 1.68, as summarized in table 157. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 158**
**Maximum long term deflection ratios AASHTO type V bridge girders in group B**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.68 |
| Girder 2 | 1.66 |
| Girder 3 | 1.65 |
| Girder 4 | 1.64 |
| Girder 5 | 1.65 |
| Girder 6 | 1.65 |
| Girder 7 | 1.67 |

**Long term effects on AASHTO type VI girder bridges in group B.** The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO type VI girders, designed for HS20-44 truck loads are summarized in table 159. The ratio of the maximum compressive stress at the top surface of girders varied between 1.23 and 1.70. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.44 and 1.71. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 159**
**Maximum long term stresses ratios AASHTO type VI bridge girders in group B**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.66 | 1.67 |
| Girder 2 | 1.63 | 1.71 |
| Girder 3 | 1.70 | 1.63 |
| Girder 4 | 1.23 | 1.44 |
| Girder 5 | 1.32 | 1.45 |
| Girder 6 | 1.65 | 1.69 |
| Girder 7 | 1.62 | 1.60 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.65 and 1.67, as summarized in table 160. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 160**
**Maximum long term deflection ratios AASHTO type VI bridge girders in group B**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.66 |
| Girder 2 | 1.66 |
| Girder 3 | 1.65 |
| Girder 4 | 1.66 |
| Girder 5 | 1.66 |
| Girder 6 | 1.66 |
| Girder 7 | 1.67 |

**Long term effects on AASHTO Bulb-Tee 54 girder bridges in group B.** The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO Bulb-Tee 54 girders, designed for HS20-44 truck loads are summarized in table 161. The ratio of the maximum compressive stress at the top surface of girders varied between 1.39 and 1.67. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.42 and 1.64. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 161**
**Maximum long term stresses ratios AASHTO Bulb-Tee 54 bridge girders group B**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.67 | 1.63 |
| Girder 2 | 1.65 | 1.63 |
| Girder 3 | 1.65 | 1.53 |
| Girder 4 | 1.39 | 1.42 |
| Girder 5 | 1.50 | 1.58 |
| Girder 6 | 1.65 | 1.63 |
| Girder 7 | 1.65 | 1.64 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.64 and 1.67, as summarized in table 162. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 162**
**Maximum long term deflection ratios AASHTO Bulb-Tee 54 bridge girders in group B**

| Fatigue | |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.67 |
| Girder 2 | 1.66 |
| Girder 3 | 1.64 |
| Girder 4 | 1.65 |
| Girder 5 | 1.65 |
| Girder 6 | 1.65 |
| Girder 7 | 1.65 |

Long term effects on AASHTO Bulb-Tee 63 girder bridges in group B. The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO Bulb-Tee 63 girders, designed for HS20-44 truck loads are summarized in table 163. The ratio of the maximum compressive stress at the top surface of girders varied between 1.32 and 1.67. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.42 and 1.69. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 163**
**Maximum long term stresses ratios AASHTO Bulb-Tee 63 bridge girders group B**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.65 | 1.65 |
| Girder 2 | 1.62 | 1.62 |
| Girder 3 | 1.70 | 1.64 |
| Girder 4 | 1.61 | 1.50 |
| Girder 5 | 1.72 | 1.51 |
| Girder 6 | 1.69 | 1.64 |
| Girder 7 | 1.62 | 1.59 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.63 and 1.65, as summarized in table 164. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 164**
**Maximum long term deflection ratios AASHTO Bulb-Tee 63 bridge girders in group B**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.63 |
| Girder 2 | 1.64 |
| Girder 3 | 1.65 |
| Girder 4 | 1.64 |
| Girder 5 | 1.63 |
| Girder 6 | 1.65 |
| Girder 7 | 1.64 |

Long term effects on AASHTO Bulb-Tee 72 girder bridges in group B. The long term stress effects of bridge girders of sugarcane truck loads on simple span bridges with seven AASHTO Bulb-Tee 72 girders, designed for HS20-44 truck loads are summarized in table 165. The ratio of the maximum compressive stress at the top surface of girders varied between 1.37 and 1.69. The ratio of the maximum tensile stress at the bottom surface of girders varied between 1.46 and 1.70. The bridges in this study with stress ratios which were greater than 1.1 would be overstressed, and might experience more cracking in the bridge girders. Such cracks would require additional inspections along with early and frequent maintenance.

**Table 165**
**Maximum long term stresses ratios AASHTO Bulb-Tee 72 bridge girders group B**

| Fatigue | Ratio Sugarcane/HS20-44 | |
|---|---|---|
| Girder Number | Compressive Stress | Tensile Stress |
| Girder 1 | 1.63 | 1.62 |
| Girder 2 | 1.63 | 1.61 |
| Girder 3 | 1.69 | 1.70 |
| Girder 4 | 1.37 | 1.46 |
| Girder 5 | 1.49 | 1.46 |
| Girder 6 | 1.68 | 1.69 |
| Girder 7 | 1.63 | 1.60 |

The ratio for deflection caused by Sugarcane truck loads as compared to HS20-44 truck loads varied between 1.65 and 1.67, as summarized in table 166. The above discussion on the ratio of the stress was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the Sugarcane trucks.

**Table 166**
**Maximum long term deflection ratios AASHTO Bulb-Tee 72 bridge girders in group B**

| Fatigue | Deflection Ratios |
|---|---|
| Girder Number | Ratio Sugarcane/HS20-44 |
| Girder 1 | 1.67 |
| Girder 2 | 1.66 |
| Girder 3 | 1.65 |
| Girder 4 | 1.65 |
| Girder 5 | 1.65 |
| Girder 6 | 1.66 |
| Girder 7 | 1.66 |

## Bridge Deck Evaluation

This part of research focuses on the strength and serviceability of bridge decks due to the impact of the heavy concentrated loads from the sugarcane trucks. Finite element analysis is used for a typical deck and girder system to determine the effects of the trucks on the stresses in the transverse and longitudinal directions, and the shear stress.

**AASHTO type IV girder bridges in group A.** The effects of sugarcane truck loads on bridge deck with five AASHTO type IV girders, designed for HS20-44 truck loads were presented in tables 167 and 168. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination "Fatigue" was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.10 and 0.89 in the transverse direction. The ratio of shear stress was 1.30. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.82, and 1.15 in the transverse direction; the ratio of shear stress was 1.39. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.82 and 1.15 in the transverse direction. The ratio of shear stress was 1.39. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.10, and 0.89 in the transverse direction; the ratio of shear stress was 1.30. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.67 and 0.94 in the transverse direction. The ratio of shear stress was 1.25. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.77, and 1.21 in the transverse direction; the ratio of shear stress was 1.52. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.77 and 1.21 in the transverse direction. The ratio of shear stress was 1.52. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.67, and 0.94 in the transverse direction; the ratio of shear stress was 1.25. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

**Table 167**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO type IV – group A**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.103 | 0.886 | 1.297 | 0.819 | 1.148 | 1.390 |
| Fatigue | 1.665 | 0.943 | 1.250 | 0.771 | 1.205 | 1.517 |

**Table 168**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO type IV – group A**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.819 | 1.148 | 1.390 | 1.103 | 0.886 | 1.297 |
| Fatigue | 0.771 | 1.205 | 1.517 | 1.665 | 0.943 | 1.250 |

**AASHTO type V girder bridges in group A.** The effects of sugarcane truck loads on bridge deck with five AASHTO type V girders, designed for HS20-44 truck loads were presented in tables 169 and 170. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination "Fatigue" was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.10 and 1.24 in the transverse direction. The ratio of shear stress was 1.24. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.74, and 0.95 in the transverse direction; the ratio of shear stress was 1.21. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.74 and 0.95 in the transverse direction. The ratio of shear stress was 1.21. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.10, and 1.24 in the transverse direction; the ratio of shear stress was 1.24. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.66 and 1.27 in the transverse direction. The ratio of shear stress was 1.04. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.65, and 0.96 in the transverse direction; the ratio of shear stress was 0.97. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.65 and 0.96 in the transverse direction. The ratio of shear stress was 0.97. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was

0.66, and 1.27 in the transverse direction; the ratio of shear stress was 1.04. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

**Table 169**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO type V – group A**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.103 | 1.241 | 1.239 | 0.735 | 0.946 | 1.213 |
| Fatigue | 0.663 | 1.274 | 1.037 | 0.654 | 0.955 | 0.971 |

**Table 170**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO type V – group A**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.735 | 0.946 | 1.213 | 1.103 | 1.241 | 1.239 |
| Fatigue | 0.654 | 0.955 | 0.971 | 0.663 | 1.274 | 1.037 |

**AASHTO type VI girder bridges in group A.** The effects of sugarcane truck loads on bridge deck with five AASHTO type VI girders, designed for HS20-44 truck loads were presented in tables 171 and 172. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination "Fatigue" was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.09 and 1.11 in the transverse direction. The ratio of shear stress was 0.92. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.71, and 0.91 in the transverse direction; the ratio of shear stress was 1.03. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.71 and 0.91 in the transverse direction. The ratio of shear stress was 1.03. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.09, and 1.11 in the transverse direction; the ratio of shear stress was 0.92. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.66 and 1.46 in the transverse direction. The ratio of shear stress was 0.96. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.63, and 0.95 in the transverse direction; the ratio of shear stress was 1.04. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.63 and 0.95 in the transverse direction. The ratio of shear stress was 1.04. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.66, and 1.46 in the transverse direction; the ratio of shear stress was 0.96. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

**Table 171**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO type VI – group A**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.090 | 1.107 | 0.915 | 0.713 | 0.912 | 1.030 |
| Fatigue | 0.661 | 1.460 | 0.964 | 0.634 | 0.949 | 1.038 |

**Table 172**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO type VI – group A**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.713 | 0.912 | 1.030 | 1.090 | 1.107 | 0.915 |
| Fatigue | 0.634 | 0.949 | 1.038 | 0.661 | 1.460 | 0.964 |

**AASHTO Bulb-Tee 54 girder bridges in group A.** The effects of sugarcane truck loads on bridge deck with five AASHTO Bulb-Tee 54 girders, designed for HS20-44 truck loads were presented tables 173 and 174. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination "Fatigue" was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.13 and 1.17 in the transverse direction. The ratio of shear stress was 1.41. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.76, and 0.99 in the transverse direction; the ratio of shear stress was 1.31. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.76 and 0.99 in the transverse direction. The ratio of shear stress was 1.31. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.13, and 1.17 in the transverse direction; the ratio of shear stress was 1.41. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.86 and 1.19 in the transverse direction. The ratio of shear stress was 1.14. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.70, and 1.03 in the transverse direction; the ratio of shear stress was 1.52. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.70 and 1.03 in the transverse direction. The ratio of shear stress was 1.22. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.86, and 1.19 in the transverse direction; the ratio of shear stress was 1.14. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

**Table 173**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO Bulb-Tee 54 – group A**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.126 | 1.174 | 1.413 | 0.760 | 0.985 | 1.314 |
| Fatigue | 0.858 | 1.194 | 1.139 | 0.704 | 1.030 | 1.217 |

**Table 174**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO Bulb-Tee 54 – group A**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.760 | 0.985 | 1.314 | 1.126 | 1.174 | 1.413 |
| Fatigue | 0.704 | 1.030 | 1.217 | 0.858 | 1.194 | 1.139 |

**AASHTO Bulb-Tee 63 girder bridges in group A.** The effects of sugarcane truck loads on bridge deck with five AASHTO Bulb-Tee 63 girders, designed for HS20-44 truck loads were presented in tables 175 and 176.   The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination “Fatigue” was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.12 and 1.07 in the transverse direction. The ratio of shear stress was 1.33. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.73, and 0.97 in the transverse direction; the ratio of shear stress was 1.27. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.73 and 0.97 in the transverse direction. The ratio of shear stress was 1.27. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.12, and 1.07 in the transverse direction; the ratio of shear stress was 1.33. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.49 and 1.22 in the transverse direction. The ratio of shear stress was 1.11. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.67, and 1.01 in the transverse direction; the ratio of shear stress was 1.16. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.67 and 1.01 in the transverse direction. The ratio of shear stress was 1.16. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.49, and 1.22 in the transverse direction; the ratio of shear stress was 1.11. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

We should consider that the locations of maximum stresses due to HS20-44 or Sugarcane truck loads may differ. Further research should be applied to obtain the ratios of the stresses at same location to evaluate the deck behavior under heavy truck loads.

**Table 175**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO Bulb-Tee 63 – group A**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.123 | 1.073 | 1.327 | 0.727 | 0.970 | 1.269 |
| Fatigue | 0.485 | 1.217 | 1.112 | 0.674 | 1.005 | 1.164 |

**Table 176**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO Bulb-Tee 63 – group A**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.727 | 0.970 | 1.269 | 1.123 | 1.073 | 1.327 |
| Fatigue | 0.674 | 1.005 | 1.164 | 0.485 | 1.217 | 1.112 |

**AASHTO Bulb-Tee 72 girder bridges in group A.** The effects of sugarcane truck loads on bridge deck with five AASHTO Bulb-Tee 72 girders, designed for HS20-44 truck loads were presented in tables 177 and 178. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination "Fatigue" was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.12 and 1.09 in the transverse direction. The ratio of shear stress was 1.23. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.70, and 0.96 in the transverse direction; the ratio of shear stress was 1.16. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.70 and 0.96 in the transverse direction. The ratio of shear stress was 1.16. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.12, and 1.09 in the transverse direction; the ratio of shear stress was 1.23. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.60 and 1.25 in the transverse direction. The ratio of shear stress was 1.09. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.65, and 0.99 in the transverse direction; the ratio of shear stress was 1.11. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.65 and 0.99 in the transverse direction. The ratio of shear stress was 1.11. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.60, and 1.25 in the transverse direction; the ratio of shear stress was 1.09. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

We should consider that the locations of maximum stresses due to HS20-44 or Sugarcane truck loads may differ. Further research should be applied to obtain the ratios of the stresses at same location to evaluate the deck behavior under heavy truck loads.

**Table 177**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO Bulb-Tee 72 – group A**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.115 | 1.091 | 1.230 | 0.702 | 0.958 | 1.155 |
| Fatigue | 0.602 | 1.253 | 1.090 | 0.652 | 0.985 | 1.107 |

**Table 178**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO Bulb-Tee 72 – group A**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.702 | 0.958 | 1.155 | 1.115 | 1.091 | 1.230 |
| Fatigue | 0.652 | 0.985 | 1.107 | 0.602 | 1.253 | 1.090 |

**AASHTO type IV girder bridges in group B.** The effects of sugarcane truck loads on bridge deck with seven AASHTO type IV girders, designed for HS20-44 truck loads were presented in tables 179 and 180. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination "Fatigue" was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.09 and 1.31 in the transverse direction. The ratio of shear stress was 1.25. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.88, and 1.10 in the transverse direction; the ratio of shear stress was 1.30. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.88 and 1.10 in the transverse direction. The ratio of shear stress was 1.30. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.09, and 1.31 in the transverse direction; the ratio of shear stress was 1.25. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.66 and 1.70 in the transverse direction. The ratio of shear stress was 1.53. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.83, and 1.21 in the transverse direction; the ratio of shear stress was 1.45. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.83 and 1.21 in the transverse direction. The ratio of shear stress was 1.45. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.66, and 1.70 in the transverse direction; the ratio of shear stress was 1.53. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

**Table 179**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO type IV – group B**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.087 | 1.311 | 1.250 | 0.878 | 1.103 | 1.295 |
| Fatigue | 1.663 | 1.704 | 1.526 | 0.827 | 1.207 | 1.452 |

**Table 180**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO type IV – group B**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.878 | 1.103 | 1.295 | 1.087 | 1.311 | 1.250 |
| Fatigue | 0.827 | 1.207 | 1.452 | 1.663 | 1.704 | 1.526 |

**AASHTO type V girder bridges in group B.** The effects of sugarcane truck loads on bridge deck with seven AASHTO type V girders, designed for HS20-44 truck loads were presented in tables 181 and 182. The load combination “Strength I max” was used to determine the short term effect of bridge deck while load combination “Fatigue” was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.07 and 1.39 in the transverse direction. The ratio of shear stress was 1.25. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.77, and 0.93 in the transverse direction; the ratio of shear stress was 1.30. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.77 and 0.93 in the transverse direction. The ratio of shear stress was 1.30. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.07, and 1.39 in the transverse direction; the ratio of shear stress was 1.25. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.47 and 0.62 in the transverse direction. The ratio of shear stress was 1.07. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.66, and 0.98 in the transverse direction; the ratio of shear stress was 1.03. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.66 and 0.98 in the transverse direction. The ratio of shear stress was 1.03. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.47, and 0.62 in the transverse direction; the ratio of shear stress was 1.07. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

**Table 181**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO type V – group B**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.071 | 1.385 | 1.248 | 0.766 | 0.927 | 1.302 |
| Fatigue | 0.468 | 0.622 | 1.067 | 0.657 | 0.976 | 1.031 |

**Table 182**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO type V – group B**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.766 | 0.927 | 1.302 | 1.071 | 1.385 | 1.248 |
| Fatigue | 0.657 | 0.976 | 1.031 | 0.468 | 0.622 | 1.067 |

**AASHTO type VI girder bridges in group B.** The effects of sugarcane truck loads on bridge deck with seven AASHTO type VI girders, designed for HS20-44 truck loads were presented in tables 183 and 184. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination "Fatigue" was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.07 and 1.20 in the transverse direction. The ratio of shear stress was 1.13. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.75, and 0.94 in the transverse direction; the ratio of shear stress was 1.21. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.75 and 0.94 in the transverse direction. The ratio of shear stress was 1.21. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.07, and 1.20 in the transverse direction; the ratio of shear stress was 1.13. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.31 and 0.71 in the transverse direction. The ratio of shear stress was 1.10. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.64, and 0.98 in the transverse direction; the ratio of shear stress was 1.00. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.64 and 0.98 in the transverse direction. The ratio of shear stress was 1.00. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.31, and 0.71 in the transverse direction; the ratio of shear stress was 1.10. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

**Table 183**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO type VI – group B**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.068 | 1.203 | 1.129 | 0.754 | 0.936 | 1.206 |
| Fatigue | 0.311 | 0.712 | 1.099 | 0.642 | 0.982 | 0.998 |

**Table 184**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO type VI – group B**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.754 | 0.936 | 1.206 | 1.068 | 1.203 | 1.129 |
| Fatigue | 0.642 | 0.982 | 0.998 | 0.311 | 0.712 | 1.099 |

**AASHTO Bulb-Tee 54 girder bridges in group B.** The effects of sugarcane truck loads on bridge deck with seven AASHTO Bulb-Tee 54 girders, designed for HS20-44 truck loads were presented in tables 185 and 186. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination “Fatigue” was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.10 and 1.41 in the transverse direction. The ratio of shear stress was 1.31. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.79, and 0.93 in the transverse direction; the ratio of shear stress was 1.38. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.79 and 0.93 in the transverse direction. The ratio of shear stress was 1.38. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.10, and 1.41 in the transverse direction; the ratio of shear stress was 1.31. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.55 and 0.73 in the transverse direction. The ratio of shear stress was 1.15. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.72, and 0.98 in the transverse direction; the ratio of shear stress was 1.12. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.72 and 0.98 in the transverse direction. The ratio of shear stress was 1.12. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.55, and 0.73 in the transverse direction; the ratio of shear stress was 1.15. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

**Table 185**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO Bulb-Tee 54 – group B**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.100 | 1.408 | 1.307 | 0.791 | 0.930 | 1.377 |
| Fatigue | 0.554 | 0.725 | 1.153 | 0.722 | 0.978 | 1.117 |

**Table 186**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO Bulb-Tee 54 – group B**

| Load Combination | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.791 | 0.930 | 1.377 | 1.100 | 1.408 | 1.307 |
| Fatigue | 0.722 | 0.978 | 1.117 | 0.554 | 0.725 | 1.153 |

**AASHTO Bulb-Tee 63 girder bridges in group B.** The effects of sugarcane truck loads on bridge deck with seven AASHTO Bulb-Tee 63 girders, designed for HS20-44 truck loads were presented in tables 187 and 188. The load combination “Strength I max” was used to determine the short term effect of bridge deck while load combination “Fatigue” was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.07 and 1.26 in the transverse direction. The ratio of shear stress was 1.27. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.78, and 0.93 in the transverse direction; the ratio of shear stress was 1.25. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.78 and 0.93 in the transverse direction. The ratio of shear stress was 1.25. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.07, and 1.26 in the transverse direction; the ratio of shear stress was 1.27. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.37 and 0.73 in the transverse direction. The ratio of shear stress was 1.10. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.70, and 0.98 in the transverse direction; the ratio of shear stress was 1.01. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.70 and 0.98 in the transverse direction. The ratio of shear stress was 1.01. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.37, and 0.73 in the transverse direction; the ratio of shear stress was 1.10. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

We should consider that the locations of maximum stresses due to HS20-44 or Sugarcane truck loads may differ. Further research should be applied to obtain the ratios of the stresses at same location to evaluate the deck behavior under heavy truck loads.

**Table 187**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO Bulb-Tee 63 – group B**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.067 | 1.263 | 1.273 | 0.776 | 0.933 | 1.247 |
| Fatigue | 0.365 | 0.733 | 1.100 | 0.698 | 0.982 | 1.011 |

**Table 188**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO Bulb-Tee 63 – group B**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.776 | 0.933 | 1.247 | 1.067 | 1.263 | 1.273 |
| Fatigue | 0.698 | 0.982 | 1.011 | 0.365 | 0.733 | 1.100 |

**AASHTO Bulb-Tee 72 girder bridges in group B.** The effects of sugarcane truck loads on bridge deck with seven AASHTO Bulb-Tee 72 girders, designed for HS20-44 truck loads were presented in tables 189 and 190. The load combination "Strength I max" was used to determine the short term effect of bridge deck while load combination "Fatigue" was used to determine the long term effect. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

Considering the short term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 1.08 and 1.54 in the transverse direction. The ratio of shear stress was 1.13. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.75, and 0.93 in the transverse direction; the ratio of shear stress was 1.21. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.75 and 0.93 in the transverse direction. The ratio of shear stress was 1.21. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 1.08, and 1.54 in the transverse direction; the ratio of shear stress was 1.13. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the transverse and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

Considering the long term effect of bridge deck, at the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.32 and 0.82 in the transverse direction. The ratio of shear stress was 1.12. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was 0.65, and 0.98 in the transverse direction; the ratio of shear stress was 1.02. At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction was 0.65 and 0.98 in the transverse direction. The ratio of shear stress was 1.02. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction was

0.32, and 0.82 in the transverse direction; the ratio of shear stress was 1.12. Those ratios which were greater than 1.1 may cause the bridge decks to experience cracks in the all three directions. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or sugarcane truck loads may differ from each other. The difference is what makes the ratio of sugarcane to HS20-44 truck for some cases to be less than 1.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, no matter whether the stresses are in the tension zone or the compression zone.

We should consider that the locations of maximum stresses due to HS20-44 or Sugarcane truck loads may differ. Further research should be applied to obtain the ratios of the stresses at same location to evaluate the deck behavior under heavy truck loads.

**Table 189**
**Effects of sugarcane truck loads on top surface of bridge deck - girder type AASHTO Bulb-Tee 72 – group B**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 1.079 | 1.541 | 1.125 | 0.746 | 0.933 | 1.208 |
| Fatigue | 0.324 | 0.824 | 1.123 | 0.646 | 0.977 | 1.017 |

**Table 190**
**Effects of sugarcane truck loads on bottom surface of bridge deck - girder type AASHTO Bulb-Tee 72 – group B**

| Load | Ratio of Max Value of Stress of Sugarcane to HS20-44 | | | | | |
|---|---|---|---|---|---|---|
| Combination | Max Tensile Stress | | | Max Compressive Stress | | |
| | Longitudinal | Transverse | Shear | Longitudinal | Transverse | Shear |
| Strength I Max | 0.746 | 0.933 | 1.208 | 1.079 | 1.541 | 1.125 |
| Fatigue | 0.646 | 0.977 | 1.017 | 0.324 | 0.824 | 1.123 |

## End and Intermediate Diaphragms

The diaphragm is defined to be a transverse stiffener, which is provided between girders in order to maintain section geometry. It has been thought to contribute to the overall distribution of live loads in bridges. The diaphragms normally were used to contribute to the overall distribution of live loads in bridges. In this study, the full-depth diaphragms were placed at both ends of bridge supports, and two full-depth intermediate diaphragms were used at the distance of 40 feet from the supports. The ratios of axial force of each diaphragm were obtained to evaluate the short term and long term effects of sugarcane truck load on the diaphragms. Similar to the stress and deflection analysis, the load combination “Strength I max” governs the short term effects; while “Fatigue” governs the long term effects on the diaphragms. Under the load combination “Fatigue,” the ratios were significantly greater than those under the load combination “Strength I max,” which means for the long term effects of sugarcane truck loads, the diaphragms could experience more cracking, which would require additional inspections along with early and frequent maintenance.