1971

Slab behavior of a prestressed concrete i-beam bridge lehighton bridge, July 1971

Chiou-Horng Chen

David A. VanHorn

Follow this and additional works at: http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports

Recommended Citation
http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports/1973

This Technical Report is brought to you for free and open access by the Civil and Environmental Engineering at Lehigh Preserve. It has been accepted for inclusion in Fritz Laboratory Reports by an authorized administrator of Lehigh Preserve. For more information, please contact preserve@lehigh.edu.
COMMONWEALTH OF PENNSYLVANIA
Department of Transportation
Bureau of Materials, Testing and Research
Leo D. Sandvig - Director
Wade L. Gramling - Research Engineer
Kenneth L. Heilman - Research Coordinator

Project 67-12: Lateral Distribution of Load for Bridges constructed with Prestressed Concrete I-Beams

SLAB BEHAVIOR
of a
PRESTRESSED CONCRETE I-BEAM BRIDGE
LEHIGHTON BRIDGE

by
Chiou-Horng Chen
David A. VanHorn

This work was sponsored by the Pennsylvania Department of Transportation; U. S. Department of Transportation, Federal Highway Administration; and the Reinforced Concrete Research Council. The opinions, findings, and conclusions expressed in this publication are those of the authors, and not necessarily those of the sponsors.

LEHIGH UNIVERSITY
Office of Research
Bethlehem, Pennsylvania
July, 1971
Fritz Engineering Laboratory Report No. 349.5
TABLE OF CONTENTS

ABSTRACT

1. INTRODUCTION 1

2. GENERAL TESTING PROCEDURE 4

   2.1 Test Bridge 4
   2.2 Strain Gage Locations 5
   2.3 Position Indicators, Timing, and Instrumentation 5
   2.4 Test Vehicle, Loading Lanes, and Test Runs 6

3. DATA REDUCTION AND EVALUATION 8

   3.1 Oscillograph Tracing Readings 8
   3.2 Evaluation of Experimental Data 8

      3.2.1 Transverse and Longitudinal Strains 8
      3.2.2 Transverse and Longitudinal Bending Stresses 10
      3.2.3 Experimental and Design Slab Bending Moments 11

         3.2.3.1 Formulation of the Experimental Slab Bending Moment 11
         3.2.3.2 Design Moments for Slab 15

4. PRESENTATION AND DISCUSSION OF TEST RESULTS 17

   4.1 Transverse Bending Moments 17

      4.1.1 Influence Lines for Transverse Bending Moments 17
      4.1.2 Comparisons of Experimental Transverse Bending Moments with Design Values 20
4.1.3 The Effect of the Corrugated Steel and the Depth of Concrete

4.2 Slab Strains

4.3 Slab Stresses

4.3.1 Influence Lines for Transverse Slab Stresses

4.3.2 Influence Lines for Longitudinal Slab Stresses

5. SUMMARY AND CONCLUSIONS

5.1 Summary

5.2 Conclusions

6. ACKNOWLEDGMENTS

7. TABLES

8. FIGURES

9. REFERENCES
ABSTRACT

This report presents the experimental results from the field test of the deck slab of a prestressed concrete I-beam slab superstructure. In the test program, SR-4 strain gages were attached to the top and bottom surfaces of the deck slab, along a superstructure cross-section near midspan. Measurements were made of the strains produced by the passage of an HS 20-44 load vehicle passing across the structure in each of nine load test lanes. Tests were conducted with the vehicle moving at speeds ranging from 2 mph to 60 mph, and moving across a midspan ramp at a speed of 10 mph. Information on strains, stresses, and bending moments is presented in the form of tables and influence lines. Experimental values of stresses were compared with the corresponding typical allowable stresses, and experimentally determined bending moments were compared with values used in the design of the slab. The effects of the midspan diaphragm, the corrugated steel stay-in-place forms, and the local stresses produced by the wheel loads are discussed. It was found that, under the static loading of the bridge, the tensile and compressive strains and stresses on the concrete slab surfaces were small, and the experimental slab bending moments were considerably less than the value used in the design. Under the impact loading, the maximum tensile stress at one gage location reached the vicinity of the ultimate tensile strength of the concrete. However, it is concluded that in
general, there is very little cracking of the slab under typical live load design conditions.
1. INTRODUCTION

In 1964, Lehigh University initiated a research investigation of the overall structural response of prestressed concrete beam-slab bridge superstructures to design vehicle loading conditions. A major thrust of the investigation was a field test program involving the testing of eight in-service superstructures. Six of the test bridges were of the spread box-beam type, and two were of the typical I-beam type. The principal objective of all of the field tests was the development of extensive information on the lateral distribution of a design vehicle load to the longitudinal beams. Extensive testing programs were conducted on each of the structures, yielding a large bank of experimental information on load distribution, as well as on a number of other aspects of structural behavior.

In 1968, the last of the spread box-beam bridges was tested. One of the phases of the test program for this bridge involved the measurement of slab strains at a number of locations, not only on the surface of the concrete, but also on some of the lateral reinforcing bars. Later, two I-beam bridges were tested, and in both cases, a part of the field test program was devoted to the measurement of slab strains. This report is devoted to a description of the slab behavior of the last test bridge, and I-beam superstructure.

The objectives of the slab investigation were (1) to
develop information on strains and stresses produced on the slab surface by a design load vehicle, (2) to develop information on the bending moments produced at several locations on the slab, (3) to compare the experimental results with values used in the design of the slab, based on the AASHO specifications, and (4) to form a bank of data for future comparison with the results from analytical methods.

Historically, following the completion of Kelley's work in 1926, and the publication of Westergaard's theory in 1930, a simplified method for the design of reinforced concrete bridge slabs was established and set forth in the AASHO specifications. With the passage of time, a number of the provisions of the AASHO specifications were modified and improved. However, the method for determining live load bending moments in slabs changed very little. Over the years, a number of analytical studies, laboratory investigations, and field tests have been conducted, for the purpose of confirming or improving the methods which are currently being used in slab design. One of the general conclusions of the earlier research work was that the current procedure is very conservative. However, since all of the analyses and test programs were based on specific bridges, a more general formulation of the design provisions has not been possible, to date.

Basically, the study reported herein is an experimental investigation of a single in-service superstructure. The overall intent is to assess the actual structural response to vehicular
loading, and to sense the possible need for additional investigations which would ultimately lead to the development of more refined specification provisions for the design of concrete bridge deck slabs for beam-slab type superstructures.
2. GENERAL TESTING PROCEDURE

2.1 Test Bridge

The test bridge (Fig. 1) is located near Lehighton, Pennsylvania, and carries L.R. 164-8 over Pohopoco Creek. The bridge consists of three simply supported spans, each 71 feet 6 inches center-to-center of bearings, with a skew of 90°.

The cross-section of the bridge (Fig. 2) is composed of six identical 24/45 prestressed concrete I-beams, with a center-to-center spacing of 6 feet 9 inches. The slab (Fig. 3), with minimum nominal thickness of 7-1/2 inches, was cast in place on the top of the girders, utilizing corrugated steel, stay-in-place construction forms. On the south side of the bridge, a safety curb was constructed on top of the slab (Fig. 2), with construction joints spaced at 18 feet 3 inches along the bridge. Diaphragms were located at the ends of the spans above the supports, and at midspan. The specified 28-day minimum compressive strength of the bridge deck was 3000 psi.

The slab provided a roadway width of 35 feet 11-1/2 inches. The bridge was designed as a three-lane bridge, although the actual usage is for two lanes of traffic. Additional detailed dimensions are given in the PennDOT Standards for Prestressed Concrete Bridges.
2.2 Strain Gage Locations

Section M (Fig. 1), which is a superstructure cross-section 3.55 feet east of midspan, was selected as the test section. Along Section M both longitudinal and transverse gages were mounted to measure surface strains. Theoretically, the maximum bending moment produced by the test vehicle was developed at this section when the drive axle of the test vehicle passed eastward over the bridge.

For the slab study, gages were applied on the top and bottom surfaces of three of the five panels of the bridge deck (Fig. 4). In each panel, transverse gages were applied near the edges of the top flanges of the beams, and at the centers of the panels. Longitudinal gages were mounted at the center of each panel.

2.3 Position Indicators, Timing Indicators, and Instrumentation

Three lateral hoses were used as the position indicators for the test vehicle. One hose was located at Section M, while the other two were located 40 feet east and west of Section M. As the wheels of the test vehicle passed over the hoses, offsets were produced to indicate the location of the truck at three points on the oscillograph records. Two additional hoses, installed at approximately 90 feet east and west of Section M, were used to actuate the timer which recorded the average speed of the test truck.
All gages applied were type A-9-3, except gages 608, 614, and 620 (Fig. 4), which were type A-3. The procedures for installing the gages, including grinding and cleaning of the concrete surface, waterproofing and mounting of the gages, as well as the setup of other instruments, were described in previous Lehigh University reports\textsuperscript{9-14}.

2.4 Test Vehicle, Loading Lanes, and Test Runs

The test vehicle consisted of a diesel-powered tractor and semi-trailer unit, provided by the Federal Highway Administration, and operated by personnel from that organization. The vehicle was located with crushed stone to approximate the AASHO\textsuperscript{3} HS 20-44 design loading. The actual loading and dimensions of the vehicle are shown in Fig. 5.

In the test, the roadway of the bridge was divided into nine loading lanes (Fig. 6). Lanes 2, 4, 6, and 8 were located along the centerlines of the girders. Lanes 1, 3, 5, and 7 were located midway between the girders, while lane 9 was situated 4 feet 9 inches from the face of the safety curb.

A total of 98 test runs were conducted in the test of the bridge (Table 1). These runs consisted of crawl runs, speed runs and impact runs.

The bridge was tested first with midspan diaphragms in place, and then with midspan diaphragms removed. The crawl runs were conducted at a nominal speed of 2 - 3 mph to investigate the
static behavior of the deck slab. The speed in the speed runs was varied from 15 mph to 60 mph. In the controlled impact runs, all conducted at a speed of 10 mph, a wooden ramp was located near Section M to produce a 2-inch drop of the axles of the test vehicle.

This report contains the results from the crawl runs and the controlled impact runs, since it was found that in the speed runs, the amplification of static effects in the slab was relatively small, as compared to amplification from the impact loading.
3. DATA REDUCTION AND EVALUATION

3.1 Oscillograph Tracing Readings

In the data reduction process, editing or identification of the oscillograph traces was the first step. After the identification was made, calibration and vertical excursions were measured. As shown in Fig. 7, the local effect of the wheels produced a sharp offset at the point where the tracing reached its maximum amplitude. Therefore, two vertical excursions, vex (1) and vex (2), were measured. Vex (1) represents the smooth vertical excursion without the local effect, and vex (2) the vertical excursion which includes consideration of the local perturbation produced by the wheel load. Details of the oscillograph trace readings and measurements were explained in references 12 and 17.

3.2 Evaluation of Experimental Data

3.2.1 Transverse and Longitudinal Strains

After the excursions vex (1) and vex (2) had been measured, a computer program for the CDC 6400 computer was developed to convert the vertical excursions of the gages into corresponding strains and stresses. For easy identification, the strain $\varepsilon_{xi}$ in the transverse direction was defined as $\varepsilon_{x1}$ or $\varepsilon_{x2}$. The subscript $i$ denotes the strain developed without the local effect, whereas
the subscript \( a \) corresponds to the net strain, including the local disturbance. The strains in the longitudinal direction \( \varepsilon_{y_1} \) were specified similarly as \( \varepsilon_{y_1} \) or \( \varepsilon_{y_2} \). For the development of stresses in the computer program, details will be presented in the next section.

In addition to the aforementioned vertical excursions, the input data for the development of the strain values consisted of the attenuation of the vertical excursions, equivalent calibration values, gage numbers, connecting cable lengths, gage resistances, gage factors, run numbers, lane numbers, and speeds of the test vehicle. Since the slab acted compositely with the girders, the longitudinal strain near the top flanges of the girders could be evaluated from the results in the lateral distribution of load\(^{17}\) in this bridge. The additional input data included the thicknesses of the slab surfaces near the flanges, locations of the neutral axes of the girders, and the strain values at the bottom of the girders.

The output of this program included the reprints of the input data, corresponding transverse and longitudinal strains, as well as punched cards which recorded the strain values. The reprints of the input data were for confirmation of the original data. The punched strain cards served as the input data cards in the development of slab bending moments, which will be covered in Section 3.2.3.
3.2.2 Transverse and Longitudinal Bending Stresses

In the following analysis, the concrete was assumed as a homogeneous, isotropic material. From the theory of elasticity\textsuperscript{18}, the bending stress for a two dimensional problem can be defined\textsuperscript{17} as

\[ \sigma_{xi} = \frac{E}{1-\nu} [\varepsilon_{xi} + \nu \varepsilon_{yi}] \quad i = 1,2 \]  

(1)

\[ \sigma_{yi} = \frac{E}{1-\nu} [\varepsilon_{yi} + \nu \varepsilon_{xi}] \quad i = 1,2 \]  

(2)

where:

- \( \sigma_{xi} \) = bending stress in transverse direction
- \( \sigma_{yi} \) = bending stress in longitudinal direction
- \( E \) = modulus of elasticity
- \( \nu \) = Poisson's ratio
- \( \varepsilon_{xi} \) = strain in transverse direction
- \( \varepsilon_{yi} \) = strain in longitudinal direction

\( \sigma_{x1} \) (index \( i = 1 \)) denotes that the stress was developed without the local effect. \( \sigma_{x2} \) represents the stress with local disturbance. \( \sigma_{y1} \) and \( \sigma_{y2} \) are defined similarly as \( \sigma_{x1} \) and \( \sigma_{x2} \). The index notation is similarly applied to other variables \( w_i, B_i \), and \( M_{xi} \), which will be covered in the next section.

In order to obtain the stresses \( \sigma_{xi} \) and \( \sigma_{yi} \), a computer program segment was inserted into the strain program (see Section 3.2.1). The added input data included the values of modulus of elasticity and Poisson's ratio of the concrete. The
output of the program were values of $\sigma_{xi}$ and $\sigma_{yi}$.

3.2.3 Experimental and Design Slab Bending Moments

3.2.3.1 Formulation of the Experimental Slab Bending Moment

It was assumed that the strain was linear across the slab section (Fig. 8). Considering the bending effect, the strain and displacement relationships were:

$$\varepsilon_{xi} = -z \frac{\partial^2 w_i}{\partial x^2}$$  \hspace{1cm} (3)

$$\varepsilon_{yi} = -z \frac{\partial^2 w_i}{\partial y^2}$$  \hspace{1cm} (4)

Where $z$ was the coordinate across the depth of the slab (Fig. 8), $\frac{\partial^2 w_i}{\partial x^2}$ and $\frac{\partial^2 w_i}{\partial y^2}$ were the curvatures in the transverse and longitudinal directions, defined as

$$\frac{\partial^2 w_i}{\partial x^2} = \frac{\varepsilon_{xib} - \varepsilon_{xit}}{d}$$  \hspace{1cm} (5)

$$\frac{\partial^2 w_i}{\partial y^2} = \frac{\varepsilon_{xib} - \varepsilon_{yit}}{d}$$  \hspace{1cm} (6)

and $d$ was the actual thickness of the slab where gages were installed. $\varepsilon_{xib}$ and $\varepsilon_{xit}$ were the strains (Fig. 8) at the bottom
and top of the slab in the transverse direction. \( \varepsilon_{yib} \) and \( \varepsilon_{yit} \) were similarly defined. From the theory of elasticity\(^{18} \) the stress and strain relationships were derived as

\[
\sigma_{xi} = \frac{E}{1-\nu} \left[ \varepsilon_{xi} + \nu \varepsilon_{yi} \right]
\]

Substituting Equations (3) and (4) into (7)

\[
\sigma_{xi} = -\frac{Ez}{1-\nu} \left( \frac{\partial^2 w_i}{\partial x^2} + \nu \frac{\partial^2 w_i}{\partial y^2} \right)
\]

If

\[
B_i = \frac{\partial^2 w_i}{\partial x} + \nu \frac{\partial^2 w_i}{\partial y}
\]

then

\[
\sigma_{xi} = -\left( \frac{EB_i}{1-\nu^2} \right) z
\]

From the equilibrium of the forces on the section (Fig. 8),

\[
\int_A \sigma_{xi} \, dA = 0
\]

The area of the reinforcing steel in the concrete and the corrugated steel on the bottom of the concrete were first transformed into concrete. The total area (in\(^2 \)) of the concrete section having a width of 5 inches (Fig. 8) was

\[
\Sigma A = 5d - 5.476 + 0.476 n + 7.21 t_s n
\]
where \( n \) was the ratio of modulus of elasticity of steel to that of the concrete, and \( t_s \) was the thickness of the corrugated steel. From equations (9), (10), and (11), the neutral axis can be established as

\[
\bar{z}_c = \frac{1}{E_A} \left[ 2.5 (d - 2)^2 + 5.0 (d - 1.167) + 0.238 (n - 1) (d - 1) \right. \\
\left. + 2.5 t_s n (d - 1) + 4.71 t_s n (d - 1) \right] 
\]

(12)

Following the definition from the theory of plates and shells\(^{19}\), the bending moment, per foot width of slab, is defined as

\[
M_{xi} = -\frac{EB_i}{5(1-v^2)} \left( 1.25 t_s n \left[ (d - \bar{z}_c)^2 + (d - z_c - 2)^2 \right] \\
+ 0.785 t_s n \left[ (d - \bar{z}_c)^2 - (d - z_c - 2)^2 \right] \\
+ 0.238 (n - 1) \left[ (d - \bar{z}_c - 3)^2 + (\bar{z}_c - 2)^3 \right] \\
+ \frac{5}{12} (d - 2)^3 + 5 (d - 2) (0.5 d - \bar{z}_c - 1)^2 \\
+ 1.25 \left[ 0.25 (d - \bar{z}_c - 2)^4 + \frac{1}{3} (d + 1 - \bar{z}_c) (d - z_c)^3 \\
- 0.25 (d - \bar{z}_c)^4 - \frac{1}{3} (d + 1 - \bar{z}_c) (d - \bar{z}_c - 2)^3 \right] \right)
\]

(14)

Positive bending moment indicates curvature which is concave upward, and negative bending moment denotes concave downward.

For a better understanding of the contribution of each
element in the slab section, several cases were considered in evaluating the experimentally-based values of slab bending moment. These cases are shown in Fig. 9.

Case I. Neglecting Corrugated Steel: Since the cast-in-place corrugated steel was primarily used as a construction form, it was not considered to contribute to the slab stiffness. Therefore, the slab bending moment evaluated in this case can be simply achieved by setting \( t_s = 0 \) in equation (14).

Case II. Considering Corrugated Steel: In this case, the corrugated steel was assumed to act compositively with the bridge deck. Hence, the thickness \( (t_s) \) of the corrugated steel in equation (14) was included as \( t_s = 0.0280 \) inches.

Case III. Solid Slab - Full Depth: Corrugated steel and reinforcing steel are replaced by the extra concrete under the section, and therefore, an isotropic, homogeneous solid rectangular section was assumed with a total depth \( d \). The bending moment (ft.-lb./ft.) was computed as \( ^{17,18,19} \).

\[
M_{xi} = \frac{Ed^3}{12(1-\nu^2)} \left( \frac{\partial^2 w_i}{\partial x^2} + \frac{\partial^2 w_i}{\partial y^2} \right) 
\]

(15)

Case IV. Solid Slab - Equivalent Depth: Case IV was
similar to Case III, except that the depth of section is one inch less than that of Case III, to approximate the average value of the depth.

In computing the bending moments, a fortran IV computer program was developed. The input data included strains (from data cards in the strain program), the ratio of the modulus of elasticity of steel to that of the concrete (n), the actual thickness of slab at gage locations, and the test run information. The output of this program consisted of the bending moments (4 cases), and the reprints of input strains.

3.2.3.2 Design Moments for Slab

The slab of the bridge was designed according to the AASHO Specifications, 1961 Edition. Accordingly, the live load moment for simple spans with main reinforcement perpendicular to traffic, and under HS 20-44 truck loading, was computed as:

\[ M = \frac{(S + 2)}{32} P_{20} \]  

(16)

where \( M \) is the positive or negative bending moment in foot-pounds per foot width of slab, \( S \) is the clear span between girders, and \( P_{20} \) is the HS 20-44 wheel load (16,000 pounds). The development of the slab design equation was essentially based on work by Westergaard\(^3\) and others\(^{20-23}\). Equation 16 represents a simply supported slab with single span\(^3\). If the slabs are continuous over three or more beam supports, then a continuity factor of 0.8
is applied and equation (16) becomes:

\[ M = \pm (0.80) \left( \frac{S + 2}{32} \right) P_{so} \]  

(17)

In considering the impact effect, the impact formula

\[ I = \frac{50}{L + 125} \leq 0.30 \]  

(18)

was applied to amplify values of M computed from Equation (17).

Finally, it would be appropriate to note that in the actual design of the slab, the total effective depth was taken as 7 in. The top \(\frac{1}{2}\) in. was considered to be ineffective, as well as the steel form and the concrete below the top of the form.
4. PRESENTATION AND DISCUSSION OF TEST RESULTS

4.1 Transverse Bending Moments

4.1.1 Influence Lines for Transverse Bending Moments

In the computation of transverse bending moments, equations (5-15) were applied. The moduli of elasticity of steel and concrete were taken as 29,000 ksi and 5,000 ksi, respectively. The Poisson's ratio of the concrete was assumed as 0.18. Four cases (I-IV) were considered in computing bending moments. The results of Case I will be presented in this Section, and Cases II-IV will be discussed in Section 4.1.3.

The bending moments obtained are interpreted with influence lines as shown in Figs. 10-16. The upper part in the figures represents the results with midspan diaphragms in place, and the results with the midspan diaphragms removed are shown in the lower part of the figures. Solid lines show the behavior of the bridge under static loads. Each value is the average result from two identical runs. The results from the impact runs are shown by the dashed lines. Each impact value was obtained from one test run. In both crawl and impact cases, the local effect is excluded.

With the use of the influence lines, the value of the transverse bending moment can be determined, with the bridge under single or multi-lane loading. Since the bridge was designed for more than one load lane, the superposition method can be applied
to find the maximum slab bending moment as long as the slab is not cracked.

An examination of the influence values indicates that the magnitude of the bending moment in the slab is greatest when a wheel load is positioned nearest the gage location. It can also be seen that the bending moment becomes smaller and sometimes changes in sign, when the wheel is located farther away from the gage location. In general, when the test results indicate that the truck load is located near the designated gages in the bridge, a positive bending moment is produced in the slab. Conversely, when the truck is positioned farther away, the bending moment is negative.

The determination of bending moments was based on the curvature of the slab in both transverse and longitudinal directions. It was found that both the stiffnesses and spans of the prestressed girders and slab affected the curvatures of the slab. From Figs. 11 and 13, when the truck was positioned over gage locations C and G (in lanes 3 and 7), with the wheels of the truck directly over the girders, positive bending moments were produced at the two locations.

Actually, the bridge deck is a form of one-way slab on elastic supports, and the magnitudes of the bending moments are greater at the center of the slab span than near the edges of the beams. The test section (M) was near the midspan diaphragm which, when in place, provided a partial restraint and reduced the bending moments at the test section. The removal of the midspan diaphragm
provided the release of the restraint and increased the bending moments at the test section. This behavior without diaphragms should parallel the behavior at a cross-section which is located away from the end and midspan diaphragms, for instance, at the quarter point of the longitudinal span where the effects of the diaphragms are minimal. It is quite clear that in the vicinity of the midspan diaphragm, the magnitudes of the transverse bending moments are greater with the midspan diaphragms removed.

The general shape of the influence lines for the impact runs are similar to those for the crawl runs. However the magnitudes of the bending moments under impact loads were, in all of the cases, greater than those produced in the crawl runs, both in the positive and negative regions. Since the bending moments with the midspan diaphragms removed were greater, the following discussion is mainly focused on this case. In the impact runs, lanes 1 and 9 were not included in the test program. Therefore, the bending moments at locations A and I did not reach the possible maximum magnitudes.

In the static behavior of the bridge, the maximum positive bending moment produced was 1645 ft.-lb./ft. (at location E), and the maximum negative bending moment was 400 ft.-lb./ft. (at location H). In the impact runs, the maximum bending moment was 4600 ft.-lb./ft. (at location E), and the maximum negative moment was 1350 ft.-lb./ft. (at position I). It is interesting to note that the maximum positive bending moment always occurred at location...
E, and the maximum negative moment always occurred in panel GHI, under both static and impact loading. The occurrence of greater negative bending moments in panel GHI is probably due to the effect of the heavy edge parapet in increasing the resistance to transverse rotation at the outer edge of the panel. Finally, it is also significant to note that in all slab panels (ABC, DEF, GHI), moment reversal was observed, both at midpanel locations B, E, and H, and at panel support locations A, C, D, F, G, and I.

4.1.2 Comparisons of Experimental Transverse Bending Moments With Design Values

Since the slab is supported by more than three flexible supports in this bridge, the design bending moments, both positive and negative, were evaluated from equation (17). The span of the slab, center-to-center of girders, was 6 feet 9 inches. Deducting the width of the top flange of the girder, 18 inches, the clear span of the slab was 5 feet 3 inches. Equation (17) renders M = 2900 ft.-lb./ft.

In considering the amplification due to impact, the impact fraction was computed from equation (18), yielding I = 0.38. Since the impact fraction actually used must be less than, or equal to, 0.30, the value 0.30 was used. Therefore, the design value for maximum positive and negative bending moment in the slab was ±M = 3770 ft.-lb./ft.

As shown in the last section, the maximum experimental
slab bending moment under static loading was 1645 ft.-lb./ft. Comparing this result with the design value of 2900 ft.-lb./ft., it is apparent that for this superstructure, the design of slab bending moment is rather conservative. However, in considering the impact behavior, the experimental slab bending moment, 4600 ft.-lb./ft., was greater than the design value, 3770 ft.-lb./ft. Based on this result, it appears that an increase in the impact factor used in the design of the bridge deck should be given consideration in future studies.

4.1.3 The Effect of the Corrugated Steel and the Depth of Concrete

As described in Section 3.2.3.1, four different assumptions (Fig. 9) were used in computing experimental values of the slab bending moments, based on measured surface strains. A comparison of values from the four cases indicates the range of a variation.

In the previous two sections, the values presented were from Case I, where it was assumed that the corrugated steel did not participate in resisting deformation from the loading. When the corrugated steel was considered to be acting compositely with the bridge deck concrete (Case II), the computations yielded an additional 7–8% to the computed experimental bending moments. Actually, the slab moment values probably lie between the values computed for Cases I and II. That is, the actual experimental
bending moments would be between 1.00 and 1.08 times the values presented in Sections 4.1.1 and 4.1.2, depending on the effective bond between the corrugated steel forms and the reinforced concrete slab. For a better understanding of the composite behavior, additional tests would be needed, in which extra gages should be applied directly to the surface of the corrugated steel, as well as on the concrete surface, in order to compare the strains at the interface.

Since the development of bending moment values is rather complicated when the reinforcing steel and corrugated steel are considered (Cases I and II), equation (15) was utilized to compute values in Cases III and IV, in which it was assumed that the material was isotropic, homogeneous, and linearly elastic\textsuperscript{9,10,11}.

In comparing the results from several test runs, it was found that the slab bending moments in Case III were 25 - 30\% greater than those computed in Case I. After the depth of the slab was reduced by one inch (Case IV), the slab bending moments were approximately 9\% less than those in Case I. Since the slab bending moment is roughly proportional to the cube of the depth of the concrete, it is estimated that if 3/4 inch was deducted from the total depth of the concrete slab, then the results from Case IV would be nearly equal to the values found in Case I.

4.2 Slab Strains

All of the strains from all of the gages were computed
and printed in the computer program output, with the superstructure under the various loading conditions. The maximum measured tensile and compressive strains at each gage location were then selected from a comparison of values for the crawl runs in all nine loading lanes, and under the two test conditions, with and without the midspan diaphragms. For the impact runs, the same procedure was followed except that the results represent only the seven loading lanes which were used for the impact runs. These maximum strain values from both crawl and impact runs are listed in Table 2. At this point, it should be noted that all gage numbers listed in Table 2 represent transverse gages. However, longitudinal strains, $\varepsilon_y$, are also listed opposite the transverse gage numbers. These longitudinal strains represent values obtained either from longitudinal gages at these locations, or from extrapolations of strains from longitudinal gages mounted along the sides of the beams.

It was found that each gage mounted on the slab was subjected to both tensile and compressive strains, depending on the location of the concentrated wheel loads during the various test runs. Gages 1014, 1008, and 1002 (Fig. 4), located on the bottom surfaces at the midspan points of three of the slab panels, were subjected to higher tensile strains than compressive strains. Conversely, gages 1015, 1009, 1003, located on the top side of the slab directly above gages 1014, 1008, and 1002, yielded larger compressive strains than tensile strains. The magnitudes
and signs of strains from the other gages, all located near the
top flanges of the girders, varied considerably.

The maximum tensile strain, neglecting the local effect,
was 30.4 micro.-in./in. in the crawl runs, and 95.0 micro.-in./in.
in the impact runs. Both values were measured at gage 1008.
When the local effect was included, the maximum tensile strain
was 58.5 micro.-in./in. in the crawl runs, and 95.0 micro.-in./in.
in the impact runs. Again, both values were measured at gage
1008. The maximum compressive strain, neglecting the local
effect, was 22.2 micro.-in./in. in the crawl runs, and 54.3
micro.-in./in. in the impact runs. These values were measured
at gages 1011 and 1009, respectively. When the local effect was
considered, the maximum values were 41.3 micro.-in./in. in the
crawl runs, and 54.3 micro.-in./in. in the impact runs.

At this point, it would be appropriate to describe the
measurement of the longitudinal compressive strains. At the mid-
points of the gaged slab panels (Fig. 4), longitudinal gages were
used to directly measure the longitudinal strains. However, the
longitudinal strains at the ends of the gaged slab panels,
directly above the edges of the supporting girders were evaluated
using the extrapolation of strain values measured along the verti-
cal surfaces of the girders. This procedure was used to yield
the maximum information from the limited number of available re-
cording channels.

Nearly all of the gages were subjected to local
disturbance when a concentrated wheel load was located near the gage. In the crawl runs, the local disturbance produced a magnification of strain values, with few exceptions. However, even though the amplification percentage was large, the resulting maximum strains were relatively low in magnitude (-58.5 micro.-in./in., and 41.3 micro.-in./in.). These values were measured at the bottom and top surfaces at the midpoint of the middle slab test panel.

In evaluating the results from the impact runs, the vigorous vibration of the bridge made it possible to distinguish the effect of the local disturbance from the oscillation due to the impact load. Therefore, it should be noted that in Table 2, the values listed with subscripts 1 and 2 were the same.

4.3 Slab Stresses

The slab stresses are presented in the form of influence lines, both in transverse and longitudinal directions. To illustrate the variation in the magnitude of the slab stresses under different locations of the load vehicle, the maximum values of the slab stresses are presented in Table 3. Listings in Table 3 parallel the strain values listed in Table 2, as described in Section 4.2.

4.3.1 Influence Lines for Transverse Slab Stresses

The slab stresses in the lateral direction, perpendicular
to the direction of traffic, are presented in the form of influence lines, (Figs. 16-42). In these figures, the solid lines denote the slab stresses which were evaluated without consideration of the local effect of the concentrated wheel load. In contrast, the dashed lines represent the stresses with the local effect included. Positive values indicate compression on the slab surface, while negative values indicate tension. Near the bottom of the figures, the stress distribution across the thickness of the slab is indicated for the various positions of the load vehicle, assuming a linear distribution from top to bottom.

In the crawl runs, including the local effect, the maximum tensile stress was 310 psi, produced at gage 1008, and the maximum compressive stress was 344 psi, at gage 1007.

In the crawl runs, neglecting the local effect, the maximum tensile stress was 160 psi, produced at gage 1008, while the maximum compressive stress was 167 psi, at gage 1007. In the impact runs, the maximum stresses were 513 psi tension, at gage 1008, and 281 psi compression, at gage 1009.

For the behavior under static loading, the influence lines are presented in Figs. 16-30, where Figs. 16-21 indicate values with midspan diaphragms in place, and Figs. 22-30, with diaphragms removed. For response to impact loading, the influence lines are given in Figs. 31-42, with Figs. 31-36 representing behavior with midspan diaphragms in place, and Figs. 37-42 with diaphragms removed.
4.3.2 Influence Lines for Longitudinal Slab Stresses

The development of influence lines for longitudinal stresses in the deck slab was similar to the development of information on transverse stresses. Figures 43 - 50 describe the variation in these stresses under crawl run conditions with the midspan diaphragms in place. Figures 51 - 58 represent a similar series of crawl runs, both with midspan diaphragms removed. The response to the impact runs, with midspan diaphragms in place, is indicated in Figs. 59 - 65, and with midspan diaphragms removed, in Figs. 66 - 72.

In general, the longitudinal stresses were found to be compressive, except for a few values measured at the ends of the slab test panels. In these exceptions, the gages indicated small values of tensile stresses when the truck was located on the opposite side of the bridge. It is also interesting to note that all of the longitudinal stresses measured at the midpoint of the slab test panels were very small, no matter where the load vehicle was located. This indicates that the primary bending at the midpoint of the slab panels occurs in the transverse direction. However, it was found that for the locations at the ends of the slab test panels, the values of the compressive stresses in both longitudinal and transverse directions were of approximately the same magnitude.

Since the longitudinal strains at the ends of the slab test panels were computed from measurements taken on beam gages,
there was no way to evaluate the local effect in the longitudinal direction. It was only possible to evaluate the local effect in the transverse direction in these regions. On the other hand, the influence lines for the midpoints of the slab test panels, with local effect included, were evaluated based on direct strain measurements. The maximum compressive stress with the local effect included was 296 psi in the crawl run series, and 426 psi in the impact series. Both measurements were obtained at gage 1015.
5. SUMMARY AND CONCLUSIONS

5.1 Summary

This investigation of slab behavior is one part of an overall investigation of the behavior of the superstructure of a prestressed concrete I-beam bridge. The principal objective of the overall investigation was to obtain information on the distribution of vehicular loads to the longitudinal beams. The main objectives of the investigation of the slab were (1) to develop information on stresses on the surface of the slab, (2) to develop information on the bending moments produced by both static and dynamic loading, (3) to compare the experimental results with design values yielded by the AASHO specifications, and (4) to establish a bank of experimental test results to form a base for comparison with analytical procedures to be developed in the future.

The superstructure of the test bridge was basically composed of six identical prestressed concrete I-beams, topped with a composite cast-in-place reinforced concrete slab. A reinforced concrete safety curb section was cast along one edge of the slab. This superstructure was designed according to PennDOT Bridge Division standards.

The test section for the structure was selected as the cross-section at which the maximum bending moment was produced by the load-vehicle. At this cross-section, located 3.55 feet from midspan, the first, third, and fifth panels of the slab were
instrumented with SR-4 strain gages. Nine loading lanes were established across the width of the slab. A diesel-powered tractor and semi-trailer unit, loaded with crushed stone to approximate AASHO HS 20-44 design loading, was used to load the test structure. Basically, the test series included passes of the load vehicle in the nine test lanes at speeds ranging from 2 mph to 60 mph, along with a series of controlled impact tests produced by driving the load vehicle over a small ramp located near the test cross-section.

A series of tests was conducted first with the midspan diaphragms in place, as originally constructed. After the first series had been completed, the midspan diaphragms were completely removed, and a second series of tests was conducted. A total of 98 passes of the load vehicle comprised the entire load test program for this superstructure.

The results from this investigation of the slab are presented in the form of tables which list the maximum measured strains and the maximum computed stresses developed from measurements made at selected locations on the top and bottom surfaces of the slab. In addition, influence lines are presented to illustrate the variation in stresses at the various gage locations, and to show the variation in the bending moments in the slab at nine different locations in the three slab test panels. These influence lines are based on positions of the load vehicle which result in wheels either directly at the midpoints of the slab spans or directly over the girders. Other positions were not included.
because of time limitations dictated by the availability of the field test equipment. Although it is possible that other positions of the wheels might have resulted in larger experimental values in a few cases, the shapes of the influence lines do not indicate the probability of maximum values greater than those measured. Finally, a comparison is made between experimentally developed bending moments produced during the load test program and the computed design bending moment.

5.2 Conclusions

The following conclusions can be drawn from the investigation of the structural response of the slab.

1. Under static loading conditions, the maximum bending moment derived from strain measurements at all gage locations in the slab was considerably less than the value computed according to the AASHO specifications.

2. Under impact loading, the bending moment exceeded the design value at only one location. However, it should be emphasized that the value of the modulus of elasticity for the concrete was assumed to be 5000 ksi in evaluating the experimentally based bending moments. This value was selected as an absolute upper bound for the slab concrete. For the concrete actually used in the slab ($f'_c = 3000$ psi), the value for the modulus of elasticity would more probably be in the range 3500 - 4000
ksi. For the more realistic value of modulus of elasticity, the experimentally based maximum bending moment under the impact loading condition would not have exceeded the design value.

It should also be emphasized that the controlled impact test was not of a type normally taken into account by design specifications. Instead, the impact factor yielded by the AASHO equation is to account for the typical vertical motion of the load vehicle developed at normal speeds.

3. The transverse negative bending moments were considerably smaller in magnitude than the maximum positive values. These maximum negative values were produced in the slab panel nearest the side of the bridge where the safety curb section was located. It is concluded that the effect of the safety curb was to restrain the torsional displacement of the exterior girder, thereby providing greater rotational restraint to the slab in the exterior panel.

4. The effect of the midspan diaphragm is to reduce the strains, stresses, and bending moments in the slab, at the near midspan cross-section. With the diaphragms in place, the local effect of the concentrated wheel load provides a larger component to the total stress than in
the case when the diaphragms are not present.

5. Based on a comparison of values computed from test results, the experimentally based bending moments produced in the slab were increased by approximately 8% in the case when the corrugated steel forms were considered to act fully compositely with the cast-in-place slab. Conversely, this would mean that full composite action between the form and the slab would result in a reduction of approximately 8% in stresses produced by the load vehicle. The maximum transverse and longitudinal compressive stresses were found to be much less than the allowable stress permitted in the concrete slab, both in the static and in the impact loading series. The maximum tensile stress produced in the static tests was less than the tensile strength of the slab concrete. At one of the nine gaged cross-sections, the tensile stress in the impact series was near to the tensile strength of the material. This would indicate that very little cracking would normally be produced in the bridge slab by vehicular loading.

6. As a result of the testing program on this superstructure, it is obvious that additional theoretical and experimental work is needed to more clearly consider and formulate the many factors which effect slab behavior
under vehicular loading conditions. Some of the factors which need to be considered are slab thickness, slab reinforcement, slab cracking, torsional stiffness of the beams, and local stresses produced by the wheels.

7. The findings from this investigation of slab behavior are the third series reported in the current overall research investigation of beam-slab type bridge behavior conducted at Lehigh University. Therefore, at this time, the results will serve as a representation of the slab behavior at three different transverse slab spans in a typical prestressed concrete I-beam superstructure. Similar results from another I-beam bridge (Bartonsville) and a spread box-beam bridge (Hazleton) will form a basis for comparison of field test results, and will provide a useful data base for the future analytical work required to develop possible revisions in specifications and procedures for deck slab design.
6. ACKNOWLEDGMENTS

This study was conducted in the Department of Civil Engineering and Fritz Engineering Laboratory, under the auspices of the Lehigh University Office of Research, as a part of a research investigation sponsored by the Pennsylvania Department of Transportation; the U. S. Department of Transportation, Federal Highway Administration; and the Reinforced Concrete Research Council.

The field test equipment was made available through Mr. C. F. Scheffey, now Director, Office of Research, Federal Highway Administration. The instrumentation of the test structure, and operation of the test equipment, were supervised by Messrs. R. F. Varney and H. Laatz, both from the Federal Highway Administration.

The basic research planning and administrative coordination in this investigation were in cooperation with the following individuals representing the Pennsylvania Department of Transportation: Mr. B. F. Kotalik, Bridge Engineer; Mr. H. P. Koretzky, and Mr. Hans Streibel, all from the Bridge Engineering Division; and Messrs. Leo D. Sandvig, Director; Wade L. Gramling, Research Engineer; and Foster C. Sankey and Kenneth L. Heilman, Research Coordinators, all from the Bureau of Materials, Testing, and Research.

The following members of the faculty and staff at
Lehigh University made major contributions in the conduct of the field tests and in the reduction and processing of the test data: Dr. C. N. Kostem, Prof. J. O. Liebig, Jr., Felix Bardo, Yan-Liang Chen, and Anton Wegmuller. The manuscript was typed by Mrs. Ruth Grimes, and the figures were prepared by John M. Gera and Mrs. Sharon Balogh.
7. TABLES
<table>
<thead>
<tr>
<th>Description</th>
<th>Nominal Speed (mph)</th>
<th>With Midspan Diaphragms</th>
<th>Without Midspan Diaphragms</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Lanes</td>
<td>No.</td>
</tr>
<tr>
<td>Crawl</td>
<td>2.0-3.0</td>
<td>1,2,3,4,5,6,7,8,9</td>
<td>18</td>
</tr>
<tr>
<td>Speed</td>
<td>15.0</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>17.5</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>20.0</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>22.5</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>25.0</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>27.5</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>30.0</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>32.5</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>35.0</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>55.0</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>57.5</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>60.0</td>
<td>2,6</td>
<td>2</td>
</tr>
<tr>
<td>Impact</td>
<td>10.0</td>
<td>2,3,4,5,6,7,8</td>
<td>7</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td>49</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
TABLE 2 MAXIMUM MEASURED SLAB STRAINS

Unit: $10^{-6}$ in./in.

<table>
<thead>
<tr>
<th>Gage No.</th>
<th>Crawl Runs</th>
<th>Impact Runs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tensile Strain</td>
<td>Compressive Strain</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_{x1}$</td>
<td>$\varepsilon_{x2}$</td>
</tr>
<tr>
<td>1017</td>
<td>-6.9</td>
<td>-11.8</td>
</tr>
<tr>
<td>1016</td>
<td>-11.1</td>
<td>-18.5</td>
</tr>
<tr>
<td>1015</td>
<td>-4.0</td>
<td>-4.0</td>
</tr>
<tr>
<td>1014</td>
<td>-20.8</td>
<td>-47.2</td>
</tr>
<tr>
<td>1013</td>
<td>-5.3</td>
<td>-54.6</td>
</tr>
<tr>
<td>1012</td>
<td>-19.6</td>
<td>-19.7</td>
</tr>
<tr>
<td>1011</td>
<td>-11.4</td>
<td>-37.0</td>
</tr>
<tr>
<td>1010</td>
<td>-18.6</td>
<td>-21.7</td>
</tr>
<tr>
<td>1009</td>
<td>-11.8</td>
<td>-26.5</td>
</tr>
<tr>
<td>1008</td>
<td>-30.4</td>
<td>-58.5</td>
</tr>
<tr>
<td>1007</td>
<td>-9.6</td>
<td>-9.6</td>
</tr>
<tr>
<td>1006</td>
<td>-22.8</td>
<td>-26.7</td>
</tr>
<tr>
<td>1005</td>
<td>-6.7</td>
<td>-6.7</td>
</tr>
<tr>
<td>1004</td>
<td>-16.4</td>
<td>-13.8</td>
</tr>
<tr>
<td>1003</td>
<td>-8.6</td>
<td>-13.0</td>
</tr>
<tr>
<td>1002</td>
<td>-11.3</td>
<td>-34.3</td>
</tr>
<tr>
<td>1001</td>
<td>-7.9</td>
<td>-7.9</td>
</tr>
<tr>
<td>1000</td>
<td>-12.1</td>
<td>-22.2</td>
</tr>
</tbody>
</table>
### TABLE 3 MAXIMUM COMPUTED SLAB STRESSES

Unit: psi

<table>
<thead>
<tr>
<th>Gage No.</th>
<th>Crawl Runs</th>
<th>Impact Runs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tensile Stress</td>
<td>Compressive Stress</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{x_1}$</td>
<td>$\sigma_{x_2}$</td>
</tr>
<tr>
<td>1017</td>
<td>-23.9</td>
<td>-37.8</td>
</tr>
<tr>
<td>1016</td>
<td>-44.7</td>
<td>-81.6</td>
</tr>
<tr>
<td>1015</td>
<td>-36.4</td>
<td>-36.4</td>
</tr>
<tr>
<td>1014</td>
<td>-105.2</td>
<td>-241.7</td>
</tr>
<tr>
<td>1013</td>
<td>-22.0</td>
<td>-258.3</td>
</tr>
<tr>
<td>1012</td>
<td>-89.4</td>
<td>-89.1</td>
</tr>
<tr>
<td>1011</td>
<td>-74.2</td>
<td>-181.3</td>
</tr>
<tr>
<td>1010</td>
<td>-89.7</td>
<td>-106.4</td>
</tr>
<tr>
<td>1009</td>
<td>-56.6</td>
<td>-165.0</td>
</tr>
<tr>
<td>1008</td>
<td>-160.2</td>
<td>-309.9</td>
</tr>
<tr>
<td>1007</td>
<td>-87.4</td>
<td>-87.4</td>
</tr>
<tr>
<td>1006</td>
<td>-112.2</td>
<td>-132.3</td>
</tr>
<tr>
<td>1005</td>
<td>-28.6</td>
<td>-28.6</td>
</tr>
<tr>
<td>1004</td>
<td>-77.8</td>
<td>-64.6</td>
</tr>
<tr>
<td>1003</td>
<td>-42.3</td>
<td>-65.1</td>
</tr>
<tr>
<td>1002</td>
<td>-55.9</td>
<td>-168.2</td>
</tr>
<tr>
<td>1001</td>
<td>-31.6</td>
<td>-31.6</td>
</tr>
<tr>
<td>1000</td>
<td>-54.2</td>
<td>-106.2</td>
</tr>
</tbody>
</table>
8. FIGURES
Fig. 1 Elevation of Test Bridge
Fig. 2 Cross-Section of Test Bridge
Fig. 3 Transverse Cross-Section of Slab
Fig. 4 Location of SR-4 Gages
Axle and Wheel Spacing

Front
10.425k
10.200k

Drive
32.325k
32.200k

Rear
32.340k (1)
32.675k (2)

Axle Loads

(1) Load: July 15-24, 1969
(2) Load: July 25, 1969

Fig. 5 Test Vehicle
Fig. 7 Typical Oscillograph Record - Crawl Run
Fig. 8 Distribution of Strains
Fig. 9 Assumptions in Development of Experimental Slab Bending Moments
Fig. 10 Influence Lines for Transverse Bending Moment - Location A
Fig. 11 Influence Lines for Transverse Bending Moment - Location C
Fig. 12 Influence Lines for Transverse Bending Moment - Location E
Fig. 13 Influence Lines for Transverse Bending Moment - Location G
Fig. 14 Influence Lines for Transverse Bending Moment - Location H
Fig. 15 Influence Lines for Transverse Bending Moment - Location I
Fig. 16 Influence Lines for Transverse Stress Crawl Runs - Diaphragm in Place - Gages 1017 and 1016

--- Local Effect Included

--- Local Effect Excluded

Variation in Stresses Across the Depth
Fig. 17 Influence Lines for Transverse Stress Crawl Runs — Diaphragm in Place — Gages 1013 and 1012

--- Local Effect Excluded

----- Local Effect Included

Variation in Stresses Across the Depth
Fig. 18 Influence Lines for Transverse Stress Crawl Runs - Diaphragm in Place - Gages 1009 and 1008
Fig. 19 Influence Lines for Transverse Stress Crawl Runs - Diaphragm in Place - Gages 1005 and 1004
Fig. 20 Influence Lines for Transverse Stress Crawl Runs - Diaphragm in Place - Gages 1003 and 1002
Fig. 21 Influence Lines for Transverse Stress Crawl Runs - Diaphragm in Place - Gages 1001 and 1000
Fig. 22 Influence Lines for Transverse Stress Crawl Runs - Diaphragm Removed - Gages 1017 and 1016
Fig. 23 Influence Lines for Transverse Stress Crawl Runs - Diaphragm Removed - Gages 1015 and 1014
Fig. 24 Influence Lines for Transverse Stress Crawl Runs - Diaphragm Removed - Gages 1013 and 1012

--- Local Effect Excluded

--- Local Effect Included
Fig. 25 Influence Lines for Transverse Stress Crawl Runs - Diaphragm Removed - Gages 1011 and 1010
Fig. 26 Influence Lines for Transverse Stress Crawl Runs - Diaphragm Removed - Gages 1009 and 1008
Fig. 27 Influence Lines for Transverse Stress Crawl Runs - Diaphragm Removed - Gages 1005 and 1004
Fig. 28 Influence Lines for Transverse Stress Crawl Runs - Diaphragm Removed - Gages 1003 and 1002

--- Local Effect Excluded

--- --- Local Effect Included
Fig. 29 Influence Lines for Transverse Stress Crawl Runs - Diaphragm Removed - Gages 1001 and 1000
Fig. 30 Influence Lines for Transverse Stress Impact Runs - Diaphragm in Place - Gages 1017 and 1016
Fig. 31 Influence Lines for Transverse Stress Impact Runs - Diaphragm in Place - Gages 1013 and 1012
Fig. 32 Influence Lines for Transverse Stress Impact Runs - Diaphragm in Place - Gages 1009 and 1008
Fig. 33 Influence Lines for Transverse Stress Impact Runs - Diaphragm in Place - Gages 1005 and 1004
Fig. 34 Influence Lines for Transverse Stress Impact Runs - Diaphragm in Place - Gages 1003 and 1002
Fig. 35 Influence Lines for Transverse Stress Impact Runs - Diaphragm in Place - Gages 1001 and 1000
Fig. 36 Influence Lines for Transverse Stress Impact Runs - Diaphragm Removed - Gages 1017 and 1016

---

Variation in Stresses Across the Depth

- - - Local Effect Excluded
- - - - Local Effect Included

Test Lanes 1 2 3 4 5 6 7 8 9

Stress (psi)

Gage 1017

Gage 1016
Fig. 37 Influence Lines for Transverse Stress Impact Runs - Diaphragm Removed - Gages 1013 and 1012

--- Local Effect Excluded

--- Local Effect Included

Variation in Stresses Across the Depth

Test Lanes 1 2 3 4 5 6 7 8 9

STRESS (psi)

-200 -100 0 100 200

Gage 1013

Gage 1012

- Local Effect Excluded

- Local Effect Included
Fig. 38 Influence Lines for Transverse Stress Impact Runs - Diaphragm Removed - Gages 1009 and 1008
Fig. 39 Influence Lines for Transverse Stress Impact Runs - Diaphragm Removed - Gages 1005 and 1004

--- Local Effect Included or Excluded (Coincident)
Fig. 40 Influence Lines for Transverse Stress Impact Runs - Diaphragm Removed - Gages 1003 and 1002
Fig. 41 Influence Lines for Transverse Stress Impact Runs - Diaphragm Removed - Gages 1001 and 1000
Fig. 42 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm in Place - Gages 1017 and 1016

-82-
Fig. 43 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm in Place - Gages 1013 and 1012
Fig. 44 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm in Place - Gages 1011 and 1010
Fig. 45 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm in Place - Gages 1009 and 1008
Fig. 46 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm in Place - Gages 1007 and 1006
Fig. 47 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm in Place - Gages 1005 and 1004
Fig. 48 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm in Place - Gages 1003 and 1002
Fig. 49 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm in Place - Gages 1001 and 1000
Fig. 50 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1017 and 1016
Fig. 51 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1015 and 1014
Fig. 52 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1013 and 1012
Fig. 53 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1011 and 1010
Fig. 54 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1009 and 1008

Test Lanes 1 2 3 4 5 6 7 8 9

STRESS (psi)

Long. Stress at Gage 1009

Variation in Stresses Across the Depth

--- Local Effect Excluded

--- Local Effect Included

Fig. 54 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1009 and 1008
Fig. 55 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1007 and 1006
Fig. 56 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1005 and 1004
Fig. 57 Influence Lines for Longitudinal Stress Crawl Runs - Diaphragm Removed - Gages 1001 and 1000
Fig. 58 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm in Place - Gages 1017 and 1016
Fig. 59 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm in Place - Gages 1013 and 1012
Fig. 60 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm in Place - Gages 1011 and 1010
Fig. 61 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm in Place - Gages 1007 and 1006
Fig. 62 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm in Place - Gages 1005 and 1004
Fig. 63 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm in Place - Gages 1003 and 1002
Variation in Stresses Across the Depth

Test Lanes 1 2 3 4 5 6 7 8 9

-100
0
100
200

Long. Stress at Gage 1001

Long. Stress at Gage 1000

-local effect excluded
-local effect included

Fig. 64 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm in Place - Gages 1001 and 1000
Fig. 65 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm Removed - Gages 1017 and 1016
Fig. 66 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm Removed - Gages 1015 and 1014
Fig. 67 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm Removed - Gages 1013 and 1012
Fig. 68 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm Removed - Gages 1011 and 1010

---

Test Lanes 1 2 3 4 5 6 7 8 9

STRESS (psi)

200

0

-200

Long. Stress at Gage 1011

STRESS (psi)

200

0

-200

Long. Stress at Gage 1010

Variation in Stresses Across the Depth

With or Without Local Effect
Fig. 69 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm Removed - Gages 1007 and 1006
Fig. 70 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm Removed - Gages 1005 and 1004
Fig. 71 Influence Lines for Longitudinal Stress Impact Runs - Diaphragm in Place - Gages 1001 and 1000
10. REFERENCES

1. Kelley, E. F.
   EFFECTIVE WIDTH OF CONCRETE BRIDGE SLABS SUPPORTING CONCENTRATED LOADS, Public Roads, March, 1926.

2. Westergaard, H. M.
   COMPUTATION OF STRESSES IN BRIDGE SLABS DUE TO WHEEL LOADS, Public Roads, March, 1930.

3. American Association of State Highway Officials

4. Reese, R. T.
   LOAD DISTRIBUTION IN HIGHWAY BRIDGE FLOORS: A SUMMARY AND EXAMINATION OF EXISTING METHODS OF ANALYSIS AND DESIGN OF LOAD DISTRIBUTION IN HIGHWAY BRIDGE FLOORS, M.S. Thesis, Brigham Young University, March, 1966.

5. Moorman, R. B. B.
   BENDING MOMENT DETERMINATION IN HIGHWAY BRIDGE SLABS, Engineering News Record, 113, 3, July 19, 1954.

6. Aktas, Z. and VanHorn, D. A.
   BIBLIOGRAPHY ON LOAD DISTRIBUTION IN BEAM-SLAB HIGHWAY BRIDGES, Fritz Engineering Laboratory Report No. 349.1, September, 1968.

7. Newmark, N. M., Seiss, C. P., Perman, R. R.
   STUDIES OF SLAB AND BEAM HIGHWAY BRIDGES PART I, TESTS OF SIMPLE-SPAN RIGHT I-BEAM BRIDGES, University of Illinois Engineering Experiment Station Bulletin No. 363, March 8, 1946.

8. Pennsylvania Department of Transportation, Bridge Division,
   STANDARDS FOR PRESTRESSED CONCRETE BRIDGES, 1960.
9. Guilford, A. A. and VanHorn, D. A.

10. Guilford, A. A. and VanHorn, D. A.

11. Schaffer, T. and VanHorn, D. A.
STRUCTURAL RESPONSE OF A 45° SKEW PRESTRESSED CONCRETE BOX-GIRDER HIGHWAY BRIDGE SUBJECTED TO VEHICULAR LOADING - BROOKVILLE BRIDGE, Fritz Engineering Laboratory Report 315.5, October, 1967.

12. Lin, Cheng-Shung and VanHorn, D. A.
THE EFFECT OF MIDSPAN DIAPHRAGMS ON LOAD DISTRIBUTION IN A PRESTRESSED CONCRETE BOX-BEAM BRIDGE - PHILADELPHIA BRIDGE, Fritz Engineering Laboratory Report 315.6, June, 1968.

13. Guilford, A. A. and VanHorn, D. A.

14. VanHorn, D. A.

15. Motarjemi, D. and VanHorn, D. A.
THEORETICAL ANALYSIS OF LOAD DISTRIBUTION IN PRESTRESSED CONCRETE BOX-BEAM BRIDGES, Fritz Engineering Laboratory Report 315.9, October, 1969.

16. Chen, Chiou-Horng and VanHorn, D. A.
17. Wegmuller, A. W. and VanHorn, D. A.
SLAB BEHAVIOR OF A PRESTRESSED CONCRETE I-BEAM BRIDGE -
BARTONSVILLE BRIDGE, Fritz Engineering Laboratory

18. Timoshenko, S. P. and Goodier, J. N.
THEORY OF ELASTICITY, McGraw-Hill Book Company,
New York, 1951.

THEORY OF PLATES AND SHELLS, McGraw-Hill Book

20. Jensen, V. P.
SOLUTIONS FOR CERTAIN RECTANGULAR SLABS CONTINUOUS OVER
FLEXIBLE SUPPORTS, University of Illinois Bulletin
No. 303, June, 1938.

21. Newmark, N. M.
A DISTRIBUTION PROCEDURE FOR THE ANALYSIS OF SLABS
CONTINUOUS OVER FLEXIBLE BEAMS, University of Illinois
Bulletin No. 304, June, 1938.

22. Jensen, V. P.
MOMENTS IN SIMPLE SPAN BRIDGE SLABS WITH STIFFENED
EDGES, University of Illinois Bulletin No. 315,
August, 1939.

23. Jensen, V. P., Kluge, R. W., and Williams, C. B.
HIGHWAY SLAB BRIDGES WITH CURBS, LABORATORY TESTS
AND PROPOSED DESIGN METHOD, University of Illinois
Engineering Experimentation Station Bulletin
No. 346, 1943.

24. Ferguson, P. M.
REINFORCED CONCRETE FUNDAMENTALS, Second Edition,