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Structural and economical impacts of heavy truck loads on bridges

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STRUCTURAL AND ECONOMICAL IMPACTS OF HEAVY TRUCK LOADS ON BRIDGES

By

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ABSTRACT

The study included in this dissertation assesses the strength, serviceability, and economic impact of overweight trucks on Louisiana bridges. Truck load configurations FHWA 3S2 and FHWA 3S3 were applied to bridge models that were originally designed for HS20-44 truck configuration to determine the effects of heavy truck loads on bridges. Behaviors of bridge components including bridge girder, deck, and diaphragm were evaluated separately.

AASHTO linear approach and finite element analysis were employed to evaluate bridge girder behaviors under the heavy truck load. Bridge models with different geometric configurations were considered. Both short term and long term effects on simple span and continuous bridges were determined based on AASHTO LRFD specifications. Results indicated that the AASHTO linear approach was more conservative than the finite element approach. Results based on finite element analysis showed that the short term effect of heavy truck load on selected bridge models was limited, while the long term effect was significant.

Finite element analysis was used to perform the bridge deck evaluation. Longitudinal, transverse, and shear stress states at top and bottom surfaces of decks were obtained and evaluated. The researcher determined that bridge decks were overstressed and might experience cracks.

Statistical methods were introduced to this study in order to evaluate stress data of bridge decks. Due to the determinacy of results from finite element models, a modified factorial experiment with crossed treatment factors was created to perform the probability based statistical analysis. The sequence of significance of analysis parameters was observed. Effects of bridge girder types on deck stress performances were discovered under different bridge geometric and truck load configurations.

The diaphragm behaviors were assessed based on ratios of axial forces. The effects of heavy truck load on diaphragms were determined limited even though the ratio exceeded the criteria, since the values of axial forces were not large.

The methodology employed in the evaluation of 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) FHWA 3S2 truck with maximum GVW of 108,000 lb; b) FHWA 3S3 truck with maximum GVW of 120,000 lb; and c) FHWA 3S3 truck with GVW of 100,000 lb. with uniformly distributed load.

The researcher recommends that a) for bridges on the routes of timber, lignite coal, and coke fuel transporting, do not increase the GVW to 108,000 lb. to avoid the high bridge fatigue cost; b) for bridges on the routes of sugarcane transporting, truck configuration FHWA 3S3 is suggested to be used to haul sugarcane with GVW of 100,000 lb. uniformly distributed. This configuration 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 GVW of 120,000 lb., which will result in high fatigue cost on the network.

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CHAPTER I

INTRODUCTION

1.1 General

The rapid growth of the economy has lead to a rapid growth in the number of heavy vehicles in service, as well as a dramatic increase in the size and weight of heavy vehicles. The tug-of-war between the demand of increasing the truck weight to get more carrying capacity and reducing the risk and rehabilitation costs of the bridges has existed for a long time. Therefore, evaluating the bridge characteristics under heavy truck loads is necessary and important.

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 can be 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, or GVW 83,400 lb, at certain period during the year. Furthermore, the state legislature released the restriction to 100,000 lb for several kinds of the trailers with a nominal permit fee to meet the increasing growth of economy. Because highways have traditionally been designed for the legal load of 80,000 lb., permitted trucks of 100,000 lb., or even heavier than 100,000 lb., decrease the expected service life of the infrastructure. The results are 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 Gross Vehicle Weight (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 gross vehicle weights 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.

Prestressed and cast-in-place concrete girders slab bridges are the most common type of highway systems used in United States. The main infrastructure includes the girder, deck and diaphragm as the most important components. To evaluate the effects of heavy truck loads on bridges, the behavior of those components must be investigated.

As part of the on-going effort to determine the bridge behavior under overload trucks, the Louisiana Department of Transportation and Development and Louisiana Transportation Research Center had co-sponsored several task research programs at

Louisiana Tech University. Those programs included projects such as "The Effects of Hauling Timber, Lignite Coal, and Coke Fuel on Louisiana Highways and Bridges," in which the vehicle GVW was 108,000 lb.; and "Monitoring System to Determine the Impact of Sugarcane Truckloads on Non-Interstate Bridges," in which the vehicle GVW was 120,000 lb.. The research presented here included information from the above projects.

1.2 Research Objectives

The primary objective of this research is to assess the strength, serviceability, and economic impact of overweight trucks on Louisiana bridges. The detail evaluations for bridge girder, deck, and diaphragm must be performed to meet the goal. The GTSTRUDL finite element software was used to construct the 3-D model to simulate the response of the bridge components. The SAS statistic software was used to analyze the bridge deck data efficiently. This research program included the following activities:

- Conducted a background review on bridge behaviors, finite element modeling of bridge system and related areas.
- Investigated the typical AASHTO tee section bridge girder behaviors under the heavy truck load by simplified AASHTO line approach and finite element method.
- Investigated the bridge deck performance under the heavy truck load by finite element method.
- Conducted a statistic model and used it to evaluate the bridge deck behaviors.
- Investigated the bridge diaphragm performance under the heavy truck load.

- Constructed a bridge cost model and determine the long term effect on simple span and continuous bridges under the heavy truck load.

1.3 Organization of the Study

The background review relevant to the objectives of this research is presented in Chapter II. The methodology to construct the finite element model and influence line analysis for typical slab-on-girder concrete bridges is presented in Chapter III. The parametric studies and evaluations of typical bridge girders is presented in Chapter IV. Chapter V and VI include the studies of bridge decks by finite element analysis results and statistical methods. The bridge diaphragm performance under the heavy truck load is discussed in Chapter VII. Chapter VIII presents the studies of bridge cost model and the long term effects on remaining bridge life. The summary, conclusions, and recommendations for this study are presented in Chapter IX.

CHAPTER II

BACKGROUND REVIEW

2.1 Introduction

The heavy load caused by trucks has a great effect on the bridge system. The impact and the distribution of the live load now is becoming an important research area, especially in those states that are rich in agricultural and forest products. Several methods, including finite element analysis, long-term monitoring, and field experiment and so on, are used to do the investigation work. The literature review was used to investigate all aspects of the work that would be required to complete this study. The following five topics were identified as major areas where previous research information could be beneficial: (1) behavior of bridge under certain load combination, (2) finite element analysis and analytical modeling, (3) behavior of diaphragms, (4) the statistical method applied on bridge system evaluation, (5) experimental testing of bridges.

2.2 Behavior of Bridge Under Certain Load Combination

Background information on the development of wheel load distribution factors can be found in Hays et al. (1986), Sanders and Elleby (1970), and Stanton and Manock (1986). Chen (1995a, 1995b and 1995c) studied load distribution in bridges with unequally spaced girders. AASHTO empirical formulas for estimating live load distribution factors were compared to results from the refined method. Parametric studies were conducted with a number of field bridge examples that were simply supported, non-skewed, and had no diaphragms. Refined load distribution equations were proposed. Subsequent work by Chen and Aswad (1996) sought to review the accuracy of the formulas for live load distribution for flexure contained in the LRFD Specifications (AASHTO 1994) for prestressed concrete I-girder bridges. It was concluded that the use of a finite element analysis leads to a reduction of the lateral load distribution factor in I-beams when compared to the simplified LRFD guidelines. Fu et al. (1996) conducted comparable work by field testing four steel I -girder bridge structures under the effect of real moving truck loads. The results indicated that all the code methods, AASHTO and LRFD, produced higher distribution factors.

Khan (1996) summarized the historical developments in bridge design going back to 1938 with Newmark's distribution procedure where the whole slab of the bridge is considered to be an isotropic plate with no composite action with the supporting girders. A strip of slab is considered to be a continuous beam over flexible supports and moment distributions involving fixed-end moment, stiffness, and carry-over factors, analogous to continuous beam.

In 1986, Marx, Khachaturiian, and Gamble developed wheel load distribution equations using the finite element analysis of 108 simply supported skew slab-and-girder bridges. The research included models for bridge concrete deck and prestressed girders as eccentrically stiffened shell assembly.

El-Ali (1986) used the SAP-IV finite element program to study the wheel load distribution characteristics of simply supported skew bridges using the discretization scheme of Bishara (1986) where each I-beam girder was divided into two T-shaped beam elements, and elastic properties of these elements lumped at the centroid of the flanges. Truss systems were used to connect the two beam elements and the top beam element to the deck plate element. Such a procedure is very lengthy, and because of the limited scope of the study, no expressions were developed. In 1987, Nun, Zokaie, and Schamber analyzed multi-girder composite steel bridges using equivalent orthotropic plate and ribbed plate models and developed simplified equations that were modified and included in the 1994 AASHTO LRFD code.

Amiri (1988) did a finite element study on continuous composite skew bridges with prestressed girders and proposed some distribution equations based on linear elastic theory with a limited range of parameters specific to girder spacing.

Further revisions to load distribution equations were presented by Tarhini and Frederick (1995). Contrary to AASHTO assumptions, the finite element analysis revealed that the entire bridge superstructure acts as one unit rather than a collection of individual structural elements. The effect of cross bracing on the wheel load distribution factor was found to be negligible. The research correlated distribution factor results obtained from published field test data with the proposed formulae as well as the AASHTO method.

In A. S. Nowak, C. Eamon, and M. A. Ritter's research of "Structural Reliability of Plank Decks" (2001), they reported that given the LRFD code target reliability index is 3.5 is clear that in most cases the codes are overly conservative. That fact is primarily the result of two factors: an unrealistic load distribution model and flat-use factors that do not adequately predict plank capacity.

The current AASHTO LRFD Bridge Design Specifications (AASHTO 1998) provides a set of distribution factor formulas for estimating the distribution of bending moment and shear force effects in the interior and exterior girders of highway bridges. However, the LRFD Specifications impose strict limits on the use of its live-load distribution factor formulas.

In Paul J. Barr, Marc O. Eberhard, and John F. Stanton's (2001) research, by comparing 24 bridge models, they drew the conclusion that the live-load distribution factors calculated from the AASHTO LRFD Specifications (2^{nd} edition, 1998) were conservative. And the differences among the distribution factors from the various finite-element models were attributable to the presence of lifts, intermediate diaphragms, end diaphragms, and continuity, where continuity and intermediate diaphragms had less effect than others.

It is also proved by the results of Shin-Tai Song, Y. H. Chai, and Susan E. Hida (2003), in other conditions outside of the limits of LRFD specifications; the refined analyses using 3D models are required for design of bridges. And when the standard truck loading from the LRFD specifications was applied on box-girder bridges, the formulas from the LRFD specifications generally provide a conservative more estimate than those from the finite element analysis.

Harry Cohen, Gongkang Fu, Wassem Dekelbab, and Fred Moses (2003) raised a new method for modeling truck load spectra resulting from truck weight-limit changes, differentiating weight-out and cube-out truck traffic. The modeling was based on freight transportation behavior, and it was flexible for both across-the-board and local changes without restriction on the truck types to be impacted.

Hani H. Nassif, Ming Liu, and Oguz Ertekin (2003) studied in the Dynamic Load Factors by 3D analysis model; the results confirmed the experimental study by H. H.

Nassif and A. S. Nowak (1995) that the Dynamic Load Factors decreases as the static stress increases. For very heavy trucks, the Dynamic Load Factors did not exceed the theoretical results. They notified values of Dynamic Load Factors for the purpose of design should be based on those obtained from the most loaded interior girders. However, for situations where fatigue is the dominant mode of behavior, such as in connections, larger values of Dynamic Load Factors need to be considered.

Raid Karoumi (1996) did some research on the dynamic response of cable-stayed bridges under moving vehicles. He described a method of evaluating the response by idealizing the bridge as a Bernoulli-Euler beam on elastic supports with varying support stiffness. The analysis uses the mode superposition technique and calculates the response in time domain, utilizing an iterative scheme and providing a numerical example.

In AASHTO LRFD 1994 the load distribution factor is calculated by a new equation that is based on parametric studies and finite element analysis. However, this equation involves a longitudinal stiffness parameter, which needs an iterative procedure to correctly determine the LDF value. By the finite element analysis research of Elisa D. Sotelino, Judy Liu, Wonseok Chung, and Kitiapat Phuvoravan (2004), a simpler and sufficiently accurate equation for calculation of load distribution was given, and the longitudinal stiffness parameter and the slab thickness parameter that appear in the LRFD equation are implicitly embedded in the simplified expression, which dramatically decreases the work of the designer.

Stuart S. Chen, Amjad J. Aref, Il-Sang Ahn, Methee Chiewanichakorn, and Aaron F. Nottis (2003) used the experimental method to discover the behavior at service and ultimate loads of the continuous composite bridge. By the ¼ scale two span bridge specimen, they got that the results from the service limit test closely resembled the values attained through elastic analysis. Under ultimate loading, good post-yield behavior was observed. The researchers found good correlation with previously-developed finite element analysis predictions for the specimen behavior under selected test protocol loads.

In 2006 Structures Congress, Zaher Yousif and Riyadh Hindi presented the research of a comparison between the moments distribution factors of concrete bridges due to live load calculated in accordance with the AASHTO-LRFD (2004) formulas and finite-element analysis and gave the recommendations for specific bridge geometries of bridges built with AASHTO-PCI girders. Several three-dimensional linear elastic models were built using the structural analysis program SAP2000 to obtain the most accurate method to model the bridge superstructure.

In the recent research of Mayrai Gindy, Hani H. Nassif, and Joe Davis (2003), they tried to find a methodology for comparing the optional live load deflection limit with simulated deflections, validated using actual field deflection measurements, extrapolated to a 75-year level. They used a long-term defection-measuring system to measure the maximum girder deflection. A computer model based on the semi-continuum method was also developed and verified using a test truck of known weight and axle configuration. A weigh-in-motion system to record the actual live load information, which is used to develop statistics regarding normal truck traffic, was applied. Then the Monte-Carlo simulation technique is used to simulate truck traffic and predict the 75-year maximum girder deflection. The optional code defection limit in terms of a reliability index was evaluated by structural reliability theory.

P. J. Barr and MD. N. Amin (2006) used a full scale, single lane test bridge to evaluate a typical slab-on-girder bridge's response to shear force. The results of the shear load test provided the means to evaluate the level of detail for a finite element model that is required to accurately replicate the behavior of bridges subject to shear loads. More than 200 finite element bridge models were evaluated in the study. The finite element shear distribution factors were compared with those calculated according to the AASHTO LRFD specifications. It was found that the AASHTO LRFD procedure accurately predicted the shear distribution factor for changes in girder spacing and span length. However, the LRFD shear distribution factor for the exterior girder was found to be unconservative for certain overhang distances and overly conservative for the interior girder for higher skew angles.

Erin Hughs and Rola Idriss (2006) evaluated the shear and moment live-load distribution factors for a new, prestressed concrete, spread box-girder bridge. The shear and moment distribution factors were measured under a live-load test using embedded fiber-optic sensors and used to verify a finite element model, which was then loaded with the AASHTO design truck and used to calculate the maximum girder distribution factors and compared to those calculated from both the AASHTO standard specifications and the AASHTO LRFD bridge design specifications. Results indicated that for the study bridge, the LRFD specifications would result in a safe design, though exterior girders would be overdesigned. The standard Specifications, however, would result in an unsafe design for interior girders and overdesigned exterior girders.

2.3 Finite Element Analysis and Analytical Modeling of Bridge System

Bakht (1988) reported on a simplified procedure by which skewed bridges could be analyzed to acceptable design accuracy using methods originally developed for the analysis of straight bridges. The study concluded that beam spacing, in addition to skew angle, is an important criterion when analyzing a skew bridge as right. Results from an error analysis using experimental data indicated that the process of analyzing a skew bridge as equivalent straight bridge is conservative for longitudinal moments but is unconservative when dealing with longitudinal shears.

Jaeger and Bakht (1982) initially discussed the use of grillage analogy to conduct bridge analyses. A very detailed explanation of the theory and application was included. Wilson (1996) also examined the use of finite element models in conducting three-dimensional dynamic analyses of structures. Special emphasis was placed on dynamic analysis for earthquake engineering.

The lateral stability of prestressed girders was investigated by Saber (1998). The analyses were for long span simply supported non-skew bridges. The results indicated that the AASHTO 1996 recommendation for T-girder construction, of one intermediate diaphragm at the point of maximum positive moment of spans in excess of 40 feet, is conservative.

Barth and Bowman (1999) studied the effect of diaphragm details on the service life of bridges and found that even though some fatigue cracking might occur in certain locations, it did not reduce the service life of the bridge.

Helwig and Frank (1999) found that the bracing behavior of the shear diaphragms was significantly affected by the type of loading and noticed that procedures based upon uniform moment solutions often overestimate the capacity of diaphragm-braced beams.

Barr et al. (2000) evaluated live-load distribution factors by testing a series of three-span, prestressed concrete girder bridges and comparing to AASHTO and finite element analysis. It was found that lifts, end diaphragms, skew angle, and load type significantly decreased the distribution factors, while continuity and intermediate diaphragms had the least effect.

Yazdani and Green (2000) studied the performance of elastomeric bearing pads in precast concrete bridge girders using a parametric study on the interaction of support boundary conditions and bridge girders. The researchers found that intermediate diaphragms have the positive effect of reducing the overall midpoint deflections and maximum stresses for the bridge system, but the reductions in deflections and stresses were smaller for increasing skew angles.

L. Kwasniewski, M.M. Szerszen, and A.S. Nowak (2000) tried to create an accurate finite element model and tested the model to calibrate the parameters. They reported that boundary conditions and modulus of elasticity of concrete were the most important parameters in the modeling of the actual bridge under service load. Partial constraints at the supports can be modeled using spring element. But when researchers try to estimate load-carrying capacity, models without springs and with noncomposite action should be used.

After using non-linear finite element models to study the behavior of real segmental box girder bridges, G. Rombach and A. Specker (2000) reported that the behavior of such type of structure is dominated by the un-reinforced joints. The indentation of the shear keys can be neglected in the numerical model if the structure is loaded by bending only.

The complexity of reinforced concrete is a major factor that limits the capabilities of the finite element method, so to get an accurate finite element analysis model is very important. Chen and Aswad (1996), Mabsout et al. (1997), and Paul J. Barr et al (2001) did the investigation work, and finally Paul J. Barr et al (2001) used the frame element, shell element, and rigid link to build the model and had a better effect.

B. M. Kavlicoglu, F. Gordaninejad, M. Saiidi, and Y. Jiang (2001) did the comparative research with analysis and testing of graphite/epoxy concrete bridge girders under static loading. The results showed the use of steel stirrups as shear connection elements for composite/concrete bridge girders worked effectively, and if using proper assumptions, it was possible to model the behavior of the new graphite/epoxy/concrete girder.

Kuan-Chen Fu and Feng Lu (2003) built the nonlinear finite-element analysis model to investigate the importance of the nonlinear behavior of the concrete to the highway bridge design. The performance of the numerical model is far better than the current design method. The procedure may cover several types such as box girder bridges, cable-stayed bridges, and suspension bridges where concrete deck is constructed as an integral part to provide composite action.

Christopher Higgins (2003) used LRFD orthotropic plate model to determine the deflection and live load moment in filled grid decks. He reported a closed-form solution to the orthotropic plate problem under multiple patch loads and introduced the maximum moment envelopes for multiple load patch cases, stiffness ratios, span lengths, and grid orientations. After the finite element analysis, he notified the presented equations might significantly simplify calculation of maximum live load moment for these types of deck systems and facilitate design.

2.4 Behavior of Diaphragms

The National Cooperative Highway Research program (NCHRP) Project No.12-26, which produced the truck load distribution factors for the AASHTO LRFD Specifications (AASHTO 1994), assumed diaphragms and cross-frames had an insignificant effect on load distribution. Despite this acknowledgment, AASHTO still requires the inclusion of diaphragms at points of maximum moment for spans over 12.20 m (40-ft).

Gustafson (1966) performed the analysis of slab and girder bridges using the finite element method. The investigation by Sithichaikasem (1972) included the effects of the torsional stiffness and warping stiffness of the girders and the effects of in-plane forces in the slab. The study recommended that interior diaphragms be eliminated from most prestressed I-beam bridges unless they are required for erection purposes. The results of the study, by Wong and Gamble (1973), on the effects of diaphragms on load distribution of continuous, straight, right slab, and girder highway bridges reported the following. The diaphragms may improve the load distribution characteristics of some bridges that have a large ratio of beam spacing to span; the usefulness of the diaphragms is minimal, and they are harmful in most cases. Based on cost effectiveness, the authors recommended that diaphragms be omitted in highway bridges. The study recommended that interior diaphragms be eliminated from most prestressed I-beam bridges unless they are required for erection purposes. One of the arguments that have been raised for using diaphragms in bridges is that diaphragms help limit damage to an overpass structure that is struck transversely from below by an oversized load. There appears to be conflicting evidence as to whether the diaphragms are damage limiting or damage spreading members. However, no analyses were reported relevant to such a claim, and the analyses mentioned above were all performed on simply supported bridges.

Sengupta and Breen (1973) studied the effect of diaphragms in prestressed concrete girder and slab bridges by varying span length, skew angle, stiffness, location, and number of diaphragms. It was found that interior diaphragms were good only to distribute the load more evenly while never significantly reducing the governing design moment. The conclusion reached was that it is more economical to provide increased girder strength than to rely on improved distribution of load due to provision of diaphragms. Furthermore, the distribution factors of the 1969 AASHO specifications for live loads were found to be conservative even without diaphragms. Also, interior diaphragms made the girders more vulnerable to damages from lateral impacts. It was recommended that interior diaphragms should not be provided in simply supported prestressed concrete girder and slab bridges, and that provision of exterior diaphragms was considered necessary for reliable serviceability.

Cheung et al. (1986) reported on the apparent lack of previous research to deal with the actual increases or decreases of longitudinal moments due to diaphragms. For example, most published papers concentrated on the alleged effectiveness, or lack of a particular arrangement of diaphragms. Kennedy and Soliman (1982) had reached similar conclusions four years earlier. Based on experimental findings and parametric studies using the finite element method, reearchers observed that the effective moments of resistance along failure yield lines in the positive and negative moment regions depended on the position of the load and on the nature of the connection between the transverse steel diaphragms and the longitudinal steel beams or girders.

Much of the present design criteria on load distribution are based on the results of simply supported bridges. The provisions for the design of negative moment regions are inferred from the behavior of the positive moments. It is difficult to make direct comparison between the results of an analysis of a simply supported bridge and continuous bridge because of the effective span length due to the negative moment at the interior support. Since most highway bridges are continuous bridges, analyses of the effects of diaphragm on continuous bridges will undoubtedly provide new data and supplement the data on the design of slab and girder bridges.

Kostem and deCastro (1977) studied the effect of diaphragms on the lateral distribution of live load in simple-span non-skewed beam-slab bridges with prestressed concrete I-beams. Based on a finite element analysis of two bridges with spans of 21.8 and 20.9 m (71.5 and 68.5 ft), they found that reinforced concrete diaphragms contribute only about 20 to 30 percent of their stiffness to load distribution. They also found that when all design lanes are loaded, the contribution of the diaphragms is negligible. When maximum bending moments are produced, an increase in the number of diaphragms along the span does not necessarily correspond to a more even distribution of loads at midspan. It was also found that, if all the design lanes are loaded, the contribution of diaphragms is negligible regardless of the number of diaphragms used. A recommendation was made that vehicle overload and large skew effects be considered before eliminating the use of intermediate diaphragms.

In 1983, Kennedy and Grace studied the effects of diaphragms in skew bridges subjected to concentrated loads. They concluded that diaphragms enhance the distribution of point loads specifically in bridges with large skew angles.

Griffin, J.J. (1997), researched the influence of intermediate diaphragms on load distribution in prestressed concrete I-girder bridges. The studies included two bridges that were constructed with a 50 degree skew angle along the coal haul route system of Southeastern Kentucky. One of the bridges has concrete intermediate diaphragms, while the other bridge has no intermediate diaphragms. Bridges of similar design along coal haul routes have experienced unusual concrete spalling at the interface of the diaphragms and the bottom flange of the girders. The intermediate diaphragms appeared to be contributing more to the increased rare of deterioration and damage than reducing the moment coefficient and distributing the traffic loads. Experimental static and dynamic field testing was conducted on both bridges. All field tests were completed prior to the opening of the bridges. Once the calibration of the finite element models was completed using the test data, analyses were conducted with actual coal haul truck traffic to investigate load distribution and the cause of the spalling at the diaphragm-girder interface. Based on the results obtained in the research study, a significant advantage in structural response was not noted due to the presence of intermediate diaphragms. Although large differences were noted percentage wise between the responses of the two bridges, analyses suggested that the bridge without intermediate diaphragms would experience displacements and stresses well within AASHTO and ACI design requirements. The finite element analyses also revealed the cause of concrete spalling witnessed in the diaphragm-girder interface region. The tendency of the girders to separate as the bridge was loaded played a large role in generating high stress concentrations in the interface region. Other mitigating factors were the presence of the diaphragm anchor bars and the subjection of the bridge to the overloads of coal trucks. However, the total elimination of intermediate diaphragms was not recommended since they were required during construction and would be needed in the event the deck was to be replaced. The use of steel diaphragms was recommended as substitutes for the concrete intermediate diaphragms.

Abendroth et al. (1995) summarized research conducted by various investigators, and they reported that Sengupta and Breen investigated the role of end and intermediate diaphragms in typical prestressed concrete girder and slab bridges in 1973. Experimental variables in that study included span length; skew angle of the bridge; and number, location, and stiffness of the diaphragms. The elastic response of the bridge was studied under static, cyclic, and impact loads-with and without diaphragms. Overload and ultimate load behavior was also documented from various static load and impact load tests. Experimental results were used to verify a computer program, which in turn was used to generalize some of the results. Sengupta and Breen concluded that under no circumstances would the presence of intermediate diaphragms significantly reduce the design girder moments. In fact, in certain situations the presence of intermediate diaphragms might even increase the design moment. A recommendation that intermediate diaphragms be excluded in prestressed concrete girder and composite slab bridges was made. Abendroth et at. (1995) cited other research by Kostem and deCastro which found that when all traffic lanes were loaded, diaphragms were ineffective in distributing loads laterally. Based upon independent research work, Abendroth et al. a(1995) studied the effect of overweight vehicles on diaphragms of prestressed concrete bridges. Cases with and without diaphragms were investigated using the finite element method considering both pinned and fixed-end conditions. It was found that vertical load distribution is independent of the type and location of the intermediate diaphragms; however, the horizontal load distribution was a function of the intermediate diaphragm type and location. It was also shown that construction details at the girder supports created considerable rotational-end restraint for both vertical and horizontal loading. They also found that fabricated intermediate structural steel diaphragms provided the same type of response as reinforced-concrete intermediate diaphragms used by the Iowa Department of Transportation.

2.5 Previous Studies on Truck Weight Regulations

The truck industry is faced with the demand of increasing the truck weight to get more carrying capacity. On the other hand, bridge owners can control the loading on the bridges to limit the deterioration of the existing bridge infrastructure in the United States to keep the structure in a safe condition. To solve this problem, regulations allow the truck weights to increase to a certain range while guaranteeing the safety and serviceability of the bridge systems. The Federal legislation known as Federal-aid Highway Act introduced a program regulating truck weights. This legislation restricts the gross weights of trucks and weights of different axles and axle groups. The maximum gross weight of the vehicle is 80,000 lbs., while the limit for the single axle load is 20,000 lbs. and 34,000 lbs. for the tandem axles.

The axle group weights are regulated based on the truck weight formula, also known as "Formula B," given by

$$W = \frac{BN}{2(N-1)} + 6N + 18 \tag{2.1}$$

Where, W is the overall gross weight (Unit: lbs)

B is the length of the axle group (Unit: ft)

N is the number of axles in the axle group

By using the Formula B, the overstressing of the bridges with an HS20 design load can be avoided by more than five percent and the bridges with an H15 design load can be avoided by more than 30 percent.

This formula is based on the principle that overstressing H15 bridges by 30 percent is still acceptable for bridge safety and serviceability. Most of the H15 bridges are built on low heavy-truck volume highways while the HS20-44 bridges are usually built on interstate highways. This fact means that if the bridge is overstressed by more than 5 percent, a high risk exists. However, engineering experiences in some states and the province of Ontario show that the results of Formula B are very conservative. Many states have increased their legal loads above the standard. For example, Minnesota allows a winter increase in GVW of 10 percent during dates set by the transportation commissioner based on a freezing index. Michigan allows loads up to 154,000 lbs., and most western states allow loads up to 131,000 lbs.

The Federal Highway Administration (FHWA) supported research to develop another truck weight formula known as the TTI formula, which is based on the same overstressing criterion as the Formula B. Compared to the Formula B, the TTI formula allows higher weights for shorter vehicles, tandem, and tridem axle groups, but it allows smaller gross weights than Formula B for longer vehicles.

The TTI formula is given by

$$\begin{aligned} W &= 34 + B \text{ (Kips)} && \text{for } B < 56 \text{ ft} \\ W &= 62 + 0.5B \text{ (Kips)} && \text{for } B > 56 \text{ ft} \end{aligned} \tag{2.2}$$

In 1990, the Transportation Research Broad (TRB) finished research on a modification of the TTI formula, which reduced the limits on axle loads and allowed the higher gross weights. However, the modified TTI formula established stress limits on the bridges whose design load is the HS20 truck load without consideration of the H15 truck load; the modified formula is given by:

$$\begin{aligned} W &= 26 + 2.0B \text{ (Kips)} && \text{for } B < 23 \text{ ft} \\ W &= 62 + 0.5B \text{ (Kips)} && \text{for } B > 23 \text{ ft} \end{aligned} \tag{2.3}$$

2.6 Statistical Method for Bridge Analysis

Various analytical and experimental methods have been used to analyze the load distribution and deck behavior in highway bridges. Assumptions are made to simplify the problem and postulate a manageable solution. Some statistics methods are also introduced into the procedure to reduce the high amount of data analysis work. And recently, engineers and academics have become much more interested in this field.

The results of the study, by M. Ghosn (2000), on the truck weight regulations using the bridge reliability model, reviewed the historical truck regulations, which maintained controls on axle and gross weights with legal load formulas based on limiting allowable stresses in certain types of bridges. The stress limitations do not usually lead to consistent or defensible safety levels and also ignore the cost impact of the weight regulations. He illustrated how new truck weight regulations can be developed to provide acceptable safety levels, which were derived from the AASHTO bridge evaluations. The reliability indices were used to relate the statistics of bridge load effects. A sensitivity analysis was also performed to study the effect of errors in the database. The results demonstrated that the proposed formula is not sensitive to the assumed database if the target safety index is changed accordingly.

J. A. Laman, J. S. Pechar and T. E. Boothby (1999) did an experiment to evaluate the statistics of dynamically induced stress levels in steel through-truss bridges as a function of bridge component type, component peak static stress, vehicle type, and vehicle speed determined the dynamic load allowance for each of the instrumented bridge components for each of several truck crossings. The study examined that the DLA data was a function of component type, component location, and truck type, number of axles, truck speed, and truck direction. The DLA is dependent on truck location, component location, component type, and component peak static stress but appears to be nearly independent of vehicle speed. And the normal traffic conditions best reflected the variation of truck loading conditions and variables that induced the dynamic effects in the bridge members.

M. Schwarz and J. A. Laman (2001) studied the response of prestressed concrete I-girder bridges to live load. They presented the results of field tests conducted on three prestressed concrete I-girder bridges to obtain dynamic load allowance statistics, girder distribution factors, and service level stress statistics. They measured the bridge response at each girder for the passage of test trucks and normal truck traffic and observed that the dynamic amplification was a strong function of peak static stress and a weak function of vehicle speed and was independent of span length, number of axles, and configuration. The field based data were also compared to numerical model results and results were closely aligned.

An application of field testing for an efficient evaluation and control of live load effects on bridges was described by Nowak, A. S., Eorn, J. and Sanli, A. (2000). A procedure for measuring live load spectra on bridges was developed in the experiment. Truck weight was measured to determine the statistical parameters of the actual live load. They measured the strain and stress in various components of girder bridges to determine component-specific load, and verified the minimum load-carrying capacity by proof load tests. Authors drew the conclusions that the live load effects were strongly site specific and component specific; the measured strains were lower than analysis results; and the dynamic load factor decreased with increasing static load effect. The proof load test results indicated that the structural response was linear. Compared with the code, the dynamic load factor and girder distribution factor from the experiment were lower.

Mabsout, M. E. et al. (1998) used the finite element method to study the effect of continuity on wheel load distribution factors for typical two equal span, two lane, straight, composite steel girder bridges. The influence on the bridge continuity was investigated. They observed that interior girders carried more live loads than the outside girders. Results of two finite element modeling techniques were used to predict wheel load distribution factors, which were similar to the results from AASHTO LRFD manual but less than the old AASHTO formula.

Fafard, M. et al. (1998) investigated the effect of dynamic loads on the dynamic amplification factors of an existing continuous bridge. An 3-D analytical model to idealize the vehicle and a FE model to analyze the bridge were developed, and the results were compared with the experimental testing results. Researchers concluded that current design codes tended to underestimate dynamic amplification factors, especially for long span continuous bridges.

The development of statistical models for wood bridge structures was discussed by Eamon, C. et al. (2000). The statistical methods were used to develop rational models for loads and resistance. Reliability was used to measure structure performance, which also provided a rational basis for comparison of wood and other structural materials. The authors determined the structural reliability of selected wooden bridges designed by AASHTO codes and identified the inadequacies in load distribution and material resistance in the current specifications.

Petrou, M. F., Perdikaris, P. C. and Duan, M. (1996) studied the static behavior of noncomposite concrete bridge decks under concentrated loads. Three kinds of decks with three different reinforcement combinations were applied in the experiments. The load-deflection diagrams, cracking and yielding load level, failure mode, and some other results were observed.

Issa, M. A. (1999) investigated the cracking behaviors in concrete bridge decks at early ages. Survey, experimental work and analytical study were performed to reach the goal. Author indicated that in most cases, cracking of concrete might be attributed to the high evaporation rate and high magnitude of shrinkage. Some other factors, like high slump concrete, excessive water in the concrete, insufficient top reinforcement cover, and so on, were also the key causes of the cracking.

The effects of material properties on cracking in bridge decks were observed by Schmitt, T. R. and Darwin, D. (1999). The information collected from construction documents and field books was compared with observed levels of cracking to identify correlations between cracking and the variables studied. The results of the evaluation indicated that cracking in monolithic bridge decks increases with increasing values of concrete slump, percent of concrete volume occupied by water and cement, water content, cement content, and compressive strength, and decreasing values of air content.

Boothby, T. E. and Laman, J. A. (1999) evaluated the cumulative damage caused by vehicle loading to bridge concrete deck slabs. An analytical model was implemented to evaluate the effect of user loads on a statistical sample of bridge deck slabs while an extensive literature review was conducted to determine the state of the art for cumulative deck slab damage evaluation. Authors found the relationship between environmental and mechanical factors in bridge deck deterioration is very important; the damage to bridge decks due to mechanical loading is insignificant compared to environmental factors.

Smith-Pardo, J. P. et al. (2006) reported a parametric study about distribution of compressive stresses in transversely posttensioned concrete bridge decks. According to the study results, authors reported that the distribution of compressive stresses is mainly affected by the support conditions of the girders and the axial stiffness of the diaphragms.

Nowak, A. S. et al. (2000) researched the dynamic loads due to truck traffic on bridges. Both of analytical and experimental studies were applied. The simulation and field measurements indicated that the design dynamic load could be reduced, and the values offered by codes were conservative.

Nassif, H. H. et al. (2003) did some study on the model validation for bridge-road-vehicle dynamic interaction system. The study provided an alternate method for the development of live load models for bridge design and evaluation by a 3-D computer model that was based on the grillage approach and was applied to four steel girder bridges.

A reliability based fatigue evaluation of bridge girders was performed by Szerszen, M. M. and Nowak, A. S. in 1999. Authors presented the reliability analysis for steel and concrete bridge girders. The analysis is based on live load measured in conjunction with field tests, and it is described in the load model. The reliability index was calculated due to service life of the bridge.

DePiero, A. H. et al (2002) focused the research on details of finite element modeling of bridge deck connection. A study for assessing the loading conditions for the connection details on the bridge was performed, and the results showed significant variation in connection detail stress range, depending on the detail's longitudinal and lateral location.

Kwasniewski, L. et al. (2000) did the sensitivity analysis for slab-on-girder bridges via finite element method and reported the following parameters influence the bridge structural reliability: live load, material data, boundary conditions for the girders, and interaction between girders and the deck slab, in which the boundary condition for girders was the important factor and difficult to model.

2.7 Experimental Testing

Aktan el al. (1992) reported on the use of known weight trucks to obtain static bridge response as a basis for nondestructive bridge evaluation (NDE). Experimental data taken from the static and dynamic testing of the bridge were used to calibrate a finite element model. A prestressed flat slab bridge was tested by Cook el at. (1993). The experimental and analytical research was conducted with the primary objectives of testing the bridge for service, fatigue, and ultimate loads; developing analytical models to predict the performance of the system, and verifying the analytical results by comparing them with those obtained from experimental data. In Helba and Kennedy (1995), equations for the design and analysis of skew bridges were developed from the analysis of a prototype composite bridge subjected to Ontario-Highway Bridge Design Code (OHBDC) truck loading. One conclusion drawn from the study was that rigidly connected diaphragms produce a significant increase in the ultimate load capacity of the bridge.

Craig el at. (1994), noted that even when the wheel line was closer to the fascia stringer than to the first interior beam, strains measured on fascia stringers under decks with integral curbs were significantly lower than those measured on the first interior beam.

Law et al. (1995a) studied the effect of local damage in the diaphragm on the first modal frequency. Three types of damage were studied that constituted a reduction in the stiffness of the diaphragm(s). The study concluded that there was no noticeable change in the first modal frequency in all three cases. Law et al. (1995b) furthered the work with model tests and measurements of 13 full scale bridges. Similarly, Paultre et al. (1995) initiated a study with the following main objectives: 1) evaluating the dynamic amplification factor for different highway bridges, 2) calibrating finite element models of the bridges being tested, and 3) examining the effects of changes in the stiffness of structural elements and the influence of secondary structural elements on the dynamic response. Data from the tests demonstrated that the dynamic amplification factor could be influenced by variables such as the vehicle speed and the ratio of the vehicle weight to the total weight of the structure.

CHAPTER III

METHODOLOGY OF FINITE ELEMENT ANALYSIS

3.1 Introduction

The methodology used in the finite element analysis phase evaluated the effect of the heavy loads on the bridges from the trucks transporting heavy 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.

The effects of heavy truck load on 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 two types of certain FHWA truck configurations. 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 heavy 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 heavy truck loads used in analysis were based on the FHWA 3S2 truck configuration and FHWA 3S3 truck configuration, with the same steering axle of 12,000 lb., and maximum tandem load of 48,000 lb., maximum tridem load of 60,000 lb., respectively.

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 3S2 truck, 3S3 truck, and the design truck (HS20-44) for stresses were calculated. The serviceability criteria were evaluated based on their deflections.

3.2 Analysis Variables

Under the whole analysis procedure, seven variables were considered as the design factors:

(1) Bridge Width;

(2) Slab Thickness;

(3) Girder Type;

(4) Girder Spacing;

(5) Span Length;

(6) Bridge Skew Angle;

(7) Diaphragm Condition;

(8) Bridge Support Condition;

(9) Truck Loads on the Bridge.

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 0^{o} diaphragms. There were two types of supports of the bridge structures: simply supported, or three equal spans continuous. Based on the girder numbers, the models were divided into two groups. The structures of both groups analyzed in this study were thirty feet wide. For first group, the girders were spaced at eight feet in the middle and seven feet on the outside. The model contained five AASHTO Type IV, V, VI or Bulb-Tee 54, 63, 72 girders with the slab thickness kept eight inches as the constant; for the second group, the girders were spaced at 5/4.5 feet in the middle and six 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 eight inches as the constant. The geometry of the bridge and its deck are shown in Fig. 3.1 to Fig. 3.3.

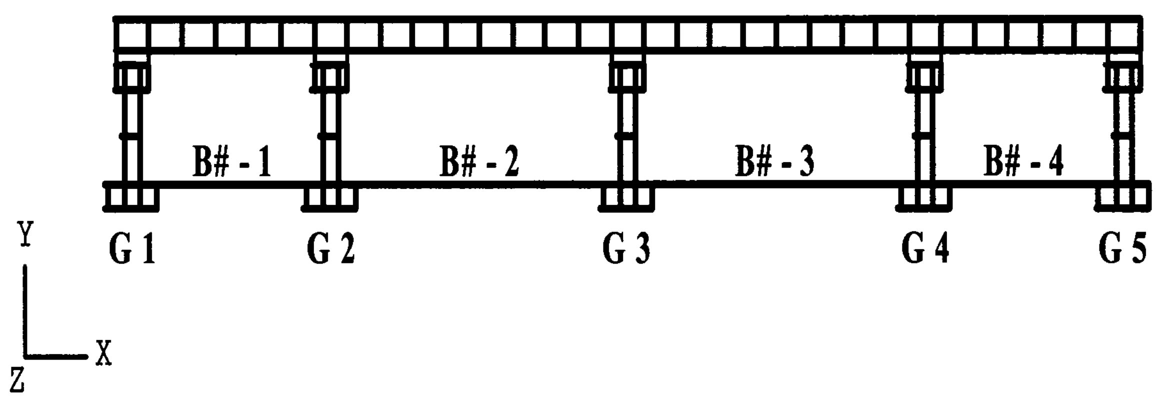


Fig. 3.1 Models Used for Bridge Analysis – Five Girders Model

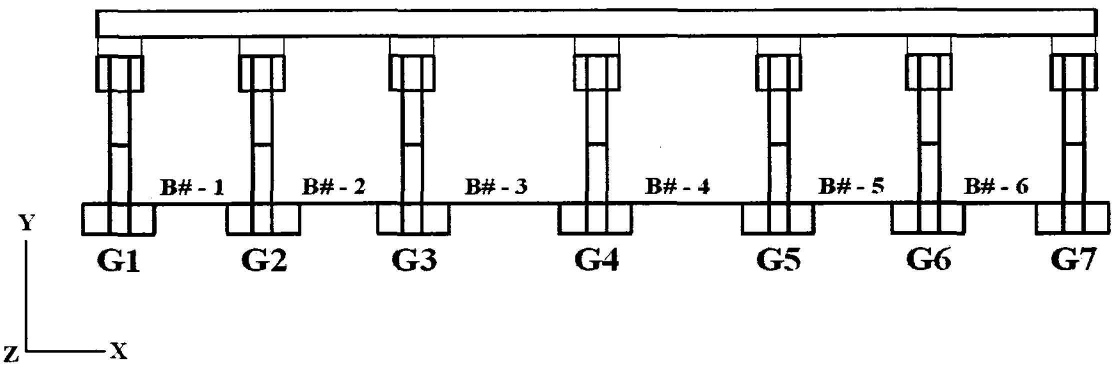


Fig. 3.2 Models Used for Bridge Analysis – Seven Girders Model

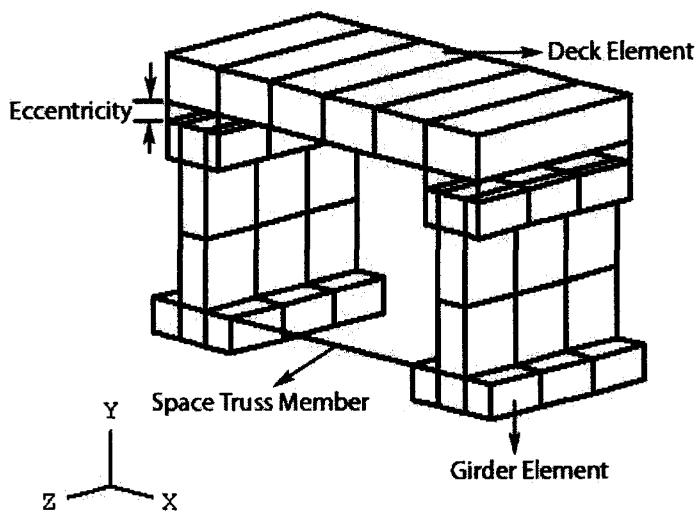


Fig. 3.3 Typical Plate and Girder Elements

The heavy truck loads used in analysis were based on the FHWA 3S2 truck configuration, with maximum tandem load of 48,000 lb. and steering axle of 12,000 lb.; and FHWA 3S3 truck configuration, with maximum tridem load of 60,000 lb. and steering axle of 12,000 lb. All truck loads were placed on the bridge as shown in Fig. 3.4 to Fig. 3.6.

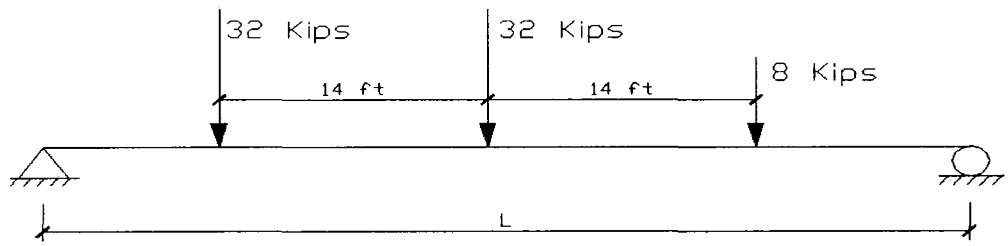


Fig. 3.4 AASHTO HS20-44 Truck Configuration with GVW=72 Kips

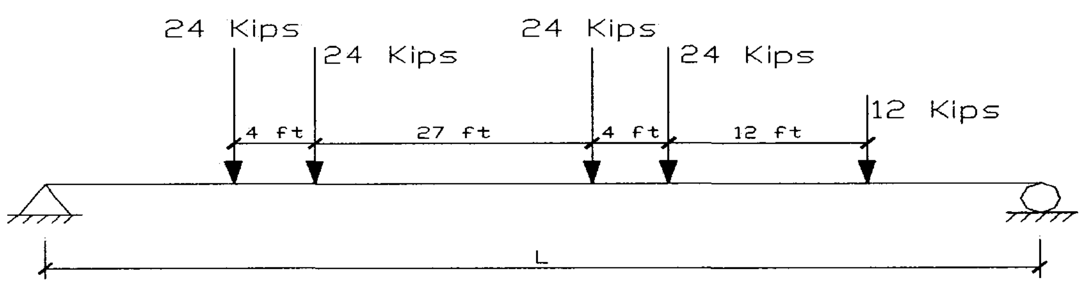


Fig. 3.5 FHWA 3S2 Truck Configuration with GVW=108 Kips

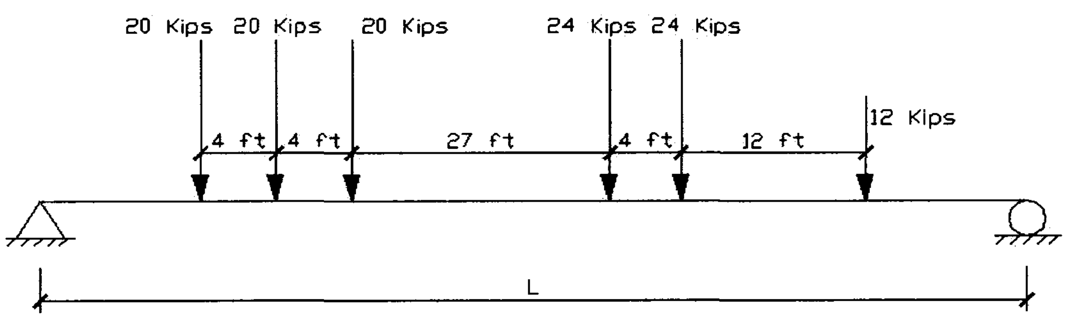


Fig. 3.6 FHWA 3S3 Truck Configuration with GVW=120 Kips

3.3 Method of Approach

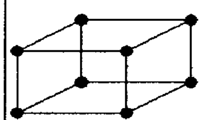
The finite element analysis of the bridge was finished by GTSTRUDL software in this study. The finite element models used for bridge in this study simulated the behavior of simple span and continuous bridges. The girders were modeled using Type-IPSL tridimensional elements available in GTSTRUDL. Type-SBCR 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.

3.3.1 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 3.1.

Table 3.1 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

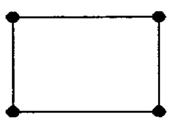
N - Output Element Nodes

3.3.2 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. These type plate finite elements were 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 3.2.

Table 3.2 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

3.3.3 Prismatic Space Truss Member

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 related 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 3.3 lists the detailed properties of the prismatic space truss member.

Table 3.3 Detail Properties of Prismatic Space Truss Member

Member Type	Member Parallel To Global 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 Applicable

3.4 Finite Element Modeling of Concrete Girder Bridges

3.4.1 Bridge Properties

For the simple span bridges, the girders were considered simply supported at the supports; for the continuous bridges, the girders were simply supported at each support while the deck was cast continuously above the girders. The properties of the girders were defined by certain parameters dependent on the bridge geometry. Seven girders were used in models with a 5/4.5 ft spacing. Five girders were used in models with eight ft spacing; however, the outside girders on the models had a narrower spacing in order to keep the bridge width 30 ft constant. Several assumptions were made in the formulation of the bridges in this study as follows:

1) The slab thickness was kept eight inches as a constant.
2) The bridge width was kept 30 ft as a constant.
3) All girders in the models were identical and parallel to each other.
4) The simple span bridge models had one span; the span length was measured between two support centers.
5) The continuous bridges were defined as the bridge had three equal spans, the girders were simply supported at two adjacent supports, and the deck was continuously cast above the girders.
6) Full composite action was assumed between the girder and the slab.

3.4.2 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.

3.4.3 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, FHWA 3S2 or 3S3 truck loading. In addition to the dead and truck loads, a future wearing surface loading of 12 psf, and a surcharge 19 psf, in total 31 psf, according to LADOTD Bridge Manual, was placed on the deck to account for future overlays. Based on AASHTO Chapter III, four kinds of load combinations were used in this study, and corresponding loading condition factors were applied to the model, as shown in Table 3.4. In the load combination "fatigue," the impact factor 1.3 was applied to all trucks, as required by the AASHTO LRFD Bridge Design Manual, chapter III.

Table 3.4 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		

3.4.4 <u>Finite Element Modeling of the Girder Over Interior Supports</u>

Since the girders are simply supported and the deck is continuous over the girders, a space will be created between the two girders, over the interior supports, during the construction of the bridge. Because the end diaphragm does not provide continuity in this case, the girder will require a two inch gap between the girders, as shown in Fig. 3.7 to Fig. 3.8.

Bridge decks contain 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 will 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 are free to rotate and act as a simply supported beam because the beam is supported by points at the end of the girder and not resting on the pad. Tensile and compressive stresses will still exist at the girder ends because of the restricted rotation of the girders.

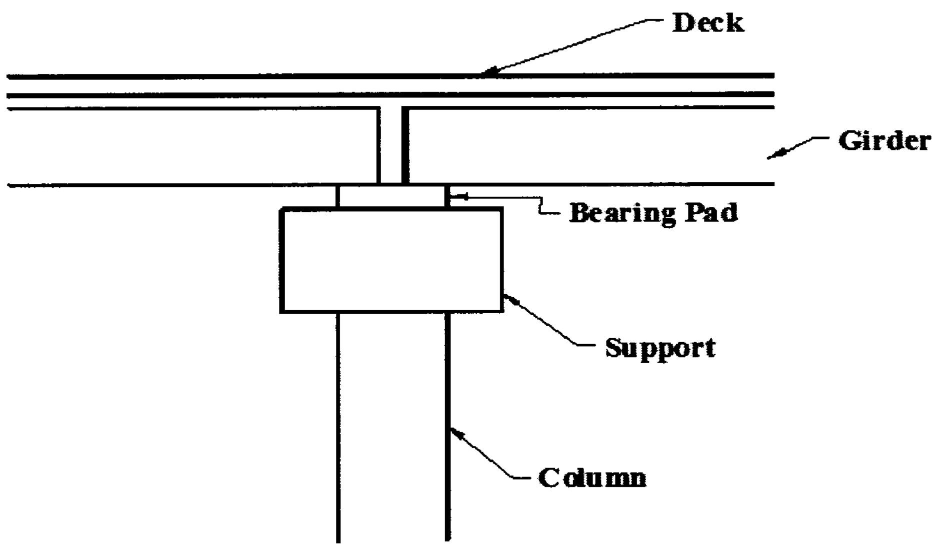


Fig. 3.7 Elevation View of Girders over Interior Support

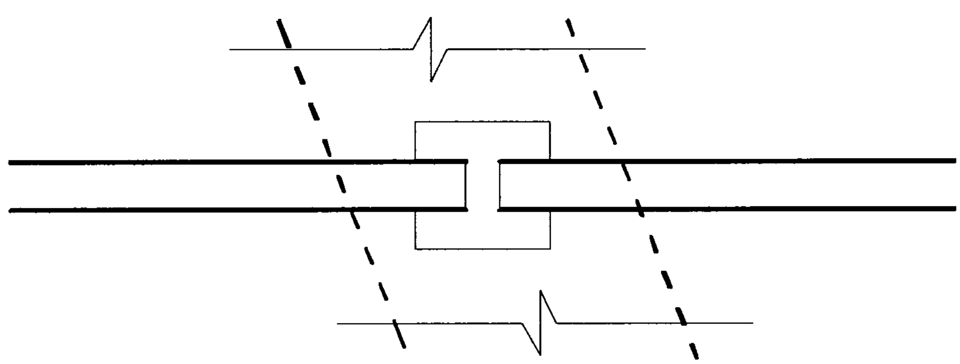


Fig. 3.8 Plan View of Girders over Interior Support

3.5 <u>Influence Line Analysis</u>

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.

3.5.1 Modeling in GTSTRUDL

In this study, HS20-44 truck loads, and typical heavy 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.

The influence lines were computed in both the longitudinal and transverse direction of the bridge. The models were constructed in GTSTRUDL, and then the unit loads were applied to the bridge. GTSTRUDL calculated the ordinates (deflection) of the maximum moment of displaced structure due to the unit loading at each joint. The results were used to generate the moment produced by each truck loading and determine the critical truck locations.

3.5.2 Determination of the Critical Truck Load Location

For the influence line generated at each joint, the maximum moment and shear force was found by moving the selected truck load through the span in one-inch increments. The maximum values due to the truck load were calculated by superposition, which took the sum of the ordinates multiplied by the magnitudes of the loads. The critical truck load location was determined by finding the location where the wheel load generated the maximum moment or shear force. Table 3.5 represented the results of critical locations of trucks on continuous bridges.

Table 3.5 Critical Location for Trucks on Continuous Bridge Girders

Span Length	HS20-44			FHWA 3S2		
	Truck Location X (ft.) (From Left Support to Front Tire)			Truck Location X (ft.) (From Left Support to Front Tire)		
(ft.)	Max Positive Moment	Max Negative Moment	Max Absolute Shear Force	Max Positive Moment	Max Negative Moment	Max Absolute Shear Force
55	8	12	26	6	25	7
60	10	15	31	8	30	12
65	12	18	36	10	34	17
70	14	21	41	11	39	22
75	17	24	46	13	66 (a)	27
80	19	27	51	15	69 (a)	32
85	21	30	56	17	72 (a)	37
90	23	32	61	20	75 (a)	42
95	25	35	66	22	78 (a)	47
100	27	38	71	24	81 (a)	52
105	29	41	76	26	35	57
110	31	44	81	28	87 (a)	62
115	33	47	86	30	90 (a)	67
120	36	50	91	32	93 (a)	72
125	38	53	96	34	96 (a)	77
130	40	56	101	36	99(a)	82

(a) The truck is traveling from left side to right side along the bridge. Otherwise from right to left.

3.6 Summary

The methodology of finite element analysis in this study was represented in this chapter. Variables of analysis and details of model properties were introduced. Factors needed for finite element analysis were given and discussed. Influence line analysis was performed, and the results were used in the upcoming chapters.

CHAPTER IV

BRIDGE GIRDER PERFORMANCE UNDER THE HEAVY TRUCK LOAD

4.1 Introduction

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 the 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 author used two typical methods described in this chapter. In following section 4.2, the simplified AASHTO line girder analysis approach was used to evaluate the girder behavior by determining the magnitude of the maximum moment and shear forces. The detailed analysis using finite element models by GTSTRUDL was performed in section 4.3; both the short term effect and the long term effect of the girder under the truck load were evaluated.

4.2 Evaluation Based on AASHTO Linear Approach

The methodology used in this analysis phase evaluated the effect of the heavy truck loads on the bridges based on LRFD and LFD design recommendations. The effects of heavy truck loads on bridges were determined by comparing the flexural, shear, and serviceability conditions of the bridges under their design load to the conditions under the FHWA 3S2 truck configuration, with maximum tandem load 48,000 lb, and steering axle load of 12,000 lb.

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 analysis, 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 FWHA 3S2 truck and the design truck (HS20-44) for flexural and shear forces or stresses were calculated. The serviceability criteria were evaluated based on their deflections.

This part of study included some contents from the Louisiana state project No. 736-9-1299 (also the LTRC project No. 05-2p). In this project approximately 2,800 bridges were involved, which were grouped in Table 4.1. The analysis for those bridges was performed, and results are presented in this section.

Table 4.1 Considered Critical Bridges and Categories

Critical Bridges for This Study		
	State Bridges	**Parish Bridges**
Category	**Number of Bridges**	**Number of Bridges**
Simple Beam	998	166
Continuous	149	1
Culvert	435	59
Others	75	20
Posted Bridges	169	302
Design Load Low (5, 10 ton)	55	3
Design Load Unknown	NA	394
Total	1881	945

4.2.1 Performance of Simple Span Bridge Girders

The influence line analysis for bridges with simple spans was performed using hand calculations and spread sheets. The standard truck configurations for HS20-44, as provided in AASHTO Chapter III, were used. The span length for bridge girders between 20 ft. and 120 ft. (at 2 ft. increments) were considered for this study.

The truck loads were placed on the bridge girder as shown in chapter III, and moved on the girder at 1 ft. increments, to calculate the absolute maximum moment and shear force. The different load conditions for the corresponding girder span lengths are shown in Table 4.2.

Table 4.2 Load Conditions for Simply Supported Bridge Girders

HS20-44 Truck Configuration		**FHWA 3S2 Truck Configuration**	
Girder Span (ft.)	**Load on Girder**	**Girder Span (ft.)**	**Load on Girder**
20 To 28	P2 (or P3)	20 To 24	P1
20 To 28	P1 & P2	20 To 26	P2 & P3
20 To 28	P2 & P3	20 To 56	P4 & P5
33 To 120	P1, P2 & P3	24 To 62	P1, P2 & P3
33 To 120	P1, P2 & P3	50 To 57	P1, P2, P3 & P4
		52 To 120	P1, P2, P3, P4 & P5

The performance of simple span bridge girders were evaluated by the values and ratios of absolute maximum shear, moment, and deflection. 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 maximum deflection was determined by the truck location on the bridge girder that caused the maximum absolute moment.

The effects of FHWA 3S2 trucks loads on bridges were evaluated by normalizing the critical conditions for each bridge span to the design load, which are presented in Fig. 4.1. The ratio of the absolute maximum moment varied between 0.98 and 1.29. The ratio of the shear forces varied between 0.97 and 1.34. Where the bridge span was similar to the length of the 3S2 truck, the ratios of the absolute maximum moment and shear were within 10 percent. This confirms the findings in the previous studies that focused 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 would be overstressed, which could experience more cracking in the bridge girders and bridge decks. Such cracks would require additional inspections along with early and frequent maintenance.

The ratio for deflection caused by FHWA 3S2 truck loads as compared to HS20-44 truck loads varied between 0.94 and 1.42. The above discussion on the ratio of the absolute moment 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 FHWA 3S2 trucks. Also, the high ratios obtained in this study could result in more cracking in the bridge girders and bridge decks. Such cracks will require additional inspections along with early and frequent maintenance.

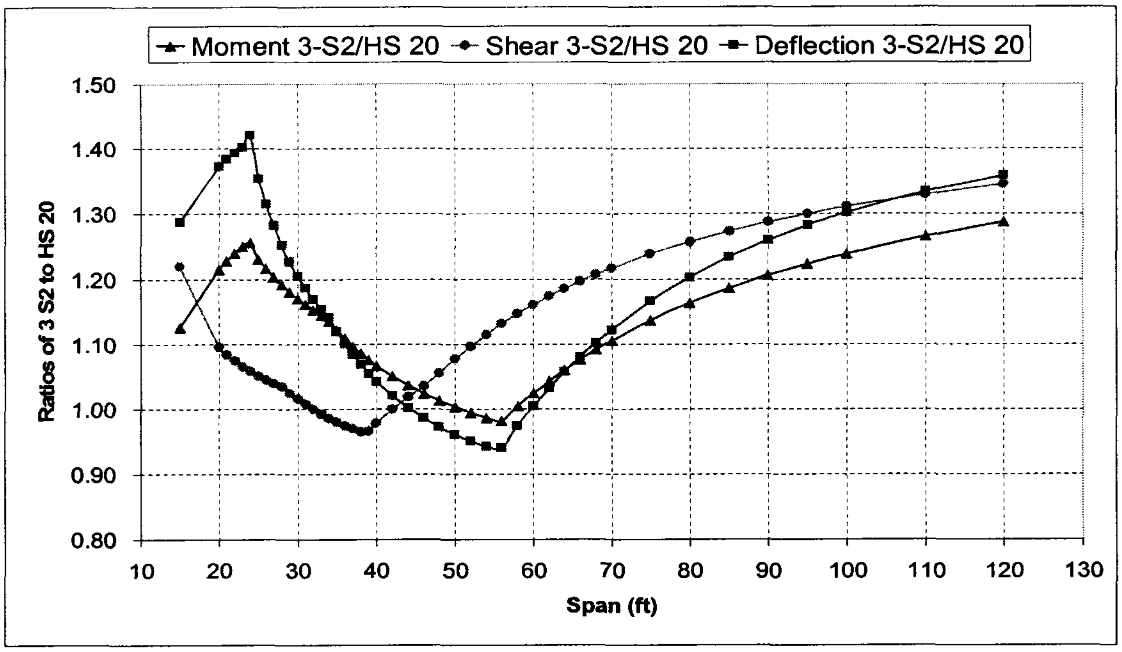


Fig. 4.1 Effects of 3S2 Truck on Simple Span Bridges with HS20-44 Design Loads

The effects of FHWA 3S2 truck loads on simple span bridges designed for HS20-44 truck loads are presented in Table 4.3. The span for most of these bridges is 20 ft.; the ratio of the absolute maximum moment and shear due to 3S2 and HS20-44 truck loads are 1.22 and 1.1, respectively. Previous studies reported that changes in the design codes and design practices could cause a margin of safety of about 5 percent to 10 percent in bridges designed for HS20-44 truck loads.

This study included 60 bridges with span lengths between 40 ft. and 66 ft. The ratio for the absolute maximum moment was within the margin of safety. There were 57 bridges with span lengths between 70 ft. and 120 ft., and 38 bridges with span lengths between 25 ft. and 35 ft. The ratio for the absolute maximum moment was larger than 1.1, or more than the 10 percent margin of safety. Therefore, the bridges in Table 4.3 with ratios that are higher than the margin of safety for bridges designed for HS20-44 truck loads could experience flexural and shear cracks in the bridge girders and bridge decks.

The bridges with span length larger than 120 ft were marked as "outliers," and were not considered in this study.

Table 4.3 Evaluated State Simple Span Bridges

Max Span Length (ft.)	Number of Bridges Design Load HS20-44	Ratio 3 S2/HS20-44		
		Moment	Shear	Deflection
20 or shorter	632	1.22	1.10	1.37
25	30	1.23	1.05	1.35
30	1	1.17	1.02	1.21
35	7	1.12	0.98	1.12
40	14	1.07	0.98	1.04
46	15	1.02	1.04	0.99
50	16	1.00	1.08	0.96
56	3	0.98	1.13	0.94
60	12	1.03	1.16	1.00
66	4	1.08	1.20	1.08
70	17	1.11	1.22	1.12
75	7	1.14	1.24	1.17
80	2	1.16	1.26	1.20
85	5	1.19	1.27	1.23
90	5	1.21	1.29	1.26
95	4	1.22	1.3	1.28
100	6	1.24	1.31	1.30
110	5	1.27	1.33	1.33
120	2	1.29	1.34	1.36
125 to 235	13	Outliers		
	Total (800)			

4.2.2 Performance of Continuous Span Bridge Girders

GTSTRUDL software was used to calculate the influence line of moment and shear at each joint along the length of the bridge girder. The bridge girder models were considered as three equal spans. The first support for the girder was considered pin support, and the remaining three supports were roller type. The span lengths considered for this study varied from 20 ft. to 130 ft. (at 5 ft. increments). All truck loads were placed on each girder to perform the analysis. Due to the symmetry of the bridge, only the left half part of the bridge girder was considered. The truck loads were applied in both directions, from left to right and from right to left.

After generating the influence line for each joint, the position of the truck loads on the bridge girder that would result in maximum positive moment, maximum negative moment, and maximum shear forces was determined. Those maximum values were calculated by moving the truck loads along the bridge girders in 1 ft. increments. The magnitudes of the moment and shear force 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 showed that the increase in the truck load on the moments in the bridge girder was insignificant for girders with spans shorter than 70 ft. However, the impact on the girders with long spans was more significant.

The effects of FHWA 3S2 trucks loads on continuous bridges were evaluated also by normalizing the critical conditions for each bridge span to the design load, which are presented in Figs. 4.2-4.3. The ratio of the maximum positive moment varied between 1.0 and 1.28. For the maximum negative moment the ratio varied between 1.0 and 1.48. The ratio of the shear forces varied between 0.98 and 1.40. Where the bridge span was similar to the length of the FHWA 3S2 truck, the ratio of the maximum positive moment and shear forces were within 10 percent. This result confirmed the findings in the previous studies that focused on bridge formulas. The previous studies increased the GVW and minimized the impact on the stresses in the bridge girders by increasing the truck length. However, bridge girders with a maximum positive moment ratio or shear larger than 1.10 would be overstressed.

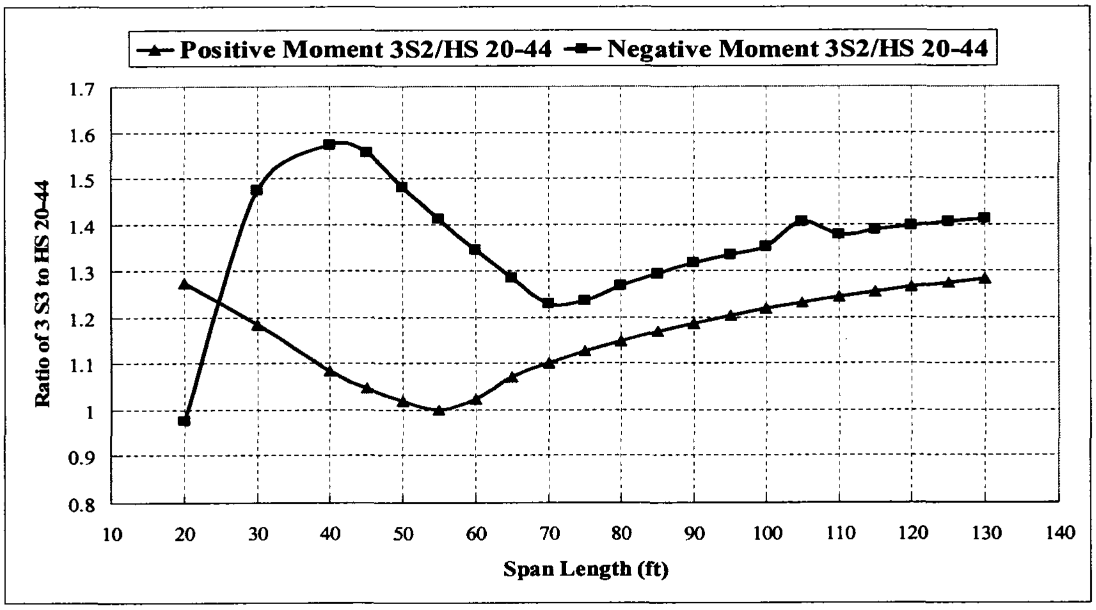


Fig. 4.2 Effects on Moment of 3S2 Truck on Continuous Bridges

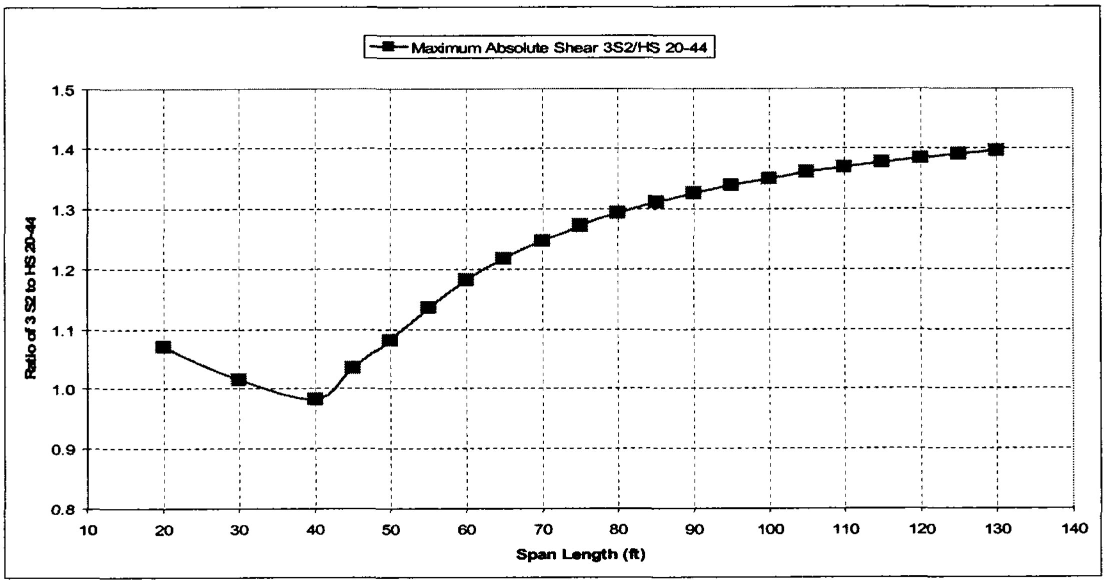


Fig. 4.3 Effects on Shear Force of 3S2 Truck on Continuous Bridges

The effects of 3S2 truck loads on continuous bridges designed for HS20-44 truck loads are presented in Table 4.4. This study included 42 bridges with span lengths between 40 ft. and 70 ft. The ratios for the maximum moment were within the margin of safety. There were three bridges with span length equal to 20 ft., and 81 bridges with span length between 70 ft and 130 ft., for which the ratio for the maximum positive moment was larger than 1.1, or more than the 10 percent margin of safety. Therefore, these bridges could experience flexural and shear cracks in the bridge girders and bridge decks. Such cracks would require additional inspections along with early and frequent maintenance. The bridges with span length larger than 130 ft were marked as "outliers," and were not considered in this study.

The ratio for the maximum negative moment was higher than the margin of safety, except for the three bridges with span lengths equal to 20 ft. The high values in negative moment would result in high compressive stresses in the bridge decks. Such conditions could result in an increase in the compression cracks and would require additional inspections and could result in early and frequent maintenance, also.

Table 4.4 Evaluated State Continuous Bridges

Max Span Length (ft.)	Number of Bridges Design Load HS20-44	Ratio 3S2/HS20-44		
		Positive Moment	Negative Moment	Shear
20	3	1.28	0.98	1.07
40	1	1.08	1.57	0.98
45	1	1.05	1.56	1.04
50	14	1.02	1.48	1.08
55	1	1.00	1.41	1.14
60	4	1.02	1.35	1.18
65	6	1.07	1.28	1.22
70	15	1.10	1.23	1.25
75	10	1.13	1.24	1.27
80	2	1.15	1.27	1.29
85	5	1.17	1.29	1.31
90	18	1.19	1.32	1.33
95	3	1.20	1.34	1.34
100	13	1.22	1.35	1.35
105	20	1.23	1.40	1.36
110	2	1.24	1.38	1.37
120	2	1.27	1.40	1.38
125	4	1.28	1.41	1.39
130	2	1.28	1.41	1.40
135 to 375	19	Outliers		
	Total (145)			

4.3 Evaluation Based on Finite Element Analysis

Finite Element Modeling (FEM) is among the most popular methods of analysis. Significant advances in computer technology allow for detailed models to be constructed and analyzed. The finite element models used in this study simulate the behavior of medium span continuous bridges. GTSTRUDL Version 28 software was used for this investigation. The modeled bridge girders were formulated using Type-IPSL, tridimensional eight node elements. The bridge deck was formulated using Type-SBCR for node plate elements. Prismatic space truss members were used to model the continuity diaphragms, as shown in chapter III.

4.3.1 Short Term Effects of Heavy Truck Load on Simple Span Bridge Girders

The results of all bridges with girder type AASHTO Type IV, V, VI, BT-54, BT-63 and BT-72 were compared to determine the short term effects of FHWA 3S3 truck load on simply supported bridge girders.

In this study, the short term effects of FHWA 3S3 truck loads on simple medium span bridges designed for HS20-44 truck loads were evaluated by computing the percent change of the maximum stress at both top and bottom surfaces of each girder. Three load combinations "Strength I max," "Strength III max," and "Strength V max," based on AASHTO LRFD bridge design specifications, were used to evaluate the short term performances of the bridge girders. By comparing the stress state of bridge girders under these three load combinations, the load combination "Strength I max" lead the maximum stresses of the girder. Therefore, we could determine that the "Strength I max" is the governing load combination for the short term effects analysis, and all the analyses below were based on it.

The bridges analyzed in this investigation were 30 ft. wide, simply supported bridges. The span length varied from 90 to 120 ft. The slab thickness considered was eight-inch. The bridge model may contain five or seven girders; the girder spacing in this model was eight or five ft., respectively. The detail models and their properties were presented in chapter III.

In this study, the short-term effects of FHWA 3S3 truck loads on simple medium span bridges designed for HS20-44 truck loads were evaluated by computing the percent change of the maximum stress at both top and bottom surfaces of each girder. Only the compressive stress was considered at top surface of the girder, while both of the tensile and compressive stresses were obtained at bottom surface of the girder.

The percent change of maximum stress of each individual girder of the model with 8-ft spacing was presented in Figs.4.4-4.9. The results indicated that the change in the compressive stress for all types of girders at the top surface of the girder did not exceed 10%. The change in the compressive stress for all types of girders at the bottom surface of the girder did not exceed 10% except girder three or four of AASHTO Type IV and Bulb-Tee models with the percent change less than 12%; the change in the tensile stress for all types of girders at the bottom surface of the girder did not exceed 10% except girder five of AASHTO Bulb-Tee models with the percent change less than 12%. The bridges in this study with stress percent change which was greater than 10% would be considered as overstressed and might experience more cracking in the bridge girders. Therefore, most girders did not meet the overstress condition; the short term effects of the heavy truck load on those medium span bridges were limited. With the exception of bridges built with AASHTO Type IV or Bulb-Tee girders, the end of the interior girder on all bridges might need a special attention to their strength.

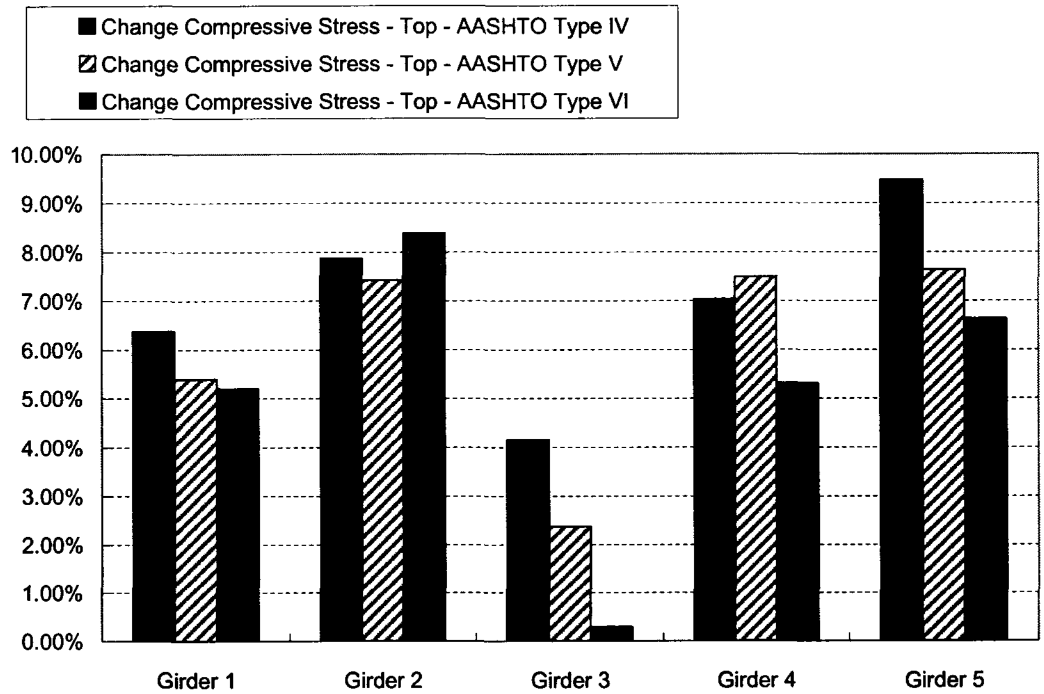


Fig. 4.4 Short Term Effects on Compressive Stresses at the Top of AASHTO I Type Bridge Girders – Girder Spacing Eight ft

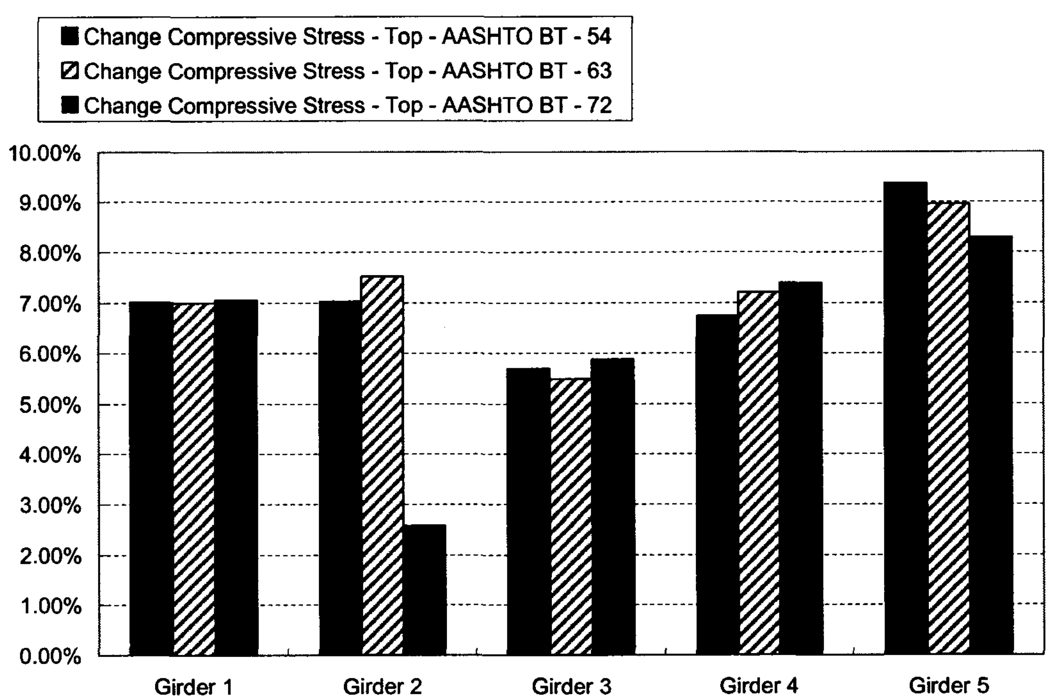


Fig. 4.5 Short Term Effects on Compressive Stresses at the Top of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Eight ft

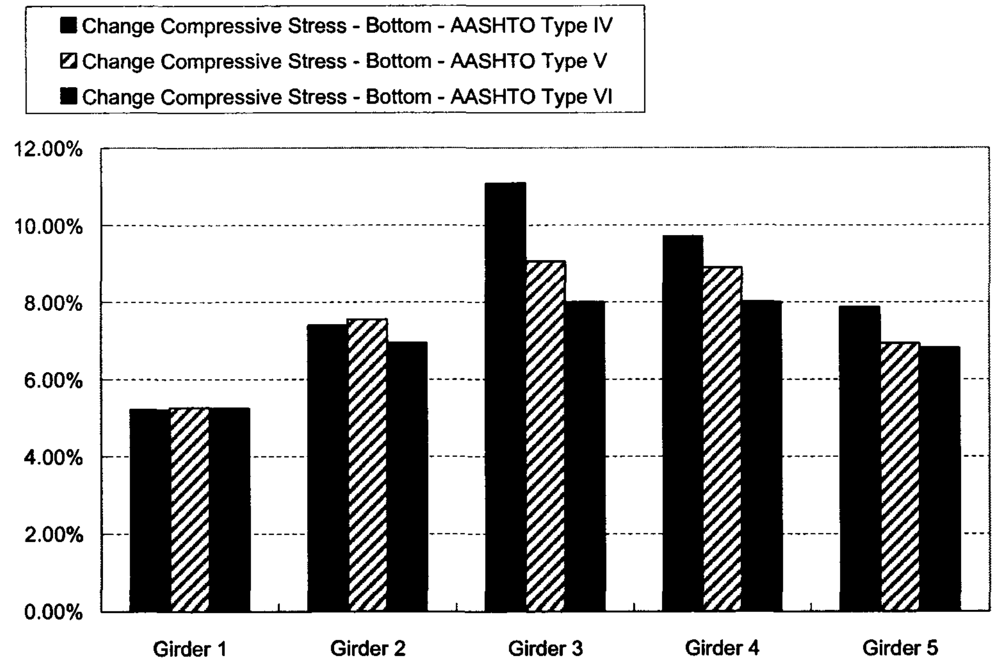


Fig. 4.6 Short Term Effects on Compressive Stresses at the Bottom of AASHTO I Type Bridge Girders – Girder Spacing Eight ft

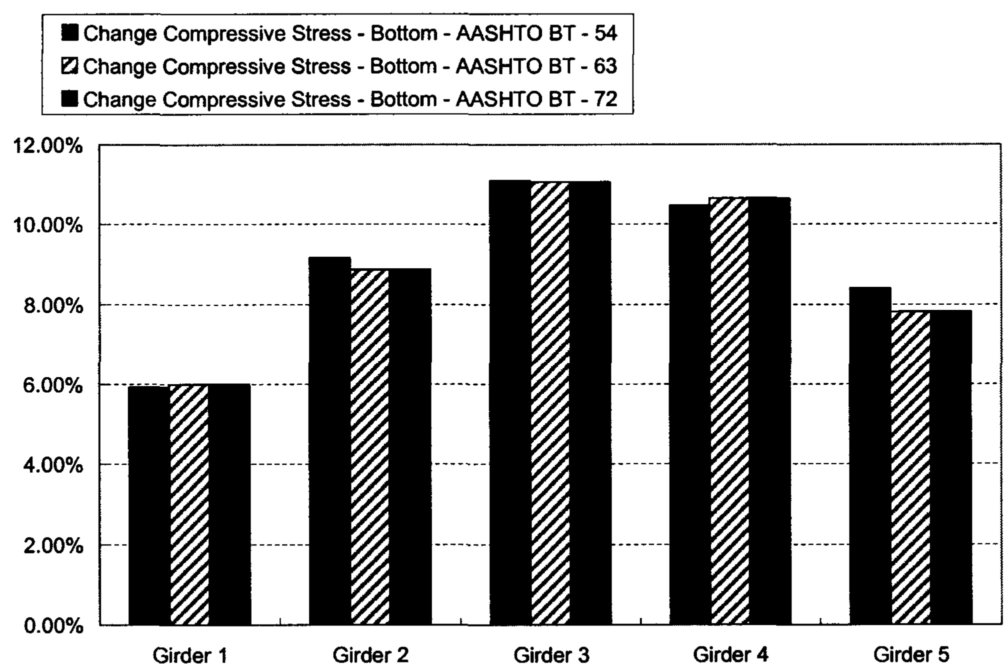


Fig. 4.7 Short Term Effects on Compressive Stresses at the Bottom of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Eight ft

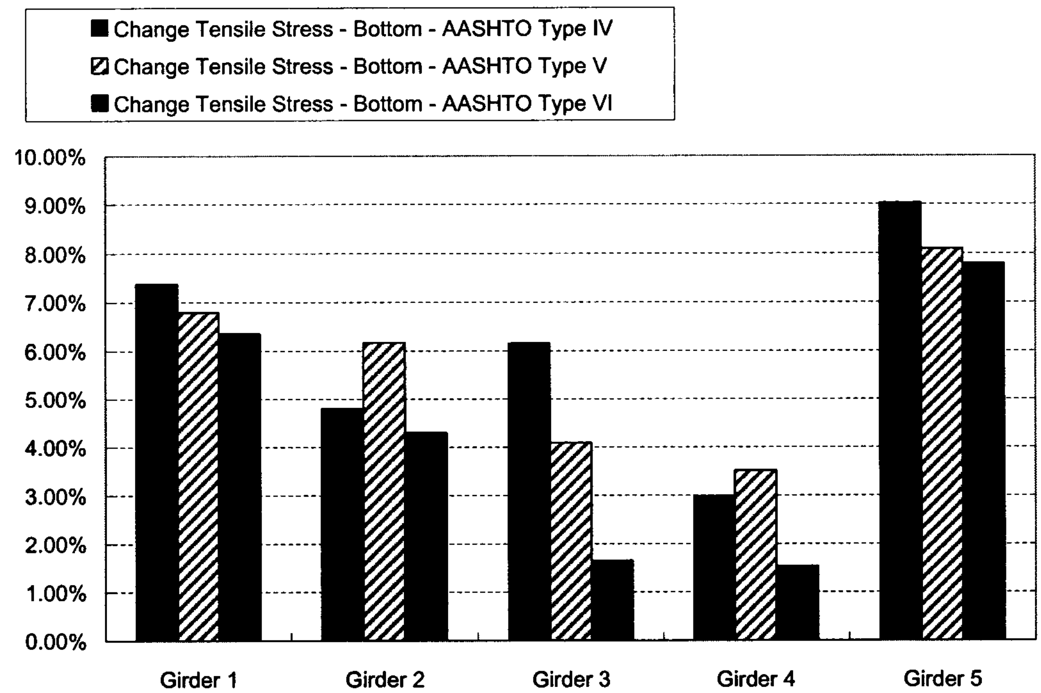


Fig. 4.8 Short Term Effects on Tensile Stresses at the Bottom of AASHTO I Type Bridge Girders – Girder Spacing Eight ft

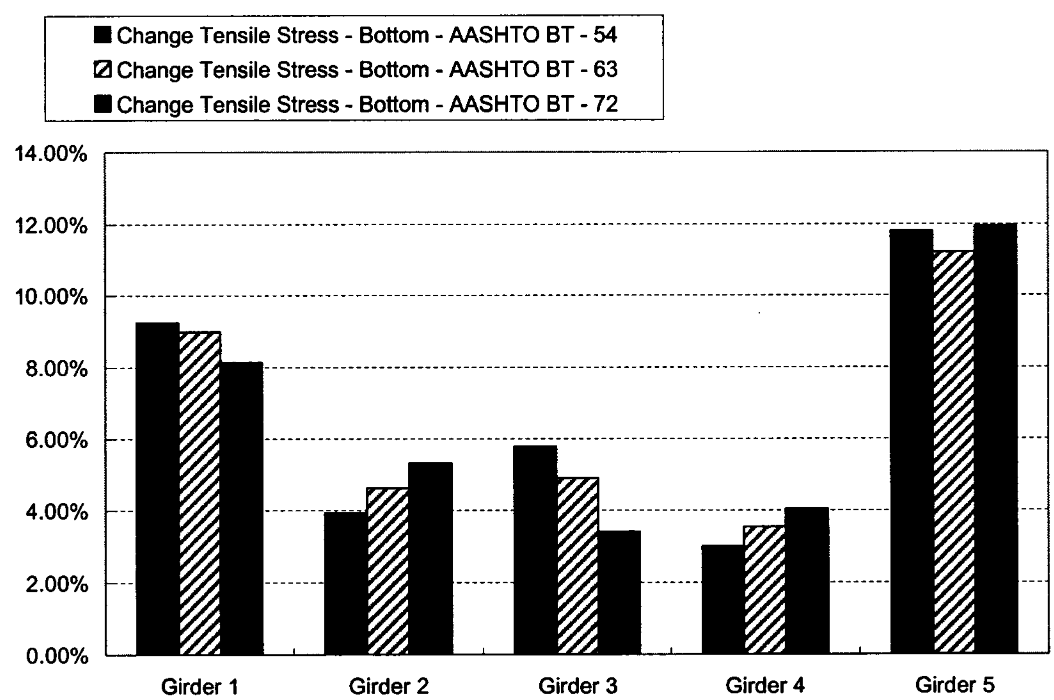


Fig. 4.9 Short Term Effects on Tensile Stresses at the Bottom of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Eight ft

The percent change of maximum stress of each individual girder of the model with 5-ft spacing was presented in Figs. 4.10-4.15. The results indicated that the change in the compressive stress for all types of girders at the top surface of the girder did not exceed 10%, the change in the compressive stress for all types of girders at the bottom surface of the girder did not exceed 10%. The change in the tensile stress for all types of girders at the top surface of the girder did not exceed 10%, either. The bridges in this study with stress percent change which was greater than 10% would be considered overstressed and might experience more cracking in the bridge girders. Therefore, the girders did not meet the overstress condition, and the short term effects of the heavy truck load on those medium span bridges were limited. It should be noticed that the percent change of stresses of those models that contained seven girders and had shorter girder spacing were smaller than the models contained five girders, which meant the models with shorter girder spacing had better capacity to resist the heavy truck load impact.

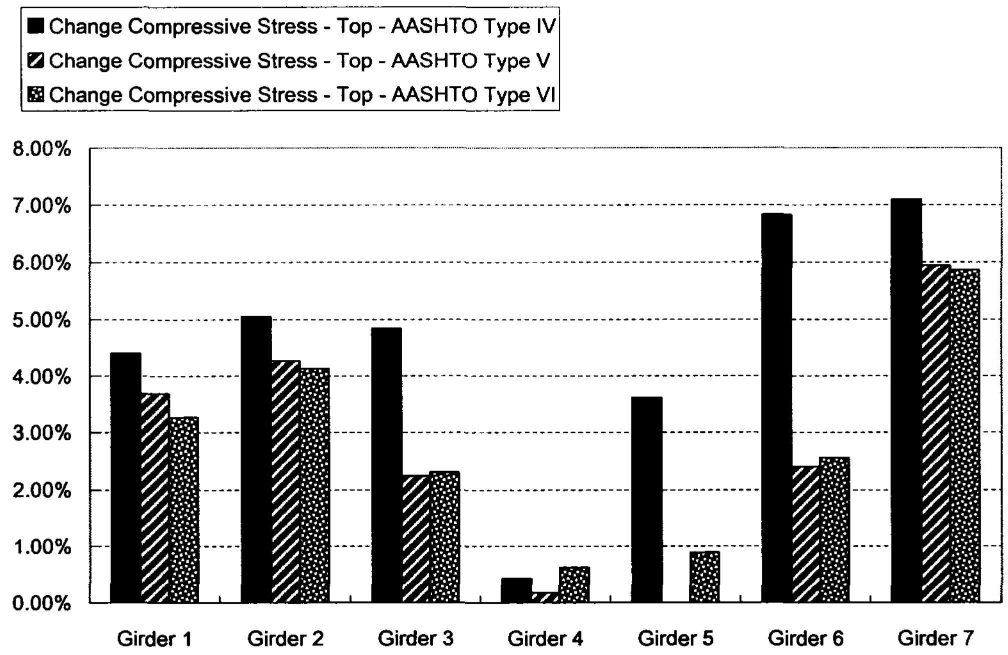


Fig. 4.10 Short Term Effects on Compressive Stresses at the Top of AASHTO I Type Bridge Girders – Girder Spacing Five ft

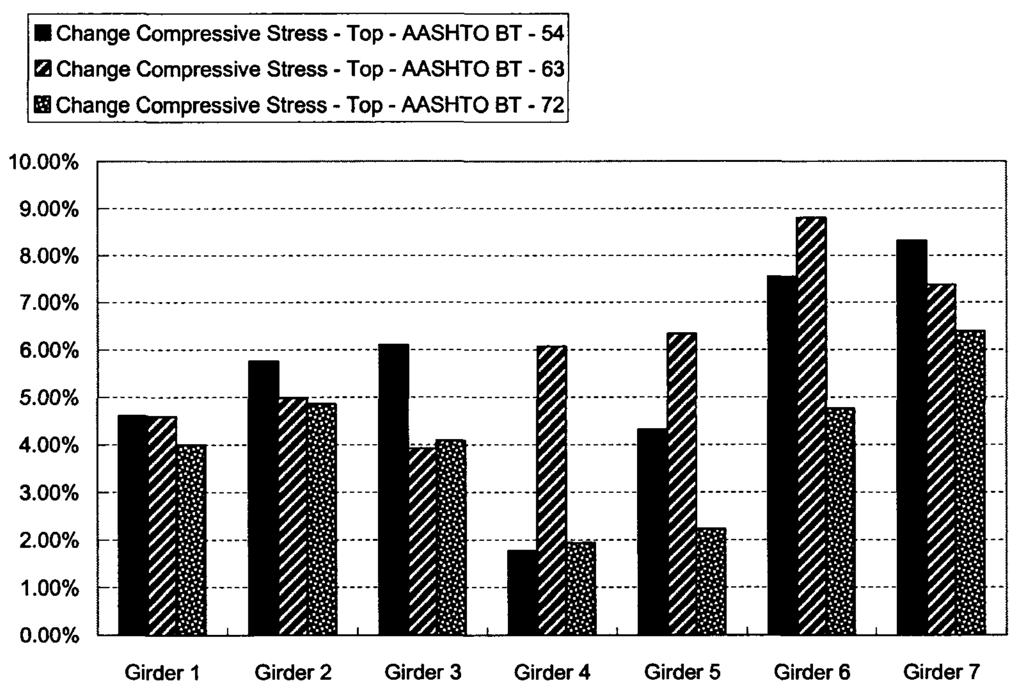


Fig. 4.11 Short Term Effects on Compressive Stresses at the Top of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Five ft

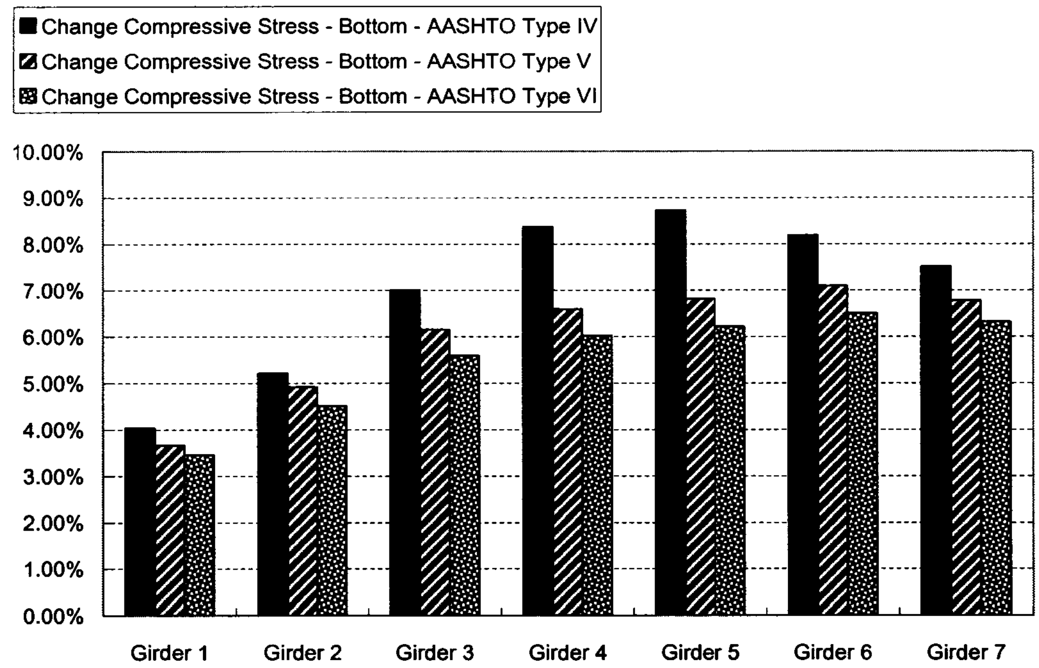


Fig. 4.12 Short Term Effects on Compressive Stresses at the Bottom of AASHTO I Type Bridge Girders – Girder Spacing Five ft

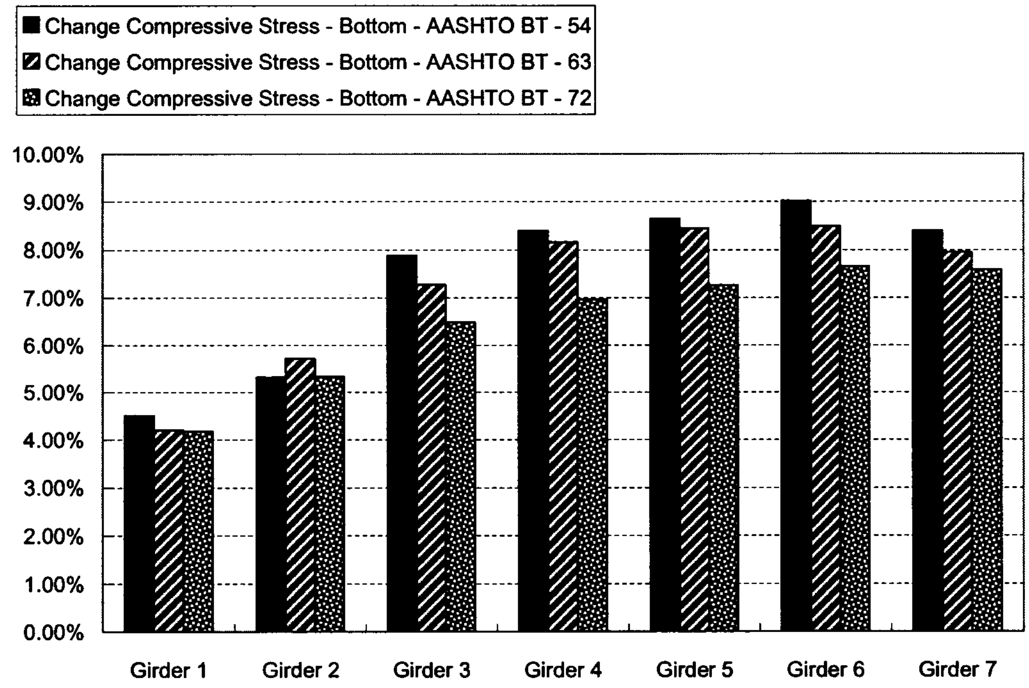


Fig. 4.13 Short Term Effects on Compressive Stresses at the Bottom of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Five ft

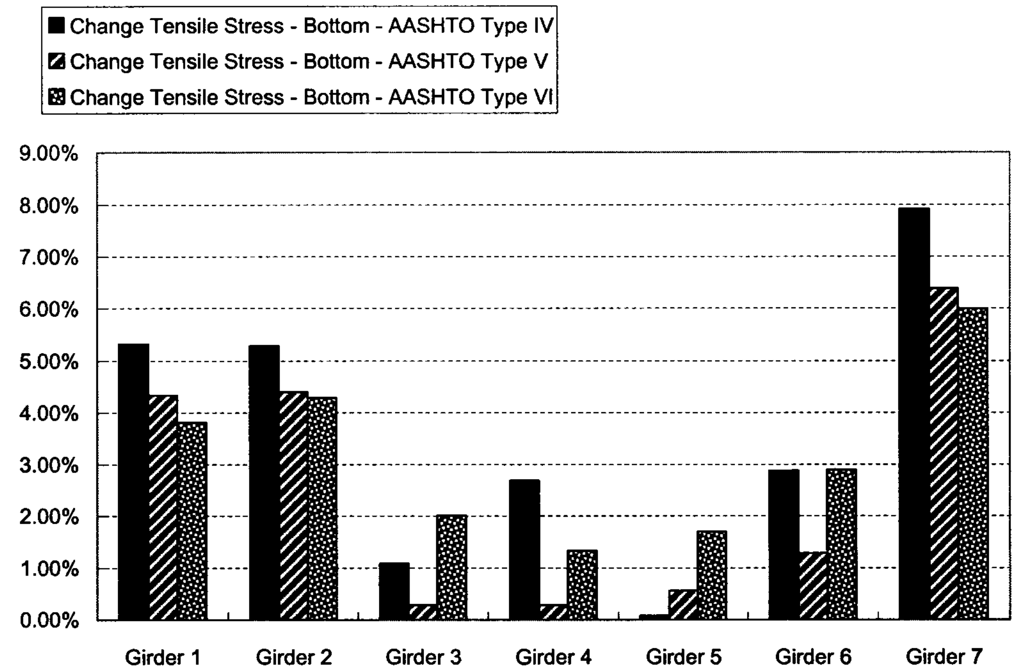


Fig. 4.14 Short Term Effects on Tensile Stresses at the Bottom of AASHTO I Type Bridge Girders – Girder Spacing Five ft

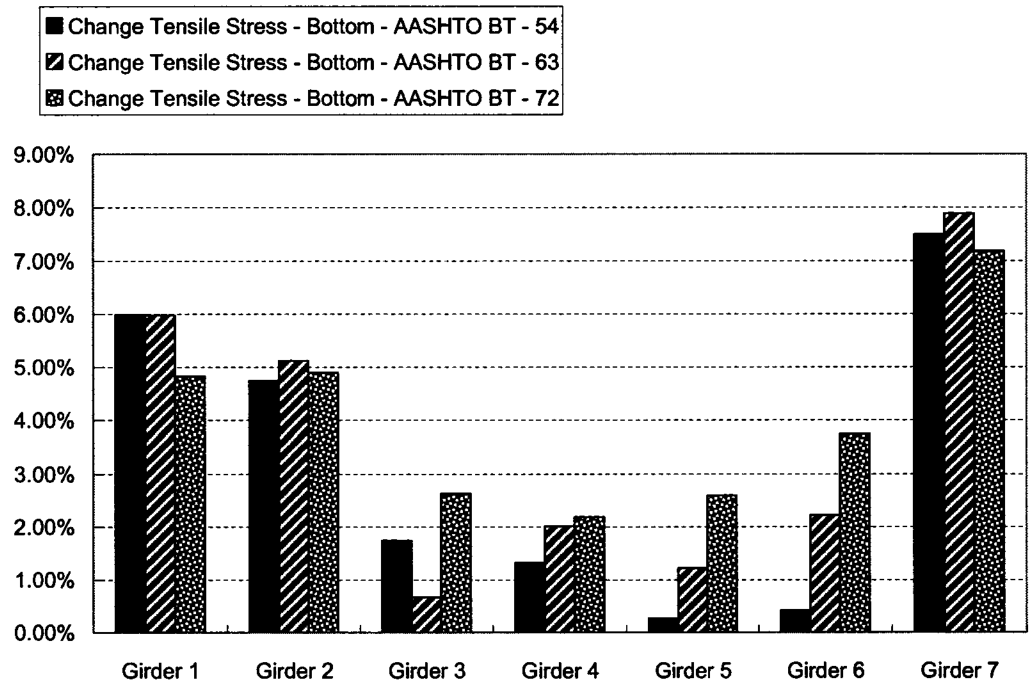


Fig. 4.15 Short Term Effects on Tensile Stresses at the Bottom of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Five ft

4.3.2 Long Term Effects of Heavy Truck Load on Simple Span Bridge Girders

The results of all bridges with girder type AASHTO Type IV, V, VI, BT-54, BT-63 and BT-72 were compared to determine the long term effects of FHWA 3S3 truck load on bridge girders.

The long term effects of FHWA 3S3 truck loads on simple span bridges designed for HS20-44 truck loads were evaluated also by computing the percent change of the maximum stress at both top and bottom surfaces of each girder. Based on AASHTO LRFD bridge design specifications, the load combination "Fatigue" was considered as the critical load combination for the long term effects analysis.

The percent change of maximum stress of each individual girder of the model with eight ft spacing was presented in Figs. 4.16-4.21. Only the compressive stress was considered at top surface of the girder, while both of the tensile and compressive stresses were obtained at bottom surface of the girder. The results indicated that the percent changes in long term stresses for all types of girders were much higher than those changes of short term stresses. The percent changes of stresses were mostly around 60%, some of which might be more than 100%. It was observed that the long term girder stress values under both 3S3 and HS20-44 truck loads were smaller than those short-term stress values, although the percent changes were significantly higher. This observation suggested that even the long-term girder stresses had not exceeded the maximum allowable stresses of the components; the effects of heavy truck loads could not be neglected. For a long period, the heavy trucks travel on the bridge would have remarkable effects on the bridge safety and serviceability. Therefore, such conditions could result in an increase in the flexural cracks on bridge girders and would require additional inspections and could result in early and frequent maintenance.

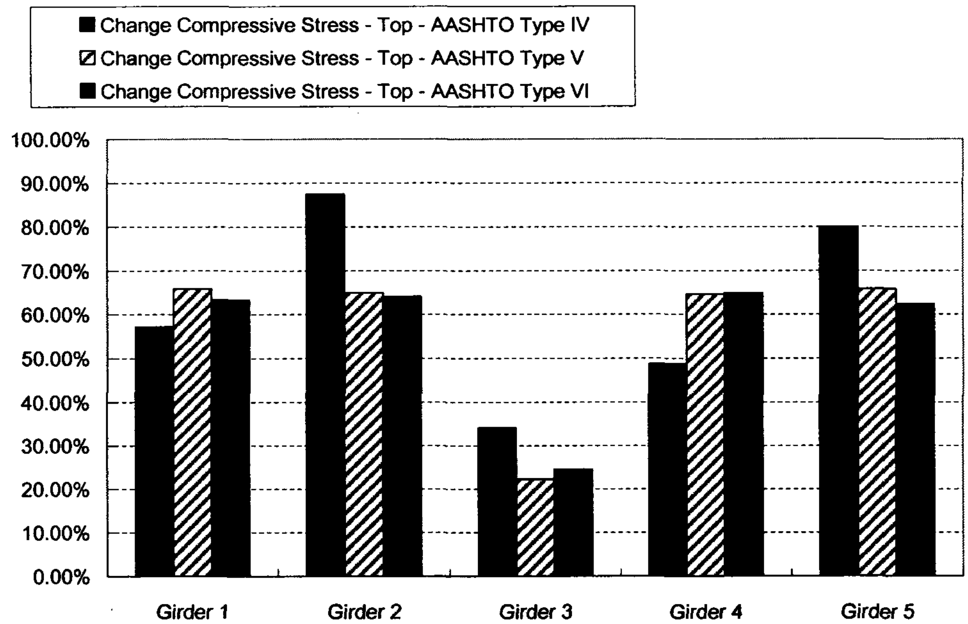


Fig. 4.16 Long Term Effects on Compressive Stresses at the Top of AASHTO I Type Bridge Girders – Girder Spacing Eight ft

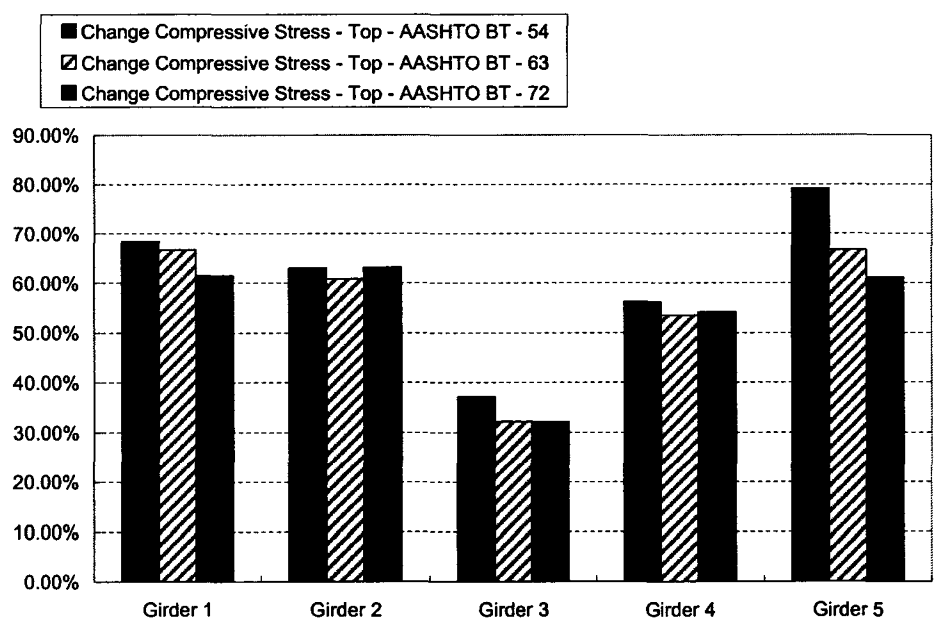


Fig. 4.17 Long Term Effects on Compressive Stresses at the Top of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Eight ft

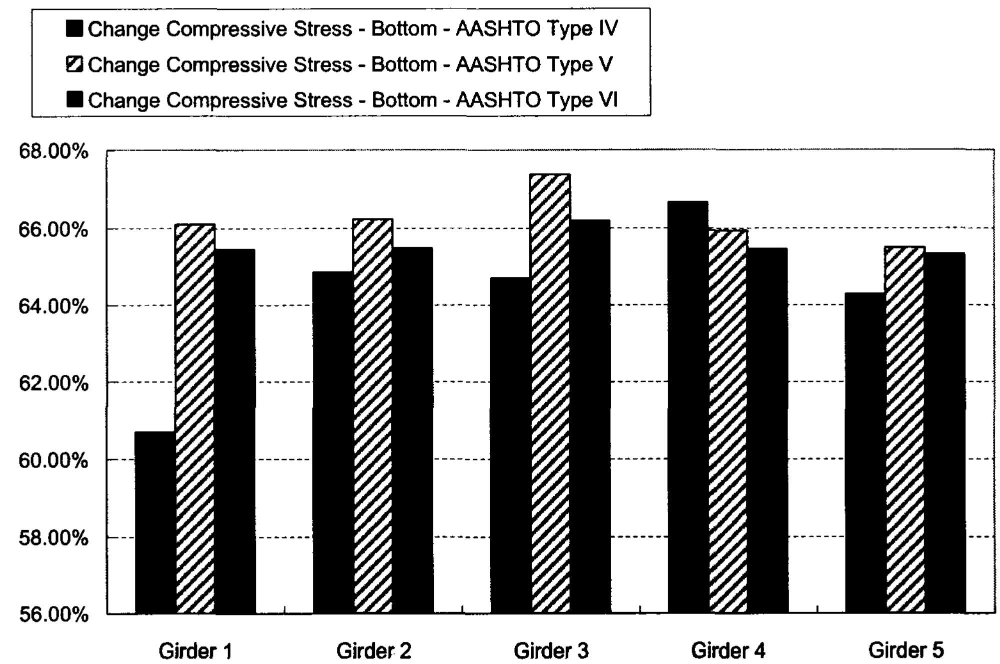


Fig. 4.18 Long Term Effects on Compressive Stresses at the Bottom of AASHTO I Type Bridge Girders – Girder Spacing Eight ft

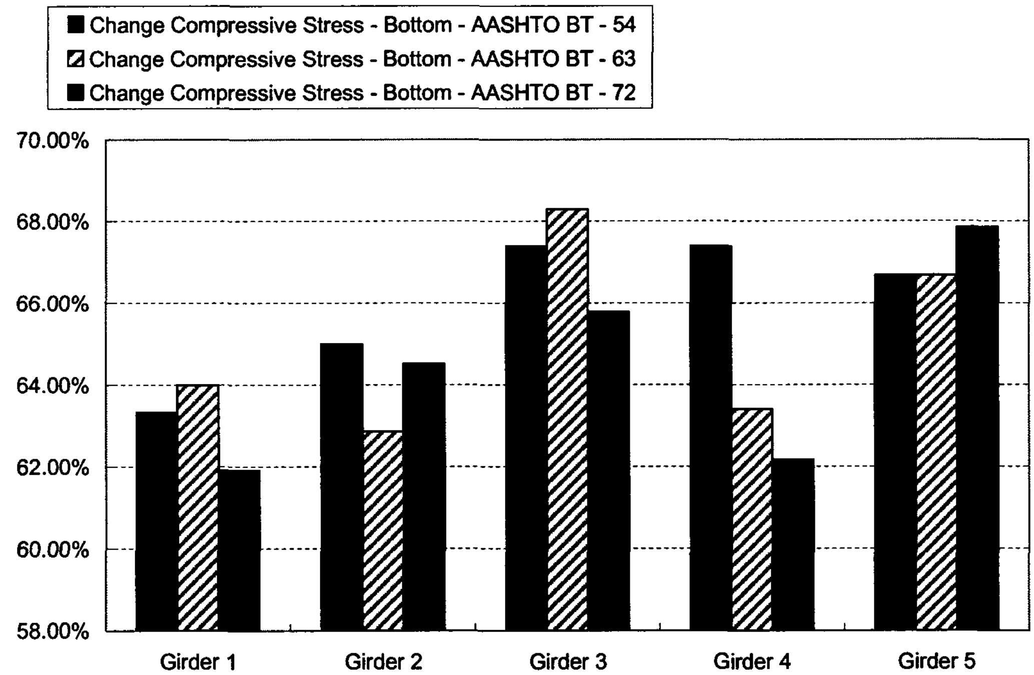


Fig. 4.19 Long Term Effects on Compressive Stresses at the Bottom of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Eight ft

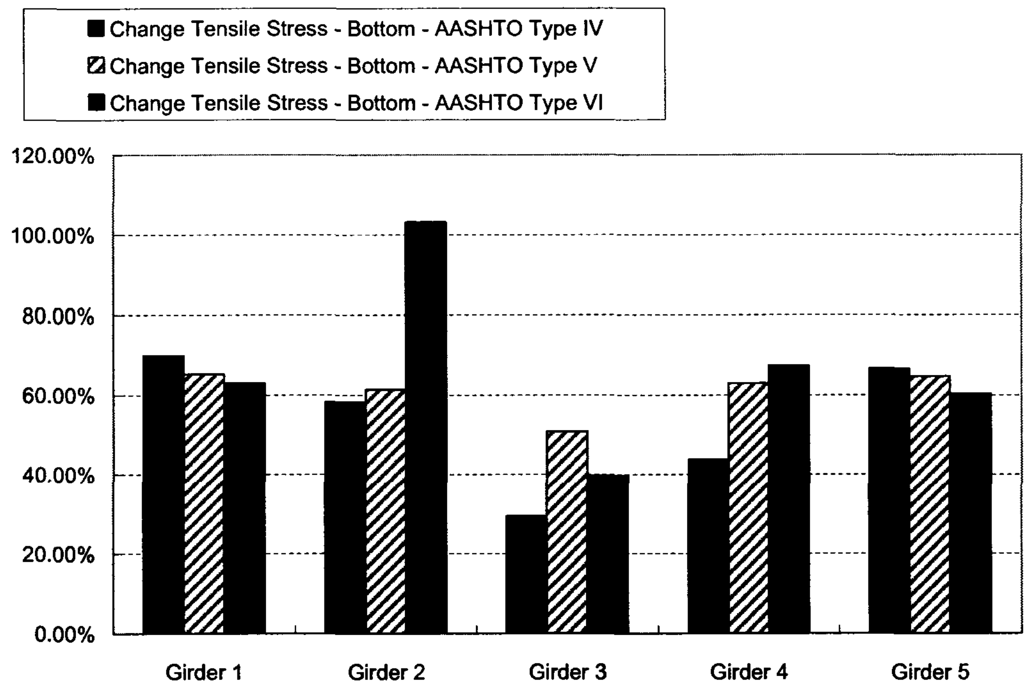


Fig. 4.20 Long Term Effects on Tensile Stresses at the Bottom of AASHTO I Type Bridge Girders – Girder Spacing Eight ft

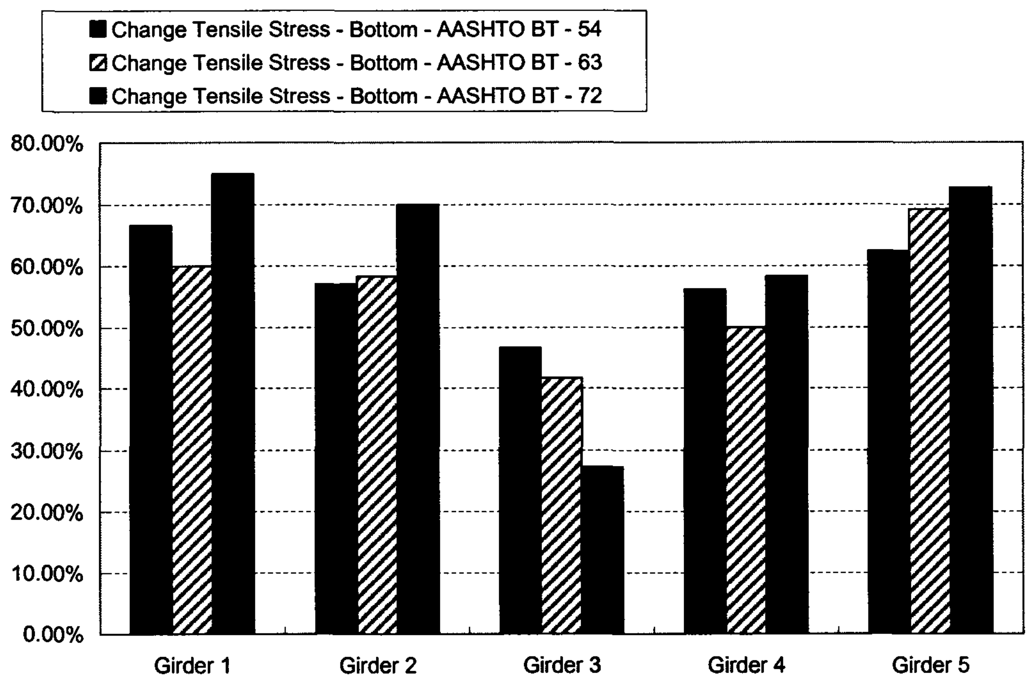


Fig. 4.21 Long Term Effects on Tensile Stresses at the Bottom of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Eight ft

The percent change of maximum stress of each individual girder of the model with five ft spacing is presented in Figs. 4.22-4.27. Only the compressive stress was considered at top surface of the girder, while both of the tensile and compressive stresses were obtained at bottom surface of the girder. The results indicated that the percent changes in long term stresses for all types of girders were much higher than those changes of short term stresses. The percent changes of stresses were mostly around 60%. Also, the long-term girder stress values under both 3S3 and HS20-44 truck loads were smaller than those short term stress values, although the percent changes were significantly higher, which indicated that even the long term girder stresses did not exceed the maximum allowable stresses of the components; the effects of heavy truck loads can not be neglected. For a long period, the heavy trucks traveled on the bridge would have remarkable effects on the bridge safety and serviceability. Therefore, these bridges might experience flexural cracks in the bridge girders. Such cracks would require additional inspections and could result in early and frequent maintenance. The percent changes of the models with five ft girder spacing were slightly smaller than the changes of the eight ft girder spacing models. This difference implied that the models with shorter girder spacing had better but limited capacity to resist the long-term heavy truck load impact.

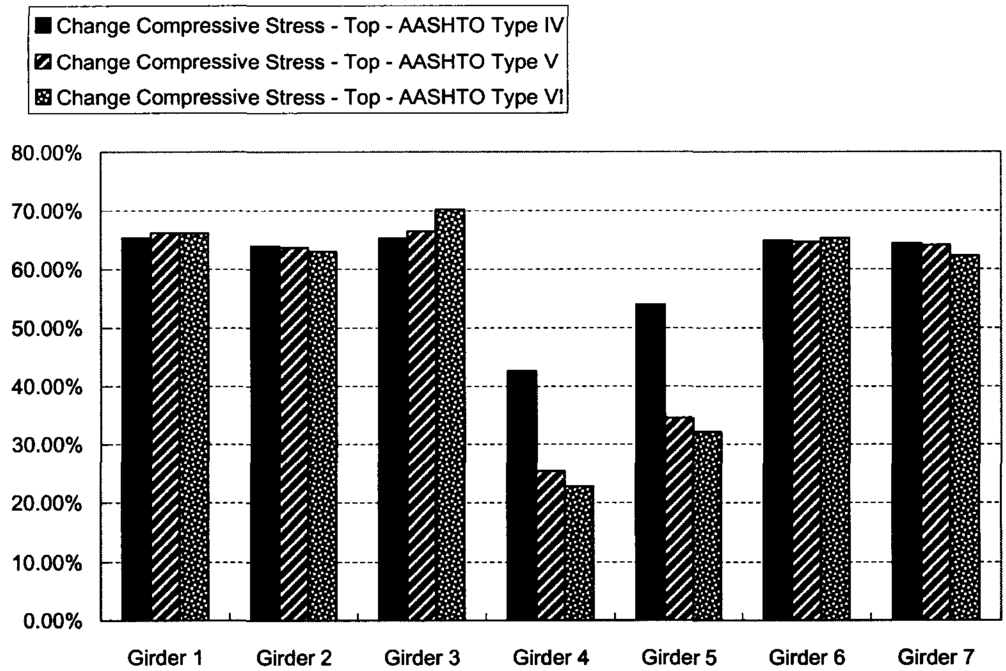


Fig. 4.22 Long Term Effects on Compressive Stresses at the Top of AASHTO I Type Bridge Girders – Girder Spacing Five ft

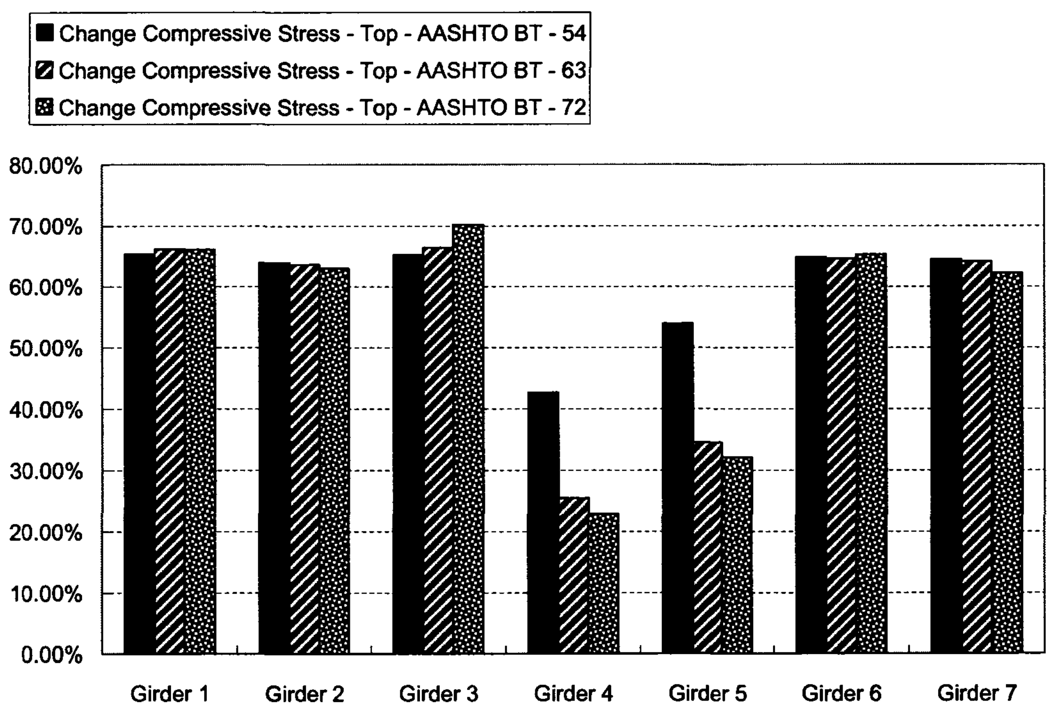


Fig. 4.23 Long Term Effects on Compressive Stresses at the Top of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Five ft

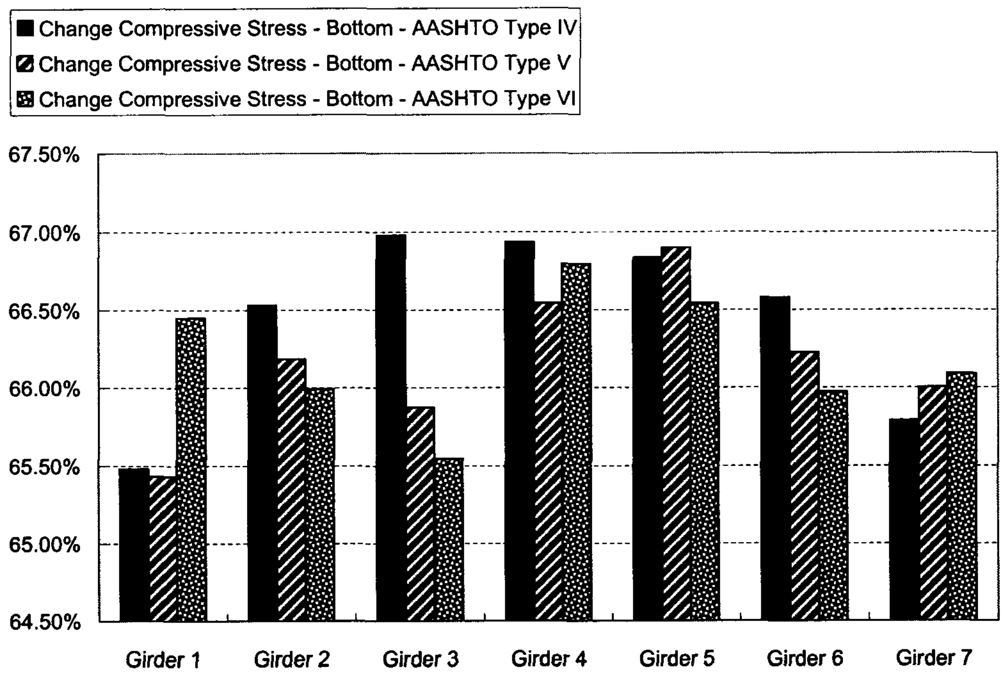


Fig. 4.24 Long Term Effects on Compressive Stresses at the Bottom of AASHTO I Type Bridge Girders – Girder Spacing Five ft

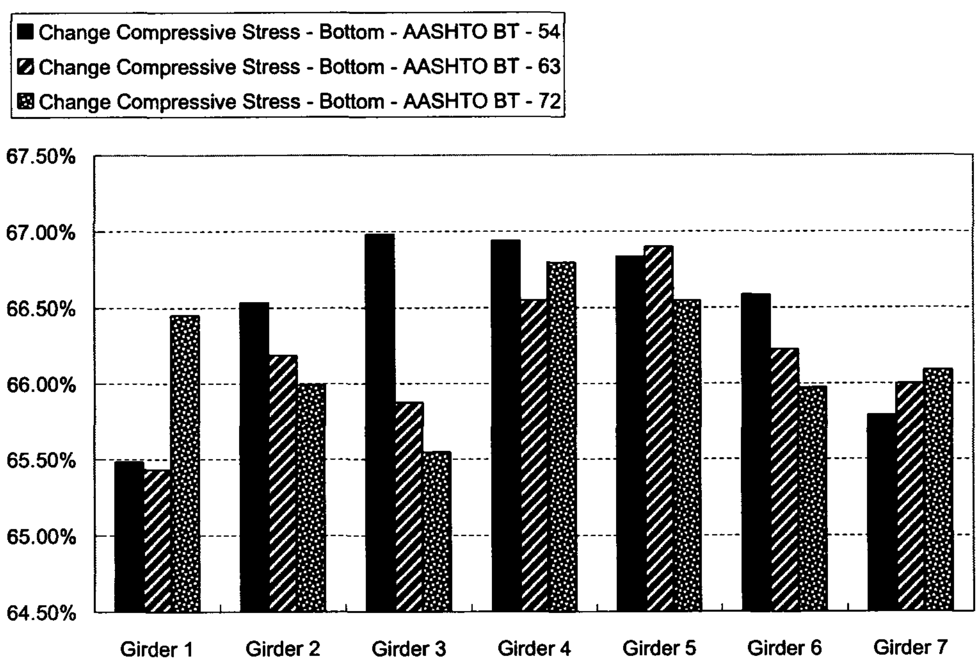


Fig. 4.25 Long Term Effects on Compressive Stresses at the Bottom of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Five ft

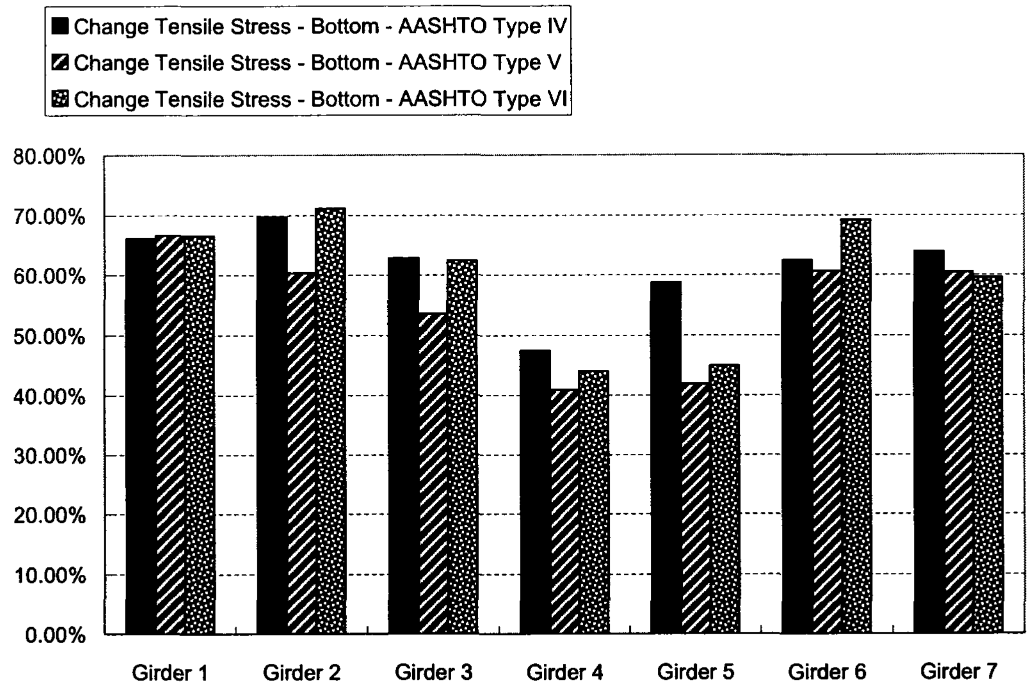


Fig. 4.26 Long Term Effects on Tensile Stresses at the Bottom of AASHTO I Type Bridge Girders – Girder Spacing Five ft

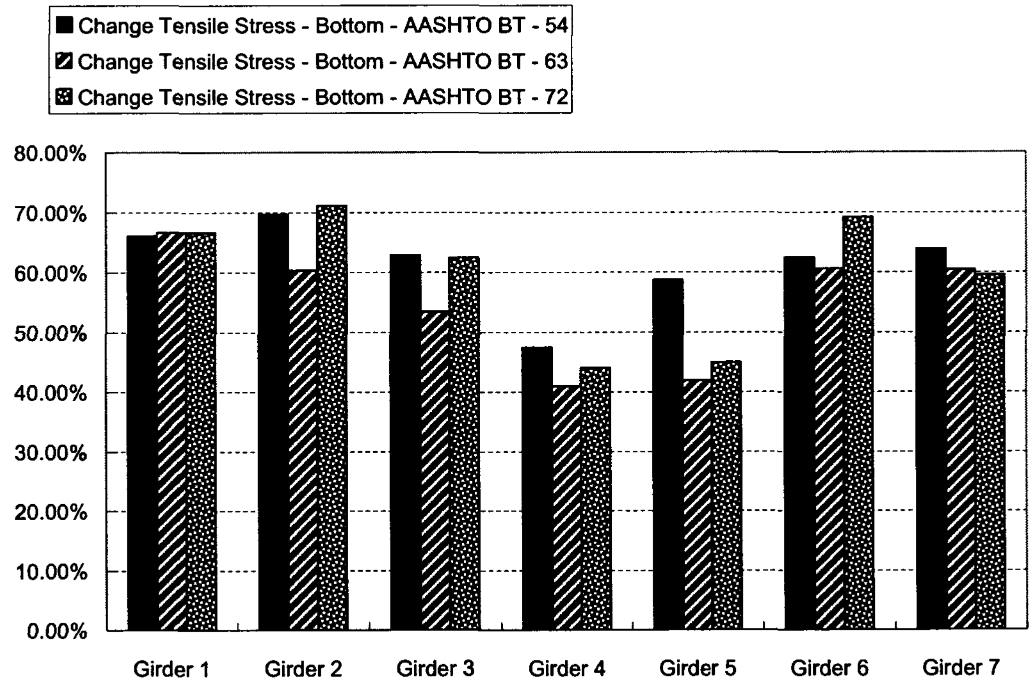


Fig. 4.27 Long Term Effects on Tensile Stresses at the Bottom of AASHTO Bulb-Tee Type Bridge Girders – Girder Spacing Five ft

4.3.3 Short Term Effects of Heavy Truck Load on Continuous Span Bridge Girders

The results of all bridges with girder type AASHTO Type IV, V, VI, BT-54, BT-63 and BT-72 were compared to determine the short term effects of FHWA 3S2 truck load on continuous span bridge girders.

In this study, the word "continuous" refers to the bridge models which have three equal span lengths, simply supported at each span, and with the continuous placed deck above the bridge girders. The methodology used in this part is similar to that used in the simply supported bridge analysis. The short term effects of FHWA 3S2 truck loads on continuous bridges designed for HS20-44 truck loads were evaluated by computing the percent change of the maximum stress at both top and bottom surfaces of each girder, then finding the maximum rate for each model. Three load combinations "Strength I max," "Strength III max," and "Strength V max," based on AASHTO LRFD bridge design specifications, were used to evaluate the short term performances of the bridge girders. By comparing the stress state of bridge girders under these three load combinations, the load combination "Strength I max" lead the maximum stresses of the girder. Therefore, we could determine that the "Strength I max" is the governing load combination for the short term effects analysis, and all the analysis below were based on it.

The bridges analyzed in this investigation were 30 ft. wide continuous bridges. The span length ranged from 20 to 105 ft. The slab thickness considered was eight-inch as the constant. The bridge model may contain five or seven girders; the girder spacing in this model was eight or five ft., respectively. The detail models and their properties were presented in chapter III.

In this study, the short-term effects of FHWA 3S2 truck loads on continuous span bridges designed for HS20-44 truck loads were evaluated by computing the percent change of the maximum stress at both top and bottom surfaces of each girder, then found the maximum rate of each model. The truck loads were placed on the deck at critical locations where the loads generated maximum positive or negative moments on the model. Only the compressive stress was considered at top surface of the girder, while the tensile stresses was obtained at bottom surface of the girder.

The percent changes of maximum stress along the bridge span length of the models with eight ft spacing were presented in Figs. 4.28-4.31. When the truck load was placed on the maximum negative moment location, the results indicated that the change in the compressive stress for all types of girders at the top surface of the girder did not exceed 10% except bridge built with AASHTO Type IV girder at the span length 20 ft.. The changes in the tensile stress for all types of girders at the bottom surface of the girder were larger than 10% while the span length was from 20 ft. to 30 ft., and less than 10% while the span length was from 30 ft. to 105 ft.. When the truck load was placed on the maximum positive moment location, the results indicated that the change in the compressive stress for all types of girders at the top surface of the girder were larger than 10% while the span length was from 20 ft. to 35 ft., and less than 10% while the span length was from 35 ft. to 105 ft.. The changes in the tensile stress for all types of girders at the bottom surface of the girder were larger than 10% while the span length was from 20 ft. to 40 ft., and less than 10% while the span length was from 40 ft. to 105 ft.. The bridges in this study with stress percent change which was greater than 10% would be considered as overstressed, and might experience more cracking in the bridge girders. Therefore, the bridges with span length 40 ft. to 105 ft. did not meet the overstress condition; the short term effects of the heavy truck load on those bridges were limited. The bridges with span length 20 ft. to 40 ft. may be overstressed while the FHWA 3S2 truck traveled on them; those bridge girders might need a special attention to their strength.

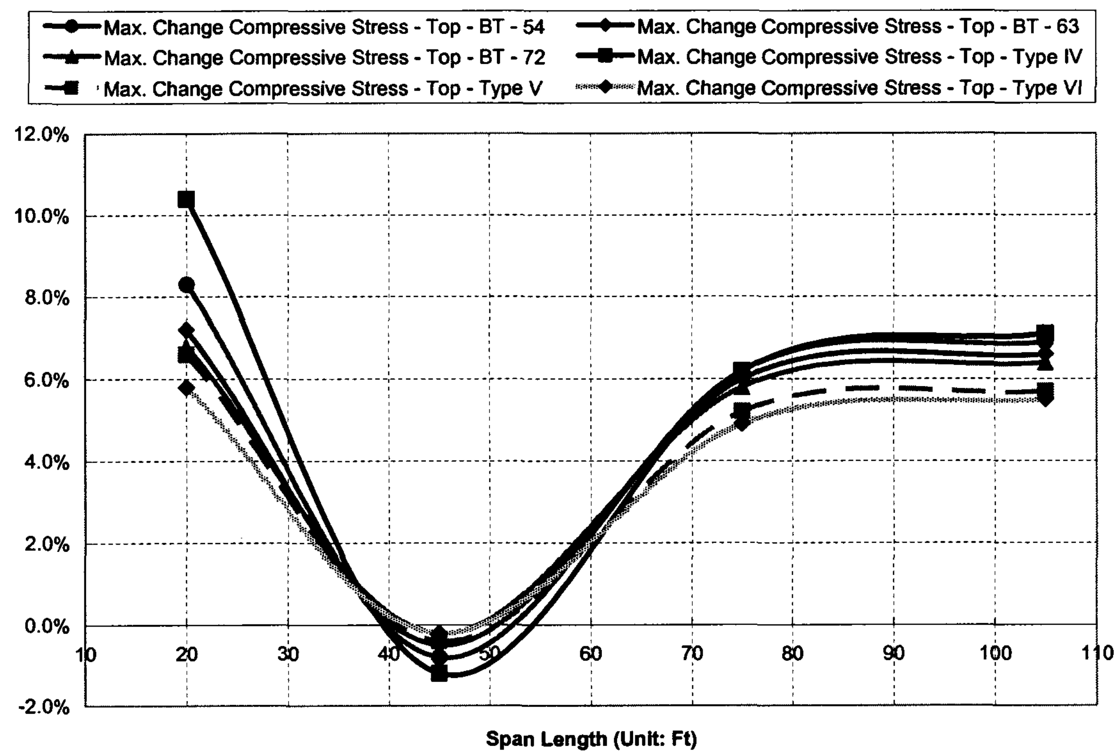


Fig. 4.28 Short Term Effects on Compressive Stresses at the Top of Bridge Girders – Negative Moment Location, Girder Spacing Eight ft

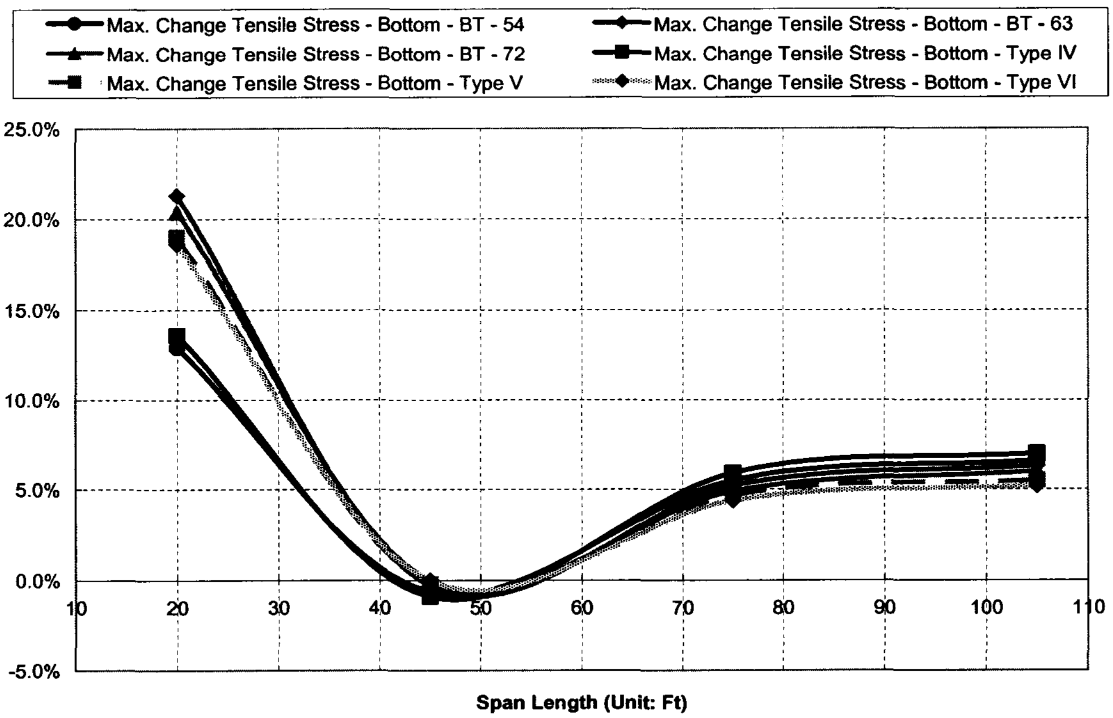


Fig. 4.29 Short Term Effects on Tensile Stresses at the Bottom of Bridge Girders – Negative Moment Location, Girder Spacing Eight ft

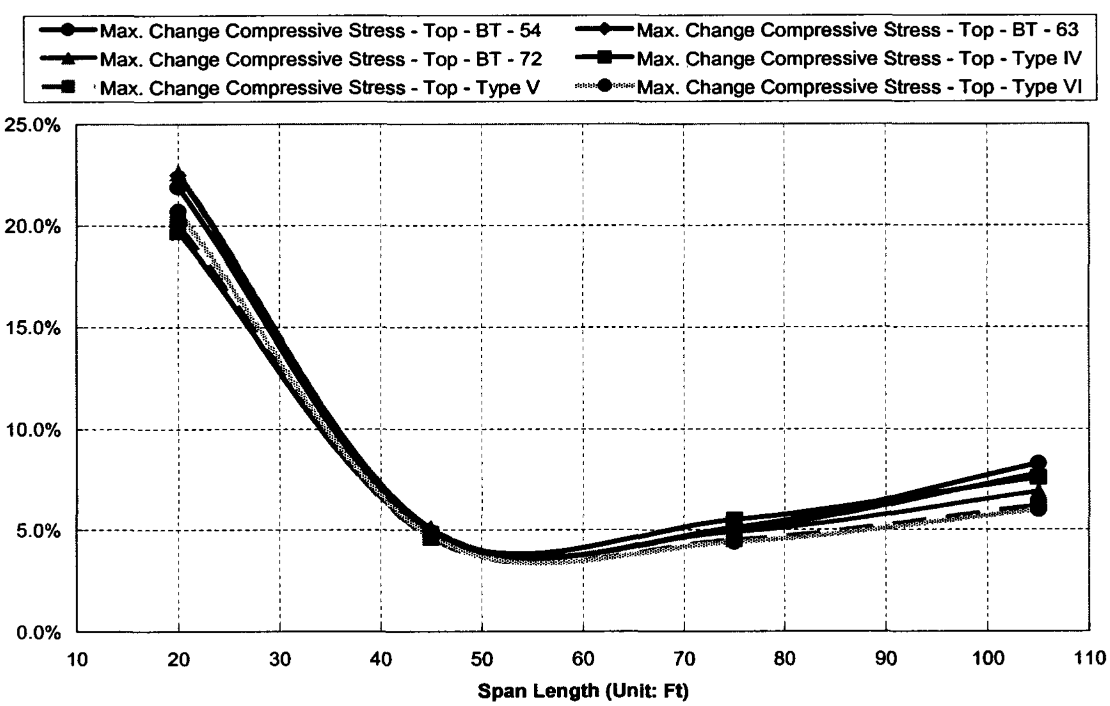


Fig. 4.30 Short Term Effects on Compressive Stresses at the Top of Bridge Girders – Positive Moment Location, Girder Spacing Eight ft

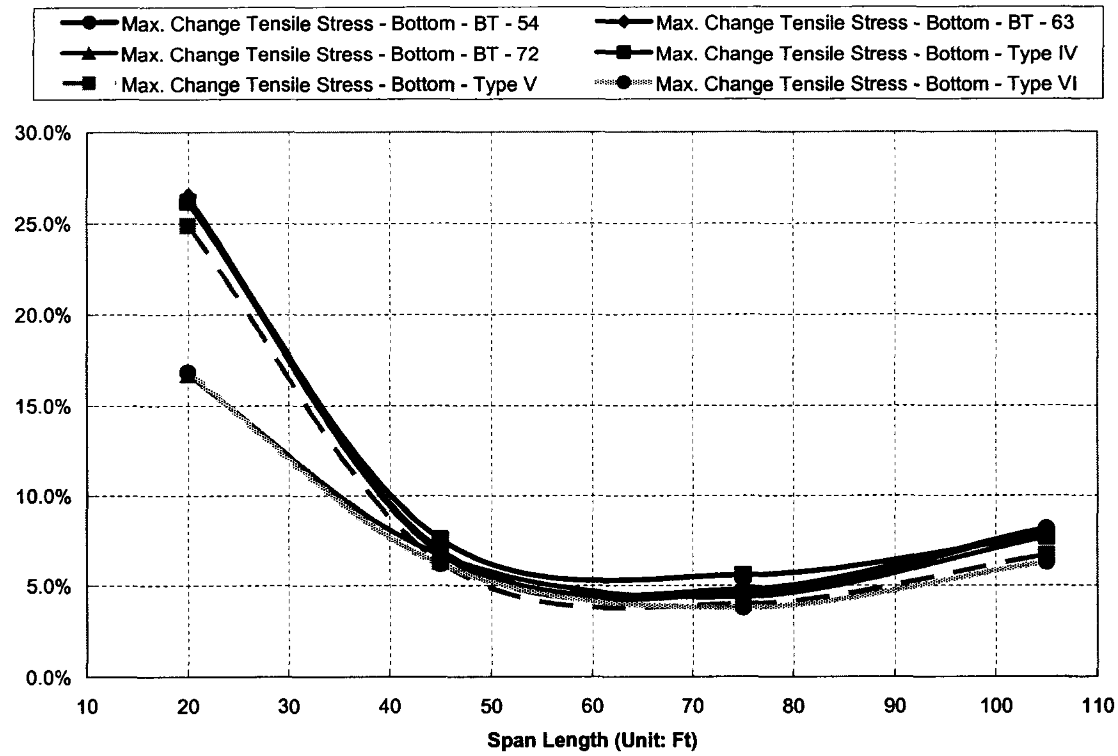


Fig. 4.31 Short Term Effects on Tensile Stresses at the Bottom of Bridge Girders – Positive Moment Location, Girder Spacing Eight ft

The percent changes of maximum stress along the bridge span length of the models with five ft spacing were presented in Fig. 4.32 to Fig. 4.35. When the truck load was placed on the maximum negative moment location, the results indicated that the change in the compressive stress for AASHTO Type IV, V and VI girders at the top surface of the girder did not exceed 10%. For bridges built with Bulb-Tee girders, the percent changes were larger than 10% when bridge span lengths were 20 ft. to 30 ft.. The changes in the tensile stress for all types of girders at the bottom surface of the girder were larger than 10% while the span length was from 20 ft. to 30 ft. except bridge built with AASHTO Type IV girder, and less than 10% while the span length was from 30 ft. to 105 ft. When the truck load was placed on the maximum positive moment location, the results indicated that for Type V and VI girder, the changes in compressive stress at the top surface of the girder were always smaller than 10%; the change in the compressive stress for other types of girders at the top surface of the girder were larger than 10% while the span length was from 20 ft. to 35 ft, and less than 10% while the span length was from 35 ft. to 105 ft. The changes in the tensile stress for all types of girders at the bottom surface of the girder were larger than 10% while the span length was from 20 ft. to 40 ft., and less than 10% while the span length was from 40 ft. to 105 ft. The bridges in this study with stress percent change which was greater than 10% would be considered as overstressed, and might experience more cracking in the bridge girders. Therefore, the bridges with span length 40 ft. to 105 ft. did not meet the overstress condition; the short term effects of the heavy truck load on those bridges were limited. The bridges with span length 20 ft. to 40 ft, built with AASHTO Type IV and Bulb-Tee girders, might be overstressed while the FHWA 3S2 truck traveled on them; those bridge girders might need a special attention to their strength.

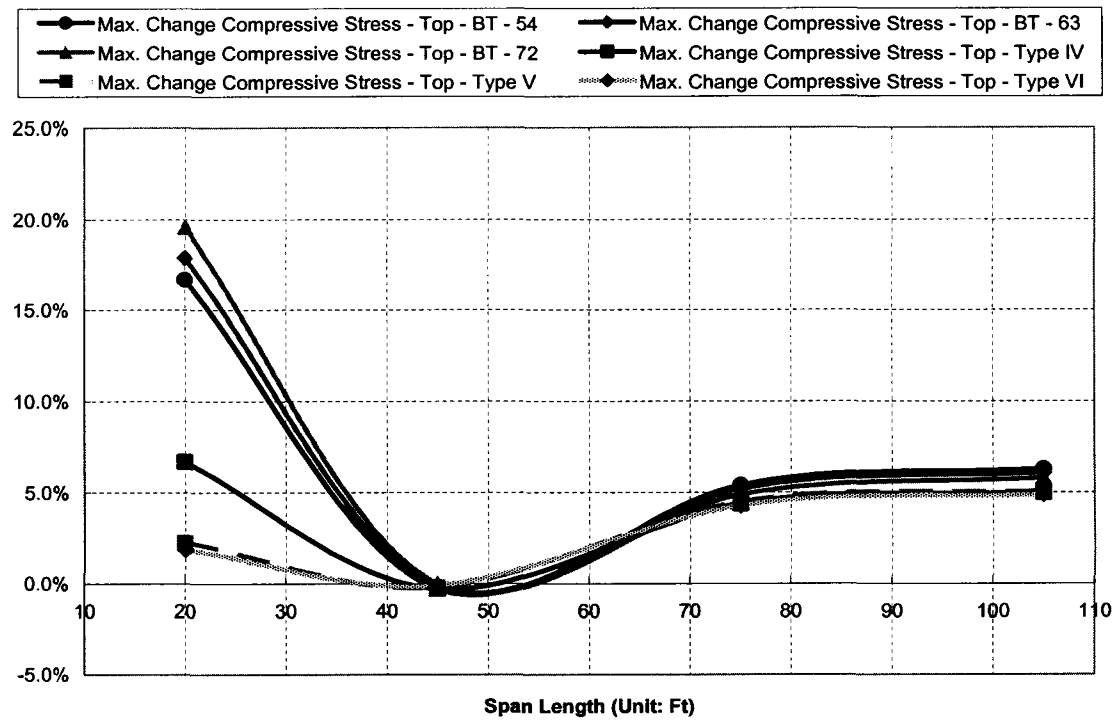


Fig. 4.32 Short Term Effects on Compressive Stresses at the Top of Bridge Girders – Negative Moment Location, Girder Spacing Five ft

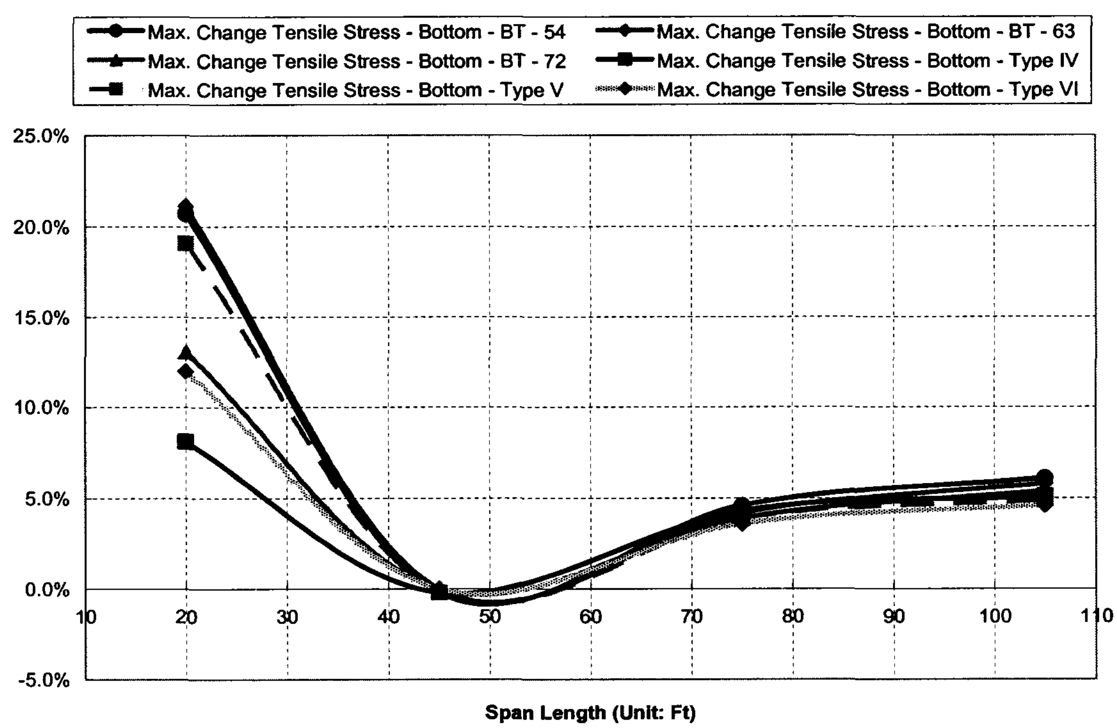


Fig. 4.33 Short Term Effects on Tensile Stresses at the Bottom of Bridge Girders – Negative Moment Location, Girder Spacing Five ft

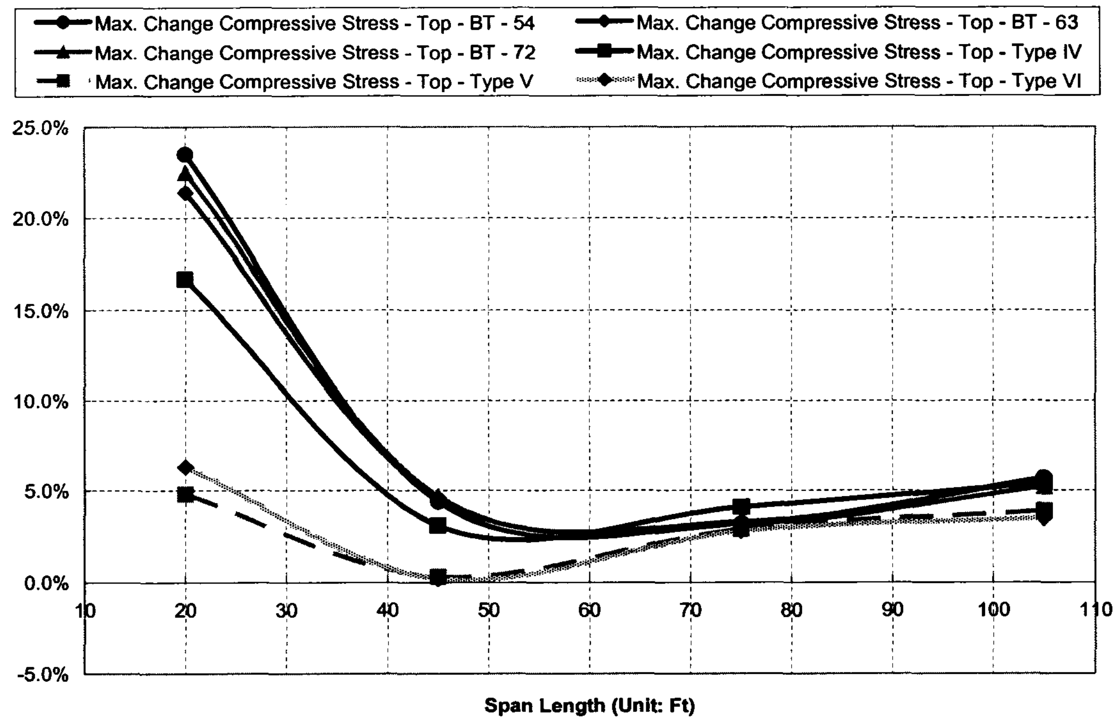


Fig. 4.34 Short Term Effects on Compressive Stresses at the Top of Bridge Girders – Positive Moment Location, Girder Spacing Five ft

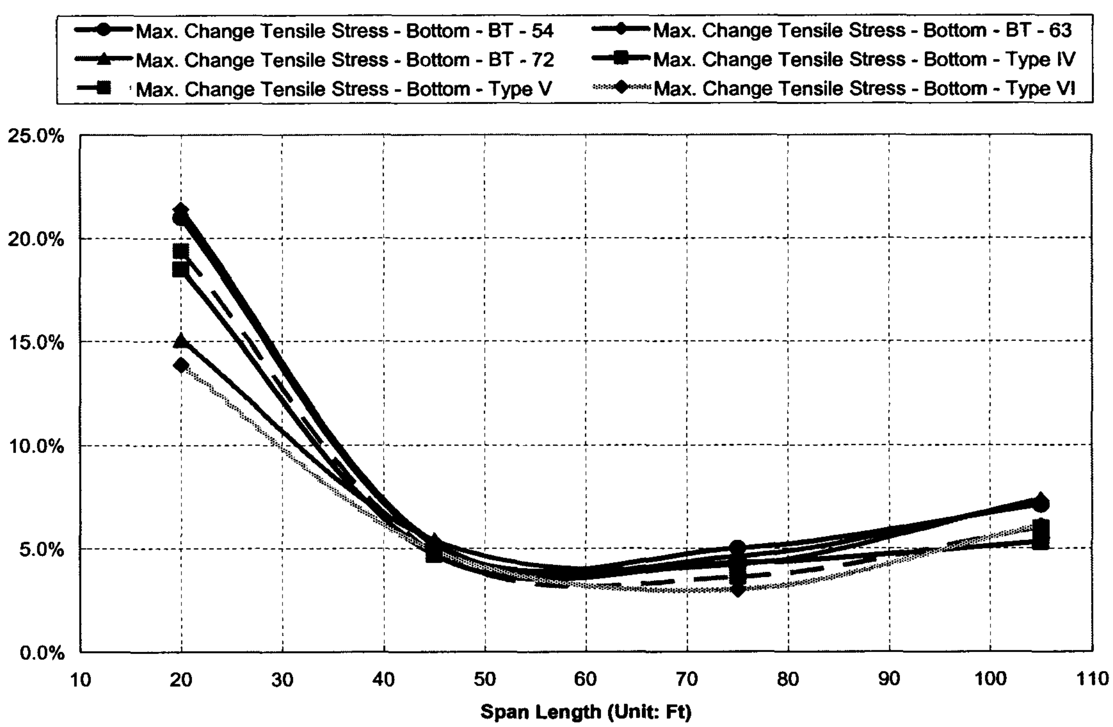


Fig. 4.35 Short Term Effects on Tensile Stresses at the Bottom of Bridge Girders – Positive Moment Location, Girder Spacing Five ft

4.3.4 Long Term Effects of Heavy Truck Load on Continuous Span Bridge Girders

The results of all bridges with girder type AASHTO Type IV, V, VI, BT-54, BT-63 and BT-72 were compared to determine the long term effects of FHWA 3S2 truck load on bridge girders.

The long term effects of FHWA 3S2 truck loads on continuous bridges designed for HS20-44 truck loads were evaluated also by computing the percent change of the maximum stress at both top and bottom surfaces of each girder, then finding the maximum rate for each model. Based on AASHTO LRFD bridge design specifications, the load combination "Fatigue" was considered as the critical load combination for the long term effects analysis.

The bridges analyzed in this investigation were 30 ft. wide continuous bridges. The span length ranged from 20 to 105 ft. The slab thickness considered was eight-inch as the constant. The bridge model may contain five or seven girders; the girder spacing in this model was eight or five ft., respectively. The detail models and their properties were presented in chapter III.

The percent changes of maximum stress along the bridge span length of the models with eight ft spacing were presented in Figs. 4.36-4.39. Only the compressive stress was considered at top surface of the girder, while the tensile stress was obtained at bottom surface of the girder. The results indicated that the percent changes in long term stresses for all types of girders were much higher than those changes of short term stresses. The percent changes of stresses were mostly around 50%, some of which might be more than 90%. It was observed that the long term girder stress values under both 3S2 and HS20-44 truck loads were smaller than those short-term stress values, although the percent changes were significantly higher. This observation suggested that even the long-term girder stresses had not exceeded the maximum allowable stresses of the components; the effects of heavy truck loads could not be neglected. Another observation was that while the span length increased the percent changes went to 50%, no matter what kind the girder was. The effects of changing girder types on the long term impact of 3S2 truck load on bridges were limited. For a long period, the heavy trucks travel on the bridge would have remarkable effects on the bridge safety and serviceability. Therefore, such conditions could result in an increase in the flexural cracks on bridge girders and would require additional inspections and could result in early and frequent maintenance.

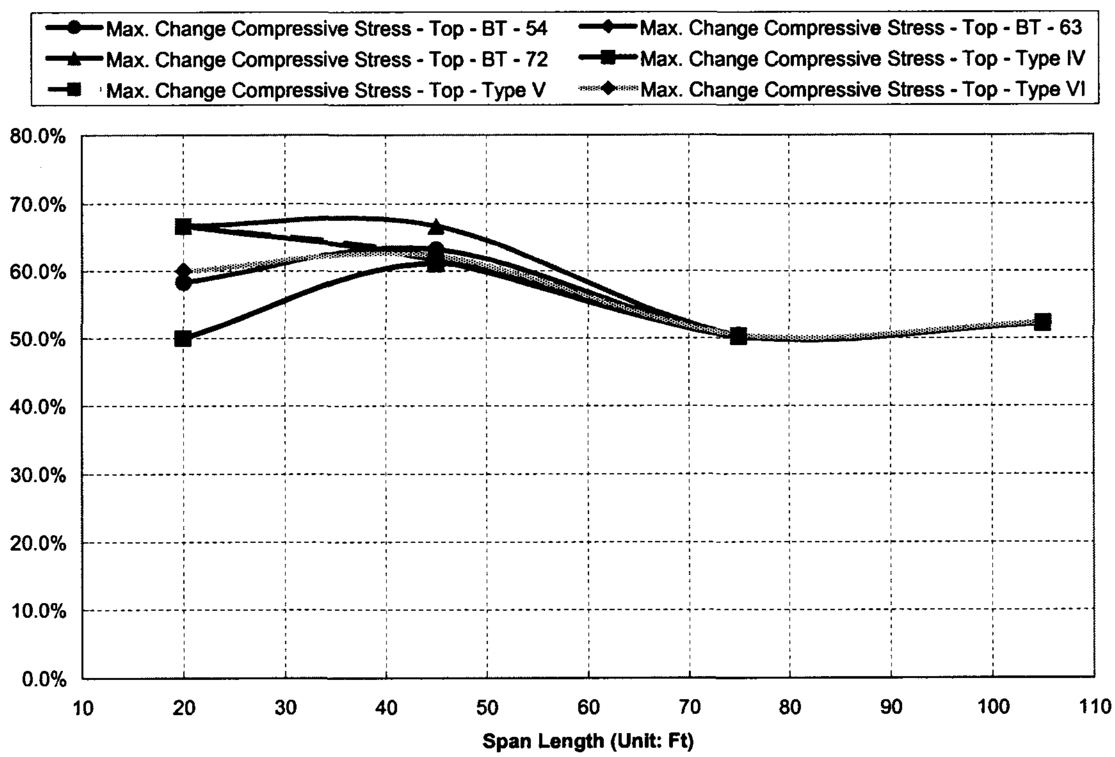


Fig. 4.36 Long Term Effects on Compressive Stresses at the Top of Bridge Girders – Negative Moment Location, Girder Spacing Eight ft

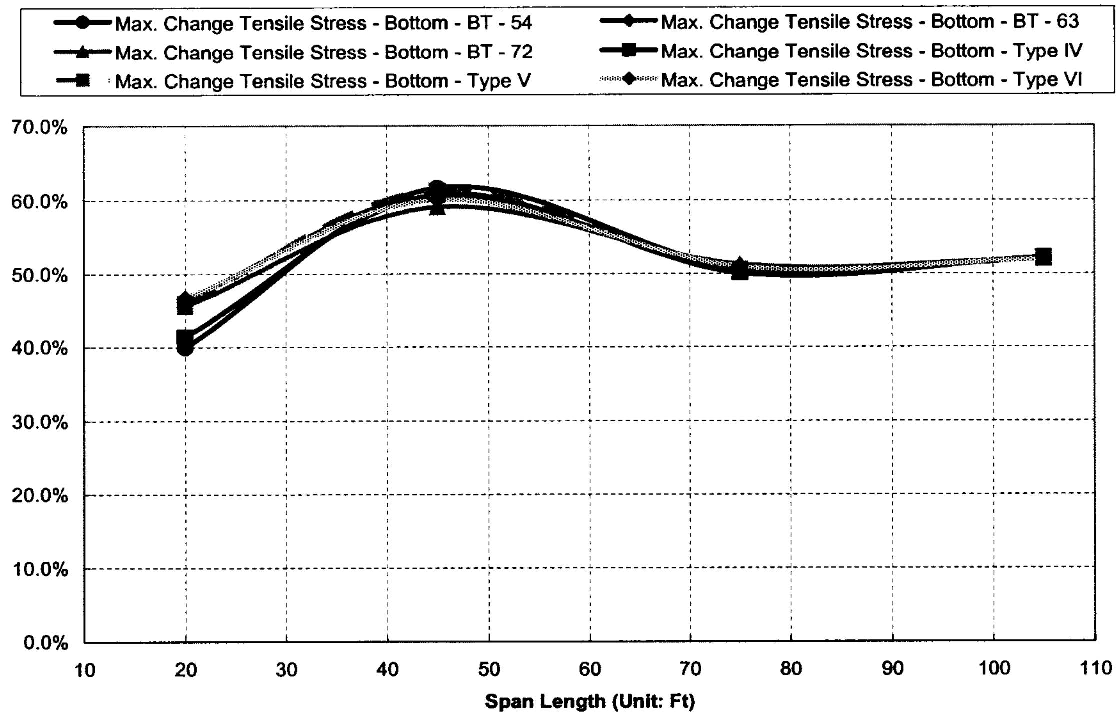


Fig. 4.37 Long Term Effects on Tensile Stresses at the Bottom of Bridge Girders – Negative Moment Location, Girder Spacing Eight ft

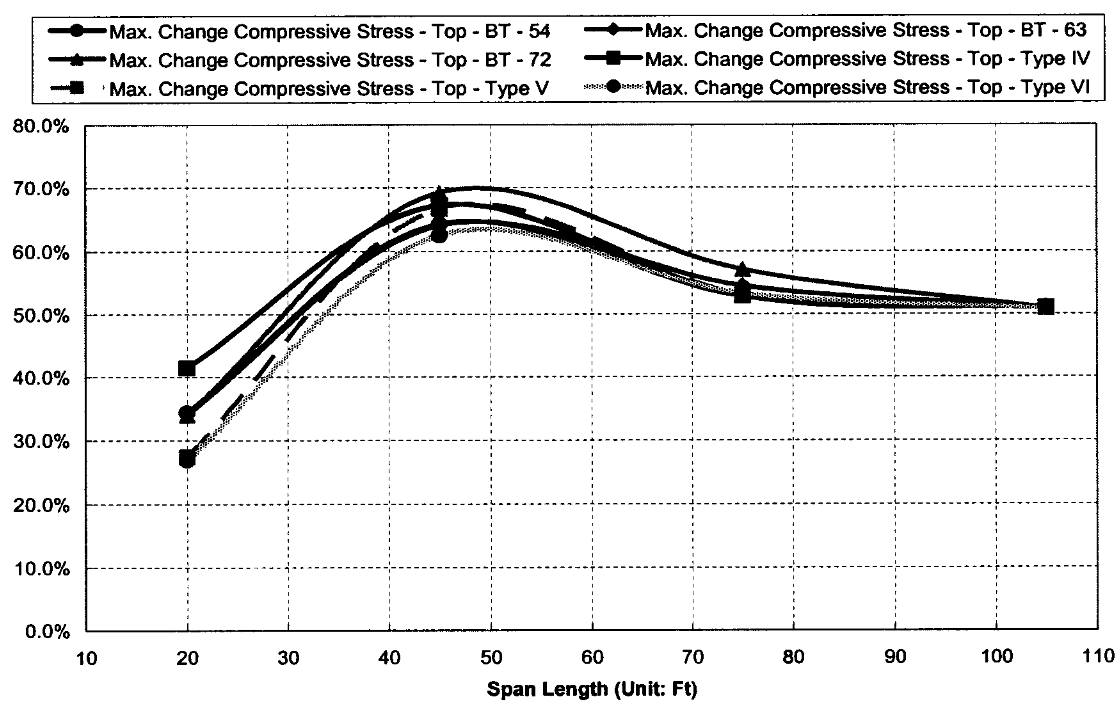


Fig. 4.38 Long Term Effects on Compressive Stresses at the Top of Bridge Girders – Positive Moment Location, Girder Spacing Eight ft

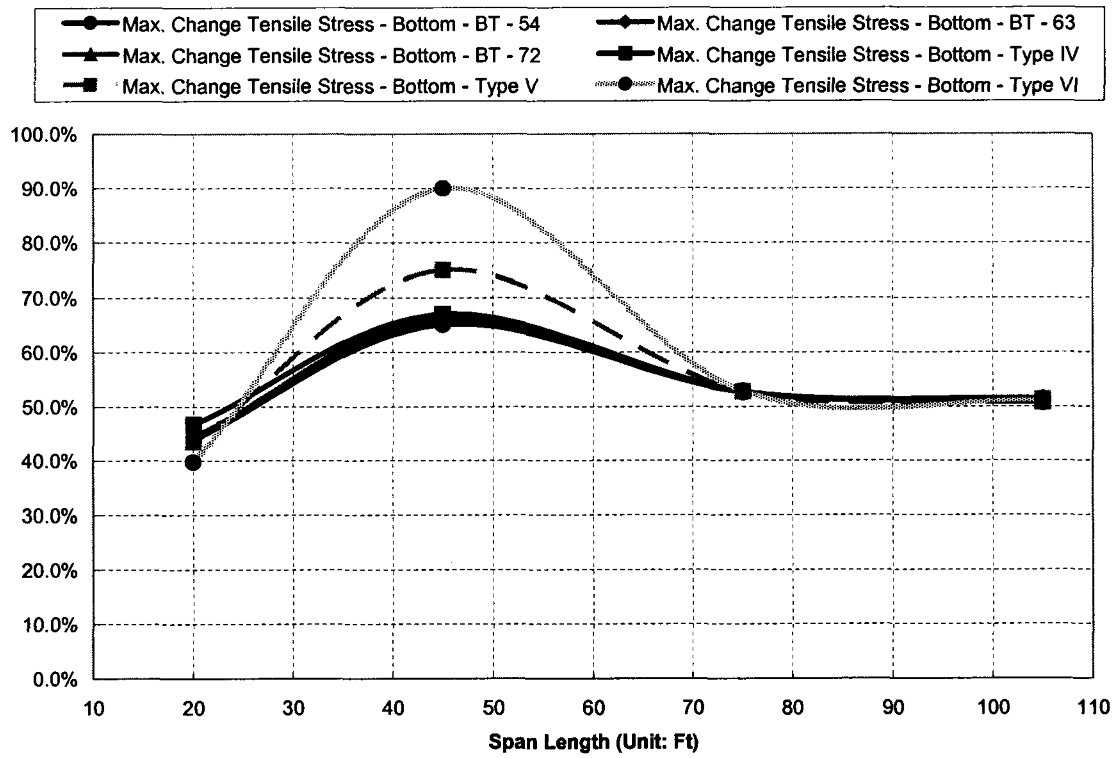


Fig. 4.39 Long Term Effects on Tensile Stresses at the Bottom of Bridge Girders – Positive Moment Location, Girder Spacing Eight ft

The percent change of maximum stress of each individual girder of the model with five ft spacing is presented in Figs. 4.40-4.43. Only the compressive stress was considered at top surface of the girder, while the tensile stress was obtained at bottom surface of the girder. The results indicated that the percent changes in long term stresses for all types of girders were much higher than those changes of short term stresses. The percent changes of stresses were mostly around 50%, some of which might be more than 100%. Also, the long-term girder stress values under both 3S2 and HS20-44 truck loads were smaller than those short term stress values, although the percent changes were significantly higher, which indicated that even the long term girder stresses did not exceed the maximum allowable stresses of the components; the effects of heavy truck loads can not be neglected. For a long period, the heavy trucks traveled on the bridge would have remarkable effects on the bridge safety and serviceability. Therefore, these bridges might experience flexural cracks in the bridge girders. Such cracks would require additional inspections and could result in early and frequent maintenance. The difference between two groups of models with different girder spacing was very little. When the span lengths were short, the models with five ft girder spacing had a worse performance than the eight ft girder spacing models. This difference implied that the girder spacing was not a governing parameter to be considered when evaluating the long term heavy truck load impact.

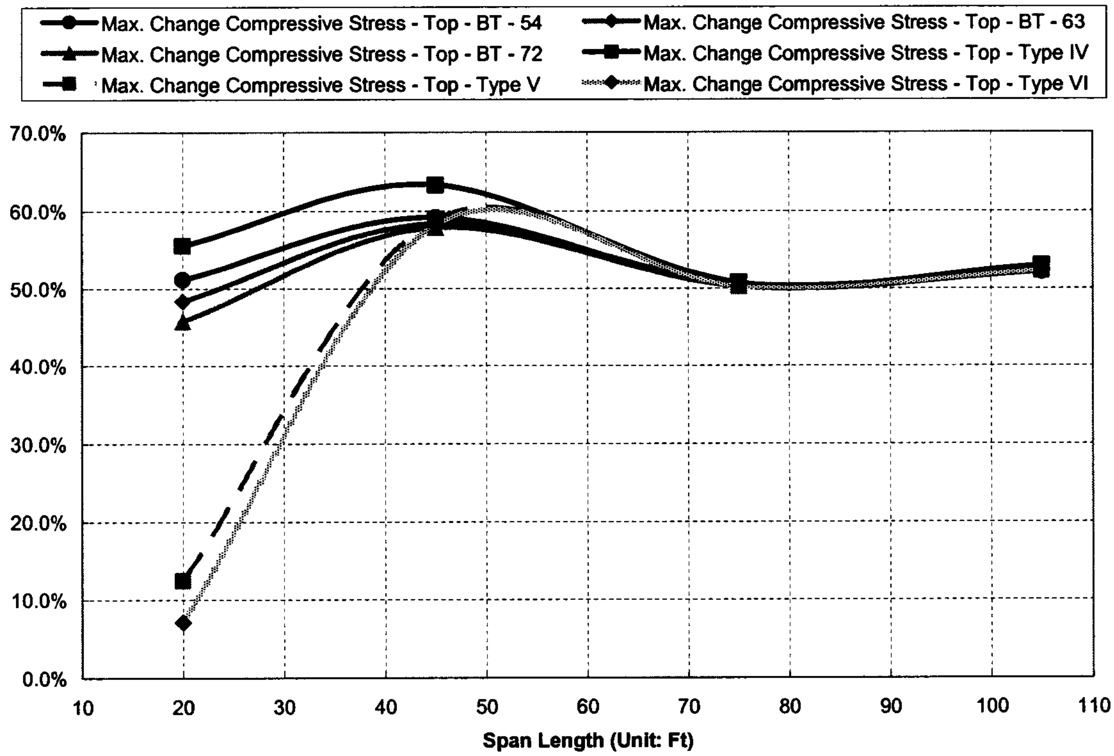


Fig. 4.40 Long Term Effects on Compressive Stresses at the Top of Bridge Girders – Negative Moment Location, Girder Spacing Five ft

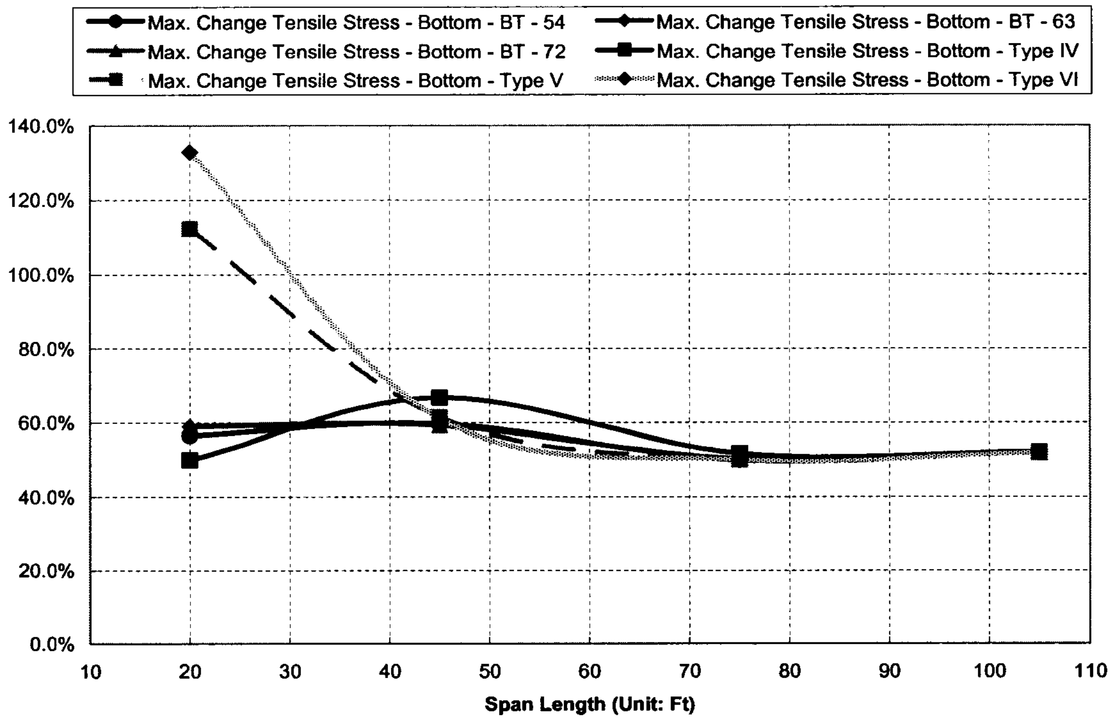


Fig. 4.41 Long Term Effects on Tensile Stresses at the Bottom of Bridge Girders – Negative Moment Location, Girder Spacing Five ft

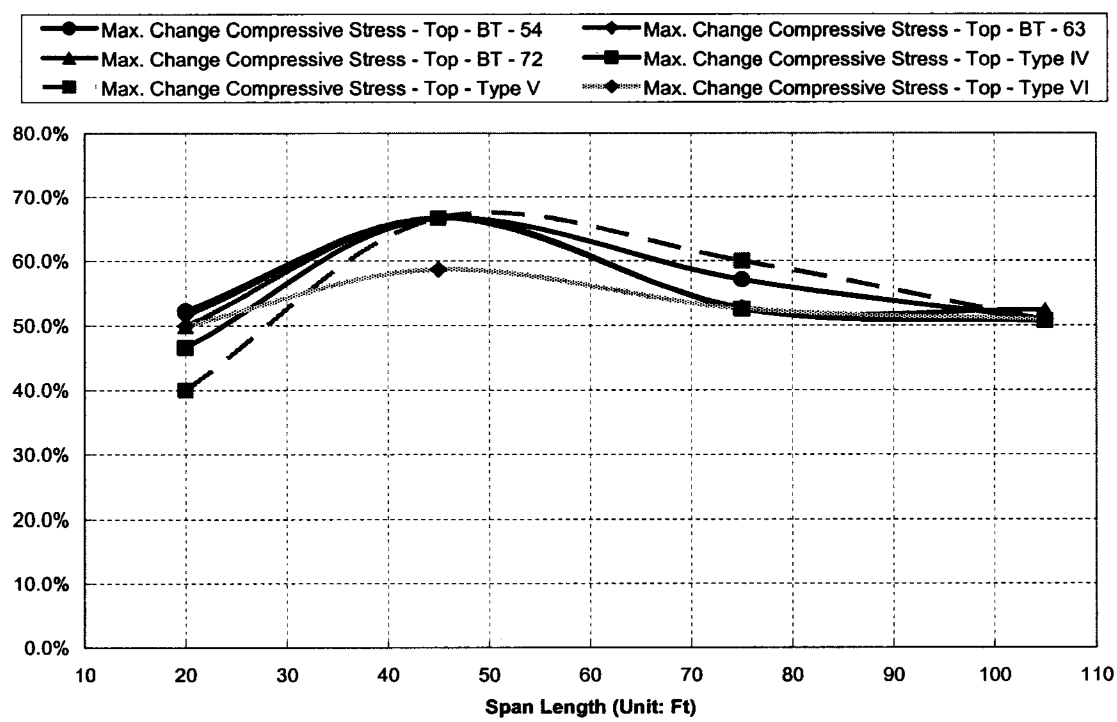


Fig. 4.42 Long Term Effects on Compressive Stresses at the Top of Bridge Girders – Positive Moment Location, Girder Spacing Five ft

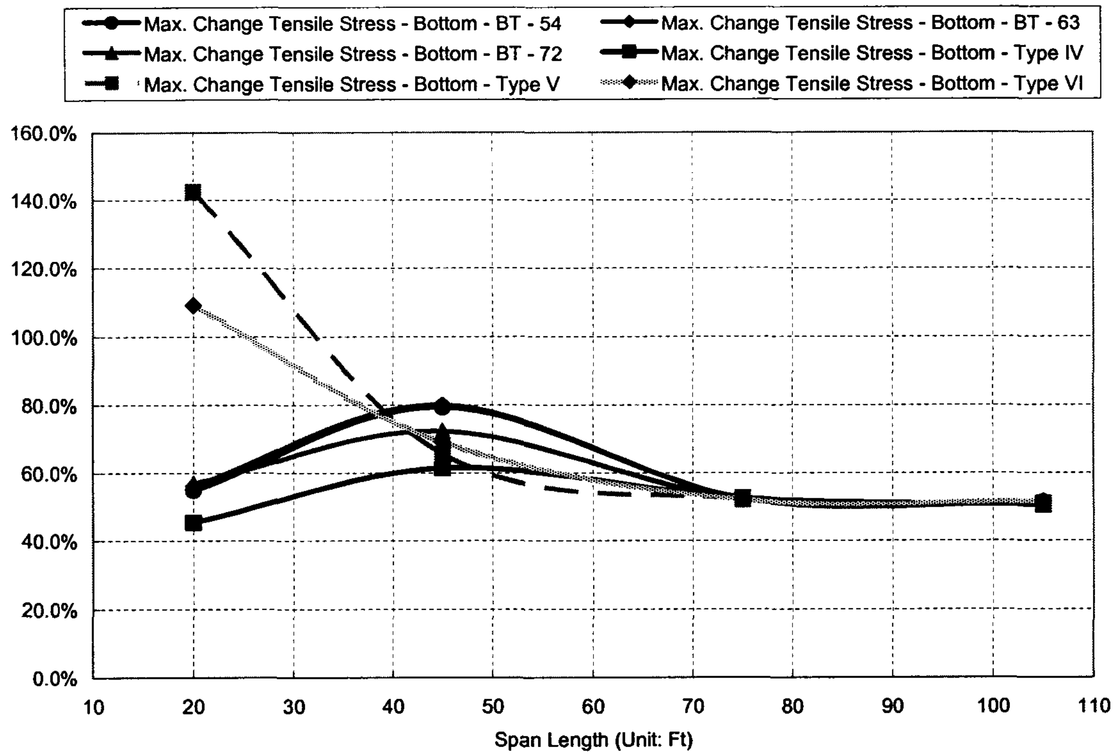


Fig. 4.43 Long Term Effects on Tensile Stresses at the Bottom of Bridge Girders – Positive Moment Location, Girder Spacing Five ft

4.4 Summary

The girder performance under the heavy truck loads were evaluated in this chapter. Simply supported bridges with span length from 20 ft. to 120 ft. and continuous bridges with span length from 20 ft. to 130 ft. were analyzed by simplified AASHTO line girder analysis approach in section 4.2. Different configurations of medium span simply supported bridge girders and continuous bridge girders were analyzed by the finite element method via GTSTRUDL in section 4.3.

Results from this chapter would be utilized in chapter VIII to perform the bridge costs evaluation.

CHAPTER V

BRIDGE DECK PERFORMANCE UNDER THE HEAVY TRUCK LOAD

5.1 Introduction

The materials in bridges are 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 damage would especially occur on those bridges that meet with the heavily traveled vehicles. In this study, the fatigue behavior of bridge decks 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 decks. According to the AASHTO specification, the fatigue factor 0.75 and the impact factor 1.3 were used. The investigation used the same finite element models as described in previous chapters. Truck loads for HS20-44, FHWA 3S2 and FHWA 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, then to be analyzed.

5.2 Evaluation of Continuous Bridge Decks under FHWA 3S2 Truck Load

This subtask focused on the strength and serviceability of bridge decks under the impact of the heavy truck loads. The evaluation considered composite and non-composite bridge systems. 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 were generated with a typical 30-ft. bridge-deck width and eight-inch thickness supported by five AASHTO type IV girders, the girders are spaced at eight ft. in the middle and seven ft. on the outside. The design load for the bridges included in this study and the loads from FHWA 3S2 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.

The finite element model used for bridge decks in this study simulated the behavior of continuous span bridges. The word "continuous" referred to the bridge models that had three equal span lengths, simply supported at each span, and with the continuous placed deck above the bridge girders. The span lengths of the bridges were in the range of 20 to 120 feet. 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. 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.

The effects of FHWA 3S2 truck loads on continuous bridge decks designed for HS20-44 truck loads are presented in Tables 5.1 and 5.2 and Figs. 5.1 to 5.6. 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.

At the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction varied between 0.91 and 1.74 and between 0.71 and 1.37 in the transverse direction. The ratio of shear stress varied between 0.87 and 1.59. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction varied between 0.58 and 1.09, and between 0.90 and 1.10 in the transverse direction; the ratio of shear stress varied between 0.98 and 2.23. The ratio of maximum compressive stress was mostly smaller than the ratio of maximum tensile stress. The ratios of maximum tensile stress in the longitudinal direction were larger than 1.15 when the span length was longer than 30 ft. Therefore, these bridge decks may experience cracks in the longitudinal direction. The ratio of maximum shear stress was usually higher than others. Thus the decks may experience more cracks in vertical direction. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or FHWA 3S2 truck loads may differ from each other. The difference is what makes the ratio of 3S2 to HS20-44 truck for some span lengths less than 1.

At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction varied between 0.58 and 1.09, in the transverse direction varied between 0.90 and 1.10, the ratio of shear stress varied between 0.98 and 2.23. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction varied between 0.91 and 1.74, in the transverse direction varied between 0.71 and 1.37; the ratio of shear stress varied between 0.87 and 1.59. The ratio of maximum tensile stress was mostly smaller than the ratio of maximum compressive stress. The ratios of maximum compressive stress in the longitudinal direction were larger than 1.15 when the span length was longer than 30 ft. Therefore, these bridge decks may experience cracks in the longitudinal direction. The ratio of maximum shear stress was usually higher than others. Thus the decks may experience more cracks in the vertical direction. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or 3S2 truck loads may differ from each other. The difference is what makes the ratio of 3S2 to HS20-44 truck for some span lengths 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 5.1 Long Term Effects of 3S2 Truck Loads on Top Surface of Continuous Bridge Decks

Ratio of Max Value of Stress of FHWA 3S2 to HS20-44						
Span Length	Max Tensile Stress			Max Compressive Stress		
(ft.)	*Longitudinal*	*Transverse*	*Shear*	*Longitudinal*	*Transverse*	*Shear*
20	0.912	0.722	1.588	0.719	0.962	2.229
30	1.150	0.707	1.266	0.577	0.896	1.145
45	1.739	1.059	0.870	0.705	1.006	0.975
60	1.599	1.168	0.970	0.711	0.950	0.996
75	1.247	1.284	1.232	0.746	1.025	1.504
90	1.356	1.324	1.348	1.092	1.062	1.295
105	1.385	1.332	1.335	0.813	1.104	1.411
120	1.430	1.371	1.370	0.997	1.093	1.384

Table 5.2 Long Term Effects of 3S2 Truck Loads on Bottom Surface of Continuous Bridge Decks

Ratio of Max Value of Stress of FHWA 3S2 to HS20-44						
Span Length	Max Tensile Stress			Max Compressive Stress		
(ft.)	*Longitudinal*	*Transverse*	*Shear*	*Longitudinal*	*Transverse*	*Shear*
20	0.719	0.962	2.229	0.912	0.722	1.588
30	0.577	0.896	1.145	1.150	0.707	1.266
45	0.705	1.006	0.975	1.739	1.059	0.870
60	0.711	0.950	0.996	1.599	1.168	0.970
75	0.746	1.025	1.504	1.247	1.284	1.232
90	1.092	1.062	1.295	1.356	1.324	1.348
105	0.813	1.104	1.411	1.385	1.332	1.335
120	0.997	1.093	1.384	1.430	1.371	1.370

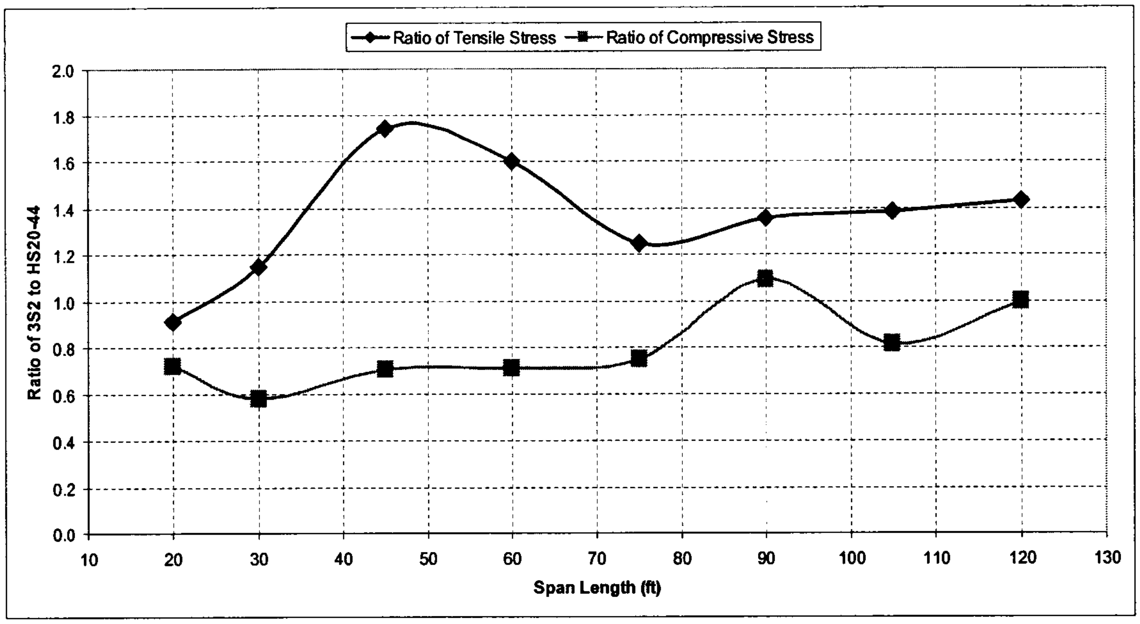


Fig. 5.1 Long Term Effects on Longitudinal Stress at Top Surface of Continuous Bridge Decks

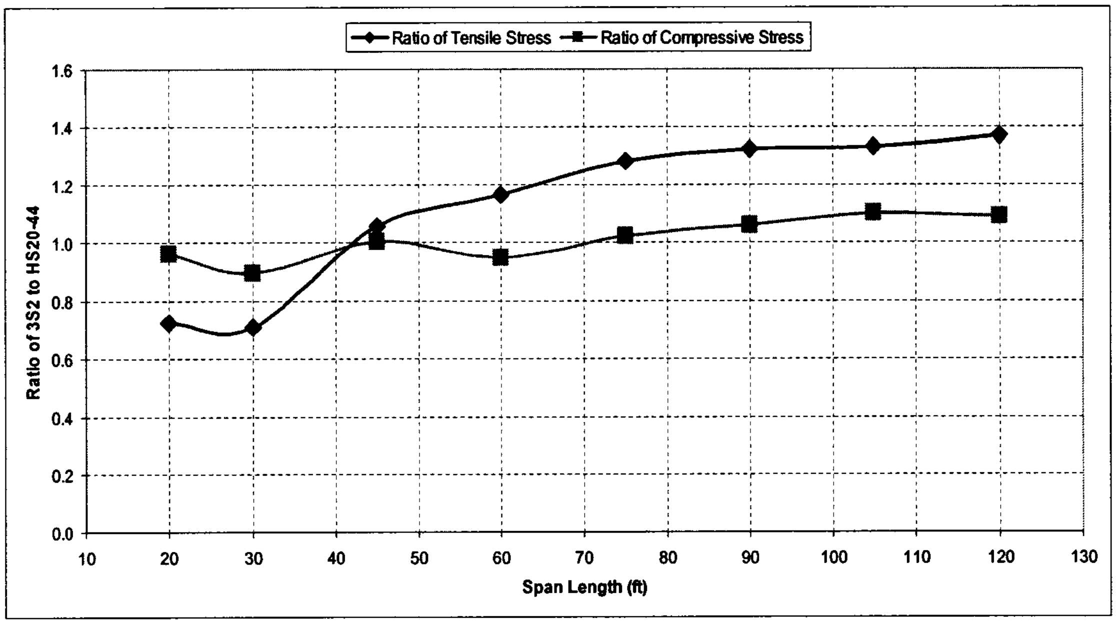


Fig. 5.2 Long Term Effects on Transverse Stress at Top Surface of Continuous Bridge Decks

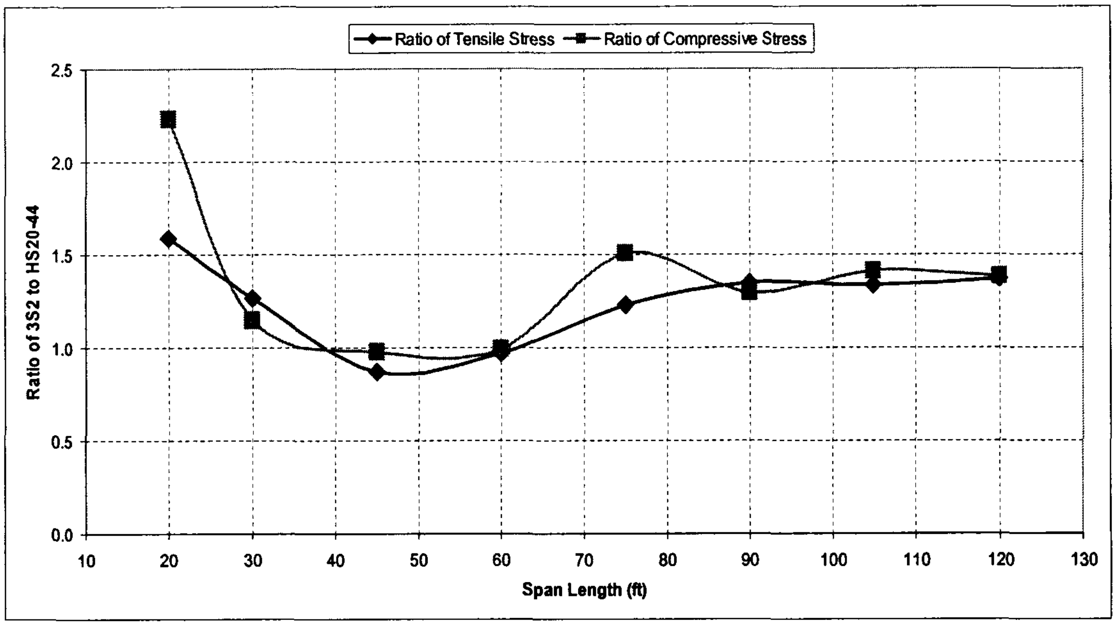


Fig. 5.3 Long Term Effects on Shear Stress at Top Surface of Continuous Bridge Decks

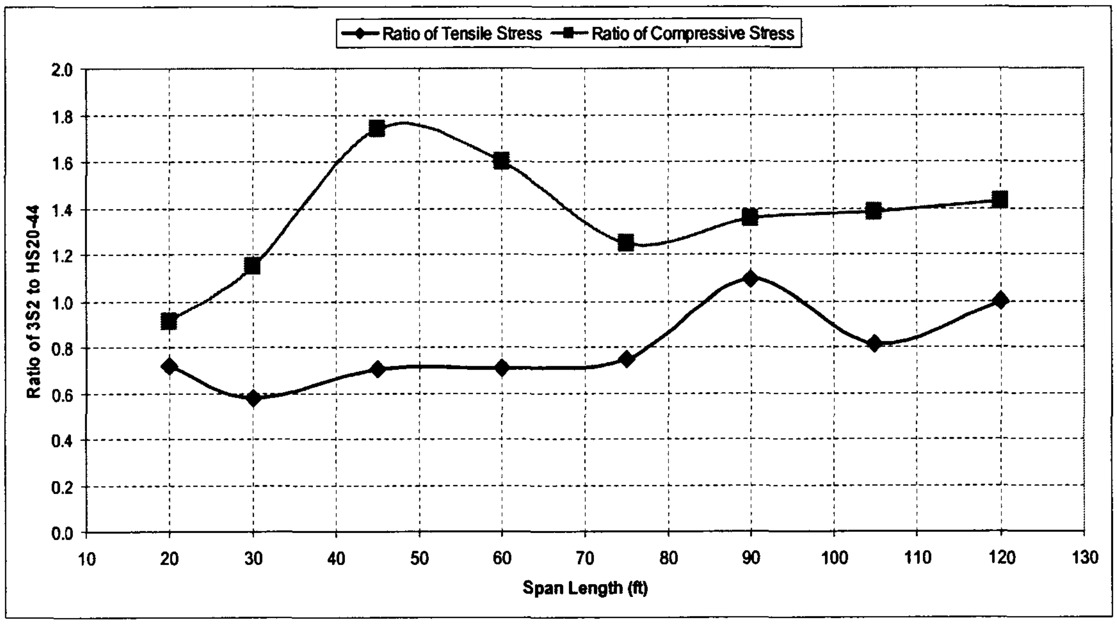


Fig. 5.4 Long Term Effects on Longitudinal Stress at Bottom Surface of Continuous Bridge Decks

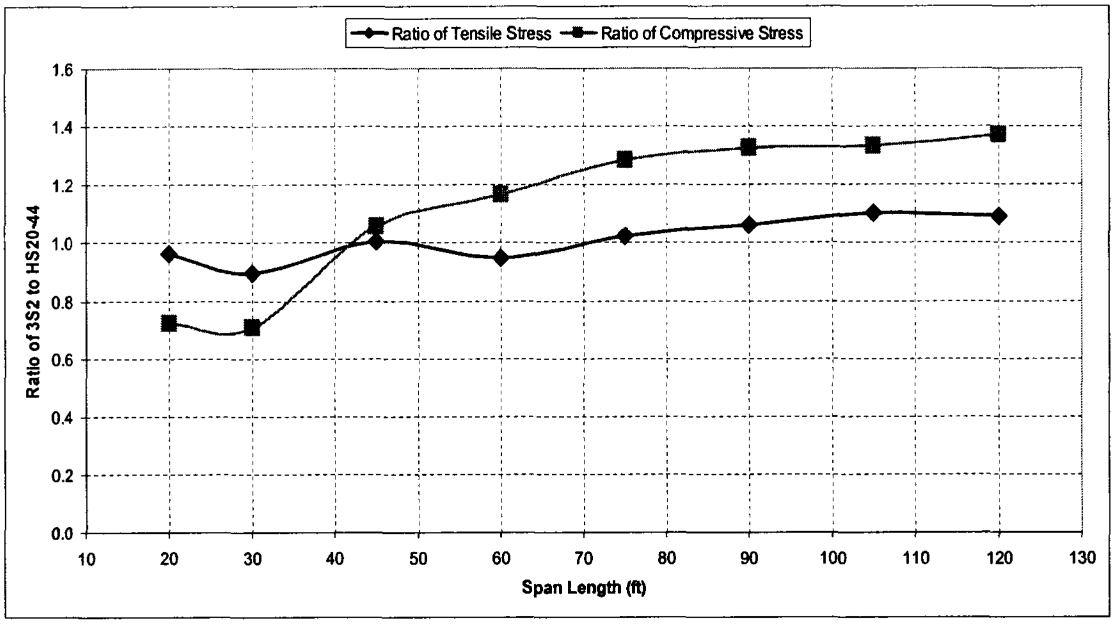


Fig. 5.5 Long Term Effects on Transverse Stress at Bottom Surface of Continuous Bridge Decks

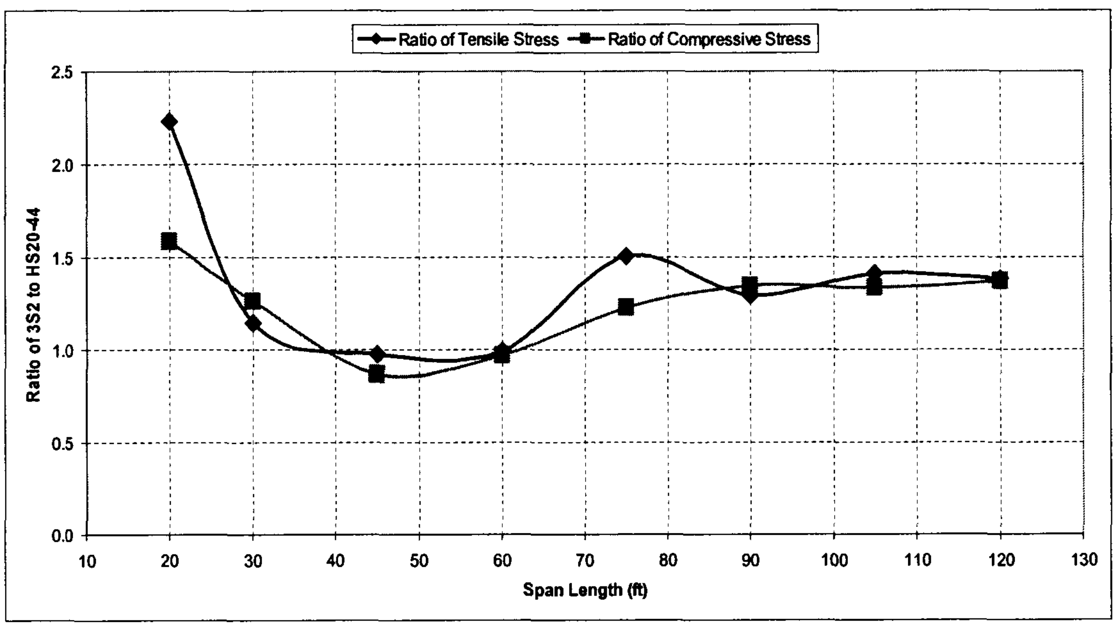


Fig. 5.6 Long Term Effects on Shear Stress at Bottom Surface of Continuous Bridge Decks

5.3 Evaluation of Bridge Decks under FHWA 3S3 Truck Load

This subtask focused on the strength and serviceability of bridge decks under the impact of the FHWA 3S3 truck load. The evaluation considered composite and non-composite bridge systems. Similar FEM analysis used in the previous chapter was also employed here for a typical deck and girder system to determine the effects of the trucks on the stresses in the transverse and longitudinal directions. In this part, both of simple span and continuous span bridges were evaluated.

5.3.1 Simply Supported Bridge Decks

All bridges considered for this study had concrete decks. The finite element models for typical bridge decks were generated with a typical 30-ft. bridge deck width and eight-inch thickness supported by five AASHTO Bulb-Tee 54, or Bulb-Tee 63, or Bulb-Tee 72 girders. The girders are spaced at eight ft. in the middle and seven ft. on the outside. The span length was fixed as 90 ft. The design load for the bridges included in this study and the loads from FHWA 3S3 truck configuration were applied to the deck. As presented in AASHTO LRFD specifications, the load combination "Strength I Max" was performed for these typical bridge deck models to determine the short term effects of 3S3 truck on bridge decks, while load combination "Fatigue" was performed for these typical bridge deck models to determine the long term effects.

The effects of FHWA 3S3 truck loads on continuous bridge decks designed for HS20-44 truck loads are presented in Table 5.3 to Table 5.6. 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.

Short term effects At the top surface of the bridge deck, for bridge built with Bulb-Tee 54 girder, the ratio of maximum tensile stress varied between 1.13 and 1.41 and between 0.76 and 1.31 for the ratio of maximum compressive stress; for bridge built with Bulb-Tee 63 girder, the ratio of maximum tensile stress varied between 1.12 and 1.33 and between 0.73 and 1.27 for the ratio of maximum compressive stress; for bridge built with Bulb-Tee 72 girder, the ratio of maximum tensile stress varied between 1.12 and 1.23 and between 0.70 and 1.16 for the ratio of maximum compressive stress. Those ratios exceeded 1.1, which means the deck was in an overstressed state and may experience cracks in all three directions; even with the larger girder sections, the ratio of maximum values become smaller.

Table 5.3 Short term Effects of 3S3 Truck Loads on Top Surface of Simple Span Bridge Decks

Girder Type	Ratio of Max Value of Stress of FHWA 3S3 to HS20-44					
	Max Tensile Stress			Max Compressive Stress		
	Longitudinal	*Transverse*	*Shear*	*Longitudinal*	*Transverse*	*Shear*
BT - 54	1.126	1.174	1.413	0.760	0.985	1.314
BT - 63	1.123	1.073	1.327	0.727	0.970	1.269
BT - 72	1.115	1.091	1.230	0.702	0.958	1.155

At the bottom surface of the bridge deck, for a bridge built with Bulb-Tee 54 girder, the ratio of maximum tensile stress varied between 0.76 and 1.31 and between 1.13 and 1.41 for the ratio of maximum compressive stress. For a bridge built with Bulb-Tee 63 girder, the ratio of maximum tensile stress varied between 0.73 and 1.27 and between 1.12 and 1.33 for the ratio of maximum compressive stress. For a bridge built with Bulb-Tee 72 girder, the ratio of maximum tensile stress varied between 0.70 and 1.16 and between 1.12 and 1.23 for the ratio of maximum compressive stress. Those ratios exceeded 1.1, which means the deck was under overstress state and may experience cracks in all three directions, even with the girder sections grow larger; the ratio of maximum values become smaller.

Table 5.4 Short term Effects of 3S3 Truck Loads on Bottom Surface of Simple Span Bridge Decks

Girder Type	Ratio of Max Value of Stress of FHWA 3S3 to HS20-44					
	Max Tensile Stress			Max Compressive Stress		
	Longitudinal	*Transverse*	*Shear*	*Longitudinal*	*Transverse*	*Shear*
BT - 54	0.760	0.985	1.314	1.126	1.174	1.413
BT - 63	0.727	0.970	1.269	1.123	1.073	1.327
BT - 72	0.702	0.958	1.155	1.115	1.091	1.230

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

Long term effects At the top surface of the bridge deck, for a bridge built with Bulb-Tee 54 girder, the ratio of maximum tensile stress varied between 0.86 and 1.14 and between 0.70 and 1.22 for the ratio of maximum compressive stress. For a bridge built with Bulb-Tee 63 girder, the ratio of maximum tensile stress varied between 0.49 and 1.22 and between 0.67 and 1.16 for the ratio of maximum compressive stress. For a bridge built with Bulb-Tee 72 girder, the ratio of maximum tensile stress varied between

0.60 and 1.25 and between 0.65 and 1.11 for the ratio of maximum compressive stress. Those ratios exceeded 1.1, which means the deck was under overstress state and may experience cracks in all three directions. Such cracks would require additional inspections along with early and frequent maintenance.

Table 5.5 Long term Effects of 3S3 Truck Loads on Top Surface of Simple Span Bridge Decks

Girder Type	Ratio of Max Value of Stress of FHWA 3S3 to HS20-44					
	Max Tensile Stress			Max Compressive Stress		
	Longitudinal	*Transverse*	*Shear*	*Longitudinal*	*Transverse*	*Shear*
BT - 54	0.858	1.194	1.139	0.704	1.030	1.217
BT - 63	0.485	1.217	1.112	0.674	1.005	1.164
BT - 72	0.602	1.253	1.090	0.652	0.985	1.107

At the bottom surface of the bridge deck, for a bridge built with Bulb-Tee 54 girder, the ratio of maximum tensile stress varied between 0.70 and 1.22 and between 0.86 and 1.14 for the ratio of maximum compressive stress. For a bridge built with Bulb-Tee 63 girder, the ratio of maximum tensile stress varied between 0.67 and 1.16 and between 0.49 and 1.22 for the ratio of maximum compressive stress. For a bridge built with Bulb-Tee 72 girder, the ratio of maximum tensile stress varied between 0.65 and 1.11 and between 0.60 and 1.25 for the ratio of maximum compressive stress. Those ratios exceeded 1.1, which means the deck was overstressed and may experience cracks in all three directions. Such cracks would require additional inspections along with early and frequent maintenance.

Table 5.6 Long term Effects of 3S3 Truck Loads on Bottom Surface of Simple Span Bridge Decks

Girder Type	Ratio of Max Value of Stress of FHWA 3S3 to HS20-44					
	Max Tensile Stress			Max Compressive Stress		
	Longitudinal	*Transverse*	*Shear*	*Longitudinal*	*Transverse*	*Shear*
BT - 54	0.704	1.030	1.217	0.858	1.194	1.139
BT - 63	0.674	1.005	1.164	0.485	1.217	1.112
BT - 72	0.652	0.985	1.107	0.602	1.253	1.090

The locations of maximum stresses due to HS20-44 or FHWA 3S3 truck loads may differ from each other. The difference is what makes the ratio of 3S3 to HS20-44 truck for some span lengths less than 1. There is no significant difference between the long term effects and the short term effects of FHWA 3S3 truck load on bridge decks. Under both situations the bridge deck may experience cracks in all three directions. 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.

5.3.2 Continuous Bridge Decks

Similar finite element models and analysis methods from chapter 5.2 were applied in this chapter, except the heavy truck load traveled on the bridge was changed into an FHWA 3S3 truck. The span lengths of the bridge models were in the range of 20 to 105 feet. Only the "Fatigue" load combination, as presented in the AASHTO LRFD specification, was performed for these typical bridge deck models. Also, the fatigue factor 0.75 and the impact factor 1.3 were used.

The effects of FHWA 3S3 truck loads on continuous bridge decks designed for HS20-44 truck loads were presented in Tables 5.7 and 5.8 and. 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.

At the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction varied between 0.95 and 1.78 and between 0.73 and 1.45 in the transverse direction. The ratio of shear stress varied between 0.90 and 1.46. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction varied between 0.71 and 1.19, and between 0.94 and 1.16 in the transverse direction; the ratio of shear stress varied between 1.04 and 1.57. The ratios of maximum shear stress were larger than 1.4 when the span length was 20 ft. Therefore, these bridge decks may experience cracks in the vertical direction. The ratios of maximum stress were mostly larger than 1.1 when the span length was longer than 30 ft. Therefore, these bridge decks may experience cracks in 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 FHWA 3S3 truck loads may differ from each other. The difference is what makes the ratio of 3S3 to HS20-44 truck for some span lengths less than 1.

Table 5.7 Long Term Effects of 3S3 Truck Loads on Top Surface of Continuous Bridge Decks

Ratio of Max Value of Stress of 3S3 to HS20-44						
Span Length	**Max Tensile Stress**			**Max Compressive Stress**		
(ft.)	***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

At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction varied between 0.73 and 1.45 and between 0.95 and 1.78 in the transverse direction. The ratio of shear stress varied between 1.04 and 1.57. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction varied between 0.94 and 1.16, and between 0.71 and 1.19 in the transverse direction; the ratio of shear stress varied between 0.90 and 1.46. The ratio of maximum shear stress was larger than 1.4 when the span length was 20 ft. Therefore, these bridge decks may experience cracks in the vertical direction. When the span length is longer than 30 ft., the ratios of maximum stress were mostly larger than 1.1, which means the deck was in an overstressed state and may experience cracks in 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 3S3 truck loads may differ from each other. The difference is what makes the ratio of 3S3 to HS20-44 truck for some span lengths less than 1.

Table 5.8 Long Term Effects of 3S3 Truck Loads on Bottom Surface of Continuous Bridge Decks

Ratio of Max Value of Stress of 3S3 to HS20-44						
Span Length	**Max Tensile Stress**			**Max Compressive Stress**		
(ft.)	***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

5.4 Summary

The bridge deck performances were evaluated in this chapter. Short term and long term effects of FHWA 3S3 truck load on simple span bridges were determined in section 5.3, while long term effects of FHWA 3S2 and 3S3 truck load on continuous span bridges were determined in sections 5.2 and 5.3. As the truck load increased, the short term or long term effects of heavy truck load on bridge decks cannot be neglected. In most cases the bridge decks are overstressed when a FHWA 3S2 or 3S3 truck traveled on them. The deck may experience cracks in longitudinal, transverse, and vertical directions. Such cracks would require additional inspections along with early and frequent maintenance.

CHAPTER VI

STATISTICAL ANALYSIS OF SIMPLE SPAN BRIDGE DECK DATA

6.1 Introduction

In the slab-on-girder bridge system, the reinforced concrete deck is one of the most important elements in distributing the service load into the longitudinal and transverse directions. Any deterioration of the deck may cause weakening or even failure in other elements, for instance, girders or diaphragms. On the other hand, the deck also plays an important role on the bridge serviceability condition. The maintenance and/or rehabilitation of the deck have a significant percentage of the bridge life cycle cost.

Two main problems induced by mechanical loading on bridge decks are overstressing and fatigue. Based on research works of Fang et al. (1990) and Petrou et al. (1994), it has been determined that the overstressing and fatigue of the bridge decks are independent phenomena. Thus, these two deterioration modes will be dealt with separately in this study.

To analyze the bridge deck stress state and strain state accurately is a complex work. Modern technology provides researchers and engineers some effective tools; the finite element method is one of the most widely used techniques. Using the finite element method, it is easily to obtain the stress and displacement in longitudinal, transverse and shear directions at each joint. Along with obtaing accurate results, researchers encounter another difficulty: a large amount of result data. It takes plentiful time and energy to find the useful information from the results, such as the extreme value of the stresses and the stress distribution among the deck surface. In this situation, the researchers and engineers should apply the statistical method to the data. Analyzing the work will give the researchers and engineers more efficiency.

The objective of this research is to develop a statistical experiment to evaluate the stress behavior of the simple span bridge deck, including the stress distribution of the bridge deck at the top and bottom surfaces in longitudinal, transverse, and shear directions; and to find the interaction between bridge deck stress behavior and other parameters, such as bridge support condition, the girders type/number, and other secondary load path elements.

6.2 Design Variables of Experiments

In this study, the following parameters were considered in the analysis procedure:

1) Bridge girder type;
2) Bridge girder number;
3) Span length;
4) The sample joints selection;
5) Truck load type.

Also, these parameters were considered as the independent variables in the statistic experiment, and the bridge deck stress behavior was considered as the response. SAS will be used to perform the statistical experiment, while GTSTRUDL will be used for the finite element analysis.

Since there are several parameters being considered for the analysis, and each parameter would influence the deck stress behavior. The statistical experiment was designed and analyzed as the factorial experiment with several crossed treatment factors. For instance, the standard model for three treatment factors is

$$Y_{ijkt} = \mu + \tau_{ijk} + \varepsilon_{ijkt},$$

$$\tau_{ijk} = \alpha_i + \beta_j + \gamma_k + (\alpha\beta)_{ij} + (\alpha\gamma)_{ik} + (\beta\gamma)_{jk} + (\alpha\beta\gamma)_{ijk}, \tag{6.1}$$

$t = 1,\dots,r_{ijk}$; $i = 1,\dots,a$; $j = 1,\dots,b$ and $k = 1,\dots,c$.

$\varepsilon_{ijkt} \sim N(0,\sigma^2)$, ε_{ijkt}'s mutually independent.

Where $\mu + \tau_{ijk}$ denotes the true mean response for the treatments; $\alpha_i, \beta_j, \gamma_k$ are the effects (positive or negative) on the response of factor A, B, C at levels i, j, k, respectively; $(\alpha\beta)_{ij}, (\alpha\gamma)_{ik}, (\beta\gamma)_{jk}$ are the additional effects of the pairs of factors together at the specified levels; and $(\alpha\beta\gamma)_{ijk}$ is the additional effect of all three factors together at levels i, j, k. The single variable ε_{ijkt} is called an error variable, where "$\sim N(0,\sigma^2)$" denotes that it has a normal distribution with mean 0 and variance σ^2.

For a factorial experiment with several crossed treatment factors, there are several different models that may be appropriate for analyzing, depending on which interactions are believed to be negligible. The investigation of the contributions that each of the factors make individually to the response were obtained. Since this research is based on the finite element method but not the real experiment, there are no observation error terms; in order to perform the analysis, some insignificant interactions would be neglected from the analysis and used as the error terms. Since these kinds of error terms do not contain the true error, we cannot say that the error follows the normal distribution with the mean 0 and appropriate variance; correspondingly, the confidence intervals of the observation cannot be obtained. The hypothesis tests can not be performed, either. To solve this difficulty, the ratio of variance of the independent variables and some interaction terms were obtained to show the importance.

6.3 Simply Supported Bridge Decks Analysis

In this chapter, decks of the simply supported bridges were analyzed via statistical methods. The statistical model was developed, then the Analyses of Variance (ANOVA) were performed, and corresponding charts were generated to evaluate the deck behavior under different bridge configurations.

6.3.1 Statistic Model Setup

For the simply supported bridges, the analyses parameters need to be considered are identified as follow:

(1) Bridge span length was fixed as 90 feet.

(2) Bridge deck thickness was fixed as eight inches.

(3) There were six types of girders considered in the analysis: AASHTO Type IV, V, VI; AASHTO Bulb-Tee 54, 63 and 72.

(4) There were two kinds of bridge models: group one included five girders, group two included seven girders, and both of them had a fixed bridge width of 30 feet. The girders were simply supported and spaced at eight feet in the middle and seven feet on the outside for the first group, while the girders were simply supported and spaced at 5/4.5 feet in the middle and six feet on the outside for the second group. The details of the models are shown in Fig. 3.1 and Fig. 3.2.

(5) Two kinds of truck loads were included: HS20-44 and FHWA 3S3 with GVW 120,000 kips, the HS20-44 truck was the original design truck load, and the FHWA 3S3 truck was used as the heavy truck load to determine the deck stress behaviors. All truck loads were placed on the bridge as shown in Fig. 3.4 and Fig. 3.6.

(6) The model considered in this study was non-skewed with 0° full depth diaphragms at the end of the bridge.

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.

After the analyses parameters were determined, the finite element model could be obtained. GTSTRUDL was used to perform the analysis; the results of deck stress states could be obtained. Following Table 6.1 lists the model details in the study.

Table 6.1 Simple Span Bridge Models and Their Specifications Used in STAT Study

Bridge Model	Girder Type	# of Girders	Span Length	# of Spans	Support Condition	Applied Truck Load
316	AASHTO Type IV	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
326	AASHTO Type V	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
336	AASHTO Type VI	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
346	AASHTO BT-54	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
356	AASHTO BT-63	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
366	AASHTO BT-72	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
376	AASHTO Type IV	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
386	AASHTO Type V	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
396	AASHTO Type VI	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
406	AASHTO BT-54	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
416	AASHTO BT-63	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3
426	AASHTO BT-72	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3

The statistical model could be determined when the stress state were obtained from finite element analysis. For the purpose of reducing the data amount, the deck was first meshed into several rectangle areas, and every rectangle area had the same length of 30 feet and the same width of 10 feet, with the area of the rectangle 300 ft^2. A typical meshed deck was showed in following Fig. 6.1. The maximum stress state in each rectangle would be identified as the representative stress state and used in the statistical analysis.

In Chapter V the researcher determined that the bridge deck was in a stable stress state. Results from FEM indicated that the difference of stresses at the top and bottom surfaces was only the sign of the stress values (positive or negative numbers), thus following research was based on the results of stresses at bottom surface of the bridge deck.

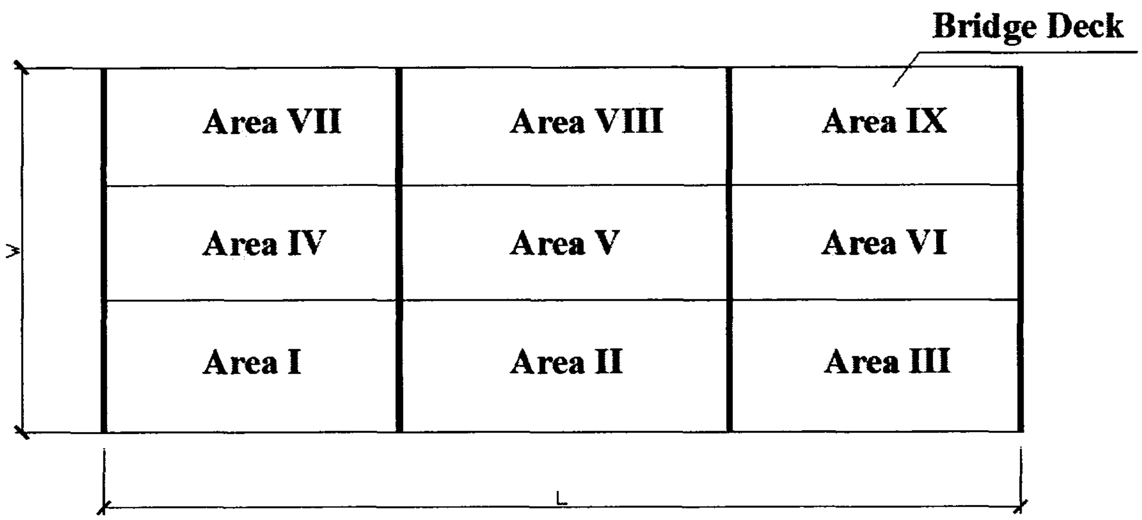


Fig. 6.1 Typical Meshed Bridge Deck for Statistical Analysis

Three treatment factors were used to set up the statistic model: girder type, girder number, and truck load type. The treatment factors and their details were listed in Table 6.2.

Table 6.2 Treatment Factors and Corresponding Observation Levels

Treatment Factors	Abbreviation	Observation Level	Details of Observation Level
Girder Type	GT	6	AASHTO Type IV, V, VI; BT-54, 63, 72
Girder Number	GN	2	5 Girders/7 Girders
Truck Load Type	TT	2	FHWA 3S3/HS20-44

There were four treatment interactions between each of the main factors or three together. The total statistical model without error terms was defined as follows

$$Y_{ijk} = \mu + (GT)_i + (GN)_j + (TT)_k + (GT \times GN)_{ij}$$
$$+ (GT \times TT)_{ik} + (GN \times TT)_{jk} + (GT \times GN \times TT)_{ijk} \quad (6.2)$$

$i = 1,...,6$; $j = 1,2$ and $k = 1,2$.

As mentioned before, the less important treatment interactions or the interactions we did not care about would be used as the error term in order to establish the proper statistical model. In formula 6.2, the component $(GT \times GN \times TT)$ is used as the initial error term due to the three effects of interaction normally thought less important than other components. The initial analysis of variance table is shown as Table 6.3, and the modified statistical model with error terms is listed:

$$Y_{ijkt} = \mu + (GT)_i + (GN)_j + (TT)_k + (GT \times GN)_{ij}$$
$$+ (GT \times TT)_{ik} + (GN \times TT)_{jk} + \varepsilon_{ijkt} \quad (6.3)$$

$t = 1$; $i = 1,...,6$; $j = 1,2$ and $k = 1,2$.

Table 6.3 ANOVA, Single Rectangle Area

Source of Variation	Degree of Freedom	Sum of Squares	Mean Square	Ratio
GT	5	*ss(GT)*	*ss(GT)/5*	*ms(GT)/msE*
GN	1	*ss(GN)*	*ss(GN)/1*	*ms(GN)/msE*
TT	1	*ss(TT)*	*ss(TT)/1*	*ms(TT)/msE*
GTxGN	5	*ss(GTxGN)*	*ss(GTxGN)/5*	*ms(GTxGN)/msE*
GTxTT	5	*ss(GTxTT)*	*ss(GTxTT)/5*	*ms(GTxTT)/msE*
GNxTT	1	*ss(GNxTT)*	*ss(GNxTT)/1*	*ms(GNxTT)/msE*
Error (GTxGNxTT)	5	*ssE*	*ssE/5*	
Total	23	*sstot*		

The results obtained from finite element analysis contained three stress components, Sxx, Sxy, and Syy. Those stresses needed to be evaluated separately. The corresponding ANOVA tables and figures were generated to determine the effects of each independent factor and/or combination. The ANOVA tables were obtained from the SAS files that only had the data for a single rectangle area, while the figures were obtained from a SAS file contained all Sxx or Sxy or Syy data to save the total work load.

6.3.2 Analysis of Variables

SAS was used to perform the statistic analysis in this study; typical SAS codes were listed in appendix C. Stress components Sxx, Sxy, and Syy were evaluated separately. For each kind of stress components, the evaluations of two factor groups were applied. In first group factors GT and GN were included to investigate the deck stress performance under different factor combinations; while in the second group treatment factors GT and TT were used.

In detailed SAS input files, the analysis procedures to draw the figures of relationship between average stresses and GT and GN (or TT) could be described as follows: 1) data were inputted by the sequence of GT (six observation levels), GN (two observation levels), TT (two observation levels), and AR (nine observation levels, which were nine divided areas of deck); 2) the average stresses of each GT and GN (or GT and TT) combination of each area were computed, and corresponding figures were generated. The SAS input files used to draw the figures were different from files which used to generate the ANOVA tables since in these files the divided area AR was added to the input and the selected error term was not $(GT \times GN \times TT)$ but some others. This result would create an ANOVA table different from Table 6.3. But the figures still stay the same because to derive the figures, the researcher did not use the results from ANOVA. The researcher used this method only simply for the purpose of saving work load. The ANOVA results were obtained from those rectangular divided areas separately.

The effects of GT and GN combinations on bridge deck stress component Sxx were presented in Fig. 6.2 to Fig. 6.10. The GT values one through six represented girder type AASHTO Bulb-Tee 54, Bulb-Tee 63, Bulb-Tee 72, Type IV, Type V and Type VI, respectively. The GN values one and two represented bridge models containing five and seven girders, respectively. The standard was whether the absolute stress value was close to zero. From the figures it is easy to determine that when the bridge models contained five or seven girders, then which type of girder the bridge was built with would give us the minimum Sxx value in the bridge deck.

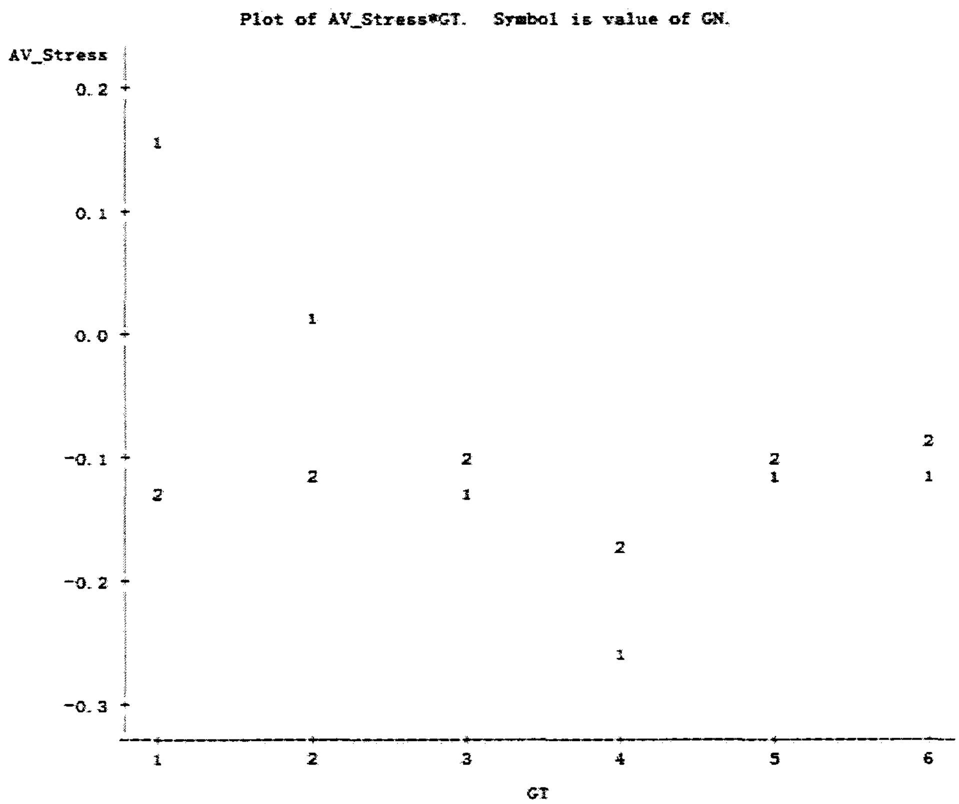


Fig. 6.2 The Effects of Treatment Factors GT and GN on Bridge Deck – Area I, Stress Component Sxx

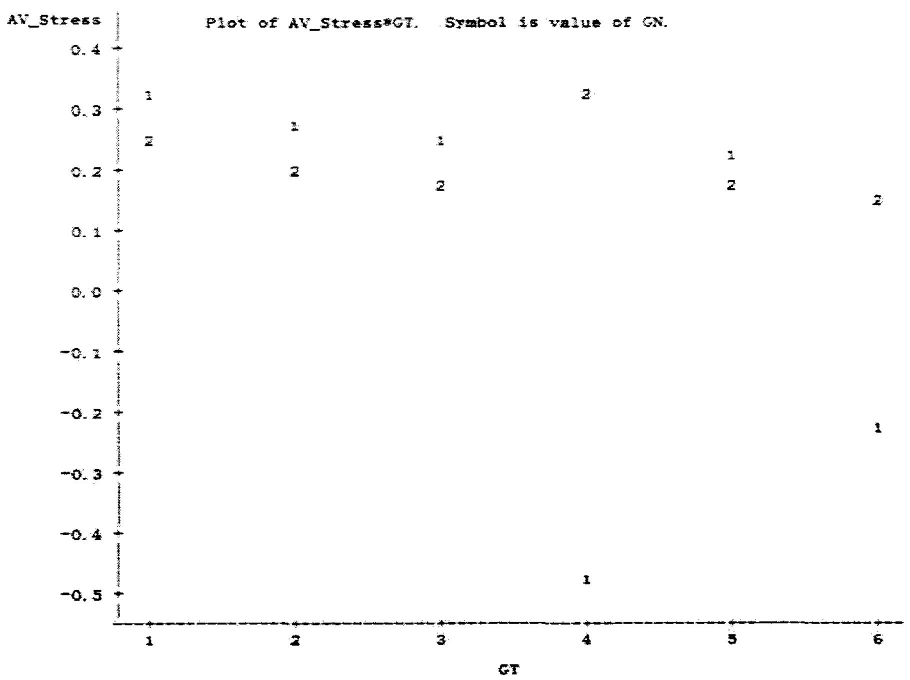


Fig. 6.3 The Effects of Treatment Factors GT and GN on Bridge Deck – Area II, Stress Component Sxx

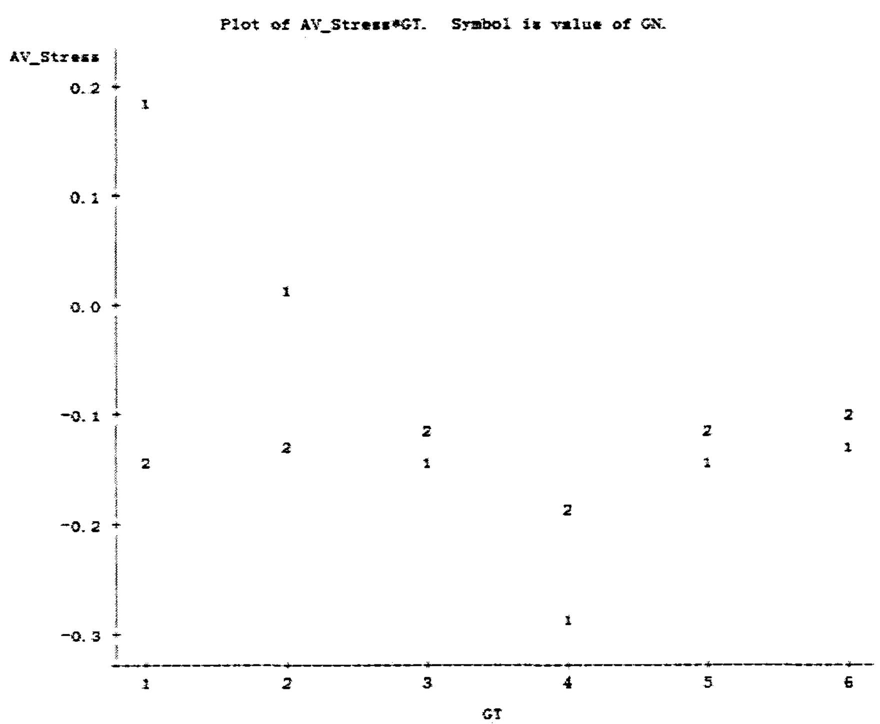


Fig. 6.4 The Effects of Treatment Factors GT and GN on Bridge Deck – Area III, Stress Component Sxx

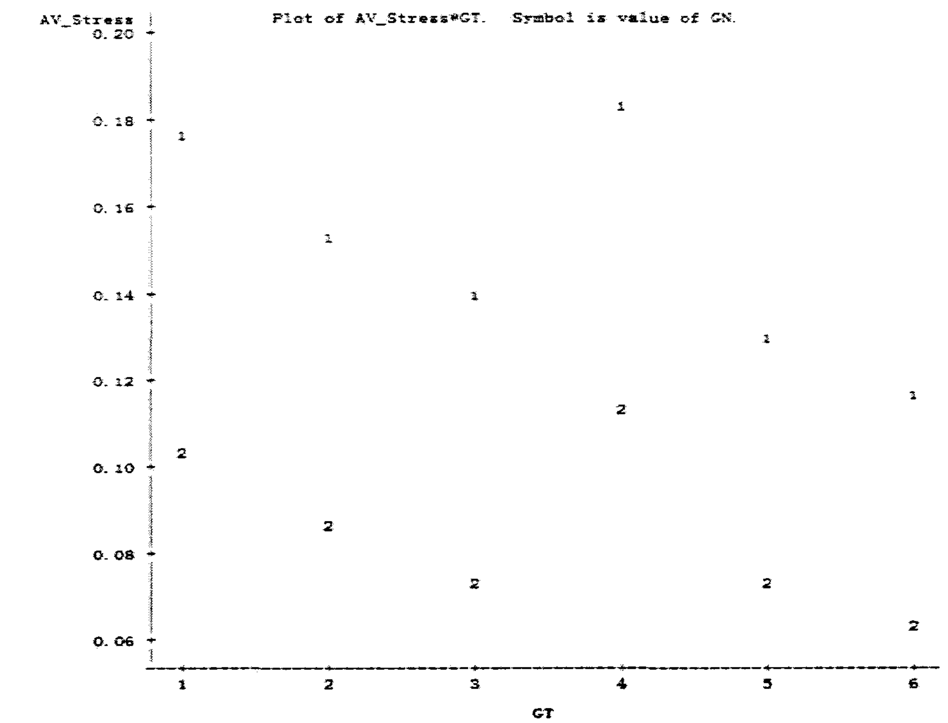


Fig. 6.5 The Effects of Treatment Factors GT and GN on Bridge Deck – Area IV, Stress Component Sxx

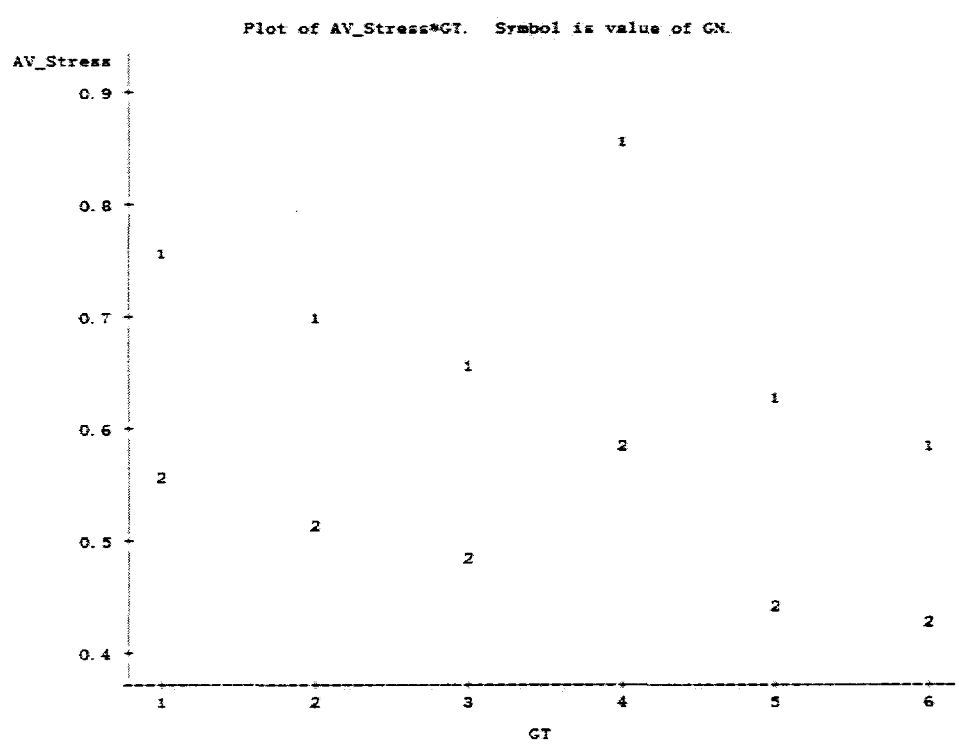


Fig. 6.6 The Effects of Treatment Factors GT and GN on Bridge Deck – Area V, Stress Component Sxx

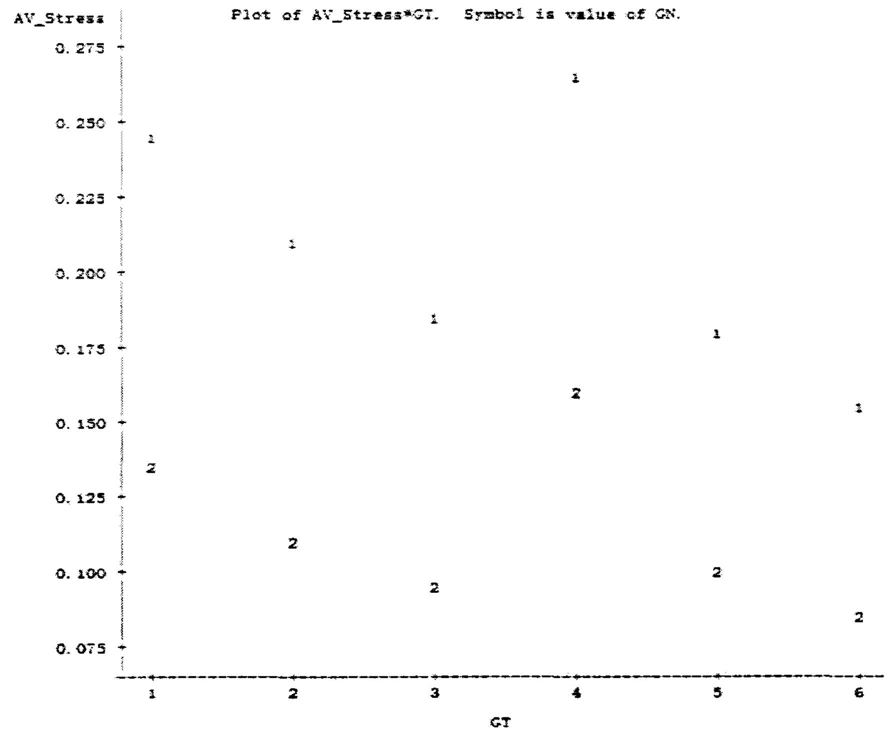


Fig. 6.7 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VI, Stress Component Sxx

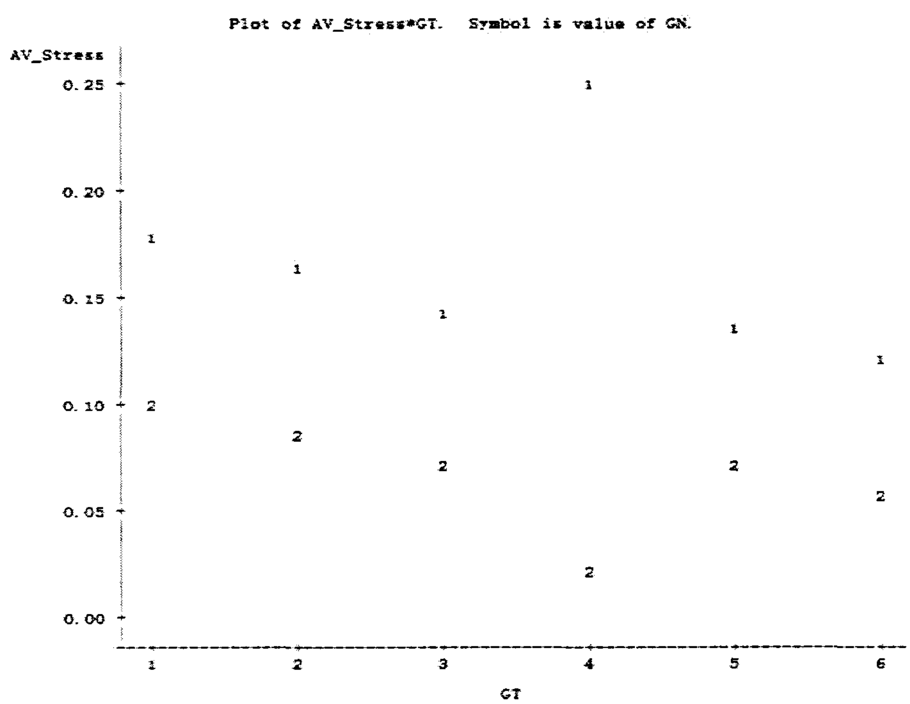


Fig. 6.8 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VII, Stress Component Sxx

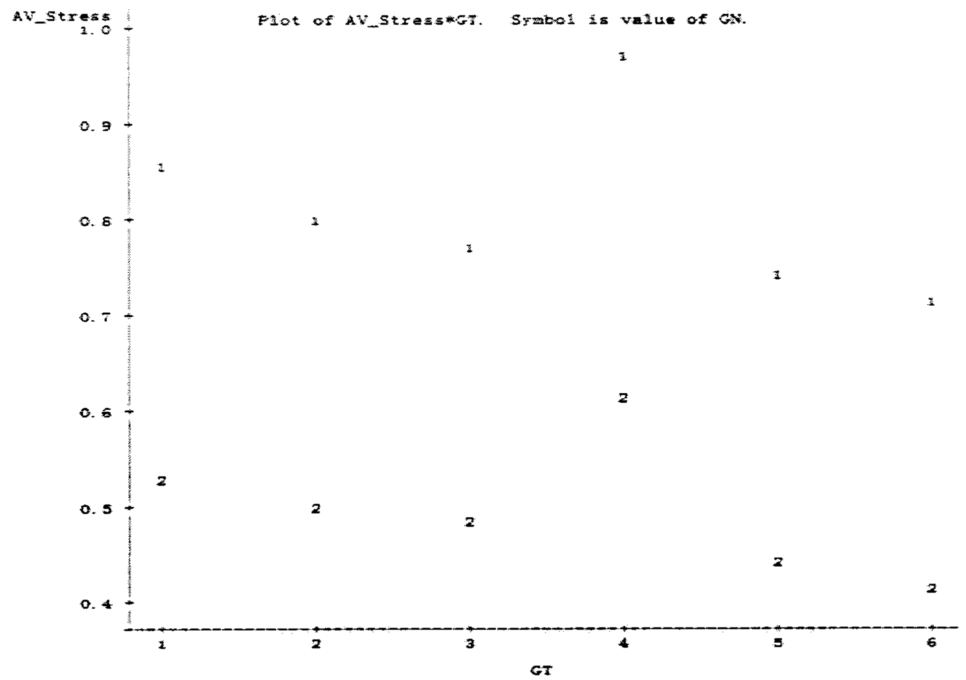


Fig. 6.9 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VIII, Stress Component Sxx

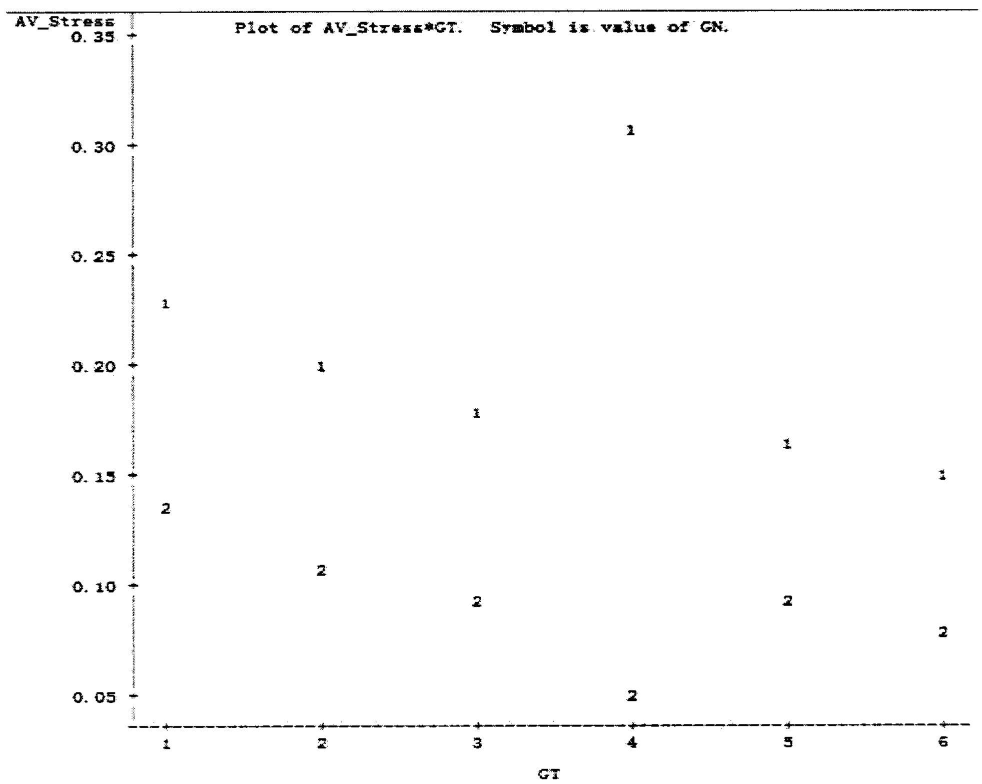


Fig. 6.10 The Effects of Treatment Factors GT and GN on Bridge Deck – Area IX, Stress Component Sxx

Comparisons were made for each divided area. Detailed results were summarized in Tables 6.4 and 6.5. In the tables the symbol ">" represented "better"; for instance, "BT-63 > Type VI" meant the absolute deck stress value of a bridge built with AASHTO Bulb-Tee 63 girder was smaller than that of a bridge built with AASHTO Type VI girder. For bridge models containing five girders, in most areas the bridge built with type VI girder had the best performance, which indicated that when AASHTO Type VI girder was used to construct the bridge, normally the bridge deck stress Sxx could be minimized. The differences of stress performances of a bridge built with AASHTO Type V and Bulb-Tee 72 were limited. And usually for a bridge built with Type IV girder, the situation was remarkably worse than other cases. In most areas, the sequence of deck stress Sxx performances was bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 and Type IV girders, from better to worse, respectively.

Table 6.4 Comparison Results – Treatment Factor GT and GN, Girder Spacing Eight ft, Deck Stress Component Sxx

Area	Comparison Results
Area I	BT-63 > Type VI > Type V > BT-72 > BT-54 > Type IV
Area II	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area III	BT-63 > Type VI > Type V > BT-72 > BT-54 > Type IV
Area IV	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area V	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VI	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VII	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VIII	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IX	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV

In Table 6.5 the comparisons were made for those bridge models which contained seven girders with girder spacing five ft. The results indicated that for most areas, the bridges built with AASHTO Type VI girder still had the best performance. The differences of stress performances of bridge built with AASHTO Type V and Bulb-Tee

72 were also limited. In one area type IV girder had the smallest value. But generally the sequence of deck stress Sxx performances involved bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 and Type IV girders, from better to worse, respectively.

Table 6.5 Comparison Results – Treatment Factor GT and GN, Girder Spacing Five ft, Deck Stress Component Sxx

Area	Comparison Results
Area I	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area II	Type VI > BT-72 > Type V > BT-63 > BT-54 > Type IV
Area III	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IV	Type VI > BT-72 > Type V > BT-63 > BT-54 > Type IV
Area V	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VI	Type VI > BT-72 > Type V > BT-63 > BT-54 > Type IV
Area VII	Type IV > Type VI > BT-72 > Type V > BT-63 > BT-54
Area VIII	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IX	Type IV > Type VI > BT-72 > Type V > BT-63 > BT-54

Similar analysis methods were applied to deck stress components Sxy. Fig. 6.11 to Fig. 6.19 show the effects of GT and GN combinations on bridge deck stress component Sxy. The GT values and GN values represented the same meaning as described before. The standard was still whether the absolute stress value closed to zero.

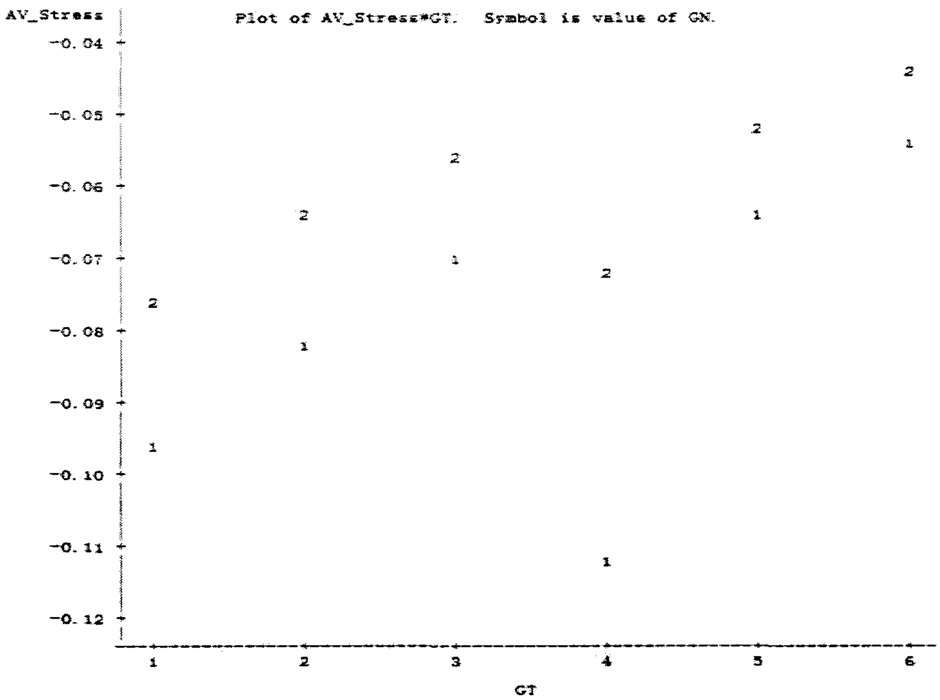


Fig. 6.11 The Effects of Treatment Factors GT and GN on Bridge Deck – Area I, Stress Component Sxy

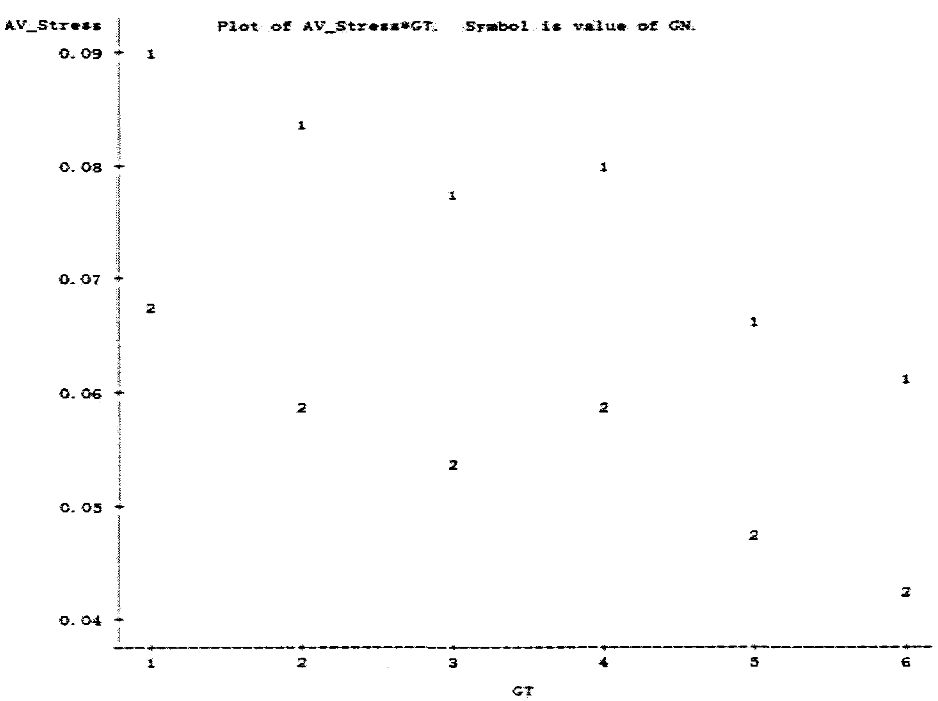


Fig. 6.12 The Effects of Treatment Factors GT and GN on Bridge Deck – Area II, Stress Component Sxy

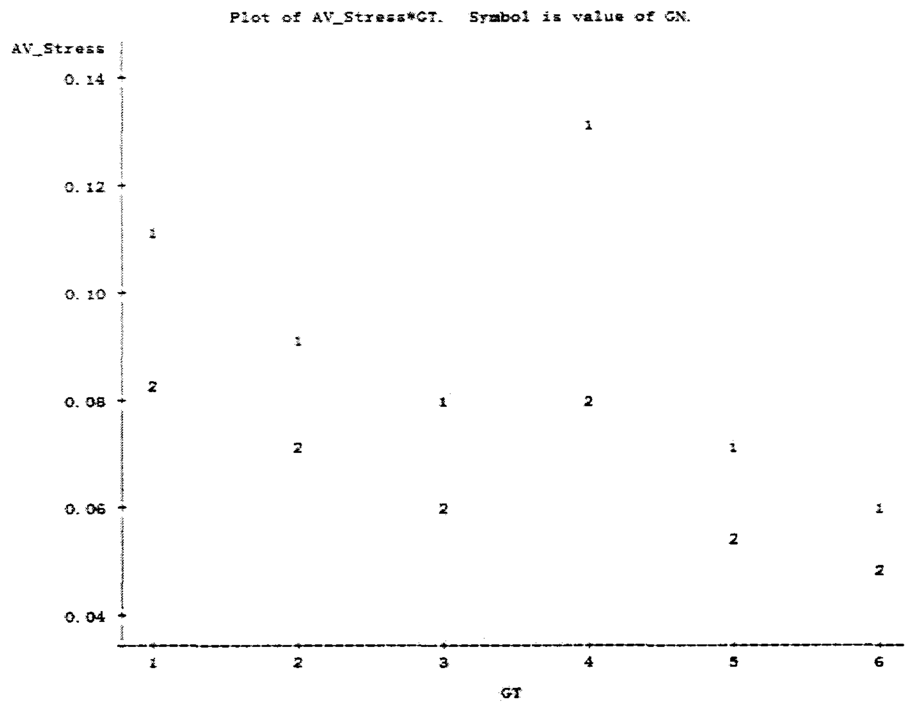


Fig. 6.13 The Effects of Treatment Factors GT and GN on Bridge Deck – Area III, Stress Component Sxy

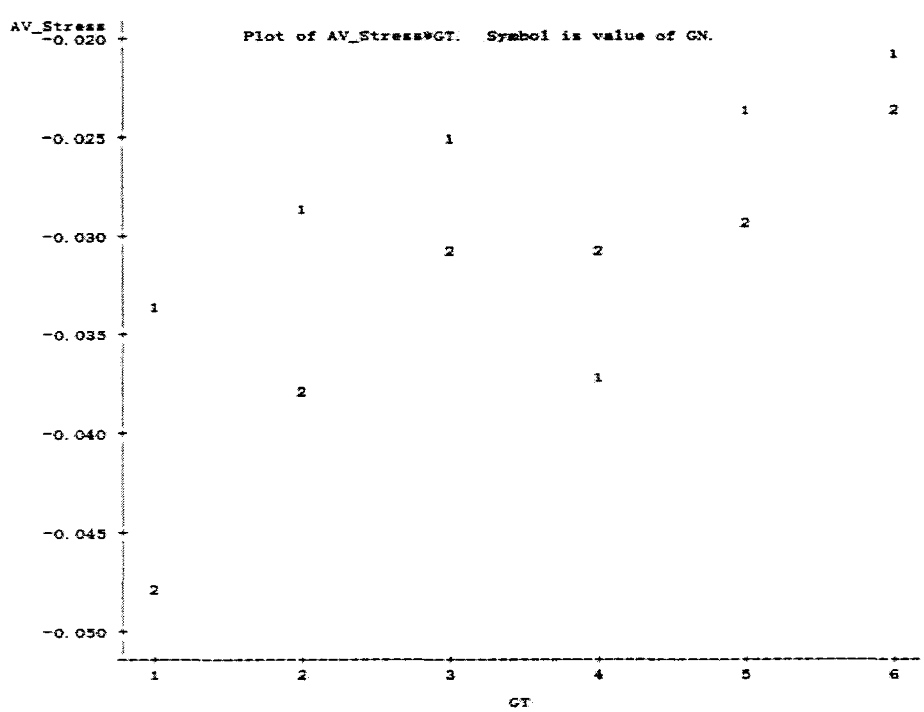


Fig. 6.14 The Effects of Treatment Factors GT and GN on Bridge Deck – Area IV, Stress Component Sxy

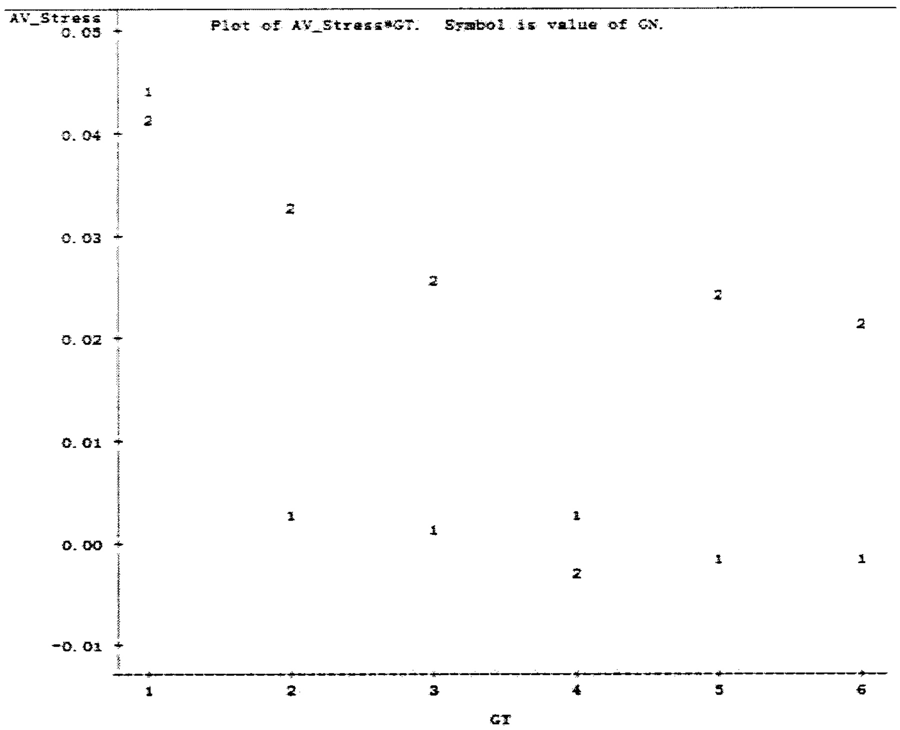


Fig. 6.15 The Effects of Treatment Factors GT and GN on Bridge Deck – Area V, Stress Component Sxy

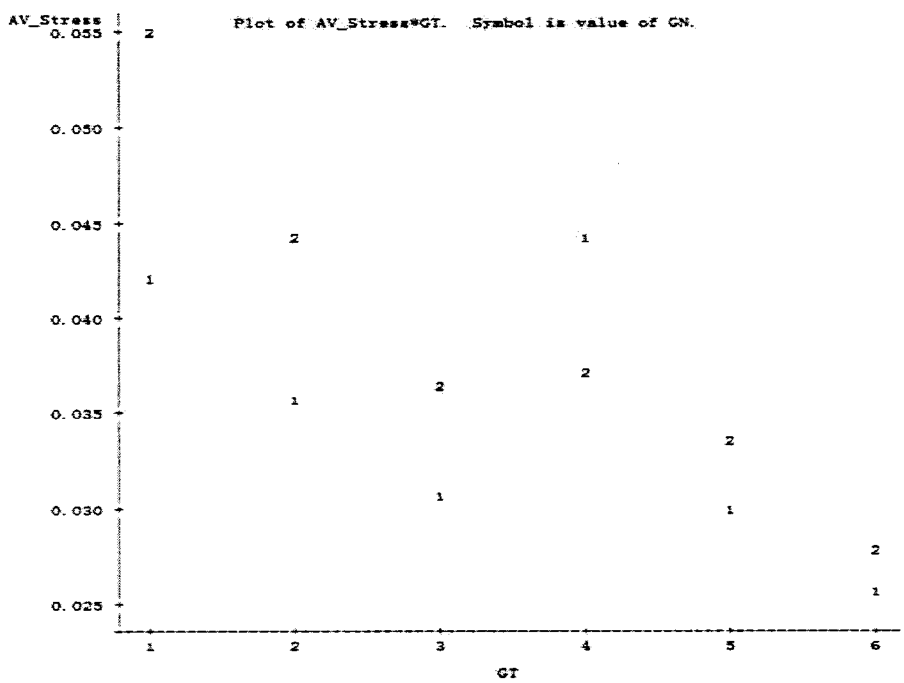


Fig. 6.16 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VI, Stress Component Sxy

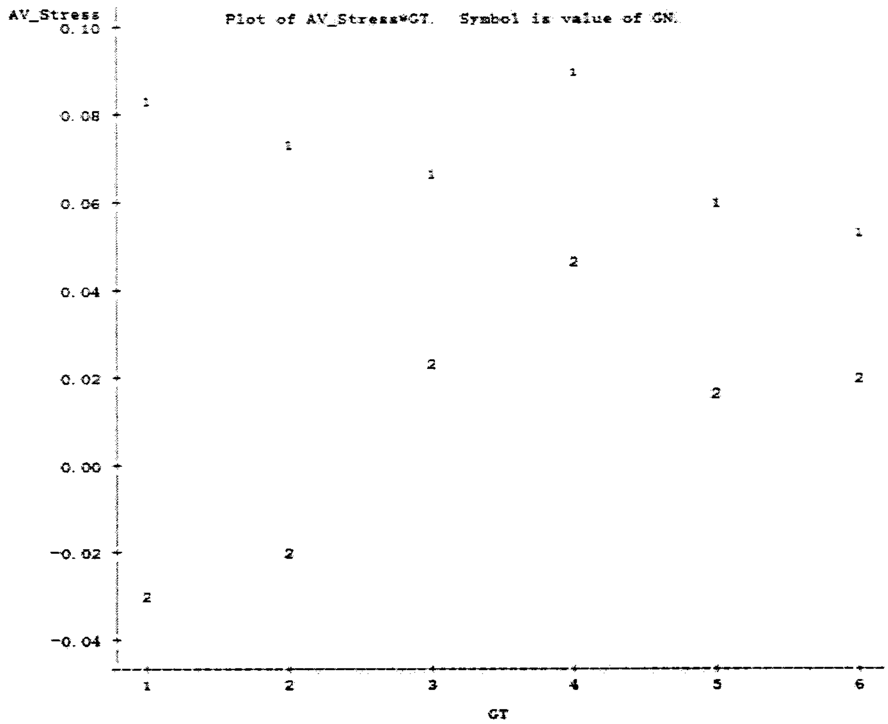


Fig. 6.17 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VII, Stress Component Sxy

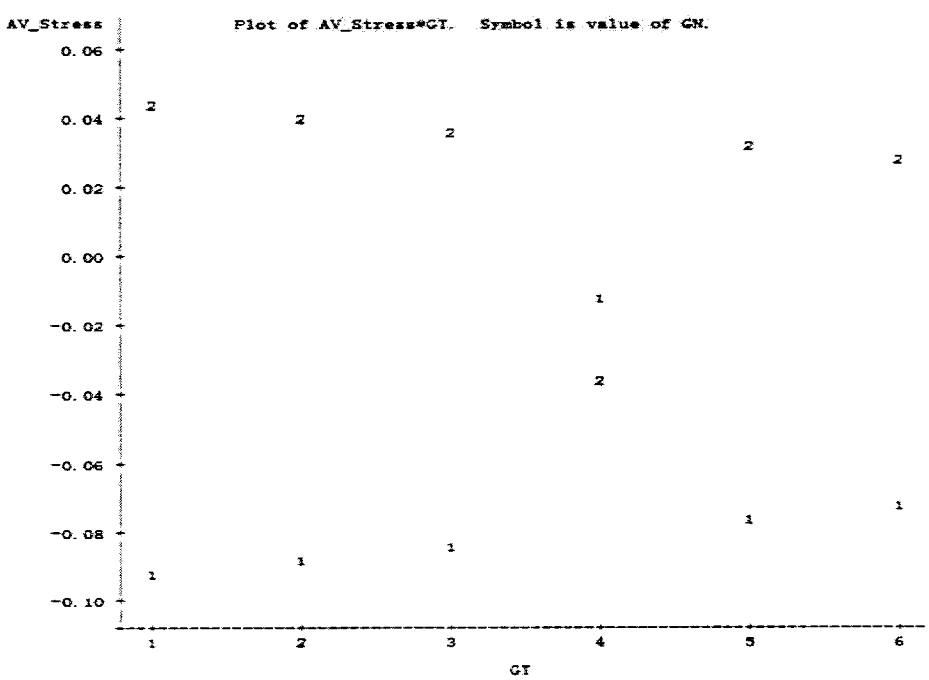


Fig. 6.18 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VIII, Stress Component Sxy

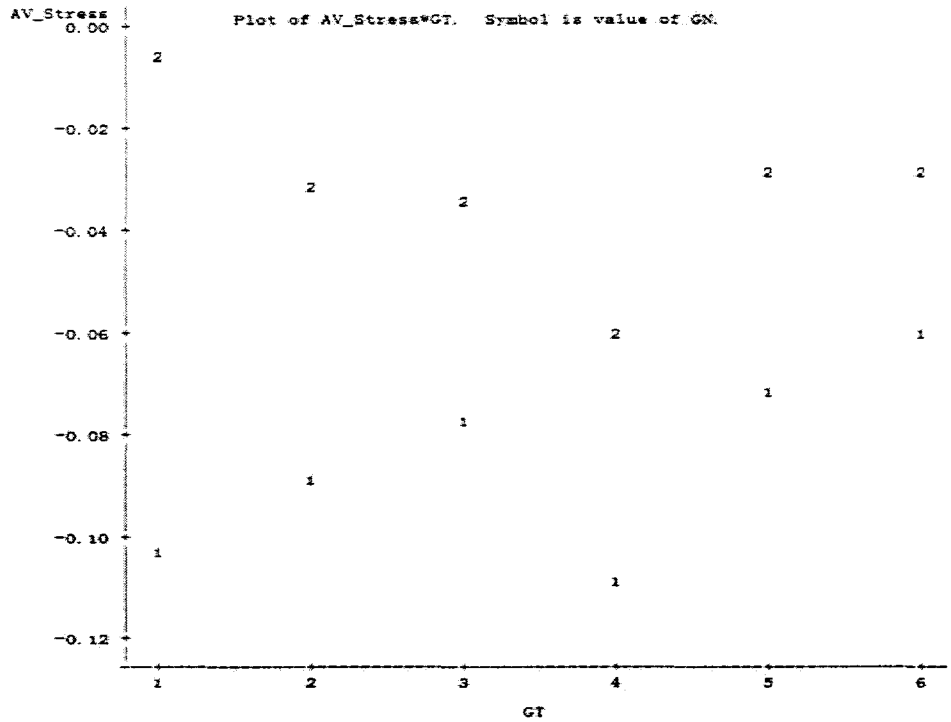


Fig. 6.19 The Effects of Treatment Factors GT and GN on Bridge Deck – Area IX, Stress Component Sxy

Comparisons were made for each divided area. Detail results were summarized in Tables 6.6 and 6.7. In the tables the symbol ">" represented the same meaning as before. For bridge models containing five girders, in most areas the bridge built with a Type VI girder had the best performance, which indicated that when the AASHTO Type VI girder was used to construct the bridge, normally the bridge deck stress Sxy could be minimized. The differences of stress performances of bridges built with AASHTO Type V and Bulb-Tee 72 were limited. And usually for bridges built with Type IV or Bulb-Tee 54 girders, the situations were worse than other cases. In most areas, the sequence of deck stress Sxy performance was bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 (or Type IV) girders, from better to worse, respectively.

Table 6.6 Comparison Results – Treatment Factor GT and GN, Girder Spacing Eight ft, Deck Stress Component Sxy

Area	Comparison Results
Area I	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area II	Type VI > Type V > BT-72 > Type IV > BT-63 > BT-54
Area III	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IV	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area V	BT-72 > Type VI > Type V > BT-63 > Type IV > BT-54
Area VI	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VII	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VIII	Type IV > Type VI > Type V > BT-72 > BT-63 > BT-54
Area IX	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV

In Table 6.7 the comparisons were made for those bridge models that contained seven girders with girder spacing five ft. The results indicated that for most areas, the bridges built with AASHTO Type VI girders still had the best performance. The differences of stress performances of bridges built with AASHTO Type V and Bulb-Tee 72 were also limited. In three areas Bulb-Tee 54 or Type IV or Type V girders had the smallest value. But generally the sequence of deck stress Sxy performance was bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Type IV and Bulb-Tee 54 girders, from better to worse, respectively.

Table 6.7 Comparison Results – Treatment Factor GT and GN, Girder Spacing Five ft, Deck Stress Component Sxy

Area	Comparison Results
Area I	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area II	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area III	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area IV	Type VI > Type V > Type IV > BT-72 > BT-63 > BT-54
Area V	Type IV > Type VI > Type V > BT-72 > BT-63 > BT-54
Area VI	Type VI > Type V > BT-72 > Type IV > BT-63 > BT-54
Area VII	Type V > Type VI > BT-63 > BT-72 > BT-54 > Type IV
Area VIII	Type VI > Type V > BT-72 > Type IV > BT-63 > BT-54
Area IX	BT-54 > Type VI > Type V > BT-63 > BT-72 > Type IV

Similar analysis methods were applied to deck stress components Syy. Fig. 6.20 to Fig. 6.28 show the effects of GT and GN combinations on bridge deck stress component Sxy. The GT values and GN values represented the same meaning as described before. The standard was still whether the absolute stress value was close to zero. In some figures the difference of stress between models built with five or seven girders was very small, thus some observations were hidden, and only one symbol was displayed in the figure.

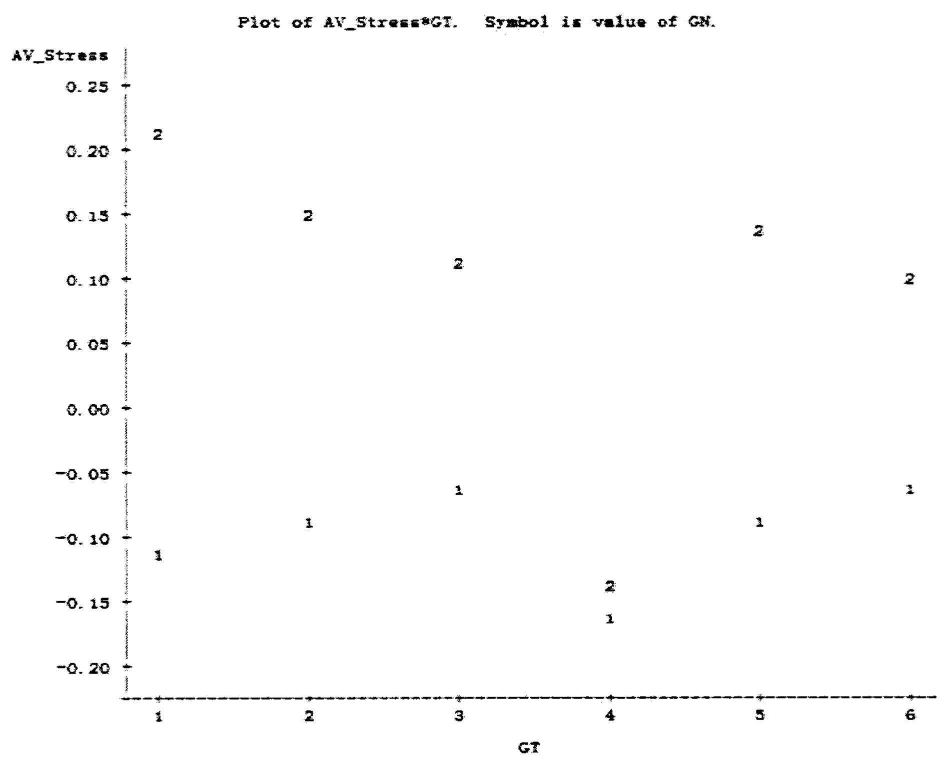


Fig. 6.20 The Effects of Treatment Factors GT and GN on Bridge Deck – Area I, Stress Component Syy

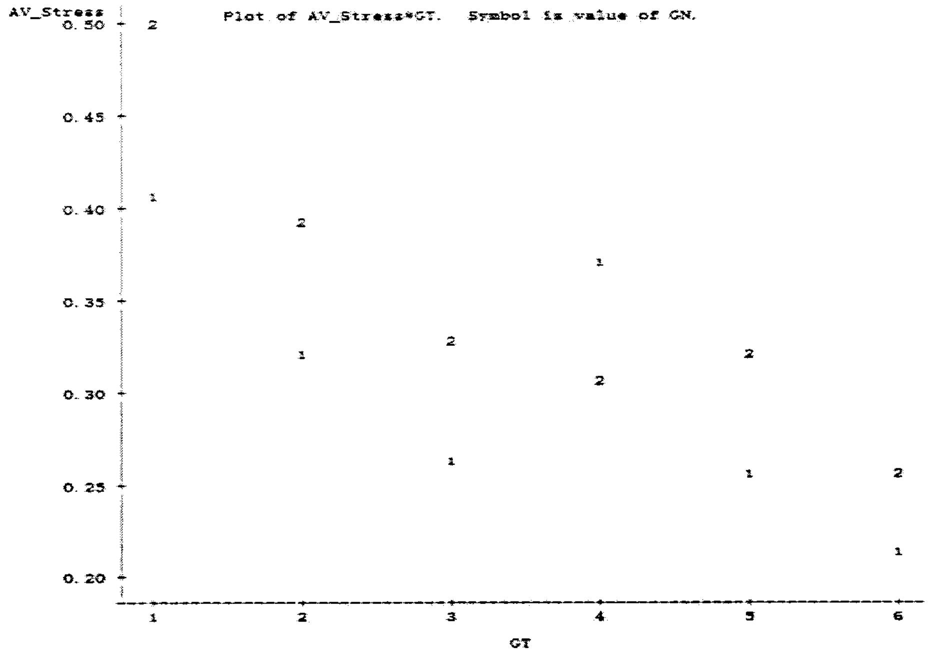


Fig. 6.21 The Effects of Treatment Factors GT and GN on Bridge Deck – Area II, Stress Component Syy

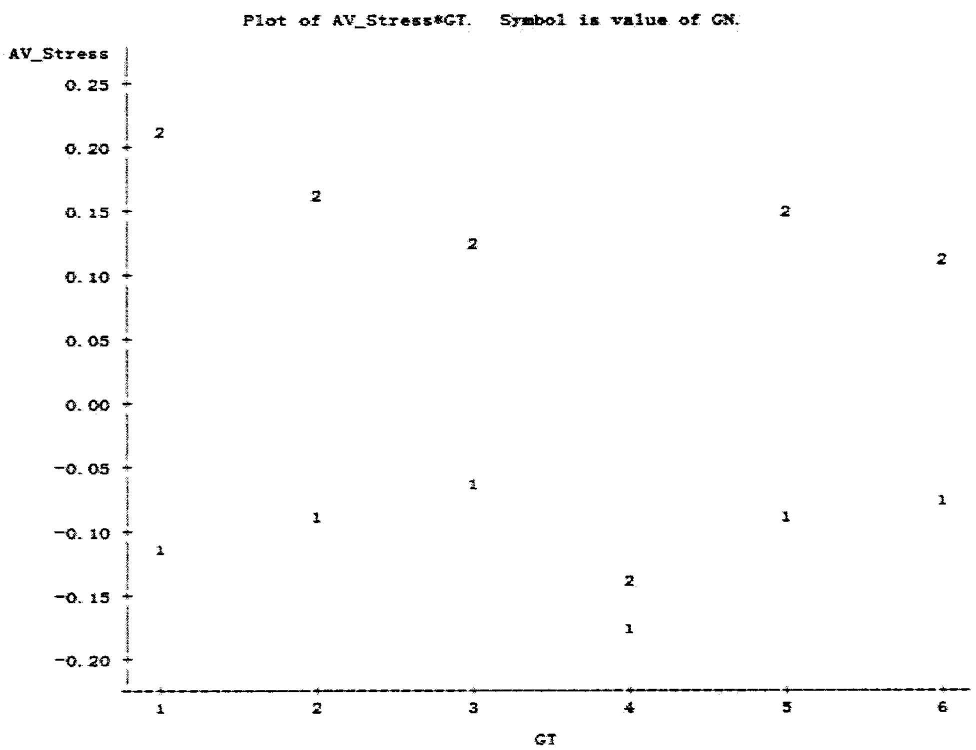


Fig. 6.22 The Effects of Treatment Factors GT and GN on Bridge Deck – Area III, Stress Component Syy

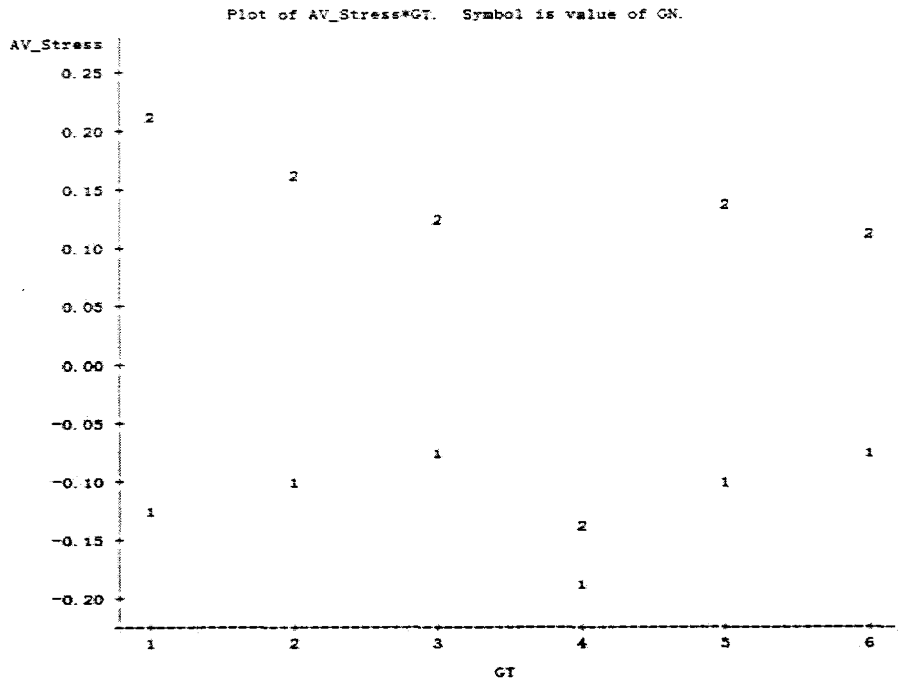


Fig. 6.23 The Effects of Treatment Factors GT and GN on Bridge Deck – Area IV, Stress Component Syy

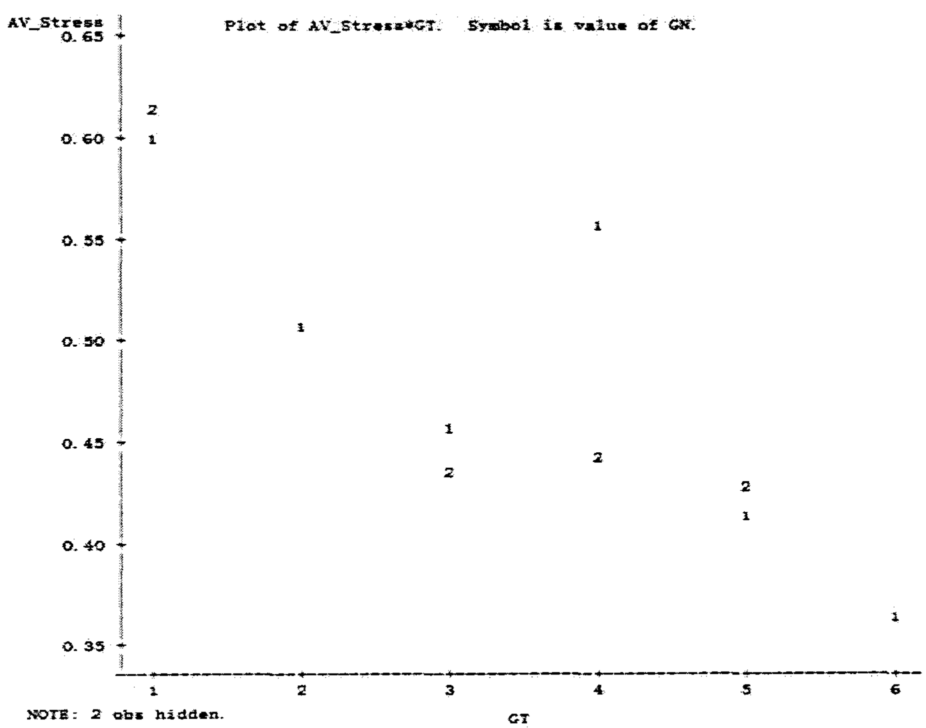


Fig. 6.24 The Effects of Treatment Factors GT and GN on Bridge Deck – Area V, Stress Component Syy

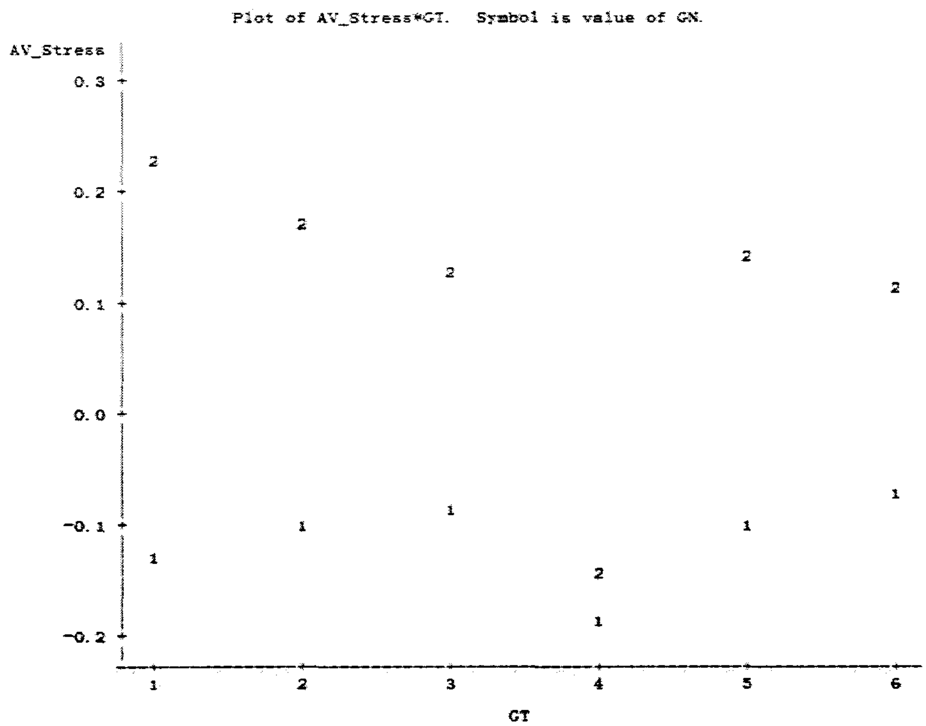


Fig. 6.25 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VI, Stress Component Syy

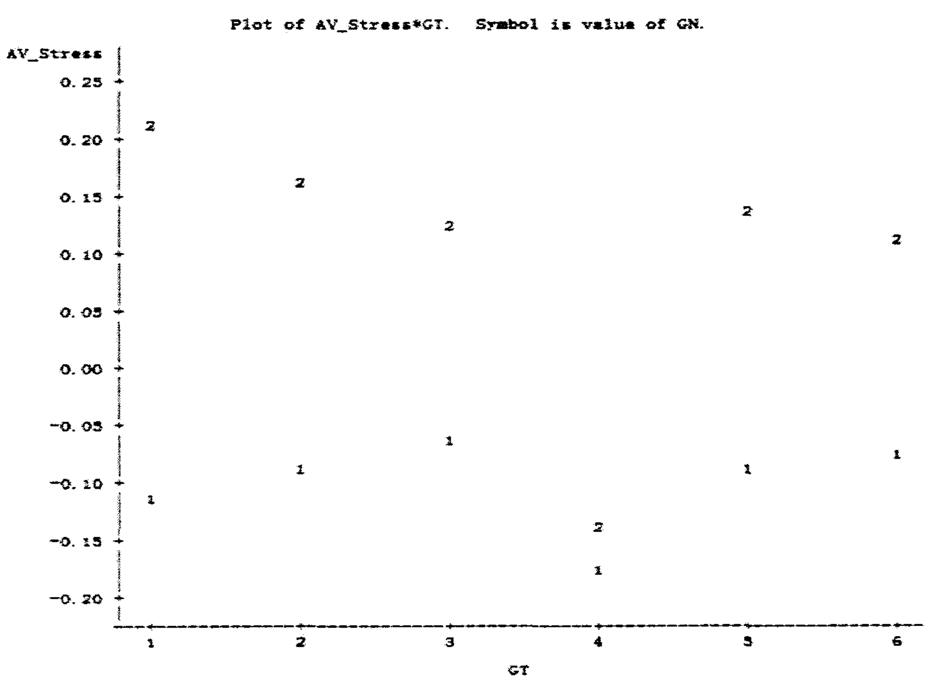


Fig. 6.26 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VII, Stress Component Syy

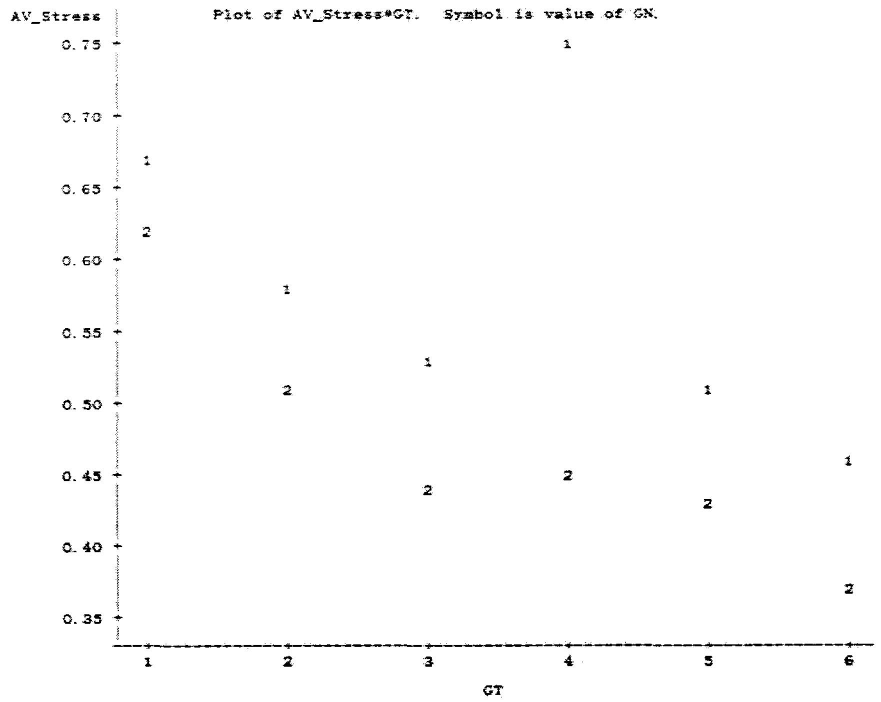


Fig. 6.27 The Effects of Treatment Factors GT and GN on Bridge Deck – Area VIII, Stress Component Syy

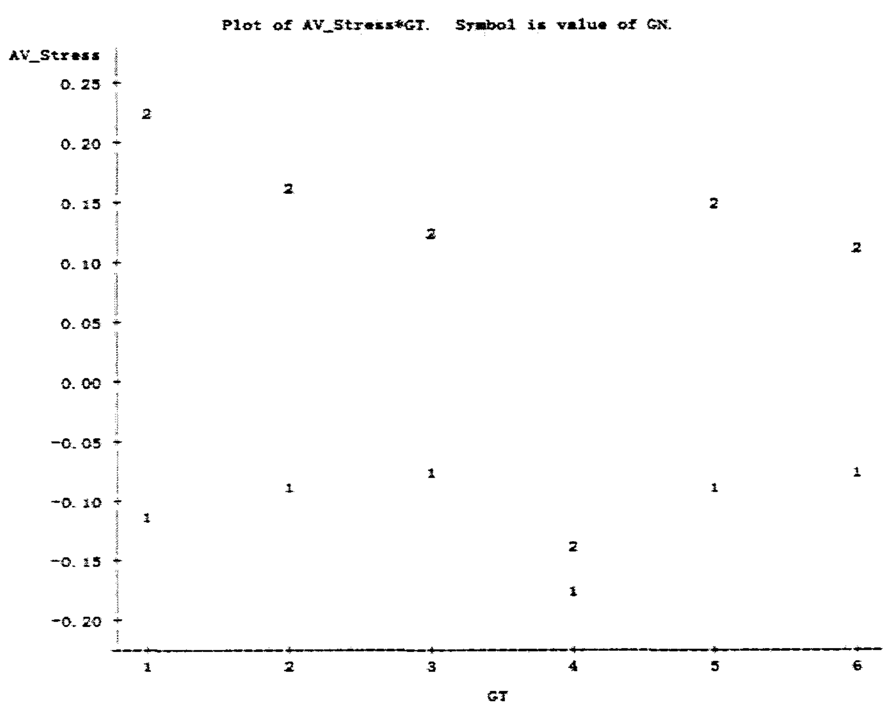


Fig. 6.28 The Effects of Treatment Factors GT and GN on Bridge Deck – Area IX, Stress Component Syy

Comparisons were made for each divided area. Detailed results were summarized in Tables 6.8 and 6.9. In the tables the symbol ">" represented the same meaning as before; while the symbol"=" represented that there was not too much difference. For bridge models containing five girders, in most areas the bridge built with Type VI and Bulb-Tee 72 girders had the best performance, which indicated that when those two kinds of girders were used to construct the bridge, normally the bridge deck stress Syy could be minimized. In some areas the differences of stress performances of bridge built with AASHTO Type V and Bulb-Tee 63 were limited. And usually for bridge built with Type IV or Bulb-Tee 54 girders, the situations were worse than other cases. In most areas, the sequence of deck stress Syy performance was bridges built with Type VI, Bulb-Tee 72, Type V, Bulb-Tee 63, Bulb-Tee 54 (or Type IV) girders, from better to worse, respectively.

Table 6.8 Comparison Results – Treatment Factor GT and GN, Girder Spacing Eight ft, Deck Stress Component Syy

Area	Comparison Results
Area I	BT-72 = Type VI > BT-63 = Type V > BT-54 > Type IV
Area II	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area III	BT-72 > Type VI > BT-63 = Type V > BT-54 > Type IV
Area IV	BT-72 > Type VI > BT-63 = Type V > BT-54 > Type IV
Area V	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area VI	Type VI > BT-72 > BT-63 = Type V > BT-54 > Type IV
Area VII	BT-72 > Type VI > BT-63 = Type V > BT-54 > Type IV
Area VIII	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IX	Type VI > BT-72 > BT-63 = Type V > BT-54 > Type IV

In Table 6.9 the comparisons were made for those bridge models that contained seven girders with girder spacing five ft. The results indicated that for most areas, the bridges built with AASHTO Type VI girder still had the best performance. The differences of stress performances of bridge built with AASHTO Type V and Bulb-Tee

72 were also limited. The stress results of Bulb-Tee 63 and Bulb-Tee 54 girder models were worse on whole deck area. Generally, the sequence of deck stress Syy performance was bridges built with Type VI, Bulb-Tee 72, Type V, Type IV, Bulb-Tee 63 and Bulb-Tee 54 girders, from better to worse, respectively.

Table 6.9 Comparison Results – Treatment Factor GT and GN, Girder Spacing Five ft, Deck Stress Component Syy

Area	Comparison Results
Area I	Type VI > BT-72 > Type V > Type IV > BT-63 > BT-54
Area II	Type VI > Type IV > Type V > BT-72 > BT-63 > BT-54
Area III	Type VI > BT-72 > Type V > Type IV > BT-63 > BT-54
Area IV	Type VI > BT-72 > Type V > Type IV > BT-63 > BT-54
Area V	Type VI > Type V > BT-72 > Type IV > BT-63 > BT-54
Area VI	Type VI > BT-72 > Type V = Type IV > BT-63 > BT-54
Area VII	Type VI > BT-72 > Type V = Type IV > BT-63 > BT-54
Area VIII	Type VI > Type V > BT-72 > Type IV > BT-63 > BT-54
Area IX	Type VI > BT-72 > Type IV > Type V > BT-63 > BT-54

Based on the above discussions, some suggestions could be given as follow:

1. For those bridge models built with five girders and girder spacing seven ft.
   a) To get the best longitudinal deck stress (Syy) performance, it is suggested to use an AASHTO Type VI or Bulb-Tee 72 girder to build the bridge; this type will give the minimum Syy stress value in the deck. Generally the sequence of suggested selection is AASHTO Type VI, Bulb-Tee 72, Bulb-Tee 63, Type V, Bulb-Tee 54 and Type IV, from better to worse, respectively.
   b) To get the best transverse deck stress (Sxx) performance, it is suggested to use an AASHTO Type VI girder to build the bridge; this type will give the minimum Sxx stress value in the deck. Generally the sequence of the

suggested selection is AASHTO Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 and Type IV, from better to worse, respectively.

c) To get the best shear deck stress (Sxy) performance, it is suggested to use an AASHTO Type VI girder to build the bridge; this type will give the minimum Sxy stress value in the deck. Generally the sequence of the suggested selection is AASHTO Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 (or Type IV), from better to worse, respectively.

2. For those bridge models built with seven girders and girder spacing five ft.

a) To get the best longitudinal deck stress (Syy) performance, it is suggested to use an AASHTO Type VI girder to build the bridge; this type will give the minimum Syy stress value in the deck. Generally the sequence of the suggested selection is AASHTO Type VI, Bulb-Tee 72, Type V, Type IV, Bulb-Tee 63 and Bulb-Tee 54, from better to worse, respectively.

b) To get the best transverse deck stress (Sxx) performance, it is suggested to use an AASHTO Type VI girder to build the bridge; this type will give the minimum Sxx stress value in the deck. Generally the sequence of the suggested selection is AASHTO Type VI, Bulb-Tee 72 (or Type V), Bulb-Tee 63, Bulb-Tee 54 and Type IV, from better to worse, respectively.

c) To get the best shear deck stress (Sxy) performance, it is suggested to use an AASHTO Type VI girder to build the bridge; this type will give the minimum Sxy stress value in the deck. Generally the sequence of the suggested selection is AASHTO Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63 (or Bulb-Tee 54 or Type IV), from better to worse, respectively.

While evaluating the effects of combinations of constructed girder types and girder numbers on bridge deck stresses, the effects of combinations of bridge girder types and truck loads applied on the bridges on bridge deck stresses were also important and need to be investigated. Similar methods described before were also used to evaluate the effects. Fig. 6.29 to Fig. 6.37 were used to evaluate the effects of GT and GN combinations on bridge deck stress component Sxx. The GT values one through six also represented girder type AASHTO Bulb-Tee 54, Bulb-Tee 63, Bulb-Tee 72, Type IV, Type V and Type VI, respectively. The TT values one and two represented the truck loads HS20-44 and FHWA 3S3, which were applied to the bridge models, respectively. The standard was whether the absolute stress value was close to zero. From the figures it is easy to determine, when HS20-44 or FHWA 3S3 truck loads traveled on bridges, which type of girder would give us the minimum Sxx value in the bridge deck.

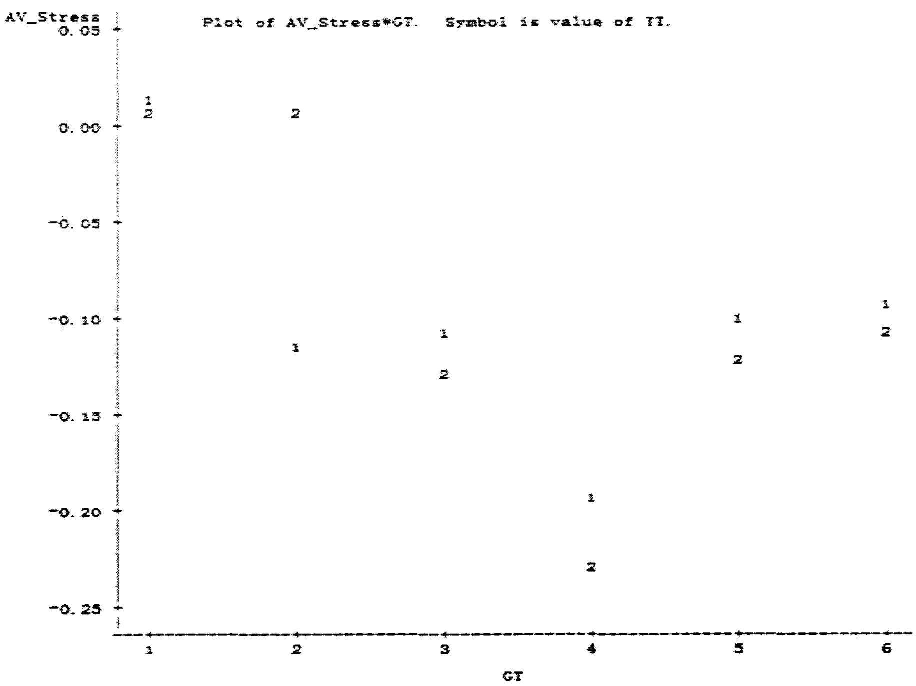


Fig. 6.29 The Effects of Treatment Factors GT and TT on Bridge Deck – Area I, Stress Component Sxx

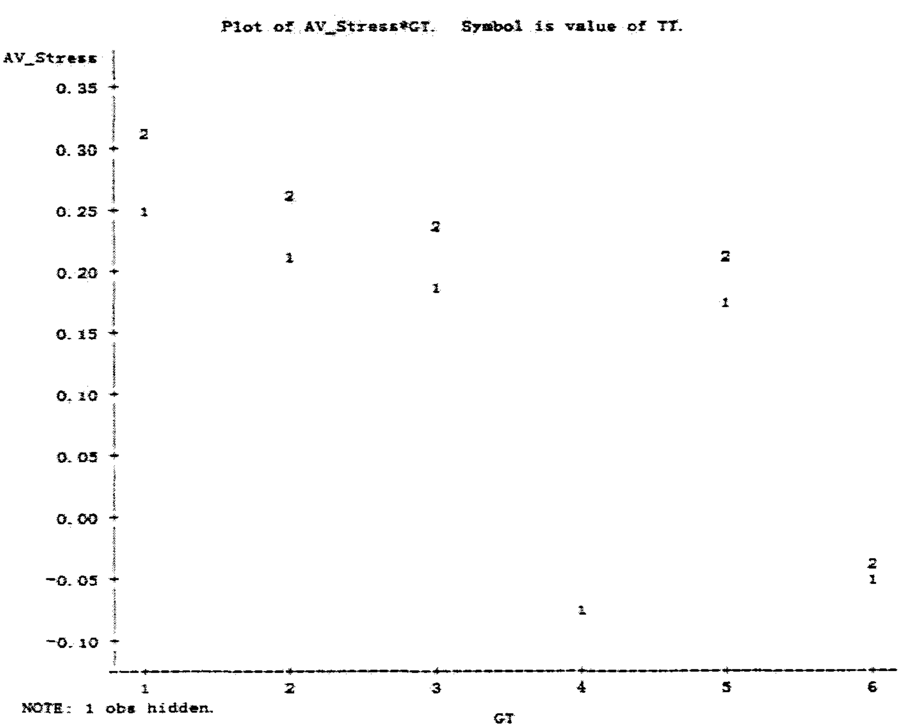


Fig. 6.30 The Effects of Treatment Factors GT and TT on Bridge Deck – Area II, Stress Component Sxx

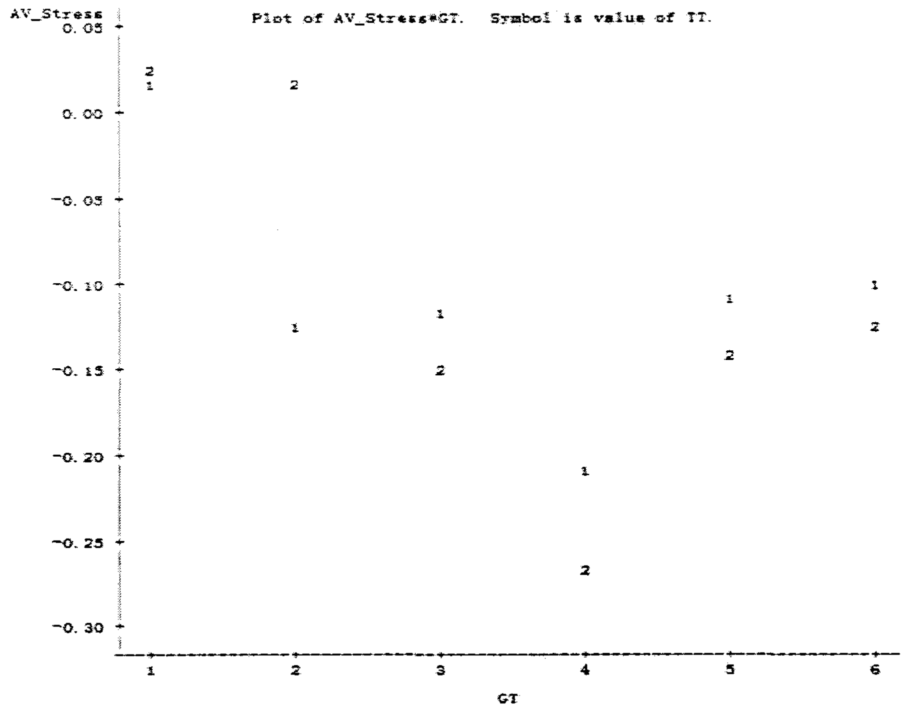


Fig. 6.31 The Effects of Treatment Factors GT and TT on Bridge Deck – Area III, Stress Component Sxx

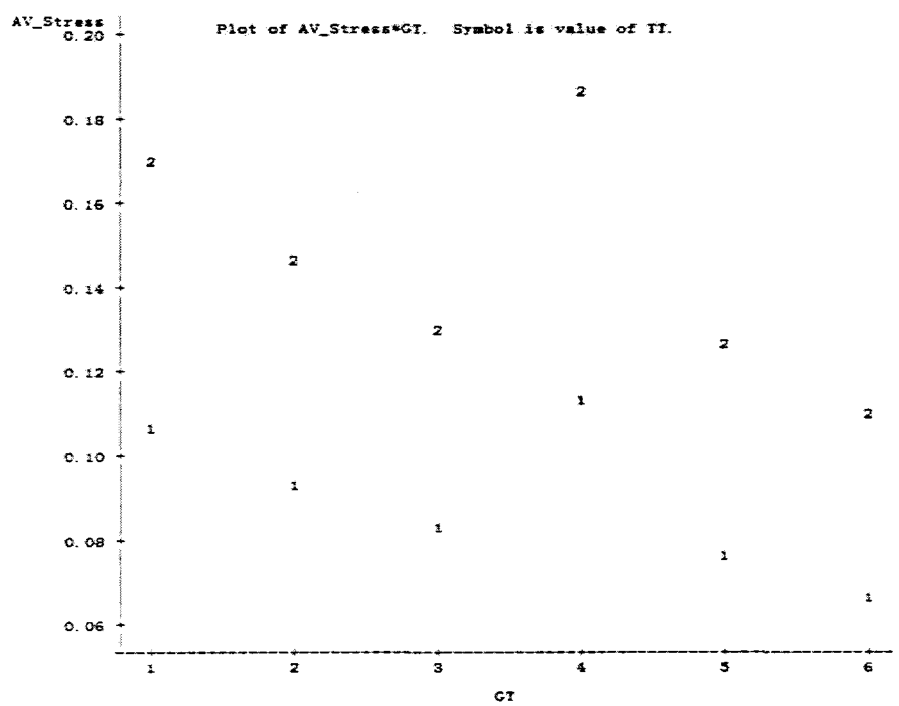


Fig. 6.32 The Effects of Treatment Factors GT and TT on Bridge Deck – Area IV, Stress Component Sxx

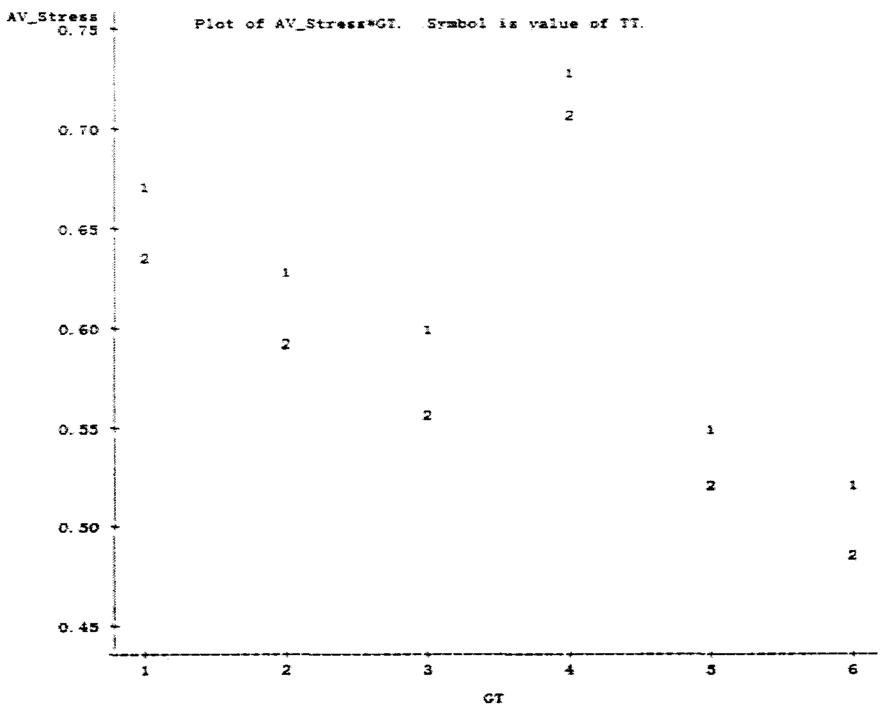


Fig. 6.33 The Effects of Treatment Factors GT and TT on Bridge Deck – Area V, Stress Component Sxx

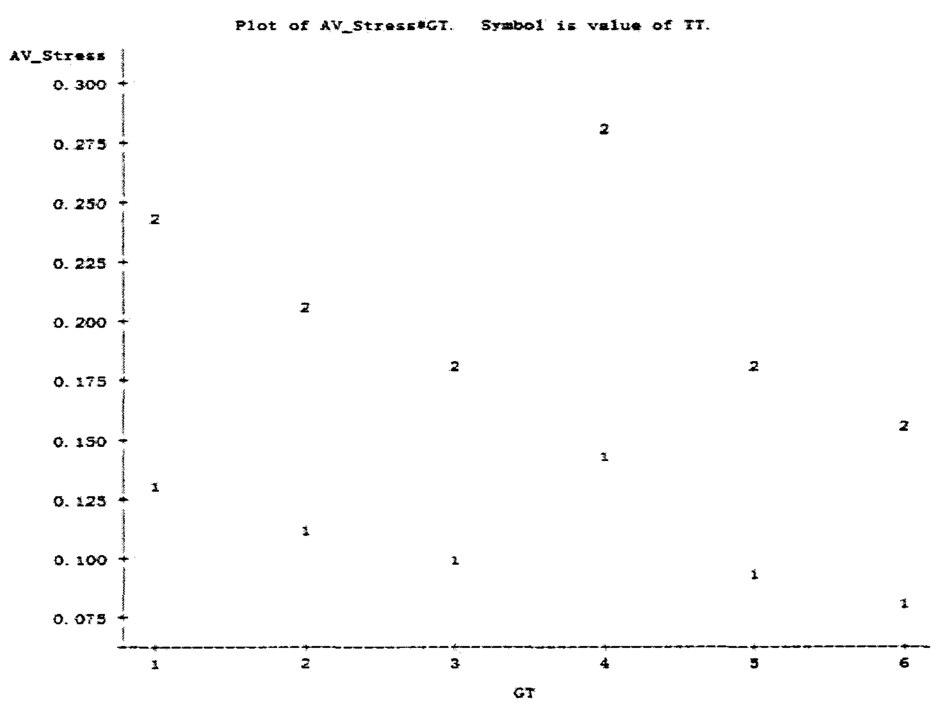


Fig. 6.34 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VI, Stress Component Sxx

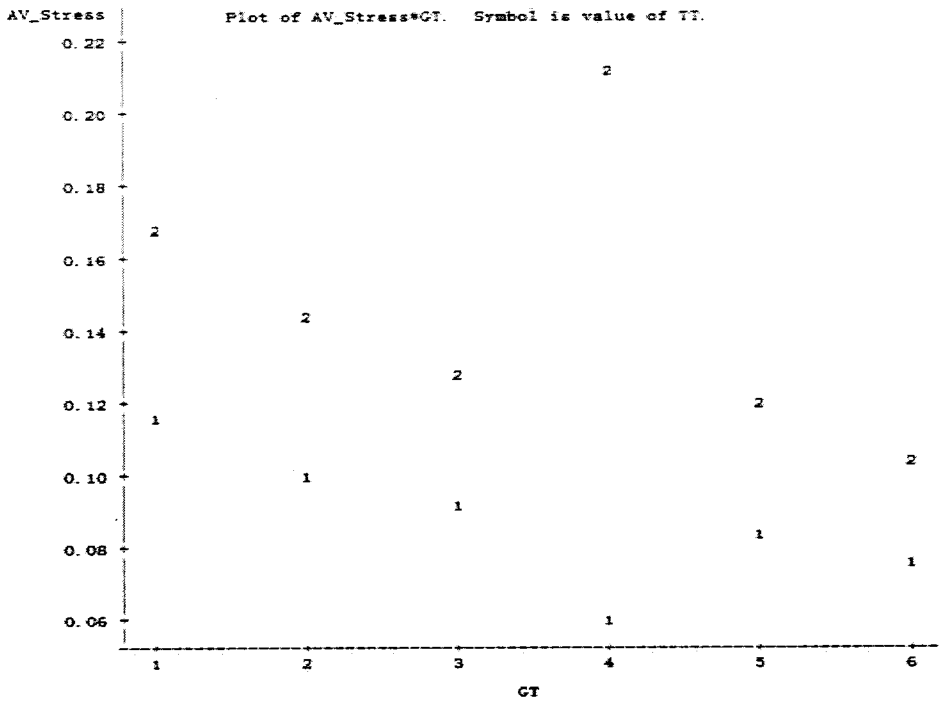


Fig. 6.35 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VII, Stress Component Sxx

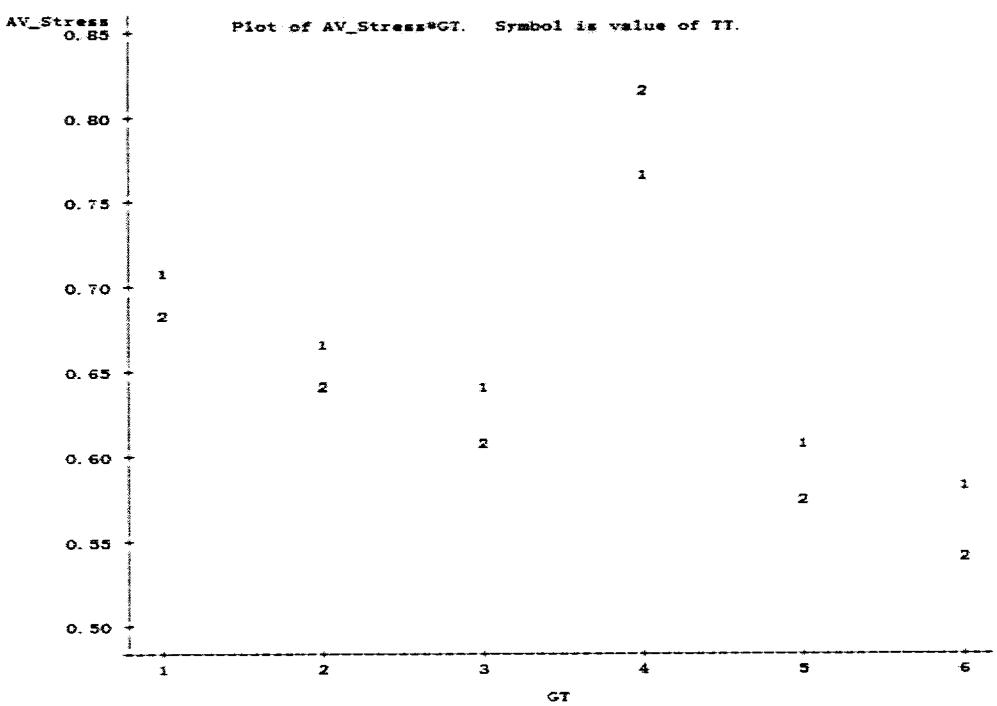


Fig. 6.36 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VIII, Stress Component Sxx

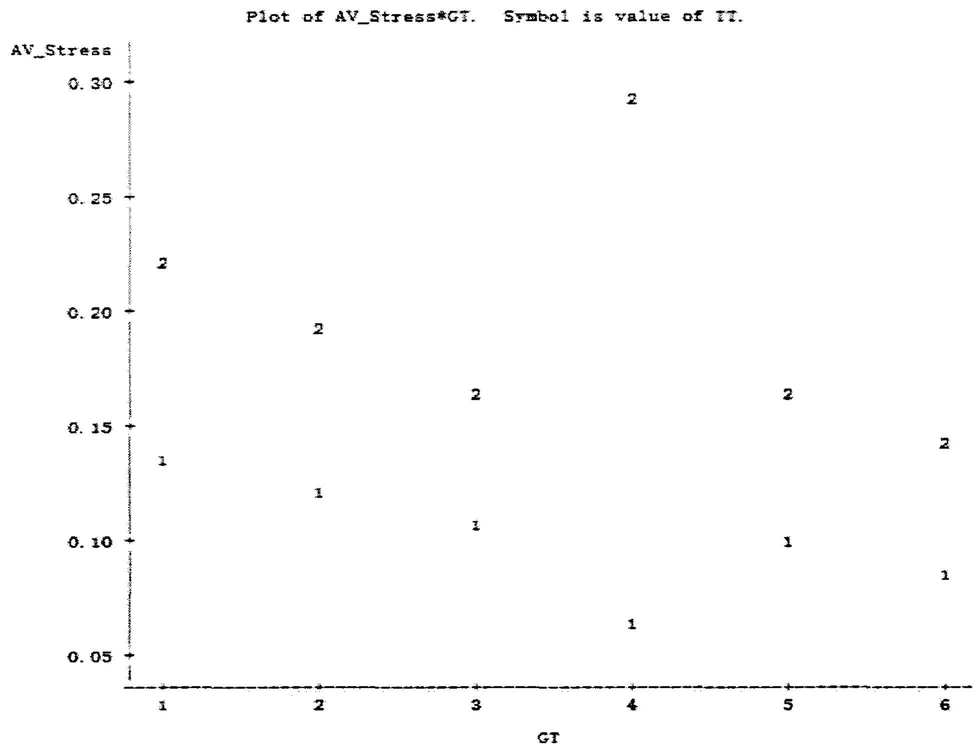


Fig. 6.37 The Effects of Treatment Factors GT and TT on Bridge Deck – Area IX, Stress Component Sxx

Comparisons were made for each divided area. Detail results were summarized in Tables 6.10 and 6.11. In the tables the symbols ">" and "=" represented the same meaning as before. When an HS20-44 truck load was applied to the bridge model, in the center areas, the bridge built with a Type VI girder had the best performance; in those parts the bridge deck stress Sxy could be minimized. At four corners the models built with Bulb-Tee 54 or Type IV girders were governing; while in other parts, usually for bridges built with Type IV or Bulb-Tee 54 girders, the situation was worse than other cases. In most areas, generally the sequence of deck stress Sxx performance was bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 and Type IV girders, from better to worse, respectively.

Table 6.10 Comparison Results – Treatment Factor GT and TT, Truck Load Type HS20-44, Deck Stress Component Sxx

Area	Comparison Results
Area I	BT-54 > Type VI > Type V > BT-72 > BT-63 > Type IV
Area II	Type VI > Type IV > Type V > BT-72 > BT-63 > BT-54
Area III	BT-54 > Type VI > Type V > BT-72 > BT-63 > Type IV
Area IV	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area V	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VI	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VII	Type IV > Type VI > Type V > BT-72 > BT-63 > BT-54
Area VIII	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IX	Type IV > Type VI > Type V > BT-72 > BT-63 > BT-54

In Table 6.11 the comparisons were made for those bridge models that had applied an FHWA 3S3 truck. The results indicated that for most areas, the bridges built with an AASHTO Type VI girder still had the best performance. The differences of stress performances of bridges built with AASHTO Type V and Bulb-Tee 72 were also limited. At two corners the model built with a Bulb-Tee 54 girder was governing, but generally the sequence of deck stress Sxx performance was bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 and Type IV girders, from better to worse, respectively.

Table 6.11 Comparison Results – Treatment Factor GT and TT, Truck Load Type FHWA 3S3, Deck Stress Component Sxx

Area	Comparison Results
Area I	BT-54 = BT-63 > Type VI > Type V > BT-72 > Type IV
Area II	Type VI > Type IV > Type V > BT-72 > BT-63 > BT-54
Area III	BT-54 > BT-63 > Type VI > Type V > BT-72 > Type IV
Area IV	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area V	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VI	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VII	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area VIII	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IX	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV

The similar analysis methods were applied to deck stress components Sxy. Fig. 6.38 to Fig. 6.46 showed the effects of GT and TT combinations on bridge deck stress component Sxy. The GT values and TT values represented the same meaning as described before. The standard was still whether the absolute stress value closed to zero. In some figures the difference of stress between models built with five or seven girders was very small, thus some observations were hidden, and only one symbol was displayed in the figure.

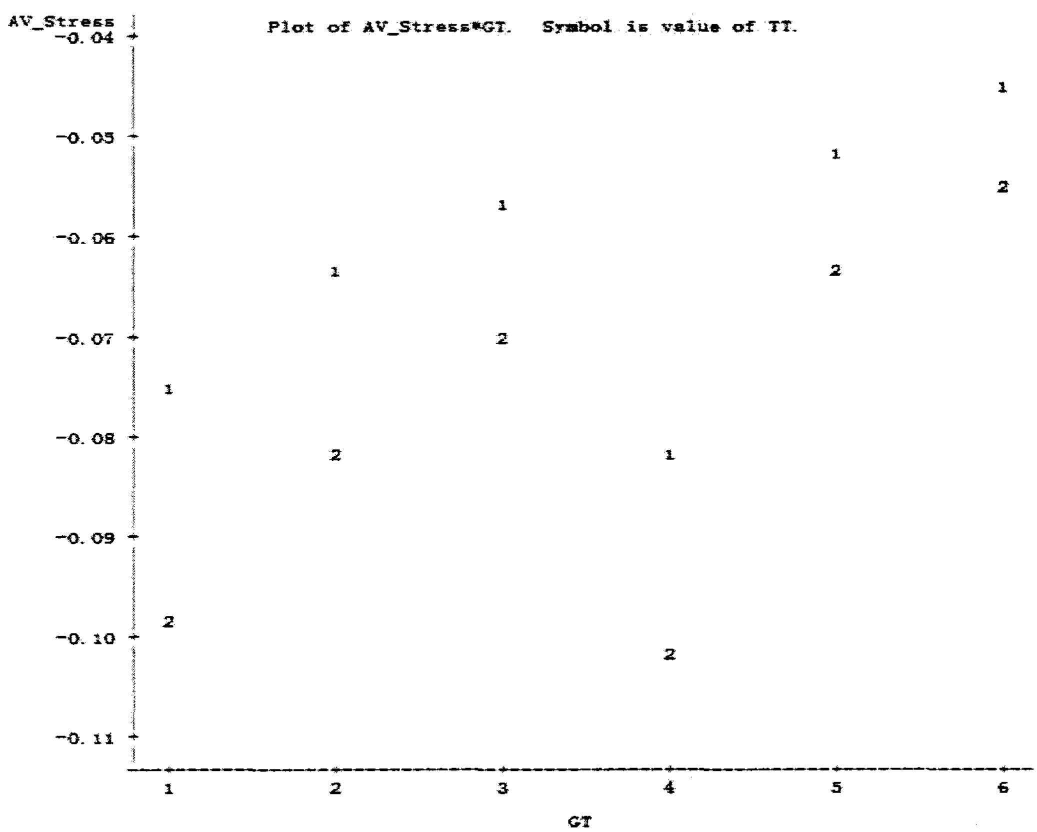


Fig. 6.38 The Effects of Treatment Factors GT and TT on Bridge Deck – Area I, Stress Component Sxy

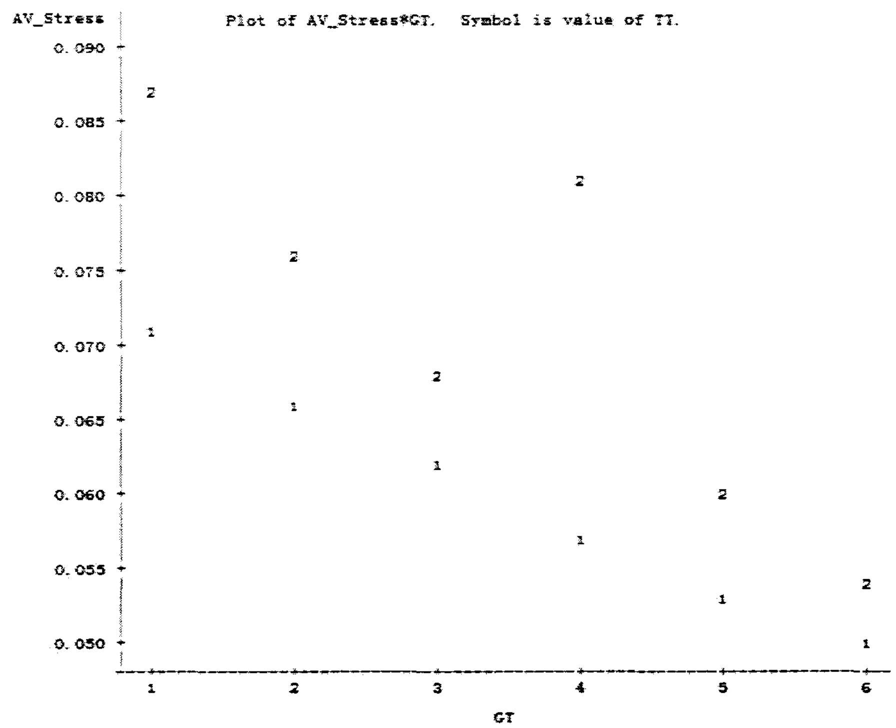


Fig. 6.39 The Effects of Treatment Factors GT and TT on Bridge Deck – Area II, Stress Component Sxy

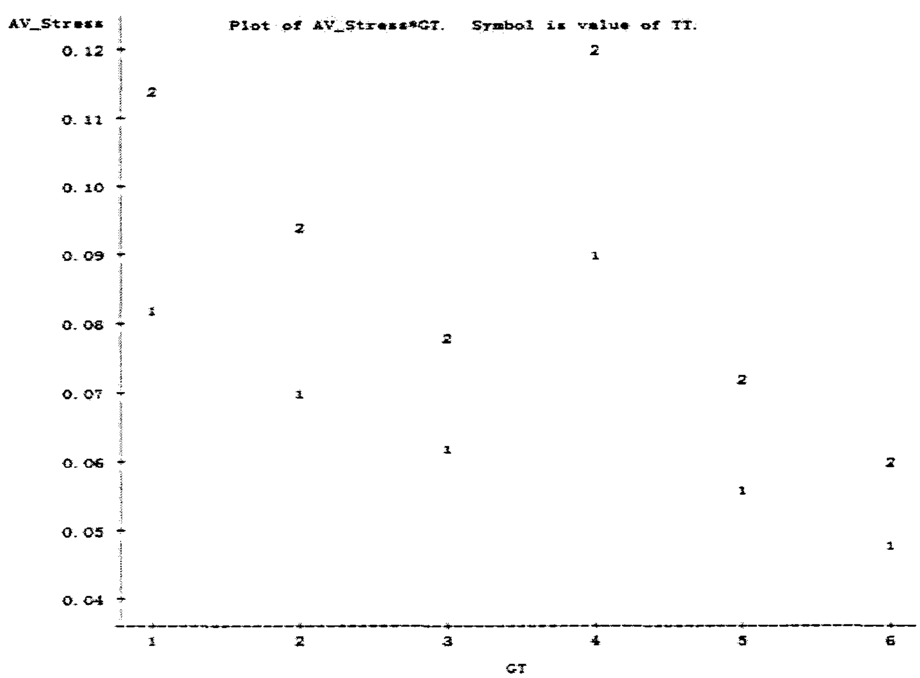


Fig. 6.40 The Effects of Treatment Factors GT and TT on Bridge Deck – Area III, Stress Component Sxy

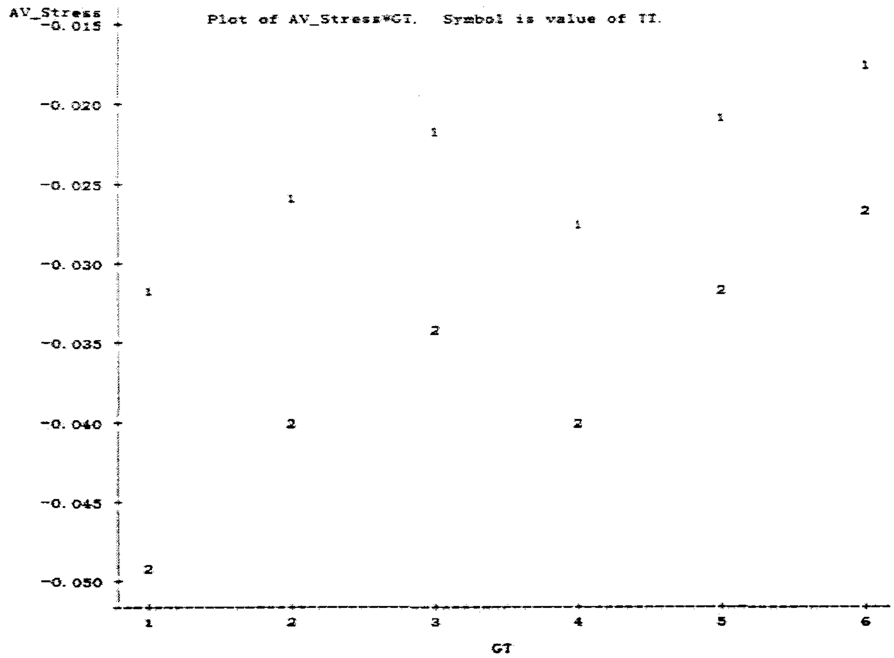


Fig. 6.41 The Effects of Treatment Factors GT and TT on Bridge Deck – Area IV, Stress Component Sxy

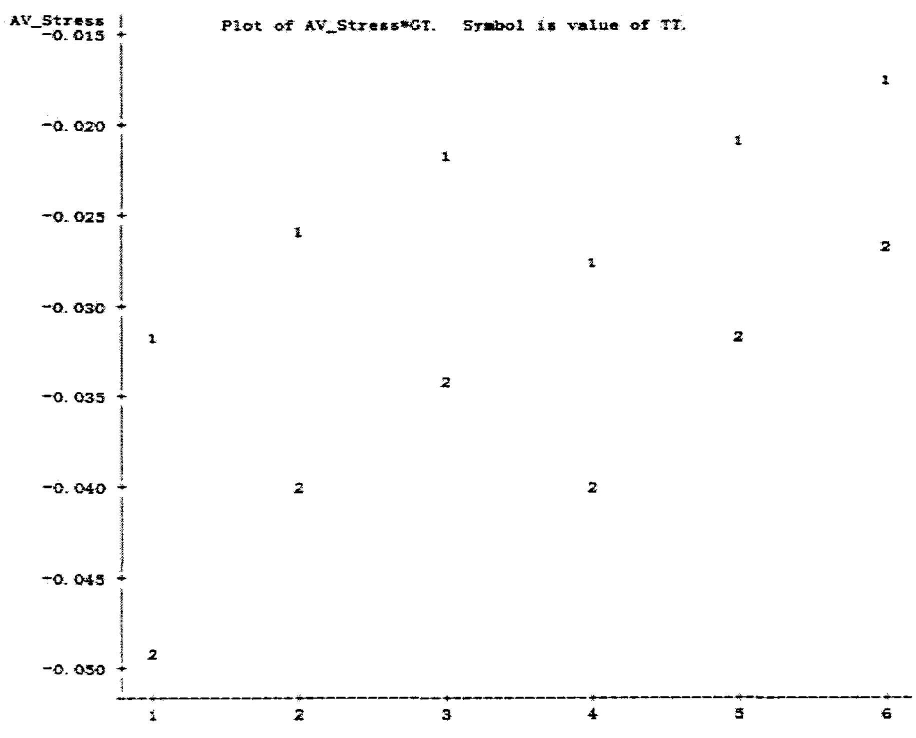


Fig. 6.42 The Effects of Treatment Factors GT and TT on Bridge Deck – Area V, Stress Component Sxy

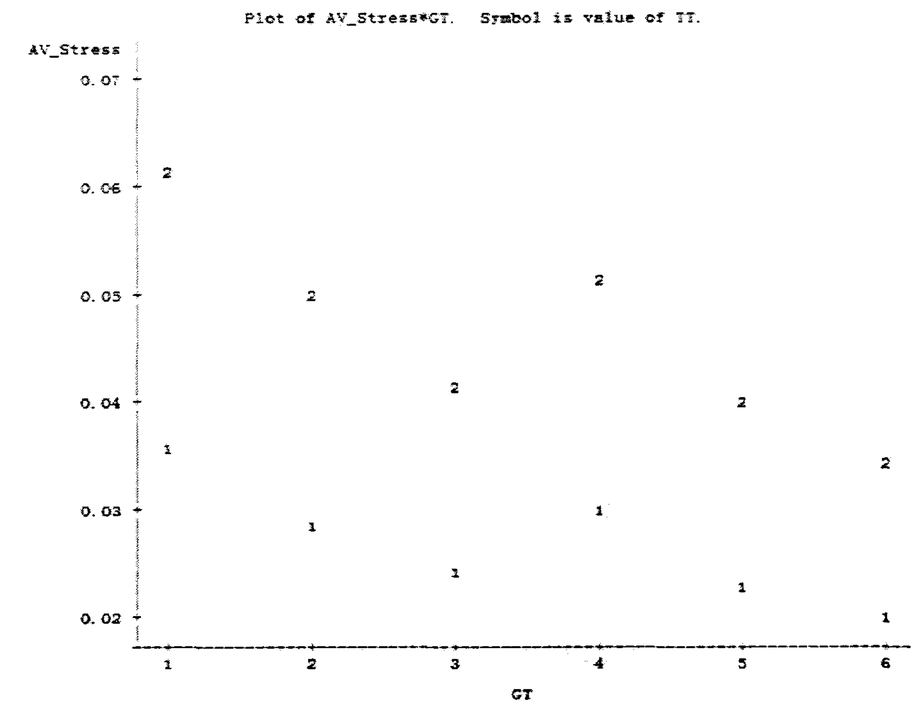


Fig. 6.43 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VI, Stress Component Sxy

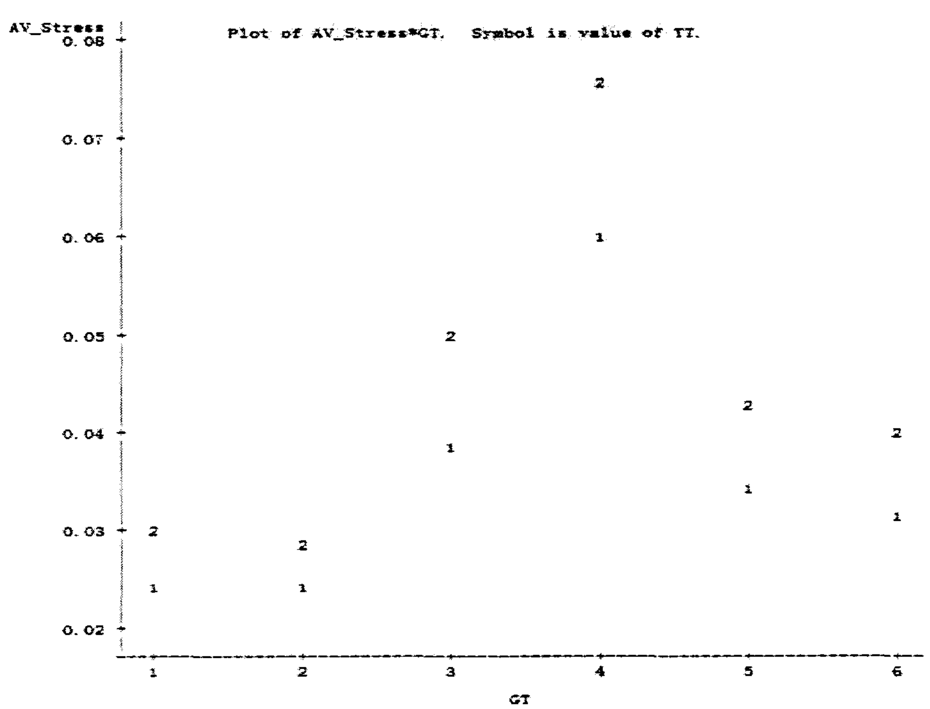


Fig. 6.44 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VII, Stress Component Sxy

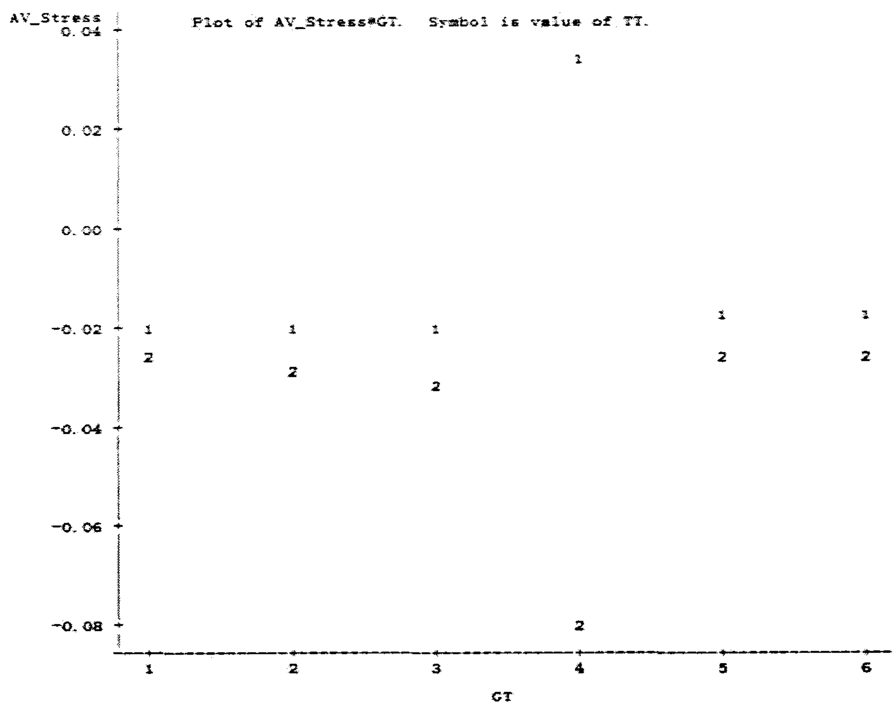


Fig. 6.45 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VIII, Stress Component Sxy

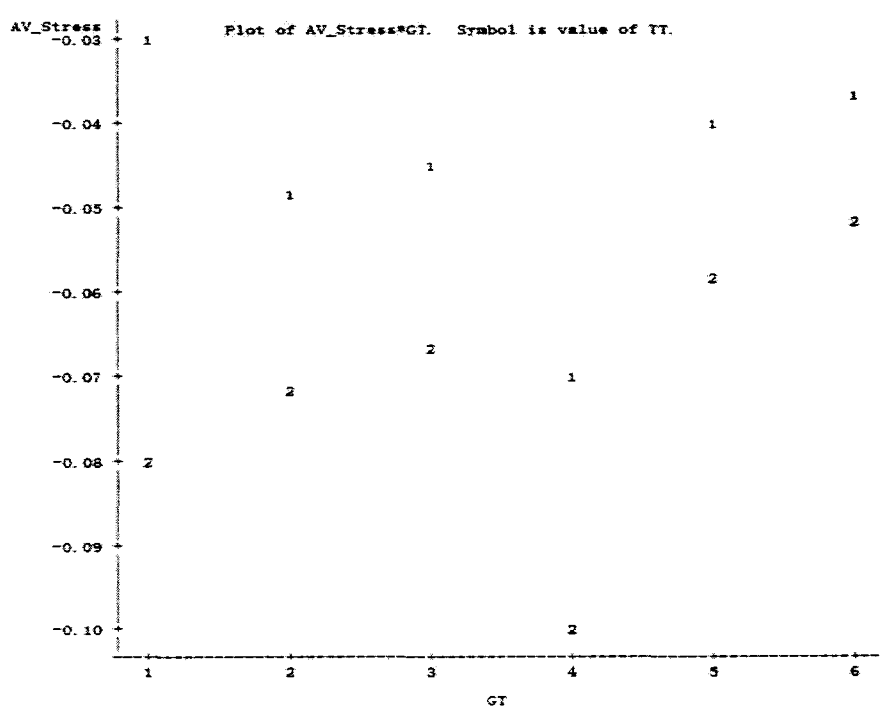


Fig. 6.46 The Effects of Treatment Factors GT and TT on Bridge Deck – Area IX, Stress Component Sxy

Comparisons were made for each divided area. Detailed results were summarized in Tables 6.12 and 6.13. In the tables the symbols ">" and "=" represented the same meaning as before. When an HS20-44 truck load was applied to the bridge model, in most areas the bridge built with a Type VI girder had the best performance; in those parts the bridge deck stress Sxy could be minimized. And usually for bridges built with Type IV or Bulb-Tee 54 girders, the situations were worse than other cases. In most areas, generally the sequence of deck stress Sxy performance was bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 (or Type IV) girders, from better to worse, respectively.

Table 6.12 Comparison Results – Treatment Factor GT and TT, Truck Load Type HS20-44, Deck Stress Component Sxy

Area	Comparison Results
Area I	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area II	Type VI > Type V > Type IV > BT-72 > BT-63 > BT-54
Area III	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IV	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area V	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area VI	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area VII	BT-63 = BT-54 > Type VI > Type V > BT-72 > Type IV
Area VIII	Type VI = Type V > BT-54 = BT-63 = BT-72 > Type IV
Area IX	BT-54 > Type VI > Type V > BT-72 > BT-63 > Type IV

In Table 6.11 the comparisons were made for those bridge models that had applied an FHWA 3S3 truck load. The results indicated that for most areas, the bridges built with AASHTO Type VI girder still had the best performance. The differences of stress performance of bridge built with AASHTO Type V and Bulb-Tee 72 were also limited. At a corner the models built with Bulb-Tee 54 and Bulb-Tee 63 girders were governing, but generally the sequence of deck stress Sxy performance was bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 (or Type IV) girders, from better to worse, respectively.

Table 6.13 Comparison Results – Treatment Factor GT and TT, Truck Load Type FHWA 3S3, Deck Stress Component Sxy

Area	Comparison Results
Area I	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area II	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area III	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Area IV	Type VI > Type V > BT-72 > BT-63 = Type IV > BT-54
Area V	Type VI > Type V > BT-72 > BT-63 = Type IV > BT-54
Area VI	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area VII	BT-63 > BT-54 > Type VI > Type V > BT-72 > Type IV
Area VIII	Type VI = Type V = BT-54 > BT-63 > BT-72 > Type IV
Area IX	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54

Similar analysis methods were applied to deck stress components Syy. Fig. 6.47 to Fig. 6.55 showed the effects of GT and TT combinations on bridge deck stress component Syy. The GT values and TT values represented the same meaning as described before. The standard was still whether the absolute stress value was close to zero. In some figures the difference in stress between models built with five or seven girders was very small, thus some observations were hidden, and only one symbol was displayed in the figure.

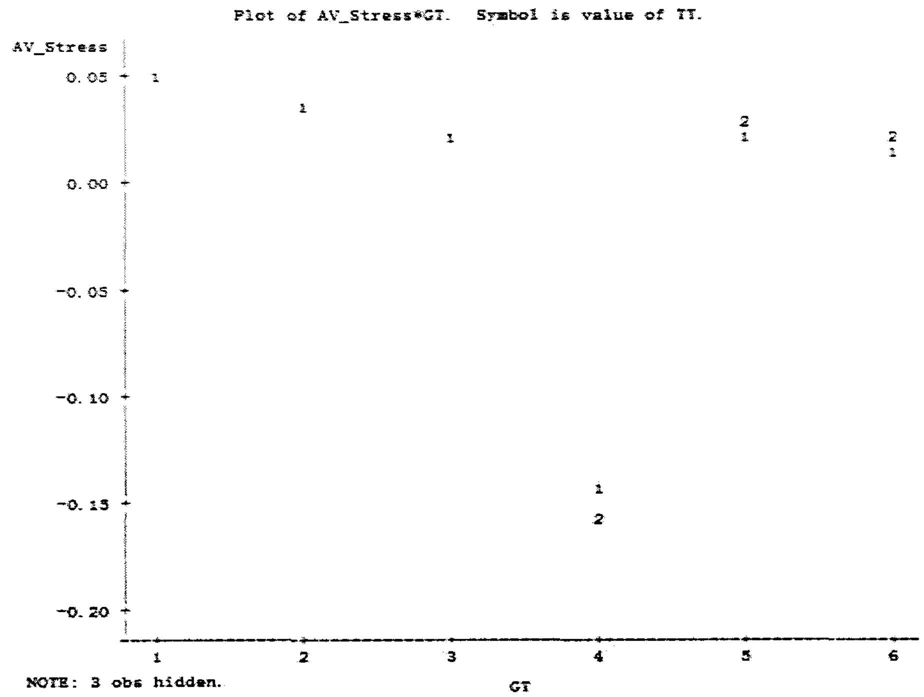


Fig. 6.47 The Effects of Treatment Factors GT and TT on Bridge Deck – Area I, Stress Component Syy

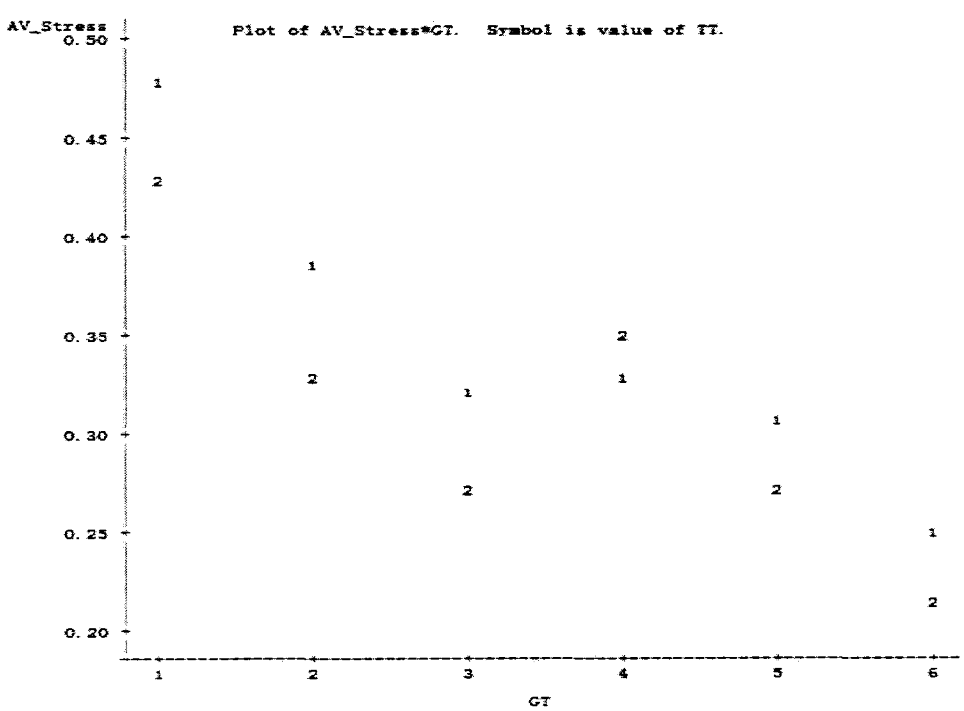


Fig. 6.48 The Effects of Treatment Factors GT and TT on Bridge Deck – Area II, Stress Component Syy

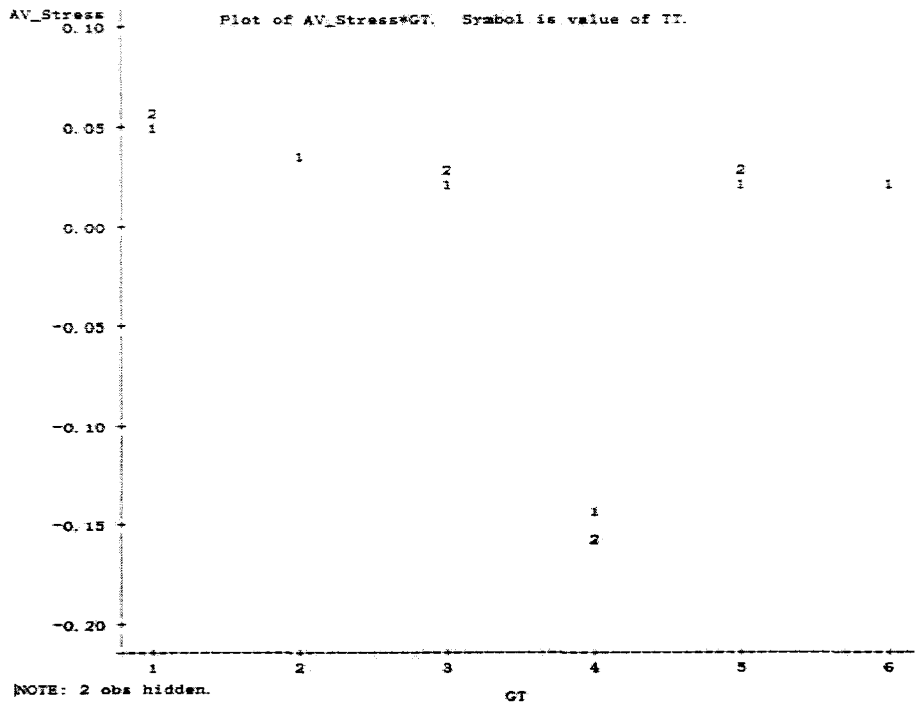


Fig. 6.49 The Effects of Treatment Factors GT and TT on Bridge Deck – Area III, Stress Component Syy

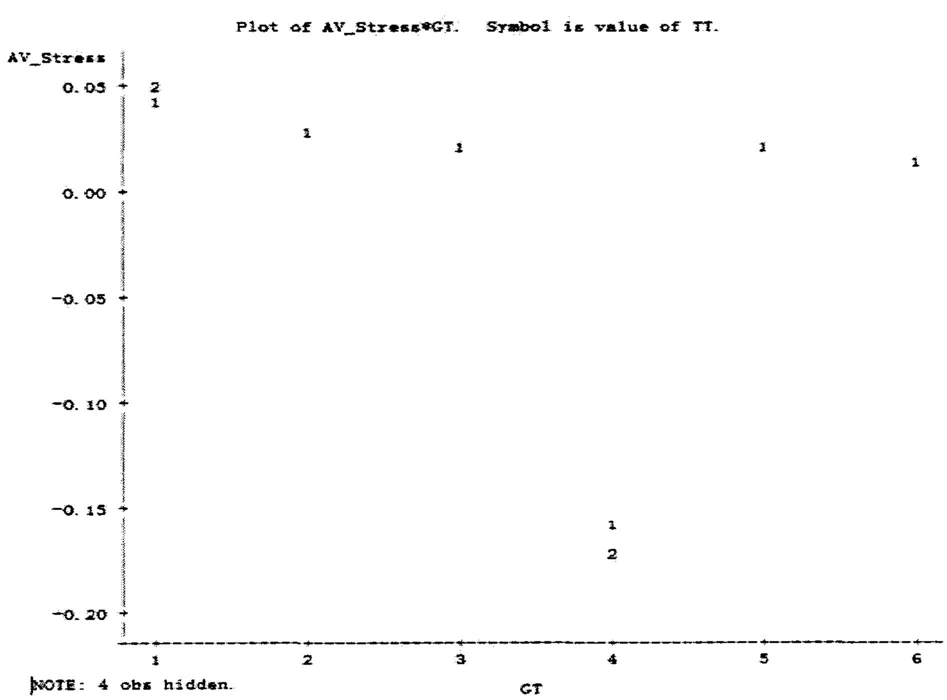


Fig. 6.50 The Effects of Treatment Factors GT and TT on Bridge Deck – Area IV, Stress Component Syy

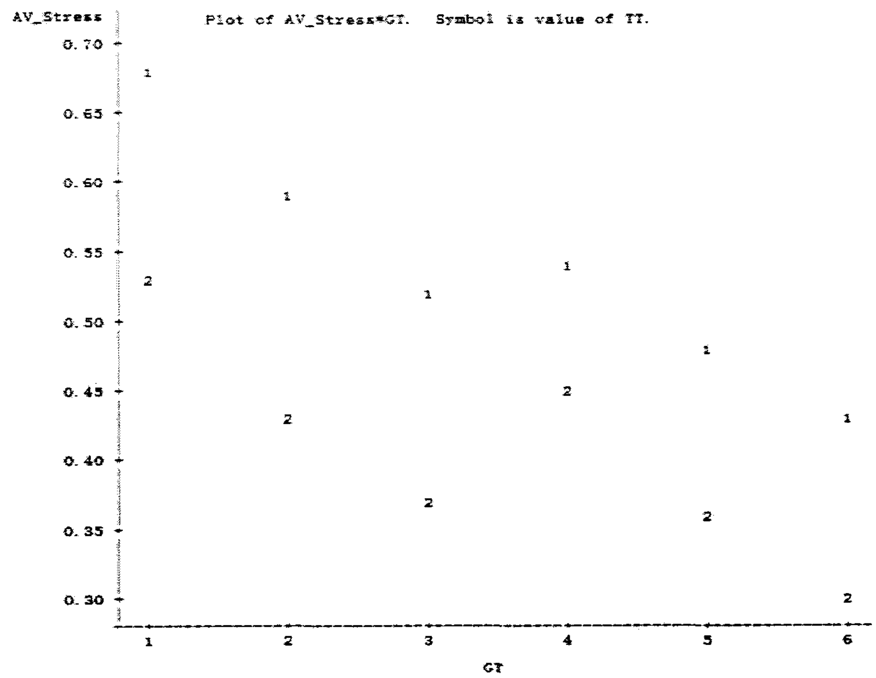


Fig. 6.51 The Effects of Treatment Factors GT and TT on Bridge Deck – Area V, Stress Component Syy

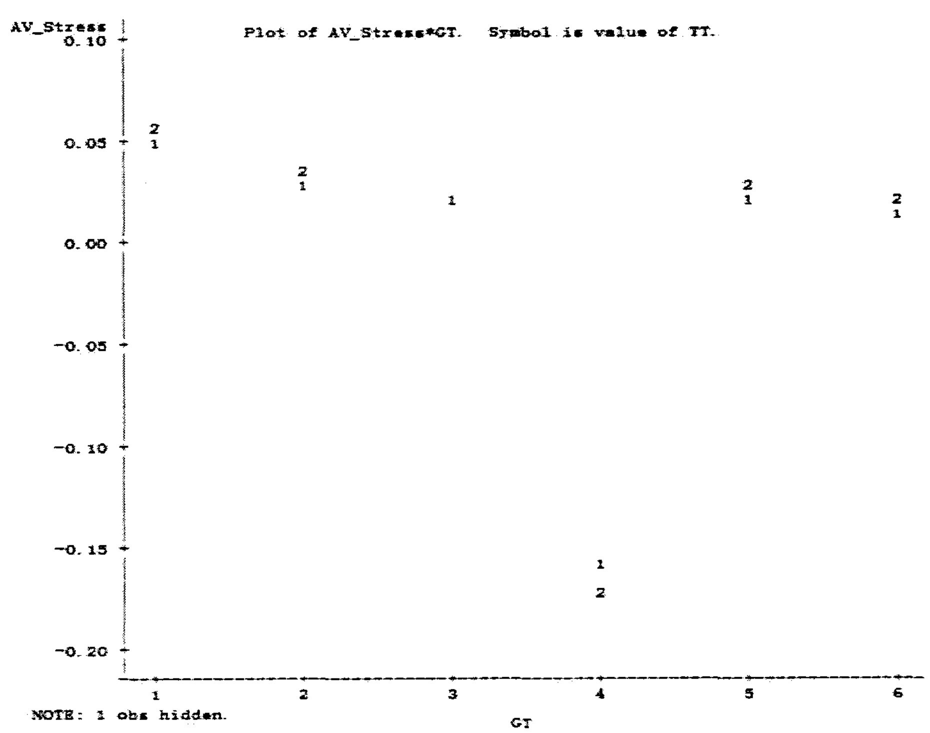


Fig. 6.52 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VI, Stress Component Syy

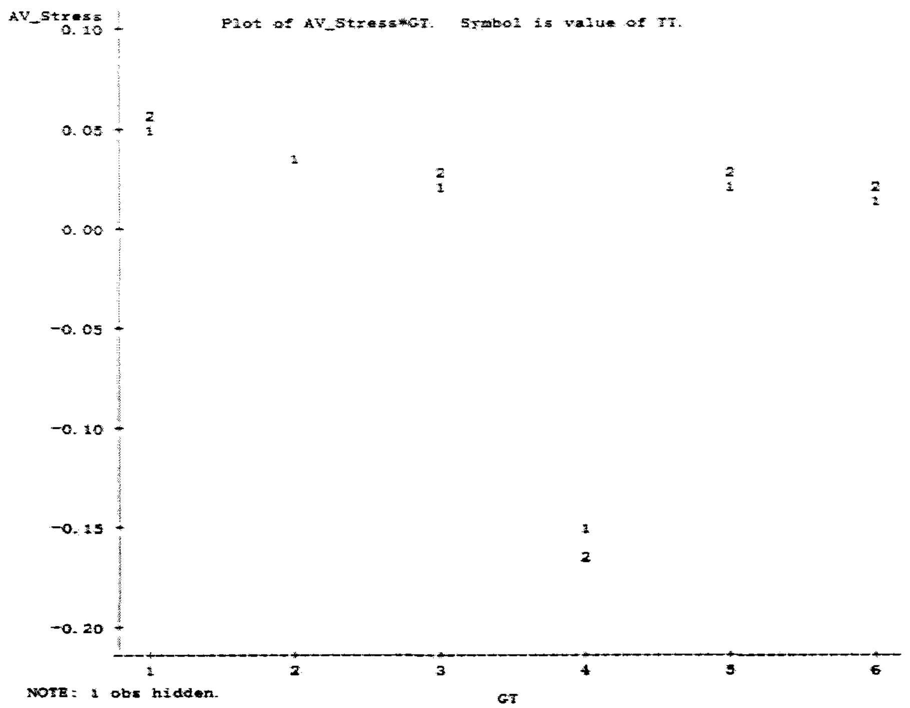


Fig. 6.53 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VII, Stress Component Syy

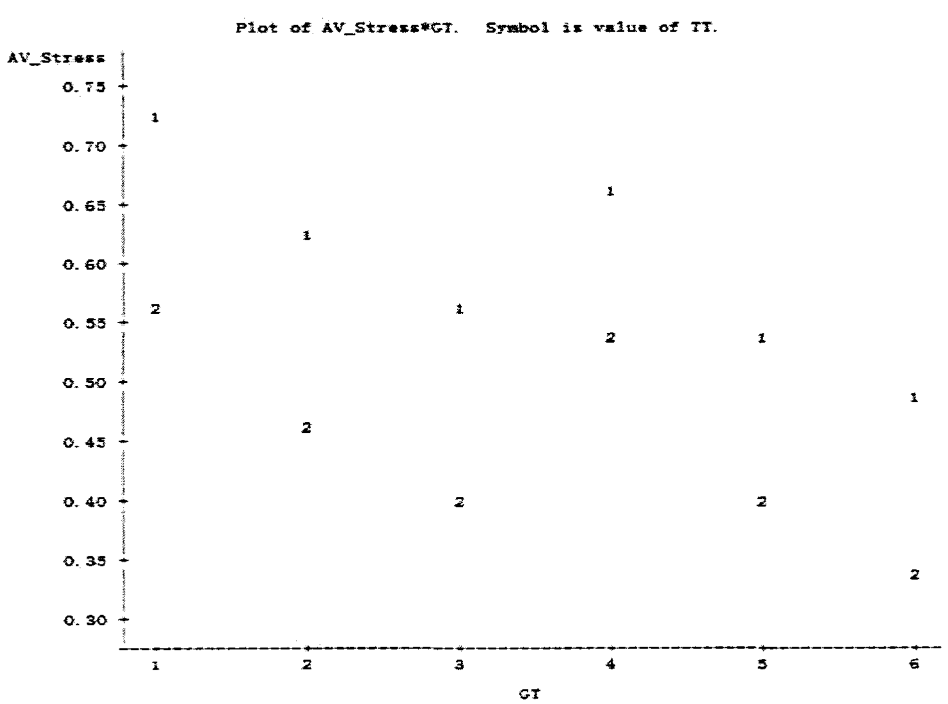


Fig. 6.54 The Effects of Treatment Factors GT and TT on Bridge Deck – Area VIII, Stress Component Syy

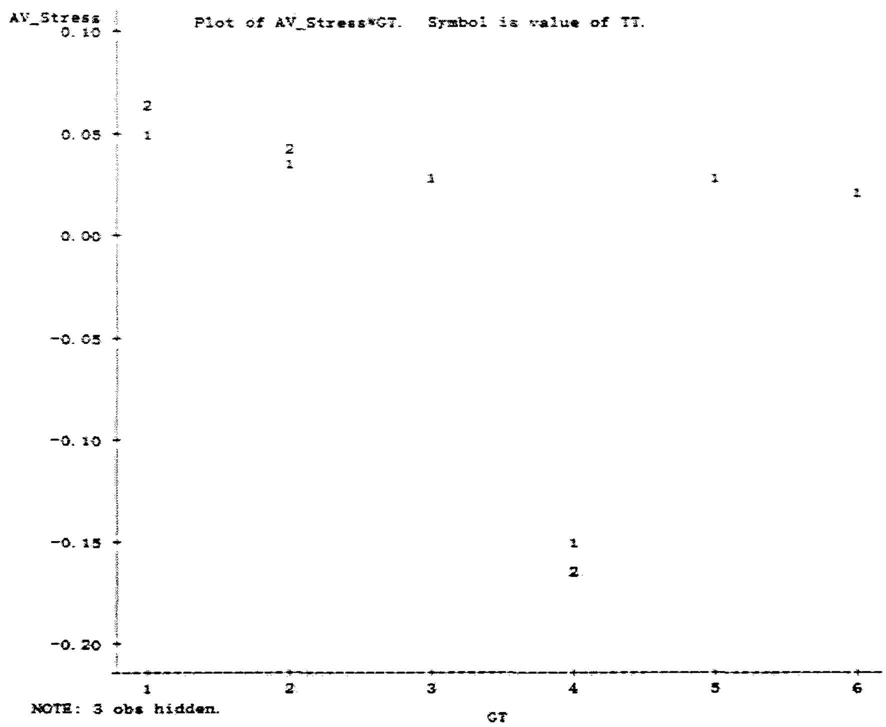


Fig. 6.55 The Effects of Treatment Factors GT and TT on Bridge Deck – Area IX, Stress Component Syy

Comparisons were made for each divided area. Detailed results were summarized in Tables 6.14 and 6.15. In the tables the symbols ">" and "=" represented the same meaning as before. When an HS20-44 truck load was applied to the bridge model, in the center areas the bridge built with the Type VI girder had the best performance; in those parts the bridge deck stress Sxy could be minimized. Usually for bridges built with Type IV or Bulb-Tee 54 girder, the situations were worse than other cases. In most areas, generally the sequence of deck stress Syy performances was bridges built with Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 (or Type IV) girders, from better to worse, respectively.

Table 6.14 Comparison Results – Treatment Factor GT and TT, Truck Load Type HS20-44, Deck Stress Component Syy

Area	Comparison Results
Area I	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV
Area II	Type VI > Type V > BT-72 > Type IV > BT-63 > BT-54
Area III	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV
Area IV	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV
Area V	Type VI > Type V > BT-72 > Type IV > BT-63 > BT-54
Area VI	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV
Area VII	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV
Area VIII	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area IX	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV

In Table 6.15 the comparisons were made for those bridge models that had applied an FHWA 3S3 truck load. The results indicated that for most areas, the bridges built with an AASHTO Type VI girder still had the best performance. The differences in stress performances of bridges built with AASHTO Type V and Bulb-Tee 72 were also limited. Usually for bridges built with Type IV or Bulb-Tee 54 girders, the situations were worse than other cases. Generally the sequence of deck stress Syy performance was bridges built with Type VI, Bulb-Tee 72, Type V, Bulb-Tee 63, Bulb-Tee 54 (or Type IV) girders, from better to worse, respectively.

Table 6.15 Comparison Results – Treatment Factor GT and TT, Truck Load Type FHWA 3S3, Deck Stress Component Syy

Area	Comparison Results
Area I	Type VI = BT-72 > Type V > BT-63 > BT-54 > Type IV
Area II	Type VI > Type V = BT-72 > BT-63 > Type IV > BT-54
Area III	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV
Area IV	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV
Area V	Type VI > Type V > BT-72 > BT-63 > Type IV > BT-54
Area VI	Type VI = BT-72 > Type V > BT-63 > BT-54 > Type IV
Area VII	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV
Area VIII	Type VI > Type V = BT-72 > BT-63 > Type IV > BT-54
Area IX	Type VI > Type V = BT-72 > BT-63 > BT-54 > Type IV

Based on above discussions, some suggestions could be given as follows:

1. When an HS20-44 truck load is applied to the bridge models:

   a) To get the best longitudinal deck stress (Syy) performance, it is suggested to use AASHTO Type VI girders to build the bridge; this girder will give the minimum Syy stress value in the deck. Even at some corners, bridges built with Bulb-Tee 63 or 54 girders would give us the best values. Generally, the sequence of suggested selection is AASHTO Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 (or Type IV), from better to worse, respectively.

   b) To get the best transverse deck stress (Sxx) performance, it is suggested to use AASHTO Type VI girders to build the bridge; this girder will give the minimum Sxx stress value in the deck. Even at some corners bridges built with Bulb-Tee 54 or Type IV girders would give us the best values. Generally, the sequence of suggested selection is AASHTO Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 and Type IV, from better to worse, respectively.

   c) To get the best shear deck stress (Sxy) performance, it is suggested to use AASHTO Type VI girders to build the bridge; this girder will give the minimum Sxy stress value in the deck. Generally, the sequence of suggested selection is AASHTO Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 and Type IV, from better to worse, respectively.

2. When an FHWA 3S3 truck load is applied to the bridge models:

a) To get the best longitudinal deck stress (Syy) performance, it is suggested to use AASHTO Type VI girders to build the bridge; this girder will give the minimum Syy stress value in the deck. Generally, the sequence of suggested selection is AASHTO Type VI, Bulb-Tee 72, Type V, Type IV, Bulb-Tee 63 and Bulb-Tee 54, from better to worse, respectively.

b) To get the best transverse deck stress (Sxx) performance, it is suggested to use AASHTO Type VI girders to build the bridge; this girder will give the minimum Sxx stress value in the deck. Generally, the sequence of suggested selection is AASHTO Type VI, Bulb-Tee 72 (or Type V), Bulb-Tee 63, Bulb-Tee 54 and Type IV, from better to worse, respectively.

c) To get the best shear deck stress (Sxy) performance, it is suggested to use AASHTO Type VI girders to build the bridge; this girder will give the minimum Sxy stress value in the deck. Generally, the sequence of suggested selection is AASHTO Type VI, Type V, Bulb-Tee 72, Bulb-Tee 63, Bulb-Tee 54 and Type IV, from better to worse, respectively.

Generally ANOVA was used for the purpose of determining the importance of treatment factors. The outputs generated by SAS had two option tables to be evaluated. Information concerning main effects and interactions was provided underneath the table under the heading "Type I" and "Type III" sums of squares. In this study the Type I and Type III sums of squares were identical when the sample sizes were equal, since the factorial effects were then estimated independently of one another, In appendix D detailed ANOVA tables were listed for all areas and stress components. The ratio of mean square and *MSE* is the F value. When the F value is larger, the corresponding factor or factor combination is more significant. Tables 6.16 to 6.18 summarized the comparison results of treatment factors GN, GT and TT. In the tables the symbol ">" represented "more significant," the symbol "=" represented "had the equal importance," and the symbol "≈" represented "almost equal".

According to Table 6.16, in total nine areas, there were six areas in which treatment factor GN had the most important effects on deck stress component Sxx; while treatment factor TT had the least important effects in six areas. This phenomenon indicated that in transverse direction, girder number was the most significant factor to influence the deck stress behavior.

Table 6.17 showed that in longitudinal direction factor GN was still the key factor. It had the largest F values in seven of nine areas. While there were two areas the treatment factor TT was governing, in rest areas the F values of factor TT were very limited, and some of the values were close to zero. Therefore, the researcher determined that the effects of truck types on deck stress in the longitudinal direction were limited.

In Table 6.18 factor GN controlled six outside areas, while in three internal areas factor TT was governing. Compared to factor girder number and truck load type, factor girder type had the least significance on the shear stress of the bridge deck.

Table 6.16 ANOVA Results Comparison – Deck Stress Component Sxx

Area	Comparison Results
Area I	GT > GN > TT
Area II	GN > GT > TT
Area III	GT > GN > TT
Area IV	GN > GT > TT
Area V	GN > GT > TT
Area VI	TT > GN > GT
Area VII	GN > TT > GT
Area VIII	GN > GT > TT
Area IX	GN > TT > GT

Table 6.17 ANOVA Results Comparison – Deck Stress Component Syy

Area	Comparison Results
Area I	GN > GT > TT (≈ 0)
Area II	GN > GT > TT
Area III	GN > GT > TT (≈ 0)
Area IV	GN > GT > TT (≈ 0)
Area V	TT > GN > GT
Area VI	GN > GT > TT (≈ 0)
Area VII	GN > GT > TT (≈ 0)
Area VIII	TT > GN > GT
Area IX	GN > GT > TT (≈ 0)

Table 6.18 ANOVA Results Comparison – Deck Stress Component Sxy

Area	Comparison Results
Area I	GN > TT > GT
Area II	GN > TT > GT
Area III	GN > TT > GT
Area IV	TT > GN = GT
Area V	TT > GN > GT
Area VI	TT > GT > GN
Area VII	GN > GT > TT
Area VIII	GN > TT > GT
Area IX	GN > TT > GT

6.4 Summary

In this chapter, statistical methods were introduced to analysis procedure to evaluate the stress behavior of the simple span bridge deck. Factorial experiment design was used to construct the statistical models in section 6.2. In section 6.3 the models were set up and corresponding evaluations were performed based on the treatment factor combinations (Girder Type, Girder Number) and (Girder Type, Truck Type).

Considering the stress behavior of the bridge deck and those two combinations, the results of the evaluation showed that 1) normally bridge built with AASHTO Type VI girder had the best performance on deck stress behavior; 2) normally bridges built with AASHTO Type IV or Bulb-Tee 54 girders had the worst performance on deck stress behavior. Detailed results could be referred to section 6.3 and chapter IX.

ANOVA was also performed, and comparisons were made among treatment factors GN, GT, and TT at the end of section 6.3. In longitudinal and transverse directions, girder number was the most important factor affecting the deck stress behavior, then factor girder type, then factor truck load type. For shear stress, both girder number and truck load type were the controlling factors, while girder type had less effects on it.

CHAPTER VII

SIMPLE SPAN BRIDGE DIAPHRAGM PERFORMANCE UNDER THE HEAVY TRUCK LOAD

7.1 Introduction

In AASHTO LRFD bridge design specifications, 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. Consequently, most bridges constructed have intuitively included diaphragms. Depending on the type of bridge, the diaphragms may take different forms. Cast-in-place concrete diaphragms are most common in prestressed concrete I-girder bridge construction. A diaphragm terminated at the end of the sloping portion of the bottom flange is called "full depth." Generally, the diaphragm is integral with the deck through continuous reinforcement, tied to the I-girder through anchor bars.

In this study, the full-depth diaphragms were terminated at the both ends of bridge supports, and two full-depth intermediate diaphragms were added at the distance of 40 feet to both of the bridge supports. The finite element models of simple span bridges presented in chapter IV were employed to generate data for evaluations, which was presented in section 7.2.

7.2 Effects of Heavy Truck Load on Simple Span Bridge Diaphragm

The finite element models of simple span bridges presented in chapter IV were employed for evaluation. Models with six types of bridge girders and two types of girder spacing were analyzed. In the finite element analysis procedure, prismatic space truss members were used to model the end and intermediate diaphragms. In GTSTRUDL, the prismatic space truss member was limited to take only the axial forces. The maximum values obtained from each model under HS20-44 and FHWA 3S3 truck loads were compared to evaluate the impact on the diaphragms. Similar to the stress state of bridge deck and girders, for the short time effects of the 3S3 truck load on the bridges, the load combination "Strength I Max" was the governing load combination to give the maximum axial forces of the diaphragms, while the load combination "Fatigue" was used to determine the long time effects of the 3S3 truck load. The diaphragm members were numbered and grouped as shown in Fig. 7.1 and Fig. 7.2.

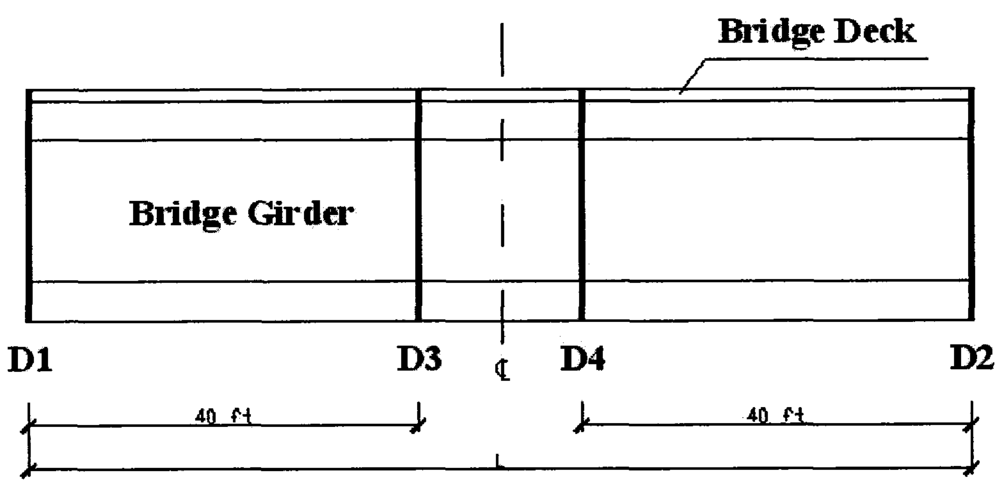


Fig. 7.1 Locations of End and Intermediate Diaphragms

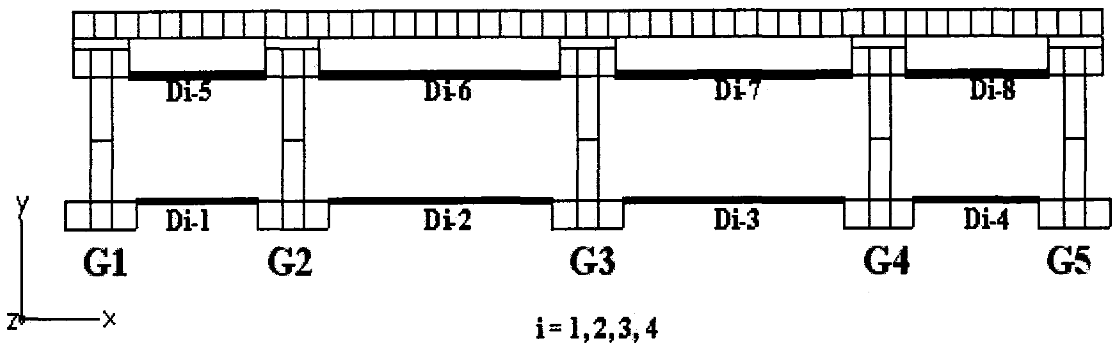


Fig. 7.2 Cross Section of Grouped Diaphragms

The maximum axial forces of each diaphragm under HS20-44 and FHWA 3S3 truck loads were computed. The maximum values were selected and the ratios of results for those two truck loads were calculated. The detail results were summarized in Table 7.1 and Table 7.2.

Table 7.1 Effects of FHWA 3S3 Truck Loads on Simple Span Bridge Diaphragms – Girder Spacing Eight ft

	Ratio of FHWA 3S3 to HS20-44	
Girder Type	***Strength I Max***	***Fatigue***
AASHTO Type IV	1.15	1.59
AASHTO Type V	1.21	1.64
AASHTO Type VI	1.27	1.64
AASHTO BT - 54	1.17	1.63
AASHTO BT - 63	1.17	1.63
AASHTO BT - 72	1.18	1.63

Table 7.2 Effects of FHWA 3S3 Truck Loads on Simple Span Bridge Diaphragms – Girder Spacing Five ft

	Ratio of FHWA 3S3 to HS20-44	
Girder Type	***Strength I Max***	***Fatigue***
AASHTO Type IV	1.23	1.63
AASHTO Type V	1.36	1.62
AASHTO Type VI	1.37	1.61
AASHTO BT - 54	1.29	1.64
AASHTO BT - 63	1.29	1.62
AASHTO BT - 72	1.37	1.61

For the bridges with girder spacing eight ft., the ratio of maximum axial force varied between 1.15 and 1.27 on the short term effects, and between 1.59 and 1.64 on the long term effects. For the bridges with girder spacing five ft., the ratio of maximum axial force varied between 1.23 and 1.37 on the short term effects, and between 1.61 and 1.64 on the long term effects. Even the bridge models were created with different types of girders; the differences of ratios were not very significant. Girder types were not the major parameters influencing effects of the heavy truck loads on simple span bridge diaphragms.

Even the values of ratios were significant, some of them were larger than 1.5. One thing should be noticed was that the maximum compressive axial force in the diaphragm was 70.67 kips, which would not lead the large axial stress which might be beyond the allowable axial stress of the concrete or reinforcement. The effects of FHWA 3S3 truck loads on the bridge diaphragms, which were designed based on the HS20-44 truck loads were limited.

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 FHWA 3S3 truck loads, the diaphragms would meet more critical situations that might need more additional inspections or frequent maintenances.

7.3 Summary

The short term and long term effects of FHWA 3S3 truck loads on simple span bridge diaphragms, which were designed based on HS20-44 truck loads, were evaluated in this chapter. The effects were determined limited, although the long term effects on the diaphragms might be more critical than the short term effects.

CHAPTER VIII

BRIDGE COSTS STUDY

8.1 Introduction

The long term effects of heavy trucks on bridges and bridge decks play an important role in the bridge life evaluation. The selected bridges 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 cost of the damage is hard to obtain since fatigue damage may lead to many actions including repairs, testing, rehabilitations, and replacements.

There were many studies done and methods used to evaluate the remaining lives of bridge structures. These studies were sponsored by federal committees such as AASHTO and NCHRP and by State DOTs. The use of these methods in this study is hindered by the amount of data needed on trucks. The site-specific information available for this study on heavy truck loads was very limited and statistically insufficient for use with the NCHRP 495 approach or the other methodologies discussed above. The approach used in this study was a similar method that was used in the study prepared for OHIO DOT, and approved during the Project Review Committee meeting 2004.

The data used in this study and presented here are based on Louisiana state project No. 736-99-1299 (also the LTRC project No. 05-2p) and Louisiana state project No. 736-99-1133 (also the LTRC project No. 03-2ST). The long term effects of FHWA 3S2 and 3S3 truck loads on Louisiana bridges were evaluated. The details can be referred to Saber et al (2005 and 2006).

8.2 Cost Model Setup

Fatigue is an important performance criterion for bridges that are evaluated. Most of the bridges in Louisiana are designed for 50 years fatigue life. Overloaded trucks will definitely shorten the life of the bridges. The bridges in this study are evaluated for fatigue cost based on the flexural and shear results of the analyses performed in chapter IV. 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 average daily traffic of the heavy truck is 2500 trucks per day.

The following equation was used to determine the percentage of the life of the bridge used when a truck crosses it:

$$\%\ of\ life = \frac{(Ratio\ from\ analysis)^3}{(2500\ trucks\ per\ day * 365\ days\ per\ year * 50\ years)} * 100 \qquad (8.1)$$

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 was considered to be $90 per square foot.

$$\textit{Cost per Trip on Bridge} = (\textit{\% of life}) * (\$90 \text{ per square foot}) \tag{8.2}$$

The effect of the heavy truck loads on the fatigue life of the bridge was ignored when the "ratio from analysis" was equal to or less than one. Therefore, the cost per trip for fatigue calculation is zero.

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 procedure used in calculating the weighted average cost per trip is presented as follows:

1. Multiply the value of the cost per trip by the number of bridges of certain span length to get the cost per trip via all certain span length bridges.
2. Multiply the value of the cost per trip by the number of main spans to get the cost per trip via all certain span length.
3. Multiply the value of the cost per trip by the number of bridges of certain span length by the number of main spans to get the total cost via all certain span length bridges.
4. Multiply the values of the number of bridges and number of main spans.
5. Sum the values of the number of bridges, number of main spans, and the value of step 4.
6. Sum the values obtained from step 1, step 2, and step 3.
7. Divide results obtained from step 6 by the values obtained from 4 and 5, respectively, to find the weighted average cost per trip.

8.3 Long Term Effects on Simple Span Bridges

8.3.1 Long Term Effects of FHWA 3S2 Truck Loads on Simple Span Bridges

The long term effects of FHWA 3S2 trucks on simple span Louisiana state bridges were calculated based on flexural analyses performed in chapter IV. The span for most of these bridges was 20 ft. and the controlling factor was the high ratio of flexural moments. This study was originally performed in Louisiana state project No. 736-99-1299, "Effects of Hauling Timber, Lignite Coal, and Coke Fuel on Louisiana Highways and Bridges." The bridge data was collected on the state bridges located on the routes mostly traveled by heavy trucks carrying those products. The results are presented in Table 8.1. The estimated fatigue cost per trip is $5.45.

Table 8.1 Fatigue Cost Based on $90psf and FHWA 3S2 Truck Load for Simply Supported Bridges with Design Load HS20-44

FHWA 3S2/HS20-44								
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Total Length * # of Bridges	**Ratio from Flexure Analysis**	% of Life	Cost per Trip (Dollars)	Cost per Trip * # of Bridges * Total Length
20 ft or shorter	419	630	8380	90220	**1.22**	0.0000039	$89	$190,339
25 ft	100	30	2500	5675	**1.23**	0.0000041	$28	$15,885
30 ft	3	1	90	90	**1.17**	0.0000035	$1	$77
35 ft	18	7	630	945	**1.12**	0.0000031	$5	$1,157
40 ft	57	14	2280	3720	**1.07**	0.0000027	$16	$12,181
46 ft	86	15	3956	5566	**1.02**	0.0000024	$25	$19,974
50 ft	79	16	3950	6400	**1.00**	0.0000022	$24	$17,265
56 ft	33	3	1848	1848	**0.98**	0.0000000	$0	$0
60 ft	51	12	3060	5220	**1.03**	0.0000024	$20	$17,893
66 ft	20	4	1320	1782	**1.08**	0.0000027	$10	$7,836
70 ft	20	17	1400	4760	**1.11**	0.0000030	$11	$11,541
75 ft	15	7	1125	2400	**1.14**	0.0000032	$10	$7,341
80 ft	11	2	880	880	**1.16**	0.0000035	$8	$3,638
85 ft	43	5	3655	3655	**1.19**	0.0000037	$36	$34,466
90 ft	12	5	1080	1710	**1.21**	0.0000038	$11	$6,307
95 ft	12	4	1140	1520	**1.22**	0.0000040	$12	$8,022
100 ft	53	6	5300	5300	**1.24**	0.0000042	$60	$77,562
105 ft	5	4	525	1050	**1.25**	0.0000043	$6	$3,336
110 ft	8	1	880	880	**1.27**	0.0000044	$11	$9,285
115 ft	4	1	460	460	**1.28**	0.0000046	$6	$2,605
120 ft	45	1	5400	5400	**1.29**	0.0000047	$68	$367,746
Sum				**149481**				**$814,457**
weighted average cost per trip								**$5.45**

8.3.2 Long Term Effects of FHWA 3S3 Truck Loads on Simple Span Bridges

The long term effects of FHWA 3S3 trucks on simple span Louisiana state bridges were calculated based on flexural analyses performed in chapter IV. Similar to the 3S2 truck loads, the controlling factor was the high ratio of flexural moments. This study was originally performed in Louisiana state project No. 736-99-1133, "Monitoring System to Determine the Impact of Sugarcane Truckloads on Non-Interstate Bridges." The bridge data was collected on the state bridges located on the routes mostly traveled by heavy trucks carrying those products. In this part of study, compared with the FHWA 3S3 truck with GVW 120 kips, an alternative type of FHWA 3S3 truck with GVW 100 kips, uniformly distributed tandem and tridem loads, was used, as shown in Fig. 8.1. The corresponding ratio of flexural moments was computed under this purpose. The results are presented in Table 8.2 and Table 8.3. The estimated fatigue costs per trip for those two types of FHWA 3S3 trucks are $11.75 and $0.90, respectively.

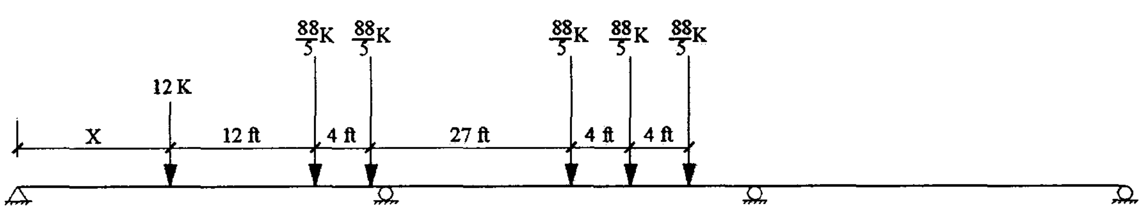


Fig. 8.1 FHWA 3S3 Truck Configuration with GVW=100 Kips, Uniformly Distributed Tandem and Tridem Loads

Table 8.2 Fatigue Cost Based on $90psf and FHWA 3S3 Truck with GVW 120 Kips for Simply Supported Bridges with Design Load HS20-44

FHWA 3S3 (GVW = 120 Kips)/HS20-44								
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Total Length * # of Bridges	**Ratio from Flexure Analysis**	% of Life (E-6)	Cost per Trip (Dollars)	Cost per Trip * # of Bridges * Total Length
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								**$11.75**

Table 8.3 Fatigue Cost Based on $90psf and FHWA 3S3 Truck with GVW 100 Kips for Simply Supported Bridges with Design Load HS20-44

FHWA 3S3 (GVW = 100 Kips)/HS20-44								
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Total Length * # of Bridges	**Ratio from Flexure Analysis**	% of Life (E-6)	Cost per Trip (Dollars)	Cost per Trip * # of Bridges * Total Length
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								**$0.90**

8.4 Long Term Effects on Continuous Span Bridges

As mentioned before, studies performed in this chapter were based on two Louisiana state projects No. 736-99-1299 and No. 736-99-1133. In project 736-99-1133, the number of continuous bridges was very limited. Therefore, only FHWA 3S2 truck load was used to evaluate the long term effects on continuous bridges. The long term effects of FHWA 3S2 trucks on continuous Louisiana state bridges were also calculated based on flexural analyses performed in chapter IV. The controlling factor was the high ratio of flexural moments. This study was also originally performed in Louisiana state project No. 736-99-1299, "Effects of Hauling Timber, Lignite Coal, and Coke Fuel on Louisiana Highways and Bridges." The bridge data was collected on the state bridges located on the routes mostly traveled by heavy trucks carrying those products. The results are presented in Table 8.4. The estimated fatigue cost per trip is $8.86.

Table 8.4 Fatigue Cost Based on $90psf and FHWA 3S2 for Continuous Bridges with Design Load HS20-44

FHWA 3S2/HS20-44												
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Average Length of each Bridge	Ratio from Flexure Analysis		% of Life		Cost per Trip		Cost per Trip * Total Length	
					Positive Moment	**Negative Moment**	Positive Moment	Negative Moment	Positive Moment	Negative Moment	Positive Moment	Negative Moment
20 or Shorter	45	3	900	300	**1.28**	**0.98**	4.54E-06	2.04E-06	$4	$0	$3,313	$0
45 ft	19	2	740	370	**1.05**	**1.56**	2.52E-06	8.25E-06	$3	$8	$1,861	$6,096
50 ft	150	14	7511	536.5	**1.02**	**1.48**	2.33E-06	7.13E-06	$3	$10	$25,310	$77,608
55 ft	3	1	166	166	**1**	**1.41**	2.20E-06	6.17E-06	$1	$3	$163	$459
60 ft	14	4	708	177	**1.02**	**1.35**	2.35E-06	5.34E-06	$1	$3	$796	$1,806
65 ft	50	6	3300	550	**1.07**	**1.28**	2.69E-06	4.65E-06	$4	$7	$13,188	$22,780
70 ft	126	15	8030	535.33	**1.1**	**1.23**	2.92E-06	4.06E-06	$4	$6	$33,893	$47,176
75 ft	120	10	4765	476.5	**1.13**	**1.24**	3.13E-06	4.13E-06	$4	$5	$19,195	$25,312
80 ft	11	2	730	365	**1.15**	**1.27**	3.32E-06	4.46E-06	$3	$4	$2,392	$3,212
85 ft	16	5	1198	239.6	**1.17**	**1.29**	3.50E-06	4.75E-06	$2	$3	$2,716	$3,685
90 ft	85	18	6728	373.78	**1.19**	**1.32**	3.67E-06	5.01E-06	$4	$5	$24,919	$33,994
95 ft	36	3	3044	1014.67	**1.2**	**1.34**	3.82E-06	5.23E-06	$10	$14	$31,893	$43,575
100 ft	102	13	9251	711.62	**1.22**	**1.35**	3.97E-06	5.42E-06	$8	$10	$70,521	$96,282
105 ft	83	20	8036	401.8	**1.23**	**1.4**	4.10E-06	6.08E-06	$4	$7	$35,749	$52,972
110 ft	19	2	1711	855.5	**1.24**	**1.38**	4.22E-06	5.73E-06	$10	$13	$16,696	$22,660
120 ft	40	2	3570	1785	**1.27**	**1.4**	4.45E-06	5.98E-06	$21	$29	$76,504	$102,938
125 ft	19	4	1558	389.5	**1.28**	**1.41**	4.55E-06	6.09E-06	$5	$6	$7,449	$9,974
130 ft	8	2	864	432	**1.28**	**1.41**	4.64E-06	6.18E-06	$5	$7	$4,676	$6,229
Sum			**62,810**								**$371,231**	**$556,759**
	Weighted Average Cost per Trip										**$5.91**	**$8.86**

8.5 Summary

The long term economic effects of heavy truck loads on Louisiana bridges were evaluated in this chapter. The methodology of estimating the weighted average cost for truck traveling on the bridges was developed. For simple span and continuous span bridges on the routes of the timber, lignite coal, and coke fuel industry, the estimated costs for FHWA 3S2 trucks crossing selected bridges were determined. For simple span bridges on the routes of the sugarcane industry, the estimated costs for two different load configurations of FHWA 3S3 trucks crossing selected bridges were determined and compared. The truck configuration FHWA 3S3 with uniformly distributed tandem and tridem loads is recommended to be used with GVW of 100,000 lb. This configuration will result in the least fatigue cost on the network.

CHAPTER IX

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

9.1 Summary

The rapid growth of the economy has led to a rapid growth in the number of heavy vehicles in service, as well as a dramatic increase in the size and weight of heavy vehicles. The tug-of-war between the demand of increasing the truck weight to get more carrying capacity and reducing the risk and rehabilitation costs of the bridges existed for a long time. Therefore, evaluating the bridge characteristics under heavy truck loads is necessary and important.

The objectives of this research were to determine the effects of heavy truck loads on simple span and continuous bridges; to study the detailed effects on bridge components, including girders, decks and diaphragms and to investigate the economic impact when higher truck loads are applied to existing bridges. In pursuit of these objectives, the effects of heavy truck loads on bridges were investigated through AASHTO linear approach, finite element analysis, and some statistical analysis. The bridge parameters that were considered in this study include support condition, girder type, girder spacing, span length, and truck load applied on the bridge model. The bridge width and slab thickness remained constant at 30 ft. and 8 in., respectively.

The evaluation of bridge girders were based on two different methods. When using AASHTO linear approach, simply supported bridges with span length from 20 ft to 120 ft and continuous bridges with span length from 20 ft to 130 ft were evaluated under FHWA 3S2 truck load. While finite element analysis approach was used, medium span length simply supported bridges and continuous bridges with span length from 20 ft to 105 ft were investigated under FHWA 3S3 truck load.

Chapter V includes the study of bridge deck performance under the heavy truck loads. Short term and long term effects of FHWA 3S3 truck load on simple span bridges were determined in section 5.3, while long term effects of FHWA 3S2 and 3S3 truck load on continuous span bridges were determined in sections 5.2 and 5.3. As the truck load increased, the short term or long term effects of heavy truck load on bridge decks cannot be neglected. In most cases the bridge decks are overstressed when 3S2 or 3S3 trucks traveled on them.

To reduce the heavy work load and investigate the effects of different bridge parameters on bridge deck stress performance, the statistical analysis was conducted in Chapter VI. Simple span bridge decks were used as the analysis sample. A modified factorial experiment with crossed treatment factors was created to perform the probability based statistical analysis due to the determinacy of results from finite element models. The sequence of significance of analysis parameters was observed. Effects of bridge girder types on deck stress performances were discovered under different bridge geometric and truck load configurations.

The short term and long term effects of FHWA 3S3 truck loads on simple span bridge diaphragms which were designed based on HS20-44 truck loads were evaluated in

Chapter VII. The effects were determined limited although the long term effects on the diaphragms might be more critical than the short term effects.

The economic impact of heavy truck loads on remaining safe life of bridges is very important and needs to be investigated since it has more practical meanings. The methodology of estimating the weighted average cost for a truck traveling on the bridges was developed. The estimated costs for simple span and continuous span bridges on the routes of the timber, lignite coal, and coke fuel industries with FHWA 3S2 truck loads were determined. The estimated costs for simple span bridges on the route of the sugarcane industry with two different load configurations of FHWA 3S3 truck load were determined and compared in Chapter VIII.

The original contributions of this research can be summarized as follows: 1) The effects of FHWA 3S2 and 3S3 truck configuration that used to haul timber and sugarcane products on bridge components were determined. 2) Statistical analyses for the evaluations of simple span bridge decks under FHWA 3S2 truck load and standard design truck load were conducted. 3) The fiscal impacts of heavy trucks hauling timber and sugarcane products on Louisiana non-interstate bridges were obtained and gave recommendations of truck configurations used to haul those agricultural products were made.

9.2 Conclusions

From research performed in previous chapters, the following conclusions are drawn:

9.2.1 <u>Bridge Girders</u>

<u>Evaluation Based on AASHTO Linear Approach</u> Simply supported bridges with span length from 20 ft. to 120 ft., and continuous bridges with span length from 20 ft. to 130 ft. were analyzed by simplified AASHTO line girder analysis approach in section 4.2. All bridges with moment or shear ratios greater than 1.1 would be considered overstressed. Results from simplified AASHTO line girder analysis approach indicated

- For simple span bridges, bridges with span length 40 ft. to 50 ft. did not exceed this limit; bridges with span length 20 ft. to 40 ft. and 50 ft. to 120 ft. exceeded this limit and were overstressed.
- For continuous span bridges, the ratios of positive moments of bridges with span length 20 ft. and 75 ft. to 120 ft. exceeded the criteria; ratios of negative moments higher than the criteria for bridges with span length 40 ft. to 130 ft.; while the ratios of shear forces of bridges with span length 55 ft. to 130 ft. exceeded the criteria. Those bridges with the higher ratios may have increased chances of cracks on systems.

<u>Evaluation Based on Finite Element Approach</u> Medium span length simply supported bridges and continuous bridges with span length from 20 ft. to 105 ft. were analyzed by finite element analysis approach in section 4.3. The criteria or overstressing was if the ratios of tensile or compressive stresses at top or bottom surface of the bridge girders were greater than 1.1.

Based on analysis results described in section 4.3, for simple span bridges, the conclusions can be drawn as follow:

- When the models contained five girders and had the girder spacing eight ft., as the truck load increased, the short-term effects of heavy truck load were limited; the long-term effects of heavy truck load were significant.
- When the models contained seven girders and had the girder spacing five ft., as the truck load increased, the short-term effects of heavy truck load were also limited; the long-term effects of heavy truck load were also significant.
- For the short-term effects of heavy truck loads, bridges with narrower girder spacing had a better capacity of resisting the impact; while for the long-term effects of heavy truck loads, bridges with narrower girder spacing had a better but limited capacity for impact resistance.

For those bridge models with three equal span length, simply supported girders, and continuous bridge decks on girders, conclusions for those models are given below:

- For those models containing five girders and with the girder spacing eight ft. and truck loads placed on the locations that would result maximum negative moment on bridge systems, as the truck load increased:
  1) The short term effects on compressive stresses at the top of bridge girders were limited. The effects could be reduced to minimum when the bridge span lengths ranged from 30 ft. to 65 ft.
  2) The short term effects on tensile stresses at the bottom of bridge girders were limited when the when the bridge span lengths ranged from 30 ft. to 105 ft. When the bridge span length varied from 20 ft. to 30 ft, the effects of higher truck loads could not be neglected.

3) The long term effects on compressive and tensile stresses were significant, no matter at the top or bottom of bridge girders. In a long time period, those bridge girders may experience compressive and tensile cracks and require additional inspections along with early and frequent maintenance.

- For those models containing five girders and with the girder spacing eight ft. and truck loads placed on the locations that would result in maximum positive moment on bridge systems, as the truck load increased:

1) The short term effects on compressive stresses at the top of bridge girders were limited when the when the bridge span lengths ranged from 35 ft. to 105 ft.. When the bridge span length varied from 20 ft. to 35 ft, the effects of higher truck loads could not be neglected.

2) The short term effects on tensile stresses at the bottom of bridge girders were limited when the when the bridge span lengths ranged from 40 ft. to 105 ft. When the bridge span length varied from 20 ft. to 40 ft, the effects of higher truck loads could not be neglected.

3) The long term effects on compressive and tensile stresses were significant, no matter at the top or bottom of bridge girders. In a long time period, those bridge girders may experience compressive and tensile cracks and require additional inspections along with early and frequent maintenance.

- For those models containing seven girders and with the girder spacing five ft. and truck loads were placed on the locations that would result in maximum negative moment on bridge systems, as the truck load increased:

1) The short term effects on compressive stresses at the top of bridge girders were limited when the when the bridge span lengths ranged from 30 ft. to 105 ft. When the bridge span length varied from 20 ft. to 30 ft, the effects of higher truck loads could not be neglected.
2) The short term effects on tensile stresses at the bottom of bridge girders were limited when the when the bridge span lengths ranged from 30 ft. to 105 ft. When the bridge span length varied from 20 ft. to 30 ft, the effects of higher truck loads could not be neglected.
3) The long term effects on compressive and tensile stresses were significant, no matter at the top or bottom of bridge girders. In a long time period, those bridge girders may experience compressive and tensile cracks and require additional inspections along with early and frequent maintenance.

- For those models containing seven girders and with the girder spacing five ft. and truck loads were placed on the locations that would result in maximum positive moment on bridge systems, as the truck load increased:

1) The short term effects on compressive stresses at the top of bridge girders were limited when the when the bridge span lengths ranged from 35 ft. to 105 ft.. When the bridge span length varied from 20 ft. to 35 ft, the effects of higher truck loads could not be neglected.
2) The short term effects on tensile stresses at the bottom of bridge girders were limited when the when the bridge span lengths ranged from 35 ft. to 105 ft. When the bridge span length varied from 20 ft. to 35 ft., the effects of higher truck loads could not be neglected.

3) The long term effects on compressive and tensile stresses were significant, no matter at the top or bottom of bridge girders. In a long time period, those bridge girders may experience compressive and tensile cracks and require additional inspections along with early and frequent maintenance.

- In general, the short term effects of heavy truck loads on continuous bridge girders with span length 40 ft. to 105 ft. were limited; bridge girders with span length shorter than 40 ft. might have more chances of cracking and need more and frequent inspections. The long term effects of heavy truck loads on continuous bridge girders with all evaluated span length were significant and cannot be neglected. Bridges with narrower spacing had better but very limited capacity of resisting the higher truck loads' impacts.
- Compared with finite element analysis, results from simplified AASHTO linear analyses methods are conservative for typical multi-girder bridges. Since linear girder analyses methods are relatively simple and conservative, they can be used as a preliminary tool. If the estimated performance of bridges did not meet the requirement, then a more detailed method, such as finite element analysis method, could be used.

9.2.2 Bridge Decks

Short term and long term effects of FHWA 3S3 truck load on simple span bridges were determined in section 5.3, while long term effects of FHWA 3S2 and 3S3 truck load on continuous span bridges were evaluated in sections 5.2 and 5.3. Conclusions related to those models are given as follows:

- While FHWA 3S2 truck load was put on the continuous span bridge models with three equal span lengths:
    1) For span length ranging from 20 ft. to 30 ft., the long term effects on longitudinal and transverse directions were limited, but effects on shear stress were significant. Those short span continuous bridges might meet more shear cracks.
    2) For span length ranging from 30 ft. to 60 ft., the long term effects on transverse stress and shear stress were limited, but effects on longitudinal stress were significant. Continuous bridges with those span lengths might meet more cracks in longitudinal direction.
    3) For span length ranging from 60 ft. to 120 ft, the long term effects on all three directions were significant. Those continuous bridges might meet more cracks in all three directions.
- While FHWA 3S3 truck load was put on the simple span bridge models with span length 90 ft. and built by AASHTO Bulb-Tee 54, 63 and 72 girders:
    1) The short term effects on longitudinal and transverse stresses were not as significant as the effect on shear stress. The bridge deck had more chances to experience shear cracks and require additional inspections along with early and frequent maintenance. The sequence of bridge deck performance were bridges constructed with AASHTO Bulb-Tee 72 girders, Bulb-Tee 63 girders, and Bulb-Tee 54 girders, from better to worse, respectively.
    2) The long term effects on longitudinal stresses were limited. The long term effects on transverse and shear stresses were significant and might cause

more cracks. Such cracks would require additional inspections along with early and frequent maintenance.

- While FHWA 3S3 truck load was put on the continuous span bridge models with three equal span lengths:

  1) For span length of 20 ft, the long term effects on longitudinal and transverse directions were limited, but effects on shear stress were significant. Continuous bridges with span lengths of 20 ft. might have more shear cracks.
  2) For span length of 30 ft, the long term effect on transverse direction was limited, but effects on other two directions were significant. Continuous bridges with this span length had a higher chance to experience cracks in longitudinal and shear directions.
  3) For span length ranging from 60 ft. to 105 ft, the long term effects on all three directions were significant. Those continuous bridges might meet more cracks in all three directions, which might need additional inspections along with early and frequent maintenance.

9.2.3 Statistic Analysis of Bridge Deck Data

Factorial experiment design was used to construct the statistical model and corresponding analyses were performed in chapter VI. Conclusions related to this model are given below:

- Evaluations based on the treatment factor combinations (Girder Type, Girder Number) and (Girder Type, Truck Type). Results indicated that generally bridges built with AASHTO Type VI girders had the best performance on deck stress behavior, while bridges built with AASHTO Type IV or Bulb-Tee 54 girders had the worst performance on deck stress behavior. Detailed results are listed in Table 9.1 to Table 9.4. In the tables the symbols ">" and "=" represented the same meaning as in chapter VI.

Table 9.1 Statistical Analyses Results – Treatment Factor GT and GN, Girder Spacing Eight ft.

Stress Type	Comparison Results
Longitudinal	Type VI > BT-72 > BT-63 > Type V > BT-54 > Type IV
Transverse	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Shear	Type VI > Type V > BT-72 > BT-63 > BT-54 = Type IV

Table 9.2 Statistical Analyses Results – Treatment Factor GT and GN, Girder Spacing five ft.

Stress Type	Comparison Results
Longitudinal	Type VI > BT-72 > Type V > Type IV > BT-63 > BT-54
Transverse	Type VI > BT-72 > Type V = BT-63 > BT-54 > Type IV
Shear	Type VI > Type V > BT-72 > BT-63 = BT-54 = Type IV

Table 9.3 Statistical Analyses Results – Treatment Factor GT and TT, Truck Load Type HS20-44

Stress Type	Comparison Results
Longitudinal	Type VI > Type V > BT-72 > BT-63 > BT-54 = Type IV
Transverse	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV
Shear	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV

Table 9.4 Statistical Analyses Results – Treatment Factor GT and TT, Truck Load Type FHWA 3S3

Stress Type	Comparison Results
Longitudinal	Type VI > BT-72 > Type V > Type IV > BT-63 > BT-54
Transverse	Type VI > BT-72 = Type V > BT-63 > BT-54 > Type IV
Shear	Type VI > Type V > BT-72 > BT-63 > BT-54 > Type IV

- Generally ANOVA was used for the purpose of determining the importance of treatment factors. The conclusion was that among three treatment factors, girder number had the most significance. Detailed results were listed in Table 9.5.

Table 9.5 Statistical Analyses Results for ANOVA

Stress Type	Comparison Results
Longitudinal	GN > GT > TT
Transverse	GN > GT > TT
Shear	GN > TT > GT

9.2.4 Bridge Diaphragms

- The short term and long term effects of FHWA 3S3 truck loads on simple span bridge diaphragms which were designed based on HS20-44 truck loads were evaluated in chapter 7.2. The conclusions were drawn as follows:

1) The long term effects on diaphragms might be more critical than the short term effects.
2) The results indicated that even ratios of axial forces might be beyond the criteria, the effects on diaphragms were limited because the axial forces in diaphragms were limited.

9.2.5 Bridge Costs

For simple span and continuous span bridges on the routes of the timber, lignite coal, and coke fuel industries, the estimated cost for FHWA 3S2 truck cross selected bridges were determined. While for simple span bridges on the routes of the sugarcane industry, the estimated cost for two different load configurations of FHWA 3S3 truck cross selected bridges were determined and compared. Evaluation results indicated the key findings below:

- If increasing the truck load into FHWA 3S2 with GVW 108,000 lb, bridge fatigue costs for simple span bridges on the route of timber, lignite coal, and coke fuel industries is $5.45 per truck per bridge.
- If increasing the truck load into FHWA 3S2 with GVW 108,000 lb, bridge fatigue costs for continuous span bridges on the route of timber, lignite coal, and coke fuel industries is $8.86 per truck per bridge.
- If increasing the truck load into FHWA 3S3 with GVW 120,000 lb, bridge fatigue costs for simple span bridges on the routes of the sugarcane industry is $11.75 per truck per bridge.
- If increasing the truck load into FHWA 3S3 with GVW 100,000 lb, where the 3S3 truck load is uniformly distributed and the steering axle is 12,000 lb, bridge fatigue costs for simple span bridges on the routes of the sugarcane industry is $0.90 per truck per bridge.

9.3 Recommendations

- AASHTO linear girder analysis methods are relatively simple and conservative; therefore, they can be used as a preliminary tool. It is recommended using finite element analysis approach to get the more accurate and detailed results.
- Based on results of the finite analysis method, for medium span length simply supported bridges, which are often traveled by FHWA 3S3 truck configurations, it is recommended to have additional inspections along with early and frequent maintenance because the long term effects of heavy truck loads on those bridges cannot be neglected.

- For continuous span bridges with girder spacing eight ft. and often traveled by FHWA 3S2 truck configurations, it is recommended to have the bridge span length varied from 40 ft. to 105 ft. to better resist the short term impact on bridge girders.
- For continuous span bridges with girder spacing five ft. and often traveled by FHWA 3S2 truck configuration, it is recommended to have the bridge span length varied from 35 ft. to 105 ft. to better resist the short term impact on bridge girders.
- The long term effects of heavy truck loads on simple and continuous bridges are significant. It is recommended to have additional inspections along with early and frequent maintenance on bridge girders.
- Continuous span bridges with span length varied from 20 ft. to 105 ft. were evaluated in this study. However, bridges with span lengths not in this range need to be investigated in further research.
- While truck load increased, all types of bridge decks are overstressed, and bridge decks may experience more cracks and require additional inspections along with early and frequent maintenance.
- Despite the economic considerations, it is recommended to use AASHTO Type VI girders to construct bridges for better deck performance. It is not recommended to use AASHTO Type IV or Bulb-Tee 54 girder to construct the bridges because of the poor performance
- For those routes used by the timber, lignite coal, and coke fuel industries, it is not recommended to increase the GVW on FHWA 3S2 vehicles to 108,000 lb. due to

the high fatigue cost. If this GVW increase is necessary, the vehicle's axle configuration should be modified

- It is recommended that truck configuration FHWA 3S3 be used to haul sugarcane with GVW of 100,000 lb. uniformly distributed. This configuration will result in the least fatigue cost on the network.
- It is not recommended that truck configuration FHWA 3S3 be used to haul sugar cane with GVW of 120,000 lb. This configuration will result in high fatigue cost on the network and could cause failure in bridge girders and bridge decks.
- It is recommended that further cost evaluations of different truck configurations on bridges be conducted.

ACRONYMS, ABBREVIATIONS, & SYMBOLS

3S2 = truck with three axles on tractor and a semi-trailer with two axles

3S3 = truck with three axles on tractor and a semi-trailer with three axles

AASHTO = American Association of State Highway and Transportation Officials

ADT = average daily traffic, vehicles/day

ANOVA = Analysis of variance

DOTD = Department of Transportation and Development

FHWA = Federal Highway Administration

ft = foot

GVW = gross vehicle weight

kip = 1,000 lb.

LA-DOTD = Louisiana Department of Transportation and Development

lb. = pound

LRFD = Load Resistance Factor Design

LTRC = Louisiana Transportation Research Center

psf = pounds per square foot

APPENDIX A

BRIDGE CHARACTERS CONSIDERED

IN ANALYSIS

Table A.1 Bridge Models used for Girder Analysis by AASHTO Linear Approach

Support Condition	Span Length Range	Truck Loads
Simply Supported	20 ft. ~ 120 ft.	HS20-44 & FHWA 3S2 (GVW 108 Kips)
Continuous	20 ft. ~ 130 ft.	HS20-44 & FHWA 3S2 (GVW 108 Kips)

Table A.2 Bridge Models used for Girder Analysis by Finite Element Approach

Support Condition	Span Length Range	Girder Number	Girder Type	Truck Loads
Simply Supported	90 ft. ~ 120 ft.	5 Girders or 7 Girders	AASHTO Type IV, Type V, Type VI, BT-54, BT-63 and BT - 72	HS20-44 & FHWA 3S3 (GVW 120 Kips)
Continuous	20 ft. ~ 105 ft.	5 Girders or 7 Girders	AASHTO Type IV, Type V, Type VI, BT-54, BT-63 and BT - 72	HS20-44 & FHWA 3S2 (GVW 108 Kips)

Table A.3 Bridge Models used for Bridge Deck Evaluation

Support Condition	Span Length Range	Girder Number	Girder Type	Truck Loads
Simply Supported	90 ft.	5 Girders	AASHTO BT-54, BT-63 and BT - 72	HS20-44 & FHWA 3S3 (GVW 120 Kips)
Continuous	20 ft. ~ 105 ft.	5 Girders	AASHTO Type IV	HS20-44 & FHWA 3S3 (GVW 120 Kips)
Continuous	20 ft. ~ 120 ft.	5 Girders	AASHTO Type IV	HS20-44 & FHWA 3S2 (GVW 108 Kips)

Table A.4 Bridge Models used for Statistic Analysis

Girder Type	# of Girders	Span Length	# of Spans	Support Condition	Applied Truck Load
AASHTO Type IV	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO Type V	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO Type VI	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO BT-54	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO BT-63	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO BT-72	5	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO Type IV	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO Type V	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO Type VI	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO BT-54	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO BT-63	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)
AASHTO BT-72	7	90 ft	1	Simply Supported	HS20-44 & FHWA 3S3 (GVW 120 Kips)

Table A.5 Bridge Models used for Bridge Diaphragm Evaluation

Support Condition	Span Length Range	Girder Number	Girder Type	Truck Loads
Simply Supported	90 ft. ~ 120 ft.	5 Girders or 7 Girders	AASHTO Type IV, Type V, Type VI, BT-54, BT-63 and BT - 72	HS20-44 & FHWA 3S3 (GVW 120 Kips)

Table A.6 Bridge Models used for Cost Study

Support Condition	**Span Length Range**	**Truck Loads**
Simply Supported	20 ft. ~ 120 ft.	HS20-44 & FHWA 3S2 (GVW 108 Kips)
Simply Supported	20 ft. ~ 95 ft.	HS20-44 & FHWA 3S3 (GVW 120 Kips)
Simply Supported	20 ft. ~ 95 ft.	HS20-44 & FHWA 3S3 (GVW 100 Kips, Uniformly Distributed)
Continuous	20 ft. ~ 130 ft.	HS20-44 & FHWA 3S2 (GVW 108 Kips)

APPENDIX B

TYPICAL GTSTRUDL INPUT FILES

B.1 GTSTRUDL Input File of Simple Span Bridge with Truck Load FHWA 3S3 and AASHTO Bulb-Tee 54 Girder

```
STRUDL 'Model 326' 'BT-54 GIRDER, 0 BRIDGE SKEW, GIRDER SPACING 8 ft, SPAN LENGTH 90
ft, 0 DIAPHRAGM SKEW'
UNITS inch
PRINT GENERATE OFF

$==========================================================================
$ Generate Joints to Connect Girder Brick Elements
$==========================================================================
GENERATE 4 JOINTS ID 1000001 1 X LIST 8 18 24 34 Y 0 Z 0
REPEAT 1 ID 4 Y 8.5
REPEAT 1 ID 100 Z -12

GENERATE 2 JOINTS ID 1000009 1 X LIST 18 24 Y 19.5 Z 0
REPEAT 2 ID 2 Y 10
REPEAT 1 ID 100 Z -12

GENERATE 4 JOINTS ID 1000015 1 X LIST 0 18 24 42 Y 49.5 Z 0
REPEAT 1 ID 4 Y 4.5
REPEAT 1 ID 100 Z -12

$==========================================================================
$ Generate Girder Brick Elements
$==========================================================================
TYPE TRIDEMINSIONAL
GENERATE 3 ELEMENTS ID 'G-100011' 1 FROM 1000001 1 TO 1000002 1 TO 1000006 1 TO
1000005 1 TO 1000101 1 TO 1000102 1 TO 1000106 1 TO 1000105 1
GENERATE 1 ELEMENTS ID 'G-100014' 0 FROM 1000006 0 TO 1000007 0 TO 1000010 0 TO
1000009 0 TO 1000106 0 TO 1000107 0 TO 1000110 0 TO 1000109 0
GENERATE 2 ELEMENTS ID 'G-100015' 1 FROM 1000009 2 TO 1000010 2 TO 1000012 2 TO
1000011 2 TO 1000109 2 TO 1000110 2 TO 1000112 2 TO 1000111 2
GENERATE 1 ELEMENTS ID 'G-100017' 1 FROM 1000013 0 TO 1000014 0 TO 1000017 0 TO
1000016 0 TO 1000113 0 TO 1000114 0 TO 1000117 0 TO 1000116 0
GENERATE 3 ELEMENTS ID 'G-100018' 1 FROM 1000015 1 TO 1000016 1 TO 1000020 1 TO
1000019 1 TO 1000115 1 TO 1000116 1 TO 1000120 1 TO 1000119 1

$==========================================================================
$ Copy Generated Girder Section and Copy it Down Entire Length of Bridge
$==========================================================================
DEFINE OBJECT 'SECTION' JOINTS 1000001 TO 1000022 1000101 TO 1000122, ELEMENTS 'G-
100011' TO 'G-100020'
COPY OBJECT 'SECTION' REPEAT 89 TIMES JOINT INCR 100 ELEMENT INCR 10 TRANSLATE Z
-12

$==========================================================================
$ Copy Generated Girder and Copy it Across The Entire Width of Bridge
$==========================================================================
DEFINE OBJECT 'GIRDER A' JOINTS EXISTING 1000001 TO 1009022, ELEMENTS EXISTING 'G-
100011' TO 'G-100910'
COPY OBJECT 'GIRDER A' REPEAT 1 TIMES JOINT INCR 1000000 ELEMENT INCR 100000
TRANSLATE X 63
```

```
DEFINE OBJECT 'GIRDER B' JOINTS EXISTING 2000001 TO 2009022, ELEMENTS EXISTING 'G-
200011' TO 'G-200910'
COPY OBJECT 'GIRDER B' REPEAT 2 TIMES JOINT INCR 1000000 ELEMENT INCR 100000
TRANSLATE X 96
DEFINE OBJECT 'GIRDER C' JOINTS EXISTING 4000001 TO 4009022, ELEMENTS EXISTING 'G-
400011' TO 'G-400910'
COPY OBJECT 'GIRDER C' REPEAT 1 TIMES JOINT INCR 1000000 ELEMENT INCR 100000
TRANSLATE X 63

$=======================================================================
$ Generate Joints to Connect Deck Plate Elements
$=======================================================================
GENERATE 2 JOINTS ID 1000023 1 X LIST 0 42 Y 54.01 Z 0
REPEAT 30 ID 300 Z -36

GENERATE 2 JOINTS ID 2000023 1 X LIST 63 105 Y 54.01 Z 0
REPEAT 30 ID 300 Z -36

GENERATE 2 JOINTS ID 3000023 1 X LIST 159 201 Y 54.01 Z 0
REPEAT 30 ID 300 Z -36

GENERATE 2 JOINTS ID 4000023 1 X LIST 255 297 Y 54.01 Z 0
REPEAT 30 ID 300 Z -36

GENERATE 2 JOINTS ID 5000023 1 X LIST 318 360 Y 54.01 Z 0
REPEAT 30 ID 300 Z -36

GENERATE 2 JOINTS ID 2000025 1000000 X LIST 132 228 Y 54.01 0 Z 0 0
REPEAT 30 ID 300 Z -36

$=======================================================================
$ Generate Deck Plate Elements
$=======================================================================
TYPE PLATE
GENERATE 5 ELEMENTS ID 'P-100011' 100000 FROM 1000023 1000000 TO 1000024 1000000 TO
1000324 1000000 TO 1000323 1000000
REPEAT 29 ID 10 FROM INCR 300 TO INCR 300

GENERATE 1 ELEMENTS ID 'P-100012' 0 FROM 1000024 0 TO 2000023 0 TO 2000323 0 TO 1000324
0
REPEAT 29 ID 10 FROM INCR 300 TO INCR 300

GENERATE 2 ELEMENTS ID 'P-200012' 100000 FROM 2000024 1000000 TO 2000025 1000000 TO
2000325 1000000 TO 2000324 1000000
REPEAT 29 ID 10 FROM INCR 300 TO INCR 300

GENERATE 2 ELEMENTS ID 'P-200013' 100000 FROM 2000025 1000000 TO 3000023 1000000 TO
3000323 1000000 TO 2000325 1000000
REPEAT 29 ID 10 FROM INCR 300 TO INCR 300

GENERATE 1 ELEMENTS ID 'P-400012' 0 FROM 4000024 0 TO 5000023 0 TO 5000323 0 TO 4000324
0
REPEAT 29 ID 10 FROM INCR 300 TO INCR 300
```

```
$===========================================================================
$ Generate Rigid Members to Connect Girder Elements to Plate Elements
$===========================================================================
TYPE SPACE TRUSS
GENERATE 2 MEMBERS ID 'R-10001' 1 FROM 1000019 3 TO 1000023 1
REPEAT 4 ID 10000 FROM INCR 1000000 TO INCR 1000000
REPEAT 30 ID 10 FROM INCR 300 TO INCR 300

$===========================================================================
$ Generate Diaphragm Members at Top of Bottom Flange
$===========================================================================
$ END DIAPHRAGMS
GENERATE 4 MEMBERS ID 'B-1011' 1 FROM 1000008 1000000 TO 2000005 1000000
GENERATE 4 MEMBERS ID 'B-1021' 1 FROM 1000018 1000000 TO 2000015 1000000
GENERATE 4 MEMBERS ID 'B-1031' 1 FROM 1009008 1000000 TO 2009005 1000000
GENERATE 4 MEMBERS ID 'B-1041' 1 FROM 1009018 1000000 TO 2009015 1000000
GENERATE 4 MEMBERS ID 'B-1051' 1 FROM 1004008 1000000 TO 2004005 1000000
GENERATE 4 MEMBERS ID 'B-1061' 1 FROM 1004018 1000000 TO 2004015 1000000
GENERATE 4 MEMBERS ID 'B-1071' 1 FROM 1005008 1000000 TO 2005005 1000000
GENERATE 4 MEMBERS ID 'B-1081' 1 FROM 1005018 1000000 TO 2005015 1000000

$===========================================================================
$ Define Supports
$===========================================================================
STATUS SUPPORT JOINTS -
 1000001 TO 5000001 BY 1000000 1000002 TO 5000002 BY 1000000 1000003 TO 5000003 BY
1000000 1000004 TO 5000004 BY 1000000 -
 1009001 TO 5009001 BY 1000000 1009002 TO 5009002 BY 1000000 1009003 TO 5009003 BY
1000000 1009004 TO 5009004 BY 1000000

$===========================================================================
$ Set Boundary Conditions
$===========================================================================
JOINT RELEASES

$ GIRDER BASE
$  END PIN CONDITIONS
 1000001 TO 1000004 2000001 TO 2000004 3000001 TO 3000004 4000001 TO 4000004 5000001 TO
5000004 MOMENT X Y
 1009001 TO 1009004 2009001 TO 2009004 3009001 TO 3009004 4009001 TO 4009004 5009001 TO
5009004 MOMENT X Y

$===========================================================================
$ Define Element Properties
$===========================================================================
MATERIAL CONCRETE ALL ELEMENTS
ELEMENT PROPERTIES
 EXISTING 'G-100011' TO 'G-100910' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'G-200011' TO 'G-200910' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'G-300011' TO 'G-300910' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'G-400011' TO 'G-400910' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'G-500011' TO 'G-500910' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'P-100011' TO 'P-100303' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-200011' TO 'P-200303' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-300011' TO 'P-300303' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-400011' TO 'P-400303' TYPE 'SBCR' THICKNESS 8
```

```
EXISTING 'P-500011' TO 'P-500303' TYPE 'SBCR' THICKNESS 8

$===========================================================================
$ Define Member Properties
$===========================================================================
MATERIAL CONCRETE ALL MEMBERS
MEMBER PROPERTIES PRISMATIC
 EXISTING 'R-10001' TO 'R-50302' AX 8

MEMBER PROPERTIES PRISMATIC
 EXISTING 'B-1011' TO 'B-1084' AX 330

$===========================================================================
$ Define Supports of Deck
$===========================================================================
STATUS SUPPORT JOINTS -
 1000023 TO 5000023 BY 1000000 1000024 TO 5000024 BY 1000000 -
 1009023 TO 5009023 BY 1000000 1009024 TO 5009024 BY 1000000

$===========================================================================
$ Set Boundary Conditions of Deck
$===========================================================================
JOINT RELEASES

$ DECK BASE
$  END CONDITIONS

1000023 TO 1000024 2000023 TO 2000024 3000023 TO 3000024 4000023 TO 4000024 5000023 TO
5000024 FORCE Y
1009023 TO 1009024 2009023 TO 2009024 3009023 TO 3009024 4009023 TO 4009024 5009023 TO
5009024 FORCE Y
1000023 TO 1000024 2000023 TO 2000024 3000023 TO 3000024 4000023 TO 4000024 5000023 TO
5000024 MOMENT X Y
1009023 TO 1009024 2009023 TO 2009024 3009023 TO 3009024 4009023 TO 4009024 5009023 TO
5009024 MOMENT X Y

$===========================================================================
$ Define Loading
$===========================================================================
UNITS LBS FT

LOADING 'DC1' 'DEAD LOAD STRUCTURAL COMPONENTS OF ELEMENTS'
 ELEMENT LOADS
  EXISTING 'G-100011' TO 'G-100910' BODY FORCES GLOBAL BY -150
  EXISTING 'G-200011' TO 'G-200910' BODY FORCES GLOBAL BY -150
  EXISTING 'G-300011' TO 'G-300910' BODY FORCES GLOBAL BY -150
  EXISTING 'G-400011' TO 'G-400910' BODY FORCES GLOBAL BY -150
  EXISTING 'G-500011' TO 'G-500910' BODY FORCES GLOBAL BY -150
  EXISTING 'P-100011' TO 'P-100303' BODY FORCES GLOBAL BY -150
  EXISTING 'P-200011' TO 'P-200303' BODY FORCES GLOBAL BY -150
  EXISTING 'P-300011' TO 'P-300303' BODY FORCES GLOBAL BY -150
  EXISTING 'P-400011' TO 'P-400303' BODY FORCES GLOBAL BY -150
  EXISTING 'P-500011' TO 'P-500303' BODY FORCES GLOBAL BY -150

DEAD LOAD 'DC2' 'DEAD LOAD STRUCTURAL COMPONENTS OF MEMBERS' DIRECTION -Y -
  MEMBERS 'B-1011' TO 'B-1014' 'B-1021' TO 'B-1024' 'B-1031' TO 'B-1034' 'B-1041' TO 'B-1044' -
```

```
'B-1051' TO 'B-1054' 'B-1061' TO 'B-1064' 'B-1071' TO 'B-1074' 'B-1081' TO 'B-1084'

LOADING 'LS' 'LIVE LOAD SURCHARGE'
 ELEMENT LOADS
  EXISTING 'P-100011' TO 'P-100303' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-200011' TO 'P-200303' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-300011' TO 'P-300303' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-400011' TO 'P-400303' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-500011' TO 'P-500303' SURFACE FORCES GLOBAL PY -31

LOADING 'LL1' 'VEHICULAR LIVE LOAD HS20-44'
 JOINT LOADS
$ Truck 1 HS20-44
  3002725 3002723 FORCE Y -4000
  3004225 3004223 FORCE Y -16000
  3005725 3005723 FORCE Y -16000

LOADING 'LL2' 'VEHICULAR LIVE LOAD HS20-44 Fatigue'
 JOINT LOADS
$ Truck 2 HS20-44 Fatigue
  3002425 3002423 FORCE Y -4000
  3003925 3003923 FORCE Y -16000
  3006925 3006923 FORCE Y -16000

LOADING 'LL3' 'VEHICULAR LIVE LOAD SugerCane'
 JOINT LOADS
$ Truck 3 SugerCane
  3002125 3002123 FORCE Y -6000
  3003325 3003323 FORCE Y -12000
  3003625 3003623 FORCE Y -12000
  3006325 3006323 FORCE Y -10000
  3006625 3006623 FORCE Y -10000
  3006925 3006923 FORCE Y -10000

LOADING 'WS' 'WIND LOAD ON STRUCTURE'
 ELEMENT LOADS
  'G-500013' TO 'G-500903' BY 10 SURFACE FORCES FACE 4 GLOBAL PX -50.13
  'G-500014' TO 'G-500904' BY 10 SURFACE FORCES FACE 4 GLOBAL PX -50.13
  'G-500015' TO 'G-500905' BY 10 SURFACE FORCES FACE 4 GLOBAL PX -50.13
  'G-500016' TO 'G-500906' BY 10 SURFACE FORCES FACE 4 GLOBAL PX -50.13
  'G-500017' TO 'G-500907' BY 10 SURFACE FORCES FACE 4 GLOBAL PX -50.13
  'G-500020' TO 'G-500910' BY 10 SURFACE FORCES FACE 4 GLOBAL PX -50.13

$==========================================================================
$ Factored Loads
$==========================================================================
LOADING COMBINATION 11 'STRENGTH I MAXIMUM - HS20-44' SPECS 'DC1' 1.25 'DC2' 1.25
'LL1' 1.75 'LS' 1.75 'WS' 0.0
LOADING COMBINATION 13 'STRENGTH I MAXIMUM - SugerCane' SPECS 'DC1' 1.25 'DC2' 1.25
'LL3' 1.75 'LS' 1.75 'WS' 0.0

LOADING COMBINATION 21 'STRENGTH III MAXIMUM - HS20-44' SPECS 'DC1' 1.25 'DC2' 1.25
'LL1' 0.0  'LS' 0.0  'WS' 1.4
LOADING COMBINATION 23 'STRENGTH III MAXIMUM - SugerCane' SPECS 'DC1' 1.25 'DC2' 1.25
'LL3' 0.0  'LS' 0.0  'WS' 1.4
```

LOADING COMBINATION 31 'STRENGTH V MAXIMUM - HS20-44' SPECS 'DC1' 1.25 'DC2' 1.25 'LL1' 1.35 'LS' 1.35 'WS' 0.4
LOADING COMBINATION 33 'STRENGTH V MAXIMUM - SugerCane' SPECS 'DC1' 1.25 'DC2' 1.25 'LL3' 1.35 'LS' 1.35 'WS' 0.4

LOADING COMBINATION 42 'FATIGUE - IMPACT 1.3 - FACTOR 0.75 - HS20-44 Fatigue' SPECS 'LL2' 0.975
LOADING COMBINATION 43 'FATIGUE - IMPACT 1.3 - FACTOR 0.75 - SugerCane' SPECS 'LL3' 0.975

$==
$ Prepare and Generate Output
$==
QUERY
STIFFNESS ANALYSIS

UNITS KIP INCH

LIST SUMMATION OF REACTIONS
LIST REACTIONS

CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-100011' TO 'G-100910'
CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-200011' TO 'G-200910'
CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-300011' TO 'G-300910'
CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-400011' TO 'G-400910'
CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-500011' TO 'G-500910'

CALCULATE AVERAGE STRESS AT TOP SURFACE FOR ELEMENTS EXISTING 'P-100011' TO 'P-500303'
CALCULATE AVERAGE STRESS AT BOTTOM SURFACE FOR ELEMENTS EXISTING 'P-100011' TO 'P-500303'

LIST FORCE MEMBERS EXISTING 'B-1011' TO 'B-1084'

B.2 GTSTRUDL Input File of Continuous Span Bridge with Truck Load FHWA 3S2 and AASHTO Bulb-Tee 54 Girder

```
STRUDL 'Model 4041' 'BT-54 GIRDER, 0 BRIDGE SKEW, GIRDER SPACING 9 ft, SPAN LENGTH
75 ft, 3S2 HS20-44'
UNITS inch
PRINT GENERATE OFF

$==========================================================================
$ Generate Joints to Connect Girder Brick Elements
$==========================================================================
GENERATE 4 JOINTS ID 1000001 1 X LIST 8 18 24 34 Y 0 Z 0
REPEAT 1 ID 4 Y 8.5
REPEAT 1 ID 100 Z -12

GENERATE 2 JOINTS ID 1000009 1 X LIST 18 24 Y 19.5 Z 0
REPEAT 2 ID 2 Y 10
REPEAT 1 ID 100 Z -12

GENERATE 4 JOINTS ID 1000015 1 X LIST 0 18 24 42 Y 49.5 Z 0
REPEAT 1 ID 4 Y 4.5
REPEAT 1 ID 100 Z -12

$==========================================================================
$ Generate Girder Brick Elements
$==========================================================================
TYPE TRIDEMINSIONAL
GENERATE 3 ELEMENTS ID 'G-100011' 1 FROM 1000001 1 TO 1000002 1 TO 1000006 1 TO
1000005 1 TO 1000101 1 TO 1000102 1 TO 1000106 1 TO 1000105 1
GENERATE 1 ELEMENTS ID 'G-100014' 0 FROM 1000006 0 TO 1000007 0 TO 1000010 0 TO
1000009 0 TO 1000106 0 TO 1000107 0 TO 1000110 0 TO 1000109 0
GENERATE 2 ELEMENTS ID 'G-100015' 1 FROM 1000009 2 TO 1000010 2 TO 1000012 2 TO
1000011 2 TO 1000109 2 TO 1000110 2 TO 1000112 2 TO 1000111 2
GENERATE 1 ELEMENTS ID 'G-100017' 1 FROM 1000013 0 TO 1000014 0 TO 1000017 0 TO
1000016 0 TO 1000113 0 TO 1000114 0 TO 1000117 0 TO 1000116 0
GENERATE 3 ELEMENTS ID 'G-100018' 1 FROM 1000015 1 TO 1000016 1 TO 1000020 1 TO
1000019 1 TO 1000115 1 TO 1000116 1 TO 1000120 1 TO 1000119 1

$==========================================================================
$ Copy Generated Girder Section and Copy it Down Entire Length of Bridge
$==========================================================================
DEFINE OBJECT 'SECTION' JOINTS 1000001 TO 1000022 1000101 TO 1000122, ELEMENTS 'G-
100011' TO 'G-100020'
COPY OBJECT 'SECTION' REPEAT 74 TIMES JOINT INCR 100 ELEMENT INCR 10 TRANSLATE Z
-12

$==========================================================================
$ Copy Generated Girder and Copy it Across The Entire Width of Bridge
$==========================================================================
DEFINE OBJECT 'GIRDER A' JOINTS EXISTING 1000001 TO 1007522, ELEMENTS EXISTING 'G-
100011' TO 'G-100760'
COPY OBJECT 'GIRDER A' REPEAT 1 TIMES JOINT INCR 1000000 ELEMENT INCR 100000
TRANSLATE X 63
```

```
DEFINE OBJECT 'GIRDER B' JOINTS EXISTING 2000001 TO 2007522, ELEMENTS EXISTING 'G-
200011' TO 'G-200760'
COPY OBJECT 'GIRDER B' REPEAT 2 TIMES JOINT INCR 1000000 ELEMENT INCR 100000
TRANSLATE X 96
DEFINE OBJECT 'GIRDER C' JOINTS EXISTING 4000001 TO 4007522, ELEMENTS EXISTING 'G-
400011' TO 'G-400760'
COPY OBJECT 'GIRDER C' REPEAT 1 TIMES JOINT INCR 1000000 ELEMENT INCR 100000
TRANSLATE X 63

$====================================================================
$ Generate Joints to Connect Deck Plate Elements
$====================================================================
GENERATE 2 JOINTS ID 1000023 1 X LIST 0 42 Y 54.01 Z 0
REPEAT 25 ID 300 Z -36

GENERATE 2 JOINTS ID 2000023 1 X LIST 63 105 Y 54.01 Z 0
REPEAT 25 ID 300 Z -36

GENERATE 2 JOINTS ID 3000023 1 X LIST 159 201 Y 54.01 Z 0
REPEAT 25 ID 300 Z -36

GENERATE 2 JOINTS ID 4000023 1 X LIST 255 297 Y 54.01 Z 0
REPEAT 25 ID 300 Z -36

GENERATE 2 JOINTS ID 5000023 1 X LIST 318 360 Y 54.01 Z 0
REPEAT 25 ID 300 Z -36

GENERATE 2 JOINTS ID 2000025 1000000 X LIST 132 228 Y 54.01 0 Z 0 0
REPEAT 25 ID 300 Z -36

$====================================================================
$ Generate Deck Plate Elements
$====================================================================
TYPE PLATE

GENERATE 5 ELEMENTS ID 'P-100011' 100000 FROM 1000023 1000000 TO 1000024 1000000 TO
1000324 1000000 TO 1000323 1000000
REPEAT 24 ID 10 FROM INCR 300 TO INCR 300

GENERATE 1 ELEMENTS ID 'P-100012' 0 FROM 1000024 0 TO 2000023 0 TO 2000323 0 TO 1000324
0
REPEAT 24 ID 10 FROM INCR 300 TO INCR 300

GENERATE 2 ELEMENTS ID 'P-200012' 100000 FROM 2000024 1000000 TO 2000025 1000000 TO
2000325 1000000 TO 2000324 1000000
REPEAT 24 ID 10 FROM INCR 300 TO INCR 300

GENERATE 2 ELEMENTS ID 'P-200013' 100000 FROM 2000025 1000000 TO 3000023 1000000 TO
3000323 1000000 TO 2000325 1000000
REPEAT 24 ID 10 FROM INCR 300 TO INCR 300

GENERATE 1 ELEMENTS ID 'P-400012' 0 FROM 4000024 0 TO 5000023 0 TO 5000323 0 TO 4000324
0
REPEAT 24 ID 10 FROM INCR 300 TO INCR 300
```

```
$=====================================================================
$ Generate Rigid Members to Connect Girder Elements to Plate Elements
$=====================================================================
TYPE SPACE TRUSS

GENERATE 2 MEMBERS ID 'R-10001' 1 FROM 1000019 3 TO 1000023 1
REPEAT 4 ID 10000 FROM INCR 1000000 TO INCR 1000000
REPEAT 25 ID 10 FROM INCR 300 TO INCR 300

$=====================================================================
$ Generate Diaphragm Members at Top of Bottom Flange
$=====================================================================
$ END DIAPHRAGMS

GENERATE 4 MEMBERS ID 'B-1011' 1 FROM 1000008 1000000 TO 2000005 1000000
GENERATE 4 MEMBERS ID 'B-1021' 1 FROM 1000018 1000000 TO 2000015 1000000
GENERATE 4 MEMBERS ID 'B-1031' 1 FROM 1007508 1000000 TO 2007505 1000000
GENERATE 4 MEMBERS ID 'B-1041' 1 FROM 1007518 1000000 TO 2007515 1000000

$=====================================================================
$ Define Supports
$=====================================================================
STATUS SUPPORT JOINTS -
 1000001 TO 5000001 BY 1000000 1000002 TO 5000002 BY 1000000 1000003 TO 5000003 BY
1000000 1000004 TO 5000004 BY 1000000 -
 1007501 TO 5007501 BY 1000000 1007502 TO 5007502 BY 1000000 1007503 TO 5007503 BY
1000000 1007504 TO 5007504 BY 1000000

$=====================================================================
$ Set Boundary Conditions
$=====================================================================
JOINT RELEASES

$ GIRDER BASE
$  END PIN CONDITIONS
 1000001 TO 1000004 2000001 TO 2000004 3000001 TO 3000004 4000001 TO 4000004 5000001 TO
5000004 MOMENT X Y
 1007501 TO 1007504 2007501 TO 2007504 3007501 TO 3007504 4007501 TO 4007504 5007501 TO
5007504 MOMENT X Y
 1007501 TO 1007504 2007501 TO 2007504 3007501 TO 3007504 4007501 TO 4007504 5007501 TO
5007504 FORCE Z

$=====================================================================
$ Copy Generated Part and Copy it Down Entire Length of Bridge
$=====================================================================
DEFINE OBJECT 'PART1' JOINTS EXISTING 1000001 TO 5007524, -
ELEMENTS EXISTING 'G-100011' TO 'G-500760', -
ELEMENTS EXISTING 'P-100011' TO 'P-500253', MEMEBRS EXISTING 'R-10001' TO 'R-50252', -
MEMEBRS EXISTING 'B-1011' TO 'B-1044'
COPY OBJECT 'PART1' REPEAT 2 TIMES JOINT INCR 100000 ELEMENT INCR 10000 MEMBER
INCR 1000 TRANSLATE Z -903

$=====================================================================
$ Generate Deck Plate Elements Above the Support
$=====================================================================
TYPE PLATE
```

```
GENERATE 5 ELEMENTS ID 'P-1001' 1000 FROM 1007523 1000000 TO 1100023 1000000 TO
1100024 1000000 TO 1007524 1000000
REPEAT 1 ID 100 FROM INCR 100000 TO INCR 100000

GENERATE 1 ELEMENTS ID 'P-1002' 0 FROM 1007524 0 TO 2007523 0 TO 2100023 0 TO 1100024 0
REPEAT 1 ID 100 FROM INCR 100000 TO INCR 100000

GENERATE 2 ELEMENTS ID 'P-2002' 1000 FROM 2007524 1000000 TO 2007525 1000000 TO
2100025 1000000 TO 2100024 1000000
REPEAT 1 ID 100 FROM INCR 100000 TO INCR 100000

GENERATE 2 ELEMENTS ID 'P-2003' 1000 FROM 2007525 1000000 TO 3007523 1000000 TO
3100023 1000000 TO 2100025 1000000
REPEAT 1 ID 100 FROM INCR 100000 TO INCR 100000

GENERATE 1 ELEMENTS ID 'P-4002' 0 FROM 4007524 0 TO 5007523 0 TO 5100023 0 TO 4100024 0
REPEAT 1 ID 100 FROM INCR 100000 TO INCR 100000

$=====================================================================
$ Define Element Properties
$=====================================================================
MATERIAL CONCRETE ALL ELEMENTS
ELEMENT PROPERTIES
 EXISTING 'G-100011' TO 'G-120760' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'G-200011' TO 'G-220760' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'G-300011' TO 'G-320760' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'G-400011' TO 'G-420760' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'G-500011' TO 'G-520760' TYPE 'IPSL' INTEGRATION ORDER 3
 EXISTING 'P-100011' TO 'P-120253' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-200011' TO 'P-220253' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-300011' TO 'P-320253' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-400011' TO 'P-420253' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-500011' TO 'P-520253' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-1001' TO 'P-1103' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-2001' TO 'P-2103' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-3001' TO 'P-3103' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-4001' TO 'P-4103' TYPE 'SBCR' THICKNESS 8
 EXISTING 'P-5001' TO 'P-5103' TYPE 'SBCR' THICKNESS 8

$=====================================================================
$ Define Member Properties
$=====================================================================
MATERIAL CONCRETE ALL MEMBERS
MEMBER PROPERTIES PRISMATIC
 EXISTING 'R-10001' TO 'R-52252' AX 8

MEMBER PROPERTIES PRISMATIC
 EXISTING 'B-1011' TO 'B-3044' AX 330

$=====================================================================
$ Define Supports of Deck
$=====================================================================
STATUS SUPPORT JOINTS -
 1000023 TO 5000023 BY 1000000 1000024 TO 5000024 BY 1000000 -
 1207523 TO 5207523 BY 1000000 1207524 TO 5207524 BY 1000000
```

```
$==========================================================================
$ Set Boundary Conditions of Deck
$==========================================================================
JOINT RELEASES

$ DECK BASE
$ END CONDITIONS

1000023 TO 1000024 2000023 TO 2000024 3000023 TO 3000024 4000023 TO 4000024 5000023 TO
5000024 FORCE Y
1207523 TO 1207524 2207523 TO 2207524 3207523 TO 3207524 4207523 TO 4207524 5207523 TO
5207524 FORCE Y
1000023 TO 1000024 2000023 TO 2000024 3000023 TO 3000024 4000023 TO 4000024 5000023 TO
5000024 MOMENT X Y
1207523 TO 1207524 2207523 TO 2207524 3207523 TO 3207524 4207523 TO 4207524 5207523 TO
5207524 MOMENT X Y

$==========================================================================
$ Define Loading
$==========================================================================
UNITS LBS FT

LOADING 'DC1' 'DEAD LOAD STRUCTURAL COMPONENTS OF ELEMENTS'
 ELEMENT LOADS
  EXISTING 'G-100011' TO 'G-120760' BODY FORCES GLOBAL BY -150
  EXISTING 'G-200011' TO 'G-220760' BODY FORCES GLOBAL BY -150
  EXISTING 'G-300011' TO 'G-320760' BODY FORCES GLOBAL BY -150
  EXISTING 'G-400011' TO 'G-420760' BODY FORCES GLOBAL BY -150
  EXISTING 'G-500011' TO 'G-520760' BODY FORCES GLOBAL BY -150
  EXISTING 'P-100011' TO 'P-120253' BODY FORCES GLOBAL BY -150
  EXISTING 'P-200011' TO 'P-220253' BODY FORCES GLOBAL BY -150
  EXISTING 'P-300011' TO 'P-320253' BODY FORCES GLOBAL BY -150
  EXISTING 'P-400011' TO 'P-420253' BODY FORCES GLOBAL BY -150
  EXISTING 'P-500011' TO 'P-520253' BODY FORCES GLOBAL BY -150
  EXISTING 'P-1001' TO 'P-1103' BODY FORCES GLOBAL BY -150
  EXISTING 'P-2001' TO 'P-2103' BODY FORCES GLOBAL BY -150
  EXISTING 'P-3001' TO 'P-3103' BODY FORCES GLOBAL BY -150
  EXISTING 'P-4001' TO 'P-4103' BODY FORCES GLOBAL BY -150
  EXISTING 'P-5001' TO 'P-5103' BODY FORCES GLOBAL BY -150

DEAD LOAD 'DC2' 'DEAD LOAD STRUCTURAL COMPONENTS OF MEMBERS' DIRECTION -Y -
  MEMBERS 'B-1011' TO 'B-1014' 'B-1021' TO 'B-1024' 'B-1031' TO 'B-1034' 'B-1041' TO 'B-1044' -
  'B-2011' TO 'B-2014' 'B-2021' TO 'B-2024' 'B-2031' TO 'B-2034' 'B-2041' TO 'B-2044' -
  'B-3011' TO 'B-3014' 'B-3021' TO 'B-3024' 'B-3031' TO 'B-3034' 'B-3041' TO 'B-3044'

LOADING 'LS' 'LIVE LOAD SURCHARGE'
 ELEMENT LOADS
  EXISTING 'P-100011' TO 'P-120253' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-200011' TO 'P-220253' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-300011' TO 'P-320253' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-400011' TO 'P-420253' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-500011' TO 'P-520253' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-1001' TO 'P-1103' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-2001' TO 'P-2103' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-3001' TO 'P-3103' SURFACE FORCES GLOBAL PY -31
  EXISTING 'P-4001' TO 'P-4103' SURFACE FORCES GLOBAL PY -31
```

```
 EXISTING 'P-5001' TO 'P-5103' SURFACE FORCES GLOBAL PY -31

LOADING 'LL1' 'VEHICULAR LIVE LOAD HS20-44'
 JOINT LOADS
$ Truck 1 HS20-44
 3002425 3002423 FORCE Y -4000
 3003925 3003923 FORCE Y -16000
 3005425 3005423 FORCE Y -16000

LOADING 'LL2' 'VEHICULAR LIVE LOAD HS20-44 Fatigue'
 JOINT LOADS
$ Truck 2 HS20-44 Fatigue
 3006625 3006623 FORCE Y -4000
 3005425 3005423 FORCE Y -16000
 3002425 3002423 FORCE Y -16000

LOADING 'LL3' 'VEHICULAR LIVE LOAD 3S2'
 JOINT LOADS
$ Truck 3 3S2
 3006625 3006623 FORCE Y -6000
 3005425 3005423 FORCE Y -12000
 3005125 3005123 FORCE Y -12000
 3002425 3002423 FORCE Y -12000
 3002125 3002123 FORCE Y -12000

$==========================================================================
$ Factored Loads
$==========================================================================
LOADING COMBINATION 11 'STRENGTH I MAXIMUM - HS20-44' SPECS 'DC1' 1.25 'DC2' 1.25
'LL1' 1.75 'LS' 1.75 'WS' 0.0
LOADING COMBINATION 13 'STRENGTH I MAXIMUM - 3S2' SPECS 'DC1' 1.25 'DC2' 1.25 'LL3'
1.75 'LS' 1.75 'WS' 0.0

LOADING COMBINATION 42 'FATIGUE - IMPACT 1.3 - FACTOR 0.75 - HS20-44 Fatigue'   SPECS
'LL2' 0.975
LOADING COMBINATION 43 'FATIGUE - IMPACT 1.3 - FACTOR 0.75 - 3S2'   SPECS 'LL3' 0.975

$==========================================================================
$ Prepare and Generate Output
$==========================================================================
QUERY
STIFFNESS ANALYSIS

UNITS KIP INCH

LIST SUMMATION OF REACTIONS
LIST REACTIONS

CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-100011'
TO 'G-120760'
CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-200011'
TO 'G-220760'
CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-300011'
TO 'G-320760'
CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-400011'
TO 'G-420760'
```

CALCULATE AVERAGE STRESS AT MIDDLE SURFACE FOR ELEMENTS EXISTING 'G-500011' TO 'G-520760'

CALCULATE AVERAGE STRESS AT TOP SURFACE FOR ELEMENTS EXISTING 'P-1001' TO 'P-520253'
CALCULATE AVERAGE STRESS AT BOTTOM SURFACE FOR ELEMENTS EXISTING 'P-1001' TO 'P-520253'

APPENDIX C

TYPICAL SAS INPUT FILES

C.1 SAS Input File of Deck Area I, Stress Component Sxx

```
Data SxxStressBlock1;
Input GT GN TT Stress;
Cards;
  1 1 1 0.137976
  1 2 1 -0.112013
  2 1 1 -0.129042
  2 2 1 -0.10026
  3 1 1 -0.122828
  3 2 1 -0.091185
  4 1 1 -0.228769
  4 2 1 -0.150275
  5 1 1 -0.11216
  5 2 1 -0.086865
  6 1 1 -0.104505
  6 2 1 -0.078736
  1 1 2 0.178222
  1 2 2 -0.15731
  2 1 2 0.154994
  2 2 2 -0.136228
  3 1 2 -0.140286
  3 2 2 -0.120689
  4 1 2 -0.275617
  4 2 2 -0.184634
  5 1 2 -0.129477
  5 2 2 -0.116646
  6 1 2 -0.117775
  6 2 2 -0.10302
;

PROC GLM;
CLASS GT GN TT;
MODEL Stress = GT GN TT GT*GN GT*TT GN*TT;

PROC SORT DATA=SxxStressBlock1;
 BY GT GN;
PROC MEANS DATA=SxxStressBlock1 NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;

PROC SORT DATA=SxxStressBlock1;
 BY GT TT;
PROC MEANS DATA=SxxStressBlock1 NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
```

```
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;

Run;
```

C.2 SAS Input File of Whole Deck, Treatment Factor GT and GN, Stress Component Sxx

```
Data SxxStresswholedeck;
Input GT GN TT AR Stress;
Cards;
  1 1 1 1 0.137976
  1 2 1 1 -0.112013
  2 1 1 1 -0.129042
  2 2 1 1 -0.10026
  3 1 1 1 -0.122828
  3 2 1 1 -0.091185
  4 1 1 1 -0.228769
  4 2 1 1 -0.150275
  5 1 1 1 -0.11216
  5 2 1 1 -0.086865
  6 1 1 1 -0.104505
  6 2 1 1 -0.078736
  1 1 2 1 0.178222
  1 2 2 1 -0.15731
  2 1 2 1 0.154994
  2 2 2 1 -0.136228
  3 1 2 1 -0.140286
  3 2 2 1 -0.120689
  4 1 2 1 -0.275617
  4 2 2 1 -0.184634
  5 1 2 1 -0.129477
  5 2 2 1 -0.116646
  6 1 2 1 -0.117775
  6 2 2 1 -0.10302
  1 1 1 2 0.305011
  1 2 1 2 0.198979
  2 1 1 2 0.264833
  2 2 1 2 0.170406
  3 1 1 2 0.234968
  3 2 1 2 0.149304
  4 1 1 2 -0.448777
  4 2 1 2 0.288019
  5 1 1 2 0.216278
  5 2 1 2 0.142064
  6 1 1 2 -0.223998
  6 2 1 2 0.121613
  1 1 2 2 0.339815
  1 2 2 2 0.281001
  2 1 2 2 0.292677
  2 2 2 2 0.243722
  3 1 2 2 0.25786
  3 2 2 2 0.216147
  4 1 2 2 -0.496034
  4 2 2 2 0.355456
  5 1 2 2 0.239672
  5 2 2 2 0.188495
  6 1 2 2 -0.225735
  6 2 2 2 0.162964
  1 1 1 3 0.156106
  1 2 1 3 -0.120407
```

2	1	1	3	-0.143468
2	2	1	3	-0.106831
3	1	1	3	-0.134399
3	2	1	3	-0.096649
4	1	1	3	-0.252442
4	2	1	3	-0.165148
5	1	1	3	-0.123958
5	2	1	3	-0.09153
6	1	1	3	-0.113766
6	2	1	3	-0.082464
1	1	2	3	0.216178
1	2	2	3	-0.17387
2	1	2	3	0.183405
2	2	2	3	-0.148561
3	1	2	3	-0.16365
3	2	2	3	-0.130452
4	1	2	3	-0.318424
4	2	2	3	-0.209756
5	1	2	3	-0.151509
5	2	2	3	-0.12417
6	1	2	3	-0.134998
6	2	2	3	-0.108545
1	1	1	4	0.133975
1	2	1	4	0.080895
2	1	1	4	0.118806
2	2	1	4	0.067315
3	1	1	4	0.107817
3	2	1	4	0.057632
4	1	1	4	0.137105
4	2	1	4	0.089254
5	1	1	4	0.098563
5	2	1	4	0.056132
6	1	1	4	0.089061
6	2	1	4	0.046286
1	1	2	4	0.217158
1	2	2	4	0.124437
2	1	2	4	0.189923
2	2	2	4	0.102968
3	1	2	4	0.169833
3	2	2	4	0.088963
4	1	2	4	0.230468
4	2	2	4	0.140538
5	1	2	4	0.160726
5	2	2	4	0.092304
6	1	2	4	0.143034
6	2	2	4	0.078495
1	1	1	5	0.772558
1	2	1	5	0.563859
2	1	1	5	0.724387
2	2	1	5	0.532386
3	1	1	5	0.685528
3	2	1	5	0.508677
4	1	1	5	0.862627
4	2	1	5	0.591357
5	1	1	5	0.642983
5	2	1	5	0.461714
6	1	1	5	0.605104

```
6 2 1 5 0.438883
1 1 2 5 0.735015
1 2 2 5 0.538747
2 1 2 5 0.68334
2 2 2 5 0.501422
3 1 2 5 0.642743
3 2 2 5 0.473283
4 1 2 5 0.838507
4 2 2 5 0.577365
5 1 2 5 0.604409
5 2 2 5 0.436592
6 1 2 5 0.566338
6 2 2 5 0.409247
1 1 1 6 0.166687
1 2 1 6 0.098708
2 1 1 6 0.147138
2 2 1 6 0.081689
3 1 1 6 0.132461
3 2 1 6 0.069259
4 1 1 6 0.176336
4 2 1 6 0.110741
5 1 1 6 0.123616
5 2 1 6 0.067271
6 1 1 6 0.110822
6 2 1 6 0.055121
1 1 2 6 0.319137
1 2 2 6 0.16795
2 1 2 6 0.27402
2 2 2 6 0.140801
3 1 2 6 0.240307
3 2 2 6 0.121649
4 1 2 6 0.35004
4 2 2 6 0.211775
5 1 2 6 0.233891
5 2 2 6 0.129793
6 1 2 6 0.203959
6 2 2 6 0.111599
1 1 1 7 0.149725
1 2 1 7 0.081351
2 1 1 7 0.134478
2 2 1 7 0.067583
3 1 1 7 0.122744
3 2 1 7 0.057682
4 1 1 7 0.214156
4 2 1 7 -0.092361
5 1 1 7 0.114324
5 2 1 7 0.056714
6 1 1 7 0.103553
6 2 1 7 0.046559
1 1 2 7 0.213589
1 2 2 7 0.125468
2 1 2 7 0.18833
2 2 2 7 0.101953
3 1 2 7 0.169214
3 2 2 7 0.084852
4 1 2 7 0.289653
4 2 2 7 0.137979
```

```
  5 1 2 7 0.156184
  5 2 2 7 0.08569
  6 1 2 7 0.139145
  6 2 2 7 0.071509
  1 1 1 8 0.861863
  1 2 1 8 0.54908
  2 1 1 8 0.818364
  2 2 1 8 0.521893
  3 1 1 8 0.78542
  3 2 1 8 0.501681
  4 1 1 8 0.909282
  4 2 1 8 0.628715
  5 1 1 8 0.764668
  5 2 1 8 0.456306
  6 1 1 8 0.73521
  6 2 1 8 0.436764
  1 1 2 8 0.849015
  1 2 2 8 0.517511
  2 1 2 8 0.793793
  2 2 2 8 0.484518
  3 1 2 8 0.752224
  3 2 2 8 0.460144
  4 1 2 8 1.04484
  4 2 2 8 0.588824
  5 1 2 8 0.72179
  5 2 2 8 0.425035
  6 1 2 8 0.686781
  6 2 2 8 0.401257
  1 1 1 9 0.174102
  1 2 1 9 0.09906
  2 1 1 9 0.154799
  2 2 1 9 0.081879
  3 1 1 9 0.139805
  3 2 1 9 0.069254
  4 1 1 9 0.238872
  4 2 1 9 -0.111026
  5 1 1 9 0.13046
  5 2 1 9 0.067766
  6 1 1 9 0.116965
  6 2 1 9 0.05531
  1 1 2 9 0.282039
  1 2 2 9 0.166014
  2 1 2 9 0.244035
  2 2 2 9 0.138729
  3 1 2 9 0.215393
  3 2 2 9 0.118694
  4 1 2 9 0.369582
  4 2 2 9 0.209185
  5 1 2 9 0.200711
  5 2 2 9 0.124599
  6 1 2 9 0.175923
  6 2 2 9 0.106011
;

PROC PRINT;
PROC GLM;
CLASS GT GN TT AR;
```

```
MODEL Stress = GT GN TT AR GT*GN GT*TT GT*AR GN*TT GN*AR TT*AR GT*GN*AR;

data one; set SxxStresswholedeck;
if AR=1;
PROC print;
PROC SORT DATA=one;
 BY GT GN;
PROC MEANS DATA=one NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;

data two; set SxxStresswholedeck;
if AR=2;
PROC print;
PROC SORT DATA=two;
 BY GT GN;
PROC MEANS DATA=two NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;

data three; set SxxStresswholedeck;
if AR=3;
PROC print;
PROC SORT DATA=three;
 BY GT GN;
PROC MEANS DATA=three NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;

data four; set SxxStresswholedeck;
if AR=4;
PROC print;
PROC SORT DATA=four;
 BY GT GN;
PROC MEANS DATA=four NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;
```

```
data five; set SxxStresswholedeck;
if AR=5;
PROC print;
PROC SORT DATA=five;
 BY GT GN;
PROC MEANS DATA=five NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;

data six; set SxxStresswholedeck;
if AR=6;
PROC print;
PROC SORT DATA=six;
 BY GT GN;
PROC MEANS DATA=six NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;

data seven; set SxxStresswholedeck;
if AR=7;
PROC print;
PROC SORT DATA=seven;
 BY GT GN;
PROC MEANS DATA=seven NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;

data eight; set SxxStresswholedeck;
if AR=8;
PROC print;
PROC SORT DATA=eight;
 BY GT GN;
PROC MEANS DATA=eight NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;
```

```
data nine; set SxxStresswholedeck;
if AR=9;
PROC print;
PROC SORT DATA=nine;
 BY GT GN;
PROC MEANS DATA=nine NOPRINT MEAN VAR;
 VAR Stress;
 BY GT GN;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT GN AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=GN;

 Run;
```

C.3 SAS Input File of Whole Deck, Treatment Factor GT and TT, Stress Component Sxy

```
Data SxyStresswholedeck;
Input GT GN TT AR Stress;
Cards;
1	1	1	1	-0.08665
1	2	1	1	-0.061807
2	1	1	1	-0.073935
2	2	1	1	-0.054377
3	1	1	1	-0.064714
3	2	1	1	-0.048352
4	1	1	1	-0.099614
4	2	1	1	-0.064004
5	1	1	1	-0.059043
5	2	1	1	-0.043654
6	1	1	1	-0.050726
6	2	1	1	-0.038718
1	1	2	1	-0.107059
1	2	2	1	-0.088641
2	1	2	1	-0.08903
2	2	2	1	-0.074788
3	1	2	1	-0.076254
3	2	2	1	-0.064317
4	1	2	1	-0.124835
4	2	2	1	-0.0788
5	1	2	1	-0.069479
5	2	2	1	-0.058848
6	1	2	1	-0.058403
6	2	2	1	-0.05045
1	1	1	2	0.083732
1	2	1	2	0.058232
2	1	1	2	0.079082
2	2	1	2	0.053148
3	1	1	2	0.075247
3	2	1	2	0.04952
4	1	1	2	0.068605
4	2	1	2	0.044737
5	1	1	2	0.063586
5	2	1	2	0.042676
6	1	1	2	0.060011
6	2	1	2	0.039936
1	1	2	2	0.097368
1	2	2	2	0.077317
2	1	2	2	0.087357
2	2	2	2	0.06512
3	1	2	2	0.080024
3	2	2	2	0.056927
4	1	2	2	0.090189
4	2	2	2	0.07191
5	1	2	2	0.069497
5	2	2	2	0.051288
6	1	2	2	0.063091
6	2	2	2	0.044962
1	1	1	3	0.096888
1	2	1	3	0.068269
```

2	1	1	3	0.081646
2	2	1	3	0.059406
3	1	1	3	0.070529
3	2	1	3	0.052376
4	1	1	3	0.110923
4	2	1	3	0.069576
5	1	1	3	0.064445
5	2	1	3	0.046983
6	1	1	3	0.054661
6	2	1	3	0.041305
1	1	2	3	0.127357
1	2	2	3	0.100133
2	1	2	3	0.103632
2	2	2	3	0.083005
3	1	2	3	0.086938
3	2	2	3	0.070377
4	1	2	3	0.153956
4	2	2	3	0.087739
5	1	2	3	0.078983
5	2	2	3	0.063685
6	1	2	3	0.065102
6	2	2	3	0.053794
1	1	1	4	-0.025914
1	2	1	4	-0.036814
2	1	1	4	-0.022309
2	2	1	4	-0.0293
3	1	1	4	-0.019732
3	2	1	4	-0.024089
4	1	1	4	-0.030765
4	2	1	4	-0.023633
5	1	1	4	-0.01889
5	2	1	4	-0.022448
6	1	1	4	-0.016774
6	2	1	4	-0.018505
1	1	2	4	-0.040535
1	2	2	4	-0.058469
2	1	2	4	-0.034244
2	2	2	4	-0.046416
3	1	2	4	-0.029756
3	2	2	4	-0.03803
4	1	2	4	-0.042971
4	2	2	4	-0.037217
5	1	2	4	-0.028392
5	2	2	4	-0.035459
6	1	2	4	-0.024564
6	2	2	4	-0.029045
1	1	1	5	0.037973
1	2	1	5	0.033053
2	1	1	5	-0.036633
2	2	1	5	0.025606
3	1	1	5	-0.036515
3	2	1	5	0.020964
4	1	1	5	-0.047212
4	2	1	5	0.027078
5	1	1	5	-0.041904
5	2	1	5	0.019385
6	1	1	5	-0.041299

6	2	1	5	0.017146
1	1	2	5	0.05034
1	2	2	5	0.049336
2	1	2	5	0.043219
2	2	2	5	0.03875
3	1	2	5	0.038129
3	2	2	5	0.031454
4	1	2	5	0.052922
4	2	2	5	-0.03356
5	1	2	5	0.03862
5	2	2	5	0.02983
6	1	2	5	0.038959
6	2	2	5	0.024357
1	1	1	6	0.030394
1	2	1	6	0.04066
2	1	1	6	0.026036
2	2	1	6	0.032534
3	1	1	6	0.022873
3	2	1	6	0.026867
4	1	1	6	0.031779
4	2	1	6	0.027153
5	1	1	6	0.022109
5	2	1	6	0.025032
6	1	1	6	0.019464
6	2	1	6	0.020704
1	1	2	6	0.05365
1	2	2	6	0.069563
2	1	2	6	0.04471
2	2	2	6	0.055447
3	1	2	6	0.038276
3	2	2	6	0.045594
4	1	2	6	0.056195
4	2	2	6	0.046475
5	1	2	6	0.037967
5	2	2	6	0.042334
6	1	2	6	0.032497
6	2	2	6	0.034694
1	1	1	7	0.07431
1	2	1	7	-0.024594
2	1	1	7	0.065914
2	2	1	7	-0.017912
3	1	1	7	0.05951
3	2	1	7	0.017939
4	1	1	7	0.078667
4	2	1	7	0.041198
5	1	1	7	0.053521
5	2	1	7	0.014389
6	1	1	7	0.047245
6	2	1	7	0.01649
1	1	2	7	0.094429
1	2	2	7	-0.033721
2	1	2	7	0.081997
2	2	2	7	-0.024459
3	1	2	7	0.072679
3	2	2	7	0.026018
4	1	2	7	0.099027
4	2	2	7	0.053382

```
5	1	2	7	0.065081
5	2	2	7	0.021861
6	1	2	7	0.05637
6	2	2	7	0.023068
1	1	1	8	-0.079503
1	2	1	8	0.040296
2	1	1	8	-0.07822
2	2	1	8	0.038783
3	1	1	8	-0.07671
3	2	1	8	0.038807
4	1	1	8	0.093464
4	2	1	8	-0.0255
5	1	1	8	-0.067535
5	2	1	8	0.032608
6	1	1	8	-0.065925
6	2	1	8	0.032431
1	1	2	8	-0.103215
1	2	2	8	0.049221
2	1	2	8	-0.098591
2	2	2	8	0.039056
3	1	2	8	-0.094384
3	2	2	8	0.032802
4	1	2	8	-0.114158
4	2	2	8	-0.043332
5	1	2	8	-0.083216
5	2	2	8	0.029952
6	1	2	8	-0.079036
6	2	2	8	0.025992
1	1	1	9	-0.085581
1	2	1	9	0.024119
2	1	1	9	-0.075058
2	2	1	9	-0.020639
3	1	1	9	-0.066936
3	2	1	9	-0.024102
4	1	1	9	-0.090074
4	2	1	9	-0.04875
5	1	1	9	-0.060342
5	2	1	9	-0.019397
6	1	1	9	-0.052673
6	2	1	9	-0.020971
1	1	2	9	-0.122452
1	2	2	9	-0.03703
2	1	2	9	-0.103837
2	2	2	9	-0.040899
3	1	2	9	-0.089985
3	2	2	9	-0.042076
4	1	2	9	-0.128927
4	2	2	9	-0.071984
5	1	2	9	-0.080402
5	2	2	9	-0.035247
6	1	2	9	-0.068236
6	2	2	9	-0.034594
;

PROC PRINT;
PROC GLM;
CLASS GT GN TT AR;
```

```
MODEL Stress = GT GN TT AR GT*GN GT*TT GT*AR GN*TT GN*AR TT*AR GT*GN*AR;

data one; set SxyStresswholedeck;
if AR=1;
PROC print;
PROC SORT DATA=one;
 BY GT TT;
PROC MEANS DATA=one NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;

data two; set SxyStresswholedeck;
if AR=2;
PROC print;
PROC SORT DATA=two;
 BY GT TT;
PROC MEANS DATA=two NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;

data three; set SxyStresswholedeck;
if AR=3;
PROC print;
PROC SORT DATA=three;
 BY GT TT;
PROC MEANS DATA=three NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;

data four; set SxyStresswholedeck;
if AR=4;
PROC print;
PROC SORT DATA=four;
 BY GT TT;
PROC MEANS DATA=four NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;
```

```
data five; set SxyStresswholedeck;
if AR=5;
PROC print;
PROC SORT DATA=five;
 BY GT TT;
PROC MEANS DATA=five NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;

data six; set SxyStresswholedeck;
if AR=6;
PROC print;
PROC SORT DATA=six;
 BY GT TT;
PROC MEANS DATA=six NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;

data seven; set SxyStresswholedeck;
if AR=7;
PROC print;
PROC SORT DATA=seven;
 BY GT TT;
PROC MEANS DATA=seven NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;

data eight; set SxyStresswholedeck;
if AR=8;
PROC print;
PROC SORT DATA=eight;
 BY GT TT;
PROC MEANS DATA=eight NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;
```

```
data nine; set SxyStresswholedeck;
if AR=9;
PROC print;
PROC SORT DATA=nine;
 BY GT TT;
PROC MEANS DATA=nine NOPRINT MEAN VAR;
 VAR Stress;
 BY GT TT;
 OUTPUT OUT=DATA2 MEAN=AV_Stress VAR=VAR_Stress;
PROC PRINT;
 VAR GT TT AV_Stress VAR_Stress;
PROC PLOT;
 PLOT AV_Stress*GT=TT;

Run;
```

APPENDIX D

DETAILED ANOVA TABLES USED IN STATISTIC ANALYSIS

Table D.1 ANOVA Results, Stress Component Sxx, Area I

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.24653141	0.01369619	3.44	0.0882
Error	5	0.01992104	0.00398421		
Corrected Total	23	0.26645245			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.10884169	0.02176834	5.46	0.0429
GN	1	0.01253981	0.01253981	3.15	0.1362
TT	1	0.00003799	0.00003799	0.01	0.926
GT*GN	5	0.09899854	0.01979971	4.97	0.0516
GT*TT	5	0.01845995	0.00369199	0.93	0.5323
GN*TT	1	0.00765344	0.00765344	1.92	0.2244

Table D.2 ANOVA Results, Stress Component Sxx, Area II

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	1.27309865	0.0707277	284.38	<.0001
Error	5	0.00124353	0.00024871		
Corrected Total	23	1.27434218			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.47381693	0.09476339	381.03	<.0001
GN	1	0.12930144	0.12930144	519.9	<.0001
TT	1	0.00796943	0.00796943	32.04	0.0024
GT*GN	5	0.65608456	0.13121691	527.6	<.0001
GT*TT	5	0.00172709	0.00034542	1.39	0.3637
GN*TT	1	0.0041992	0.0041992	16.88	0.0093

Table D.3 ANOVA Results, Stress Component Sxx, Area III

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.32641528	0.01813418	3.25	0.0982
Error	5	0.02791024	0.00558205		
Corrected Total	23	0.35432551			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.14568346	0.02913669	5.22	0.0469
GN	1	0.01389407	0.01389407	2.49	0.1755
TT	1	0.00000469	0.00000469	0	0.978
GT*GN	5	0.13157224	0.02631445	4.71	0.057
GT*TT	5	0.02584972	0.00516994	0.93	0.5325
GN*TT	1	0.0094111	0.0094111	1.69	0.2508

Table D.4 ANOVA Results, Stress Component Sxx, Area IV

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.05587679	0.00310427	198.74	<.0001
Error	5	0.0000781	0.00001562		
Corrected Total	23	0.05595489			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.01063688	0.00212738	136.2	<.0001
GN	1	0.02478444	0.02478444	1586.74	<.0001
TT	1	0.01793099	0.01793099	1147.97	<.0001
GT*GN	5	0.00031279	0.00006256	4.01	0.077
GT*TT	5	0.00061715	0.00012343	7.9	0.0203
GN*TT	1	0.00159453	0.00159453	102.08	0.0002

Table D.5 ANOVA Results, Stress Component Sxx, Area V

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.36094536	0.02005252	16522.2	<.0001
Error	5	0.00000607	0.00000121		
Corrected Total	23	0.36095143			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.12102926	0.02420585	19944.3	<.0001
GN	1	0.22620553	0.22620553	186381	<.0001
TT	1	0.0061138	0.0061138	5037.44	<.0001
GT*GN	5	0.00719428	0.00143886	1185.54	<.0001
GT*TT	5	0.00023913	0.00004783	39.41	0.0005
GN*TT	1	0.00016336	0.00016336	134.6	<.0001

Table D.6 ANOVA Results, Stress Component Sxx, Area VI

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.14087669	0.00782648	106.27	<.0001
Error	5	0.00036822	0.00007364		
Corrected Total	23	0.14124491			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.02363575	0.00472715	64.19	0.0002
GN	1	0.05152804	0.05152804	699.69	<.0001
TT	1	0.05655803	0.05655803	767.99	<.0001
GT*GN	5	0.00092163	0.00018433	2.5	0.1684
GT*TT	5	0.00272724	0.00054545	7.41	0.0232
GN*TT	1	0.005506	0.005506	74.76	0.0003

Table D.7 ANOVA Results, Stress Component Sxx, Area VII

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.11836186	0.00657566	5.36	0.036
Error	5	0.00612864	0.00122573		
Corrected Total	23	0.12449051			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00832161	0.00166432	1.36	0.3727
GN	1	0.05704881	0.05704881	46.54	0.001
TT	1	0.02083046	0.02083046	16.99	0.0092
GT*GN	5	0.02099939	0.00419988	3.43	0.1013
GT*TT	5	0.01094084	0.00218817	1.79	0.2701
GN*TT	1	0.00022075	0.00022075	0.18	0.6889

Table D.8 ANOVA Results, Stress Component Sxx, Area VIII

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.73481231	0.04082291	31.89	0.0006
Error	5	0.00640037	0.00128007		
Corrected Total	23	0.74121269			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.13418712	0.02683742	20.97	0.0023
GN	1	0.58641322	0.58641322	458.11	<.0001
TT	1	0.00247079	0.00247079	1.93	0.2234
GT*GN	5	0.00442266	0.00088453	0.69	0.6525
GT*TT	5	0.00580188	0.00116038	0.91	0.5416
GN*TT	1	0.00151664	0.00151664	1.18	0.326

Table D.9 ANOVA Results, Stress Component Sxx, Area IX

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.18741677	0.01041204	5.37	0.0359
Error	5	0.00969834	0.00193967		
Corrected Total	23	0.1971151			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.01415065	0.00283013	1.46	0.3443
GN	1	0.07229353	0.07229353	37.27	0.0017
TT	1	0.05355023	0.05355023	27.61	0.0033
GT*GN	5	0.02601187	0.00520237	2.68	0.1514
GT*TT	5	0.02121607	0.00424321	2.19	0.2052
GN*TT	1	0.00019442	0.00019442	0.1	0.7643

Table D.10 ANOVA Results, Stress Component Sxy, Area I

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.01002847	0.00055714	55.28	0.0002
Error	5	0.0000504	0.00001008		
Corrected Total	23	0.01007887			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00540959	0.00108192	107.34	<.0001
GN	1	0.00226177	0.00226177	224.4	<.0001
TT	1	0.00158942	0.00158942	157.7	<.0001
GT*GN	5	0.00062727	0.00012545	12.45	0.0075
GT*TT	5	0.0001316	0.00002632	2.61	0.1578
GN*TT	1	0.00000883	0.00000883	0.88	0.3924

Table D.11 ANOVA Results, Stress Component Sxy, Area II

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.00593229	0.00032957	556.67	<.0001
Error	5	0.00000296	0.00000059		
Corrected Total	23	0.00593525			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00195504	0.00039101	660.44	<.0001
GN	1	0.00286052	0.00286052	4831.6	<.0001
TT	1	0.00077678	0.00077678	1312.03	<.0001
GT*GN	5	0.00002582	0.00000516	8.72	0.0164
GT*TT	5	0.00029396	0.00005879	99.3	<.0001
GN*TT	1	0.00002019	0.00002019	34.1	0.0021

Table D.12 ANOVA Results, Stress Component Sxy, Area III

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.0161191	0.00089551	30.28	0.0007
Error	5	0.00014787	0.00002957		
Corrected Total	23	0.01626698			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00815667	0.00163133	55.16	0.0002
GN	1	0.00371041	0.00371041	125.46	<.0001
TT	1	0.00276692	0.00276692	93.56	0.0002
GT*GN	5	0.00114275	0.00022855	7.73	0.0212
GT*TT	5	0.0003316	0.00006632	2.24	0.1981
GN*TT	1	0.00001074	0.00001074	0.36	0.573

Table D.13 ANOVA Results, Stress Component Sxy, Area IV

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.00233517	0.00012973	134.6	<.0001
Error	5	0.00000482	0.00000096		
Corrected Total	23	0.00233999			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00083217	0.00016643	172.68	<.0001
GN	1	0.00017377	0.00017377	180.29	<.0001
TT	1	0.00101303	0.00101303	1051.05	<.0001
GT*GN	5	0.00024515	0.00004903	50.87	0.0003
GT*TT	5	0.00004752	0.0000095	9.86	0.0126
GN*TT	1	0.00002354	0.00002354	24.42	0.0043

Table D.14 ANOVA Results, Stress Component Sxy, Area V

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.02314756	0.00128598	1.87	0.2522
Error	5	0.00343446	0.00068689		
Corrected Total	23	0.02658202			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00416724	0.00083345	1.21	0.4186
GN	1	0.00145393	0.00145393	2.12	0.2055
TT	1	0.00751592	0.00751592	10.94	0.0213
GT*GN	5	0.0012412	0.00024824	0.36	0.8558
GT*TT	5	0.00103483	0.00020697	0.3	0.893
GN*TT	1	0.00773444	0.00773444	11.26	0.0202

Table D.15 ANOVA Results, Stress Component Sxy, Area VI

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.00386881	0.00021493	60.19	0.0001
Error	5	0.00001786	0.00000357		
Corrected Total	23	0.00388667			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00117949	0.0002359	66.06	0.0001
GN	1	0.00010883	0.00010883	30.48	0.0027
TT	1	0.00223874	0.00223874	626.91	<.0001
GT*GN	5	0.00023645	0.00004729	13.24	0.0066
GT*TT	5	0.00010069	0.00002014	5.64	0.0403
GN*TT	1	0.00000461	0.00000461	1.29	0.3074

Table D.16 ANOVA Results, Stress Component Sxy, Area VII

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.03378176	0.00187676	60.08	0.0001
Error	5	0.00015619	0.00003124		
Corrected Total	23	0.03393795			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00464962	0.00092992	29.77	0.001
GN	1	0.02251495	0.02251495	720.77	<.0001
TT	1	0.00049554	0.00049554	15.86	0.0105
GT*GN	5	0.00581973	0.00116395	37.26	0.0006
GT*TT	5	0.00008725	0.00001745	0.56	0.7309
GN*TT	1	0.00021466	0.00021466	6.87	0.047

Table D.17 ANOVA Results, Stress Component Sxy, Area VIII

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.08812125	0.00489562	3.87	0.0701
Error	5	0.00632733	0.00126547		
Corrected Total	23	0.09444858			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.0000385	0.0000077	0.01	1
GN	1	0.05397392	0.05397392	42.65	0.0013
TT	1	0.00431762	0.00431762	3.41	0.124
GT*GN	5	0.01778787	0.00355757	2.81	0.1406
GT*TT	5	0.00886519	0.00177304	1.4	0.3602
GN*TT	1	0.00313815	0.00313815	2.48	0.1761

Table D.18 ANOVA Results, Stress Component Sxy, Area IX

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.02935787	0.00163099	34.97	0.0005
Error	5	0.00023318	0.00004664		
Corrected Total	23	0.02959105			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.00407505	0.00081501	17.48	0.0035
GN	1	0.0177634	0.0177634	380.9	<.0001
TT	1	0.00414133	0.00414133	88.8	0.0002
GT*GN	5	0.00259136	0.00051827	11.11	0.0097
GT*TT	5	0.00078161	0.00015632	3.35	0.1052
GN*TT	1	0.00000512	0.00000512	0.11	0.7538

Table D.19 ANOVA Results, Stress Component Syy, Area I

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.38244525	0.02124696	1654.89	<.0001
Error	5	0.00006419	0.00001284		
Corrected Total	23	0.38250944			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.11116291	0.02223258	1731.66	<.0001
GN	1	0.22624261	0.22624261	17621.6	<.0001
TT	1	0.0000009	0.0000009	0.07	0.8013
GT*GN	5	0.04463496	0.00892699	695.31	<.0001
GT*TT	5	0.00012143	0.00002429	1.89	0.2505
GN*TT	1	0.00028243	0.00028243	22	0.0054

Table D.20 ANOVA Results, Stress Component Syy, Area II

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.1525996	0.00847776	284.55	<.0001
Error	5	0.00014897	0.00002979		
Corrected Total	23	0.15274856			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.11221536	0.02244307	753.3	<.0001
GN	1	0.01221444	0.01221444	409.97	<.0001
TT	1	0.00733587	0.00733587	246.23	<.0001
GT*GN	5	0.01626188	0.00325238	109.17	<.0001
GT*TT	5	0.00418497	0.00083699	28.09	0.0011
GN*TT	1	0.00038708	0.00038708	12.99	0.0155

Table D.21 ANOVA Results, Stress Component Syy, Area III

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.40572839	0.02254047	886.34	<.0001
Error	5	0.00012715	0.00002543		
Corrected Total	23	0.40585555			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.11541742	0.02308348	907.69	<.0001
GN	1	0.24196983	0.24196983	9514.76	<.0001
TT	1	0.00001646	0.00001646	0.65	0.4576
GT*GN	5	0.04748114	0.00949623	373.41	<.0001
GT*TT	5	0.00021261	0.00004252	1.67	0.2932
GN*TT	1	0.00063094	0.00063094	24.81	0.0042

Table D.22 ANOVA Results, Stress Component Syy, Area IV

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.4370695	0.02428164	1059.1	<.0001
Error	5	0.00011463	0.00002293		
Corrected Total	23	0.43718413			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.12318387	0.02463677	1074.59	<.0001
GN	1	0.26669658	0.26669658	11632.6	<.0001
TT	1	0.00000324	0.00000324	0.14	0.7224
GT*GN	5	0.04630116	0.00926023	403.91	<.0001
GT*TT	5	0.00024099	0.0000482	2.1	0.2171
GN*TT	1	0.00064367	0.00064367	28.08	0.0032

Table D.23 ANOVA Results, Stress Component Syy, Area V

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.26043016	0.01446834	612.72	<.0001
Error	5	0.00011807	0.00002361		
Corrected Total	23	0.26054823			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.13956593	0.02791319	1182.09	<.0001
GN	1	0.0014282	0.0014282	60.48	0.0006
TT	1	0.10428017	0.10428017	4416.13	<.0001
GT*GN	5	0.01109111	0.00221822	93.94	<.0001
GT*TT	5	0.00319014	0.00063803	27.02	0.0013
GN*TT	1	0.00087462	0.00087462	37.04	0.0017

Table D.24 ANOVA Results, Stress Component Syy, Area VI

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.46727855	0.02595992	545.98	<.0001
Error	5	0.00023774	0.00004755		
Corrected Total	23	0.46751629			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.12711199	0.0254224	534.67	<.0001
GN	1	0.28782426	0.28782426	6053.4	<.0001
TT	1	0.00003425	0.00003425	0.72	0.4348
GT*GN	5	0.0507818	0.01015636	213.6	<.0001
GT*TT	5	0.00033463	0.00006693	1.41	0.3584
GN*TT	1	0.00119162	0.00119162	25.06	0.0041

Table D.25 ANOVA Results, Stress Component Syy, Area VII

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.4159451	0.02310806	980.13	<.0001
Error	5	0.00011788	0.00002358		
Corrected Total	23	0.41606298			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.12258321	0.02451664	1039.88	<.0001
GN	1	0.24490743	0.24490743	10387.8	<.0001
TT	1	0.0000003	0.0000003	0.01	0.9149
GT*GN	5	0.04764992	0.00952998	404.22	<.0001
GT*TT	5	0.00024966	0.00004993	2.12	0.2148
GN*TT	1	0.00055458	0.00055458	23.52	0.0047

Table D.26 ANOVA Results, Stress Component Syy, Area VIII

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.40649594	0.02258311	576.1	<.0001
Error	5	0.000196	0.0000392		
Corrected Total	23	0.40669194			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.15066903	0.03013381	768.72	<.0001
GN	1	0.0748991	0.0748991	1910.68	<.0001
TT	1	0.13331956	0.13331956	3400.99	<.0001
GT*GN	5	0.04179472	0.00835894	213.24	<.0001
GT*TT	5	0.00122429	0.00024486	6.25	0.0329
GN*TT	1	0.00458924	0.00458924	117.07	0.0001

Table D.27 ANOVA Results, Stress Component Syy, Area IX

Dependent Variable: Stress

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	0.44707141	0.0248373	548.34	<.0001
Error	5	0.00022648	0.0000453		
Corrected Total	23	0.44729789			
Source	**DF**	**Type III SS**	**Mean Square**	**F Value**	**Pr > F**
GT	5	0.12751838	0.02550368	563.05	<.0001
GN	1	0.26636736	0.26636736	5880.67	<.0001
TT	1	0.00006231	0.00006231	1.38	0.2937
GT*GN	5	0.05160068	0.01032014	227.84	<.0001
GT*TT	5	0.00037429	0.00007486	1.65	0.2974
GN*TT	1	0.00114839	0.00114839	25.35	0.004

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