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OVERLOADING BEHAVIOR

OF

STEEL MULTIGIRDER HIGHWAY BRIDGES

FINAL REPORT

Celal N. Kostem

Research Project 77-1: Overloading Behavior of

Steel Multigirder Highway Bridges

Fritz Engineering Laboratory Report No. 435.4

LEHIGH UNIVERSITY

Fritz Engineering Laboratory

COMMONWEALTH OF PENNSYLVANIA

Department of Transportation

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Project 77-1: Overloading Behavior of Steel Highway Bridges

OVERLOADING BEHAVIOR
OF
STEEL MULTIGIRDER HIGHWAY BRIDGES

by

Celal N. Kostem

LEHIGH UNIVERSITY

Office of Research

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March, 1988

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This report presents the summary of the research program on the prediction of overload response of steel multigirder highway bridges with reinforced concrete deck. The analytical developments and parametric studies were presented in previous technical reports. This report presents the highlights of the observations made in different phases of the research. Recommendations and conclusions based on the overall research program have been enumerated with appropriate referencing to the detailed description of the relevant problem area.

An in-depth study of the research program summarized in the report requires close scrutiny of the technical reports referenced herein.

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ABSTRACT

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An in-depth study of the research program summarized in the report requires close scrutiny of the technical reports referenced herein.

NOTICE:

Computer program BOVAS (Bridge OVerload Analysis - Steel) referred to in this report is fully operational at Lehigh University Computing Center's Control Data Corporation CYBER 850 Computer (Network Operating System; FORTRAN-IV compiler). However, this program is not operational in Pennsylvania Department of Transportation's computer systems.

I. INTRODUCTION

Most bridges are occasionally loaded beyond the load levels for which they were designed. The overloading of bridge superstructures can occur (1) due to the transport of heavy industrial, construction or farm equipment, (2) due to legal, across-the-board raises in vehicular weight limits, and (3) due to additional permit overloads. Another source of overloading, that tends to be overlooked as far as the possible response of the bridge is concerned, is the traverse of vehicles with a limited axle spacing and a limited number of wheels as compared to the design vehicle. The total weight of the vehicle may or may not be less then the total weight of the design vehicle, but the load is applied over a smaller area then that assumed by the designer.

The overloading of the bridge can not be simplistically related to the gross vehicular weight. A heavy "vehicle" with front axle, "multiple axle front dolly," and "multiple axle rear dolly" may be interpreted as an overloaded vehicle, if the distance between the dollies is small and if the span length of the bridge is long. Conversely, for a short span bridge, if the distance between the dollies is sufficiently long, then overloading may not be the case. This is due to the fact that only one dolly may be over the superstructure at any given time. The situation will again change if the superstructure is a multi-span continuous bridge. It can be concluded that the true assessment of the "overloading" of a given superstructure should not be limited to an examination of the gross vehicular weight, number of axles, and axle spacing, but to the "study" of the superstructure and the vehicle. This proposition immensely complicates the rating and/or overload permit application processing for a given bridge for a specific vehicular configuration.

Another type of overloading often overlooked is the case of deteriorated bridge superstructures. A bridge may loose some of its inherent strength. The original design vehicle will correspond to a case of overloading in the traverse of the "design" vehicle over the deteriorated bridge.

The observations by the district bridge engineers in Pennsylvania and the forecasts made by various investigators have clearly indicated that the overloading of bridges occurs frequently. It is prudent to assume that the frequency of overloading will increase

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(Ref. 10). About a decade ago an across-the-board increase of allowable truck weights changed the vehicles that used to be considered overloaded to legally loaded vehicles. It is also expected that similar legal increases in truck weights may again take place in the not too distant future. The legalization of higher load levels, if not accompanied by appropriate programs to rate and strengthen existing bridges, will cause all bridges to be "overloaded" when subjected to this truck traffic.

It has also been recognized that a substantial number of bridges is in need of repair and rehabilitation; especially the **bridge deck slabs**. Through the loss of strength of the deck slab it is possible that the deterioration of these bridges is being accelerated when they are subjected to design vehicular loading, let alone vehicles that are in excess of the design vehicle.

1.1 Design Conservatism

The loading configurations considered in the design of superstructures in some other countries are more severe than those employed in the United States and, more specifically, in the Commonwealth of Pennsylvania (Ref. 17). The frequency of overloading in some of these countries is not as great as that in the United States. Consequently, the problem that is being confronted, and which will present even greater problems in the future, is the reduction in the reserve strength of the bridge superstructure and/or its components. This reserve strength, which is due to the conservative dimensioning of the structure, has taken care of the adverse effects of possible design inaccuracies, construction oversights, and limited deterioration of the superstructure. A diminishing reserve strength margin, however, will make the adverse effects of the sources listed above critical, thereby, possibly requiring major bridge rehabilitation programs.

1.2 Live Load vs. Fatigue Damage

Fatigue crack initiation, fatigue crack propagation, fracture, etc. are all related to the "stress range" in a given member or detail (Refs. 4 and 5). Since the stress range is a direct function of the live loading of the superstructure, any increases in the live loading, e.g. overloaded vehicles, and the frequency of

loading will further compound the fatigue-related problems. Thus, it is absolutely essential that overloading should be considered in conjunction with "fatigue."

1.3 Proof-Loading and Ultimate Strength

Some researchers and bridge engineers in the United States and in some other countries proofloaded the old and new bridges by vehicles of various gross weights, which varied from the design "level" up to very large load levels (Ref. 23). In, majority of these field tests it was noted that the bridge did not collapse, furthermore, in most cases no major permanent damage was noted. This led to the conclusion that the bridges are far stronger and more resilient than was originally thought. This has further led to another conclusion that the weight of the regular "truck traffic" could be increased without any adverse effect to the superstructure.

In most cases the instrumentation of the bridges for the above referred tests was limited. Thus, it has not been possible to get the full picture of stresses and displacements in the superstructure. However, the tests indicated that the ultimate strength of the bridge superstructures is very high.

In other investigations either the full or a part of the bridge girders were cut-out from the original bridge superstructure (Ref. 23). These "specimens" were tested in the laboratories. Again, it was found that the girders had high inherent ultimate strength. However, these tests do not fully reveal the true overload response of bridge superstructures. To determine the "load paths," i.e. flow of forces from the vehicle to the supports, it is essential to investigate the full structure. Identification of the "critical members" and their overload response require the investigation of the full structure.

It can be noted that even though there are some test results available on the excessive loading of bridge superstructures, the available data are of greater interest on the ultimate strength of the superstructure, rather than the true overload response.

1.4 Objectives of the Reported Research

The studies have indicated that the current practice of computing the load that will be carried for different components of the bridge is not as realistic as it should be. The simplest form of this is the computation of the lateral live load distribution factor (Ref. 1). The behavior of the bridge superstructure assumed by the designer versus the actual bridge behavior as can be observed in the field are different. This problem is further compounded when the superstructure is subjected to overload vehicles of various sizes and shapes. The AASHTO Overload Provisions may be used for some slight and infrequent overloading, due to the lack of any other method (Refs. 1 and 2). However, when the overload vehicle has an uncommon weight and axle configuration there exists no reliable tool to predict the effects of the vehicle on the superstructure. This phenomenon led the Overload Permit Officers to issue or deny permits based more on their intuition than the results of an application of scientifically proven methods.

The first objective of the reported research was to develop a computer based analysis tool which could simulate the response of the bridge superstructure from dead weight load level up to the vehicular load level that would induce the collapse of the superstructure (Refs. 6 and 7). It was also required to provide information for various load levels between zero live load and collapse load. The emphasis was to be placed from zero live load level up to the load levels which will cause permanent damage in the superstructure. Furthermore, it was imperative to define the load level which would induce damage, recoverable or not, to the superstructure, the type and location of the damage, and its spread for increased load levels (Refs. 5, 6, and 7).

The second objective was to document the computer program that was developed as a part of the first objective of the research program (Ref. 14).

The third objective was to conduct a limited parametric investigation using the above referred computer program. The data from these parametric computer runs were to be extracted, and presented in a tabular form (Ref. 13).

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The fourth objective was to identify important findings from all
phases of the research, and present them in concise form for the
use of bridge engineers. This report, i.e. the final report of
the project, fulfills this objective.

1.5 Scope and the Organization of the Report

The theoretical developments carried out both within the reported
research project, and additional peripheral pilot projects, are
extremely detailed in nature (Refs.4-7, 9, 13-16, 18, 21, and
22). All the analytical research eventually led to the develop-
ment and use of computer programs (Refs. 14 and 15). The output
of the computer-based studies is too extensive to be reported in
this report. Even the reports cited in the later sections of
this report contain only the summary of the computer results.
For example, the parametric investigation on the overload be-
havior of steel multigirder highway bridges resulted in about
7,000,000 pieces of data (Ref. 13). In depth investigation of
select bridges with and without damaged girders resulted in ap-
proximately 40,000,000 pieces of data (Refs. 18 and 21). It can
then be easily seen that this report can, and will, present only
the key findings.

Chapter II summarizes the various phases of the research project,
and identifies the key references for each phase. Chapter III
presents the findings of the research program, except the
parametric investigation. Chapter IV describes the parametric
study and the findings.

II. THE RESEARCH PROGRAM

2.1 Analytical Developments

The research project required the development of a "tool" to pre-
dict the elastic and inelastic response of simple span and con-
tinuous steel multigirder highway bridges with monolithic rein-
forced concrete deck. It was proposed and required that the
analyses to be conducted would be as realistic as possible, and
would simulate the true behavior of the "full bridge
superstructure." This requirement resulted in the use of finite
element method to model the superstructure. The additional re-
quirement of the consideration of the inelastic behavior neces-
sitated the development of a new finite element program to con-
sider all forms of nonlinearities in the bridge superstructure.
These nonlinearities included the nonlinear stress-strain curve
for steel and concrete, yielding of steel, cracking and crushing
of concrete, buckling of the web of the steel girders and com-
pression flanges, etc.

Earlier experience with the overload analysis of simple span
prestressed concrete I-beam bridges have permitted the transfer
of all analytical tools and computer program segments to the
reported research project.

The key features that had to be addressed in the development of a
new finite element method and computer program were (Refs. 6, 7,
9, 15, and 16):

* The inelastic response of deep girders, i.e. "Timoshenko
   beam" vs. "Bernoulli-Navier beam."

* Partial composite construction.

* Slip between the girders and the deck slab.

* Buckling and post-buckling behavior of the girder web.

* Buckling and post-buckling behavior of the flanges.

* Damage initiation and propagation mechanism at negative
   moment region.

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* Fatigue checks.

All other nonlinear phenomenon and considerations are described in reports by Kostem (Refs. 6, 11, 12, 14, 15, 16, 18, 19, 20, 21, and 22); thus no attempt will be made to re-present the material in this report. However, a few salient points will be presented in the following sections.

2.2 Shear Punching of the Deck Slab

Bridge decks are not susceptible to shear punch failure (Ref. 11). Prior to the attainment of the load level that can cause the shear punch failure, the deck will undergo almost "total damage" due to flexure.

The primary structural response mode, as well as the failure initiation mode in the deck slab, is due to flexure. This observation is applicable to the deck slabs designed in accordance with the current AASHTO Standard Specifications for Highway Bridges (Ref. 23) and the current bridge design practice in the Commonwealth of Pennsylvania. If the bridge decks are designed using provisions that are substantially different in basic design philosophy from that employed in the AASHTO Specifications, it is possible that the mode of failure of the bridge decks could be due to other modes, such as the shear punch failure.

Another critical issue regarding the shear punch failure of the bridge decks also needs to be considered, that is, the internal tire pressure and contact geometry of the tire on the slab surface (Ref. 11). Detailed parametric studies indicated that it is possible to have shear punch failure; provided that, the contact area of the tire will be "small." This, for example, corresponds to tires with internal pressure in excess of 100-200 psi range. Most of the overloaded vehicles have multiple axles with multiple wheels on each axle. Furthermore, the tires employed in these vehicles are "low pressure tires." Low pressure tires result in a large contact area, thereby reducing the possibility of the shear punch failure. It should be recognized that the prime cause of the shear punch failure is the transmittal of a large force over an area with a limited perimeter.

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It is recognized that a new design approach was established in conjunction with the development of the Ontario Bridge Code. This approach basically employs lighter deck slab reinforcement as compared to the "standard" AASHTO (Ref. 1) philosophy. "Ontario" researchers reported that the tests conducted on the "ultimate strength" of the slabs resulted in "shear punch failure" for the large majority of the tests. That might very well be the case. Extensive interaction of the researcher with bridge engineers during the past two decades at the local, state, and federal level have not uncovered any reported "shear punch failure" of the bridge deck due to the "overloaded truck traffic." It should be noted that there exists a discrepancy in the findings of two different research teams, i.e. "shear punch" vs. flexure.

2.3 Partial Composite Action and Slip

During the inception of the research project all bridge experts have unequivocally indicated that at the "post-linear elastic regime" of the bridge superstructure there will be a slip between the girders and the deck slab. It was further stated that in the case of "partial composite construction" this slip will be more pronounced (Ref. 22). In addition it was asserted that a "universal force-slip equation" will be available in a matter of months. In the development of the analytical solution scheme and the computer program extensive effort was put forth to accommodate the "slip-phenomenon." During the conduct of the research it was noted that the researchers elsewhere could not develop a "universal slip equation." It was further shown by the author through the study of the available test data and pilot studies, that no noticeable slip takes place for fully composite and partially composite construction (Refs. 6 and 7). In the case of full non-composite construction, the slip does not take place until the structure undergoes substantial deformations.

In view of the above, the reported findings, unless indicated otherwise, employed the "full composite" assumption.

The terminology employed above requires clarification to prevent misinterpretations of the term "partial composite." In the case of full composite, there is a sufficient number of shear connectors distributed over the full length of the girder to provide a "monolithic interaction" between the top flange of the girder and

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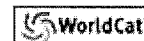
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the deck slab. In the case of partial composite action, there
exists a confusion and the lack of unified definition. Within
the framework of the reported research, the partial composite im-
plies any one of the following two, or their combination: (a)
Number of shear connectors are fewer and more flexible as com-
pared to the full composite construction, such that there might
be a slight relative longitudinal movement between the flange and
the slab. (b) There are limited "regions" over the girder flange
where the shear connectors might not have been employed.

2.4 Verification and Accuracy of the Analysis Scheme

The reported research is analytical in nature. All equations
emanating from the analytical developments required the rear-
rangement of the equations to permit them to be solved by
mainframe digital computers. Regardless of the high precision of
computer output, it is essential to check on whether the results
correspond to the "actual physical problem" on hand. Thus, at
all phases of the research the computer generated "numbers" were
spot-checked using the "slide-rule approach." This prevented
"major blunders," whether accidental or systematical, to go un-
noticed.

Reported full scale field or scaled-down laboratory model testing
of steel multigirder bridges is extremely limited. This is espe-
cially true if the needed test data is to include the post
linear-elastic behavior of the superstructure, then the available
data pool exponentially shrinks (Refs. 6, 7, and 9). To verify
the developed computer-based analysis scheme it was essential to
simulate the actual test bridges to be sure that the computer
program correctly predicts the actual bridge response.

In view of limited test data, the developed computer program was
compared against all known experimental full scale, bridge model,
and bridge component tests. Additional verification studies for
the developed computer program were carried out using (1) other
verified computer programs, (2) simplified modeling techniques,
like grid-analogy, and (3) "over-simplified" so-called "slide-
rule approaches" (Refs. 6 and 7).

All above referred comparisons indicated that the developed com-
puter program to predict the elastic and inelastic response of
the bridge superstructure yield fully acceptable values.

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III. FINDINGS

The findings of the research project are summarized below. For
each finding a reference is provided for the detailed description
of the finding and the pertinent details. It should be noted
that all findings are based on the investigations carried out
using bridges designed in accordance with the prevailing AASHTO
Standards (Refs. 1 and 2). The bridge response, and thus the
findings, would have been different if the dimensioning of the
bridge members substantially deviating from the provisions of the
AASHTO Standard Specifications for Highway Bridges

In depth investigations have been carried out for 100 ft. and
150 ft. span length bridges. These bridges were taken from
FHWA's Standard Plans for Highway Bridges (Ref. 3). Pilot inves-
tigations using bridges with various span lengths using the
bridges from the above sources indicated that the observations
based on 100 ft. bridges would yield highly reliable results on
the identification of the "trends" of the bridge behavior. Un-
less noted otherwise, the primary source of the findings listed
below are based on 100 ft. bridges. Bridges with different span
lengths were also analyzed to confirm the observed trends.

3.1 Simulation of the Overload Response
The analytical investigation on the elastic and inelastic
response of simple span and continuous steel multigirder bridges
resulted in computer program **BOVAS** (Bridge OVerload Analysis -
Steel). The information on the use of the program is described
in Ref. 14. In addition, a microcomputer based computer program
was developed. This program is called PREBOV. PREBOV is a
"conversational program." The program poses a series of ques-
tions regarding the bridge and the vehicle (Ref. 15). Based on
the "answers" to these questions, it generates the data set to be
used by program BOVAS (Ref. 14).

Program **PREBOV** and **BOVAS** can be used to predict the overload
response of steel multigirder bridges.

3.2 Composite vs. Noncomposite Behavior

All available field test observations and analytical investigations showed that regardless of the type of design assumption and construction practice under the regular traffic the bridge superstructure responds as a fully composite unit. Both partial and fully composite construction behaves as fully composite under both regular truck traffic and overloading. Under excessive overloading, which may propel the structural response into largely inelastic regime, intermittent slip between the deck and the girders may occur. This partial slip is arrested for additional loads, and another limited slip may occur for further increases of the live loading (Ref. 22). For all practical purposes it can be assumed that bridge behavior, under both design and overloading, is composite.

3.3 Residual Stresses

The pattern and the magnitude of the residual stresses in the girders does not have any discernible effect on the "initial straight line" segment of the load-deflection curve for the bridge. The same is true for the ultimate strength of the bridge. However, the transition zone of the load deflection curve, i.e. the part of the curve between the initial straight line segment and the horizontal plateau for the ultimate load, is substantially effected by the residual stresses. If the residual stresses are ignored, then the superstructure responds as a "stiff system." This results in the girders under the live load taking a large share of the vehicular load. If the residual stresses are considered, then the overall structure is more flexible, and live load is partly shed to other girders.

Both the magnitude and the pattern of residual stresses in the girders are a function of "manufacturing process." This process can change from one fabricator to another. Under the circumstances, a simple and exact method to determine the residual stresses in existing bridges is not available.

The residual stresses will not substantially (less than 5%) alter the stiffness of the superstructure at "service load levels," and at the "ultimate load level." These stresses will noticeably (20-30%) alter the stiffness of the bridge superstructure after deck damage and limited yielding.

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3.4 Vertical and Horizontal Web Stiffeners

The only role of vertical and horizontal web stiffeners is the retardation of the critical buckling load of the web (Refs. 6 and 7). These secondary members do not provide any strength and stiffness to the primary behavior of the girders (Ref. 5). The primary members, i.e. girders, essentially respond in planar bending in the plane of the web.

3.5 Local Instability

In deep girders near the supports, especially in the case of continuous construction, "web panel buckling" occurs shortly after the damage to the deck slab (Ref. 7). After the web panel buckling, the "truss action" takes place in the web. Thus, this buckling should not be construed as the "collapse" of the superstructure. The "panel size" in the web is defined by the longitudinal distance between the vertical web stiffeners. Close spacing of the web stiffeners near the supports retards, but not prevents, the panel buckling. Upon the removal of the live load, the web panel bounces back to its original form.

The lateral instability of the compression flange can become an issue at load levels far in excess of the load levels that cause the web panel buckling.

Under the current design and construction practices as defined by the AASHTO Standard Specification, web and flange buckling should not constitute an area of major concern in the case of the overloading of the steel multigirder bridge superstructure.

If the bridge structure is to be monotonically loaded with increasing live load levels up to the "ultimate load carrying capacity," then the "web panel buckling" must be considered. For increased live load levels, especially if the vertical web stiffeners are not close enough, the "exclusion" of these stability consideration gives incorrect results. The actual bridge is far softer than an analytical model which ignores the web panel buckling.

3.6 Cross Framing

The effectiveness of cross framing, e.g. X-bracing, in the distribution of the live load is dependent upon the load location. Under most **favorable conditions** the cross framing may reduce the maximum midspan deflection of the "loaded girder" by about 10%. Under normal circumstances the contribution of the cross framing in the lateral distribution of live loads is about 5% (Refs. 5 and 18).

Under the current design, detailing, and construction practices (Ref. 1) cross framing creates a problem. Even though the forces carried by the cross framing members are relatively small (as compared to the force flow in the primary members), these members create "out-of-plane" stress and deformation fields in the vicinity of their connection to the girders. For increased live load levels, e.g. frequent overloading, the stress range is directly proportional to the vehicular loading.

Doubling and quadrupling the cross section of "normally designed" cross bracings reduces the stresses in these members (Ref. 18). This does not noticeably improve the lateral distribution of the live load. The increase in these cross sectional properties adversely effects the out-of-plane distortions of the girder web.

Positive contributions of cross framing in "uniform" lateral distribution of live load are not significant. The "displacement induced fatigue" problems can be alleviated through proper retrofit of existing structures, or by using a different approach in the design of "connection details," in the case of new bridges. Guidelines provided in Reference 23 provide valuable tools to reduce the susceptibility to displacement induced fatigue failure of steel girders.

Total elimination of X-bracing should eliminate the displacement induced fatigue problem in the vicinity of the X-bracing to girder connection detail. However, this creates a new set of problems, i.e. absence of the positive contribution of the X-bracings during the erection phase of the bridge.

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3.7 Lateral Live Load Distribution Mechanism

In any given steel multigirder bridge, with or without cross bracings, lateral distribution of live load, design or overload, is primarily accomplished through the structural contribution of the bridge deck slab. The deck slab undergoes substantial transverse bending to transfer the live load from the loaded area to the unloaded girders.

If, for example, a vehicle (HS20-44 or PennDOT Permit Vehicle) is placed on the exterior lane at midspan to cause the largest flexural response, the loaded girder(s) will have the largest vertical deflection. The other exterior girder, which is farthest from the loaded area, will have an uplift. The differential vertical displacements between the girder directly under the vehicle and the girder that are not loaded is always extremely large. This indicates the large transverse bending action.

If the transverse deflection profile of the bridge at quarter points of the span is investigated, the differential vertical deflections between the loaded and unloaded girders are less than 1/5 to 1/10 of the midspan differential deflections (Ref. 16). The deck slab "dishes" immediately under the live load. Depending upon the bridge and loading geometry the deck slab also exhibits points (actually curves) of counterflexure. The "extreme fibers" of the deck slab are the most stressed members.

3.8 Longitudinal Support Conditions

A factor that has a great impact on the longitudinal girder stresses, e.g. bottom flange stresses in simple span bridges, is the presence or absence of restraint against the **longitudinal movement** of the bottom flange of the girder. The supports of a simple span bridge are designed and constructed as "**fixed-expansion**," where the bottom flange of the girder may move "a fraction of an inch" in the longitudinal direction. This move "permits" the "expansion" of the bottom flange without developing any "longitudinal stresses." However, if the bottom flange of the bridge can **not** move a minute amount at "support points," than large axial stresses in the longitudinal direction at the level of the bottom flange can develop. A typical factor that partially inhibits the movement of the bottom flange at the support points is the "freezing" of the supports. If the supports are designed as "expansion-expansion" or "fixed-expansion," and if

the supports are "frozen" against any longitudinal movement of the bottom flange of the girders, then bottom flange stresses at midspan are reduced!

The above phenomenon is due to the fact that when a girder is subjected to flexure the bottom chord elongates, and the top chord gets shorter. The magnitude of this elongation is extremely small; however, if this elongation is prevented it corresponds to the application of a very large axial force in longitudinal direction at each end of the bottom flange. These forces can also be interpreted as if the girder is subjected to "positive" end-moments, or as if the structure is subjected to post-tensioning.

Analyses were carried out on pilot studies for all three longitudinal support conditions, i.e. "expansion-expansion," "fixed-expansion," and "fixed-fixed." The parametric studies were carried out for "expansion-expansion" condition only, since this corresponds to the most conservative assumption.

A bridge with "fixed-fixed" support conditions will have 10-20% less midspan deflection and midspan bottom flange stresses as compared to the "expansion-expansion" type support assumption. It is strongly recommended that the actual behavior of the longitudinal support conditions of steel multigirder bridges be investigated for a better assessment of the true strength of the superstructure.

The above findings have been observed in limited field tests. In addition, these field tests were simulated by computer models to identify the "contribution" of the support behavior. It is noted that all bridges, due to friction at "expansion" support, exhibit some amount of longitudinal restraint. Some old bridges, with poor maintenance records have substantial longitudinal restraints. This observation raises an interesting question, which has not been answered in the reported research, since it was outside the scope of the investigation: "Would it be 'less' harmful, if the 'frozen' supports of the 'old bridges' are left untouched?" If the supports are frozen, then longitudinal forces that develop due to the live load action will be less detrimental, should the supports have been perfectly free to "expand." Since the contribution of the supports was not a part of the

reported research, no attempts were made to identify the relationship between the magnitude of the lower flange forces vs. loading vs. the bridge geometry.

3.9 Load Placement

The bridges of various span lengths, all of which were taken from FHWA Standard Plans, were analyzed for AASHTO HS20-44 Standard Vehicle, PennDOT 204-kip permit vehicle, and a host of a series of "dolly" configurations. The vehicles were placed at exterior lanes and at the centerline of the bridge. Exterior lane loading always resulted in more adverse response, as compared to center-line loading.

The worst loading case, considering the **unfactored** vehicular loading, was due to PennDOT's 204-kip permit vehicle on the outermost lane. However, the longitudinal axial stresses in the deck slab due to the dead weight of the structure are 20-50 times greater than the live load induced stresses. Lateral translation of the bottom flange of the girders (See Section 3.6) is dependent on the live load only. These translations due to dead weight of the superstructure are negligible.

3.10 Maximum Stress and Damage vs. Support Conditions

Depending upon the assumed longitudinal support conditions the peak stresses, and consequently the damage initiation, in the superstructure changes. If the support conditions are "fixed-fixed," or "fixed-expansion," the most adverse stress field will be due to the transverse bending of the deck slab. If the "expansion-expansion" conditions are assumed, than the most adversely stressed members will be the deck slab (transverse bending) and bottom flange of the girders at the tip of the bottom coverplate. This observation reinforces the proposition to have a better assessment of the support conditions in steel multigirder bridges.

3.11 Serviceability Limits

Any rating or parametric investigations that are based on the "serviceability limits" and "fully conservative assumptions" will terminate the analyses prematurely. To have a better understanding of the structural response of the bridges the analyses should

consider realistic, not the conservative, assumptions. Any analyses that are based on fully conservative assumptions will reveal little information (e.g. Ref. 13).

3.12 Damage vs. Loading

The following findings were observed to be the case for 5, 6, and 7 girder bridges. Span lengths varied from 80 ft. to 220 ft.. The width of the bridges were always 44 ft. If the bridge geometry is drastically changed, e.g. 3 or 4 girder bridge with one, and perhaps maximum of two lanes wide, and having span length of 200 ft. to 300 ft., then the applicability of the specific numbers given below may not be true.

The first yielding of the most heavily loaded girder when the bridge is loaded at the exterior lane by an "HS20-44 like vehicle" occurs when the magnitude of the load level is about 4-5 times of HS20-44. If the bridge is loaded by a PennDOT 204-kip permit vehicle, the yield load level is increased by about 30%. There is about 40% reserve capacity above the first yield live load for both AASHTO HS20-44 and PennDOT 204-kip permit vehicle.

The vertical deflection of the bridge is less for 204-kip PennDOT permit vehicle as compared to the vertical deflection due to HS20-44 vehicle.

Live load versus midspan deflection profiles for all girders are essentially linear up to the first yielding of the girder.

If the loading is on the exterior lane, the girders on the exterior lane at the other side of the bridge are unaffected by the loading.

After the initiation of the yielding of a girder under the load, the yielding quickly spreads on the girder that has yielded, rather than the initiation of the yielding in an adjacent girder.

For an equivalent increment of load above yield, the HS20-44 vehicle load produces "deeper" yielding while the PennDOT 204-kip permit vehicle loading produces "wider" yielding.

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3.13 Loading Envelope

It is noted that certain "dolly" loadings, e.g. three axles spaced four feet, cause adverse response in the bridge super-structure, as compared to AASHTO HS20-44 or PennDOT 204-kip permit vehicle. However, in general, PennDOT's 204-kip permit vehicle provides a practical and sufficiently adverse response in the overall bridge superstructure.

IV. PARAMETRIC STUDY

A series of multigirder highway bridges with "rolled girders" or "welded girders" were taken from Ref. 3, and were analyzed using program BOVAS (Ref. 14). Three different vehicular loadings were considered: AASHTO HS20-44 standard design vehicle, 204 kip PennDOT permit vehicle, and a multi-axle dolly. The results of these parametric investigations were presented in Reference 13. The tabular values in Reference 13 were subjected to statistical analysis. This chapter presents the results of this statistical analysis.

4.1 Terminology

4.1.1 Axles
To have a better understanding of the presented results some definitions are in order. The vehicles are grouped under two "classifications":

* The vehicles with distinct axle spacing, e.g. HS20-44. (Also PennDOT Permit Vehicle on long span bridges.)

* Dollies, e.g. four-axle dolly. (Also PennDOT Permit Vehicle on short span bridges.)

It can be assumed that in the case of **vehicles with spaced axles** the total number of axles in a given axle group is less than three. Spacing between these axles is not less than 4 ft. The **axle weight** for such a configuration corresponds to the total weight of the axle group. For example, in the case of the "rear axle group" of HS20-44, the number quoted will be for the total weight of the rear axle group, even though there may be one or two axles in this axle group.

In the case of **dollies** it can be assumed that there are at least three axles in a given axle group, and these axles are four feet apart. The axle weight in the case of dollies refers to the **weight of a single axle** in any given axle group.

All numerical values given in this chapter are based on the vehicular weight multiplied by the appropriate impact factor.

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4.1.2 Elastic vs. Cracked States

In the presentation of the results reference made to "elastic" indicates that the structural response of the full superstructure is linear elastic, and there are no damages to the bridge. In the case of the term "cracked," under that given loading condition the concrete cover of the reinforcing bars of the bridge deck slab has fully cracked. It should be noted that, as elaborated upon in Reference 13, the values corresponding to both elastic and cracked states are extremely conservative. This conservatism is more pronounced in the estimation of the damage to the bridge deck slab. The given values for the stresses in the girders are more realistic.

4.1.3 Bridge vs. Vehicle

The statistical analysis was conducted without discriminating between the bridges and vehicles. This approach showed a relatively large standard deviation, indicating the "rough approximations" involved in the results. Even though these are crude numbers, they are still useful for a rough assessment of the bridge rating.

Additional statistical analyses were conducted for (a) spaced axle vehicles on rolled girders, (b) spaced axle vehicles on welded girders, (c) dollies on rolled girders, and (d) dollies on welded girders. Grouping of the data as described resulted in noticeably smaller standard deviations, indicating the better clustering of the results.

In the presentation of the results in the remainder of the chapter unscientific, but descriptive, terms of "poor correlation" and "good correlation" will be used. The former indicates that the standard deviation is large, and the latter indicates a good statistical match.

4.2 Axle Weights

The findings will be presented in an abbreviated format, as follows:

* Regardless of the type of bridge, if the axle weight of a
    spaced axle vehicle is 43 kips or less, the bridge will
    remain elastic. (Poor correlation.)

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* Regardless of the type of bridge, if the axle weight of a spaced axle vehicle is 84 kips or more, the bridge deck will crack. (Good correlation.)

* Regardless of the type of bridge, if the axle weight of a dolly is 14 kips or less, the bridge will remain elastic. (Poor correlation.)

* Regardless of the type of bridge, if the axle weight of a dolly is 32 kips or more, the bridge deck will crack. (Poor correlation.)

* On rolled girder bridges if the axle weight of a spaced axle vehicle is 35 kips or less, the bridge will remain elastic. (Poor correlation.)

* On welded girder bridges if the axle weight of a spaced axle vehicle is 41 kips or less, the bridge will remain elastic. (Excellent correlation.)

* On rolled girder bridges if the axle weight of a spaced axle vehicle is 69 kips or more, the bridge deck will crack. (Poor correlation.)

* On welded girder bridges if the axle weight of a spaced axle vehicle is 81 kips or more, the bridge deck will crack. (Excellent correlation.)

4.3 Bottom Flange Tensile Stresses

The peak tensile stresses at the bottom flange due to dead load plus live load amplified by impact factor yield the following:

* Regardless of the type of bridge and vehicle, prior to the initiation of any distress to the bridge, the peak stress is 12 ksi. (Poor correlation.)

* Regardless of the type of bridge for spaced axle vehicles, prior to the initiation of any distress to the bridge, the peak stress is 15 ksi. (Good correlation.)

* Regardless of the type of bridge, for spaced axle vehicles, at load levels which cause the cracking of the bridge deck, the peak stress is 24 ksi. (Excellent correlation.)

* Regardless of the type of bridge for dollies, prior to the initiation of any distress to the bridge, the peak stress is 10 ksi. (Good correlation.)

* For rolled girder bridges with spaced axle vehicles, prior to the initiation of any distress to the bridge, the peak stress is 12 ksi. (Poor correlation.)

* For welded girder bridges with spaced axle vehicles, prior to the initiation of any distress to the bridge, the peak stress is 15 ksi. (Excellent correlation.)

* For rolled girder bridges with spaced axle vehicles, at the load level which cracks the bridge deck, the peak stress is 19 ksi. (Excellent correlation.)

* For welded girder bridges with spaced axle vehicles, at the load level which cracks the bridge deck, the peak stress is 23 ksi. (Excellent correlation.)

* For rolled girder bridges with dolly, prior to the initiation of any distress to the bridge, the peak stress is 10 ksi. (Good correlation.)

* For welded girder bridges with dolly, prior to the initiation of any distress to the bridge, the peak stress is 10 ksi. (Excellent correlation.)

* For rolled girder bridges with dolly, at the load level which cracks the bridge deck, the peak stress is 16 ksi. (Good correlation.)

* For welded girder bridges with dolly, at the load level which cracks the bridge deck, the peak stress is 12 ksi. (Excellent correlation.)

4.4 Maximum Web Tensile Stresses

The study also provided peak tensile stresses in the web of the girders. These stresses are due to dead load plus the live load amplified by the impact factor. The primary interest for the web stresses would be due to fatigue considerations. The fatigue consideration requires the "stress range," rather than the total stresses. Thus, the findings on web stresses are substantially condensed, because of their possible redundancy. It should be noted that, without any exception, the peak web tension occurs at the immediate vicinity of the peak flange tension.

* Regardless of the bridges, for spaced axle vehicles prior to any distress to the bridge, the peak stress is 12 ksi. (Excellent correlation.)

* Regardless of the bridges, for dollies, prior to any distress to the bridge, the peak stress is 8.2 ksi. (Excellent correlation.)

* Regardless of the type of bridge with a spaced axle vehicle, at the load that the bridge deck cracks, the peak stress is 20 ksi. (Good correlation.)

* Regardless of the type of bridge with a dolly, at the load that the bridge deck cracks, the peak stress is 12 ksi. (Poor correlation.)

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V. RECOMMENDED FUTURE RESEARCH

For potential future research it is possible to identify and list
many items; some of which may have eventual benefits. A typical
example would be the development of a "universal force-slip"
relation for partially composite and noncomposite construction.
However, the research has showed that this is not a critical
issue as far as the serviceability and performance of the bridges
are concerned. Thus, all research areas without immediate direct
impact on bridge engineering are omitted from the following list.
The critical areas of research are as follows:

1. There exists very limited data originated from the full scale
   field testing of fully instrumented steel multigirder high-
   way bridges. In most available tests the attention was
   focused on a very limited area of interest. Even though
   these completed tests cost substantial sums of money, there
   are no results recorded or reported for the overall bridge
   behavior.

   It is strongly urged that in-service bridges should be
   tested with extensive instrumentation. The computer based
   research can be carried out on all forms of bridges.
   However, in order to verify the computer models field test
   data is essential. (It should be noted that the term
   "verify" is used here. The field test results should never
   be used to "calibrate" the computer models. The calibration
   merely corresponds, in rather crude terms, to the use of
   "fudge-factor.")

2. In the past the interest was on the "working stress range,"
   whereas the current interest is on "load and resistance fac-
   tor approach," i.e. ultimate strength of the bridge. A con-
   veniently and often overlooked "regime" of the bridge
   response is the "post-linear elastic range." At this range
   the serviceability limits of the bridge superstructure be-
   come a critical issue.

   It is recommended that all future research should not be
   limited to the ultimate strength of the bridge, but also to
   the range between the elastic proportional limit and the ul-
   timate strength.

3. A number of transportation agencies in various states and provinces have "proofloaded" the bridge superstructures with extremely heavy loads. These tests, for the sake of speed, did not employ extensive instrumentation. The usual conclusion drawn was that "the bridges are far stronger than originally assumed." The reported research, and other research programs, have showed that the ultimate strength of the bridges is very high.

   It is recommended that rather than having hundreds of inconclusive tests, in-depth testing programs be initiated. Great attention should be paid to the response ranges where the serviceability limits might be violated.

4. It is found that the primary live load distribution mechanism in steel multigirder highway bridges is the bridge deck slab. A similar observation was noted for steel two-girder bridges, and prestressed concrete spread box-beam and I-beam bridges. Both analytical and experimental research are recommended to have a better understanding of the behavior of the deck slab in a given bridge superstructure. These studies should not be limited to the isolated study of slabs; this phenomenon is well understood. However, how the bridge deck interacts with the girders is only qualitatively observed.

5. The research showed that if the supports of a steel multigirder bridge provide longitudinal restraints, i.e. in the direction of the bottom flange, than the stresses in the bridge will alter. An in-depth study, both analytical and experimental, should be conducted to quantify the restraints provided by the supports.

6. Additional case studies using program BOVAS should be executed. In any generic bridge configuration only two span lengths were considered. To have a better understanding of the effect of span length on the bridge response, at least a series of analyses should be conducted using a span length noticeably different from those used in the reported research.

7. In conjunction with #6 above, the studies should be initiated with axle spacings noticeably different from AASHTO HS20-44. For this recommendation at least one more vehicular configuration should be employed.

8. In conjunction with #6 and #7 above, a simple but a series of critical change should be made to program BOVAS. In the conduct of the reported research every single "assumption" had to be on the absolutely conservative side, as per the requirement of the sponsoring agency. This resulted in all reported results being on the extremely conservative side. In some cases the conservatism is so high that it may make the findings impractical.

    It is recommended that tests be undertaken to "loosen" some of the assumptions, so that the results will be "reasonably conservative," rather than "absolutely conservative." A typical example would be the method of computing the bridge deck slab stresses.

9. In conjunction with #6-#8, additional tests should be conducted to quantify the stress ranges in the areas where fatigue and fracture can be critical.

V. ACKNOWLEDGMENTS

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