THE OVERLOAD BEHAVIOR
OF
DAMAGED STEEL MULTIGIRDER BRIDGES

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ABSTRACT

This report describes the structural response of fractured steel multigirder bridges under loads from the design service level to the serviceability limit level. The research is conducted in two major phases, each employing separate analytical models of multigirder bridge behavior.

In Phase I the major factors effecting the behavior of damaged multigirder bridges are examined using a general purpose linear elastic finite element program. Detailed three dimensional models are developed including most of the structural detail of standard short, medium, and long span FHWA multigirder bridges. The multigirder bridge response is quantified in terms of deflection profiles in the longitudinal and transverse directions, girder lower flange stress, slab stress parallel and perpendicular to the steel girders, cross bracing stresses, and support reactions. This research provides bridge engineers with valuable information concerning: (1) the general behavior of multigirder bridges with a severely damaged main load carrying member, (2) the effect of span length on the response of a damaged multigirder bridge, (3) the response of a damaged multigirder bridge when overloaded, (4) the response of a multigirder bridge to limited damage to the tension flange and/or web, and (5) the effect of various support conditions on the response of a damaged multigirder bridge. A large amount of quantitative data has been provided in graphical and tabular form supporting the conclusions of this study.

The Phase II research investigates the response of damaged multigirder bridges beyond the limits of linear elastic behavior. An analytical model is developed which permits the complete overload analysis of fractured multigirder bridges. While this new modeling technique includes
only the bridge's primary load carrying members, it realistically incorporates inelastic stress-strain relationships, cracking and crushing of concrete, yielding and strain hardening of reinforcing and girder steel, buckling of flanges, and buckling of girder webs. This nonlinear model is employed in studying the response of damaged multigirder bridges to normal loads and overloads. The response is quantified in terms of load versus deflection, longitudinal and transverse deflection profiles, and girder lower flange stress. Failure areas are identified, and post-failure stress redistribution is monitored for the reinforced concrete deck and steel girders. The key quantitative findings of this phase are provided in graphical form to facilitate its use by bridge engineers.
1. INTRODUCTION.

1.1. Introduction.

Field observations of steel highway bridges indicate that the girders can be susceptible to fracturing. Steel girders with tension flange cracks perpendicular to the primary stress field are considered critical cases, requiring the immediate determination of the bridge's safety and serviceability. The subsequent closing of the bridge to make necessary repairs is a dreaded, but often necessary, action.

Throughout recent civil engineering literature, numerous examples have been cited of bridge structures having serious damage undetected for undetermined periods of time. One of the more dramatic and publicized occurrences was the 11 foot crack of a main girder on the I-79 bridge discovered in January, 1977 (Ref. 42). A large number of smaller structures have been identified which have functioned with significant damage. Csagoly and Sweeney have cited examples of substantial damage going undetected in highway (Ref. 9) and railroad bridges (Ref. 43), respectively. Fisher amassed information on cracks existing in nearly 150 different bridge structures in 20 states and Ontario (Ref. 14).

More recently, Daniels, et al, obtained and reviewed information from over 130 fatigue and/or fracture damaged steel bridges in the U.S., Canada, and Japan (Ref. 10). Detailed case studies were provided of the steel girder bridge damage for many of these bridges. Although moderate to severe cracking had developed in 13 bridge girders, and near full-depth fracture of four bridges, no collapse of a span had occurred. All the bridges remained in
service without undue problems until the damage was discovered.

These investigations confirm the existence of severe cracking on in-service highway bridges. It is also known that the majority of highway bridges are periodically subjected to overloads, i.e., live loads in excess of the service loads used in the design computations. With slight overloading, the structure may remain within the limits of linear elastic response. However, the overloading may propel its structural response to a level above the proportional limit, but yet still remain far below the collapse load. The fact that bridge structures with cracks are subjected to overloads make it imperative that bridge engineers know the reserve strength, load redistribution, and non-recoverable damage of these bridges.

1.2. Previous Research.

During the last decade, substantial progress has been made in determining the overload response of uncracked steel multigirder bridges.

Research by Wegmuller and Kostem (Refs. 47, 48, and 49) developed a technique for predicting the elastoplastic response of eccentrically stiffened plates. A significant development in this research was the use of layered elements to monitor the spread of yielding throughout the structure. Based on this work, Kulicki and Kostem (Refs. 25, 26, 27, 28, and 29) extended the model and the technique to incorporate eccentrically located reinforced or prestressed concrete beams. In this analysis the response characteristics of the concrete beams were realistically modeled, including the cracking and crushing of concrete and yielding of the reinforcing steel. Sub-
sequently, Peterson and Kostem (Refs. 36, 37, 38, and 39) further extended the analysis technique to accurately simulate the biaxial behavior of reinforced concrete slabs, and thus in the end, to reliably predict the overload response of concrete highway bridge superstructures. These studies provided the theoretical and analytical foundation for subsequent studies of steel highway bridges.

Extension of these concrete bridge studies to steel bridges began in earnest in the mid-1970's. Tumminelli and Kostem developed a general finite element formulation to be used to perform linear elastic analysis of composite single or multibeam, simple or continuous bridge superstructures (Refs. 44, 45, and 46). This formulation could include the effects of slip between slab and beams, shear deformation of the beams, and shear lag in the deck.

The works of Tumminelli/Kostem and Peterson/Kostem were integrated by Hall and Kostem to develop a model for predicting the inelastic overload response of intact steel multigirder bridges (Refs. 16, 17, 18, and 19). This model included the complexities in material behavior and losses in stiffness due to yielding, cracking, crushing, strain hardening, flange buckling, and web buckling.

In bridge engineering literature there still exists a major gap in the prediction of a bridge's response having existing damage. Heins and Hou used a simplified space frame finite element model to report on the effect of several bracing configurations on the maximum deflection of a multigirder system when a fracture develops in one of the girders (Ref. 20). Heins and Kato also used a finite element model to examine the effect of bracing on the load distribution of a two girder bridge system when one of the main girders was subjected to an induced flange crack (Ref.
21). Both of the above reports used linear elastic finite element models; neither addressed overloads.

Sangare and Daniels conducted a computer study of a steel deck truss bridge (Ref. 51) where post-elastic member behavior was considered. In the investigation, even though the bottom chord of one truss was assumed to be completely severed at midspan, all members of both main trusses remained elastic.

It appears that much work has yet to be done in quantifying steel bridge response when damaged and/or overloaded. This situation exists despite the availability of computer models which can replicate the sophisticated manner in which steel multigirder bridges behave. This research will attempt to fill this gap in the level of engineering knowledge in this critical area of concern to bridge engineers.

1.3. Purpose.

The purpose of the reported research is to fully investigate the structural response of fractured steel multigirder bridges in the region from design load levels to the upper limit of serviceability. The results will:

1. Provide an analysis tool which permits the bridge engineer to predict both the elastic and inelastic response of a damaged bridge superstructure in terms of live load versus deformation, material failure, and instability.

2. Allow bridge engineers to have a qualitative and quantitative understanding of actual performance of commonly used multigirder bridges after fractures have developed.
1.4. Scope.

Due to the complexity and range of the research, it has been divided into two major phases.

1.4.1. Phase I.

Phase I examined the major factors effecting the behavior of damaged multigirder bridges. Finite element models were developed for the most common types of simply supported highway bridges. These models were thorough three dimensional finite element models including as much of the actual structural detail as possible to include cover plates, cross bracing members, diaphragms, transverse stiffeners, curbs, and supports. Models included short, medium, and long multigirder bridges. Loads were kept within the range of commonly accepted loads and overloads. The multigirder bridge response was quantified in terms of deflection profiles in the longitudinal and transverse directions, girder lower flange stress, slab stress parallel and perpendicular to the steel girders, cross bracing stress, and support reactions.

Phase I provides qualitative and quantitative answers to the following questions of concern to bridge engineers:

1. How is multigirder bridge behavior effected by severe damage to a main load carrying member?

2. How does span length effect the response of a damaged multigirder bridge under normal loading?

3. How does the response of a damaged multigirder bridge change when overloaded?

4. If the damage level existing on a multigirder bridge is reduced from the most severe scenario, does the response change significantly? Can the new response be predicted?
5. How is the response of a damaged multigirder bridge effected by changes in the design support conditions?

Due to the sophistication of the model, the number of parameters to be analyzed, and the relatively low live load levels; the models for the Phase I research were developed using a general purpose linear elastic finite element modeling program.

The Phase I research is described in Chapters 2 and 3 of this report. Chapter 2 addresses the modeling of multigirder bridges for linear elastic damage analysis; Chapter 3 describes the results of the parametric study using these models.

1.4.2. Phase II.

The Phase I research identified several sources of nonlinear behavior for damaged multigirder bridges. To fulfill the study’s overall purpose, the Phase II research investigated the response of damaged multigirder bridges incorporating this nonlinear behavior.

The development of an analytical model for the nonlinear analysis of damaged multigirder bridges is a critical Phase II research task. This modeling technique must permit the performance of a complete overload analysis of fractured multigirder bridges. To realistically duplicate actual bridge behavior beyond the linearly elastic range, the new model must incorporate inelastic stress-strain relationships, cracking and crushing of concrete, yielding and strain hardening of the reinforcing and mild steel, buckling of flanges, and buckling of girder webs. Because of the complexity of the nonlinear damage technique and the high computational costs, only primary load carrying members (steel girders and reinforced concrete slab)
were incorporated into the model.

Following its development, the nonlinear model was employed to study the response of damaged multigirder bridges to loads and overloads. Load levels approached the structure's serviceability limit level. The response was quantified in terms of load versus deflection, longitudinal and transverse deflection profiles, and girder lower flange stress. Failure areas were identified, and post-failure stress redistribution was monitored for the reinforced concrete deck and steel girders.

Chapter 4 and 5 provide the details of the Phase II research. Chapter 4 describes the development of the nonlinear damage model; Chapter 5 presents the results of using this model for analyzing the nonlinear behavior of multigirder bridges.
2. MODELING OF COMMON MULTIGIRDER HIGHWAY BRIDGES FOR LINEAR ELASTIC DAMAGE ANALYSIS.

2.1. The Actual Structure.

The focus of the research was on the superstructures containing multiple welded A36 steel girders and reinforced concrete desk slab; designed for HS20-44 vehicle loading using the AASHTO Standard Specifications for Highway Bridges of the 1960's (Ref. 2). The cross section of the simple span - multigirder bridges analyzed is shown in Fig. 1. The figure is extracted from Ref. 13. This reference contains all pertinent details for the multigirder bridges investigated. These bridges were selected because they represent an excellent sampling of the type of interstate highway bridge structure in use during the last two decades. Some of the more critical structural details of these simple span bridges will be summarized in this section.

All bridges have six girders spaced at 94 inches center-to-center and a 44 foot wide clear roadway. Each has a 7.5 inch reinforced concrete slab designed to develop complete interaction with the main girders. The slab overhangs the outside girders of all the bridges by 41 inches. A 18 inch by 12 inch curb lies along both edges of the slab. The bridges' six plate girders are nonprismatic in nature with 3/8 inch thick webs.

To identify the effect of span length during Phase I of the research, span lengths of 100 feet, 140 feet, and 180 feet were analyzed. The major difference between the three spans is the geometry of their six parallel welded girders, and the location of cross bracing and other secondary members. Most of these differences are depicted
in Fig. 2, also extracted from Ref. 13. As seen in Fig. 2, the webs of the nonprismatic girders have depths of 66 inches, 94 inches, and 120 inches for the 100 foot, 140 foot, and 180 foot spans respectively. The dimensions of the upper and lower flanges of the welded girders vary along their lengths with the dimensions shown in Fig. 2. In addition, the number and location of cross bracing frames differ for each of the three different spans. The 100 foot bridge has five frames spaced at 300 inches; the 140 foot bridge has seven frames spaced at 280 inches; and the 180 foot bridge has nine frames spaced at 270 inches. The location and size of transverse and longitudinal stiffeners also differ as depicted in Fig. 2.

2.2. The Three Dimensional Model.

2.2.1. Overview.

Even though the field testing of actual structures is desirable, the conduct of the reported research in the field is impractical at best. Consequently, computer simulation employing finite element modeling was used throughout the investigation. In the finite element method of structural analysis the structure is divided into an assembledsemblage of discrete subunits called finite elements. These elements are interconnected at discrete node points. The behavior of each finite element can be described by the element stiffness matrix which relates node point forces to node point displacements. By stacking all of the element stiffness matrices and considering the applied node point loads and node point constraints, a set of equilibrium equations results. A detailed treatment of the finite element method as applied to structural analysis can be found
in numerous references. Several of these titles are referenced in this work (Refs. 5, 7, 11, 15, and 50).

Due to the complex nature of the expected response of the bridges, a full three dimensional model was required. The two major components of this three dimensional model are:

1. Two dimensional girders modeled with multiple elements over their length and depth as shown in Fig. 3.
2. A reinforced concrete slab modeled with multiple two dimensional plate bending elements over its length and width as shown in Fig. 4.

Joining these two major two dimensional components at right angles at their common node points creates a three dimensional structure.

In addition to the extremely detailed finite elements used to simulate the primary components, most of the secondary details of the bridge were also modeled. This includes components such as stiffeners, diaphragms, cross bracing, curbs, and supports; members which are normally excluded from analytic bridge models. With these secondary members added, the full finite element discretization of the bridge is shown in Fig. 5.

Even though the structure is symmetric about both the roadway centerline and midspan, the entire structure was modeled. This was necessary to allow for the application of unsymmetrical loads. By not invoking symmetry in the model, analysis of the effects of unsymmetrical support conditions was also possible.

2.2.2. Nodal Pattern and Numbering.

During this phase of the research, the global x, y, and z axes were oriented in the longitudinal, vertical,
and transverse directions respectively. The Phase I program used node numbers as the basis for numbering the nodal displacements. Hence, the bandwidth of the overall stiffness matrix depends upon the largest difference between any two external node numbers for a single element. To minimize the model's bandwidth, the nodes were generally numbered successively in the smallest dimensions first. Hence, nodal numbering was done successively in the y, then z, and then x directions.

Fig. 6 shows the nodal numbering along a transverse cut at the near bearing end of the bridge. The orientation of this cut is the same as the actual bridge plans previously seen in Fig. 1. From Fig. 6 the reader can see the three distinct layers of the bridge's model. The top layer is at the mid-depth of the concrete slab (and the girder's upper flange); the bottom layer at the girder's lower flange; and the middle layer is at the middle depth of the girder. The dependence of the model on the assumption of full composite action between the slab and girder is obvious.

On the right side of the bridge model, duplicate nodes were established near the outside girder at all node levels. Most of these duplicate nodes were defined in anticipation of creating fractures on the outside girder as will be discussed in section 2.3. Some of the duplicate nodes were created to maintain a convenient nodal numbering pattern.

The nodal pattern depicted in Fig. 6 was repeated at regular intervals along the longitudinal ("x") direction of the model. The longitudinal interval between adjacent transverse nodal patterns is 50 inches, 70 inches, and 90 inches for the 100 foot, 140 foot, and 180 foot models respectively. Near the midspan, these intervals were
reduced as seen in Figs. 3, 4, and 5 for better damage simulation in the vicinity of the girder fracture. This damage simulation will be discussed further in section 2.3.

2.2.3. Element Modeling.

The finite element analyses in Phase I were conducted using program SAPIV (Ref. 6). The structure was modeled using a combination of truss, beam, and plate bending elements from the SAPIV element library. A summary of the finite element model is listed in Table 1 and discussed below. All structural steel is A36 steel with an assumed modulus of elasticity of 29000 ksi and a Poisson's ratio of 0.30.

2.2.3.1. The Reinforced Concrete Slab.

The SAPIV plate bending element can simulate both in-plane and plate bending behaviors. The rationale for using this element for modeling the slab is obvious.

Full composite action between the 7.5 inch thick slab and the plate girder is assumed. That is, the translation and rotation of all node points common to the slab and the plate girder are assumed to be exactly the same.

The slab's width is 46 feet for all three spans of interest. This width is divided into twelve transverse elements: two elements between each of the six parallel plate girders, and one element overhanging each edge girder. The longitudinal length is subdivided into 30 elements. The length of these elements vary for each span, depending on their proximity to the midspan. Hence the slab is subdivided into the 360 elements depicted in Fig. 4. The modulus of elasticity of the slab material is assumed to be 3320 ksi (corresponding to a concrete strength of 3000 psi) and the Poisson's ratio is assumed to be 0.20.
2.2.3.2. The Plate Girder Web.

Plate bending elements were used for the plate girders' webs because significant transverse deflection and beam-slab interaction was expected, especially after damage initiation.

Fig. 3 shows each girder web modeled with two plate bending elements through the web depth, and 30 elements in the longitudinal direction, for a total of 360 plate bending elements. The dimensions of these 3/8 inch thick elements vary for each of the spans; and depending on their proximity to the midspan.

2.2.3.3. The Plate Girder Flanges.

Since the girder web was modeled as plate elements, the lower flange was modeled as a beam element to allow more realistic bending action than could be provided with truss elements.

In the case of the upper flange of the plate girder, it was unnecessary for this compression flange to be modeled using beam elements. It is assumed that this flange would act compositely with the reinforced concrete slab, and therefore have less global effect on the structural response.

Due to the nonprismatic nature of the plate girder, each bridge model used three different sized elements corresponding to the actual flange of the structure. Thirty elements were used across the length of each of the girders for their upper and lower flanges. Hence a total of 180 beam elements were used to model the lower flange; and 180 truss elements were used to model the upper flange.
2.2.3.4. The Curb.

The curb was modeled with beam elements over its full length on both edges of the bridge slab. This permitted the realistic addition of its significant bending stiffness to the overall structural stiffness.

Each of the two curbs were modeled with 30 beam elements for a total of 60 elements. Since the center of gravity of the curb is actually 12.5 inches above the central plane of the slab, the moment of inertia of the curb was increased by an amount equal to its area multiplied by 12.5² inches squared. The modulus of elasticity of the curb material is assumed to be 3320 ksi and the Poisson's ratio is assumed to be 0.20.

2.2.3.5. Secondary Members.

The transverse stiffeners, cross bracing, and diaphragms were modeled as truss elements. As secondary members, it was felt that one dimensional truss elements were sufficient to capture these members' gross effect on the overall structural behavior.

The transverse stiffeners were positioned as close as possible to their actual location on the structure. In some cases the model does not correspond exactly with the actual structure since the transverse stiffeners were placed at the nearest location where nodes already existed.

The cross bracing and diaphragms were placed at their true locations with respect to the longitudinal axis of the bridge. However, they are modeled to frame between the upper and lower layers of node points. Hence, the model differs slightly from reality as seen by the actual framing points depicted in Fig. 1. These differences should have a minimal effect on the overall structural be-
The number of secondary members varied widely for each of the different spans. These differences are highlighted in Table 1.

2.2.3.6. Superstructure Supports.

Boundary elements (one dimensional truss-like elements) were used at the bearing ends to capture all applicable reactions at the supports. Three boundary elements were placed at the two bearing ends of each of the six plate girders for a total of 36 boundary elements. When activated, these boundary elements could prevent the translation of the lower girder flange in the direction specified. The use of these boundary elements in the study of the effect of varying support conditions will be discussed in section 2.5.

2.2.4. Model Validation.

The integrity of the entire SAPIV model was examined by producing a large scale plot of the entire bridge model using program SPLT (Ref. 23). While a successful SPLT plot indicates proper geometric modeling of the structure, it certainly does not guarantee the correct overall modeling of a structure. For this reason, it is mandatory that the model be subjected to loadings to examine how it responds compared to non-finite element solutions.

Two test loads were applied to each of the three SAPIV models to compare with manual computations. The first was a "knife-edge" load of 100 kips applied vertically downward to the slab at the midspan of each of the six girders. The second was a uniform pressure of 0.000906 ksi applied vertically downward on the entire slab surface. This numeric value was chosen to correspond to a uniform...
line load of 6 kips per linear foot of bridge (or 1 kip per linear foot of girder) for comparable manual calculations.

The test loading results were very acceptable for all three bridge spans. The reactions at the bearing supports were reviewed for all SAPIV models to ensure that the results were symmetrical and logical. For all test loads on all models, this was the case.

The deflection and stress at representative points on the girder and slab of the SAPIV model were examined. These deflections and stresses were compared to calculations based on standard one-dimensional composite beam theory. These manual calculations had to account for the nonprismatic nature of the composite girders. The comparisons were acceptable for all of the bridge models. Table 2 compares the results using the SAPIV model and composite beam theory of the 100 foot bridge with the uniform load. All of the other models give similar results. The reader is cautioned from putting too much weight on the accuracy of the manual calculations. While composite beam theory has been shown to yield acceptable engineering results for simple loads on simple structures, there is no reason to assume that this theory is more accurate than a properly modeled bridge using finite element analysis. The sole reason for presenting the results is to demonstrate the compatibility of the SAPIV results with non-finite element method results.

Because of the close results between the two independent analysis methods for these simple loads, it was concluded that the SAPIV model appears to adequately represent the real structure as far as overall behavior and major component response.
2.2.5. Model Summary and Conventions.

Table 1 is a summary of the number and types of elements used in the three bridge models together with some modeling statistics. Before proceeding with the discussion, some useful conventions and nomenclature will be listed for use throughout the remaining discussion of the Phase I procedures and results:

1. The $x$, $y$, and $z$ coordinate axes are oriented in the longitudinal, vertical, and transverse directions as seen in Fig. 5. The origin of the coordinate system is at the mid-height of the slab, directly above the edge girder (not at the edge of the slab).

2. The six parallel girders will be referred to as $G_1$ through $G_6$. $G_1$ lies in the plane defined by $z=0$; $G_6$ in the plane defined by $z=470$.

3. The symbol "L" will refer to the span of a specific model. Hence, $L$ equals 1200 inches, 1680 inches, and 2160 inches for the three bridge spans respectively.

4. The bridge "centerline" refers to the plane defined by $z=235$; that is, the plane parallel and midway between the webs of $G_3$ and $G_4$.

5. The bridge "midspan" will refer to a plane defined by $x=L/2$.

6. In some discussions the bridge will be referenced from the perspective of a viewer standing in the plane defined by $x=0$ and looking toward the other end of the bridge (defined by the plane $x=L$). As such, the "near" end will be defined by the line $x=0$, and the "far" end by the line $x=L$. The "left" edge of the bridge will be the edge nearest and parallel to $G_1$; the "right" edge of the bridge will be the edge nearest and parallel to $G_6$.

7. To avoid confusion and to provide the most general results of use to bridge engineers, all live loads
will be applied without impact factor. That is, all vehicle loads will be applied as static loads with no consideration for dynamic effects. Since an accepted manner of incorporating the impact effect at a given location is by multiplying the live load by a constant which is a function of span (see paragraph 3.8.2.1 of Ref. 3), this can be accomplished easily by the reader wishing to include these effects.

8. This study uses analytic models to predict the response of actual multigirder bridge superstructures. The source of most of the results contained in this report will be these analytic models, and not experiments on actual bridges. As such, the use of the words "bridge", "model", "superstructure", and "structure" are synonymous within the context of this report. Exceptions to this word usage will be clearly identified in the report narrative.

2.3. Damage States.

2.3 Overview.

In Phase I of the research, four different damage states were investigated. In each of the four cases the damage was invoked at the midspan of G6. The midspan was chosen since it represents the location of the maximum dead load moment, as well as being close to (or at) the location of the maximum live load moment. To maximize the effects of damage, an edge girder was selected since it has fewer neighboring girders to "share" the effects of damage. Damaging the edge girder also maximizes the transverse damage effects. These transverse effects can be considerable, as will be seen in the next chapter. As will be discussed in section 2.4, the live load is positioned to
create the maximum effects at the location of this damage. In this section of the report, the procedure for modeling the various damage states will be discussed.

The four different structural states investigated in this phase of the research include:

1. The undamaged or intact structure.
2. The structure with a simulated lower flange fracture at the midspan of the exterior girder.
3. The structure with a simulated lower flange fracture and half web depth crack at the midspan of the exterior girder.
4. The structure with a simulated lower flange fracture and full web depth fracture at the midspan of the exterior girder.

It is important to note that this study does not include crack tip stresses and fatigue crack propagation. For example, in this Phase I study of the girder with a severed lower flange, there would certainly be stress concentrations and crack penetration. This study does not examine this. In a sense, this work "freezes" the damage and conducts the analysis of the structure in this "frozen" state.

As previously stated, duplicate nodes were located in the vicinity of G6 at each transverse nodal pattern to help model the damage conditions listed above. These duplicate nodes were assigned the exact same coordinate position as other nodal points. However, the duplicate nodes were created in anticipation of modeling a given damage state. When the model is intact, the duplicate nodes have no function and their degrees-of-freedom are totally restrained.
Fig. 7 is a representation of a small portion of the model of G6 in the vicinity of the midspan. The line defined by nodes 472, 473, and 474 is in the midspan plane. The line defined by nodes 442, 472, and 502 is at the level of the upper girder flange which coincides with the mid-thickness of the concrete slab. Nodes 444, 474, and 504 are at the level of the lower girder flange; nodes 443, 473, and 503 define the midheight of the girder web. The parenthetical numbers show duplicate nodes, i.e. nodes which have the same coordinate position as active nodes. Hence, node 478 has the exact same coordinate position as node 473; nodes 479 and 480 have the same coordinate position as node 474.

The bridge is modeled by placing discrete elements between node points. Some of the elements located in the vicinity of the G6 midspan are depicted in Fig. 7. These elements are identified by the "boxed in" alphabetical symbols for easy reference. Elements UFA and UFB depict the two truss elements representing the upper girder flange on each side of the G6 midspan; LFA and LFB are the two beam elements representing the lower girder flange neighboring the G6 midspan. UWA, UWB, LWA, and LWB are the four plate bending elements defining the girder web adjacent to the G6 midspan.

For the intact bridge model, the truss, beam, and plate bending elements are connected to the non-parenthetical nodes shown in Fig. 7. For example, plate bending element LWA is defined by the nodes 443-473-474-444; beam element LFB is defined by the nodes 474-504. For the intact model, the duplicate nodes (478, 479, and 480) are not connected to any element on the structure; and their degrees-of-freedom are fixed.

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2.3.2. Modeling the Girder for Different Levels of Damage.

The degrees-of-freedom of nodes 479 and 480 were activated for the modeling of a lower flange fracture at the midspan of G6. Node 478 remains inactive for this state of damage. Note that the original coordinate position of these two nodes is the same as node 474. The lower flange fracture was modeled by connecting element LFA from node 444 to node 479; and element LFB from node 480 to node 504. Elements LWA and LWB were defined in the exact same manner as they were defined for the intact structure, i.e., by nodes 443-473-474-444 and 473-503-504-474 respectively. Hence, a discontinuity was forced at the previous junction between elements LFA and LFB, simulating a lower flange fracture in the girder.

Modeling a half web depth crack only requires the activation of the degrees-of-freedom of node 479. Duplicate nodes 478 and 480 remain inactive and unused for this analysis. Plate bending element LWA is defined by nodes 443-473-474-444; and its adjacent lower flange by nodes 444-474. Plate bending element LWB is defined by nodes 473-503-504-480; and its adjacent lower flange by nodes 480-504. Hence, full continuity is maintained between the lower web and its adjacent lower flange; yet the continuity is broken between the lower web/flange elements on either side of the midspan.

Duplicate nodes 478 and 479 are needed for the modeling of a full web depth fracture; node 480 remains inactive and unused for this analysis. On the near side of the midspan on G6, plate bending element UWA is defined by nodes 442-472-473-443; LWA defined by nodes 443-473-474-444; and the lower flange by nodes 444-474. On the far side of the midspan of G6, plate bending element UWB is defined by nodes 472-502-503-478; element LWB by nodes 478-
503-504-479; and the lower flange by nodes 479-504. Hence, even though the upper flange maintains its full continuity over its full length and with its adjacent web elements, the continuity is broken between all web and lower flange elements on either side of the midspan of G6.

2.3.3. Damage State Nomenclature.

Many of the tables and figures to be presented in subsequent sections of this paper will refer to the different damage states described above. The abbreviations to be used for the damage states will be as follows:

1. "IN"--- the undamaged or intact structure.
2. "FL"--- the structure with a simulated lower flange fracture at the midspan of G6.
3. "WB"--- the structure with a simulated lower flange fracture and half web depth crack at the midspan of G6.
4. "WW"--- the structure with a simulated lower flange fracture and full web depth fracture at the midspan of G6.

2.4. Loads.

2.4.1. General.

Each bridge was subjected to seven different loads. The load cases were the HS20-44 design vehicle, a 4-axle 128-kip dolly load, a Pennsylvania Department of Transportation 8-axle 4-kip permit vehicle, the structure's dead weight, and dead load plus the indicated live loads. The seven load cases will often be referenced by the following load case numbers and abbreviations:

1. Load Case #1 (LC1): HS20-44 vehicle without
dead load.

2. Load Case #2 (LC2): 128-kip Dolly vehicle without dead load.

3. Load Case #3 (LC3): 204-kip PennDOT Permit vehicle without dead load.

4. Load Case #4 (LC4): HS20-44 vehicle with dead load.

5. Load Case #5 (LC5): 128-kip Dolly vehicle with dead load.

6. Load Case #6 (LC6): 204-kip PennDOT Permit vehicle with dead load.

7. Load Case #7 (LC7): Dead load alone.

The assumed axle loads and axle spacing for each of the live loads are depicted in Fig. 8. Each axle load is assumed to be equally divided between wheel loads which are 6 feet apart.

Not only do these live loads differ in their gross weights, they also have significant differences in their longitudinal load distributions. While the 204-kip PennDOT Permit vehicle is almost three times as heavy as the HS20-44 vehicle, it is distributed over about twice the area. In contrast, two 128-kip Dolly loads can fit within the footprint of a single HS20-44 vehicle. As such, these three vehicles represent an excellent range of vehicles in both weight and dimensions. It is hoped that most vehicles crossing highways today will fit within the range of vehicles used in this study.

2.4.2. Longitudinal Placement of the Live Loads.

As stated in Chapter 1, the purpose of this research is not to design multigirder bridges in accordance with accepted bridge engineering practice. This research presupposes the presence of extensive damage at a critical
location (the midspan of an edge girder), and seeks to study the worst effects of realistic live loads on the structure's response. As such, the research does not include detailed calculations of influence lines and surfaces as in normal bridge design. The author used engineering judgment in determining the ideal live load location to ensure the worst practical effects in the vicinity of the anticipated damage location.

The maximum live load moment for a simple span loaded with a single vehicle is achieved by bisecting the line joining the vehicle's center of gravity and the axle nearest the center of gravity, with the midspan of the bridge. When doing this, the maximum moment occurs under the axle nearest the center of gravity. However, for this research it is desirable to have the maximum moment occur at the damage location, i.e., the midspan. The midspan is also the location of the maximum dead load moment for a simple span with uniform dead load. Hence, all live loads were positioned longitudinally so that the maximum moment occurred at the midspan. This was accomplished by placing the axle nearest the center of gravity over the midspan.

2.4.3. Transverse Placement of the Load.

Determining the optimal transverse location of the live load is not as straightforward as the longitudinal placement. The exact transverse distribution of live load is uncertain. A short study was conducted to produce limited analytic data concerning this topic (Ref. 52).

In this study, the SAPIV program was used to determine the response of an intact multigirder bridge to eight different transverse positions of an HS20-44 vehicle load. These positions varied from being close to the bridge's curb to being symmetrical about the bridge's cen-
terline. The longitudinal position of all eight loads were as described in the previous section. For each of the load positions, the maximum deflection and maximum lower flange stress was examined of each of the six parallel girders. This limited study showed that the maximum deflection and girder stress were achieved by placing the live load as close to the edge of the bridge as possible. As expected, the maximum absolute effects were created at G6, the edge girder.

This limited study confirmed what engineering judgment would expect. Hence, for the purpose of this study all live loads were positioned transversely so that they were as close to the bridge's edge as practically possible. The vehicle's outside wheels were positioned 24 inches from the edge of the lane width in accordance with current practice (Ref. 3). Since the curb of the bridge is 12 inches wide, this meant that the vehicle's outside wheels were place 36 inches from the edge of the model. The author considered this to be the closest practical position of the vehicle to the outside edge of the bridge.

Figs. 9, 10, and 11 are plan views of the 100 foot bridge model with HS20-44 vehicle, 128-kip Dolly vehicle, and 204-kip PennDOT Permit vehicle loads respectively. The figures are generally to scale based on the coordinate axes shown. The six girder lines are identified, together with the position of the live load based upon the discussion of this section. In all cases, the vehicle's outside wheels are slightly to the outside of G6 and the vehicle's inside wheels are between G5 and G6. The large differences between the axle spacing for the three live loads is portrayed very dramatically in these three figures.

In summary, the live loads are positioned so that
the maximum midspan response is achieved in the vicinity of the anticipated damage. The vehicles are assumed to be oriented with their front axles closest to the bridge's far edge \((x=L)\), their rear axles closest to the near edge \((x=0)\). The damaged girder is G6; the live load is positioned as close as practically possible to the edge adjacent to G6. The wheel loads which are not located directly over a nodal point were applied in an equivalent static manner to the neighboring nodal points.

2.5. Support Conditions.

2.5.1. Overview.

Consideration of the realistic analytical modeling of support conditions to represent the actual physical situation is often ignored in engineering practice. For elementary analyses, civil engineers often idealize simple spans as one dimensional structures with length and certain cross sectional properties. The one dimensional beam's external load and support reactions are assumed to be at the centroid of the section. For simple spans, a one dimensional beam model normally assumes translational restraints in the vertical direction at both ends of the beam, and translational restraint in the longitudinal direction at one end. These restraints are assumed to be at the cross section's centroid. If only vertical external loads are applied to the beam, the longitudinal reaction at the one end of a simple span is zero by overall equilibrium considerations. Engineers have found that these idealizations yield acceptable results for many engineering problems.

However, the actual three dimensional situation is very different from this one dimensional idealization.
Figure 12 shows the bearing details of typical multigirder bridge extracted from Ref. 13. The bridge's actual restraints are not located at the section's centroid, but at the outer surface of the lower girder flange. The bridge is designed with a fixed bearing at one end of all six girders; an expansion bearing at the other end of all girders.

From Fig. 12 it is obvious that both the fixed and expansion supports restrain the lower flange of the girder from translating vertically. In addition, the fixed bearing support is designed to restrain the lower flange from translating longitudinally; the expansion support is designed to allow a limited amount of longitudinal translation. For both fixed and expansion supports, translation in the transverse direction is restrained.

Even though a typical multigirder bridge is designed for the bearing details shown in Fig. 12, the physical situation at the bridge site is often extremely different. The effects of age, weather, and workmanship often create conditions far different from the design. It is common to find expansion supports which have become "frozen" due to rust, deterioration, or poor maintenance. Fixed supports can slip from their anchors and become unrestrained in the longitudinal direction, especially in a bridge with severe damage. Hence, it is conceivable that fixed supports become expansion supports and vice versa during normal operation.

This research attempts to capture the complex nature of the three dimensional response of intact and damaged multigirder bridges with loads that are unsymmetrical in both the longitudinal and transverse directions. To accomplish this, the three bridge models must take the actual bridge support into consideration. The support as-
Assumptions of simple beam theory are not adequate.

As stated previously, three boundary elements have been positioned at both ends of all six girders. These boundary elements allow all ends of all girders to be restrained from translation in the vertical (y), longitudinal (x), and transverse (z) directions. These restraints are located at the lower girder flange to properly duplicate their actual physical location on the structure. These boundary elements can be used or "deactivated" depending upon the type of support condition the user wishes to simulate. This research will study the three different combinations of support conditions detailed in the following sections.

2.5.2. Expansion Supports at Both Ends.

The one dimensional model of a simple beam under vertical loading develops no support reactions in the longitudinal direction. This is the analytical model often used by bridge engineers in their initial design phase. It is also the model of the design bridge if the fixed supports have slipped off their anchors.

Hence, one of the models investigated will be the bridge with minimal longitudinal restraint. Of the twelve boundary elements oriented in the longitudinal direction at the two ends of the six girders, only one will be activated. Under the vertical dead and live loads, this sole longitudinal reaction will always have a zero value. Its only purpose is to prevent a mathematical singularity in the global stiffness formulation of the structure. All vertical and transverse boundary elements will be activated; that is, the bridge will be restrained from translating vertically and transversely at both ends of all six girders.
Intuitively, the bridge engineer would expect that these boundary conditions would allow the largest amount of longitudinal bending and, hence, maximize the normal stress in the longitudinal direction. This was a prime motivation behind using this support condition. The validity of this intuitive feel was investigated in the next chapter.

2.5.3. Mixed Boundary Conditions.

A second support condition investigated was the one dictated by the design as viewed in Fig. 12. All six girders will have expansion supports at the level of the lower flange at their near end, and fixed supports at their far end. Those boundary elements oriented in the longitudinal direction at the near end of the bridge are deactivated; all other boundary elements are activated.

2.5.4. Fixed Supports at Both Ends.

The last support condition to be investigated will have longitudinal translation restrained at both ends of all six girders. This could occur if the designed expansion supports became "frozen" as previously discussed. This is the highest degree of restraint of all three support conditions investigated. All 36 boundary elements are activated in this model.

2.5.5. Terminology.

In summary, three different support conditions will be investigated in this study. In all three cases, both ends of the steel girders are restrained from translation in the vertical and transverse direction. The amount of longitudinal restraint differs in the three cases. All restraints are applied at the level of the lower flange of
the steel girders.

In future graphs and discussion, the three support cases will often be referenced by the following terms and abbreviations:

1. **Expansion-expansion (EE):** as described in section 2.5.2.
2. **Expansion-fixed (EF):** as described in section 2.5.3.
3. **Fixed-fixed (FF):** as described in section 2.5.4.

The reader is warned against confusing the use of the word "fixed" in this work with the common use of "fixed" in one-dimensional beam theory. In beam theory, the word often applies to a rotational restraint about the major bending axis. For this research, no explicit rotational constraints were imposed on the ends of the girders. In this study, "fixed" assumes the common bridge engineering definition described previously and depicted in Fig. 12.
3. PARAMETRIC STUDY OF MULTIGIRDER BRIDGES USING LINEAR ELASTIC ANALYSIS.

3.1. Introduction.

3.1.1. Overview of the Parametric Study.

The previous chapter described the modeling of three multigirder bridge models for linear elastic damage analysis. Details were provided on how the model was developed, tested, loaded, and supported. The technical aspects of how various levels of damage were modeled were described. In addition, various conventions and terminology to be used in this chapter were itemized.

This chapter provides the results of the many different finite element analyses conducted using these three bridge models. The underlying goal of these analyses was to qualitatively and quantitatively describe how the performance of multigirder bridges is effected by extensive damage to a critical structural component. In order to pursue this primary goal, the study examined these significant parameters effecting intact and damaged multigirder bridge response:

1. The effect of varying the live load type.
2. The effect of varying span length.
3. The effect of intermediate levels of damage.
4. The effect of varying the support conditions.

This chapter initially examines the major response changes when an intact multigirder bridge becomes severely damaged at a critical location. It will then proceed to detail the results of evaluating the four parameters itemized above.
3.1.2. Data Gathering Procedure.

The finite element analyses were conducted using program SAPIV on a CYBER 850 computer running the NOS Version 2.4.3 operating system. The smallest bridge model has 974 nodes, 1387 elements, and 4309 degrees of freedom. When the program was modified to increase the COMMON block size up to a practical limit such that the computer runs were not in deferred mode, a single run required about 250 CPU seconds. Due to the large size of the model and the number of load cases; voluminous output resulted for each unique structural configuration. For example, the output file for the intact 100 foot bridge with expansion-expansion supports is 552 pages long. Twenty four output files of this length had to be produced and analyzed to gather the data for this chapter.

"Postprocessing" programs were developed to expedite the data reduction and to strip the "nonessential" data from the output files. This postprocessing program also converted element forces and moments into normal and shear stresses based on the elements' geometry. The select, but still extensive, data was imported into LOTUS 1-2-3 (Ref. 34) spreadsheets. These spreadsheets permitted the tabular and graphical display of the results. Through inspection of the tables and graphs it became possible to identify the trends in the structural response. This approach permitted selection of specific data from the output for further study. Without the use of this approach, making sense of the reams of printout would have been an extremely time-consuming and error prone task.

3.1.3. Primary Items of Interest.

Engineering judgment was used to determine what portions of the voluminous output data were of greatest use
to bridge engineers. Only by limiting the items of interest could a digestible report be produced.

It was felt that the following analysis items would be of the most use to bridge engineers in understanding intact and damaged multigirder bridge response:

1. The deflection profile. Deflection profiles were developed for many simulated longitudinal and transverse cuts of the structure. These deflection profiles were based on vertical displacements at the upper (or slab) level nodal points. By doing this, more data points could be generated for transverse cuts of the structure. These profiles are important to bridge engineers since they give the best "feel" for how the structure physically moves in response to a given structural and loading configuration.

2. The normal stress in the lower girder flange. The lower girder flange stress was monitored at multiple locations on all six steel girders. The girders are the primary longitudinal transmitters of load to the supports. High tensile stresses develop in the lower flange of multigirder bridges which serves as a prime area of interest for bridge engineers.

3. Normal stress in the slab in the longitudinal directions. The slab carries compressive stresses resulting from the major axis bending of the composite beam of an intact simple span bridge. However, if a steel girder were to suffer severe damage, the slab adjacent to the damaged girder carries the high tensile stresses normally developed in the steel girder. Once, this study monitored normal stresses in the slab in the longitudinal direction at the lower slab surface. The lower slab surface was selected for monitoring as a probable location of high tensile stresses for the damaged structure. It was felt that these
stresses were of more importance to bridge engineers than the high compressive concrete stresses likely to occur in the top surface of the slab.

4. Normal stress in the slab in the transverse direction. Due to the unsymmetrical live load location, tensile stresses were expected in the upper slab surface in the transverse direction. These stresses were expected to be amplified after damage occurred at the midspan of an edge girder and, therefore, would be of interest to bridge engineers.

5. Cross bracing stresses. Cross bracing stresses are often ignored in analytic bridge studies. However, it was felt that these secondary members might play a significant role in the redistribution of the load once damage was invoked. These stresses were monitored to get some perspective of their contribution in load redistribution.

6. Support reactions. A significant portion of the Phase I study dealt with the effect of various support conditions on the response of intact and damaged multigirder bridges. Quantifying the reaction forces at each support is critical in understanding how support conditions affect the overall bridge response.

In future sections of this report, the author will qualitatively and quantitatively describe the response of multigirder bridges to various parameters. Most of the data cited in this report is derived from the analysis of graphical data generated from the finite element runs. When appropriate, the author will reference the graphical basis for the data being presented by citing a graph number either explicitly in the narrative or as a parenthetical remark immediately following the referenced data.

The reader is reminded that the data for this
section is based upon a linear elastic finite element model. It is obvious in several cases that the response will propel the structure into the region of nonlinear response as a result of the probable cracking/crushing of the concrete slab and failure of cross-bracing members. Hence, some of the results must be interpreted more qualitatively than quantitatively.

3.1.4. AASHTO Allowable Stresses.

The results in the subsequent sections are presented with only infrequent comparisons to the allowable stresses based upon current specifications. This is done intentionally to allow bridge engineers to use the results in an absolute sense without the possible confusion introduced by varying factors of safety. However, as a frame of reference, the following allowable stresses extracted from the current bridge specification (Ref. 1) should be kept in mind as the results are reviewed:

1. The allowable stresses based upon AASHTO "inventory rating" loads (i.e. normal design loads) are:
   a. 20 ksi for a tension member fabricated with A36 steel.
   b. 20 ksi for the extreme fiber tension in an A36 girder.
   c. 20 ksi for the extreme compression in an A36 girder when the compression flange is fully embedded in concrete.

2. For bridge rating, AASHTO makes provisions for heavier than normal loads governed by special permits (Ref. 1). These loads are referred to as the "operating rating" loads. For structures loaded with "operating loads", AASHTO permits the allowable stresses to be up to 35% higher than the "inventory rating" allowable stresses.
Since the 128-kip Dolly vehicle and 204-kip PennDOT Permit vehicle loads are generally considered to be permit-type loads, they can take advantage of the "operating rating" provisions of AASHTO. Specifically, the allowable stresses based upon the AASHTO "operating rating" loads is increased from 20 ksi to 27 ksi for the three cases previously specified.

3. The AASHTO allowable stress of compression members must also consider stability of the member. No single allowable stress can be stated for this type of member. The allowable stress for specific compression members will be addressed later in this report.

3.2. The Response of a Multigirder Bridge to a Full Web Depth Fracture.

3.2.1. Overview.

In this section, the study will examine the overall changes in the response of a multigirder bridge before and after damage occurs. The words "damaged", "fractured", or "cracked" in this section refers to the most severe damage state previously discussed in Chapter 2. That is, the imposed damage will consist of severing the bottom flange and the full web of the exterior girder (G6) at its midspan. While it is expected that rarely would such a fracture ever develop without being noticed, this condition serves as a useful upper bound. If the bridge can survive such serious damage, it can be assumed that lesser damage to a main girder will not cause the immediate collapse of the superstructure.

Since the presence and absence of damage is the main parameter to be studied in this section; all other
parameters will be kept constant. The basic bridge of interest in this section will be a 100 foot span with expansion-expansion supports, and HS20-44 vehicle loading.

3.2.2. Vertical Deflection.

Vertical deflection profiles of each of the main girder of the 100 foot bridge with intact and fractured girder are shown in Figs. 13 and 14, respectively. The location along the bridge's longitudinal (x) axis has been normalized with respect to the span length. The midspan live load deflection of the exterior girder almost doubles (93% increase) when damaged. Therefore, the apparent damaged stiffness is about half of the intact stiffness. The fractured girder's deflection profile is similar to two straight lines intersecting at the midspan. Its apparent discontinuous slope at its midpoint is similar to the profile characteristic of an internally hinged beam. In contrast, the curvature of the displacement profile of the interior girders is quite smooth, indicating the major contribution of the bridge deck in the lateral distribution of live loading.

Figs. 13 and 14 clearly show the upward deflection of the girders farthest from the vehicle resulting from the live load. There is much less change in the remote girders as a consequence of damage. For G3, G4, and G5, the increases in midspan deflection were 38%, 55%, and 68% respectively.

The deflection profile along transverse cuts of the bridge at one-third span and midspan confirm these same points (Figs. 15 and 16). These transverse profiles show the large transverse curvature in the vicinity of the centerline of the bridge and near the edge of the bridge.

The total load results (live load plus dead load)
show the same general patterns, except that the girder upward deflections are harder to discern (Figs. 17, 18, 19, and 20). Under dead and live load, the maximum deflection of G6 increase by about 85% (almost doubles) after damage occurs. This is slightly less that the percentage increase (93%) when only live load is considered.

Under dead load alone, the maximum deflection of the edge girder increases about 80% as a result of damage. Also, there is an upward deflection of G1 and G2 as a result of damage alone (and not as a result of live load). Specifically, there is a deflection decrease of 12% and 4% in G1 and G2 respectively.

3.2.3. Lower Flange Stress.

In Figs. 21 and 22 the variation of the lower (tension) flange stress can be seen for the six girders of the 100 foot multigirder bridge for both the intact and damaged configurations. The four locations of sudden stress change are due to the change in the cross sectional properties of the nonprismatic girders.

The maximum live load stress in the exterior girder (G6) is reduced from 4.2 ksi (at midspan) to about 2.0 ksi (at quarter-span) after fracturing. As expected, there is a dramatic rise (from 2.8 ksi to 5.8 ksi) in the maximum stress of the first interior girder next to the fractured girder during the same process. It is evident that the loss of stiffness in the exterior girder results in about a 40% increase in the maximum stress in the structure. The live load stress in G5 more than doubles (increases 105%) when damaged. Figs. 21 and 22 clearly show the load distribution mechanism. The large moments originally carried by the exterior girder are shifted to the adjacent girder. The pronounced redistribution of the live load is obvious.
These live load stresses in G5 increase its susceptibility to fatigue problems, especially at locations of cover plate end welds and the welds attaching intermediate stiffeners.

If one includes total (dead plus live) load effects, there is a decrease in the lower flange stress in G1 and G2, unchanged stress in G3, minor increase in G4, and large (80%) increase in G5 after damage occurs (Figs. 23 and 24). If dead load effects are considered only, the maximum stress in G5 increases by about 72% when G6 is damaged. Correspondingly, there is a 15% decrease in the maximum stress in G1 as a result of damage in G6. Again, the damage causes an upward deflection at the remote girders, independent of the live load.

3.2.4. Normal Slab Stress in the Longitudinal Direction.

Figs. 25 through 32 show the normal stresses (membrane plus bending stresses) at the bottom of the deck slab in the direction parallel to the girders. Figures are not included for the slab stress in the vicinity of G1 and G2. Neither of these girders have lower slab live load stresses exceeding 20 psi while either intact or damaged.

For the intact structure, the stresses away from the damage have a smooth parabolic shape (Figs. 25 and 26). After fracturing these stresses only increase by about 30%; in no case exceeding 100 psi.

Near the loaded area there is a big jump in the longitudinal slab stresses after damage. For example, maximum live load stress changes from 100 psi in compression to 600 psi in tension near G6. This indicates that there will be cracks perpendicular to the girders at midspan near the damage (Figs. 27 and 28). From Figs. 25 through 28, it can be seen that damage to the structure has significant effects only in the middle quarter of the slab, and to the
damaged side of the centerline.

Unlike the previous observations concerning vertical deflection and lower flange stress in sections 3.2.1 and 3.2.2, the effect of damage can be considerably more than 100%. In the near vicinity of the damage (within an approximate 10 foot radius), there is a drastic change in both the magnitude and direction of the stress. Outside this radial area, change in stress as a result of damage stays below 100%.

Adding dead load stresses does not change these trends. The major difference is the increase of the maximum tensile stress to 1700 psi in the vicinity of damage, indicating major cracking problems (Figs. 29, 30, 31 and 32).

3.2.5. Normal Slab Stress in the Transverse Direction.

Figs. 33 through 35 show the top fiber live load slab stresses in the direction perpendicular to the girders. These figures show the stress along transverse lines at various distances along the bridge span.

Even at locations away from the loaded and fractured area, the effects of the damage can be noted (Fig. 33). However, the magnitude of the live load stresses are very low even at about one-third of the span, never exceeding 100 psi (C) or (T) (Fig. 34).

These stresses grow considerably as the loaded and fractured area is approached. Near midspan there is an almost five-fold increase in the live load tensile stress for the damaged bridge (Fig. 35). Dead load further increases the peak stresses, indicating that cracks will certainly develop in the slab parallel to the girders (Figs. 36, 37, and 38). At midspan, these cracks are most likely to occur in the top surface between G4 and G5, and at the
bottom surface near G6.

3.2.6. Cross Bracing Stresses.

Whether intact or damaged, the highest cross-bracing stresses are located across the midspan of the 100 foot multigirder bridge with expansion-expansion supports. The members discussed below are located along the transverse line across the midspan of the model. The data used in this subsection of the report is based upon the tabulated data of Table 3. In Table 3 and 4, "TYPE" refers to whether the member is a diagonal ("/" or "\") or horizontal ("-") cross-bracing member. As a means of comparing this data with accepted allowable stress levels used in bridge engineering practice, the following was computed in accordance with Ref. 1:

1. The allowable compressive stress based upon the AASHTO inventory rating (i.e. normal traffic) is approximately 8.1 ksi and 12.9 ksi for the horizontal and diagonal cross-bracing members respectively. These maximum allowable compressive stresses incorporate a factor of safety of 2.12.

2. The maximum allowable tensile stress for either a horizontal cross-bracing member or a diagonal cross-bracing member is 20.0 ksi.

The live load stress (LC1) for the intact structure is small in all members; the largest value being 3.43 ksi (compression) in the horizontal cross-bracing member between G3 and G4. The maximum total (live load plus dead; LC4) stress in any member for the intact structure is 5.16 ksi (compression), also in the horizontal cross-bracing member between G3 and G4.

If only live load effects were considered, the only members to exceed their allowable stress for the
damaged bridge would be the horizontal cross-bracing members between G3 and G4, and between G4 and G5. With live load effects only, the allowable stress was exceeded only by 14.6% and 31.4% for these two members respectively.

The total (live plus dead) load situation must be examined since the AASHTO allowable stresses are not specified for live load only. After damage, all of the midspan horizontal cross-bracing members between girders 2 and 6 exceed the AASHTO allowable compressive stress based upon total (live plus dead load; LC4) stress. The largest value in these members is 29.2 ksi (compression) in the member between G4 and G5. This exceeds the allowable stress by 259% and the buckling strength of the member; and is a 572% increase over the intact stress in this same member. The effect of damage is obviously very severe for the cross framing members.

The largest total stress (LC4) for diagonal cross-bracing members in the damaged structure is between G5 and G6. These two diagonals have total stresses of 29.2 ksi (compression) and 21.45 ksi (tension) respectively; exceeding their allowable values for compressive and tensile stress by 127% and 7.3% respectively. The only other diagonal cross-bracing member in the damaged structure exceeding its allowable compressive stress is located between G3 and G4; having a 14.8 ksi total stress. It should be noted that the AASHTO allowable stresses are exceeded in several members on the damaged structure under dead load alone. This indicates the possibility of failure even when the bridge is closed to traffic.

While the absolute stress levels in these secondary members are of interest, they cannot be interpreted literally because of the probable onset of inelastic behavior. The most significant fact is the great increase in
the cross bracing stresses after damage occurs. These cross bracing members play a significant role in the redistribution of the load to adjacent girders after the exterior girder is damaged.

3.2.7. Reactions.

This portion of the Phase I research assumes expansion-expansion type supports. As such, no longitudinal (x) reaction forces could develop as discussed in section 2.5.2. The other reaction forces are discussed below.

3.2.7.1. Vertical Reactions.

Fig. 39 depicts the vertical support reactions for the intact and damaged 100 foot multigirder bridges with expansion-expansion supports. This figure is not a true two dimensional curve in the same sense as the previous graphs shown. While the figure's vertical axis gives the magnitude of a specific support reaction; the horizontal axis simply names the support number of the data point directly above it. The horizontal axis label are defined as follows: the number refers to the girder which is restrained by the support; the letter refers to whether the given support is located on the near (x=0) or far (x=L) end of the girder. For example, "4F" refers to the support at the far (x=1200") end of the fourth girder (G4). A positive vertical reaction is directed in the positive y direction, i.e., upwards.

As expected, the damage causes a decrease in vertical reaction at G6, and corresponding increase at adjacent girders. This is true whether or not dead load is included in the analysis (Figs. 39 and 40). As it has been seen previously, the live load causes uplift reactions near
the left edges (away from the damaged girder) of the bridge. However, any such uplift is more than compensated for by the weight of the structure, as seen in Fig. 40.

3.2.7.2. Transverse Reactions.

As discussed in section 2.5, for both fixed and expansion bearing supports, the bridge is assumed to be restrained in the transverse direction. These transverse restraints are located at both the near and far ends of all six girders.

Figs. 41 and 42 depict the transverse (z direction) reactions on the multigirder bridge with expansion-expansion supports for live and total loads respectively. The sign convention deserves notice. A positive transverse reaction force is actually directed in the negative z direction. For example, Fig. 41 shows positive transverse reaction forces for the near and far ends of G6. This means that both of the G6 transverse reaction forces are directed toward the left end of the structure, i.e., towards G1.

The most important point to note at this time is that transverse reaction forces do exist, even though they are small. The most plausible physical rationale for their existence is to resist the transverse dishing action of the slab. This dishing resistance is due primarily to the positioning of the transverse restraint at the lower flange of the girder, well below the slab level. Of course, this is where these restraints are located on an actual multigirder bridge.

Damage causes a decrease in reaction force at G6, and a corresponding increase at the adjacent girders in the same manner noted for the vertical reactions (Figs. 39 and 40). However, these transverse reactions are very small.
for expansion-expansion type supports. The diaphragms and cross bracing at the bearing ends of the girders effectively resist these small forces.

3.3. The Effect of Live Load Type on the Response of Intact and Damaged Multigirder Bridges.

3.3.1. Overview.

The three live loads employed in this study are the HS20-44 vehicle, the 128-kip Dolly vehicle, and the 204-kip PennDOT Permit vehicle. A full description of these three different load types has been given in section 2.4 of this study. Type of vehicular load will be the main parameter varied in this portion of the Phase I study; all other parameters will be kept constant. The basic multigirder bridge of interest will be a 100 foot span with expansion-expansion supports in either the intact or damaged state. "Damaged", "fractured", or "cracked" again refers to the most severe damage state discussed in chapter 2, i.e., severing the bottom flange and the full web of the exterior girder at its midspan.

3.3.2. Vertical Deflections.

Figs. 43 through 48 are live load deflection profiles from G1 through G6 respectively. The legends of these graphs use a new notation. The first letter in the legend refers to the load type ("H", "D", and "P" for the HS20-44 vehicle, 128-kip Dolly vehicle, and 204-kip PennDOT Permit vehicle loads respectively); the second letter refers to the damage state ("I" for intact/undamaged and "D" for damaged). These abbreviations will be used for multiple graph legends throughout this chapter. The reader
is also cautioned to pay particular attention to the vertical scales used for each graph. While the vertical scales were chosen to emphasize the relative differences between curves; the absolute values are often very small.

As shown in Fig. 43, 44, and 45, the live load deflections of G1, G2, and G3 are small (less than 0.30 inches) in all cases. Also, G1 and G2 deflect upwards for all three live loads on the intact or damaged bridge.

The live load deflection of the other three girders vary significantly based upon load type. For example, the G4 and G5 deflection of the damaged structure with HS20-44 vehicle loading is less severe than the deflection profile of the intact structure with either the 128-kip Dolly vehicle or the 204-kip PennDOT Permit vehicle (Figs. 46 and 47). For G6, the deflection of the damaged structure with HS20-44 vehicle is about the same as the intact structure with the 128-kip Dolly vehicle load (Fig. 48). Despite widely varying magnitudes, all load types cause the characteristic hinge-like profile in G6 when it is fractured (Fig. 48).

For all load types, there is almost a doubling of the maximum live load deflection (occurring at the midspan of the G6) as a result of damage. Specifically for the HS20-44 vehicle it is 93%, for the 128-kip Dolly vehicle it is 99%, and for the 204-kip PennDOT Permit vehicle it is 87%. Doubling the maximum intact response is generally a good rough estimate of the maximum damaged response. For dead and live load, there is also almost a doubling of the maximum deflection for all load types as a result of damage. Specifically for the HS20-44 vehicle it is 85%, for the 128-kip Dolly vehicle it is 89%, and for the 204-kip PennDOT Permit vehicle it is 84%. These percentages are all less than the percentage increase for live load.
alone.

The structure's resistance to deformation for a unit of live load (stiffness) is greatest for the 204-kip PennDOT Permit vehicle; least for the 128-kip Dolly vehicle. As the live load's compactness increases, the structure's live load stiffness decreases. That is, the deflection of the bridge is not proportional to the gross vehicle load. However, for the three vehicles with a given damage state, the absolute maximum response is achieved by one 204-kip PennDOT Permit vehicle load; the least response is from one HS20-44 vehicle.

The maximum live load deflection of the intact structure increases by 87% when the load type is changed from HS20-44 vehicle to 128-kip Dolly vehicle. For a damaged structure, the increase is very similar (92%). In both cases the increase is greater than the percentage increase in the total gross live load which is 78%. The maximum live load deflection of the intact structure increases by 132% when the load type is changed from HS20-44 vehicle to 204-kip PennDOT Permit vehicle. If damaged, the increase is very similar (125%). In both cases the increase is less than the percentage increase in the total gross live load which is 183%. If total (dead and live) load are considered, these percentages are all decreased. Specifically, for a load change to the 128-kip Dolly vehicle, the deflection increases 26% and 28% for the intact and damaged structure respectively; for a change to the 204-kip PennDOT Permit vehicle there is a 39% change for either the intact or damaged structure. In no case does a change in load type increase the total (dead plus live load) response by more than 40%.

The transverse deflection profiles depicted in Fig. 49 show that all load cases exhibit pronounced
transverse curvatures in the vicinity of G4 and G6 at the midspan of the damaged structure. As a result, cracks in the deck slab might be expected to be developed in a longitudinal direction in the vicinity of these girders. Transverse curvature is very mild at the quarter span line (Fig. 50).

3.3.3. Lower Flange Stress.

A separate graph has been prepared to show the lower flange stress of each girder of the intact and damaged multigirder bridge with all three load types. Figs. 51 through 56 are for live load stresses only; Figs. 57 through 62 are for total stress.

For all load types, compressive live load stresses exist in G1, reflecting the upward deflection of that girder. For all load types, the smallest load stresses are in G2. For G3 and G4, the live load stresses are ordered in the same way as the live load deflections, i.e., H-I, H-D, D-I, P-I, D-D, and P-D. For the given damage state, the lower flange stress in all girders is greatest for one 204-kip PennDOT Permit vehicle load; least for HS20-44 vehicle load.

As seen in Fig. 55, there is an approximate doubling of the G5 stress for all live load cases when the structure is damaged. Specifically, the increases are 106%, 104%, and 104% for the HS20-44 vehicle, 128-kip Dolly vehicle, and 204-kip PennDOT Permit vehicle respectively. With dead load included with the live load, the percent increase in the total flange stress when damage occurs is smaller. Specifically, when damage occurs, the G5 flange stress increases by 80% for the HS20-44 vehicle with dead load, by 85% for the 128-kip Dolly vehicle with dead load, and by 86% for the 204-kip PennDOT Permit vehicle with dead
load. Hence, doubling the maximum intact response of G5 would be conservative when predicting the damaged response.

The effect of changing load type is severe, but not in the same proportion as the increase in the magnitude of the live load. For example, the G5 maximum stress increases by 100% when the load on the intact structure is changed from HS20-44 vehicle to 128-kip Dolly vehicle. A change from HS20-44 vehicle to 204-kip PennDOT Permit vehicle induces a 120% increase. The two percentages are very similar (99% and 119%) for the damaged structure. These percent increases are less when dead load is included with the live load. Specifically, the percent increase for the intact structure is 25% and 30% for the 128-kip Dolly vehicle and 204-kip PennDOT Permit vehicle respectively. When damaged these percentages change very slightly, i.e., to 28% and 34% respectively.

A graphical analysis shows that the greatest live load stress per unit live load is the 128-kip Dolly vehicle load; the least value is the 204-kip PennDOT Permit vehicle load. However, one 204-kip PennDOT Permit vehicle causes a higher stress than either one HS20-44 or one 128-kip Dolly vehicle.

3.3.4. Normal Slab Stress in the Longitudinal Direction

For all load types, the significant effects of damage on the longitudinal stress at the lower slab surface exist to the damaged side of the centerline. G1 live load stress never exceeds 30 psi in (tension); G2 never exceeds 45 psi (compression); and G3 never exceeds 130 psi (compression); regardless of load type. As seen in Figs. 63 through 66, the significant effects of damage are confined to the middle quarter of the span length. This is especially apparent when dead load effects are considered.
When measuring slab stress per unit of live load, the HS20-44 vehicle and 128-kip Dolly vehicle loads have much greater effects than the 204-kip PennDOT Permit vehicle load. However, one 204-kip PennDOT Permit vehicle load consistently exhibits the greatest absolute response for any given damage state.

The differences between load states are less apparent when dead load is included in the analysis as seen in Figs. 67 through 70.

3.3.5. Normal Slab Stress in the Transverse Direction.

Figs. 71 through 76 picture the upper surface slab stress in the transverse direction for all load types with and without dead load. The smooth live load stress profiles become much more complex after damage.

All load types exhibit similar effects of damage away from the damaged area. The ratio of damaged to intact response is relatively high compared to other responses previously measured (i.e. vertical deflection, lower flange stress, and longitudinal slab stress). In the majority of cases, the ratio exceeds 200%; in the vicinity of the damage it often exceeds 500%.

The 128-kip Dolly vehicle and 204-kip PennDOT Permit vehicle loads are relatively similar in their effects after damage, especially as the midspan is approached (Fig. 73). Other trends are consistent with the findings of other previous sections of this report. They include:

1. The structure's absolute response is generally bounded by the response to one HS20-44 vehicle and one 204-kip PennDOT Permit vehicle load.

2. All load types (with and without dead load) indicate that cracking of the deck slab will probably occur
in the damaged structure; parallel to the girders in the vicinity of G4 and G6; especially in the central third of the span (Figs. 73 and 76).

3. Inclusion of dead load stresses tends to lessen the differences between load cases (Figs. 74, 75, and 76).

3.3.6. Cross Bracing Stresses.

As indicated in 3.2.6, whether intact or damaged, the highest cross-bracing stresses are generally located across the midspan of the structure. The members discussed below are located transversely across the midspan of the model.

The allowable compressive stress based upon the AASHTO inventory rating (i.e. normal traffic) is approximately 8.1 ksi and 12.9 ksi for the horizontal and diagonal cross-bracing members respectively. These maximum allowable compressive stresses incorporate a factor of safety of 2.12. The maximum allowable tensile stress for either a horizontal cross-bracing member or a diagonal cross-bracing member is 20.0 ksi.

As discussed previously in section 3.1.4, AASHTO makes provisions for heavier than normal loads governed by special permits. These loads are referred to as the "operating rating" loads; and the allowable stresses of these loads are up to 35% higher than the allowable stresses of the "inventory rating" load. Since the 128-kip Dolly vehicle (LC2) and 204-kip PennDOT Permit vehicle (LC3) loads are generally considered to be permit-type loads, this entire section will make stress comparisons based upon the "operating rating" provisions of AASHTO unless otherwise indicated in the text. The allowable operating rating compressive stress (for special permit
vehicles) is approximately 10.2 ksi and 16.0 ksi for the horizontal and diagonal cross-bracing members respectively. These maximum allowable stresses incorporate a factor of safety of 1.70. The maximum allowable tensile stress for either an horizontal cross-bracing member or diagonal cross-bracing member is 27.0 ksi.

None of the three load types create cross-bracing stresses exceeding the operating rating allowable stress when the structure is intact (Table 3). For all three load cases the maximum total (live plus dead load) compressive stress occurs in the horizontal cross-bracing member between G3 and G4; its value is 5.2 ksi, 8.3 ksi, and 9.4 ksi for the HS20-44 vehicle (LC4), 128-kip Dolly vehicle (LC5), and 204-kip PennDOT Permit vehicle (LC6) loads respectively.

Depending upon the manner in which the cross-bracing members are constructed, these member may carry only live load stresses or both live and dead load stresses. Hence, the bridge engineer is interested in the separate effects.

With live load alone, both the 128-kip Dolly vehicle (LC2) and 204-kip PennDOT Permit vehicle (LC3) loads exhibit very similar effects on the damaged structure. Both have four horizontal cross-bracing members at midspan exceeding the operating allowable compressive stress. The maximum horizontal cross-bracing member live load compressive stress for the 128-kip Dolly vehicle and 204-kip PennDOT Permit vehicle loads is 39.5 ksi and 42.0 ksi. As before, the 128-kip Dolly vehicle load has the highest stress per uni of live load; one 204-kip PennDOT Permit vehicle and one HS20-44 vehicle loads provide the maximum and minimum absolute effects for cross-bracing stress caused by the three vehicular loads investigated.
With live load considered only, both the 128-kip Dolly vehicle and 204-kip PennDOT Permit vehicle loads have one diagonal cross-bracing member (the member between G5 and G6) exceeding the operating allowable compressive stress on the damaged structure. The live load stress in this member is 19.5 ksi and 21.8 ksi for the 128-kip Dolly vehicle and 204-kip PennDOT Permit vehicle loads respectively.

With live and dead load included, all three load types have all of their horizontal cross-bracing members on the damaged structure from G2 to G6 in excess of the rating allowable compressive stress. In all cases the maximum stress occurs between G4 and G5, with values of 29.2 ksi, 39.5 ksi, and 42.0 ksi for the HS20-44 vehicle (LC4), 128-kip Dolly vehicle (LC5), and 204-kip PennDOT Permit vehicle (LC6) loads respectively. With dead and live load included, the 128-kip Dolly vehicle and 204-kip PennDOT Permit vehicle loads have three diagonal cross-bracing members in excess of the rating allowable stress on the damaged structure. In contrast, the HS20-44 vehicle load has only one. The maximum compressive stress for a diagonal cross-bracing member always occurs between G5 and G6, and reaches 29.2 ksi, 38.7 ksi, and 41.0 ksi for the HS20-44 vehicle, 128-kip Dolly vehicle, and 204-kip PennDOT Permit vehicle loads respectively.

In summary, overloads on the intact structure do not cause yielding or buckling of cross bracing members. However, extensive damage to the midspan of the exterior girder makes extensive yielding and buckling of cross bracing members very likely for all three load cases.
3.3.7. Reactions.

Only vertical reactions will be discussed in this section of the report. Since expansion-expansion supports are assumed, longitudinal reaction are non-existent for this model. Transverse reactions are present, but small. The variation in the transverse reactions with load type will not be addressed.

The 128-kip Dolly vehicle and 204-kip PennDOT Permit vehicle loads exhibit the same general characteristics already discussed concerning vertical reactions induced by the HS20-44 vehicle load (see subsections 3.2.6 and 3.2.7). The only noteworthy difference is the disparity between the near and far reactions on G6 with the 204-kip PennDOT Permit vehicle load (Figs. 77 and 78). This results from the extreme nonsymmetry of the 204-kip PennDOT Permit vehicle load about the midspan as described in section 2.4.1.

In all cases, the maximum and minimum absolute response results from one 204-kip PennDOT Permit vehicle and one HS20-44 vehicle load respectively.

3.4. The Effect of Span Length on the Response of Intact and Damaged Multigirder Bridges.

3.4.1. Overview.

This portion of the Phase I study addresses how span length influences the response of intact and damaged multigirder bridges. Before presenting the results, it is important to remind the reader that this study compares three standard design bridges extracted from Ref. 13. As such, the researcher is not conducting a study of span length in the sense normally associated with a "parametric"
study. No attempt will be made to keep such things as steel girder properties, intervals of cross bracing patterns, or size of transverse stiffeners constant while varying the span length. This study will simply compare models of three standard design bridge structures of different spans and different geometric properties. The differences between these bridges was previously delineated in section 2.2.3.

Since span length is the main parameter varied in this portion of the Phase I study, all other parameters will be kept constant. Hence, the basic multigirder bridge of interest will be an intact or damaged structure with expansion-expansion supports, and loaded with one HS20-44 vehicle load. "Damaged", "fractured", or "cracked" again refers to severing the bottom flange and the full web of the exterior girder at its midspan.

3.4.2. Vertical Deflections.

Figs. 79 through 82 are longitudinal live load deflection profiles for selected girders under HS20-44 vehicle loading. The legends used in these graphs has changed from the previous section. The numbers in the legend indicate the span of interest ('"00", "40", and "80" refer to the 100, 140, and 180 foot spans respectively); the letter refers to the damage state ("I" for intact and "D" for damaged).

All span lengths have an upward live load deflection at G1, have only a slight deflection of G2 and G3, and exhibit the characteristic hinge-like profile in G6 when it is fractured.

The ratio of damaged to intact response decreases slightly as the span increases; for all span lengths the effect is less than doubling. Specifically, the maximum
live load deflection increases by 93%, 89%, and 84% for the 100', 140', and 180' spans after damage is imposed as can be seen in Fig. 82. When dead load is also included, these percentages are reduced to 85%, 77%, and 72% respectively.

Under dead load alone, the effect of damage decreases as the span increases. The percentage change in maximum deflection is 81%, 74%, and 69% for the 100', 140', and 180' spans respectively. Under dead load alone, all spans exhibit an upward deflection of Gl as a result of damage. Specifically, there is a Gl deflection decrease of 12%, 15%, and 16% after damage occurs.

The transverse live load deflection profiles shown in Figs. 83 and 84 point out that all spans share common characteristics. That is, all have zero live load deflection immediately to the right of G2, and all profiles also show the existence of large transverse curvature at the midspan of all damaged spans in the vicinity of the bridge centerline and near G6.

In summary, the maximum live load deflection of the intact structure increases by 17% when the span changes from 100' to 140'; 30% when changed from 100' to 180'. For the damaged structures these percent changes are 14% and 24% respectively. In other words, for each unit increase in span, the incremental change in live load deflection decreases. However, the absolute live load deflection increases as the span increases.

3.4.3. Lower Flange Stress.

For all span lengths, there is a compressive live load stress in the lower flange of G1, reflecting the upward deflection of that girder as can be seen in Fig. 85. For all span lengths, there is an extremely small live load stress in G2 (Fig. 86). The effect of damage is ap-
approximately a doubling of stresses for all span lengths as can be seen in Figs. 87 and 89. Specifically, the G5 live load stress increases 106%, 110%, and 112%, respectively, when the 100', 140', and 180' spans are damaged. When dead and live load are combined, these percentages become 80%, 83%, and 83% for the 100, 140, and 180 foot spans respectively (Fig. 90).

For a given damage state, the live load stress decreases as the span increases as can be seen from examination of Figs. 85 through 89. That is, the maximum live load response is the 100' span; the minimum live load response is the 180' span. Specifically, the G5 stress is reduced 14% when the intact span is changed from 100' to 140'; 27% when changed from 100' to 180'. For a damaged structure these two percentages become 12% and 25% respectively.

When live and dead load are considered, the lower flange stress increases as the span increases. Specifically, the G5 stress is increased 12% when the intact span is increased from 100' to 140'; 19% when changed from 100' to 180'. For a damaged structure these two percentages are 14% and 21% respectively (Fig. 90).

The last two paragraphs indicate a general principal of bridge engineering; that is, the relative contribution of dead load to total load increases as span increases.

3.4.4. Normal Slab Stress in the Longitudinal Direction

When live load is considered only, the differences in longitudinal slab stress at the bottom surface of the deck between the different span lengths is minimal throughout the structure (Figs. 91 through 94). The general characteristics discussed in paragraph 3.2.4 for
the 100' span hold true. For all spans, the significant effects of damage exist only within the middle quarter of the span; and to the damaged side of the centerline of the bridge. This is especially apparent when dead load effects are combined with live load effects as can be seen in Figs. 95 through 98.

When live load is considered only, the greatest absolute effects of damage are experienced by the 100' span; the least by the 180' span (Figs. 92, 93, and 94). This is because the girders in the longer spans are much stiffer than the girders in the shorter spans in resisting live load by itself. When live and dead load are combined, the greatest absolute effects of damage are generally experienced by the 180' span; the least by the 100' span (Figs. 97 and 98). This is due to the greater dead load of the longer span.

3.4.5. Normal Slab Stress in the Transverse Direction.

As far as the transverse stress in the upper slab surface is concerned, the general trends of paragraph 3.2.5 hold. There is no significant qualitative difference between the various span lengths. Fig. 99 show that, in absolute terms, the greatest live load effects are generally experienced by the smallest span. When dead load is included with the live load, there is very little difference between the response of the different spans as can be seen in Fig. 100.

3.5. The Effect of Intermediate Levels of Damage on the Response of Multigirder Bridges.
3.5.1. Overview.

Up to this point in the presentation of the Phase I results, only the intact bridge and the severely damaged bridge, in which a vertical fracture has penetrated the bottom flange and the full web of the exterior girder at its midspan, have been considered. The purpose of this portion of the Phase I study is to quantify the response of a multigirder bridge containing a less severe fracture/crack damage.

The four different structural states to be compared during this part of the research are:

1. The undamaged (intact) bridge. (INTACT)
2. The bridge with a simulated lower flange fracture at the midspan of the exterior girder. (D-LF)
3. The bridge with a simulated lower flange and half web depth crack at the midspan of the exterior girder. (D-LW)
4. The bridge with a simulated lower flange and full web depth fracture at the midspan of the exterior girder. (D-WW)

The parenthetical letters following each of the four previous items are the symbolic abbreviations for identifying these damage states in the supporting graphs (see Section 2.3.3).

Since the degree of damage is the main parameter in this portion of the report, all other parameters will be kept constant. The basic bridge of interest will be a 100 foot span with expansion-expansion supports, and HS20-44 vehicle loading.

The purpose of this portion of the report is to relate damage states consisting of intermediate levels of damage to the two extreme conditions previously discussed in this report. The author will minimize the repetition of
trends already presented in this chapter, and focus solely on the relationship between these different damage states.

3.5.2. Vertical Deflections.

There is very little difference between the intact bridge and the lower flange fractured bridge. As can be seen from Figs. 101 and 102, for the 100' bridge with the HS20-44 vehicle load, there is only a 3.9% increase of the live load maximum deflection of G6; 3.0% increase in G5. These increases are almost identical for the 128-kip Dolly vehicle and the 204-kip PennDOT Permit vehicle loads. The influence of dead load is insignificant in this analysis. When dead and live load are combined (Figs. 103 and 104), the percent increase of the flange damaged bridge with respect to the intact bridge is 2.0% and 3.4% for G5 and G6 respectively. Hence, fracturing of the lower flange alone has only a minor effect on the deflection response of a multigirder bridge. In contrast, partial web damage has a very significant effect.

In all cases, the half web depth crack response is bounded by the intact and full web depth fracture. A very good estimate of the deflection response of the half web depth response is to average the intact and the full web depth fracture responses. When this was done for the HS20-44 vehicle load on the 100' span, the results were extremely good. For all node deflections on the entire bridge surface, the approximation was generally within 2% of the actual live load model response. Specifically, for G5 and G6 the approximation was only an 0.56% and 1.25% overestimate of the actual model behavior. The approximation is equally valid when dead load is included in the analysis. It is also significant that the averaging approximation generally overestimates the response. That is,
they are slightly conservative.

For comparison purposes, an approximation was also calculated based upon averaging the flange fractured and full web depth fractured responses. While this approximation was also very good, it was slightly worse than the approximation based upon averaging the intact and full web depth damage responses. Specifically, this new approximation overestimated the model's actual response for G5 and G6 by 1.69% and 2.58%. This trend was evident for all load types, with and without the inclusion of dead load.

Hence, an approximation based upon the two extreme cases is more accurate. It is also more practical in its application. This will be discussed more fully in section 3.5.7.

3.5.3. Lower Flange Stress.

Except at the damaged girder itself, there is only minor differences between the lower flange stress of the intact structure and the structure with the lower flange severed (Figs. 105 through 107). As can be seen in Fig. 107, the G5 maximum live load stress only increases by 4.2% as a result of the flange fracturing in G6. Even for the damaged girder (G6), substantial differences between the stress profiles appear only within about 4 feet of the damage location as can be seen in Fig. 108. These trends are similar for all load types, with and without the inclusion of dead load.

Partial web damage can significantly increase the stress in the girders adjacent to the damaged girders as shown in Figs. 106 and 107. Averaging the intact and full web depth fracture responses yields an extremely good estimate of the flange stress in all of the girders other
than the damaged girder itself. Specifically, the averaging technique overestimates the G5 model’s stress by only 1.4%. This averaging technique remains valid for G1 through G5 and for all load cases with and without dead load. The averaging technique consistently overestimates the actual model stress, i.e. it is conservative.

As can be seen in Figs. 108 and 110, the averaging technique is not valid on the damaged girder itself in close proximity to the damage. The averaging estimate is very good (less than 2%) outside 8 feet longitudinally from the damage location. Within about 8 feet from the damage, the averaging technique yields unsatisfactory results. The above trend is consistent for all load cases, with and without dead load.

The reader should be advised to exert care in the interpretation of the finite element modeling data in close vicinity to the damage. The flange stress data in the vicinity of the damage should not be interpreted literally, as it is based upon the average of nodal forces/couples in a high stress gradient region.

3.5.4. Normal Slab Stress in the Longitudinal Direction

The fracturing of the lower flange has no significant effect on the longitudinal slab stress. In most cases the profiles of the longitudinal slab stress of the intact and the flange damaged states are nearly identical. This is true everywhere on the structure, including close proximity to the damage location as can be seen in Figs. 111 through 114. Superposition of dead load does not alter this trend (Figs. 115 and 116).

Partial web damage can significantly change the longitudinal slab stress in the vicinity of the damage. The largest changes in stress are most apparent within a 10
feet radial distance from the damage location as can be seen in Figs. 111 through 114.

Averaging the intact and full web depth fractured responses is a very good approximation outside a 10 foot radial distance of the damage; inside this radius it can be a very erratic approximation. However, at all locations the model's actual half web damage response is bounded by the intact and the full web depth damage response.

3.5.5. Normal Slab Stress in the Transverse Direction.

Flange fracturing alone has an insignificant effect on the transverse stress at the upper slab surface; even in close proximity to the damage. Partial web damage can significantly change the transverse slab stress in the vicinity of the damage. The largest changes in stress are most apparent within about 10 feet radially of the damage as can be seen in Figs. 117 through 122.

Averaging the intact and full web depth fractured responses is a very good approximation outside a 10 foot radial distance of the damage; inside this radius it can be a very erratic approximation. However, at all locations the model's actual half web damage response is bounded by the intact and the full web depth damage response.

3.5.6. Cross Bracing Stresses.

For all four structural conditions, the highest cross-bracing stresses are located across the midspan of the structure. The members discussed below are located transversely across the midspan of the model. The allowable compressive stress based upon the AASHTO inventory rating (i.e. normal traffic) is 8.1 ksi and 12.9 ksi for the horizontal and diagonal cross-bracing members respectively. The maximum allowable tensile stress for either a
horizontal cross-bracing member or a diagonal cross-bracing member is 20.0 ksi.

Fracture of the lower flange alone has only a minor effect on the cross-bracing stresses as seen in Table 3. The maximum total (LC4) compressive stress increased from 5.16 ksi to 5.88 ksi (in the horizontal cross-bracing member between G3 and G4), an increase of only 14%.

A half-depth web crack has a more severe effect on the cross-bracing stresses. This type of damage shifted location of the maximum total (LC4) compressive stress to the horizontal cross-bracing member from G4 to G5, resulting in a maximum value of 16.4 ksi. This exceeds the maximum allowable AASHTO operating stress and represents an increase of 277% with respect to this member's stress in the intact structure. In comparison, the stress in this same member is 4.4 ksi when the structure is intact; 29.2 ksi when the web is totally severed.

The technique of approximating the effect of a half web depth crack by averaging the intact and total web depth fracture is not as consistently accurate as previously discussed for deflection and lower flange stress data. The error in this technique ranged between a 10% overestimate and a 6% underestimate. No clear trend was observed regarding the occurrence of an underestimate rather than a overestimate.

3.5.7. Application

The practical advantages of estimating partial web depth damage from the intact and full web depth responses is very important to understand. The most economical damage states to model are the intact and full web depth damage states. Full modeling of partial web depth cracks of various depths dictate inclusion of a large
number of web elements. This can be very time consuming for the modeler; and very costly in terms of computer central processing unit (CPU) time. Using the estimating technique will allow the user to approximate various web depth crack behavior without having to model the girder in great detail. The only required computer analyses would be for the intact and the full web depth damage states.

3.6. The Effect of Support Conditions on the Response of Intact and Damaged Multigirder Bridges.

3.6.1. Overview.

The results reported in the previous sections of this chapter were based upon the assumption that all of the bridge models had expansion-expansion supports. This assumption dictates that no longitudinal reaction forces will act at the bridge supports. The expansion-expansion support condition has been used thus far because it was felt that these supports would maximize the bridges response to live and dead load. The expansion-expansion supports also create conditions most similar to those used in making calculations for one-dimensional beams with simple supports.

This portion of the Phase I research investigated the effect of using other supports to restrain the multigirder bridge model. The three different support conditions to be investigated were previously described in detail in section 2.5 of this study. They include:

1. The expansion-fixed support, i.e., longitudinal restraints at the far end (x=L) of all six girders and no longitudinal restraint at the near end. (OX)

2. The expansion-expansion support, i.e., no longitudinal reaction forces can develop to vertical loads.
3. The fixed-fixed support, i.e., longitudinal restraint at both the near and far ends of all six girders.

The parenthetical letters following each of the three previous items are the symbolic abbreviations to be used in the supporting graphs for this section.

The reader is reminded that all three of the support conditions restrain both ends of all girders (at the lower flange) from translating in the vertical (y) and transverse (z) directions. No rotational restraints are applied to the end of any girder.

Since the support condition is the main parameter in this portion of the report, all other parameters will be kept constant. The basic bridge of interest will be an intact or damaged 100 foot multigirder bridge with HS20-44 vehicle loading. "Damaged", "fractured", or "cracked" refers to severing the bottom flange and the full web of the exterior girder at its midspan.

The previously reported results have provided a large quantity of data on the response of multigirder bridges with expansion-expansion supports. Hence, the author will focus on how the expansion-fixed and fixed-fixed bridge response differs from the expansion-expansion supported structure.

3.6.2. Vertical Deflections.

Figs. 123 through 130 show selected deflection profiles for intact and damaged multigirder bridges supported in three different manners. The first two letters of the legend symbols used in these graphs were discussed in the previous subsection; the third letter of the legend refers to the damage condition ("I" for intact and "D" for
damaged). The various deflection responses will be con­
trasted below.

3.6.2.1. Expansion-Fixed Supports.

The expansion-fixed supports create a more com­
plex live load deflection profile in the girders remote
from the damage as can be seen in Figs. 123 and 124. G1's
deflection profile looks like a full sine wave with inflec­
tion point at midspan when the model is either intact or
damaged. G2 does not have an upward deflection under live
load as previously seen with expansion-expansion supports.

The expansion-fixed supports reduce vertical
deformation for all girders on the damaged side of the
bridge centerline. Specifically, there is an 18.8% reduc­
tion of the maximum live load deflection of the intact
bridge when the supports become expansion-fixed as can be
seen in Fig. 128.

Most of this reduction results from live load. If
dead load were considered alone, the deflection profiles of
the expansion-expansion and expansion-fixed intact bridges
are indistinguishable. When dead and live load effects are
combined, there is only a 5.93% reduction of maximum
deflection from expansion-expansion supports to expansion­
fixed supports for the intact 100' bridge as shown in Fig.
130.

When the bridge is damaged, the expansion-fixed
supports have a more significant impact on reducing defor­
mation. The maximum live load deflection is reduced 28.9%
from the expansion-expansion condition. When live and dead
load are combined, the reduction is 17.4%.

Changing the support to expansion-fixed supports
reduces the relative difference between the intact and
damaged states. While there was almost a doubling of the
maximum live load deflection when the model with expansion-expansion supports is damaged; with expansion-fixed supports there is a 69.2% increase with damage. This increase is 62.3% when dead load effects are also included.

3.6.2.2. Fixed-Fixed Supports.

Qualitatively the deflection profiles for the fixed-fixed supports are very similar to the expansion-expansion supported models, i.e. there is an upward deflection of G1 and G2, and half sine wave type deflection profiles. However, there is a substantial decrease in the magnitude of the deflection of all girders when the supports are changed from expansion-expansion to fixed-fixed. This is true for both the intact and damaged bridges, with and without dead load. Specifically, the maximum live load deflection on the intact structure is reduced 51.0%; on the damaged structure it is reduced 63.6% when changed from expansion-expansion to fixed-fixed supports.

There is a considerable difference between the expansion-expansion and fixed-fixed deflection responses to dead load alone. For example, there is a 56.0% and 68.2% reduction of the deflection of the intact and damaged bridges respectively from expansion-expansion to fixed-fixed supports when dead load acts alone. When live and dead load are combined these two percentages change to 54.5% and 66.8%; hence dead and live load effects are independently significant.

The fixed-fixed support condition minimizes the difference between the intact and damaged bridges. Under live load alone there is a 43.7% increase of the maximum live load deflection (much less than the doubling of the expansion-expansion supports); a 35.0% increase of the maximum live plus dead load deflection.
3.6.2.3. Comparison.

Both the expansion-fixed and fixed-fixed supports reduce deformation with respect to the expansion-expansion supports. The expansion-fixed reductions are primarily dependent on live load; the fixed-fixed reductions are for both live and dead load. The fixed-fixed reductions of deformation are so large that the damaged bridge with fixed-fixed supports generally deflects less than the intact bridge with expansion-expansion or expansion-fixed supports.

3.6.3. Lower Flange Stress.

Figs. 131 through 142 depict the lower flange stress for the three support conditions on selected girders of the 100 foot multigirder bridge. The important results are presented below.

3.6.3.1. Expansion-Fixed Supports.

The expansion-fixed supports create a very unsymmetrical response with respect to the midspan. This is apparent when viewing the live load stress profiles of all girders, both before and after damage. Away from the live load (G1 and G2), the nonsymmetrical supports induce a live load tensile stress at the fixed end. However, even after damage of the bridge, this tensile stress stays below about 3.0 ksi.

In the vicinity of the live load (G5 and G6), the maximum flange stress (located at midspan) is reduced by changing the model's supports from expansion-expansion to expansion-fixed. For example, the maximum G5 stress is reduced by 15.6% for the intact and 26.5% for the damaged model when the supports are expansion-fixed. These reduc-
tions are not as large when dead load and live loads are combined.

However, in the vicinity of the live load (G5 and G6), there are live load compressive stresses at the flange near the fixed support which did not exist for the expansion-expansion support condition. These stresses can be very significant. For example, for the intact structure with expansion-fixed supports, the magnitude of the G6 live load compressive stress at the fixed support is approximately equal to the same girder's midspan tensile stress as can be seen in Fig. 136.

In all cases the magnitude of the flange stress in the vicinity of the support increases as the result of damage. This is true for both live load alone and combined live and dead load. However, the differences between the expansion-fixed and expansion-expansion stress response is mainly attributable to live load. If dead load only is considered, the differences are minimal.

3.6.3.2. Fixed-Fixed Supports.

The stress profile with the fixed-fixed supports looks very similar to that of a prestressed concrete beam as can be seen in Fig. 134. This is apparent for all girders after dead load has been combined in the analysis.

The maximum live load flange stress in the intact structure (at G6) is cut in half (47.8% reduction) as a result of the supports changing from expansion-expansion to fixed-fixed. The reduction is even greater (56%) when dead load is combined with live load.

The relative stress reduction is even greater for the damaged structure. The maximum G5 live load stress is reduced 64.1% by changing the supports from expansion-expansion to fixed-fixed; with dead load the reduction is
68.3%. The stress reduction is not dependent upon the existence of an unsymmetrical live load. With dead load alone, the maximum G5 stress is reduced 61.3% on the intact structure; 70.0% on the damaged structure.

The difference between the intact and damaged states are minimized by changing from expansion-expansion to fixed-fixed supports. With fixed-fixed supports, there is only a 53.4% difference between the intact and damaged maximum G5 live load response. With dead load included the difference between intact and damaged is only 39.3%. From viewing the stress profiles of all girders (with and without dead load), it is obvious that the fixed-fixed supports minimize the difference between the intact and damaged states. This is especially apparent for the girders remote from the damage (G1 through G4).

The "cost" of these beneficial effects is the existence of flange stresses in the vicinity of both the near and far supports. The lower flange stresses existing in the vicinity of the bridge supports will be referred to as the "flange support stresses."

The live load flange support stress is very small for G1 through G3. However, when dead load is included in the analysis, even the G1 through G3 flange support stresses can be quite large. For all of these girders the flange support stresses are about 8 ksi (compression). This is about triple their respective midspan tensile stress.

For all girders, with either the intact or damaged state, with or without dead load; the magnitude of the flange support stresses exceed the magnitude of the corresponding midspan stress. In many cases the magnitude of the flange support stresses are more than double the magnitude of the midspan stress. The largest flange sup-
port stresses occur on G6 when the structure is damaged; about 5 ksi (compression) for live load alone, about 16 ksi (compression) for combined live and dead loads.

While the flange support stresses increase for all girders after damage, these increases are less than the corresponding increase of the midspan stress. For example, while the G5 midspan total stress increases 39.3% after damage, the G5 flange support stresses increase 18.0%. Hence, the flange support stresses are not as sensitive to damage.

3.6.3.3. Comparison.

Both the expansion-fixed and fixed-fixed supports reduce the maximum flange stress compared with the expansion-expansion support. The reduction resulting from fixed-fixed supports is much more considerable. The expansion-fixed stress reduction are primarily a live load phenomena; the fixed-fixed stress reductions are not.

Both the expansion-fixed and fixed-fixed support conditions have flange support stresses at the longitudinal restraints. The flange support stresses are generally higher for the fixed-fixed supports than the expansion-fixed supports.

3.6.4. Normal Slab Stress in the Longitudinal Direction

The graphs of the longitudinal slab stress for both live and total load (Figs. 143 through 148) indicate one prevailing trend, i.e., there is no appreciable difference between the expansion-expansion, expansion-fixed, and fixed-fixed support conditions outside the middle quarter of the span. Even within the central quarter of the span, there is very little difference between the slab
stress profile for the intact structure with either expansion-expansion, expansion-fixed, or fixed-fixed supports.

Within the central quarter of the damaged span, the longitudinal slab stress is reduced significantly if the expansion-expansion supports are changed to either expansion-fixed or fixed-fixed supports. Specifically, the live load tensile stress at midspan near G6 is reduced by 33.6% and 73.2% when changed to the expansion-fixed and fixed-fixed supports respectively. When dead load is also included, these percentages change to 22.4% and 84.7% respectively.

3.6.5. Normal Slab Stress in the Transverse Direction.

The intact structure's transverse slab stress is very similar for all three support conditions, as true whether or not dead load is included in the analysis as seen in Figs. 149 and 150.

The transverse slab stress is reduced throughout the damaged structure when the supports are changed from expansion-expansion to either expansion-fixed or fixed-fixed supports. For example, the peak live load tensile stress at midspan near G4 is reduced by 24.6% and 52.3% when the supports become expansion-fixed and fixed-fixed respectively. With dead load included these percentages become 18.5% and 65.0% respectively. Notice that the fixed-fixed supports are still influenced more by dead load than the expansion-fixed supports.

3.6.6. Cross Bracing Stresses.

Whether intact or damaged, the highest cross-bracing stresses are generally located across the midspan of the structure (Table 4). The members discussed below
are located transversely across the midspan of the model. The allowable compressive stress based upon the AASHTO inventory rating (i.e. normal traffic) is approximately 8.1 ksi and 12.9 ksi for the horizontal and diagonal cross-bracing members respectively. The maximum allowable tensile stress for either a horizontal cross-bracing member or a diagonal cross-bracing member is 20.0 ksi.

Cross-bracing stresses were generally reduced throughout the structure as a result of changing from the expansion-expansion support to either the expansion-fixed or fixed-fixed supports (Table 4). This was true for all load cases on both the intact and damaged structure. For example, the maximum total (live plus dead load) cross-bracing stress in the expansion-expansion damaged model occurred in the horizontal cross-bracing member between G4 and G5. The value of this stress is 29.2 ksi, exceeding the allowable inventory compressive stress by 259%. The stress in this same member is reduced to 23.5 ksi (188% over allowable) and 10.3 ksi (27% over allowable) for the expansion-fixed and fixed-fixed supports respectively.

The largest total (live plus dead load) diagonal cross-bracing member compressive stress for the damaged model with expansion-expansion supports is in the diagonal between G5 and G6. Its specific value is 29.2 ksi, which exceeds the allowable inventory rating stress by 127%. Changing supports reduces this members stress to 23.5 ksi (83% over allowable) and 9.8 ksi (24% under allowable) for the expansion-fixed and fixed-fixed supports respectively. For the damaged structure, most of the cross bracing stresses were reduced by about 20% when changing from expansion-expansion to expansion-fixed supports. The reduction generally exceeded 60% when changing from expansion-expansion to fixed-fixed supports.
The ratio of damaged to intact response was reduced most significantly when the supports were changed to fixed-fixed supports. For example, the very highly stressed horizontal cross-bracing member between G4 and G5 had a stress increase of approximately 940% when the model was changed from intact to damaged, for either the expansion-expansion or the expansion-fixed supports. However, for the fixed-fixed supports the intact stress increased by a much lower 388% after it was damaged.

3.6.7. Reactions.
Most of the previous paragraphs of the last section discussed quantitative results with little or no explanation of their cause. This was done intentionally. The key to understanding the effect of different support conditions is understand where and why different reactive forces develop. The following paragraphs will address these issues.

3.6.7.1. Vertical Reactions.
Figs. 151 and 152 display the vertical reactions for the intact and damaged bridges respectively. The terminology and sign convention used in these graphs was presented in subsection 3.2.7.

The vertical reactions for the expansion-fixed supports are characterized by a large disparity between the magnitude of the near and far reactions. For example, on G6 of the intact model with expansion-fixed supports, the live load vertical reactions are 14.7 kips and 25.7 kips for the expansion and fixed supports respectively. In contrast, the same reactions for the expansion-expansion supported structure are 21.6 kips and 17.3 kips. In other words, the nonsymmetrical longitudinal restraints are "attracting" the live load at the far (restrained) end of
the bridge.

After damage, this difference between the near and far support's live load response is even greater; specifically, they become 4.3 kips and 23.4 kips respectively for the expansion-fixed model in contrast to value of 12.4 kips and 8.1 kips for the expansion-expansion supported model. Notice that after damage of the expansion-fixed model there is only a minor change in the fixed support reaction, but a very large reduction of the expansion support reaction. The damage is dividing G6 into two distinct portions: a very flexible half connected to the near support with no longitudinal restraint, and a stiffer half connected to the far end trying to continue to support the live load.

The fixed-fixed vertical reactions are characterized by an ability to support a larger proportion of the live load at the edge girder. This is especially true after damage occurs. For example, the near and far supports of the fixed-fixed damaged model carry 65.8% and 81.5% more live load force than the corresponding expansion-expansion model. Of course, this means that less vertical reactive force is transferred to the neighboring G5 supports for the fixed-fixed conditions than for the expansion-expansion condition. The longitudinal restraints at both lower flanges of G6 are reducing the longitudinal curvature, with a corresponding increase in stiffness.

3.6.7.2. Longitudinal Reactions.

Our standard model for the results reported in sections 3.2 through 3.5 had expansion-expansion supports and hence no longitudinal reactions. This is not true for the expansion-fixed and fixed-fixed supported models as seen in Figs. 153 and 154. The sign convention used in
these figures is as follows: positive longitudinal reactive forces push the model in at the support, i.e., they act to cause compression at the supported girder's flange.

The live load reactive forces push on the structure in the vicinity of the live load (G5 and G6), and pull on the structure away from the live load (G1 and G2) for both the expansion-fixed and fixed-fixed supports. This is true both before and after damage. The basic cause of this is the "dishing" effect in the longitudinal direction. The lower flange of the loaded girders wants to push out as a result of the vertical load. The longitudinal restraints at both ends of G5 and G6 resist this and "push" on the lower flange. The girders on the opposite (unloaded) edge of the bridge pull on the lower flange to ensure equilibrium.

Compared to the expansion-fixed support, the fixed-fixed support reactions have a relatively smooth distribution of force from girder to girder, whether intact or damaged, with or without dead load. With the fixed-fixed supports, the longitudinal reactive force decreases gradually from G6 to G1. This is due to the symmetrical nature of the fixed-fixed supports which tend to spread the load evenly to the near and far ends.

The effect of damage is less for the fixed-fixed supports than the expansion-fixed supports. For example, the G5 far live load reaction increases 68.7% after damage when the model has expansion-fixed supports; only 30.8% with fixed-fixed supports. With dead load included these two percentages are 33% and 17.0% respectively. Unlike the expansion-fixed supports, damage to the fixed-fixed supported bridge does not divide the structure into flexible and stiff segments. The midspan damage and symmetrical supports tend to reduce the difference between
near and far sides.

The expansion-fixed reactive force is effected much less by dead load than the fixed-fixed supports (Figs. 155 and 156). This is certainly expected. The expansion-fixed supports do not prevent the uniform longitudinal "dishing" resulting from the relatively constant and symmetrical dead load; the fixed-fixed supports provide complete resistance to the dead load "kicking out". For example, the far reaction on G6 for the intact structure with expansion-fixed supports increases from 59.1 kips to 75.8 kips (or 28.3%) after dead load is included in the analysis. This same reactive force increases from 71.8 kips to 251.7 kips (or 250.6%) after dead load is included. The same type of change also occurs in the damaged structure.

3.6.7.3. Transverse Reactions.

Transverse reactions do exist at both the near and far ends of all girders, for all three supports conditions (expansion-expansion, expansion-fixed, or fixed-fixed) as can be seen in Figs. 157 through 160. The reader is reminded that a positive transverse reaction acts in the negative z direction, i.e. a positive transverse reaction tries to push the girder away from the G6 edge of the model.

Transverse reactions due to live load are very small for expansion-expansion supports (the maximum is 0.33 kips), slightly larger for fixed-fixed supports (the maximum is 0.51 kips), and large for the expansion-fixed supports (the maximum is 8.31 kips). It appears that the size of the transverse reaction is effected by the nonsymmetry of the support conditions in the longitudinal direction. Symmetrical supports reduce the amount of
transverse curvature. This reduced curvature results in reduced restraining forces in the transverse direction at the lower flange of the girders.

The transverse reactions for the model with expansion-fixed supports are increased after damage is imposed; the fixed-fixed supported model is effected very little by damage. For example, the near G5 live load reaction for the expansion-fixed model increases by 55.3% after damage, the fixed-fixed model's near reaction only increases 4.2%.

The relatively large transverse reactions for the expansion-fixed supported model are not significantly effected by the inclusion of dead load. For example, the near G5 reaction on the intact model only changes from 5.28 kips to 5.97 kips (an increase of 13.1%) after dead load is included. For the other types of supports, the increase can be as great as 500%. The magnitude of the transverse reaction for both the expansion-expansion and fixed-fixed supports remain small with and without dead load (always less than 1.5 kips).

3.7. Summary of Significant Finding From the Parametric Study of Multigirder Bridges Using Linear Elastic Analysis.

The following five subsections will provide a summary of the significant findings of the study of multi-girder bridges using the SAPIV model. The first subsection will provide the most general findings, while the last four subsections will treat the four important parameters that were addressed in the study. For each of these summaries, the reader is advised to note the exact model upon which the findings are based.
3.7.1. General Summary (100 Foot Multigirder Bridge, Intact or Damaged, HS20-44 Vehicle Load, Expansion-Expansion Supports).

1. After substantial damage to an exterior girder the load is redistributed, primarily among the structural components in the immediate vicinity of the damage. The changes in the structural response (deflections, flange stresses, slab stress, etc.) of the bridge sufficiently away from the damaged girder, are small whether there is a crack or not.

2. The live or total load deflection of the exterior girder after it is severely damaged at the midspan is about twice the deflection when there is no damage. The exterior girder's smooth parabolic shape changes to a internally hinged shape with the onset of damage.

3. The maximum live load tensile stress in the 100 foot superstructure due to the HS20-44 vehicle is about 4.2 ksi and located at G6's midspan. After substantial damage to the exterior girder, the peak live load stress is about 5.8 ksi. This stress is registered at the girder adjacent to the damaged girder (G5).

4. The peak live load tensile stresses in the first interior girder (G5) at the midspan approximately doubles after the development of a severe fracture at the "loaded" exterior girder. The peak total stress in this same girder increases by about 80%.

5. The lower flange live load stresses in the girder to the left of the bridge's centerline are always less than 1 ksi.

6. The maximum reinforced concrete slab deck stresses in the direction parallel to the girders near the girder of concern jumps from 100 psi in compression to 600 psi in tension, after the formation of the deep fracture in
the exterior girder. A similar pattern is observed for the slab stresses perpendicular to the girders. The high tensile stresses in the deck concrete indicate that there may be substantial cracking of the slab in the immediate vicinity of the damage. However, the slab sufficiently away from the damaged location does not exhibit any unusually large increases in the tensile stresses.

7. The highest cross bracing stresses are located across the midspan of the 100 foot multigirder bridge. The live and total stresses for the intact structure is small in all cross bracing members. After damage, all of the midspan horizontal cross bracing members between G2 and G6 exceed the AASHTO allowable compressive stress based upon total stress. Both diagonals between G5 and G6 also exceed the AASHTO allowable stresses. The cross bracing members appear to play a significant role in the redistribution of load after the exterior girder is damaged.

8. Small transverse reactions do exist on a multigirder bridge with expansion-expansion supports.

9. The overall results have indicated that simple span steel multigirder bridges possess very large internal redundancy. After the development of serious damage in an exterior girder, large increases in the stresses and deformations in the immediate vicinity of the damage were noted. However, none of these increases appear to be high enough to result in the collapse of the superstructure. It seems that the bridge slab plays a critical role in the redistribution of the stresses and the development of new load paths.
3.7.2. Summary: The Effect of Various Load Types (100 Foot Multigirder Bridge, Intact or Damaged, Expansion-Expansion Supports).

1. The greatest absolute effects from all the measured responses is achieved by loading the 100 foot multigirder bridge with expansion-expansion supports with one 204-kip PennDOT Permit vehicle. One HS20-44 vehicle yields the minimum absolute effects.

2. The largest response (deformation or lower flange stress) per unit of live load is achieved by loading the model with the 128-kip Dolly vehicle; the least by the 204-kip PennDOT Permit vehicle.

3. Doubling the maximum intact deflection response is generally a good rough estimate of the maximum damaged deflection response for both live load alone or total load.

4. All load cases exhibit pronounced transverse curvature in the vicinity of G4 and G6 at the midspan of the damaged structure. As a result, cracks might be expected in the longitudinal direction in the vicinity of these girders.

5. There is an approximate doubling of the G5 stress for all live load cases when the structure is damaged. These increases are less than 100% when total stress is considered.

6. For all load types, the significant effects of damage on the slab stress in the longitudinal direction are located on the damaged side of the centerline and in the middle quarter of the span length.

7. Even with dead load included on the intact model, none of the three load types cause cross bracing stresses exceeding the operating rating allowable stress. On the damaged model, all three load types have all of
their horizontal cross bracing members between G2 and G6 in excess of the rating allowable compressive stress. The overload type vehicles (204-kip PennDOT Permit vehicle and 128-kip Dolly vehicle) have several diagonal cross bracing members in excess of their rating allowable stress; the HS20-44 vehicle has only one.

3.7.3. Summary: The Effect of Span Length (Multigirder Bridges, Intact or Damaged, HS20-44 Vehicle Load, Expansion-Expansion Supports).

1. For a given damage state, the live load response decreases as the span increases; the total load response increases as the span increases.

2. While the maximum live load and total load deflections increase as the span increases, the ratio of damaged to intact deflection response decreases as the span increases.

3. There is an approximate doubling of the G5 stress for all span lengths when the structure is damaged. These increases are less than 100% when total stress is considered.

4. For all span lengths, the significant effects of damage on the slab stress in the longitudinal direction are located on the damaged side of the centerline and in the middle quarter of the span length.

3.7.4. Summary: The Effect of Intermediate Damage Levels (100 Foot Multigirder Bridge, HS20-44 Vehicle Load, Expansion-Expansion Supports).

1. Fracturing of the lower flange alone has only a minor effect on the response of a multigirder bridge with expansion-expansion supports under HS20-44 vehicle loading.

2. Half web depth cracks have a very significant
effect on the response of multigirder bridges.

3. A very good estimate of the deflection (or the lower flange stress) of the half web depth cracked bridge is to average the intact and the full web fracture deflection (or lower flange stress) responses. The approximation slightly overestimates the response, and is equally valid when live load or total load is included in the analysis. The averaging technique should not be used to approximate the lower flange stress on the fractured girder within about 8 feet of the damage.

4. Averaging the intact and full web depth fractured responses for determining the slab stress in a half web depth cracked structure is accurate outside a 10 foot radial distance of the damage location. Inside this radius, it can yield erroneous results.

5. Applying the averaging technique to the cross bracing stresses produced errors ranging from +10% to -6%. Hence, the conservative approach would be to increase the averaged stress by at least 10% to estimate the cross bracing stress resulting from a half web depth crack to the exterior girder.

6. The averaging technique is a very practical method of estimating the structural response to partial web depth cracks. The only required complete computer production runs would be for the intact and the full web depth damage states.

3.7.5. Summary: The Effect of Support Conditions (100 Foot Multigirder Bridge, Intact or Damaged, HS20-44 Vehicle Load).

1. Both the expansion-fixed and fixed-fixed supports reduce deformation in comparison to the expansion-expansion supports; the reduction is substantially greater.
for the fixed-fixed supports.

2. The response due to the expansion-fixed supports is primarily a live load phenomena; the fixed-fixed response is attributable to both live load and dead load effects.

3. Both the expansion-fixed and fixed-fixed supports reduce the maximum flange stress compared with the expansion-expansion supports. The reduction resulting from the fixed-fixed supports is considerable.

4. While both the expansion-fixed and fixed-fixed support conditions have flange support stresses at the longitudinal restraints, they are generally much higher for the fixed-fixed support.

5. There is no appreciable difference between the longitudinal slab stresses in an intact multigirder bridge with expansion-expansion, expansion-fixed, and fixed-fixed supports. For a damaged multigirder bridge, the differences are only significant within the central quarter of the midspan. Within the central quarter of the midspan, both the expansion-fixed and fixed-fixed supports reduce longitudinal slab stress in comparison to the expansion-expansion supports; the reduction is substantially greater for the fixed-fixed supports.

6. Both the expansion-fixed and fixed-fixed supports reduce the cross bracing stresses on both the intact and damaged bridges compared with the expansion-expansion supports. The reduction resulting from the fixed-fixed supports is much more considerable.

7. The key to understanding the effects of various support conditions is knowing where and why different reactive forces develop for the different support conditions. Getting a "feel" for the push and/or pull of the orthogonal restraints at the near and far ends of the
six parallel girders at level of the lower flange is critical to understanding how the multigirder bridge are responding.
4. MODELING OF COMMON MULTIGIRDER HIGHWAY BRIDGES FOR NONLINEAR INELASTIC DAMAGE ANALYSIS.

4.1. Introduction.

The Phase I model using program SAPIV assumed linear elastic properties for all structural materials. The structural steel and the slab concrete were given specific material properties which remained fixed throughout the analysis. No consideration was made for cracking or crushing of the reinforced concrete, yielding of the steel girders, or buckling of any structural component. While this type of linear elastic analysis is often quite adequate for an intact bridge under normal loads, its total accuracy is questionable once the structure is overloaded and/or damaged.

The Phase I results detailed in the previous section chapter indicate that there may be several sources of nonlinear-inelastic action in an overloaded and/or damaged multigirder bridge. The most prominent include:

1. High tensile stresses in the concrete in the longitudinal direction in the vicinity of the damaged girder (G6) indicating that there may be slab cracking in the transverse direction near the midspan of the exterior girder.

2. High tensile stresses in the transverse direction in the area around G4 and G5 indicating that there may be slab cracks in the longitudinal direction near these two girders. This high tensile stress occurs when overloading the intact structure or under normal loading of the damaged structure.

3. High tensile stresses in the lower flange of G5 exceeding the AASHTO allowable "operating rating" ten-
sile stress when the damaged structure is overloaded, indicating the possibility of girder yielding.

While the above items are the most prominent sources of nonlinear behavior evidenced from the Phase I study, there is another source of concern anytime cracking, crushing, yielding, or instability occurs. That is, how is the load redistributed once failure of a specific portion of a structure occurs. While the Phase I study can indicate what areas are initially overstressed, it gives very little assistance to the analyst in determining how the load is redistributed after initial material failure occurs. It also fails to identify the extent of damage occurring as a result of this redistribution. Hence, an analysis tool is needed that provides for the inelastic behavior of structural components so that these issues can be examined. These are some of the areas to be addressed in Phase II of this study as described in Chapters 4 and 5 of this report.

4.2. Overview of the Original Version of Bridge Overload Analysis - Steel (BOVAS) Model.

4.2.1. Introduction.

Hall and Kostem developed and verified a mathematical model which predicts the overload response resulting from the placement of overload vehicles on simple span or continuous multigirder highway bridge superstructures with steel I-girder and a reinforced concrete deck (Ref. 18). A computer program with the acronym BOVAS (Bridge Overload Analysis - Steel) was developed in order to solve the mathematical model. This program has been fully tested and verified, and developed into a recognized production
tool for practicing bridge engineers (Ref. 22).

BOVAS employs the finite element method for the analytical modeling of the superstructure. The bridge superstructure is divided into a series of plate and beam finite elements (Fig. 161) which are interconnected at discrete node points (Fig. 162). These finite elements are then further subdivided into layers in order to facilitate the inclusion of material nonlinearities in the analysis (Fig. 163).

Inclusion of material nonlinearities necessitates adoption of a particular solution scheme other than that used for linearly elastic problems. Thus BOVAS uses a tangent stiffness, or piecewise linear solution process, to simulate the expected inelastic response. The loads are applied in a series of load increments or load steps in order to allow for changes in the overall structural stiffness due to nonlinear response. Within each of these load increments iterations may take place to ensure convergence of the solution. This tangent stiffness solution process in BOVAS provides a continuous description of the structural response from initial load levels in the elastic range up to the termination load levels.

The BOVAS model has demonstrated that it can adequately reflect the structural characteristics of the actual structure (Ref. 22). BOVAS reliably describes the inelastic response of beam-slab highway bridge superstructures with steel girders and a reinforced concrete deck slab by including the following phenomena in its formulation:

1. The out-of-plane or flexural behavior of the structure.

2. The in-plane response of the girders and slab due to eccentricity of the girders.
3. The coupling action of the in-plane and out-of-plane responses.
5. Shear deformation of the girders.
6. Local instability of the girder and/or girder flanges or webs, and any associated post-buckling behavior.

The application of out-of-plane loads to the bridge superstructure produces both in-plane and out-of-plane response in the slab and girders. This interdependence between in-plane and out-of-plane actions is commonly referred to as coupling action. While coupling has little effect on the structural response in the elastic region, it does have a significant effect on the inelastic or nonlinear structural response as explained in detail in Fritz Laboratory Report No. 400.20 (Ref. 37).

Since the response due to overloading is expected to cause nonlinear stress-strain behavior, the complete stress-strain relationships of the component materials must be included. The analysis scheme developed by Hall and Kostem for BOVAS utilizes the biaxial stress-strain relationships developed in Refs. 30, 32, 33, 35, and 40 to describe the inelastic behavior of concrete slabs, and in addition, utilizes the uniaxial stress-strain relations developed in Refs. 16, 17, 25, 27, and 41 to describe the inelastic response of steel.

To account for the variation of material properties through the depth of the slab and the girders, the finite elements used in BOVAS are subdivided into a series of layers (Fig. 53). Each layer is assumed to have its own distinct material properties and is also assumed to be either in a state of uniaxial or biaxial stress. Thus the progression of nonlinear behavior through the structure can be monitored in BOVAS by defining the stress-strain
relationship on a layer by layer basis.

The basic BOVAS model also reflects the effects of shear deformation since plate girders with thin webs often deflect considerably more than standard beam theory would predict. In addition, because plate girders are of thin walled open-sections, they are susceptible to local buckling phenomena prior to attaining maximum stress conditions. Thus, BOVAS is capable of predicting the occurrence of local buckling and any post-buckling strength of such sections.

The above paragraphs present the major structural phenomena that effect the behavior of steel bridge superstructures. The developers of BOVAS excluded some structural phenomena which were of secondary importance to the model's development. Excluded from the BOVAS analysis technique are minor axis bending of the girders, shear punch failure of the slab, torsional stiffness of the girders, and superelevation.

4.2.2. BOVAS Model Assumptions.

This section will address only the assumptions pertinent to the specific features particular to the BOVAS program. These assumptions are described in detail in Fritz Laboratory Report No. 432.6 (Ref. 18) and are summarized here for the sake of completeness. A detailed treatment of the general finite element method as applied to BOVAS is presented in a number of related reports and references (Refs. 17, 18, 25, 27, 37, 40, and 46) and will not be repeated herein.

The following assumptions were made in the development of the BOVAS program. This analytical model is presently capable of analyzing steel bridges having the following characteristics:
1. The bridge can be of simple span or continuous construction.

2. Fully or partially composite interaction between the deck slab and the girders is assumed.

3. The bridge deck must be a monolithic reinforced concrete slab.

4. Steel girders of varying or constant cross-section may be considered.

5. Girder spacings must be constant for a given bridge.

6. The contributions of the diaphragms and cross-bracing to the stiffness of the structure are not included.

7. The effects of the vertical and longitudinal stiffeners are considered to be local and are neglected in the overall structural model. However, the effects of the vertical stiffeners are included in the shear panel buckling analysis.

8. It is assumed that the bridge girders may deform in shear and major axis bending.

9. The stresses in the slab are due to the biaxial bending of the slab and the axial forces that may develop in the deck slab in the longitudinal and transverse directions.

10. The bridge superstructures to be analyzed are limited to right bridges.

11. Plane section remain plane before and after deformation of the slab and girder except that a Timoshenko approach has been used to include shearing deformation in the girder.

12. The plate and beam finite elements are layered, each layer having its own stiffness properties, so as to accurately model material nonlinearities and progress-
sive material failure.

13. When the average stress of all the compression flange layers of any beam elements exceeds the critical buckling stress, the compression flange is assumed to buckle. In order to model the post-buckling strength, layers which exceed the critical buckling stress are assigned low stiffness values. Similarly, when the average stress of the web plate panel reaches the critical stress (buckling stress) all of the web layers of the entire web plate panel are assigned lower stiffness values.

14. Other less significant assumptions can be found in the reports related to BOVAS development (Refs. 16, 17, and 18).

4.2.3. BOVAS Solution Scheme.

The BOVAS solution scheme solves the overload problem in a logical sequence of operations. The solution process consists of four main phases:

1. Problem definition.
2. Dead Load Solution.
4. Overload Solution Procedure.

A simplified logical flow chart of the sequence of operations for program BOVAS is shown in Fig. 164. More detailed descriptions of the four main phases are presented in the following subsections based upon the information in Ref. 18 and included here for completeness and continuity.

4.2.3.1. Problem Definition.

This phase defines the particular problem that will be solved. To define the problem, two groups of information are required to be input into the program. They are:
1. Bridge description.
2. Bridge loadings.

The amount of information required to define the bridge is structure dependent. In order to fully describe the bridge superstructure and loadings the following information must be provided:

1. Bridge superstructure geometry.
2. Finite element discretization.
3. Slab description and material properties.
4. Girder description and material properties by layer.
5. Location of any web plate panels.
6. Location and type of any fatigue details.
7. Dead load acting on the structure.
8. Live load ("vehicle") to be investigated by the program.

In order to define the loading, the magnitude and the location of all loads must be provided. The live load is typically positioned such that a worst case analysis results.

4.2.3.2. Dead Load Solution.

Since the analytical modeling scheme employed by BOVAS considers material nonlinearities, which are stress dependent, an accurate assessment of the stress state prior to the application of the live load is required. Because of the possible nonlinear behavior of the structure, BOVAS cannot employ the principle of superposition. Therefore, prior to the application of the live load, the structure must be analyzed by BOVAS to obtain the stresses in the slab and girders due to the dead load. The initial stress state and any material failures or nonlinearities due to the application of these dead loads will thus be reflected.
by the BOVAS program prior to the application of the live load.

4.2.3.3. Scaling Procedure.

As long as the initial solution due to the overload produces a linearly elastic response, BOVAS increases the load proportionally to the lowest load level corresponding to one of the following element stress limitation:

1. 60% of the compressive strength of concrete.
2. 90% of the tensile strength of concrete.
3. 97.5% of the yield strength of the steel.
4. 100% of the buckling stress.

Because BOVAS scales up the initial load level, only one elastic solution is obtained. Thus, the number of elastic solutions are kept to a minimum. Subsequent solutions will exhibit a nonlinear response.

However, if the initial solution causes any material or stability failure, the initial live load is scaled down in order that a linear elastic solution can be obtained. Then the scaled down solution is incremented until nonlinear response occurs. Once nonlinear response begins, the overload solution is employed.

4.2.3.4. Overload Solution.

The overload solution is solved using a tangent stiffness approach (a piecewise linearization of the nonlinear phenomena). In such an approach the system of equations is assumed to be linear in a given load increment. By computing the tangent to a stress-strain curve for each layer based upon the current stress state, the layer stiffnesses, element stiffness, and ultimately the global stiffness is calculated. After calculating the nodal point
displacements and element layer strains for the load increment, the corresponding element layer stresses are obtained by BOVAS for the load increment by employing the material stress-strain relationships. These incremental stress values are added to the total stress state which existed prior to the application of the load increment, thus arriving at a new current stress state. The process is repeated (iterated) with a new current stress state until the solution for the increment converges. If a layer fails during the application of the load increment, the load increment is scaled down so that the layer stress causes incipient failure. Thus, the stiffness matrices are continually updated with each load increment or step. The initial solution of each load cycle within BOVAS is based upon zero stress and displacement increment values; thus, the first iteration of each load step is based upon the stiffness matrix of the previous load cycle. The overload analysis process terminates when one of the specified termination checks is exceeded.

Allowable limits on deflections, live loads, stresses, strains, number of cracked/crushed/yielded layers, and deck slab crack widths can be specified for the reinforced concrete slab and/or steel girders to define the serviceability limits of the bridge superstructure. These checks are used to terminate the BOVAS overload solution if any of the specific serviceability limits is exceeded.

4.2.4. Practical Considerations Concerning the BOVAS Program.

The present version of BOVAS has its origins in the early 1970's based upon the work of Kulicki, Peterson, and Kostem (Refs. 25 and 37) dealing with the inelastic analysis of reinforced and prestressed concrete beams.
During the subsequent decade their original program was modified and adapted by several other researchers with various programming styles and different end uses. During this same period of time, widespread and frequent changes in computer operating systems and programming languages occurred. Program BOVAS reflects the diversity of its development. Modifications to this program must be approached with extreme care.

The resulting program is relatively large and complex as is characteristic of many nonlinear finite element programs. For example, the program has 80 subprograms contained in ten different overlays (Ref. 12). A compressed listing of the program's source code is 188 pages long. Understanding the content and flow of the program is a challenge to even the most experienced programmer/engineer.

The input data stream must be prepared and formatted with great care. The user manual instructions for data input (Ref. 24) is in excess of 200 pages long.

Executing the program for a single structural configuration (1400 degrees-of-freedom) and load case is costly and time consuming. An execution run typical of this research used about 5500 CPU seconds using a CYBER 850 computer with the NOS Version 2.4.3 operating system. In terms of real time, a single program execution took about 7.5 hours.

In conclusion, BOVAS is a long, complex program created by multiple researchers over more than a decade of development and revision. Use and modification of this program must be approached with great care by the researcher.
4.2.5. Exercising the Original BOVAS Program.

Before attempting to modify program BOVAS to suit the purposes of this research, the author fully exercised program BOVAS to ensure it was operational using the CYBER 850 computer and its NOS Version 2.4.3 operating system at Lehigh University (Ref. 8). Minor cosmetic changes were made in the output data for better user readability. All user manual examples were attempted in addition to several self-generated bridge problems. The resulting output data verified the accuracy and operability of program BOVAS for properly modeled structures.

4.3. Bridge Overload Analysis - Steel - Damaged (BOVAS-D).

4.3.1. Development of BOVAS-D.

4.3.1.1. Introduction.

Program BOVAS is an excellent analytic tool that can be used to model the nonlinear-inelastic behavior of multigirder bridges. Unfortunately, BOVAS cannot be utilized for the nonlinear inelastic analysis of DAMAGED multigirder bridges with severe damage.

Section 4.2 provided an overview of the primary features of BOVAS and the assumptions upon which it is based. BOVAS assumes full composite action between the deck slab and its underlying steel girders, and full continuity of the structure at all internal node points. The last phrase is emphasized since it goes to the heart of the difficulty of using BOVAS to model damaged multigirder bridges.

As seen in Fig. 162, beam nodes connect adjacent...
beam elements. The BOVAS program in its fundamental formulation assumes that there is only one beam node between adjacent beam elements and the degrees-of-freedom associated with that beam node are unique. This is certainly a basic assumption for all continuous structures; unfortunately, severe damage dictates that the structure's continuity be broken - an impossibility when using the BOVAS program.

The reader is cautioned about thinking of program BOVAS in the same manner as today's general purpose finite element programs. Unlike a general purpose finite element program, BOVAS does not permit the user to define node location, node numbering, element numbering, or element connectivity. The user input primarily addresses geometric and material properties. The only major finite element modeling required of the user is selecting the number of longitudinal finite elements in the structure (even the number of transverse elements are dictated by the program). As a special purpose program which addresses multigirder bridges only, BOVAS performs all the other modeling functions such as assigning node coordinates, numbering node points, determining the number of transverse slab elements, identifying fixed and free degrees-of-freedom, and numbering the beam/slab elements. However, from its inception and throughout its formulation, BOVAS assumes full continuity of the structure.

4.3.1.2. Procedure.

A major task of this research was to modify the BOVAS program so that it would allow the modeling of damaged multigirder bridges with severe damage to the steel girders(s). The most straightforward method of accomplishing this was to break the continuity between beam elements:
adjacent to the damage at their common node point; in violation of the basic assumption of the BOVAS's formulation.

In order to break the continuity between adjacent beam elements, two (rather than one) node points were defined at each damage location. Each node point had its own unique degrees-of-freedom. These two nodes shared the same initial coordinate location, but were not connected in any manner. The adjacent beam elements were then defined to have different end nodes at their adjacent edges bordering the damage.

Fig. 165 is a simple portrayal of this process. The top diagram of Fig. 165 is an elevation view of a portion of the intact bridge model parallel to a steel girder. Four neighboring elements are portrayed, i.e. SLAB A, SLAB B, BEAM A, and BEAM B. The adjacent slab elements are joined at node 3, the adjacent beam elements at node 4.

The bottom diagram of Fig. 165 portrays the bridge modeled for severe damage between BEAM A and BEAM B. An extra node (node 7) has been created with the same coordinates as node 4. Nodes 4 and 7 have unique degrees-of-freedom. Element BEAM A is defined by nodes 2 and 4; element BEAM B by nodes 7 and 6. Hence, the girder has now been modeled for a full girder depth fracture between BEAM A and BEAM B.

While the concept is very basic in concept, it was very difficult in application. Since the entire 188 page BOVAS program was written based upon full structural continuity, the entire program had to be checked for dependence upon this assumption.

Several new algorithms had to be written to establish the damage modeling technique and many of the original processes had to be modified to properly incor-
porate the new procedure. At the same time, the author tried to maintain all the beneficial features of the original BOVAS program which performed a very sophisticated analysis of multigirder bridges for nonlinear-inelastic behavior.

After considerable time and programming effort, the above mission was accomplished. The resulting program was named Bridge Overload Analysis - Damaged (BOVAS-D) in recognition of its origin and its new capabilities.

4.3.2. Features of BOVAS-D.

The new program has maintained most of the beneficial features of the BOVAS program as described in section 4.2 of this report. The new features incorporated into the BOVAS-D program of interest to the user are described below.

1. BOVAS-D can be used for the nonlinear-inelastic analysis of either intact or damaged multigirder bridges. Even though program BOVAS models intact multigirder bridges, program BOVAS-D can be used for either intact or damaged multigirder bridges. The features of BOVAS-D incorporating damage modeling are overridden if the user is modeling an intact structure.

2. BOVAS-D provides for the existence of a full girder depth fracture at the damage location. This means that the damaged model assumes that the lower flange, entire web, and upper flange is fractured. However, the slab above the fracture is assumed to be intact when loading begins. The reader is reminded that the linear elastic damage model employed in Chapters 2 and 3 of this report employed a lower flange and full web depth fracture; the upper flange remained intact. In additional, a special feature has been built into the BOVAS-D model which can
provide for partial web depth cracks. This is discussed in section 4.3.3 to follow.

3. The BOVAS-D damage does not have to be at any specific longitudinal location on the girder. There is also no requirement that the damage be on a specific girder. Hence, the damage can be at any longitudinal location on any of the parallel girders of a multigirder bridge.

4. BOVAS-D provides for damage at one or two damage locations. The user is not limited to one damage location as was done with the previous SAPIV model. If there is more than one damage location, there is no predetermined relationship between the two damage locations. That is, they can be on the same girder, neighboring girders, nonadjacent girders, etc.; the only requirement is that the damage locations be different.

5. The damage input data provided by the user is extremely simple. The user inputs the girder number where the damage occurs and the longitudinal coordinate of the damage. The program will internally compute the location of the nearest nodal point to the damage, add the appropriate nodal point(s), break the continuity, and readjust all internal parameters and algorithms for the new connectivity.

6. In support of the BOVAS-D program, a complete interactive program called "PREBOV" has been written to prompt the user for all required input for the BOVAS-D program. This program produces a properly formatted input data file required by the BOVAS-D program. It is hoped that this program will relieve the user of some of the tedium of preparing a properly organized and formatted data file; and make the program more acceptable for general use. The details of the required input data file for BOVAS-D and the BOVAS-D "preprocessor" are described in Ref. 31. It
should be mentioned that the BOVAS-D preprocessing program can also be used to prepare input for the original version of the BOVAS program.

7. A complete interactive program called "POSTBOV" has also been written in support of BOVAS-D to assist the user in analyzing the output file resulting from executing the BOVAS-D program. This "postprocessing" program allows the user to extract output data that is of interest to him/her in a convenient and readable form. The user is relieved of the time consuming task of finding and extracting data from lengthy BOVAS-D printouts. The program was not specifically designed for the models discussed in the next chapter, and should be of assistance to any user of the BOVAS-D program. This program can also be used to postprocess data from output files of the original version of the BOVAS-D program.

4.3.3. Additional Feature - The Bilinear Spring.

In the past decade, elastic-plastic fracture mechanics has made advances in relating crack-tip opening displacement (CTOD) to the applied stress and crack length (Ref. 4). To incorporate present and future advances in elastic-plastic fracture mechanics, the BOVAS-D model has been modified to allow for the placement of a translational spring between the two beam nodes on either side of the girder crack(s).

The model allows each spring to have the properties shown in Fig. 166. Each spring can be thought of as a axial force member (truss member) with uniaxially bilinear properties. Its initial stiffness is defined by $K_0$ which is valid until the force in the spring is $P_M$. When the spring force exceeds $P_M$, the stiffness is $K_T$. There is no requirement that the spring have elastic-
perfectly plastic properties with $K_T$ equal to zero.

The user input to BOVAS-D is actually in terms of initial modulus ($E_0$), tangent modulus ($E_T$), spring area ($A$), spring length ($L$), and cut-off stress ($\Sigma M$). Any convenient values can be chosen by the user such that they are related to initial stiffness ($K_0$), tangent stiffness ($K_T$), and cut-off force ($P_M$) by:

$$K_0 = E_0 \times A / L$$

$$K_T = E_T \times A / L$$

$$P_M = \Sigma M \times A$$

The user must know the characteristics of the spring(s) to be used based upon the user's CTOD analysis. Each damage location can have its own separate bilinear translational spring. The "preprocessing" and "postprocessing" programs mentioned in the previous subsection can be used in conjunction with the bilinear spring model.

Several test runs were conducted to ensure that the model responded correctly for various values of $K_0$, $K_T$, and $P_M$. For very large values of $K_0$, the behavior of the model is almost identical to the intact structure. For low $K_0$ values, the response parallels the complete girder depth damaged structure. Intermediate values of $K_0$ produced a response between these two extreme levels. These intermediate values could logically be used to simulate partial girder cracks of lengths determined by a complete CTOD analysis. The results achieved were logical. The author is confident that the model can be used satisfactorily once the CTOD parameters are properly selected for a given crack.
condition.

Further investigation of the CTOD approach is beyond the scope of this dissertation and left for future study. However, it is to emphasized that the model has already been modified to accept a researcher's CTOD parameters. No further manipulation of the program code is required.

4.3.4. Limitations of BOVAS-D.

The assumptions of the BOVAS model were itemized in section 4.2.2 of this chapter. The only assumption which does not hold for the BOVAS-D model is the assumption of full continuity of the structural model. All of the other assumptions/limitations of BOVAS hold for the BOVAS-D program.

In addition to the limitations inherent with BOVAS, the BOVAS-D program has some limitations unique to its use. Some of the more prominent limitations include:

1. BOVAS-D program will analyze only simple span bridges. In contrast, the BOVAS program addresses simple or continuous multigirder bridges.

2. In its present form, the user of BOVAS-D cannot identify more than two damage locations.

3. The damage invoked by the BOVAS-D program is a full girder-depth fracture (except as provided in section 4.3.3).

4. Nodal points must exist at the presumed damage. If this is not the case, the program will place the damage at the closest beam nodal point on the given girder.

5. Due to the arbitrary nature of the damage location, the program will not allow any symmetry options to be used as in the original BOVAS program.
Other special considerations for the use of the BOVAS-D program are outlined in Ref. 31.

4.4. The Three Dimensional Model Investigated.

4.4.1. Actual Structure.

The basic bridge of investigation in Phase II of this research will be the 100 foot multigirder bridge with welded A36 steel girders and reinforced concrete deck slab; designed for HS20-44 vehicle loading as extracted from Ref. 13. This is the one of the three multigirder bridges used during Phase I of this research. Section 2.1 of this report provided a full description of the 100 foot bridge to be modeled in this phase of the research. While the bridge used for both phases is the same, the reader is cautioned from inferring that the model is the same in the two cases. This is definitely not the case as will be discussed in subsequent sections of this chapter.

4.4.2. Modeling of the Steel Girders.

The modeling of the multigirder bridge's parallel girders is depicted in Figs. 167, 168, and 169. Fig. 167 shows that the model includes six parallel girders spaced at 94 inches center-to-center. Each of these six girders has been divided into 18 longitudinal beam elements of varying lengths as depicted in Fig. 168. The finer element discretization is in the vicinity of the midspan where the damage to the girder and the maximum longitudinal response is anticipated.

Each of the beam elements is further subdivided into 10 layers are depicted in Fig. 169. Four layers comprise the girder's web; three layers for each of the
girder's flanges. The depth of the layers was dictated by the nonprismatic nature of the bridges girders. Flange dimensions which were less than the flange's midspan dimensions were modeled by giving certain layers of the steel girders an artificially low modulus of elasticity; as if they did not exist.

A summary of the important material properties used in the Phase II model of the steel girders is included in Table 5. Note that the steel properties are more detailed than those required for a linear elastic analysis.

4.4.3. Modeling the Concrete Slab.

Fig. 170 depicts how the slab deck was modeled for the BOVAS-D analysis. There are 18 longitudinal slab elements to insure proper connectivity with the 18 beam elements on each girder. The model has 12 transverse slab elements; two elements between each adjacent girder and one element overhanging each exterior girder. The SAPIV model used in Phase I had the same transverse discretization.

Each slab element is further subdivided into 10 layers; six concrete layers and four reinforcement layers. Fig. 171 depicts the six concrete layers of the slab. The dimensions of these layers and important concrete material properties have been listed in Table 6. The thickness of these layers has been computed to provide a minimum concrete cover of 2.50 inches on the top slab surface, and 1.00 inches at the bottom slab surface.

The four layers of reinforcing steel are described in Table 7. The most important aspect of the reinforcing steel layers that should be noted is the orientation of these layers. The top and bottom steel layers are oriented in the transverse direction, i.e. perpendicular to the girders; the middle two layers in the lon-
gitudinal direction, i.e. parallel to the girders. Of course, the slab is reinforced more heavily in the transverse direction than in the longitudinal direction.

4.4.4. The Model's Boundary Conditions.

Since the boundary conditions used in Phase I of the study were discussed in great detail (see section 2.5), the BOVAS-D support conditions will be briefly described. Prior to doing this, the reader must be informed of the coordinate system used by the BOVAS-D model as depicted in Fig. 162; which differs from the Phase I coordinate system.

When using BOVAS-D, the x, y, and z coordinate axes are oriented in the longitudinal, transverse, and vertical directions respectively.

The BOVAS-D model includes minimal longitudinal restraint at the ends of the girders. The only nodes that are prevented from translating in the longitudinal (x) direction are on the far side of the bridge model at the slab level above the beams. Transverse (y) displacement is prevented for all nodes at both the near and far end of the structure. In addition, 12 nodes are prevented from vertical (z) translation on both the near and far ends of the bridge model. These are the 6 beam nodes and the 6 slab nodes directly above beam nodes.

No rotation is allowed about the x (torsional) axis or the z (minor axis bending) axis at any node at either end of the structure. Rotation is permitted about the y (major axis bending) axis.

These boundary conditions do not exactly match any of the three support conditions used in the Phase I study of multigirder bridges. However, the BOVAS-D support conditions are closest to the expansion-expansion conditions used with the SAPIV model and described in section 110.
2.5.2 of this report.

4.4.5. The Damage States Investigated.

Section 4.3 described the various features of the BOVAS-D permitting up to two damage locations on any girder at any longitudinal location. This section also discussed the possibility of using a bilinear spring at any damage location to model partial web depth cracks. However, this research will employ the BOVAS-D model to investigate a damage configuration very similar to that used in Phase I of this research.

The only damage state that will be pursued for the remainder of this report is severe damage to one exterior girder of the 100 foot multigirder bridge. The imposed damage will consist of severing the bottom flange, full web depth, and top flange of the exterior girder at its midspan. The motivation for doing this is the same as expressed for Phase I of the study. That is, while it is expected that only a few bridges might ever develop such fractures without the damage being noticed, this condition serves as a useful upper bound. This "damaged" or "fractured" condition will be compared to the same bridge model when undamaged/intact.

The reader is again reminded that this study does not include the crack tip stresses and fatigue crack propagation. In a sense, the damage is "frozen", and then the analysis is conducted.


Due to the anticipated nonlinear behavior of the structure, BOVAS-D cannot apply the principle of superposition. A more detailed discussion of this is included in section 4.2.3.2. Therefore, prior to the application of
the live load, the structure must be analyzed by BOVAS-D to obtain the stresses in the slab and girders due to the dead load. As such, all responses reported in the next chapter are total effects, i.e., the dead load plus whatever live load is placed on the structure at that time.

The live (vehicle) loads are applied to the model in increments as discussed in sections 4.2.3.3 and 4.2.3.4 of this report. Phase II of the study will use two different live loads; the HS20-44 vehicle and the 204-kip PennDOT Permit vehicle. These live loads were chosen because they provided the minimum and maximum absolute response for the parameters investigated in the Phase I study (see section 3.3). It was also felt that these two live load types were of the most interest to practicing bridge engineers.

The live loads were positioned in a similar manner to the method used in Phase I of the study. That is, the live loads were positioned to ensure the worst practical effects in the vicinity of the damage location. Figs. 8, 9, and 10 picture the location of the two vehicles. The live load were applied as a series of rectangular wheel loads using the guidelines of section 3.30 of Ref. 3 and without application of an impact factor.

4.4.7. Trial Runs.

Several test runs were made of the BOVAS-D model of the 100 foot multigirder bridge to test the programs operability and to see if reasonable results were achieved. Trial runs were critical since there was no actual physical structure with which to compare the model's performance. The major source of comparison was the SAPIV model used in Phase I of this study.

It is important to remind the reader that the SAPIV and BOVAS-D models are fundamentally very different.
The models differ in the number of longitudinal elements, dimension of the elements, application of the dead load, and support conditions. The employment of secondary members is completely different for the two models. Even the theoretical formulation of the beam and slab elements are fundamentally different for the two models. Of course, what is most significant is that the SAPIV model assumes complete linear elastic behavior of the bridge model, while no such assumption is made using the BOVAS-D model.

However, since one would expect generally comparable results for simple loads of small magnitude on the intact models, limited comparisons were made. The purpose of the comparisons is simply to confirm the general accuracy of the BOVAS-D model. The reader is reminded that comparisons have already been made of the SAPIV model with conventional composite beam theory (see section 2.2.4).

The deflection data is the best comparison between the two models. Comparing stress at a specific location is only approximate since the reported BOVAS stress is an average stress over an entire layer of finite dimensions; the SAPIV stresses have been extrapolated to a specific point.

The primary trial runs consisted of applying a uniform line load of 1 kip per foot over each of the six girders in both the SAPIV and the BOVAS-D models. For the intact structure the results are reasonably close as can be seen from Table 8. All deflection data is with 8% of each other; the stress comparisons are also reasonably close. The intact BOVAS-D model appears to be more flexible than the intact SAPIV model. This is certainly anticipated since the BOVAS-D model is without secondary members and is governed by nonlinear-inelastic material properties.

As expected, the SAPIV and BOVAS-D models behave
quite differently after damage occurs. Once the BOVAS-D model is damaged, the structural response is very nonlinear as will be shown in Chapter 5. After damage, the deflection response away from the damage (in the vicinity of G3) is very similar for the SAPIV and BOVAS-D models as can be seen from Table 9. The stress results are reasonably close.

After damage, the deflection response in the vicinity of the damage (in the vicinity of G5 and G6) is considerably different for the SAPIV and BOVAS-D models (Tables 10 and 11). The response of the damaged BOVAS-D model can be two to three times greater than the damaged model.

Looking at Tables 9 through 11, it appears that the damaged SAPIV model distributes the load effects more to the remote girders than the damaged BOVAS-D model. As a result, the BOVAS-D model creates greater effects in the vicinity of the damage.

A final comparison was made between the SAPIV and BOVAS-D models by applying the actual dead load and a single HS20-44 vehicle load to the intact structure. Tables 12 and 13 show that the cited data corresponds within about 16%. Again, the BOVAS-D model appears to create the greatest effects in the vicinity of the live load; the SAPIV model appear to distribute the largest effects away from the live load.

In summary, the limited comparisons between substantially different models of the same 100 foot multigirder bridge are reasonably close. The validity of the BOVAS-D model to represent the actual nonlinear-inelastic behavior of the intact and damaged structure is confirmed.
5. THE NONLINEAR INELASTIC RESPONSE OF A MULTIGIRDER BRIDGE

5.1. Parametric Study Procedure.

5.1.1. Data Gathering Process.

The previous chapter addressed how BOVAS-D was used to model the 100 foot multigirder bridge for nonlinear-inelastic analysis. The subsequent sections of this chapter will discuss the results of using this model for the overload and/or damage analysis of this structure when loaded with a HS20-44 vehicle or a 204-kip PennDOT Permit vehicle.

The finite element analyses were conducted using program BOVAS-D on a CYBER 850 computer using the NOS Version 2.4.3 operating system. The BOVAS-D execution runs were costly and time consuming. The average execution time for 1400 degrees-of-freedom with 30 load steps was 5500 CPU seconds for the several runs forming the basis of the Phase II research.

Data output from the execution runs was long and detailed. Each output file produced for the reported research is in excess of 2000 pages long. A "postprocessing" program called POSTBOV (see section 4.3.2) was developed to expedite the data reduction and to strip "nonessential" data from the output file. Much of this postprocessed data was incorporated into LOTUS 1-2-3 (Ref. 34) spreadsheets for fabrication of graphical displays of the results. These iterations of data reduction produced a final display of the critical data of most interest to the researcher.
5.1.2. Primary Analysis Items.

Section 3.1.3 discussed the primary items of interest to bridge engineers in understanding intact and damaged bridge response using the SAPIV model. While most of these same items continue to be of interest when using the BOVAS-D model, some of them were not applicable or inappropriate. It was felt that the following analysis items were of greatest use to bridge engineers:

1. The deflection profile at the level of the slab. As previously discussed for the SAPIV model (see section 3.1.3), deflection profiles give the best "feel" for how the structure physically deforms.

2. The normal stress in the lower girder flange. The rationale for including this is the same as previously outlined in section 3.1.3.

3. Load versus deflection graphs. These graphs give a very clear picture of the incremental response for each load step. Limits of linear behavior can be readily identified and associated with other meaningful limit states. These graphs sometimes give an indication of the maximum load supportable by a structure.

4. Progression of yielding in the steel girders by layers. The BOVAS-D model has a clear advantage over most finite element models. It allows the monitoring of the load redistribution once nonlinearities begin to occur. Since girder yielding is such a critical structural limit state, its spread was traced as the live load increased.

5. Progression of cracking/crushing in the slab. Since the BOVAS-D model continually adjusts the stress in a given layer based upon its cracking/crushing state, monitoring the slab stress level throughout the structure becomes complex and unmanageable for analysis. Of much greater use to the engineer is monitoring the spread of
cracking/crushing in the slab. This gives a clear picture of load redistribution as the live load is applied and incremented.

5.1.3. Nomenclature and Conventions.

The BOVAS-D program applies the total load to the structure in two stages: the dead load on the structure followed by various increments of the live load. The live load is not necessarily (and not normally) applied in increments matching the vehicle's gross weight. However, the live load is always applied via wheel loads in the same proportion as that existing for one actual vehicle. For example, if an increment of load for the HS20-44 vehicle load was 36 kips (or half of the vehicle's gross weight), it would be applied by incrementing each wheel load by half its actual full vehicle weight. Hence, proper proportionality of all live loads is maintained. Section 4.2.3 provides further details on this process.

The live load used were the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads with gross weights of 72 kip. and 204 kips respectively. The researcher found that it was useful to normalize these two live loads using the same normalization factor. The normalization factor used was 72.0 kips, the actual gross weight of one HS20-44 vehicle load.

A normalized load will be identified by the letter "H" following a numeric value. For example, a live load of "5.6H" has a magnitude of 403.0 kips (i.e. 5.6 * 72.0). The context of the narrative will clearly identify whether this load is an HS20-44 vehicle or 204-kip PennDOT Permit vehicle load.

As a final note, even though the coordinate system of the BOVAS-D model is different from the SAPIV model.
(see section 4.4.4), the other conventions of the SAPIV model will be used. This includes the girder numbering system and use of such terms as "midspan", "centerline", "near", "far", "left", and "right" as defined in section 2.2.5.

5.2. The Inelastic Overload Response of an INTACT Multigirder Bridge.

5.2.1. Overview.

The remainder of this chapter will deal with presenting the results of the nonlinear-inelastic analysis of the 100 foot multigirder bridge. The results will be presented in two major sections. The present section will address the response of the intact multigirder bridge to HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads; the next section (section 5.3) will discuss the damaged multigirder bridge. The detailed discussion will concern the HS20-44 vehicle load; followed by shorter discussions emphasizing the major differences between the 204-kip PennDOT Permit vehicle and HS20-44 vehicle load responses.

Table 14 is a tabular summary of all load cycles occurring during the analysis of the intact 100 foot multigirder bridge for HS20-44 vehicle and 204-kip PennDOT Permit vehicle loading. Even though the results are shown in the same table, the analyses for the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads were accomplished on independent runs of the BOVAS-D program.

Each of the execution runs was terminated after 28 cycles (increments of load) since it was felt that the structure was approaching the upper level of serviceability. Continuing the loading for more cycles beyond
this would involve increasingly small load increments. The research was primarily interested in the bridge response from zero live load to just passed the serviceability limits. Prediction of response beyond this load level is of "academic interest". This did not seem warranted based upon the uncertain benefit for the high cost in terms of computer resources required.

The first load cycle (i.e. load increment) for each load type indicates a value of 0.0. This first cycle gives the results when the dead load alone acts on the structure; its actual total weight being 734.1 kips. Subsequent load cycles add increments of live load to this dead load.

Table 14 expresses the live load in two ways; its absolute value in kips and its normalized ("H") value obtained by dividing the absolute live load value by 72.0 as previously described. The table shows that the intact structure was loaded to a final live load of 6.2H and 8.6H when loaded with the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads respectively. The value of the live load at this final cycle of loading has no special physical significance; i.e. it is not the ultimate or collapse load of the structure.

Presentation of the results for all 28 load cycles shown in Table 14 will only be done for select cases. A large number of load cycles is difficult or impossible to present in a form "digestible" to the reader. Hence, the author has selected six load cycles from the total 28 load cycles for closer examination. An attempt was made to choose these six "data points" to represent an adequate range of both the linear and nonlinear range of bridge response.

The six data points selected for each live load
type are listed in Table 15. Data point number one (DP1) in both cases is the dead load response; data point six (DP6) is the final load cycle. Data point four (DP4) in both cases is the cycle at which girder yielding was first observed. The other three data points were chosen to complete the range of expected linear and nonlinear response.

Before proceeding with more detailed results, a preliminary comparison will be noted based upon the data of Table 15. Note that the first yield occurs at a 30% higher gross live load for the 204-kip PennDOT Permit vehicle load (457.8 kips) compared with the HS20-44 vehicle load (353.3 kips). While this is expected based upon the different distribution of the loads; the percentage value of 30% appears to surface in other results presented in the following sections. It is also noteworthy that this exact percentage was previously noted as the difference between the maximum total lower flange stress of the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loaded model of the 100 foot intact bridge using the SAPIV model.

5.2.2. Vertical Deflection of an Intact Multigirder Bridge.

5.2.2.1. HS20-44 Vehicle Loading.

Fig. 172 depicts the live load versus total (DL + LL) midspan deflection curves for all six girders of the 100 foot intact bridge with HS20-44 vehicle loading. The initial (LL = 0.0) deflection is not zero since there is a dead load deflection when the live load is first applied.

All live load versus midspan deflection curves appear to be linear up to the first girder yield which occurs at 4.9H. This linear behavior exists until this point even though extensive cracking exists prior to this as will
be seen in section 5.2.5.

Gl and G2 are almost unaffected by the live load at the other side of the structure. This is shown very dramatically in Figs. 174 and 176 which show the Gl and G2 deflection profiles for all 28 cycles of live load from 0.0H through 6.2H. These figures are not presented for extraction of a specific deflection at a specific load cycle; but to demonstrate the narrow band of Gl response for widely varied loads. As can be seen in Fig. 173 the maximum Gl deflection shows a very slight upward deflection from 1.057" at 0.0H to 1.036" at 4.9H. From 4.9H to 6.2H the Gl maximum deflection remains at 1.036". The G2 maximum deflection changes from 0.993" to 1.029" (a 4% increase) from 0.0H to 4.9H (Fig. 175). After G6 yields at 4.9H, the G2 deflection only increases to 1.036" as the load increases to 6.2H.

G3 and G4 are only significantly influenced by the live load prior to the yielding of G6. This is shown most dramatically in Figs. 178 and 180. In each case one can clearly see the first nine cycles of response prior to the yield of G6. After the yield of G6 at 4.9H, further deflection response of either G3 or G4 is limited to a narrow band of behavior even though the live load is increasing from 4.9H to 6.2H. Figs. 177 and 179 show the above response in more detail. Specifically, there is a 17% and 54% increase in the G3 and G4 deflection responses respectively as the load changes from 0.0H to 4.9H. However, from 4.9H through 6.2H there is no further change in the G3 response and only a 2% increase in the G4 response.

G5's deflection profile increases significantly throughout the entire 28 cycles of loading. There is a pronounced change of slope of G5's live load vs. deflection curve (Fig. 185) after G6 yields at 4.9H. Specifically the
slope of the live load vs. deflection curve (a pseudo-spring stiffness) is about 258 kips per inch prior to the yield of G6; about 140 kips per inch near the final cycles of the load.

The G5 deflection increases by 138% from 0.993" at 0.0H to 2.363" at 4.9H (the yielding load of G6) as can be seen in Fig. 181. After yielding of G6, G5 deflects further to 2.908" as the load increases to 6.2H. While there has been a 61% increase in the total load (dead load plus live load) from 0.0H to 6.2H, there has been a 193% increase in G5's deflection response. In contrast, the G3 and G4 deflection increases are 17% and 56% for the same increase in the total (dead plus live) load.

As expected, the most significant pre-yielding and post-yielding effects occur at the edge girder, G6, as can be seen in Figs. 183, 184, and 186. The G6 deflection increases from 1.057" to 3.392" (a 221% increase) as the load changes from 0.0H to 4.9H (the yield load). The deflection increases further to 4.813" at the final load cycle of 6.2H. Hence a 61% increase in the total load (dead load plus live load) has increased the total G6 deflection response by 355%.

The slope of the live load vs. deflection curve (Fig. 186) for G6 changes drastically after it yields. Prior to yielding at 4.9H, the slope of the live load vs. deflection curve is approximately 150 kips per inch. Over the final cycle of loading, the slope is only about 36 kips per inch. Extension of the live load vs. deflection curve indicates that a zero slope will probably occur prior to the load reaching 7.0H, or about 40% over the first yield level. Whether or not 7.0H represents the actual "ultimate" live load, the live load vs. deflection curve clearly points out that the structure has significant post-
yielding reserve strength.

Figs. 187 and 188 give a different perspective of the deflection response of a 100 foot multigirder bridge under HS20-44 vehicle loading. These figures show the transverse deflection profile at midspan for all (Fig. 188) and selected (Fig. 187) load cycles. These curves reinforce the points made previously:


In addition, these curves highlight the large transverse curvature in the vicinity of G4. This is even more pronounced than previously demonstrated for the SAPIV models (see Chapter 3).

5.2.2.2. 204-kip PennDOT Permit Vehicle Loading.
The live load vs. midspan deflection curves for the 204-kip PennDOT Permit vehicle loading (Fig. 189) is qualitatively very similar to that for the HS20-44 vehicle loading (Fig. 172). The major difference is that the gross live load limit of linear behavior for the 204-kip PennDOT Permit vehicle load is 30% higher than for the HS20-44 vehicle load. However, the deflection at the limiting load of linear behavior is very close for the two load cases; 3.39" for the HS20-44 vehicle load and 3.52" for the 204-kip PennDOT Permit vehicle load (only a 4% difference).

As previously seen with the SAPIV model, for the same gross vehicle weight the maximum deflection response is less for the 204-kip PennDOT Permit vehicle load than for the HS20-44 vehicle load. For example, at a load of 4.0H (within the linear range for both load types) the G6
deflection is 2.94" and 2.59" for the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads respectively. Outside the linear range, the differences are much greater.

Figs. 190 through 199 are submitted with little additional explanation. They exhibit the same characteristics as the equivalent figures for the HS20-44 vehicle load. Specifically, they show:

4. Large transverse curvature in the vicinity of G4, especially after the yield of G6.

It is interesting to compare the midspan deflection of all six girders at yielding of G6. The comparison is shown in Table 16. Note the small difference between the midspan deflections of all girders even though the load levels are 30% different. It appears that the deflection profile of the structure at first yield is independent of live load type.

The live load vs. deflection curve of G6 for the intact bridge with 204-kip PennDOT Permit vehicle loading (Fig. 197) shows a pre-yielding slope of about 186 kips per inch. Over the final two cycles of load the slope is reduced to about 30 kips per inch. It appears that the asymptote of this curve is approximately $P = 9.0H$. It is interesting to note that this asymptote is about 30% higher than the asymptote for the HS20-44 vehicle loading ($P = 7.0H$); approximately the same percent difference as the percent difference between the two loading's initial yield loads. The live load vs. deflection curve clearly depicts the significant post-yielding capacity of the in-
tact structure which appears to be almost 40% higher than the first yield load.

5.2.3. Girder Lower Flange Stress in an Intact Multigirder Bridge.

5.2.3.1. HS20-44 Vehicle Loading.

The Gl and G2 lower flange stresses are barely effected by increases in the live load as can be seen in Figs. 200, 201, 202, and 203. In fact, both of these girders have a 5% decrease in their midspan stress as the live load increases from 0.0H to 4.9H (the G6 yield). After G6 yields, the Gl and G2 lower flange stress remains almost constant for the remaining load cycles. G3 is also effected very little by live load increases (Fig. 205). The lower flange stress is 8.44 ksi at 0.0H and 8.60 ksi at 6.2H. This represents only a 2% stress increase as the total (dead load plus live load) load increases by 61%.

G4 exhibits a particularly interesting behavior as seen in Figs. 206 and 207. When the load changes from 0.0H to 4.9H (the G6 yield), the G4 flange stress increases 44% from 8.44 ksi to 12.17 ksi. Further increases of load above 4.9H results in a decrease in the G4 flange stress such that it is 11.73 ksi at the last load cycle of 6.2H. Hence the increase of live load from 4.9H to 6.2H (a 26% increase) results in a small decrease (4%) in stress. Hence, while there is a significant pre-yielding stress increase, the post-yielding stress change is of little engineering significance.

G5's flange stress increases appreciably both before and after the yielding of G6 as can be seen in Figs. 208 and 209. Its lower flange stress increases 163% from 8.82 ksi at 0.0H to 23.21 ksi at 4.9H (the G6 yield).
After G6 yields, as the live load increases from 4.9H to 6.2H (a 26% increase), G5's lower flange stress increases from 23.21 ksi to 29.31 ksi (a 26% increase also). Hence G5 is the only girder experiencing a significant post-yielding increase in its midspan flange stress.

While it is somewhat interesting to note that the percent increase in G5's post-yield stress is approximately equal to the percent increase in the post-yield live load (26% in both cases), this is only coincidental. The post-yield relationship between live load and the G5 maximum lower flange stress is not linear as can be seen in Fig. 210. It is interesting to note that the extension of this figure appears to indicate the yielding of G5 will occur at approximately 7.0H. Note that a load of 7.0H was the approximate asymptote of the G6 live load vs. deflection curve for the intact structure as pointed out in section 5.2.2.1.

Figs. 211 and 212 show the G6 lower flange stress as the live load increases. As expected, G6 has the greatest lower flange stress because of the live load's location. Even though the maximum flange stress remains the same after the load of 4.9H is reached, there is appreciable longitudinal load redistribution as the live load is increased. The extent of this "plastification" will be described in more detail in section 5.2.4.

In summary, Figs. 213 through 218 show the relationship between the lower flange stress of all girders at each of the six selected load cycles. These figures reinforce the points previously detailed:

1. The G1, G2, and G3 lower flange stresses are virtually unaffected by live load increases occurring at the other edge of the structure.
2. The G4 lower flange stresses only increase
significantly prior to the first yield of G6; subsequent stress changes are minor.

3. While both the G5 and G6 flange stresses increase appreciably prior to the first yield; only G5 experiences significant post-yielding midspan stress increases.

5.2.3.2. 204-kip PennDOT Permit Vehicle Loading.

Figs. 219 through 230 for the 204-kip PennDOT Permit vehicle loading of the intact model are very similar to the equivalent curves already discussed for the HS20-44 vehicle loading. Both type of loads clearly demonstrate the same qualitative characteristics itemized in the last paragraph of the previous subsection.

Table 17 lists the midspan lower flange stress of G1 through G5 for both the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads at the yield load of G6. Even though the gross loads are about 30% different at yield, the midspan lower flange stress are very close for all girders. This might indicate that the maximum flange stress of each girder at first yield in the structure is independent of the live load type.

Comparison of Figs. 211 and 224 points out a qualitative difference between the G6 lower flange stress response for the 204-kip PennDOT Permit vehicle compared to HS20-44 vehicle loads. There is a more significant longitudinal load distribution for the 204-kip PennDOT Permit vehicle than the HS20-44 vehicle loading. While being noted here, this point will be examined more closely in the following section.
5.2.4. The Girder Yield Pattern in an Intact Multigirder Bridge Model.

5.2.4.1. HS20-44 Vehicle Loading.

Figs. 231, 232, and 233 are elevation views of G6 at load levels of 4.9H, 5.6H, and 6.2H. While these figures are not drawn to scale in the vertical direction, they do properly depict the relative size of the elements in the longitudinal direction. The blackened areas depict layers of elements which have yielded as of a given load level.

Fig. 231 shows the initial yielding of G6 which occurs at a load of 4.9H. Yielding is through about half the lower flange depth near midspan, and over a longitudinal length of about 75 inches. The yielding quickly spreads to neighboring flange elements/layers. The first web element to yield occurs near midspan at a load level of 5.3H. Fig. 232 shows that by a load of 5.6H there has been extensive yielding of web layers and the extreme flange fibers have yielded over about one-fourth of the span.

The final load cycle is depicted in Fig. 233. By a load of 6.2H, G6 has yielded through about half of its depth around midspan. There is extensive flange and web yielding for about one-fourth of the span.

These three figures give a clear idea of how yielding progresses as the load increases, and how it is rapidly spread once it initiates at 4.9H. It is important to remind the reader that yielding has been confined to spreading within G6, and has not spread to adjacent girders.
5.2.4.2. 204-kip PennDOT Permit Vehicle Loading.

As previously stated, the first yield of the intact bridge with HS20-44 vehicle loading is at a gross load of 4.9H; while G6 yields at 6.4H with 204-kip PennDOT Permit vehicle loading as seen in Fig. 234. Initial girder web yielding occurs at about 7.0H for the 204-kip PennDOT Permit vehicle loading; compared to 5.3H for the HS20-44 vehicle loading. Hence, the G6 web yields at about a 30% higher gross live load for the 204-kip PennDOT Permit vehicle load than for the HS20-44 vehicle load. Besides these quantitative differences, there is a qualitative difference in how the yield spreads longitudinally.

Fig. 233 depicts G6 of the intact bridge with HS20-44 vehicle loading at a gross load of 6.2H, or about 1.3H above its first yield of 4.9H. Fig. 235 shows the same girder with 204-kip PennDOT Permit vehicle load at a gross load of 7.7H, or about 1.3H above its first yield of 6.4H. Hence, the figures show G6 at the same increment of load above yield for each of the load types. While the HS20-44 vehicle loaded intact bridge with a load of 6.2H has about 13% of its web yielded to its half depth; none of the 204-kip PennDOT Permit vehicle loaded bridge with a load of 7.7H has yielded to that degree. The HS20-44 vehicle loaded bridge has flange yielding over about 33% of G6's length; the 204-kip PennDOT Permit vehicle loaded bridge has flange yielding over about 50% of G6's length. Hence, for an equivalent increment of load above yield, the HS20-44 vehicle load produces deeper yielding while the 204-kip PennDOT Permit vehicle loading produces wider yielding.

As expected, the wider pattern of yielding for the 204-kip PennDOT Permit vehicle load is very nonsym-
metrical with respect to the midspan. The yielding is much more pronounced at the far half of the structure compared to the near half. This is due to the fact that the front four axles of the 204-kip PennDOT Permit vehicle load are far forward as seen in Fig. 11.

The final cycle of 204-kip PennDOT Permit vehicle loading is depicted in Fig. 237. This load is about 2.2H above G6's first yield. Note that the midspan of the girder has yielded more than half way through, and flange yielding has extended to about 58% of the G6 length.

5.2.5. Reinforced Concrete Slab Cracking/Crushing Patterns for an Intact Multigirder Bridge Model.

5.2.5.1. Overview.

Presentation of the reinforced concrete slab results will be very qualitative. It is felt that this will give the reader an appreciation for slab behavior on multigirder bridges. The reader is reminded that comparing the BOVAS-D model with the SAPIV model for slab stress is very imprecise; the BOVAS-D model takes cracking and crushing into effect while the SAPIV model does not.

This paper will give "snapshots" of the slab status at the six "data points" discussed in section 5.2.1. At each of the six data points every slab element was reviewed to see if it had cracked or crushed, and the number of layers involved. The analysis data has been pictorially represented on a plan view of the reinforced concrete slab. The figure has been drawn to approximate scale in both the longitudinal and transverse direction. As previously discussed, there are 12 transverse elements and 18 longitudinal elements. The bold longitudinal lines (from left to right in the figure) represent the orientation of
each of the steel girders. Six possible conditions are represented for each slab element. They are:

1. No layers cracked or crushed.
2. One layer cracked.
3. Two adjacent layers cracked. This would be an indication that the reinforcing steel was exposed.
4. Three, four, or five, adjacent layers cracked. This would be an indication that the reinforcing steel was exposed and that the slab was cracked at least half way through.
5. Six adjacent layers cracked. This would indicate that the slab was cracked all the way through its depth.
6. Combined cracking of two or more adjacent concrete layers combined with the crushing of two or more adjacent layers in the same element. This would be an indication of the possible formation of a "yield line" in the concrete slab with the concurrent cracking and crushing of multiple concrete and/or steel layers.

If a specific element had conditions (2) through (6) existing, the element was marked with the appropriate pattern as shown near the bottom of Fig. 238. In this way the reader can get a qualitative feel for the progression of slab damage as the live load increases.

5.2.5.2. HS20-44 Vehicle Loading.

While no slab cracking/crushing occurs when the slab is loaded by dead load alone (Fig. 238), initial slab cracking does occur at the relatively small load of 1.1H (Fig. 239). Cracking initiates in the area between G4 and G5 near midspan in the top slab surface. These cracks are in the longitudinal direction caused by high tensile stresses in the transverse direction. Cracking also occurs
at the lower surface near the G6 left support. This slab crack is diagonal to the longitudinal direction of the superstructure, resulting from the differential movement of the slab's nodes at unsupported edges relative to the slab nodes over the supports/girders.

By the time the load reaches 3.2H, most of the elements in the neighborhood of G4 have cracked as can be seen in Fig. 240. These cracks are a result of the large transverse curvature in the vicinity of G4. This large transverse curvature was evidenced in both the SAPIV model (Fig. 20) and the BOVAS-D model (Fig. 188). Hence these cracks are at the top slab surface in the longitudinal direction. Notice that several of the elements have exposed reinforcement since two layers are cracked. Cracking has also continued in the vicinity of the supports at G5 and G6. These cracks result from the high shear force in the vicinity of the supports resulting in cracks inclined at about 40° to 50° to the longitudinal direction.

Fig. 241 shows the slab status at the yield load of G6. Most of the elements neighboring G4 have two or three of their top layers cracked. Cracking has also begun at the top surface around G3, caused by high transverse tensile stresses near the top surface. Diagonal tension cracking continues to increase in the vicinity of the supports. In fact, all six slab layers have cracked in the vicinity of the G6 right support. Longitudinal cracks are also present near the midspan of G6 at the load of 4.9H.

Figs. 242 and 243 depict the slab situation at the post-yielding of 5.6H and 6.2H respectively. The same cracking trends already identified continue to dominate slab behavior. By 6.2H, most of the longitudinal cracks near G4 extend half way through the slab. These cracks tend to divide the structure into two separately acting
structures:

1. The top structure defined by G5 and G6 (and neighboring slab) which actually carries most of the post-yield live load.

2. The bottom structure defined by G1, G2, and G3 which does not participate in supporting the live load.

Cracks also continue to grow in the vicinity of the G5 and G6 supports at both end of the structure. These cracks are primarily directed diagonally to the longitudinal direction.

5.2.5.3. 204-kip PennDOT Permit Vehicle Loading.

The cracking pattern of the slab on the intact structure under 204-kip PennDOT Permit vehicle loading is very similar to the pattern under HS20-44 vehicle loading. While there are minor differences between the HS20-44 vehicle and 204-kip PennDOT Permit vehicle slab patterns, both are dominated by the following types of cracks:

1. Longitudinal cracks at the upper slab surface in the central 75% of the span in neighboring G4 caused by the large transverse curvature of the slab due to the eccentric live load.

2. Diagonal cracks in the lower slab surface near the G5 and G6 supports resulting from the differential movement of edge nodes relative to nodes over the supports/girders.

For both load cases, initial slab cracking occurs at a gross load of 1.1\textsuperscript{u}. The similar patterns for the two load types can be seen by comparing Figs. 240 and 246. The gross loads are comparable for these two "snapshots", and both loads are less than their respective loads at the first yield of G6. The cracking of the HS20-44 vehicle loaded slab is only slightly more severe than the 204-kip...
PennDOT Permit vehicle loaded slab.

The similarity between load types at the first yield of G6 can be seen from comparing 274 and 281. The cracking of the 204-kip PennDOT Permit vehicle slab is comparable even though its gross load is 1.5H more than the HS20-44 vehicle loaded slab.

The final "snapshot" of the 204-kip PennDOT Permit vehicle loaded slab is shown in Fig. 249 at 8.6H. All slab elements neighboring G4 and the supports at G5 and G6 have cracked through at least half of the slab depth.
5.3. The Inelastic Overload Response of a DAMAGED Multigirder Bridge.

5.3.1. Overview.

While the previous section described the response of the intact 100 foot multigirder bridge model, this section describes the damaged bridge response using the BOVAS-D model. Table 18 is a summary of all load cycles when loading the model with the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads. The model loaded with HS20-44 vehicle underwent 28 cycles; the model with 204-kip PennDOT Permit vehicle had 27 cycles.

Table 19 lists the six data points selected for closer examination for each of the load types. DP1 in both cases is the dead load response; DP6 is the final load cycle. DP3 and DP4 are the cycles at which the reinforcing steel first yields and the girder steel first yields respectively. The other two data points were selected to complete the range of expected linear and nonlinear response.

Before proceeding with the presentation of the detailed results, some preliminary comparisons will be made. As seen from Table 19, the first yield of girder steel on the damaged model occurs at 2.6H and 3.4H for the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads respectively. Each of these live loads represent an approximate 46% reduction from their intact models levels. In other words, for both load cases the live load at first girder yield on the damaged model is about half of the intact live load at first girder yield. The reduction is expected; however, the percent reduction being the same for each load type is noteworthy.

Also of interest is that the first girder yield
for the damaged model loaded with 204-kip PennDOT Permit vehicle occurs at approximately a 30% higher live load level than the damaged model loaded with the HS20-44 vehicle. The is the same percentage as the same relationship for the intact model as described in section 5.2.1.

There are two major qualitative differences between the response of the intact and damaged structures that should be noted from the beginning of this section. First, the intact structure had no observable nonlinear behavior under dead load alone. This is not true for the damaged structure. As will be seen in the subsequent subsections, under dead load alone the damaged structure has large deflections, high girder stresses, and substantial cracking of the slab. Hence, a critical state for analysis is when dead load acts on the structure alone.

The second major difference is that the first girder yielding on the damaged model occurs on G5, the girder adjacent to the damaged girder rather than on the exterior girder itself. This was expected based on the Phase I study (see section 3.2.3).

5.3.2. Vertical Deflection of a Damaged Multigirder Bridge.

5.3.2.1. HS20-44 Vehicle Loading.

The live load vs. midspan deflection curves (Fig. 250) give an excellent feel for the overall deflection of a damaged multigirder bridge. G1 through G4 are virtually unaffected by an increase in the live load in the vicinity of G5 and G6. The response of G5 is linear up to the load that G5 yields (2.6H); G6 is approximately linear up to the load at which the reinforcement yields (1.8H).

The dead load deflection of G1 is virtually identical for either the intact or damaged bridge (about 1.06")
as can be seen by comparing Fig. 172 with Fig. 251. In both cases there is a slight upward deflection of G1 as the live load increases. Even at a live load of 4.3H, G1's midspan deflection is about the same for both the intact (1.03") and the damaged (1.02") structures. Fig. 252 depicts the very small range of deflection response of G1 for a wide range of live loads varying from 0.0H to 4.3H.

G2 and G3 also display an upward deflection as the live load increases. The dead load deformation on the damaged structure is slightly less than the dead load deflection for the intact structure as can be seen from Figs. 175 and 253. That is, the damage itself creates a slight upward deflection at the remote girders. However, in the case of both G2 and G3, the change in the deflection profile is minor throughout all 28 cycles of load.

The deflection profile of G4 (Figs. 257 and 259) is significantly influenced by the live load only after the yielding of G5. As the load changes from 0.0H to 2.6H (the G5 yield), G4's deflection only increases 2% from 0.88" to 0.90". Upon yielding of G5 at 2.6H, G4's deflection response increases more rapidly as the live load increases; reaching a maximum value of 1.12" at 4.3H.

G5's dead load deflection is 59% higher for the damaged structure than the intact structure as can be seen by comparing Figs. 181 and 259. G5 then experiences significant deflection increases throughout the entire 28 cycles of loading (Fig. 260). Specifically, the maximum deflection increases from 1.57" at 0.0H to 5.44" at 4.3H, a 246% increase in the total deflection for a 42% increase in the total load. By examining G5's live load vs. deflection curve (Fig. 263), one can see that its response is linear with a slope of about 115 kips per inch up through its yield load of 2.6H. After it yields, there is a pronounced
and continual slope decrease; approaching an approximate asymptote of about 5.0H. While this asymptote is only an approximation, it does show that even the damaged structure has significant post-yielding reserve strength. A 40% increase in the post-yielding strength would appear to be conservative.

The maximum dead load deflection of G6 on the damaged structure is over four times higher than its maximum dead load deflection on the intact structure as can be seen from Figs. 183 and 261. G6 experiences very large deflection increases throughout the entire 28 cycles of loading. Its deflection profile is in the exact same shape as previously seen with the SAPIV model, i.e., the "middle hinge" type profile. Its maximum deflection increases from 4.38" at 0.0H to 19.2" at 4.3H, a 338% increase. Its live load vs. deflection curve (Fig. 264) is approximately linear (slope of 29 kips per inch) up to the first yield of the reinforcement steel at 1.8H. By the final load of 4.3H, the live load vs. deflection curve's slope is only about 12 kips per inch; with an approximate asymptote of 5.5H.

Figs. 265 and 266 show the transverse deflection profile at midspan for the damaged multigirder bridge. These curves dramatically point out the key points discussed previously concerning the damaged bridge deflection response:


In addition, these curves highlight the large
transverse curvature between G4 and G5. The point of greatest transverse curvature appears to have shifted toward G5 in comparison to its location close to G4 for the intact structure as can be seen by comparing Figs. 188 and 265. The severity of this curvature is very important in understanding the overall response of the damaged structure. Finally, note the large transverse curvature near G5 in the presence of dead load alone, which was certainly not the case for the intact structure.

5.3.2.2. 204-kip PennDOT Permit Vehicle Loading.

The deflection response of the damaged bridge under 204-kip PennDOT Permit vehicle loading (Fig. 267) is qualitatively very similar to that for the HS20-44 vehicle loading on the damaged bridge (Fig. 250). The major difference is that the gross load limit for G5's linear behavior for the 204-kip PennDOT Permit vehicle load (3.4H) is 30% higher than for the HS20-44 vehicle load (2.6H). A 30% difference also existed between the gross yield loads for the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads on the intact structure (see section 5.2.2.2).

As seen with the BOVAS-D model of the intact bridge, the maximum deflection response of the damaged bridge for the same gross vehicle weight is more for the HS20-44 vehicle load than for the 204-kip PennDOT Permit vehicle load. For example, at a load of 2.5H, the G5 midspan deflection is 3.16" and 2.82" for the HS20-44 vehicle and 204-kip PennDOT Permit vehicle loads respectively. After G5 yielded, the difference between the deflection profiles of G5 for the same gross vehicle weight widens.

Fig. 268 through 277 are submitted with little additional explanation. They exhibit the same general
characteristics as the equivalent figures for the HS20-44 vehicle load on the damaged bridge. Specifically, they show:

4. Large transverse curvature between G4 and G5.

It is interesting to compare the midspan deflection of all six girders at the initial yield of G6 (see Table 20). At the initial yield of G5, the maximum deflection of the two load types are very close even though the gross weight is very different. This indicates that the deflection profile of the structure might be independent of live load type.

Unfortunately, no clear asymptote is discernible from the live load vs. deflection curve of G5 and G6 on the damaged bridge with 204-kip PennDOT Permit vehicle loading. Hence no comparison can be made with the asymptote of the live load vs. deflection curve of the damaged bridge with HS20-44 vehicle load discussed in section 5.2.2.

5.3.3. Girder Lower Flange Stress in a Damaged Multigirder Bridge.

5.3.3.1. HS20-44 Vehicle Loading.

As previously noted, the primary difference between the lower flange stress in the intact and damaged models is the location of the maximum stress. That is, the maximum stress occurs in G6 for the intact structure; in G5 for the damaged structure. With dead load alone, the maximum flange stress increases 69% when the structure is
damaged as can be seen by comparing Figs. 213 and 290. A maximum intact stress of 9.56 ksi (in G6) compared to a maximum damaged stress of 16.21 ksi (in G5).

The G1 flange stress is virtually unchanged as the live load increases (Fig. 278). The midspan flange stress of 9.65 ksi at 0.0H decreases by only 1% to 9.53 ksi at 4.3H. The narrow range of the G1 response is dramatically reflected in Fig. 279.

G2 and G3 experience a gradual decrease in their flange stress as the live load increases as can be seen from Figs. 280 and 282. This decrease results from the upward deflection discussed in section 5.3.2.2. Specifically, there is a 16% decrease in the midspan lower flange stress of G2 from 8.50 ksi to 7.15 ksi as the live load increases from 0.0H to 4.3H. The G3 stress decreases by 24% from 7.70 ksi to 5.82 ksi during the same loading process. A large portion of this change occurs prior to the initial yielding of G5.

The behavior of G4 is particularly interesting, and demands close examination of Fig. 284. Under the initial increments of live load, G4 behaves much like G2 and G3; i.e. the flange stress decreases as the live load increases. However, after the initial yield of G5 at 2.6H, subsequent increases in the live load induce an increase in the G4 lower flange stress. Specifically, as the load increases from 0.0H to 2.6H, the G4 midspan flange stress decreases from 7.19 ksi to 6.03 ksi (a 16% decrease). Subsequently, increases of load after G5 yields from 2.6H to 4.3H causes an increase of stress in G4 from 6.03 ksi to 7.80 ksi (a 29% increase). The net effect of total load increases of 42% from 0.0H to 4.3H is a net 9% increase in the G4 stress. In other words, the yielding of G5 causes an increased participation of G4 in supporting the live
load.

The behavior of G5 on the damaged structure is very similar to that of G6 on the intact structure. This is most clearly observed in Figs. 212 and 287. The most significant difference is that the initial dead load stress is much higher for the damaged structure than for the intact structure. After its initial yield at 2.6H, G5 develops a definite yield plateau as had been observed with G6 after it yielded at 4.9H on the intact structure. The extent of this longitudinal "plastification" will be discussed further in the next section of this report.

G5 has a unique response which was not observed with G6 on the intact structure. Figs. 286 and 287 show that G5's maximum flange stress reaches stress levels exceeding the yield stress of 36 ksi. This is possible because BOVAS-D incorporates strain-hardening behavior into its material modeling. Because of limited load increments at the strain-hardening regime of the stress-strain curve, this response will not be studied in any detail.

G6 on the damaged model demonstrates the expected behavior and profile as previously observed for the damaged SAPIV model as can be seen by comparing Fig. 288 and 24. In both the SAPIV and BOVAS-D models the maximum flange stress for one HS20-44 vehicle with dead load occurs at about 0.30L. For both models, the maximum stress is about 7 ksi for one HS20-44 vehicle with dead load.

The BOVAS-D model has a grosser element discretization in the vicinity of the damage than the SAPIV model. While the midspan SAPIV beam elements for the 100 foot bridge are 6 inches long, the BOVAS-D midspan elements are 25 inches. Since the stress portrayed in Fig. 288 is an average element stress, the reader should not expect the midspan element stress to be zero even though they are ad-
jacent to the girder fracture.

From Fig. 288 it is interesting to note that G6 has significant load carrying reserve even though it is severely damaged. The composite nature of the construction allows G6 to contribute to supporting live load despite its full girder depth fracture.

Figs. 290 through 295 depict the relationship between the lower flange stress of all six girders at each of six selected load cycles. These figures reinforce the important points previously discussed:

1. The G1 stress is virtually unaffected by live load increases.
2. The G2 and G3 stress decreases in magnitude as live load increases. The G2 and G3 lower flange stresses are less than the G1 stress.
3. The G4 lower flange stress decreases prior to G5's yield, then increases after G5's yield.
4. The G5 lower flange stresses are the highest of all the girders on the damaged structure. G5 first yields at the relatively small live load of 2.6H.
5. Both G4 and G6 demonstrate appreciable flange stress increases after the yielding of G5.

5.3.3.2. 204-kip PennDOT Permit Vehicle Loading.

Figs. 296 through 307 for the 204-kip PennDOT Permit vehicle loading are very similar to the equivalent curves already discussed for the HS20-44 vehicle loading on the damaged structure. Both type of load clearly demonstrate the same qualitative characteristics previously itemized in the last paragraph of the last subsection.

Table 21 lists the midspan stress of all unyielded girders at the initial yield of G5. Even though the gross live load is about 30% different at yield, the
lower flange stresses near midspan are very close for all girders. It appears that the midspan lower flange stress of all girders at yield are independent of load type.

By comparing Figs. 288 and 301 for the damaged structure, the reader can see that the HS20-44 vehicle loads and 204-kip PennDOT Permit vehicle loads distribute themselves much differently on G6, the damaged girder. Since the 204-kip PennDOT Permit vehicle load has its four front axles well forward of the midspan (see Fig. 11), a stress "plateau" has been created at about \( x = 0.75L \). The stresses here are much higher than at the same point for the HS20-44 vehicle loading of the damaged bridge. For example, at the same gross load of 2.5H, the G6 lower flange stress at \( x = 0.75L \) is 9.70 ksi and 7.50 ksi for the 204-kip PennDOT Permit vehicle and HS20-44 vehicle loads respectively. This is one of the only times we have seen greater effects for the 204-kip PennDOT Permit vehicle load in comparison to the HS20-44 vehicle load at the same gross load. For example, at \( x = 0.25L \) and a load of 2.5H on the damaged structure, the stress is 7.60 ksi and 9.44 ksi for the 204-kip PennDOT Permit vehicle and HS20-44 vehicle loads respectively; more similar to the relationship we have noted in the previous data examined in this chapter.

We will examine the longitudinal stress pattern of the yielded girder more thoroughly in the next section.

5.3.4. The Girder Yield Pattern in a Damaged Multigirder Bridge Model.

5.3.4.1. HS20-44 Vehicle Loading.

Fig. 308 shows the initial yielding of G5 occurring at a load of 2.6H. Yielding spreads very rapidly with increased live load. By a load of 2.7H, the entire flange
has yielded at midspan and for about 150 inches along the longitudinal direction as can be seen in Fig. 309.

Initial girder web yielding of G5 occurs at the midspan at a live load of 3.0H. By 3.4H web yielding has spread to about 17% of the girder's length; in addition about 30% of the lower flange's length has yielded (Fig. 310).

The final girder "snapshot" (Fig. 311) is portrayed at the final cycle of 4.3H. Note that the web has yielded more than halfway through at midspan and that 30% of the web's length has partially yielded. Almost half of the lower flange has completely yielded.

Comparison of the yield patterns of G5 on the damaged bridge with G6 on the intact bridge show very similar results for a given increment of load above yield load. The biggest difference is that yield in the damaged structure initiates at 2.6H, in the intact structure yielding begins at 4.9H.

5.3.4.2. 204-kip PennDOT Permit Vehicle Loading.

As already stated, the first yield of the damaged bridge with HS20-44 vehicle loading is at a gross load of 2.6H; while G5 yields at 3.4H with 204-kip PennDOT Permit vehicle loading (a 30% increase). The initial yielding pattern is shown in Fig. 312. Initial girder web yielding of the damaged bridge with 204-kip PennDOT Permit vehicle loading occurs at the midspan at 3.9H, a 30% increase over the live load level at web yielding on the damaged bridge with HS20-44 vehicle loading.

Fig. 313 depicts G5 at a load of 4.4H after extensive yielding of its lower flange and lower web. This figure is very similar to the HS20-44 vehicle loading of the damaged bridge at 3.4H (see Fig. 310) even though there
is a 30% difference in the live load. The main difference is that for the HS20-44 vehicle loading the spread of the yield is short of the midspan; the 204-kip PennDOT Permit vehicle load spreads the yield on the far side of the midspan.

Fig. 315 shows the yield pattern at 5.1H. It is interesting to compare this figure with Fig. 311 for the HS20-44 vehicle loading of the damaged bridge at 4.3H. The two figures are almost mirror images of each other except that the HS20-44 vehicle loaded bridge has deeper yielding at midspan. As expected, the 204-kip PennDOT Permit vehicle loaded bridge is more heavily yielded forward of midspan; the HS20-44 vehicle loaded bridge on the near side of midspan.

The yielding pattern of G5 under 204-kip PennDOT Permit vehicle loading of the damaged bridge also appears to have a more compact pattern than the yielding pattern of G6 on the intact bridge. Figs. 235 and 314 depict the yielding patterns for G6 on the intact bridge and G5 on the damaged bridge. Both figures represent the girder status when the 204-kip PennDOT Permit vehicle load is 1.3H above their respective initial yield load. Note that the yield pattern is more compact and deeper for the G5 yielding pattern; wider and less deep for the G6 yielding pattern. Hence, the midspan damage is shifting the effects toward the midspan of the structure.

5.3.5. Reinforced Concrete Slab Cracking/Crushing Patterns for a Damaged Multigirder Bridge Model.

5.3.5.1. Dead Load Response.

The most significant difference between the slab response of the intact and damaged multigirder bridges is
the amount of cracking and crushing under self-weight alone. As already observed there is no cracking/crushing under dead load alone for the intact bridge; minor longitudinal cracks first appear at 1.1H. The damaged model behaves much differently.

Fig. 316 shows the large amount of cracking existing on the damaged bridge without application of any live load. The slab can be regarded as two distinct areas separated by a line running longitudinally midway between G3 and G4. Half of the slab is completely uncracked and undamaged; the other half on the damaged side of the bridge has at least two adjacent layers cracked for each element. This would indicate that the slab reinforcement is exposed throughout the damaged half of the structure.

With only self-weight acting on the damaged bridge, the cracking/crushing patterns generally fall into the following categories:

1. CATEGORY L: Longitudinal cracks caused by high tensile stresses in the transverse direction at the upper slab surface. These cracks are primarily located on both sides of G4 and G5, in the central 75% of the longitudinal length. These cracks could be expected based upon the large transverse curvature observed and discussed in section 5.3.2 of this paper.

2. CATEGORY D: Diagonal cracks at the top and/or bottom surface of the slab in the vicinity of the G5 and G6 supports. These cracks also appear in the neighborhood of G6 at about one-third the span.

3. CATEGORY T: Transverse cracks at the bottom surface of those elements on either side of G6, and about 25 inches away from the midspan. These cracks are caused by high tensile stresses in the longitudinal direction induced by girder damage in their near proximity.
4. CATEGORY Y: Concurrent crushing of the top slab surface together with cracking of the bottom slab surface in the slab on either side of G6 and adjacent to the midspan. Since G6 is fully cracked at this location, the slab is supporting the full major axis bending action of the composite beam. Hence, both high tensile and compressive stresses are acting in the longitudinal direction, creating a "yield line" type crack at the midspan over G6.

5. CATEGORY C: Cracking by combined action in those "transition" areas between the various zone characterized by the categories defined by 1 through 4 above.

Fig. 317 is a sketch of the slab with the approximate zones of behavior marked to give the reader a more qualitative perspective of slab behavior.

5.3.5.2. HS20-44 Vehicle Loading.

Application of live load increases the severity of the cracking and crushing of the slab. However, the qualitative nature of the cracking/crushing remains about the same as the first increments of live load are applied.

Fig. 318 gives the slab status at a load of 1.1H. Almost all of the slab on the damaged side of G4 has half slab depth cracks. A few elements near the G5 and G6 supports have cracked through all six layers. The slab of the damaged bridge with a load of 1.1H is much more severely cracked/crushed than the intact bridge with a load of 6.2H.

At a load of 1.8H, almost all of the elements on the damaged half of the bridge have cracked half way through as can be seen from Fig. 319. More than 30% of the elements neighboring G6 have cracked through all six layers.

A significant result at a load of 1.8H is that the slab reinforcement has yielded for the first time.
Specifically, the top layer of transverse reinforcement neighboring G5 at midspan has yielded. This yielding results from the high tensile stresses in the transverse direction due to the large concave curvature around G5. Since the concrete has cracked through the top half of the slab, the reinforcing steel is the sole remaining load carrier.

By a load of 2.6H, all elements on the damaged half of the bridge are cracked half way through as can be seen in Fig. 320. Over 65% of the elements neighboring G6 are cracked all the way through. These cracks are generally oriented diagonally to the longitudinal axis of the bridge. Yield of the top transverse reinforcing steel layer has spread for a longitudinal distance of about 100 inches on each side of the midspan near G5. In addition, by a load of 2.6H there has been compression yielding of the longitudinal steel at midspan around the G6 crack. This yielding has developed due to the high compressive forces that the steel must support after the adjoining concrete has crushed.

Fig. 320 also shows that the concurrent cracking and crushing of two or more layers has spread to the midspan area near G5 by a live load of 2.6H. However, this "yield line" type behavior is much different from the "yield line" previously existing around G6 at midspan. The one near G5 runs in the longitudinal direction caused by large transverse bending; the one near G6 runs in the transverse direction caused by large longitudinal bending around the G6 crack.

By a load of 3.4H a very well defined "yield line" has formed as can be seen in Fig. 321. It runs transverse to the bridge around the G6 midspan, and then runs parallel and near to G5. In fact, the transverse
reinforcement has yielded to a longitudinal distance of about 200 inches on each side of the midspan near G5; that is, for about 33% of the span. Both the top and bottom transverse reinforcement has yielded in those elements bordering the G5 midspan due to the large amount of cracking/crushing in the neighboring concrete.

The final slab "snapshot" is shown in Fig. 322 for a load of 4.3H. The longitudinal yield line has now spread to more elements neighboring G5. The top transverse reinforcement has yielded to a distance of 250 inches on each side of the midspan near G5; that is, to over 40% of the span. In addition, both the top and bottom layers of longitudinal steel have yielded near the G6 midspan. Minor cracks have also started to appear for the first time in the longitudinal direction around G3 at the top slab surface.

5.3.5.3. 204-kip PennDOT Permit Vehicle Loading.

The cracking/crushing patterns of the damaged structure under 204-kip PennDOT Permit vehicle loading is markedly similar to the pattern under HS20-44 vehicle loading. There are even less differences than those cited previously between 204-kip PennDOT Permit vehicle and HS20-44 vehicle loads on the intact bridge. This is so because the slab cracking/crushing response of the damaged bridge is dominated by the dead load effects. These severe initial effects are independent of the live load.

The reader should note the close similarities between Figs. 318 through 322 and Figs. 323 through 327. For both the HS20-44 vehicle and 204-kip PennDOT Permit loads, as the live load increases, both slabs crack/crush in a very similar manner. The only major difference is that the gross load level is greater for the HS20-44 vehicle load at
the equivalent slab cracking/crushing levels.

The similarity extends through the last reported load cycle for the two load types (see Figs. 322 and 327). The cracking/crushing pattern is slightly more advanced for the HS20-44 vehicle load, even though the gross 204-kip PennDOT Permit vehicle load is about 18% higher.

5.4. Summary of Significant Findings From the Study of Multigirder Bridges Using Nonlinear Inelastic Analysis.

1. The first girder yield for the INTACT 100 foot steel multigirder bridge model with composite reinforced concrete deck loaded with a static HS20-44 vehicle occurs at 4.9H.

2. When DAMAGED, the gross live load at first girder yield is about half of the INTACT level.

3. When the load is changed to the 204-kip PennDOT Permit vehicle load, the gross live load at first yield for both the INTACT and DAMAGED models is increased by approximately 30%.

4. There appears to be a minimum of 40% reserve capacity above the first yield live load for either the INTACT or DAMAGED bridge models with either HS20-44 vehicle or 204-kip PennDOT Permit vehicle loading.

5. For the same gross vehicle weight, the deflection response is less for the 204-kip PennDOT Permit vehicle load than for the HS20-44 vehicle load.

6. The deflection and lower flange stress profiles of the INTACT or DAMAGED structure at first yield indicates a possible independence of live load type.

7. For the INTACT bridge model, the live load versus midspan deflection curves for all girders are linear up to the first girder yield.
8. The deflection profiles of the INTACT model for either live load show:
   d. Large transverse curvature in the vicinity of G4, especially after the yield of G6.

9. The lower flange stress profiles for the INTACT structure with either load type show:
   a. The G1, G2, and G3 lower flange stresses are virtually unaffected by live load increases occurring at the other edge of the structure.
   b. The G4 lower flange stresses only increase significantly prior to the first yield of G6; subsequent stress changes are minor.
   c. While both the G5 and G6 flange stresses increase appreciably prior to the first yield; only G5 experiences significant post-yielding midspan stress increases.

10. After initial yielding occurs on either the INTACT or DAMAGED models, the yielding quickly spreads to neighboring flange and web elements/layers.

11. The spread of girder yielding is confined to the initially yielded girder for a large increment of live load beyond the initial yield level for both the INTACT and DAMAGED bridge models.

12. For an equivalent increment of load above yield, the HS20-44 vehicle load produces deeper yielding while the 204-kip PennDOT Permit vehicle loading produces wider yielding.
13. While no slab cracking/crushing occurs when the INTACT structure is loaded by dead load alone, initial slab cracking does occur at the relatively small load of 1.1H for either live load type.

14. By the time the INTACT model reaches its first girder yield load for either load type, the slab has undergone considerable cracking resulting from the large transverse curvature in the vicinity of G4. Most of the elements neighboring G4 have two or three of their top layers cracked. Very severe diagonal tension cracking exists in the vicinity of the supports. In fact, all six slab layers have cracked in the vicinity of the G6 right support.

15. The slab of the INTACT structure loaded with either the HS20-44 vehicle or 204-kip PennDOT Permit vehicle load is dominated by the following types of cracks:
   a. Longitudinal cracks at the upper slab surface in the central 75% of the span in neighboring G4 caused by the large transverse curvature of the slab due to the eccentric live load.
   b. Diagonal cracks in the lower slab surface near the G5 and G6 supports resulting from the differential movement of the slab's unsupported nodes relative to the nodes over supports/girders.
   c. For both load cases, initial slab cracking occurs at a gross load of 1.1H.

16. Under dead load alone the DAMAGED structure has large deflections, high girder stresses, and substantial cracking of the slab.

17. The first girder to yield on the DAMAGED model occurs on G5, the girder adjacent to the damaged girder rather than on the exterior girder itself. The yielding does not spread to either adjacent girder within the
range of loads of this research.

18. The response of G5 on the DAMAGED structure is linear up to the load that G5 yields; G6 is approximately linear up to the load at which the reinforcement yields.

19. The maximum dead load deflection of G6 on the DAMAGED structure is over four times higher than its maximum dead load deflection on the INTACT structure.

20. The deflection profiles of the girder on the DAMAGED model with either live load type indicate:
   d. Large transverse curvature between G4 and G5. The point of greatest transverse curvature appears to have shifted toward G5 in comparison to its location close to G4 for the INTACT structure.

21. The lower flange stress profiles of the girders on the DAMAGED structure show:
   a. The G1 stress is virtually unaffected by live load increases.
   b. The G2 and G3 stress decreases in magnitude as live load increases. The G2 and G3 lower flange stresses are less than the G1 stress.
   c. The G4 lower flange stress decreases prior to G5's yield, then increases after G5's yield.
   d. The G5 lower flange stresses are the highest of all the girders on the DAMAGED structure.
   e. Both G4 and G6 demonstrate appreciable flange stress increases after the yielding of G5.

22. With dead load alone on the DAMAGED struc-
ture, the maximum flange stress increases 69% when the structure is DAMAGED; a maximum INTACT stress of 9.56 ksi (in G6) compared to a maximum DAMAGED stress of 16.21 ksi (in G5).

23. The behavior of G5 on the DAMAGED structure is very similar to that of G6 on the INTACT structure for a given live load type.

24. G6 on the DAMAGED structure has significant load carrying reserve even though it is severely damaged. The composite nature of the construction allows G6 to contribute to supporting live load despite its full girder depth fracture.

25. The yielding pattern of G5 under either live load type on the DAMAGED bridge has a more compact pattern than the yielding pattern of G6 on the INTACT bridge.

26. The most significant difference between the response of the INTACT and DAMAGED multigirder bridges is the amount of slab cracking and crushing under self-weight alone. The slab on the DAMAGED structure can be regarded as two distinct areas separated by a line running longitudinally midway between G3 and G4. Half of the slab is completely uncracked and undamaged; the other half on the damaged side of the bridge has at least two adjacent layers cracked for each element, indicating that the slab reinforcement is exposed throughout the damaged half of the structure.

27. For the DAMAGED structure, the first yield of slab reinforcement occurs prior to the first yield of girder steel (at 1.8H on the model with HS20-44 vehicle loading). This first occurs in the top layer of reinforcing steel in the transverse direction near the G5 midspan. In addition, at a slightly higher load (2.6H on the model with HS20-44 vehicle loading) compression yielding of the
longitudinal steel at midspan around the G6 fracture.

28. The formation of a "yield line" begins under dead load alone in the DAMAGED structure. By the final cycles of recorded load, this "yield line" is well defined and runs transverse to the bridge around the G6 midspan, and then runs parallel and near to G5.
6. SUMMARY AND CONCLUSIONS.


In Chapter 1 it was noted that a major gap exists in the engineering literature in the prediction of the structural response of bridges having existing damage. While models existed for performing accurate overload analyses of steel beam-concrete slab highway bridges, little work had been done in quantifying steel bridge response when damaged.

The reported research fully investigated the structural response of fractured steel multigirder bridges in the region from design load levels to the upper level of serviceability. The research was conducted in two major phases, each employing separate analytical models of multigirder bridge behavior.

In Phase I the major factor effecting the behavior of damaged multigirder were examined using a general purpose linear elastic finite element finite element program. Detailed three dimensional models were developed which included most of the structural detail of actual short, medium, and long span FHWA multigirder bridges. The multigirder bridge response was quantified in terms of deflection profiles in the longitudinal and transverse directions, girder lower flange stress, slab stress parallel and perpendicular to the steel girders, cross bracing stresses, and support reactions.

The Phase I research provided bridge engineers with valuable information concerning:

1. The behavior of multigirder bridges with severe damage to a main load carrying member.

2. The effect of span length on the response of
a damaged multigirder bridge.

3. The response of a damaged multigirder bridge when overloaded.

4. The change in the response of a multigirder bridge if the damage involves only the fracturing of the tension flange or only a partial web depth crack.

5. The influence of various support conditions on the response of a damaged multigirder bridge.

Key quantitative data has been provided in support of the five issues addressed above. Much of this data has been summarized in section 3.7 of this report.

The Phase II research investigated the response of damaged multigirder bridges beyond the limits of linear elastic behavior. An analytical model was developed which permitted the complete overload analysis of fractured multigirder bridges. While this new modeling technique only included the bridge’s primary load carrying members, it realistically incorporates inelastic stress-strain relationships, cracking and crushing of concrete, yielding and strain hardening of reinforcing and mild steel, buckling of flanges, and buckling of girder webs.

This nonlinear model was employed to study the response of damaged multigirder bridges to normal loads and overloads. The response was quantified in terms of load versus deflection, longitudinal and transverse deflection profiles, and girder lower flange stress. Failure areas were identified, and post-failure stress redistribution was monitored for the reinforced concrete deck and steel girders. The key quantitative findings of this phase of the overall research problem have been summarized in section 5.4 of this report.
6.2. Conclusions.

Based upon the analytical data resulting from the two separate and independent models of multigirder bridge behavior, the following observations and conclusions can be noted:

1. After substantial damage at the midspan of an exterior girder of a multigirder bridge the load is redistributed, primarily among the structural components in the immediate vicinity of the damage. The response of the bridge sufficiently away from the damaged girder changes imperceptibly.

2. Simple span steel highway bridges possess very large amounts of internal redundancy, i.e. reserve strength. After the development of severe damage in an exterior girder, large increases in the stresses and deformations in the immediate vicinity of the damage are noted. However, none of these increases is high enough to result in the collapse of the superstructure when loaded with a single HS20-44 vehicle, 128-kip Dolly vehicle, or 204-kip PennDOT™ Permit vehicle.

3. The significant effects of damage are located on the damaged side of the bridge centerline. This includes deformation, girder stress, concrete slab stress, and reinforcing steel stress as evidenced by both the Phase I and Phase II results.

4. The bridge deck slab plays a critical role in the redistribution of the stresses and in the development of new load paths. This is especially apparent from the behavior of the slab of the nonlinear model which does not incorporate secondary members in its formulation.

5. The determination of the stresses and deformations of steel multigirder bridges with fractured girders can be determined with a high degree of accuracy using
three dimensional linear elastic finite element models. The presence or absence of fractures at predefined locations can be controlled via element connectivity. The computational requirement in terms of CPU-time is at least an order of magnitude faster than the nonlinear finite element model.

6. The linear elastic model is useful and practical in predicting damaged multigirder response for single vehicle loads and overloads up to and including one 204-kip PennDOT Permit vehicle. Load levels in excess of this will approach girder yield and should employ a nonlinear finite element model.

7. BOVAS-D provides a flexible analytical modeling tool for damaged multigirder bridges loaded at levels up to the ultimate load of the structure. This program can be used with full girder fractures at one or two random locations on any steel girder.

8. For the same load level, the nonlinear model produces a greater response in terms of deformation and lower flange stress than the linear elastic model. Research data based on the nonlinear model should be considered conservative.

9. The cross bracing members appear to play a significant role in redistribution of load after the exterior girder is damaged. The relative participation of cross bracing versus reinforced concrete deck is impossible to determine from this research.

10. Of the three prototype live loads considered in this research, the greatest absolute effects are achieved from loading the intact or damaged models with one 204-kip PennDOT Permit vehicle. For the same gross load, the greatest effects are gotten from an axle configuration and distribution using the 128-kip Dolly vehicle.
11. The pronounced transverse curvature in the damaged bridge model is a critical behavior to note and understand. This is a major source of cracking and non-linearity in the reinforced concrete deck for both the intact and damaged models.

12. For a given damage state, the live load response decreases as the span increases; the total load response increases as the span increases.

13. Fracturing of only the lower flange at the midspan of the "loaded" girder of a multigirder bridge has only a minor effect on the response of a multigirder bridge, even in the near vicinity of the damage. Half web depth cracks have a very significant effect on the response of multigirder bridges. A very good estimate of the half web depth response is to average the intact and full web depth fracture response. This technique should not be used for determining stresses (girder or slab) in close proximity to the damage area.

14. Deformations, maximum girder stress, midspan cross bracing stress, and slab stresses are reduced on both the intact and damaged structure by changing the support conditions from expansion-expansion to either expansion-fixed or fixed-fixed. The reduction is greatest for the fixed-fixed supports. However, the expansion-fixed and fixed-fixed supports create compressive flange stress in the vicinity of the fixed supports which must be accounted for in the design process.

15. Several quantitative trends of use to practicing bridge engineers were apparent from this study. They include the following:

a. Doubling the maximum intact deflection is generally a good rough estimate of the maximum damaged deflection response for either live load alone or total
load for loads up to a single 204-kip PennDOT Permit vehicle. This estimate should not be used for total loads in excess of this since this grossly underestimates the maximum deflection for the damage bridge using BOVAS-D.

b. The peak midspan stress in the first interior girder approximately doubles after the development of a severe fracture at the adjacent exterior girder. This was evident from both the linear and nonlinear models and provides a useful, yet slightly conservative, "rule of thumb." This result has practical significance since the first interior girder is the most critically stressed girder after damage occurs.

c. Changing the live load from the HS20-44 vehicle to the 204-kip PennDOT Permit vehicle increases the gross live load at first yield by about 30% for either the intact or damaged structure.

d. The gross live load at first girder yield on the damaged structure is about half of the gross live load at first girder yield on the intact structure. This could be deduced from both the Phase I and Phase II results.

e. There appears to be a minimum of 40% reserve capacity after the first yield live load for either the intact or damaged bridge models.

16. After initial girder yielding on either the intact or damaged bridges, only the girder(s) neighboring the yielded girder are notably effected (in terms of stress changes) by subsequent live load increases.

17. After initial girder yielding on either the intact or damaged bridges, further increases of the live load cause the spread of yielding only within the initially yielded girder for a large additional increment of live load. The yielding quickly spreads to the neighboring
areas of the flange and web.

18. Under dead load alone, the damaged structure has large deflections, high girder stresses, and substantial cracking of the slab. The formation of a "yield line" begins under dead load alone also. As the live load is applied and increased, this yield line runs transverse to the bridge around the G6 midspan, and then parallel and near to G5.

19. Even though considerable cracking/crushing has occurred, the overall linearity of the load versus deflection response of the multigirder bridge is a function of steel yielding.

This research has fully investigated the structural response of fractured steel multigirder bridges in the region from design load levels to the upper limit of serviceability. The results will allow bridge engineers to have a qualitative and quantitative understanding of actual performance of commonly used multigirder bridges after fractures have developed. In addition, an analysis tool has been developed which permits the bridge engineer to predict both the elastic and inelastic response of a damaged bridge superstructure in terms of load versus deformation, material failure, and instability.


The observations and conclusions presented in this report are those that were evident from the models studied as part of this research. It would be expected that further analytical results would confirm these conclusions. However, because the results come from a limited number of analytic models, the following recommendations are made for future research:
1. Investigate the contribution of cross bracing and lateral bracing in the redistribution of load in fractured multigirder bridges. For the initial research in this area, a linear elastic model should be sufficient for capturing the approximate behavior under normal loads and overloads. This linear model can be used to study the damaged bridges response with and without bracing, and for bracing of various stiffness.

2. Develop a more sophisticated nonlinear damage model which includes cross bracing and lateral bracing members. The model should incorporate both realistic post-yielding and buckling behavior of these members.

3. Examine the possible beneficial effects of designing multigirder bridges using fixed supports at all bearing ends. In addition to expanding upon the limited study provided in Chapter 3 of this report, the design of the supports to resist longitudinal and transverse reaction forces should be addressed.

4. Expand the present study of damaged multigirder bridges by using two HS20-44 vehicles rather than one for both the linear and nonlinear models. These vehicles should be placed side by side to cause the greatest effects in the vicinity of the damage. The resulting data would be of considerable use to practicing bridge engineers.

5. Investigate and verify more fully the "rules of thumb" suggested by the reported research. In particular, the following patterns of behavior should be examined:

   a. The relationship between the maximum intact deflection and the maximum damaged deflection.

   b. The ratio of peak stress on the intact and damaged structures. It is suggested that the study focus on the response of the girder adjacent to the damaged
girder rather than the damaged girder itself.

c. The relationship between the peak girder stress for the HS20-44 vehicle and the 204-kip PennDOT Permit vehicle on the intact and damaged structures.

d. The relationship between the ultimate ("collapse") load and the load at the first girder yield for the intact and damaged structures.

6. Implement the use of the BOVAS-D model for less severe damage states. While the BOVAS-D model already employs bilinear springs at each damage location, research must be completed on relating the springs' material properties to an accurate CTOD model.

7. Investigate the use of the "averaging technique" for predicting the behavior of less severe damage states. This study should accurately model several different depths of web cracks and compare the results with predictions based on knowing only the intact and fully damaged structural response.

8. Examine the effect of multiple damage locations on the response of multigirder bridges. This investigation should also include damage locations that are not prepositioned at the bridge's midspan.

9. Quantify the response of intact and damaged multigirder bridges as they approach "collapse" rather than terminating at the upper level of serviceability.

If all of this research is conducted, a more complete understanding of the response characteristics of damaged steel multigirder bridge superstructures will be established. Thus the bridge engineer should then have an even better capacity for making an accurate assessment of the resistance of common highway bridges if they become damaged.
TABLES
### Table 1. SUMMARY OF MULTIGIRDER BRIDGE MODELING USING SAPIV.

#### MODEL STATISTICS:

| Number of Nodes: | 974 |
| Number of Degrees of Freedom: | 4309 (intact) 4320 (damaged) |
| Number of Elements: | 1387 (100 ft. span) 1429 (140 ft. span) 1471 (180 ft. span) |

#### ELEMENT SUMMARY:

<table>
<thead>
<tr>
<th>Component</th>
<th>Element Type</th>
<th>100'</th>
<th>140'</th>
<th>180'</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinf Concrete Slab</td>
<td>Plate Bending</td>
<td>360</td>
<td>360</td>
<td>360</td>
</tr>
<tr>
<td>Lower Girder Flange</td>
<td>Beam</td>
<td>180</td>
<td>180</td>
<td>180</td>
</tr>
<tr>
<td>Plate Girder Web</td>
<td>Plate Bending</td>
<td>360</td>
<td>360</td>
<td>360</td>
</tr>
<tr>
<td>Upper Girder Flange</td>
<td>Truss</td>
<td>180</td>
<td>180</td>
<td>180</td>
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<td>Curb</td>
<td>Beam</td>
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<td>60</td>
<td>60</td>
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<td>Stiffeners</td>
<td>Truss</td>
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<td>Diaphragms</td>
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<td>10</td>
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<td>Cross Bracing</td>
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<td>135</td>
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<td>Supports</td>
<td>Boundary</td>
<td>36</td>
<td>36</td>
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</tbody>
</table>

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Table 2. MODEL TESTING OF INTACT 100 FOOT MULTIGIRDER BRIDGE WITH UNIFORM LOAD.

<table>
<thead>
<tr>
<th>Item</th>
<th>Longitudinal Location (x = )</th>
<th>Composite Beam Theory</th>
<th>SAPIV Model</th>
<th>% Difference</th>
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<td>1%</td>
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<td></td>
<td>400.0&quot;</td>
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<td></td>
<td>500.0&quot;</td>
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<td>0.718&quot;</td>
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<tr>
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<td>Lower Flange Stress</td>
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<td>6.64 ksi</td>
<td>6.16 ksi</td>
<td>7%</td>
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<td></td>
<td>512.5&quot;</td>
<td>7.12 ksi</td>
<td>6.76 ksi</td>
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<td></td>
<td>562.5&quot;</td>
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<td>5%</td>
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<tr>
<td>Upper Slab Stress, Longitud.</td>
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<tr>
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<td></td>
<td>562.5&quot;</td>
<td>0.360 ksi</td>
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Table 3. CROSS BRACING STRESS (KSI) ACROSS MIDSPAN FOR 100 FOOT MULTIGIRDER BRIDGES WITH VARIOUS DAMAGE CONDITIONS.

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<th>TYPE</th>
<th>LC#</th>
<th>IN</th>
<th>FL</th>
<th>WB</th>
<th>WW</th>
</tr>
</thead>
<tbody>
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<td>G1-G2</td>
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<td>-0.80</td>
<td>-0.84</td>
<td>-1.25</td>
<td>-1.74</td>
</tr>
<tr>
<td>G2-G3</td>
<td>-</td>
<td>1</td>
<td>-2.37</td>
<td>-2.50</td>
<td>-3.89</td>
<td>-5.51</td>
</tr>
<tr>
<td>G3-G4</td>
<td>-</td>
<td>1</td>
<td>-3.43</td>
<td>-3.66</td>
<td>-6.27</td>
<td>-9.33</td>
</tr>
<tr>
<td>G4-G5</td>
<td>-</td>
<td>1</td>
<td>-2.60</td>
<td>-2.92</td>
<td>-6.52</td>
<td>-10.70</td>
</tr>
<tr>
<td>G5-G6</td>
<td>-</td>
<td>1</td>
<td>-0.67</td>
<td>-0.87</td>
<td>-3.19</td>
<td>-5.36</td>
</tr>
</tbody>
</table>

| G1-G2 | /    | 1   | 1.06 | 1.12 | 1.67 | 2.33 |
| G2-G3 | /    | 1   | 0.58 | 0.62 | 1.13 | 1.73 |
| G3-G4 | /    | 1   | -0.55| -0.52| -0.20| 0.18 |
| G5-G6 | /    | 1   | -1.68| -2.00| -5.38| -9.99|

| G1-G2 | \   | 1   | -1.54| -1.61| -2.41| -3.34|
| G2-G3 | \   | 1   | -1.96| -2.08| -3.39| -4.94|
| G3-G4 | \   | 1   | -1.43| -1.60| -3.45| -5.60|
| G4-G5 | \   | 1   | 0.95 | 0.81 | -0.76| -2.53|
| G5-G6 | \   | 1   | 0.95 | 1.16 | 4.32 | 7.27 |

| G1-G2 | -   | 2   | -1.51| -1.58| -2.41| -3.37|
| G2-G3 | -   | 2   | -4.50| -4.75| -7.49| -10.69|
| G3-G4 | -   | 2   | -6.57| -7.04| -12.21| -18.21|
| G4-G5 | -   | 2   | -4.98| -5.63| -12.73| -20.95|
| G5-G6 | -   | 2   | -1.31| -1.71| -6.25| -10.44|

| G1-G2 | /   | 2   | 2.01 | 2.11 | 3.21 | 4.50 |
| G2-G3 | /   | 2   | 1.12 | 1.21 | 2.21 | 3.38 |
| G3-G4 | /   | 2   | -0.98| -0.92| -0.28| 0.47 |
| G4-G5 | /   | 2   | -5.04| -5.14| -6.25| -7.58|
| G5-G6 | /   | 2   | -3.05| -3.69| -10.41| -19.51|

| G1-G2 | \  | 2   | -2.91| -3.05| -4.63| -6.46|
| G3-G4 | \  | 2   | -2.82| -3.16| -6.82| -11.05|
| G4-G5 | \  | 2   | 2.00 | 1.71 | -1.38| -4.85|
| G5-G6 | \  | 2   | 1.85 | 2.28 | 8.48 | 14.17|

| G1-G2 | -  | 3   | 0.83 | -1.91| -2.82| -3.88|
| G2-G3 | -  | 3   | -5.40| -5.67| -8.68| -12.21|
| G3-G4 | -  | 3   | -7.66| -8.18| -13.85| -20.48|
| G4-G5 | -  | 3   | -5.86| -6.57| -14.37| -23.45|
| G5-G6 | -  | 3   | -1.67| -2.10| -7.11| -11.80|
TABLE 3 (CONTINUED)

CROSS BRACING STRESS (KSI) ACROSS MIDSPAN
FOR 100 FOOT MULTIGIRDER BRIDGES
WITH VARIOUS DAMAGE CONDITIONS.

<table>
<thead>
<tr>
<th>Z=</th>
<th>TYPE</th>
<th>LC#</th>
<th>IN</th>
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<th>WB</th>
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<tr>
<td>G1-G2</td>
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TABLE 3 (CONTINUED)

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FOR 100 FOOT MULTIGIRDER BRIDGES
WITH VARIOUS DAMAGE CONDITIONS.

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| G3-G4 | / | 1 | -0.43 | 0.19 | -0.55 | 0.18 | -0.27 | 0.10 |
| G4-G5 | / | 1 | -2.30 | -3.07 | -2.58 | -3.86 | -2.02 | -2.35 |
| G5-G6 | / | 1 | -1.43 | -7.23 | -1.68 | -9.99 | -1.17 | -4.14 |

| G1-G2 | \ | 1 | -1.31 | -2.46 | -1.54 | -3.34 | -1.04 | -1.58 |
| G2-G3 | \ | 1 | -1.72 | -3.72 | -1.96 | -4.94 | -1.45 | -2.44 |
| G3-G4 | \ | 1 | -1.31 | -4.26 | -1.43 | -5.60 | -1.16 | -2.69 |
| G4-G5 | \ | 1 | 0.89 | -1.67 | 0.95 | -2.53 | 0.86 | -0.50 |
| G5-G6 | \ | 1 | 0.81 | 5.34 | 0.95 | 7.27 | 0.69 | 3.04 |

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| G3-G4 | - | 2 | -5.66 | -13.98 | -6.57 | -18.21 | -5.01 | -9.20 |
| G4-G5 | - | 2 | -4.30 | -15.50 | -4.98 | -20.95 | -3.82 | -9.90 |
| G5-G6 | - | 2 | -1.10 | -7.67 | -1.31 | -10.44 | -0.96 | -4.56 |

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| G2-G3 | / | 2 | 0.99 | 2.57 | 1.12 | 3.38 | 0.95 | 1.82 |
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</tr>
<tr>
<td>Young's Modulus:</td>
<td>29,000.0 ksi</td>
</tr>
<tr>
<td>Ramberg-M:</td>
<td>0.670</td>
</tr>
<tr>
<td>Ramberg-N:</td>
<td>400.0</td>
</tr>
<tr>
<td>Strain Hardening Modulus:</td>
<td>900.0 ksi</td>
</tr>
<tr>
<td>Strain Hardening Strain:</td>
<td>0.014 in/in</td>
</tr>
<tr>
<td>Ultimate Stress:</td>
<td>58.0 ksi</td>
</tr>
<tr>
<td>Ultimate Strain:</td>
<td>0.120 in/in</td>
</tr>
<tr>
<td>Shear Modulus:</td>
<td>11,154.0 ksi</td>
</tr>
</tbody>
</table>

Table 6. SLAB GEOMETRY/MATERIAL PROPERTIES USED IN BOVAS-D MODEL OF 100 FOOT MULTIGIRDER BRIDGE.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial Compressive Strength:</td>
<td>3,000.0 psi</td>
</tr>
<tr>
<td>Direct Tensile Strength:</td>
<td>270.0 psi</td>
</tr>
<tr>
<td>Initial Modulus of Elasticity:</td>
<td>3,156.0 ksi</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete Layer</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.25 inches</td>
</tr>
<tr>
<td>2</td>
<td>1.25 inches</td>
</tr>
<tr>
<td>3</td>
<td>2.00 inches</td>
</tr>
<tr>
<td>4</td>
<td>2.00 inches</td>
</tr>
<tr>
<td>5</td>
<td>0.50 inches</td>
</tr>
<tr>
<td>6</td>
<td>0.50 inches</td>
</tr>
</tbody>
</table>

Table 7. REINFORCING STEEL GEOMETRY/MATERIAL PROPERTIES USED IN BOVAS-D MODEL OF 100 FOOT MULTIGIRDER BRIDGE.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus:</td>
<td>29,000.0 ksi</td>
</tr>
<tr>
<td>Yield Strength:</td>
<td>60.0 ksi</td>
</tr>
<tr>
<td>Ramberg-M:</td>
<td>0.70</td>
</tr>
<tr>
<td>Ramberg-N:</td>
<td>300.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Layer</th>
<th>Bar #</th>
<th>Spacing (inches)</th>
<th>Thick. (inches)</th>
<th>Distance (Note)</th>
<th>Orientation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>5.0000</td>
<td>0.06200</td>
<td>2.8125</td>
<td>Transverse</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>13.4286</td>
<td>0.01486</td>
<td>3.3750</td>
<td>Longitudinal</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>7.4545</td>
<td>0.04159</td>
<td>5.5625</td>
<td>Longitudinal</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>5.0000</td>
<td>0.06200</td>
<td>6.1875</td>
<td>Transverse</td>
</tr>
</tbody>
</table>

Note: Distance from top surface of slab in inches.
Table 8. COMPARISON OF SAPIV AND BOVAS-D RESULTS IN THE VICINITY OF G3: INTACT 100 FOOT MULTIGIRDER BRIDGE WITH UNIFORM LOAD OF 1 KIP PER FOOT OVER EACH GIRDER.

<table>
<thead>
<tr>
<th></th>
<th>SAPIV</th>
<th>BOVAS-D</th>
<th>%DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection @ x=350&quot;</td>
<td>0.601&quot;</td>
<td>0.647&quot;</td>
<td>+8%</td>
</tr>
<tr>
<td>Deflection @ x=600&quot;</td>
<td>0.741&quot;</td>
<td>0.799&quot;</td>
<td>+8%</td>
</tr>
<tr>
<td>Lower Flange Stress Near Midspan (ksi)</td>
<td>6.92</td>
<td>7.22</td>
<td>+4%</td>
</tr>
<tr>
<td>Longitudinal Slab Stress at Lower Surface Near Midspan (ksi)</td>
<td>0.200</td>
<td>0.224</td>
<td>+12%</td>
</tr>
</tbody>
</table>

Table 9. COMPARISON OF SAPIV AND BOVAS-D RESULTS IN THE VICINITY OF G3: DAMAGED 100 FOOT MULTIGIRDER BRIDGE WITH UNIFORM LOAD OF 1 KIP PER FOOT OVER EACH GIRDER.

<table>
<thead>
<tr>
<th></th>
<th>SAPIV</th>
<th>BOVAS-D</th>
<th>%DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection @ x=350&quot;</td>
<td>0.636&quot;</td>
<td>0.619&quot;</td>
<td>-3%</td>
</tr>
<tr>
<td>Deflection @ x=600&quot;</td>
<td>0.781&quot;</td>
<td>0.767&quot;</td>
<td>-2%</td>
</tr>
<tr>
<td>Lower Flange Stress Near Midspan (ksi)</td>
<td>7.34</td>
<td>6.78</td>
<td>-8%</td>
</tr>
<tr>
<td>Longitudinal Slab Stress at Lower Surface Near Midspan (ksi)</td>
<td>0.227</td>
<td>0.262</td>
<td>+15%</td>
</tr>
</tbody>
</table>
Table 10. COMPARISON OF SAPIV AND BOVAS-D RESULTS IN THE VICINITY OF G5: DAMAGED 100 FOOT MULTIGIRDER BRIDGE WITH UNIFORM LOAD OF 1 KIP PER FOOT OVER EACH GIRDER.

<table>
<thead>
<tr>
<th></th>
<th>SAPIV</th>
<th>BOVAS-D</th>
<th>%DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection @ x=350&quot;</td>
<td>0.812&quot;</td>
<td>1.003&quot;</td>
<td>+24%</td>
</tr>
<tr>
<td>Deflection @ x=600&quot;</td>
<td>1.039&quot;</td>
<td>1.249&quot;</td>
<td>+20%</td>
</tr>
<tr>
<td>Lower Flange Stress Near Midspan</td>
<td>12.72 ksi</td>
<td>12.71 ksi</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>Longitudinal Slab Stress at Lower Surface Near Midspan</td>
<td>0.249 ksi</td>
<td>0.409 ksi</td>
<td>+64%</td>
</tr>
</tbody>
</table>

Table 11. COMPARISON OF SAPIV AND BOVAS-D RESULTS IN THE VICINITY OF G6: DAMAGED 100 FOOT MULTIGIRDER BRIDGE WITH UNIFORM LOAD OF 1 KIP PER FOOT OVER EACH GIRDER.

<table>
<thead>
<tr>
<th></th>
<th>SAPIV</th>
<th>BOVAS-D</th>
<th>%DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection @ x=350&quot;</td>
<td>0.901&quot;</td>
<td>1.896&quot;</td>
<td>+110%</td>
</tr>
<tr>
<td>Deflection @ x=600&quot;</td>
<td>1.333&quot;</td>
<td>3.029&quot;</td>
<td>+127%</td>
</tr>
<tr>
<td>Lower Flange Stress Near Midspan</td>
<td>3.483 ksi</td>
<td>3.433 ksi</td>
<td>-1%</td>
</tr>
<tr>
<td>Longitudinal Slab Stress at Lower Surface Near Midspan</td>
<td>0.993 ksi</td>
<td>0.000 ksi</td>
<td>-100%</td>
</tr>
</tbody>
</table>
Table 12. COMPARISON OF SAPIV AND BOVAS-D RESULTS IN THE VICINITY OF G3: INTACT 100 FOOT MULTIGIRDER BRIDGE WITH HS20-44 VEHICLE LOAD AND DEAD LOAD.

<table>
<thead>
<tr>
<th></th>
<th>SAPIV</th>
<th>BOVAS-D</th>
<th>%DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>@ x=350&quot;</td>
<td>0.772&quot;</td>
<td>0.805&quot;</td>
<td>+4%</td>
</tr>
<tr>
<td>Deflection</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>@ x=600&quot;</td>
<td>0.953&quot;</td>
<td>0.995&quot;</td>
<td>+4%</td>
</tr>
<tr>
<td>Lower Flange Stress</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Near Midspan</td>
<td>8.97 ksi</td>
<td>8.54 ksi</td>
<td>-5%</td>
</tr>
</tbody>
</table>

Table 13. COMPARISON OF SAPIV AND BOVAS-D RESULTS IN THE VICINITY OF G6: INTACT 100 FOOT MULTIGIRDER BRIDGE WITH HS20-44 VEHICLE LOAD AND DEAD LOAD.

<table>
<thead>
<tr>
<th></th>
<th>SAPIV</th>
<th>BOVAS-D</th>
<th>%DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>@ x=350&quot;</td>
<td>1.065&quot;</td>
<td>1.235&quot;</td>
<td>+16%</td>
</tr>
<tr>
<td>Deflection</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>@ x=600&quot;</td>
<td>1.315&quot;</td>
<td>1.519&quot;</td>
<td>+16%</td>
</tr>
<tr>
<td>Lower Flange Stress</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Near Midspan</td>
<td>13.14 ksi</td>
<td>14.71 ksi</td>
<td>+13%</td>
</tr>
</tbody>
</table>
Table 14. SUMMARY OF THE LOAD CYCLES FOR THE INTACT BRIDGE WITH HS20-44 VEHICLE AND 204-KIP PENNDOT PERMIT VEHICLE.

<table>
<thead>
<tr>
<th>CYCLE</th>
<th><strong>HS20-44</strong></th>
<th><strong>PERMIT</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LL</td>
<td>LL/72.0</td>
</tr>
<tr>
<td>1</td>
<td>0.0</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>38.2</td>
<td>0.53</td>
</tr>
<tr>
<td>3</td>
<td>76.3</td>
<td>1.06</td>
</tr>
<tr>
<td>4</td>
<td>114.5</td>
<td>1.59</td>
</tr>
<tr>
<td>5</td>
<td>152.7</td>
<td>2.12</td>
</tr>
<tr>
<td>6</td>
<td>190.9</td>
<td>2.65</td>
</tr>
<tr>
<td>7</td>
<td>229.0</td>
<td>3.18</td>
</tr>
<tr>
<td>8</td>
<td>267.2</td>
<td>3.71</td>
</tr>
<tr>
<td>9</td>
<td>305.4</td>
<td>4.24</td>
</tr>
<tr>
<td>10</td>
<td>343.6</td>
<td>4.77</td>
</tr>
<tr>
<td>11</td>
<td>353.3</td>
<td>4.91</td>
</tr>
<tr>
<td>12</td>
<td>358.3</td>
<td>4.98</td>
</tr>
<tr>
<td>13</td>
<td>363.0</td>
<td>5.04</td>
</tr>
<tr>
<td>14</td>
<td>369.0</td>
<td>5.12</td>
</tr>
<tr>
<td>15</td>
<td>375.3</td>
<td>5.21</td>
</tr>
<tr>
<td>16</td>
<td>381.5</td>
<td>5.30</td>
</tr>
<tr>
<td>17</td>
<td>390.4</td>
<td>5.42</td>
</tr>
<tr>
<td>18</td>
<td>396.3</td>
<td>5.50</td>
</tr>
<tr>
<td>19</td>
<td>402.3</td>
<td>5.59</td>
</tr>
<tr>
<td>20</td>
<td>403.0</td>
<td>5.60</td>
</tr>
<tr>
<td>21</td>
<td>412.7</td>
<td>5.73</td>
</tr>
<tr>
<td>22</td>
<td>417.6</td>
<td>5.80</td>
</tr>
<tr>
<td>23</td>
<td>418.7</td>
<td>5.81</td>
</tr>
<tr>
<td>24</td>
<td>430.8</td>
<td>5.98</td>
</tr>
<tr>
<td>25</td>
<td>433.6</td>
<td>6.02</td>
</tr>
<tr>
<td>26</td>
<td>435.6</td>
<td>6.05</td>
</tr>
<tr>
<td>27</td>
<td>441.3</td>
<td>6.13</td>
</tr>
<tr>
<td>28</td>
<td>446.1</td>
<td>6.20</td>
</tr>
</tbody>
</table>
Table 15. SUMMARY OF THE CYCLES SELECTED FOR CLOSER REVIEW OF THE INTACT 100 FOOT MULTIGIRDER.

<table>
<thead>
<tr>
<th>GROUP</th>
<th>CYCLE</th>
<th>LL (kips)</th>
<th>LL/72.0</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.0</td>
<td>0.00</td>
<td>Dead load only.</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>76.3</td>
<td>1.06</td>
<td>Before girder yield.</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>229.0</td>
<td>3.18</td>
<td>Before girder yield.</td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>353.3</td>
<td>4.91</td>
<td>First yield of girder steel.</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td>403.0</td>
<td>5.60</td>
<td>After girder yield.</td>
</tr>
<tr>
<td>6</td>
<td>28</td>
<td>446.1</td>
<td>6.20</td>
<td>Final load cycle.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>GROUP</th>
<th>CYCLE</th>
<th>LL (kips)</th>
<th>LL/72.0</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.0</td>
<td>0.00</td>
<td>Dead load only.</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>75.2</td>
<td>1.04</td>
<td>Before girder yield.</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>225.6</td>
<td>3.13</td>
<td>Before girder yield.</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>457.8</td>
<td>6.36</td>
<td>First yield of girder steel.</td>
</tr>
<tr>
<td>5</td>
<td>21</td>
<td>566.5</td>
<td>7.87</td>
<td>After girder yield.</td>
</tr>
<tr>
<td>6</td>
<td>28</td>
<td>619.9</td>
<td>8.61</td>
<td>Final load cycle.</td>
</tr>
</tbody>
</table>
### Table 16. Deflection Comparison of All Girders at the Initial Yield of G6 on the Intact Multigirder Bridge for Both the HS20-44 Vehicle and 204-Kip PennDOT Permit Vehicle Loads.

<table>
<thead>
<tr>
<th>Girder #</th>
<th>Deflection (HS20-44 @ 4.9H)</th>
<th>Deflection (PennDOT @ 6.4H)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.04&quot;</td>
<td>1.05&quot;</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>2</td>
<td>1.03&quot;</td>
<td>1.04&quot;</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>3</td>
<td>1.13&quot;</td>
<td>1.14&quot;</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>4</td>
<td>1.48&quot;</td>
<td>1.52&quot;</td>
<td>3%</td>
</tr>
<tr>
<td>5</td>
<td>2.36&quot;</td>
<td>2.44&quot;</td>
<td>3%</td>
</tr>
<tr>
<td>6</td>
<td>3.39&quot;</td>
<td>3.53&quot;</td>
<td>4%</td>
</tr>
</tbody>
</table>

### Table 17. Lower Flange Stress Comparison of Unyielded Girders at the Initial Yield of G6 on the Intact Multigirder Bridge for Both the HS20-44 Vehicle and 204-Kip PennDOT Permit Vehicle Loads.

<table>
<thead>
<tr>
<th>Girder #</th>
<th>Stress (HS20-44 @ 4.9H)</th>
<th>Stress (PennDOT @ 6.4H)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.04 ksi</td>
<td>9.04 ksi</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>2</td>
<td>8.35 ksi</td>
<td>8.32 ksi</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>3</td>
<td>8.86 ksi</td>
<td>8.88 ksi</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>4</td>
<td>12.17 ksi</td>
<td>12.50 ksi</td>
<td>3%</td>
</tr>
<tr>
<td>5</td>
<td>23.21 ksi</td>
<td>23.20 ksi</td>
<td>&lt;1%</td>
</tr>
</tbody>
</table>
Table 18. SUMMARY OF THE LOAD CYCLES FOR THE DAMAGED BRIDGE WITH HS20 -44 VEHICLE AND 204-KIP PENNDOT PERMIT VEHICLE.

<table>
<thead>
<tr>
<th>CYCLE</th>
<th><em><strong><strong>HS20-44</strong></strong></em></th>
<th><em><strong><strong>PERMIT</strong></strong></em></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LL/72.0 (kips)</td>
<td>LL/72.0 (kips)</td>
</tr>
<tr>
<td>1</td>
<td>0.0 0.00</td>
<td>0.0 0.00</td>
</tr>
<tr>
<td>2</td>
<td>25.2 0.35</td>
<td>34.9 0.48</td>
</tr>
<tr>
<td>3</td>
<td>50.5 0.70</td>
<td>69.8 0.97</td>
</tr>
<tr>
<td>4</td>
<td>75.7 1.05</td>
<td>104.6 1.45</td>
</tr>
<tr>
<td>5</td>
<td>101.0 1.40</td>
<td>139.5 1.94</td>
</tr>
<tr>
<td>6</td>
<td>126.2 1.75</td>
<td>174.4 2.42</td>
</tr>
<tr>
<td>7</td>
<td>151.5 2.10</td>
<td>209.3 2.91</td>
</tr>
<tr>
<td>8</td>
<td>176.7 2.45</td>
<td>244.2 3.39</td>
</tr>
<tr>
<td>9</td>
<td>189.4 2.63</td>
<td>250.6 3.48</td>
</tr>
<tr>
<td>10</td>
<td>192.8 2.68</td>
<td>256.8 3.57</td>
</tr>
<tr>
<td>11</td>
<td>197.5 2.74</td>
<td>261.7 3.63</td>
</tr>
<tr>
<td>12</td>
<td>204.1 2.83</td>
<td>262.9 3.65</td>
</tr>
<tr>
<td>13</td>
<td>210.1 2.92</td>
<td>264.2 3.67</td>
</tr>
<tr>
<td>14</td>
<td>213.6 2.97</td>
<td>278.1 3.86</td>
</tr>
<tr>
<td>15</td>
<td>219.1 3.04</td>
<td>285.3 3.96</td>
</tr>
<tr>
<td>16</td>
<td>223.7 3.11</td>
<td>290.9 4.04</td>
</tr>
<tr>
<td>17</td>
<td>230.8 3.21</td>
<td>298.0 4.14</td>
</tr>
<tr>
<td>18</td>
<td>236.7 3.29</td>
<td>313.7 4.36</td>
</tr>
<tr>
<td>19</td>
<td>241.7 3.36</td>
<td>320.9 4.46</td>
</tr>
<tr>
<td>20</td>
<td>242.4 3.37</td>
<td>323.8 4.50</td>
</tr>
<tr>
<td>21</td>
<td>250.6 3.48</td>
<td>326.4 4.53</td>
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<td>22</td>
<td>256.4 3.56</td>
<td>333.0 4.63</td>
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<td>262.2 3.64</td>
<td>337.2 4.68</td>
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<td>267.9 3.72</td>
<td>341.4 4.74</td>
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<tr>
<td>25</td>
<td>293.2 4.07</td>
<td>350.9 4.87</td>
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<tr>
<td>26</td>
<td>297.1 4.13</td>
<td>360.2 5.00</td>
</tr>
<tr>
<td>27</td>
<td>308.1 4.28</td>
<td>364.3 5.06</td>
</tr>
<tr>
<td>28</td>
<td>309.9 4.30</td>
<td></td>
</tr>
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Table 19. SUMMARY OF THE CYCLES SELECTED FOR CLOSER REVIEW OF THE DAMAGED 100 FOOT MULTIGIRDER.

*****HS20-44 Vehicle Loading*****

<table>
<thead>
<tr>
<th>GROUP</th>
<th>CYCLE</th>
<th>LL (kips)</th>
<th>LL/72.0</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.0</td>
<td>0.00</td>
<td>Dead load only.</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>75.7</td>
<td>1.05</td>
<td>Before girder yield.</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>126.2</td>
<td>1.75</td>
<td>First reinforcing steel yield.</td>
</tr>
<tr>
<td>4</td>
<td>9</td>
<td>189.4</td>
<td>2.63</td>
<td>First yield of girder steel.</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td>242.4</td>
<td>3.37</td>
<td>After girder yield.</td>
</tr>
<tr>
<td>6</td>
<td>28</td>
<td>309.9</td>
<td>4.30</td>
<td>Final load cycle.</td>
</tr>
</tbody>
</table>

*****Permit Vehicle Loading*****

<table>
<thead>
<tr>
<th>GROUP</th>
<th>CYCLE</th>
<th>LL (kips)</th>
<th>LL/72.0</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.0</td>
<td>0.00</td>
<td>Dead load only.</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>104.6</td>
<td>1.45</td>
<td>Before girder yield.</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>174.4</td>
<td>2.42</td>
<td>First reinforcing steel yield.</td>
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<tr>
<td>4</td>
<td>8</td>
<td>244.2</td>
<td>3.39</td>
<td>First yield of girder steel.</td>
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<tr>
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<td>18</td>
<td>313.7</td>
<td>4.36</td>
<td>After girder yield.</td>
</tr>
<tr>
<td>6</td>
<td>27</td>
<td>364.3</td>
<td>5.06</td>
<td>Final load cycle.</td>
</tr>
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</table>
Table 20. DEFLECTION COMPARISON OF ALL GIR德RS AT THE INITIAL YIELD OF G5 ON THE DAMAGED MULTIGIRDER BRIDGE FOR BOTH THE HS20-44 VEHICLE AND 204-KIP PENNDOT PERMIT VEHICLE LOADS.

<table>
<thead>
<tr>
<th>Girder #</th>
<th>Deflection (HS20-44 @ 2.6H)</th>
<th>Deflection (PennDOT @ 3.4H)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.06&quot;</td>
<td>1.07&quot;</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>2</td>
<td>0.92&quot;</td>
<td>0.93&quot;</td>
<td>1%</td>
</tr>
<tr>
<td>3</td>
<td>0.81&quot;</td>
<td>0.82&quot;</td>
<td>1%</td>
</tr>
<tr>
<td>4</td>
<td>0.90&quot;</td>
<td>0.93&quot;</td>
<td>3%</td>
</tr>
<tr>
<td>5</td>
<td>3.25&quot;</td>
<td>3.26&quot;</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>6</td>
<td>11.32&quot;</td>
<td>11.32&quot;</td>
<td>1%</td>
</tr>
</tbody>
</table>

Table 21. LOWER FLANGE STRESS COMPARISON OF UNYIELDRED GIR德RS AT THE INITIAL YIELD OF G5 ON THE DAMAGED MULTIGIRDER BRIDGE FOR BOTH THE HS20-44 VEHICLE AND 204-KIP PENNDOT PERMIT VEHICLE LOADS.

<table>
<thead>
<tr>
<th>Girder #</th>
<th>Stress (HS20-44 @ 2.6H)</th>
<th>Stress (PennDOT @ 3.4H)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.72 ksi</td>
<td>9.72 ksi</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>2</td>
<td>7.73 ksi</td>
<td>7.73 ksi</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>3</td>
<td>6.36 ksi</td>
<td>6.36 ksi</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>4</td>
<td>6.03 ksi</td>
<td>6.23 ksi</td>
<td>3%</td>
</tr>
<tr>
<td>6</td>
<td>9.38 ksi</td>
<td>8.91 ksi</td>
<td>5%</td>
</tr>
</tbody>
</table>
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Plan View, Multigirder Bridge, 100'

128K Dolly Load

Transverse Location (Inches)

Longitudinal Location (Inches/1000)

- Wheel Footprint
- Girder Line
PLAN VIEW, MULTIGIRDER BRIDGE, 100'

204K PERMIT VEHICLE

LONGITUDINAL LOCATION (INCHES/100)

WHEEL FOOTPRINT

GIRDER LINE

TRANVERSE LOCATION (INCHES)
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Figure 55. Lower Flange Stresses, Girder 5, 100 Foot Intact and Damaged Bridge, Various Vehicles Without Dead Load

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Figure 72. Transverse Slab Stresses At X = 0.27L, 100 Foot Intact and Damaged Bridges, Various Vehicles Without Dead Load
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Figure 74. Transverse Slab Stresses At X = 0.10L, 100 Foot Intact and Damaged Bridges, Various Vehicles With Dead Load
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- 552"
- 1200"
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Figure 208. Lower Flange Stresses, Girder 5, 100 Foot Intact Bridge, HS20-44 Vehicle Load (Six Cycles)

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LIVE LOAD VS STRESS, GIRDER #5, X = L/2

100' BRIDGE, INTACT, LOAD = HS20-44

TOTAL STRESS (KSI)

LIVE LOAD (72 KIPS = 1.00)

TOTAL STRESS (KSI)
Figure 211. Lower Flange Stresses, Girder 6, 100 Foot Intact Bridge, HS20-44 Vehicle Load (Six Cycles)

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Figure 214. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 1.1H
Figure 215. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 3.2H

Figure 216. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 4.9H
Figure 217. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 5.6H

Figure 218. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 6.2H
Figure 219. Lower Flange Stresses, Girder 1, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load (Six Cycles)

Figure 220. Lower Flange Stresses, Girder 2, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load (Six Cycles)
Figure 221. Lower Flange Stresses, Girder 3, 100 Foot In-tact Bridge, PennDOT Permit Vehicle Load (Six Cycles)

Figure 222. Lower Flange Stresses, Girder 4, 100 Foot In-tact Bridge, PennDOT Permit Vehicle Load (Six Cycles)
Figure 223. Lower Flange Stresses, Girder 5, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load (Six Cycles)

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Figure 225. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 0.0H

Figure 226. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 1.1H
Figure 227. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 3.1H

Figure 228. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 6.4H
Figure 229. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 7.9H

Figure 230. Lower Flange Stresses, All Girders, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 8.6H
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Figure 232. Yielding of Steel Girder Layers, Girder 6, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 5.6H
Figure 233. Yielding of Steel Girder Layers, Girder 6, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 6.2H
Figure 234. Yielding of Steel Girder Layers, Girder 6, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 6.4H

Figure 235. Yielding of Steel Girder Layers, Girder 6, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 7.7H
Figure 236. Yielding of Steel Girder Layers, Girder 6, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 7.9H

Figure 237. Yielding of Steel Girder Layers, Girder 6, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 8.6H
Figure 238. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 0.0H

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Figure 241. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 4.9H

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Figure 242. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 5.6H

Figure 243. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, HS20-44 Vehicle Load, Live Load = 6.2H
Figure 244. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 0.0H

Figure 245. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 1.1H
Figure 246. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 3.1H

Figure 247. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 6.4H
Figure 248. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 7.9H

Figure 249. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Intact Bridge, PennDOT Permit Vehicle Load, Live Load = 8.6H
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Figure 272. Longitudinal Deflection Profiles, Girder 5, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load (Six Cycles)

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Figure 280. Lower Flange Stresses, Girder 2, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (Six Cycles)

Figure 281. Lower Flange Stresses, Girder 2, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (27 Cycles)
Figure 282. Lower Flange Stresses, Girder 3, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (Six Cycles)

Figure 283. Lower Flange Stresses, Girder 3, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (27 Cycles)
Figure 284. Lower Flange Stresses, Girder 4, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (Six Cycles)

Figure 285. Lower Flange Stresses, Girder 4, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (27 Cycles)
Figure 286. Lower Flange Stresses, Girder 5, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (Six Cycles)

Figure 287. Lower Flange Stresses, Girder 5, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (27 Cycles)
Figure 288. Lower Flange Stresses, Girder 6, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (Six Cycles)

Figure 289. Lower Flange Stresses, Girder 6, 100 Foot Damaged Bridge, HS20-44 Vehicle Load (27 Cycles)
Figure 290. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 0.0H

Figure 291. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 1.1H
Figure 292. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 1.8H

Figure 293. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 2.6H
Figure 294. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 3.4H

Figure 295. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 4.3H
Figure 296. Lower Flange Stresses, Girder 1, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load (Six Cycles)

Figure 297. Lower Flange Stresses, Girder 2, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load (Six Cycles)
Figure 298. Lower Flange Stresses, Girder 3, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load (Six Cycles)

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Figure 303. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 1.5H
Figure 304. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 2.4H

Figure 305. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 3.4H
Figure 306. Lower Flange Stresses, All Girders, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 4.4H

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Figure 310. Yielding of Steel Girder Layers, Girder 6, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 3.4H

Figure 311. Yielding of Steel Girder Layers, Girder 6, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 4.3H
Figure 312. Yielding of Steel Girder Layers, Girder 6, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 3.4H

Figure 313. Yielding of Steel Girder Layers, Girder 6, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 4.4H
Figure 314. Yielding of Steel Girder Layers, Girder 6, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 4.7H

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Figure 321. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 3.4H
Figure 322. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Damaged Bridge, HS20-44 Vehicle Load, Live Load = 4.3H

Figure 323. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 1.5H
Figure 324. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 2.4H

Figure 325. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 3.4H
Figure 326. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 4.4H

Figure 327. Reinforced Concrete Slab Cracking and Crushing, 100 Foot Damaged Bridge, PennDOT Permit Vehicle Load, Live Load = 5.1H
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