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AFTER-FRACTURE REDUNDANCY
OF
TWO-GIRDER STEEL BRIDGES

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OF
TWO-GIRDER STEEL BRIDGES

by

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This Report is a reproduction of the MS Thesis entitled "After-Fracture Redundancy Rating of Two-Girder Steel Bridges to Determine the Requirements of the Alternate Load Path" by Mr. Hugh F. Hegarty. Mr. Hegarty's thesis was presented to the Graduate Committee of Lehigh University in May 1988 in partial fulfillment of the degree of master of Science in Civil Engineering.

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ABSTRACT

This report presents new concepts for determining redundancy in two-girder steel bridges. These concepts are needed in order to develop guidelines which can assist the bridge engineer in establishing inspection, repair, rehabilitation and replacement priorities.

A need exists to develop relatively simple after-fracture analytical models as well as an additional rating level, in addition to the AASHTO Operating and Inventory levels, which would evaluate bridge redundancy with respect to a particular fracture scenario. This paper suggests a Redundancy Rating level and concentrates mainly on the related analytical models and procedures.

The current technique of computing a Rating Factor for each member of a bridge is not considered practical for application to Redundancy Rating. In view of the much more complex analytical models required, the usual rating analysis methods need to be simplified for practical use. The approach used in this report is to determine the requirements of the alternate load path in terms of a Redundancy Rating Factor equal to unity for a given rating vehicle, number of lanes loaded, etc.

The alternate load path is evaluated in terms of both strength and serviceability. The strength requirement is
based on the current AASHTO Allowable Stress and Load Factor Methods. The Serviceability Method is new and is based on a limiting deflection-to-span-length ratio.

This research is limited to simple span noncomposite two-girder bridges with bottom lateral bracing, cross bracing, and top lateral bracing. The requirements of these members are developed for the practical range of existing two-girder bridges with this configuration.

It is concluded that serviceability is only a factor if a very restrictive limiting deflection-to-span-length ratio is used. The Load Factor Method results in a lower required area of bottom lateral diagonal in all cases. Therefore, the Load Factor Method controls if the resulting deflection is within the limiting deflection-to-span-length ratio. If the Load Factor Method results in more deflection than the bridge engineer is willing to tolerate, the Allowable Stress Method determines the requirements of the alternate load path members.
1. INTRODUCTION

1.1 Background

1.1.1 AASHTO Definition of Redundancy

The allowable stress ranges which are used in the design of steel bridges against fatigue resulting from repetitive live loads depend on whether the bridge is considered to be a redundant or nonredundant load path structure (1)*. Article 10.3.1 of the AASHTO Bridge Specifications (1) defines redundant load path structures as "structure types with multi-load paths where a single fracture in a member cannot lead to the collapse". Nonredundant load path structures are defined as structure types "where failure of a single element could cause collapse". The "element" referred to is defined as a "main load carrying component subject to tensile stress".

As a guide to bridge engineers, AASHTO, in Art. 10.3.1 of Ref. 1, gives examples of structures which are considered either redundant or nonredundant. For example, AASHTO classifies multi-girder bridges as redundant and two-girder bridges as nonredundant. However, two case studies of two-girder steel bridges which suffer major fracture of one girder show that collapse did not occur and the bridges remained relatively serviceable under normal highway traffic (2,3).

The AASHTO examples of redundant and nonredundant load path structures are based on unrealistic beliefs widely held by bridge

* References begin on page 169 of this report
engineers on the behavior of bridges under dead and live loads. These beliefs are based on the usual over-simplified assumptions used in the design and rating of steel girder bridges.

1.1.2 AASHTO Design and Rating Models

In the design and rating by AASHTO (1,4) of the girders of two-girder steel bridges, the two girders are considered in the simplified analytical model of the bridge to be the only load paths available for transmitting all vertical dead, live and impact loads from the deck, floorbeams and stringers to the substructure. Secondary members, such as lateral bracing, diaphragms and cross bracing, are not assumed to participate in transmitting vertical loads. Although these members are, in reality, subjected to stresses from the vertical loads, they are designed basically to resist lateral wind loads and to maintain rigidity of the cross section, particularly during construction.

This analytical model greatly simplifies both the design and rating of two-girder bridges and provides a lower bound, or conservative, solution for static loading. The lower bound theorem basically says that if a structure is shown how it can carry the applied static loads, it can safely carry at least this much load. Therefore, a conservative (often overly conservative) design or rating is achieved without the need to consider the three-dimensional interaction of all the bridge components.

However, it should be noted, the simplified analytical model can
result in unsafe results for dynamic loading. The stresses and displacements due to static loading must be multiplied by a dynamic load factor to obtain the true stresses and displacements due to dynamic loading (5).

1.1.3 AASHTO Rating Procedures

Bridges are rated at two levels (4):

1. Operating Rating Level: Absolute maximum permissible load level for the bridge.

2. Inventory Rating Level: The "normal" capacity of the bridge, representing the maximum load level which may safely traverse the structure for an indefinite period of time.

AASHTO bridge ratings are based on the standard H or HS loading, or one of the three typical truck loading configurations shown in Fig. 1 (4).

Bridges are rated at the two levels noted above using one or both of two methods (4):

1. Allowable Stress Method: The simplified model of the bridge structure is analyzed under service dead, live and impact load combinations (1) using linear elastic theory. The live load Rating Factor (RF) for a member is determined such that the maximum stress in the member does not exceed the specified allowable stress.

For example, for noncomposite bridge girders the RF's for both the Operating and Inventory levels are given by (4),
where \( f_{\text{all}} \) = Allowable Stress  
\( f_D \) = Dead Load Stress  
\( f_L \) = Live plus Impact Load Stress (caused by rating vehicle)

Different allowable stresses are used for the Operating and Inventory Rating levels.

2. Load Factor Method: The simplified model of the bridge structure is analyzed under factored dead, live and impact load combinations (1) using linear elastic theory. The live load Rating Factor (RF) for a member is determined such that the load effect (bending moment, for example) does not exceed the strength of the member (including a strength reduction factor).

For example, for noncomposite bridge girders the RF for the Operating Rating level is given by (4),

\[
RF = \frac{\phi S_u - \gamma_D D}{\gamma_L (L+I)}
\]

(1.2)

where, \( \phi \) = Strength Reduction Factor  
\( S_u \) = Member Strength (maximum moment capacity, for example)  
\( D \) = Dead load effect (bending moment, for example)  
\( L+I \) = Live load plus impact effect (bending moment for example)  
\( \gamma_D \) = Load factor for dead load = 1.3  
\( \gamma_L \) = Load factor for live plus impact loads = 1.3

The corresponding RF for the Inventory Rating Level is,
RF = \left( \frac{0.8 \cdot 1.3D}{1.3(L+I)} \right) \quad (1.3)

1.1.4 Need for Redundancy Rating

AASHTO Operating and Inventory Ratings are performed for bridges in which the simplified analytical model used in the design is still applicable for rating. That is, except for corrosion damage, limited fatigue cracking, missing rivets, bent flanges, etc., the connectivity of the structural members is essentially the same as that assumed in the design. For this reason, the assumptions on load distribution, etc., are virtually identical even though significant changes in traffic conditions may have occurred.

A vastly different situation arises as a result of fracture of a main load carrying member such as one of the girders of a simple span two-girder bridge. In this case the dead and live loads are redistributed in such a way that the three-dimensional behavior of the entire superstructure is involved (6). It is possible, in some cases, to find suitable alternate load paths which bypass the fractured girder, but this suggests a much different analytical model than that used in the traditional AASHTO rating analyses (7).

Also different is the expectation that after fracture occurs the bridge should continue to function indefinitely under normal traffic conditions. Although the fractured bridge should be expected to function under normal daily traffic conditions until the fracture is discovered, the time between fracture and detection is probably very
short (day, week, month) in relation to the usual life expectancy of a bridge (many years). Recent experience suggests that the fracture would be detected within a relatively short period of time either as a result of excessive deflections, other visible signs of distress, or during bridge maintenance and/or inspection (2,3).

There is clearly the need for an additional rating level which would address bridge redundancy with respect to a particular fracture scenario. The term Redundancy Rating (RR) level is suggested (8). The proposed RR would be performed along with the Operating and Inventory Ratings of an existing two-girder steel bridge. The RR can be based on either a worst case fracture scenario or on one or more plausible fracture scenarios as revealed by design conditions and/or inspections for fatigue cracking (8). The RR procedure developed here is based on a worst case fracture scenario.

1.2 Previous Research

Heins and Kato conducted an investigation of load redistribution in cracked girders (2). The study focused on two-girder bridges where one girder is assumed to be fractured near midspan. It is concluded that the influence of the bottom lateral bracing on load redistribution is significant. Further, the study concludes that utilization of the secondary members (cross bracing and bottom lateral bracing) effectively creates redundancy in two-girder bridges.
Daniels, Wilson and Chen conducted theoretical research into the behavior of two-girder steel bridges following a nearly full-depth midspan fracture of one of the girders (6). The study shows that as the fractured girder deflects under the loads, forces are transmitted into the bottom lateral bracing system. These forces are transmitted through the cross bracing to the deck which is subjected to in-plane bending. The deck is also subjected to torsion due to differential displacement between the two girders. The study shows that the after-fracture behavior of the superstructure is quite complex and involves the three-dimensional interaction between all the components making up the superstructure.

The analyses performed led to an understanding of load redistribution in two-girder bridges with a fractured girder and to the identification of the alternate load paths that develop. The study developed a linear elastic analytical procedure which can be used by bridge engineers to proportion the bottom lateral and cross bracing systems to ensure redundancy in the event of a near full-depth midspan fracture of one girder. The design example of a simple span two-girder bridge shows that the required redundancy can be provided with a relatively small increase in the sizes of the bottom lateral bracing members.

Daniels, Wilson and Kim discussed the importance of the results of research into redundancy for the rating of existing bridges (7). The approach recommended in that report is to identify the existing viable alternate load path(s) for the existing bridge. For each of
these load paths the live load rating would be calculated using the same philosophy contained in the present AASHTO Manual (4). It is observed that problems arise because the alternate load path(s) may not be complete or may have severe load level or fatigue restrictions, primarily because they were not originally designed for the purpose of providing redundancy.

Daniels, Hegarty, Kim and Wilson presented new concepts for determining redundancy in two-girder steel bridges (5). This report proposes a Redundancy Rating level and concentrates mainly on the related analytical models and procedures which are new to the bridge engineer. The approach suggested is to determine the requirements of the alternate load path in terms of a Redundancy Rating factor equal to unity for a given rating vehicle, number of lanes loaded, etc. The report shows how the requirements of the bottom lateral bracing diagonals can be determined in terms of both strength and serviceability.

1.3 Objective and Scope

The objective of this research is to develop the requirements of the secondary members needed to provide a desired level of redundancy in two-girder steel bridges. The bridge engineer can establish inspection, repair, rehabilitation and replacement priorities by comparing the requirements of members for a given level of redundancy to the existing members for a given bridge.

The scope of this research is limited to simple span
noncomposite two-girder bridges with bottom lateral bracing, cross bracing, and top lateral bracing. The bottom and top lateral bracing are assumed to be X-shaped. The requirements of these members are developed for the practical range of existing two-girder bridges with this configuration.
2. REDUNDANCY RATING

Traditional AASHTO design and rating of a two-girder steel bridge deals with two unfractured girders. Redundancy Rating deals with one unfractured girder and one fractured girder. The probability of both girders fracturing simultaneously or one girder containing two simultaneous fractures is assumed to be low enough not to be a consideration.

2.1 Alternate Load Path Concept

In order for redundancy to be possible, the structure must contain at least one viable alternate load path, which must be capable of safely supporting the specified dead and live loads as well as maintaining serviceability of the deck following fracture of one of the two girders. A viable alternate load path needs to be found for various two-girder bridge types. This load path can include secondary members such as lateral bracing, cross bracing, cross frames, and diaphragms. Also, a composite deck acting together with the fractured and unfractured girders may be included in the alternate load path. For the bridge configuration dealt with in this research, the alternate load path consists of the bottom lateral bracing, cross bracing and top lateral bracing.
2.2 Proposed Definition of Redundancy

The term redundancy used in this research refers to the definitions of redundant and nonredundant load path structures as defined in Art. 10.3.1 of Ref. 1. A definition of redundancy was proposed in Ref. 8 which took into account the need for a viable alternate load path as discussed in the previous section:

Redundant Load Path Structure: New, existing, or rehabilitated steel bridges where at least one alternate load path exists and is capable of safely supporting the specified dead and live loads and maintaining serviceability of the deck following the fracture of a main load carrying member.

The intention of this research is to develop the requirements of the members comprising the alternate load path to satisfy this definition.

2.3 Unit Redundancy Rating Factor

The current technique of computing a Rating Factor for each member of a bridge is not considered practical for application to RR. In view of the much more complex analytical models required, the usual rating analysis approach needs to be simplified for practical use. Also, many existing noncomposite two-girder steel bridges will likely yield a Redundancy Rating Factor (RRF) of zero or less (i.e. the bridge cannot support its own dead load after fracture of a girder). This is because either the members and connections of the alternate load path cannot carry the required loads or no suitable alternate load path can be found. An RRF of zero is of little use to the bridge engineer who is interested in establishing bridge
inspection, repair, rehabilitation and replacement priorities. The engineer is more likely to be interested in knowing what modifications of the members and connections on the alternate load path are necessary to achieve the required level of redundancy.

An alternate approach, one that more directly meets the needs of the bridge engineer, is to determine the requirements of the alternate load path in terms of an RRF equal to unity for a given rating vehicle, number of lanes loaded, etc (8). Redundancy classifications as well as bridge inspection, repair, rehabilitation and replacement priorities can more easily be established in terms of the resulting requirements of the alternate load path.

2.4 Proposed Redundancy Rating Methods

The alternate load path is evaluated in terms of both strength and serviceability (8). The strength requirement is based on the current AASHTO Allowable Stress and Load Factor Methods (4). The serviceability requirement is new and is based on a permissible in-service after-fracture deflection and/or transverse slope of the deck. Both are incorporated in terms of a limiting deflection-to-span-length ratio. The establishment of a serviceability requirement is outside the scope of this research, although reasonable values are suggested in Article 3.4.4.
2.5 Components of the Alternate Load Path

The suitable alternate load path which incorporates both the unfractured and fractured girders must carry the required dead, live and impact loads safely and prevent excessive deflections in order to maintain after-fracture serviceability of the deck. The alternate load path for simple span two-girder bridges therefore must contain three basic components (8).

1. A horizontal plane near the top of the girders which provides lateral stiffness and strength and which is connected to the bearings through vertical planes at the ends of the girders.

2. A horizontal plane near the bottom of the girders which develops the forces released at the fracture.

3. Vertical planes at regular intervals along the span which connect the top and bottom horizontal planes.

These three components are shown schematically in Fig. 2 for the bridge configuration considered in this research. The horizontal plane at the top of the girders is provided by a top lateral bracing system for a noncomposite two-girder steel bridge. The horizontal plane at the bottom is provided by a bottom lateral bracing system. The vertical planes are provided by cross bracing as shown in the figure. The vertical planes could be provided by cross frames or diaphragms but these configurations are not considered in this research.

Figure 3 shows a typical top lateral bracing system configuration. It consists of n equal length panels where the length
of each panel is defined by the distance between two adjacent vertical planes. The girder spacing is $S$ and the span length is $\lambda$ as shown in the figure. The top lateral bracing functions like a truss and must consist of web members as shown in the figure plus chord members. The girder flanges function as the chord of the truss. For this reason the top lateral bracing must be near enough to the top flanges in order to efficiently develop the forces in the diagonal web members.

Similarly Fig. 4 shows a typical bottom lateral bracing system configuration. Except for the midspan fracture of the bottom flange of the fractured girder, the geometric configuration of the top and bottom lateral bracing systems are similar. The bottom lateral bracing must be near enough to the bottom flanges in order to develop the forces in the diagonals.

Figure 5 shows typical variations of top and bottom lateral bracing configurations.

Examples of cross bracing and truss bracing configurations which provide the vertical planes are shown in Fig. 6.

There are many configurations of existing two-girder bridges. Fig. 7(a) shows a common example of a bridge with cross bracing providing the vertical planes. This is a common bridge configuration for existing two-girder steel bridges. Some bridges may not contain one or more of the three components required for redundancy. For example, Fig. 7(b) shows a noncomposite two-girder bridge with truss bracing. Many bridges with this configuration do not have a top
lateral bracing system. In order to achieve a desired level of redundancy, a top lateral bracing system can be installed. It is likely that these top laterals can be located at a level just below the top flanges of the stringers as shown in the figure.

A two-girder steel bridge which does not possess the three basic components required for the alternate load path is considered to be nonredundant. It is assumed, however, that most existing bridges can be made redundant with the installation of the required components and the strengthening of the appropriate connections.

A bottom lateral bracing system is a requirement for redundancy for any simple span two-girder bridge. The bottom lateral bracing system is the component of the alternate load path which develops the forces released at the fracture. Therefore, the first step in the Redundancy Rating of a simple span two-girder bridge is to develop the requirements of the bottom lateral bracing system.
3. REQUIREMENTS OF THE BOTTOM LATERAL BRACING SYSTEM

3.1 Computer Model

A computer model was developed to assist in the development of the requirements of the bottom lateral bracing system. The model developed was based on the bridge in Ref. 10. A cross section of the bridge from Ref. 10 showing the noncomposite girders is shown in Fig. 8(a). An elevation view showing the nonprismatic girders is shown in Fig. 8(b). The span length is 150 ft. For this particular span of the bridge in Ref. 10, the flange splice is at quarterspan as noted in the figure. The bridge has X-shaped top and bottom lateral bracing as shown in Fig. 8(c). The girder spacing is 18 ft. Cross bracing spacing is 20 ft. except for the two midspan panels where it is 15 ft. as shown in Fig. 8(c). The floorbeam spacing is 10 ft. Girders are 10 ft. deep. The computer studies were performed using the Computer Aided Engineering Laboratory facility at Fritz Engineering Laboratory and the GTS/STRUDL finite element analysis program.

The support boundary conditions used for the computer model are shown in Fig. 9(a). For simplicity, the bridge is shown with five panels of bottom lateral bracing. The supports are in a horizontal plane at the level of the bottom lateral bracing. The three horizontal restraints shown result in an externally statically determinate structure which is geometrically stable. These support conditions result in no horizontal reaction forces and symmetric
displacements. The expected displacements for these support boundary conditions are shown in Fig. 9(b). Vertical supports are provided at each end of the two girders.

In the computer model the bottom lateral diagonals are considered to be the only system available to develop the forces released at the fracture. Therefore the areas of the cross bracing diagonals and the top lateral bracing are reduced to nearly zero (0.001 in\(^2\)). The model is also adjusted to prevent any relative movement between the two girders so that forces in the bottom lateral diagonals can be developed. This is accomplished by the following two adjustments which are also shown in Fig. 10:

1. Increasing the moment of inertia of the unfractured girder bottom flange about its major axis to practically infinite (1\(0^6\) in\(^4\)).

2. Increasing the area of the cross bracing horizontal to practically infinite (1\(0^6\) in\(^2\)).

These adjustments prevent transverse movement of the girders bottom flanges so that all of the forces released at the fracture can be developed by the bottom lateral bracing diagonals.

The concrete deck and floorbeams are considered as dead load only. The dead load is transferred to the two girders by deck link members as shown in Fig. 11.

A sketch of the finite element mesh used in the modeling is shown in Fig. 12. A summary of the finite elements used for the members is shown in Table 1.
In the computer model the fracture is assumed to extend through the bottom (tension) flange and through the full web depth as shown in Fig. 13. The top (compression) flange is assumed to be intact and capable of resisting the remaining after-fracture compressive force in the girder and the relatively small live load shear at midspan. Inactive nodes are placed throughout the girder web as shown in the figure so the fracture can easily be moved to different locations.

3.2 Allowable Stress Method

The initial development of the Allowable Stress Method considers only the case of midspan fracture of one of the girders. Equations for the forces and corresponding required areas of the bottom lateral diagonals are developed for midspan fracture. These equations are then modified to take into account the effect of different fracture scenarios.

3.2.1 Forces Developed After Fracture

Figure 14 shows a two-girder bridge with five panels of top and bottom lateral bracing. In the figure the girder spacing is S, the girder depth is d and the span length is L. The number of panels of top and bottom lateral bracing is n. The areas of all the bottom lateral diagonal members shown in Fig. 14(a) are assumed to be equal.

The loads and reactions acting on the fractured and unfractured girders are shown in Fig. 15. The weight of the structure is assumed to be applied as a uniform line load, w, on each girder. The
resultant of the live loads is assumed to be at midspan. The fraction of total live, \( L \), plus impact, \( I \), load on the fractured girder is \( \beta \). Therefore \( \beta (L+I) \) and \( (1-\beta)(L+I) \) are the resultant concentrated live loads located at midspan of the fractured and unfractured girders respectively. The unfractured girder is supported at points A and B and the fractured girder's supports are located at points C and D as shown in the figure. By symmetry, the resulting reactions at C and D on the fractured girder are equal. The reactions are found by summing moments about line AB along the unfractured girder and are shown in Fig. 15.

After midspan fracture occurs, the force applied to the bottom flange of the fractured girder on half the span by the bottom lateral bracing diagonals is \( \sum F = F_1 + F_2 + F_3 \) as shown in Fig's. 14(b) and (c). Although the cross bracing may also apply supporting forces to the fractured girder these forces are ignored, which is consistent with the lower bound approach (6). The force \( \sum F \) calculated on the condition of zero bending moment at midspan of the fractured girder is,

\[
\sum F = F_1 + F_2 + F_3 = \frac{1}{d} \left[ \frac{wL^2}{8} + \frac{\beta(L+I)}{4} \right] \quad (3.1)
\]

The summation of forces, \( \sum F_{BL} \), of each of the bottom lateral diagonal members on half the span is equal to \( \alpha(\sum F) \), where \( \alpha \) is the ratio of the length of a bottom lateral diagonal member to the length of the panel. Substituting this into Eq. 3.1, the forces in the bottom
lateral diagonals must sum up to,

$$\sum F_{BL} = \frac{C}{d} \left[ \frac{w^2}{8} + \frac{c(L+1)}{4} \right]$$  \hspace{1cm} (3.2)

Forces $F_1$ and $F_2$ are each developed by two bottom lateral diagonals, one in tension and one in compression as shown in Fig. 14(c). The force $F_3$ is developed by only one member in tension. Studies show that the forces in the diagonal members decrease from midspan to the end of the girder (6,7). That is $F_1 > F_2 > F_3$. Thus, for assumed equal areas of the diagonal members, the required area as governed by tension is determined by the tension force in the diagonals at midspan as shown in Fig. 14(c). Similarly the required area as governed by compression is determined by the compression force in the diagonal in the adjacent panel, as shown in the figure.

Consider, for now, only the tension force in the bottom lateral diagonals at midspan. If all diagonals had equal forces, the force in the diagonal at midspan would be that given in Eq. 3.2 divided by $n$. To account for the increase in force in this diagonal as discussed above, Eq. 3.2 can be multiplied again by a coefficient $v$. Since the coefficient $v$ is different for dead and for live load effects, the coefficient can be separated into a coefficient for dead load, $v_D$, and a coefficient for live load, $v_L$. Thus the dead load force, $F_{BLD}$, and live load plus impact force, $F_{BLL}$, in the tension diagonal at midspan are given by,
\[ F_{BLD} = \frac{\alpha\omega_0^2}{8d} \times \frac{V_D}{n} \quad (3.3) \]

\[ F_{BLL} = \frac{\alpha\varepsilon[L+I]}{4d} \times \frac{V_L}{n} \quad (3.4) \]

The extreme values of \( V_D \) and \( V_L \) can be determined as follows. If the two girders are assumed to have infinite cross sectional areas, then compatibility requires that all bottom lateral diagonal members have equal forces \((6,7)\). In this case \( V_D = V_L = 1.0 \). Similarly if the two girders are assumed to have zero cross sectional areas \( V_D = V_L = n \) and the tension force in the diagonals at midspan carry all of the loads. All other diagonals have zero forces. In what follows, values of \( V_D \) and \( V_L \) between these two limits will be established for practical two-girder bridges.

### 3.2.2 Required Area for Midspan Fracture

As indicated in Art. 2.3, the approach used in this research for the after-fracture evaluation of an existing two-girder steel bridge is to determine the requirements of the alternate load path in terms of an RRF equal to unity. The following Rating Factor for the Allowable Stress Method was previously given in Eq. 1.1,

\[ RF = \frac{f_{all} - f_D}{f_L} \quad (3.5) \]
The stresses in the midspan bottom lateral diagonals from Eq's. 3.3 and 3.4 are,

\[ f_D = \frac{\alpha w^2}{8 d A_{BL}} \cdot \frac{v_D}{n} \]

\[ f_L = \frac{\alpha q (L+I) \lambda}{4 d A_{BL}} \cdot \frac{v_L}{n} \]

where \( A_{BL} \) = area of bottom lateral diagonal.

Substituting the above equations into Eq. 3.5, the RRF for the midspan bottom lateral diagonals is,

\[ \text{RRF} = \frac{f_{all} \left( \frac{\alpha w^2}{8 d A_{BL}} \cdot \frac{v_D}{n} \right)}{\frac{\alpha q (L+I) \lambda}{4 d A_{BL}} \cdot \frac{v_L}{n}} \]  \hspace{1cm} (3.6)

The required area is found by setting the RRF equal to unity.

Setting Eq. 3.6 equal to one, the required area, \( A_{BL} \), of the bottom lateral diagonal members for midspan fracture is,

\[ \text{Required } A_{BL} = \frac{\alpha \lambda}{8 d n f_{all}} \left( v_D w^2 + 2 v_L (L+I) \right) \]  \hspace{1cm} (3.7)

The force, \( F_{BL} \), in the controlling midspan tension diagonal is found by multiplying both sides of Eq. 3.7 by the allowable stress, \( f_{all} \).
3.2.3 Appropriate v Factors

Practical values of $v_D$ and $v_L$ need to be determined by studying existing bridges. A computer study was performed, using the computer model based on the bridge in Ref. 10 and described in Art. 3.1, to determine the variation in $v_D$ and $v_L$ for typical bridges with this configuration and midspan fracture imposed.

Several bridges based on Ref. 10 were modeled for computer analysis, covering practical ranges of span length and number of panels of lateral bracing, but maintaining the 18 foot girder spacing of the bridge in Ref. 10. The computer models included bridges with spans of 100, 150 and 200 feet. The span length to girder depth ratio ($\lambda/d$) was kept constant at 15. The X-shaped bottom lateral bracing is shown in Fig. 16(a). The same relative location of flange splice, at quarterspan, is maintained as shown in Fig. 16(b). The number of panels and the assumed area, $A_{BL}$, of bottom lateral bracing diagonals were varied in each model. Details of the bridges used in the computer study are shown in Table 2. Eighteen different cases are modeled as shown in the table.

For midspan fracture, the bottom lateral diagonal in tension at midspan proves to be more critical than the governing compression member in the adjacent panels in all cases in the computer study. That is, when the tension diagonal is at its allowable tensile

$$F_{BL} = \frac{\alpha \lambda}{8dn} (v_D w + 2v_L \beta [L+1])$$

(3.8)
stress, the compression diagonal is always below its allowable compressive stress assuming that it is braced at mid-length by the tension diagonal in that panel. Therefore parametric studies were performed to determine simple expressions for $v_D$ and $v_L$ for the tension diagonals at midspan for the above 18 cases.

Values of $v_D$ and $v_L$ were obtained by substituting the values of $F_{BLD}$ and $F_{BLL}$ from the computer output into Eq's. 3.3 and 3.4. These thirty six values of $v_D$ and $v_L$ are plotted as a function of the stiffness parameter $R_k$ and the number of panels, $n$, in Fig's. 17 and 18.

Where,

$$R_k = \frac{A_{BL}}{\alpha^3 A_f}$$

The stiffness parameter, $R_k$, is a function of the ratio of the axial stiffness of a bottom lateral bracing diagonal member to the axial stiffness of the effective area, $\bar{A}_f$, of the bottom flange,

where,

$$\bar{A}_f = A_f + 0.3 A_w$$

$A_f$ = Average area of one girder bottom flange

$A_w$ = Area of girder web

Using a trial and error procedure and maintaining the condition that the RRF must equal unity, the curves in Fig's. 17 and 18 were used together with Eq. 3.7 to compute the points plotted in Fig's. 19 and 20. The procedure is as follows. First an $A_{BL}$ is assumed. With this value of $A_{BL}$, $R_k$ is calculated and values of $v_D$ and $v_L$ are obtained from Fig's. 17 and 18. These values of $v_D$ and $v_L$ are then substituted into Eq. 3.7 and the required $A_{BL}$ is calculated. The
assumed value of \( A_{BL} \) is then compared to the required value. This process is continued until convergence of the assumed and required \( A_{BL} \). This procedure is performed for the six combinations of span length and number of panels used in the study as shown in Table 2. The coefficients \( v_D \) and \( v_L \) are plotted for two assumed values of allowable stress in Fig's. 19 and 20 as a function of the three different span lengths used in the study, and with two values of \( n \) for each span length.

The straight lines shown in Fig's. 19 and 20 represent a conservative best fit of the data points. They can also be used to determine the coefficients \( v_D \) and \( v_L \) for other practical span lengths and allowable stresses. The equations of these straight lines is as follows,

\[
\begin{align*}
v_D &= 0.8 + 0.36 \lambda / f_{all} \\
v_L &= 0.8 + 0.36 \lambda / f_{all}
\end{align*}
\]  

(3.9)  

(3.10)

where \( \lambda \) is in feet and \( f_{all} \) is in ksi.

Table 3 shows a comparison of the required \( A_{BL} \) using the data points in Fig's. 19 and 20 to the results obtained using Eq's. 3.9 and 3.10. For rows *2 and *4 the values on the rows labeled "computer analysis" were calculated using the data points. The values below these were computed using the coefficients \( v_D \) and \( v_L \) given by Eq's. 3.9 and 3.10. A similar procedure is used to determine the required areas in rows *1 and *3. The four levels of allowable stress were chosen to determine if Eq's. 3.9 and 3.10 would provide results reasonably close to those obtained by computer
analysis for practical ranges of \( f_{all} \). The simplified equations result in conservative estimates of \( A_{BL} \), and are within 9% of the computed value.

The results of the equations for midspan fracture of one of the girders are checked for each of the six bridges with the combinations of span length and \( n \) shown in Table 2. The required area of bottom lateral diagonal from Eq's. 3.7, 3.9 and 3.10, using \( f_{all} = 27 \text{ ksi} \), is input into the computer model for each of the six combinations of span length and number of panels. The resulting bottom lateral forces for the six bridges are shown in Fig's. 21(a) thru (f). The results of this computer output are summarized in Table 4. The maximum tensile stress is below the allowable of 27 ksi for five of the six bridges as shown in Table 4A. There is a slight over stress in the 200 ft. bridge with thirteen panels. As expected, the midspan tensile diagonal is more critical than the governing compression diagonal for all six bridges. The ratio of the maximum compressive stress to the maximum tensile stress in a bottom lateral diagonal for each of the six bridges is shown in Table 4B.
3.2.4 Critical Fracture Scenarios

Everything done up to this point considers only the case of midspan fracture of one of the girders. A computer study is now performed, using the computer model based on the bridge in Ref. 10 and described in Art. 3.1, to determine if there are more critical fracture scenarios than the midspan fracture. Fractures are introduced in panels other than at midspan on three of the six bridges. The area of bottom lateral diagonal used for each fracture scenario is the required area for midspan fracture as calculated in Eq's. 3.7, 3.9 and 3.10. Table 5 shows the results of this study.

3.2.4.1 Required Area as Governed by Tension

Examination of Table 5A shows that the critical fracture scenario as governed by tension is either midspan fracture or fracture in the panel adjacent to midspan. The biggest increase in maximum tensile stress for a fracture in the panel adjacent to midspan is in the 100 ft. bridge with seven panels. The increase is small (2%) and the maximum tensile stress is still below $f_{all}$.

A study is performed to determine the effect of $A_{BL}$ on the difference in the maximum tensile stress between midspan fracture and adjacent to midspan fracture. Upper and lower bounds of the required $A_{BL}$ for RRF = 1.0 for midspan fracture (Eq's. 3.7, 3.9, and 3.10) are input into the computer model. The upper and lower bounds are ±25% of the required $A_{BL}$. The results of the study are shown in Table 6. The lower bound area results in an increase of 6.3% in the maximum tensile stress of a bottom lateral diagonal. Therefore, to account
for the possible increase in stress due to fracture in the panel adjacent to midspan, an amplification factor is needed. A conservative amplification factor of $\bar{\phi} = 1.1$ is suggested.

The required area of bottom lateral diagonal as governed by tension, for the critical fracture scenario, is obtained by modifying Eq. 3.7 by the amplification factor,

$$A_{BL} = \frac{1.1 \lambda}{8d n f_{all}} (v_D w) + 2v_L (\beta[L+I])$$  \hspace{1cm} (3.11)

where $v_D$ and $v_L$ are calculated from Eq's. 3.9 and 3.10.

3.2.4.2 Maximum Compressive Stress

Examination of Table 5B shows that the critical fracture scenario as governed by compression is fracture in the first interior panel. This fracture scenario is studied for all six bridges with the results summarized in Table 7. From this data it can be seen that compression is most critical for shorter spans with higher values of $\alpha$. The most critical case is the 100 ft. span with seven panels.

It must now be checked to see if compression governs for this case. The buckling model of the compression diagonal, considering it to be braced at mid-length by the tension diagonal, is shown in Fig. 22. The force, $P_T$, in the tension diagonal and the force, $P_{cr}$, in the compression diagonal are shown in the figure. The critical load, $P_{cr}$, of the compression member about its in-plane axis is related to the tension member that braces it at the center. Tests have shown that when the two members have the same size and are made of the same
material, the tension member is equivalent to an unyielding support. Thus the compression member buckles into a full sine wave as shown in the figure, at a load, $P_{cr}$, equal to four times that without any center bracing (11).

Therefore, the length of the column for the buckling model of the compression diagonal is taken as $L$, or half of the length of the diagonal. The end of the compression diagonal where it meets the tension diagonal can be considered a pinned end. The end which frames into the girder is a riveted, bolted, or welded connection. AASHTO (1) recommends the following effective length factors:

- $K = 0.75$ for riveted, bolted, or welded connections.
- $K = 0.875$ for pinned ends

To be conservative, use an effective length factor of $K = 0.875$. Therefore, the effective length of the compression diagonal is taken as $(0.875)L$, where $L$ is half the length of the diagonal.

The 100 ft. span bridge with seven panels is checked to see if the maximum compressive stress in a bottom lateral diagonal as a result of fracture in the first interior panel exceeds the allowable compressive stress. Using the Operating Rating level and assuming that the yield strength is 36 ksi, the allowable stress for compression in a concentrically loaded column is given by,

$$f_{all,c} = 21180 - 0.67(KL/r)^2$$  \hspace{1cm} (3.12)

The forces in the bottom lateral diagonals for the bridge with fracture in the first interior panel are shown in Fig. 23. The
maximum compressive stress is,

\[ f_c = \frac{279 \text{k}}{16.0 \text{ in}^2} = -17.4 \text{ ksi} \]

Equating this stress with the allowable stress given by Eq. 3.12 yields,

\[ 17400 \leq 21180 - 0.67(KI/r)^2 \]
or,

\[ (KI/r) \geq 75 \]

Therefore the required radius of gyration for the bottom lateral diagonal is,

\[ r > K/75 = (0.875)(138 \text{ in.})/75 = 1.61 \text{ in.} \]

The required area as governed by tension (Eq's 3.9, 3.10, and 3.11) is 17.6 in\(^2\). The most economical section based on this required area is,

WT 6 X 60, \( A = 17.6 \text{ in}^2 \), \( r_y = 1.57 \text{ in.} \)

This section is adequate for tension but the minimum radius of gyration (1.57 in) is slightly below the required value for compression (1.61 in). Therefore, a formula needs to be developed for the maximum compressive force in a bottom lateral diagonal.

The data from Table 7 is fitted with conservative straight lines as a function of \( \alpha \) and \( \lambda \) in Fig. 24. The equation of these lines is,

\[ \frac{f_c}{f_t} = \frac{f_c}{f_t} = \alpha(0.58 - 0.0014\lambda) \quad (3.13) \]
where \( f_c \) = maximum compressive stress due to fracture in the first interior panel

\( f_t \) = maximum tensile stress due to midspan fracture

Therefore, the maximum compression force in a bottom lateral diagonal for the critical fracture in the first interior panel is given by,

\[
F_{BLc} = \bar{\Phi}_c F_{BL} \tag{3.14}
\]

where, \( \bar{\Phi}_c \) is given by Eq. 3.13

\( F_{BL} \) is from Eq. 3.8, with \( v_D \) and \( v_L \) from Eq's 3.9 and 3.10.

The design engineer can check if the existing bottom lateral diagonal is sufficient for this compression force. If retrofitting is being considered, the section chosen to satisfy \( RRF = 1.0 \) for tension (Eq's. 3.9, 3.10 and 3.11) can be checked if it is satisfactory for the maximum compression force given by Eq. 3.14.

3.3 Load Factor Method

Figure 25 shows the model used for the Load Factor Method with midspan fracture. It is assumed that all of the bottom lateral diagonals in tension are yielded and that all of the diagonals in compression are buckled.

3.3.1 Required Area for Midspan Fracture

The number of bottom lateral diagonals subjected to the total force \( \sum F_{BL} \) (Eq. 3.2), is \((n+1)/2\) as shown in Fig. 25. There is no need for a coefficient, \( v \), because all the tension members have
yielded and carry the same load. Therefore the dead load force, $F_{BLD}$ and live load plus impact force, $F_{BLL}$ in any tension diagonal are given by,

$$F_{BLD} = \frac{2\alpha}{n+1} \frac{w'^2}{8d} \quad (3.15)$$

$$F_{BLL} = \frac{2\alpha}{n+1} \frac{\beta [I+1]}{4d} \quad (3.16)$$

The following Rating Factor for the Load Factor Method was previously given in Eq. 1.2, (4)

$$RF = \frac{\phi u - \gamma_D}{\gamma_L (I+1)} \quad (3.17)$$

In this case,

$$\phi u = (f_y) (A_{BL})$$

where $f_y$ = yield stress level

Therefore the Redundancy Rating Factor (RRF) for the tension bottom lateral diagonals is found by substituting Eq's. 3.15 and 3.16 into Eq. 3.17,

$$RRF = \frac{f_y A_{BL} \frac{2\gamma_D \alpha w'^2}{8d(n+1)}}{\frac{2\gamma_L (\beta[I+1])}{4d(n+1)} f_y}$$

The required area, $A_{BL}$ of bottom lateral diagonal for midspan fracture is found by setting the RRF equal to unity,
3.3.2 Critical Fracture Scenario

The critical fracture scenario which creates the maximum tensile force in the bottom lateral diagonal members needs to be determined. Figure 26 shows the model for the Load Factor Method for fracture in a panel other than midspan. It is assumed that all compression diagonals are buckled and all tension diagonals are yielded on the short side of the fracture as shown in Fig. 26(a). The number of tension diagonals to carry \( \sum F \) for the short span is \( \left[ \frac{(n+1)}{2} - i \right] \), where \( i \) is the number of panels from midspan at which the fracture occurs as shown in the figure.

The forces and reactions acting on the fractured girder, ignoring the cross bracing forces, are shown in Fig. 26(b). The summation of forces, \( \sum F \), applied to the bottom flange of the fractured girder by the bottom lateral bracing diagonals is calculated on the condition of zero bending moment at the point of fracture on the fractured girder,

\[
\sum F = \frac{wL}{2} (1/2-i/n) \lambda - \frac{w}{2} (1/2-i/n) \lambda^2 + \frac{1}{2} (1/2+i/n) \beta (L+I) (1/2-i/n) \lambda \]

This equation can be simplified to,

\[
\sum F = \left[ 1 - (2i/n)^2 \right] \left[ \frac{wL^2}{8d} \cdot \frac{\beta (L+I) \lambda}{4d} \right]
\]
Each of the yielded tension diagonals on the short side of the fracture carry the same force. Since the number of tension diagonals to carry $\sum F$ is $[(n+1)/2 - i]$ and $\sum F_{BL} = \sigma \sum F$, the maximum force in a bottom lateral tension diagonal for a fracture in the i'th panel is,

$$F_{BL} = \frac{\sigma [1 - (2i/n)^2]}{[(n+1)/2 - i]} \left[ \frac{w^2}{8d} + \frac{\beta (L+1)}{4d} \right]$$  (3.19)

Therefore, the amplification factor, $\bar{\Phi}$, to take into account the location of fracture is $F_{BL}$ from Eq. 3.19 divided by $F_{BL}$ for midspan fracture which is given by Eq. 3.15 plus Eq. 3.16,

$$\bar{\Phi} = \frac{[1 - (2i/n)^2]}{[(n+1)/2 - i]} \cdot \frac{(n+1)/2}{(n+1)/2 - i}$$  (3.20)

Table 8 shows the values of $\bar{\Phi}$ for different combinations of n and i. The maximum, or very near to maximum, value of $\bar{\Phi}$ occurs in the i = (n-3)/2 panel. Therefore the critical fracture scenario for the Load Factor Method is fracture at i = (n-3)/2 panels from midspan, or the first interior panel.

3.3.2.1 Required Area

Substituting i = (n-3)/2 into Eq. 3.20 yields,

$$\bar{\Phi}_{max} = 0.75(1 + 1/n)(2 - 3/n)$$

To take into account that $\bar{\Phi}_{max}$ is slightly larger than the value at i = (n-3)/2 for larger numbers of panels as seen in Table 8, the above
Eq. is modified slightly,

$$\bar{\Phi}_\text{max} = 0.77(1 + 1/n)(2 - 3/n)$$  \hspace{1cm} (3.21)

Therefore the required area of bottom lateral diagonal is the value required for midspan fracture (Eq. 3.18) multiplied by the amplification factor (Eq. 3.21),

$$\text{Req'd } A_{BL} = \frac{\bar{\Phi}_\text{max} \lambda}{4d(n+1)f_Y} [\gamma_D^w \lambda + 2\gamma_L \beta(L+I)]$$  \hspace{1cm} (3.22)

The force, $F_{BL}$, in each of the yielded tension diagonals is found by multiplying both sides of Eq. 3.22 by the yield stress, $f_Y$,

$$F_{BL} = \frac{\bar{\Phi}_\text{max} \lambda}{4d(n+1)} [\gamma_D^w \lambda + 2\gamma_L \beta(L+I)]$$  \hspace{1cm} (3.23)
3.4 Serviceability Method

The Serviceability Method is only used to determine the requirements of the bottom lateral bracing system. The required area, $A_{BL}$, of bottom lateral diagonal is determined to satisfy a $(\Delta/l)_{\text{lim}}$. The design engineer can choose the maximum permissible deflection-to-span-length ratio he will tolerate. The Serviceability Method will tell the engineer the required $A_{BL}$ to limit the deflection to this value. If serviceability controls, the requirements of the cross bracing and top lateral bracing are found from the Allowable Stress Method equations using the value of $F_{BL}$ from the Serviceability Method.

For the Serviceability Method it is assumed that each half span of the fractured girder remains straight after fracture. It is also assumed that there is no lateral displacement of the girders. The unfractured girder is assumed to remain straight. The requirements of the bottom lateral bracing system are first determined for the case of midspan fracture.

3.4.1 Required Area for Midspan Fracture

Figure 27 shows the displacement relationships for the fractured girder and the bottom lateral bracing. From Fig. 27(a) it can be seen that,

$$\frac{\Delta}{l} = \frac{h}{2d} \quad (3.24)$$

where, $h =$ horizontal displacement of fractured girder at midspan as
shown in the figure

\[ \triangle = \text{vertical displacement of fractured girder at midspan} \]

From Fig. 27(b), the strain of the bottom lateral tension diagonal at midspan is,

\[ \varepsilon_{BL} = \frac{h/\alpha}{\alpha h/n} = \frac{nh}{\alpha} \]

The stress in the bottom lateral diagonal is,

\[ f_{BL} = E \varepsilon_{BL} = \frac{E nh}{\alpha^2} \quad (3.25) \]

Coefficients, similar to \( v_D \) and \( v_L \) in the Allowable Stress Method, are needed for the Serviceability Method. The coefficients for the Serviceability Method are defined as \( \frac{f_D}{n} \) for dead load and \( \frac{f_L}{n} \) for live plus impact. The dead load force, \( F_{BLD} \), and live load plus impact force, \( F_{BLI} \), in the bottom lateral tension diagonal at midspan for the Serviceability Method are found by replacing \( v \) with \( f \) in the Allowable Stress Method equations (Eq's. 3.3 and 3.4), as follows,

\[ F_{BLC} = \frac{\alpha w^2}{8d} \ast \frac{f_D}{n} \quad (3.26) \]

\[ F_{BLI} = \frac{\alpha (gL+I)}{4d} \ast \frac{f_D}{n} \quad (3.27) \]
Dividing Eq's. 3.26 and 3.27 by $A_{BL}$ and substituting for $f_{BL}$ in Eq. 3.25 gives,

$$h = \frac{\alpha^2}{E_n A_{BL}} \left[ \frac{\alpha w L^2}{8d_n} + \frac{\alpha \zeta (L+I) L}{4d_n} \right]$$

Substituting this value of $h$ into Eq. 3.24 gives,

$$\frac{\Delta}{\lambda} = \frac{\alpha^3 \lambda^2}{16E_n d^2 A_{BL}} \left[ \frac{\zeta_D w L}{\lambda} + 2\zeta_L \zeta (L+I) \right] \quad (3.28)$$

In the Serviceability Method, the requirements of the alternate load path are determined by satisfying a $\Delta/\lambda$ limit. Solving Eq. 3.28 for the required area of bottom lateral diagonal,

$$\text{Req'd } A_{BL} = \frac{\alpha^3 \lambda^2}{16E_n d^2 (\Delta/\lambda)_{\text{lim}}} \left[ \frac{\zeta_D w L}{\lambda} + 2\zeta_L \zeta (L+I) \right] \quad (3.29)$$

It was found that it is easier to develop equations for ($\approx/n$)$\zeta_D$ and ($\approx/n$)$\zeta_L$ than for $\zeta_D$ and $\zeta_L$. Therefore $u_D = (\approx/n)\zeta_D$ and $u_L = (\approx/n)\zeta_L$ are substituted into Eq. 3.29,

$$\text{Req'd } A_{BL} = \frac{\alpha^2 \lambda^2}{16E_n d^2 (\Delta/\lambda)_{\text{lim}}} \left[ u_D w L + 2u_L \zeta (L+I) \right] \quad (3.30)$$
3.4.2 Appropriate u Factors

Suitable values of $\delta_D$ and $\delta_L$ are found for the Serviceability Method in a similar manner as $v_D$ and $v_L$ were found for the Allowable Stress Method. This is done by obtaining the value of $\Delta$ due to dead load from the computer, substituting it into Eq. 3.29 with the bridge data and the dead loads only, and solving for $\delta_D$ for each of the eighteen cases shown in Table 2. The same procedure is used to find $\delta_L$. These thirty six values of $\delta_D$ and $\delta_L$ are plotted as a function of the stiffness parameter, $R_k$, and $n$ in Fig's. 28 and 29.

A trial and error procedure is used to determine values of $\delta_D$ and $\delta_L$ for the study bridges for different $(A/\lambda)$ limits. The procedure is as follows. First an $A_{BL}$ is assumed. With this value of $A_{BL}$, $R_k$ is calculated and values of $\delta_D$ and $\delta_L$ are obtained from Fig's 28 and 29. These values of $\delta_D$ and $\delta_L$ are then substituted into Eq. 3.29 and the required $A_{BL}$ is calculated. The assumed value of $A_{BL}$ is then compared to the required value from Eq. 3.29. This process is continued until convergence of the assumed and required $A_{BL}$. This procedure is performed for the six combinations of span length and number of panels for $A/\lambda$ limits of 1/200 and 1/300. The values of $u_D$ and $u_L$ are found from $u_D = (\lambda/n)\delta_D$ and $u_L = (\lambda/n)\delta_L$. The results are summarized in Table 9.

The maximum values of $u_D$ and $u_L$ for each span length shown underlined in Table 9 are plotted as 1/u versus span length for each $A/\lambda$ limit in Fig's. 30 and 31. The straight lines shown in Fig's. 30 and 31 represent a conservative best fit of the data points. The
equations of these lines is of the form,

\[ \frac{1}{u_D} = 0.5 + C_D \quad (3.31) \]

\[ \frac{1}{u_D} = 0.5 + C_L \quad (3.32) \]

The equations for the slope of these lines, \( C_D \) and \( C_L \), as found from a best fit of the data are,

\[ C_D = 0.03 - 7 \times 10^{-5} (\lambda/\Lambda) \]

\[ C_L = 0.035 - 7 \times 10^{-5} (\lambda/\Lambda) \]

Substituting these equations for \( C_D \) and \( C_L \) into Eq's. 3.31 and 3.32 yields,

\[ u_D = \frac{100}{50 + 3 - \frac{0.007}{(\Delta/\lambda)_{\text{lim}}}} \quad (3.33) \]

\[ u_L = \frac{100}{50 + 3.5 - \frac{0.007}{(\Delta/\lambda)_{\text{lim}}}} \quad (3.34) \]

Therefore, the required area of bottom lateral diagonal for midspan fracture is given by Eq. 3.30 with values of \( u_D \) and \( u_L \) from Eq's. 3.33 and 3.34.
3.4.3 Critical Fracture Scenario

A computer study is performed to determine the critical fracture scenario for the Serviceability Method. The critical fracture is defined as the one which results in the maximum end slope, $\theta$, of the bridge. Figure 32(a) shows the case of midspan fracture. Fig. 32(b) shows a fracture in a panel other than midspan.

3.4.3.1 Maximum Slope

An amplification factor, $\overline{\Phi}_e$, for the increase in slope due to fracture location is introduced,

$$\overline{\Phi}_e = \frac{\theta_{fs}}{\theta_{mf}}$$

where, $\theta_{fs} =$ girder slope due to fracture in a panel other than midspan

$\theta_{mf} =$ girder slope due to midspan fracture

The ratio of deflection, $\Delta_{fs}$, as a result of non-midspan fracture to the deflection, $\Delta_{mf}$, as a result of midspan fracture is defined as $\overline{\Phi}_d$. From Fig. 32,

$$\overline{\Phi}_e = \frac{\theta_{fs}}{\theta_{mf}} = \frac{\Delta_{fs} / (0.5 - i/n)}{\Delta_{mf} / 0.5}$$

or,

$$\overline{\Phi}_e = \frac{\overline{\Phi}_d}{1 - 2i/n}$$

Examination of the data from the computer study on fracture
scenario for the Allowable Stress Method reveals that the critical fracture scenario for maximum girder slope is fracture in the end panel. Fracture is introduced in the end panel of three of the bridges in the computer study. The $A_{BL}$ used is calculated from Eq's. 3.30, 3.33 and 3.34 for midspan fracture. Three values of $A_{BL}$ are calculated for each bridge using $\Delta/\lambda$ limits of 1/100, 1/200 and 1/300. The results of the study are summarized in Table 10. The assumed value of $(\Delta/\lambda)_{lim}$ is at the top of each column. The actual values of $(\Delta/\lambda)$ from the computer output are shown in parenthesis in the column labeled midspan fracture. Equations 3.30, 3.33 and 3.34 result in conservative values of $\Delta/\lambda$ in all cases.

The values of $\Phi_e$ vary from 1.79 to 1.57. The following equation is developed to fit the data,

$$\Phi_e = 1.8 - 1.6/n$$

Therefore, the critical girder slope from fracture in the end panel can be found from,

$$\Theta_{cr} = \Phi_e \Theta_{mf} = [1.8 - 1.6/n](2)(\Delta/\lambda)_{lim}$$

or,

$$\Theta_{cr} = [3.6 - 1.6/n](\Delta/\lambda)_{lim} \quad (3.35)$$

Where, $(\Delta/\lambda)_{lim}$ is the $\Delta/\lambda$ limit for midspan fracture.

The critical slope (Eq. 3.35) for fracture in the end panel for a given bridge can be checked based on the chosen $(\Delta/\lambda)_{lim}$. The bridge engineer can decide if the critical girder slope is satisfactory based on serviceability considerations.
3.4.4 Suitable \((\Delta/\lambda)\) Limits

A reasonable range of \((\Delta/\lambda)_{\text{lim}}\) needs to be established. The lower bound of \((\Delta/\lambda)_{\text{lim}}\) is based on the existing deflections in bridges. The deflection of the unfractured bridge is shown in Fig. 33(a). In the unfractured bridge the only deflection, \(\Delta_L\), is due to live load because it is assumed that the bridge is cambered to equal the dead load deflection. The live load deflection-to-span-length ratio is limited by AASHTO \((1)\) to 1/800.

The deflection of the fractured bridge is shown in Fig. 33(b). In this case the \((\Delta/\lambda)_{\text{lim}}\) is based on \((\Delta_L + \Delta_D)/\lambda\). It is assumed that \(\Delta_L/\lambda\) is equal to its limiting value of 1/800. Therefore, if the dead load deflection, \(\Delta_D\), is equal to the live load deflection, \(\Delta_L\), the total deflection-to-span-length ratio is, \((1/800) + (1/800) = (1/400)\).

However, in existing bridges the dead load deflection is usually significantly larger than the live load deflection. Assuming that \(\Delta_D\) is twice \(\Delta_L\),

\[
\frac{\Delta_D + \Delta_F}{\lambda} = \frac{3}{800} = \frac{1}{267}
\]

Therefore the lower bound of \((\Delta/\lambda)_{\text{lim}}\) is suggested as \((\Delta/\lambda)_{\text{lim}} = (1/300)\).

The upper bound of \((\Delta/\lambda)_{\text{lim}}\) is based on the maximum amount of deflection at which vehicles can still safely traverse the bridge. This limit is a matter of judgement on the part of the bridge engineer. A maximum value of \((\Delta/\lambda)_{\text{lim}} = 1/100\) is suggested in this
Therefore it is suggested to use a \((\Delta/\lambda)_{\text{lim}}\) between 1/100 and 1/300 for the Serviceability Method.

3.5 Resulting \((\Delta/\lambda)\) from the Strength Methods

Formulas are now developed for the \((\Delta/\lambda)\) values resulting from determining the requirements of the bottom lateral bracing by each of the strength methods. This will enable the bridge engineer to know the \((\Delta/\lambda)\) corresponding to a required \(A_{\text{BL}}\) based on strength considerations. If the resulting \((\Delta/\lambda)\) is too high in the opinion of the engineer, a new \(A_{\text{BL}}\) can be determined using the Serviceability Method equations with a satisfactory value of \((\Delta/\lambda)_{\text{lim}}\).

3.5.1 Allowable Stress Method

A formula needs to be developed for the resulting \((\Delta/\lambda)\) from the required area, \(A_{\text{BL}}\), of bottom lateral diagonals for the Allowable Stress Method. This value of \((\Delta/\lambda)\) can be determined from the Serviceability Method equations (Eq's 3.30, 3.33 and 3.34). Solving Eq. 3.30 for \((\Delta/\lambda)\) yields,

\[
\frac{\Delta}{\lambda} = \frac{\alpha^2 L^2}{16E nd^2 A_{\text{BL}}} \left[ u_D w \lambda + 2u_L \beta (L+I) \right] \tag{3.36}
\]

The value of \((\Delta/\lambda)\) can be found by substituting the \(A_{\text{BL}}\) from the Allowable Stress method equations (Eq's. 3.7, 3.9 and 3.10) into Eq. 3.36. This requires a trial and error procedure because the
equations for $u_D$ and $u_L$ (Eq's. 3.33 and 3.34) are also in terms of $(\Delta/\lambda)$. Another equation needs to be developed which will solve for $(\Delta/\lambda)$ without the need of a trial and error solution. The plots of $\zeta_D$ and $\zeta_L$ versus the stiffness parameter, $R_K$, shown in Fig's. 28 and 29 are fit with conservative curves,

$$\zeta_D = \left[ 1 + 0.55(n^2R_K) \right]$$

$$\zeta_L = \left[ 1 + 0.30(n^2R_K) \right]$$

Substituting $(\gamma/n)\zeta_D$ and $(\gamma/n)\zeta_L$ for $u_D$ and $u_L$ in Eq. 3.36 yields,

$$\frac{\Delta}{\lambda} = \frac{\alpha^2 L^2}{16Ed^2A_{BL}} \left[ \frac{\alpha}{n} (1 + 0.55n^2R_K)w_0 + \frac{2\alpha}{n} (1 + 0.30n^2R_K)\beta[L+I] \right]$$

Substituting $R_K = A_{BL}/(\alpha^2A_f)$ into the above equation and simplifying yields,

$$\frac{\Delta}{\lambda} = \frac{L^2}{16Ed^2} \left[ \frac{\alpha^3}{n^2A_{BL}} + \frac{0.55}{A_f} \right] w_0 + \frac{2\alpha^3}{n^2A_{BL}} + \frac{0.6}{A_f} \beta[L+I] \quad (3.37)$$

Where $A_{BL} =$ Area from Allowable Stress Method (Eq. 3.7)

$\bar{A}_f = A_f + A_w$

$A_f =$ Average area of one girder bottom flange

$A_w =$ Area of girder web
Therefore, the resulting \((\Delta/\lambda)\) for the Allowable Stress Method is given by Eq. 3.37. The equation is checked for the six combinations of span length and number of panels used in the computer studies and shown in Table 2. The results are summarized in Table 11. The first column shows the results obtained using Eq. 3.37. The second column shows the results using the trial and error procedure with Eq's 3.30, 3.33 and 3.34 described above. The third column shows the results obtained from the computer output from the Allowable Stress Method study. Table 11 shows that Eq. 3.37 results in a conservative estimate of \((\Delta/\lambda)\) in all six cases.

3.5.2 Load Factor Method

An equation needs to be developed for the resulting \((\Delta/\lambda)\) from the required area, \(A_{BL}\), of bottom lateral diagonals for the Load Factor Method. The equation for \((\Delta/\lambda)\) is derived assuming that the bottom lateral diagonal in the end panel has just yielded. The equation is developed with this assumption because the diagonal in the end panel will be the last one to reach yield.

Figure 34 shows the displacements of the fractured girder and the bottom lateral system after fracture on a bridge with seven panels. In Fig. 34(a) the displacements of the bottom lateral system are shown. The compression diagonals are assumed to be buckled. The horizontal displacement of the bottom flange at the fracture is assumed as \(h\) as shown in the figure. The horizontal displacements due to girder shortening on the fractured girder are \(s_1\) through
\[ s_{(n+1)/2} \] as shown in the figure. Since no girder shortening occurs between joint A and the fracture, \( s_1 = 0 \). The horizontal displacements of joints F, G, H and I on the unfractured girder due to girder elongation are \( e_1, e_2, e_{(n-1)/2} \) and \( e_{(n+1)/2} \) respectively.

The force distribution in the bottom lateral diagonals is shown in Fig. 34(b). Each yielded tension diagonal carries the same force, \( (F_{BL}/A_{BL}) \). There are \( (n+1)/2 \) tension diagonals to carry the total force, \( F \), applied to the bottom flange of the fractured girder on half the span.

Figure 34(c) shows the force distribution along the girder flanges. The displacement, \( u \), of joint B relative to joint A is (7),

\[
U = NL \left[ \frac{1}{A \cdot E} + \frac{d^2}{4 I \cdot E} \right] \tag{3.38}
\]

Where, \( N = \) sum of the forces applied at joints B through E
\( E = \) Young's Modulus
\( L = \) bay length \((\lambda/n)\)
\( d = \) girder depth
\( A \) = area of girder
\( I \) = moment of inertia of girder

The elongation of the end bottom lateral diagonal needs to be found since the equation for \( (\Delta/\lambda) \) is being developed based on this member just reaching its yield. From Fig. 34(a), the elongation, \( e_{eq} \) of the end diagonal is,
\[ e_{ed} = \frac{1}{\alpha} \left[ h - s(n+1)/2 - e(n-1)/2 \right] \quad (3.39) \]

Equation 3.38, with values of \( N \) from the force distribution in Fig. 34(c), is used to calculate the values of \( s(n+1)/2 \), \( e(n-1)/2 \) and \( e_1 \).

\[ s(n+1)/2 = \left[ \frac{F}{n+1} \right] \left[ \sum_{i=1}^{(n-1)/2} (2i) \right] \left[ \frac{\lambda}{n} \right] \left[ \frac{1}{A_gE} + \frac{d^2}{4I_gE} \right] \quad (3.40) \]

\[ e(n-1)/2 = \left[ \frac{F}{n+1} \right] \left[ \frac{n-3}{2} + \sum_{i=1}^{(n-1)/2} 2i \right] \left[ \frac{\lambda}{n} \right] \left[ \frac{1}{A_gE} + \frac{d^2}{4I_gE} \right] \quad (3.41) \]

\[ e_1 = \left[ \frac{F}{n+1} \right] \left[ \frac{\lambda}{2n} \right] \left[ \frac{1}{A_gE} + \frac{d^2}{4I_gE} \right] \quad (3.42) \]

The girders of the bridge are nonprismatic. To take this into account, use an average area, \( A_g \), and moment of inertia, \( I_g \), of the girders in the above equations.

Substituting Eq's. 3.40 and 3.41 into Eq. 3.39, the elongation of the end diagonal is,

\[ e_{ed} = \frac{1}{\alpha} \left[ h - \left[ \frac{F}{n+1} \right] \left[ \frac{\lambda}{n} \right] \left[ \frac{1}{A_gE} + \frac{d^2}{4I_gE} \right] \left[ \sum_{i=1}^{(n-1)/2} 2i + \sum_{i=1}^{(n-3)/2} 2i + \frac{n-3}{2} \right] \right] \]
The above formula can be simplified to,

\[
e_{\text{ed}} = \frac{1}{\alpha} \left[ h - \frac{(n-2)F_L}{2nE} \left[ \frac{1}{A_g} + \frac{d^2}{4I_g} \right] \right]
\]

(3.43)

The length of a bottom lateral diagonal is \( \lambda/n \). Therefore, the strain, \( \epsilon_{\text{ed}} \), in the end diagonal is,

\[
\epsilon_{\text{ed}} = \frac{e_{\text{ed}}}{\lambda/n}
\]

Substituting this into Eq. 3.43 yields,

\[
e_{\text{ed}} = \frac{n}{\lambda^2} \left[ h - \frac{(n-2)F_L}{2nE} \left[ \frac{1}{A_g} + \frac{d^2}{4I_g} \right] \right]
\]

(3.44)

Assuming that the end diagonal has just reached the yield strain \( \epsilon_y = f_y/E \), the force in each bottom lateral diagonal is \( F_{BL} = (A_{BL})(f_y) \). Since there are \((n+1)/2\) yielded tension diagonals, the total force, \( F \), applied to the bottom flange of the fractured girder on half the span is,

\[
F = \left[ \frac{1}{\alpha} \right] \left[ \frac{n+1}{2} \right] (A_{BL})(f_y)
\]

(3.45)

Substituting Eq. 3.45 into Eq. 3.44 with \( \epsilon_{\text{ed}} = (f_y)/E \) yields,
\[
\frac{f_y}{E} = \frac{n}{2} \left[ h - \frac{(n-2)(n+1)}{2nE} \left[ \frac{1}{\alpha} \right] A_{BL} f_y \left[ \frac{1}{A_y} + \frac{d^2}{4I_y} \right] \right]
\]

Solving for \(h\) and simplifying,

\[
h = \frac{f_y}{nE} \left[ \alpha^2 + \frac{(n+1)(n-2)A_{BL}}{4} \left[ \frac{1}{A_y} + \frac{d^2}{4I_y} \right] \right] \tag{3.46}
\]

From the fractured girder elevation shown in Fig. 27, \((\Delta/\lambda) = (h/2d)\). Substituting \(h\) from Eq. 3.46 yields,

\[
\frac{\Delta}{\lambda} = \frac{f_y}{2Edn} \left[ \alpha^2 + \frac{(n+1)(n-2)A_{BL}}{4\alpha} \left[ \frac{1}{A_y} + \frac{d^2}{4I_y} \right] \right] \tag{3.47}
\]

Therefore the resulting \((\Delta/\lambda)\) for the Load Factor Method is given by Eq. 3.47. This equation is checked to see the resulting \((\Delta/\lambda)\) values for the six combinations of span length and number of panels. The results are summarized in Table 12. The resulting \((\Delta/\lambda)\) varies from 1/183 to 1/275. As was the case in the Allowable Stress Method, the shorter spans have a more severe \((\Delta/\lambda)\) value.

3.5.2.1 Ratio of the Midspan Diagonal Strain to Yield Strain

It is important to know how far past yield the midspan tension diagonals have yielded. From Fig. 34(a), the elongation, \(e_{md}\), of the midspan diagonal is,

\[
e_{md} = \frac{1}{\alpha} (h + e_1) \tag{3.48}
\]
Substituting Eq. 3.45 into Eq. 3.42 yields,

\[ e_1 = \frac{A_{BL} f_y}{4 \alpha n E} (n-3) \left\{ \frac{1}{A_g} + \frac{d^2}{4I_g} \right\} \]

Substituting the above value of \( e_1 \) and \( h \) from Eq. 3.46 into Eq. 3.48 gives the elongation of the midspan diagonal,

\[ e_{md} = \frac{f_y}{\alpha n E} \left[ \alpha^2 + (n^2 - n - 5) \frac{A_{BL}}{4 \alpha} \left\{ \frac{1}{A_g} + \frac{d^2}{4I_g} \right\} \right] \]

The strain of the midspan diagonal is,

\[ \epsilon_{md} = \frac{e_{md}}{\lambda/n} \]

or, \( \epsilon_{md} = \frac{f_y}{E} \left[ 1 + \frac{1}{4 \alpha^3 (n^2 - n - 5) A_{BL}} \left\{ \frac{1}{A_g} + \frac{d^2}{4I_g} \right\} \right] \) \hspace{1cm} (3.49)

The ratio of the mispan diagonal strain, \( \epsilon_{md} \), to the yield strain, \( \epsilon_y \), is found by dividing Eq. 3.49 by \( \epsilon_y = (f_y)/E \),

\[ \frac{\epsilon_{md}}{\epsilon_y} = 1 + \frac{1}{4 \alpha^3 (n^2 - n - 5) A_{BL}} \left\{ \frac{1}{A_g} + \frac{d^2}{4I_g} \right\} \] \hspace{1cm} (3.50)

Equation 3.50 is used on the six combinations of \( \lambda \) and \( n \) used in the computer studies. The results are summarized in Table 12. The
values of $\left( \frac{\varepsilon_{md}}{\varepsilon_y} \right)$ vary from 1.42 to 2.21. These are reasonable values and show that the midspan diagonals are not too far past yield when the end diagonal reaches the yield point.

3.5.3 Comparison of Results

The resulting $(\Delta / \lambda)$ for both of the Strength Methods are compared for the six combinations of $\lambda$ and $n$. The results are summarized in Table 13. The Load Factor Method results in 1.4 to 1.5 times more deflection than the Allowable Stress Method. These are reasonable results. The Allowable Stress Method considers service loads and includes all the bottom lateral diagonals. The Load Factor Method considers factored loads and only the tension diagonals, all of which are assumed to be yielded.
4. REQUIREMENTS OF THE CROSS BRACING SYSTEM

4.1 Transfer of Forces to the Cross Bracing

The cross bracing system must transfer the bottom lateral bracing forces into the top lateral bracing system. All the end and interior cross bracings are assumed to be identical. Components of the forces in the bottom lateral diagonals act on the cross bracing. The forces from the bottom lateral diagonals acting on the unfractured and fractured girders are U and F as shown in Fig 35(a). These forces must be developed by the cross bracing system.

It is assumed that the forces in the two diagonal members of a cross bracing are equal, one in tension and one in compression as shown in the figure. Two configurations of cross bracing are examined in this research. Type A cross bracing consists of X- or K-shaped cross bracing as shown in Fig. 35(b). Type B cross bracing is K-shaped cross bracing as shown in Fig. 35(c).

Assuming that the forces in the two cross bracing diagonals are equal and opposite, the force, $F_{CBH}$, in the cross bracing horizontal and the force, $F_{CBD}$, in the cross bracing diagonals for Type A cross bracing are given by,

$$F_{CBH} = \frac{k_d (U+F)}{2} \tag{4.1}$$

$$F_{CBD} = \frac{k_d (U-F)}{2} \tag{4.2}$$

Where $k_d = \frac{\text{length of a cross bracing diagonal}}{\text{length of a cross bracing horizontal (girder spacing)}}$
For Type B cross bracing the forces are given by,

\[ F_{CBH} = U \]  
\[ F_{CBD} = k_d(U-F)/2 \]  

The force, \( F_{CBD} \), in the cross bracing diagonals is identical for Type A and Type B cross bracing.

4.2 Allowable Stress Method

4.2.1 Cross Bracing Forces for Midspan Fracture

The initial development of the requirements of the cross bracing system for the Allowable Stress Method considers only the case of midspan fracture of one of the girders. The forces in the bottom lateral diagonals for a bridge with five panels are shown in Fig. 36(a). The force, \( F_{BL} \), is the force in the tension diagonal at midspan due to midspan fracture.

The force, \( F_{BL} \), in the controlling midspan tension diagonal was given by Eq. 3.8,

\[ F_{BL} = \frac{\alpha \lambda}{8d_n} (v_D + 2v_L + (I+I)) \]  

Where \( v_D \) and \( v_L \) are given by Eq's. 3.9 and 3.10.

As discussed in Chapter 3, \( \phi_{t1} \) and \( \phi_{c1} \) shown in Fig. 36(a) are less than one and decrease from midspan to the end of the girder. The critical cross bracings for midspan fracture are the cross bracings on either side of the fracture as shown in Fig. 36(a).

The components of the forces in the bottom lateral diagonals
acting at the critical cross bracing location are shown in Fig. 36(b). From the figure,

\[ U = F_{BL} (1 + \phi_{tl}) k_H \]  \hspace{1cm} (4.6)
\[ F = F_{BL} (1 - \phi_{cl}) k_H \]  \hspace{1cm} (4.7)

where

\[ k_H = \frac{\text{length of a cross bracing horizontal}}{\text{length of a bottom lateral diagonal}} \]

The force, \( F_{CEH} \), in the cross bracing horizontal and the force, \( F_{CBD} \), in the cross bracing diagonals for Type A cross bracing are found by substituting Eq's. 4.6 and 4.7 into Eq. 4.1 and Eq. 4.2,

\[ F_{CEH} = (1 + 0.5(\phi_{tl} - \phi_{cl})) \]  \hspace{1cm} (4.8)
\[ F_{CBD} = \pm 0.5(\phi_{tl} + \phi_{cl}) k_{Hd} F_{BL} \]

The product \( k_{Hd} \) can be simplified to,

\[ k_D = k_H k_d = \frac{\text{length of a cross bracing diagonal}}{\text{length of a bottom lateral diagonal}} \]

Substituting this into the above equation for \( F_{CBD} \) yields,

\[ F_{CBD} = \pm 0.5(\phi_{tl} + \phi_{cl}) k_D F_{BL} \]  \hspace{1cm} (4.9)

The forces for Type B cross bracing are found by substituting Eq's. 4.6 and 4.7 into Eq's. 4.3 and 4.4,

\[ F_{CEH} = (1 + \phi_{tl}) k_F F_{BL} \]  \hspace{1cm} (4.10)
The force, $F_{CBD}$, in a bottom lateral diagonal is the same for Type A and Type B cross bracing and is given by Eq. 4.9.

Examination of Eq's. 4.8 and 4.10 reveals that $F_{CBD}$ is a function of $(\phi_{t1} - \phi_{c1})$ for Type A cross bracing and a function of $\phi_{t1}$ for Type B cross bracing. Equation 4.9 for $F_{CBD}$ is the same for Type A and Type B cross bracing and is a function of $(\phi_{t1} + \phi_{c1})$.

All of the computer output for the forces in the bottom lateral diagonals due to midspan fracture, from the computer studies in Chapter 3, is gathered to determine the variation of $\phi_{t1}$, $(\phi_{t1} - \phi_{c1})$ and $(\phi_{t1} + \phi_{c1})$. The data is summarized in Table 14. There are three values of $A_{BL}$ used for each combination of span length and number of panels. The middle value, area required for RRF = 1.0, is found from Eq's. 3.7, 3.9 and 3.10 with $f_{all} = 27$ ksi. The resulting bottom lateral forces for the six bridges with these areas were shown in Fig's. 21(a) thru (f). The other two areas are an upper bound, designated RRF > 1.0, and a lower bound, RRF < 1.0. Examination of the data in Table 10 shows that,

 maximum $\phi_{t1}$ due to dead load = 0.78
 maximum $\phi_{t1}$ due to live load = 0.87
 maximum $(\phi_{t1} - \phi_{c1})$ due to dead load = 0.70
 maximum $(\phi_{t1} - \phi_{c1})$ due to live load = 0.53
 maximum $(\phi_{t1} + \phi_{c1})$ due to dead load = 1.20
 maximum $(\phi_{t1} + \phi_{c1})$ due to live load = 1.60

To be conservative, and recognizing that dead load has a greater influence than live load, the following values are chosen,
\[ \phi_{tl} = 0.80 \]
\[ \phi_{tl} - \phi_{cl} = 0.70 \]
\[ \phi_{tl} + \phi_{cl} = 1.30 \]

These values are substituted into Eq's. 4.8 thru 4.10 to give the equations for the maximum cross bracing forces due to midspan fracture,

Type A: \[ F_{\text{CBH}} = 1.35(k_{H}F_{\text{BL}}) \] (4.11)

Type B: \[ F_{\text{CBH}} = 1.80(k_{H}F_{\text{BL}}) \] (4.12)

Type A and Type B: \[ F_{\text{CBD}} = 0.65(k_{D}F_{\text{BL}}) \] (4.13)

Where, \( F_{\text{BL}} \) is from Eq. 4.5

4.2.2 Other Fracture Scenarios

Examination of the computer data from the fracture scenario study done in Chapter 3 reveals that midspan fracture is critical for the cross bracing horizontal and cross bracing diagonal. The maximum values of \( F_{\text{CBH}} \) and \( F_{\text{CBD}} \) are in the cross bracings adjacent to midspan for the case of midspan fracture. Therefore, the maximum forces in the cross bracing for any fracture scenario are given by Eq's. 4.11, 4.12 and 4.13.
4.3 Load Factor Method

The force, \( F_{CH} \), in the cross bracing horizontal is given by Eq. 4.1 for Type A cross bracing and Eq. 4.3 for Type B cross bracing. The force, \( F_{CBD} \), in the cross bracing diagonal is given by Eq. 4.2 for Type A and Type B cross bracing. The cross bracing forces are first developed for the case of midspan fracture.

4.3.1 Cross Bracing Forces for Midspan Fracture

The bottom lateral diagonal forces for the Load Factor model for a seven panel bridge are shown in Fig. 37(a). As previously noted, all of the compression diagonals are assumed to be buckled and all the compression diagonals are assumed to be yielded.

The force, \( F_{BL} \), in each of the yielded tension diagonals as shown in Fig. 37(a) is found by multiplying each side of Eq. 3.18 by the yield stress, \( f_y \),

\[
F_{BL} = \frac{\alpha \lambda}{4d(n+1)} \left[ \gamma_D W + 2 \gamma_L (L+I) \right]
\]  

(4.14)

The critical cross bracing locations are adjacent to midspan as shown in Fig. 37(a). The components of the forces in the bottom lateral diagonals acting at the critical cross bracing location are shown in Fig. 37(b),

\[
U = 2(k_H F_{BL})
\]

\[
F = k_H F_{BL}
\]

The forces on the cross bracing for Type A cross bracing are found by substituting the above values of \( U \) and \( F \) into Eq's. 4.1 and 4.2,
\[ F_{\text{CEH}} = 1.5(k_H F_{\text{BL}}) \quad (4.15) \]
\[ F_{\text{CBD}} = 0.5(k_D F_{\text{BL}}) \quad (4.16) \]

Similarly, the forces for Type B cross bracing are found by substituting into Eq's. 4.3 and 4.4,

\[ F_{\text{CEH}} = 2.0(k_H F_{\text{BL}}) \quad (4.17) \]
\[ F_{\text{CBD}} = 0.5(k_D F_{\text{BL}}) \]

### 4.3.2 Critical Fracture Scenario

Now the fracture scenario which creates the maximum force in a bottom lateral diagonal is investigated. It was shown in Section 3.3.2 that the critical fracture is in the first interior panel as shown in Fig. 38(a). The amplification factor for this fracture scenario, \( \bar{\phi}_{\text{max}} \), is given by Eq. 3.21. Therefore the force \( F_{\text{BL,max}} \) resulting from fracture in the first interior panel is \( F_{\text{BL}} \) due to midspan fracture (Eq. 4.14) multiplied by \( \bar{\phi}_{\text{max}} \) (Eq. 3.21),

\[ F_{\text{BL,max}} = \bar{\phi}_{\text{max}} F_{\text{BL}} = \frac{\bar{\phi}_{\text{max}}}{4d(n+1)} \left[ \gamma_D \right] + 2 \gamma_L \left( \zeta + \zeta + \zeta \right) \]

(4.18)

where \( \bar{\phi}_{\text{max}} = 0.77(1 + 1/n)(2 - 3/n) \) from Eq. 3.21

Upon examination of the bottom lateral diagonal forces shown in Fig. 38(a), the critical cross bracing is either the first interior (AC) or end (BD) cross bracings. The components of the forces in the bottom lateral diagonals acting on the cross bracing are shown in Fig. 38(b) for the first interior system and in Fig. 38(c) for the
end cross bracing.

The forces on the cross bracing members are found by substituting the values of $U$ and $F$ in Fig's. 38(b) and (c) into Eq's. 4.1, 4.2 and 4.3. These forces are shown under the cross bracing systems in Fig's. 38(b) and (c). Examination of these forces reveals that the first interior cross bracing is critical for the cross bracing horizontal and the end cross bracing is critical for the cross bracing diagonal. Therefore, the maximum forces for Type A cross bracing are given by,

$$F_{CBH} = k_c F_{BL} \left( \bar{\sigma}_{\max} + 0.5\phi_t \right)$$

$$F_{CBD} = 0.5(k_c F_{BL \cdot \max})$$

(4.19)

(4.20)

The maximum forces for Type B cross bracing are,

$$F_{CBH} = k_c F_{BL} \left( \bar{\sigma}_{\max} + \phi_t \right)$$

$$F_{CBD} = 0.5(k_c F_{BL \cdot \max})$$

(4.21)

Comparison of the equations for $F_{CBD}$ for fracture in the first interior panel, Eq. 4.20, with the equation for $F_{CBD}$ for midspan fracture, Eq. 4.16, shows that fracture in the first interior panel is critical for the cross bracing diagonals. This is because $\bar{\sigma}_{\max}$ is always greater than one. Therefore the maximum force in a cross bracing diagonal for Type A and Type B cross bracing is given by Eq. 4.20.

Comparison of Eq's. 4.19 and 4.21 with Eq's. 4.15 and 4.17
reveals that it is not clear which fracture is critical for the cross bracing horizontal. For Type A cross bracing midspan fracture controls if,

$$\bar{\phi}_{\text{max}} + 0.5 \phi_t \leq 1.5$$  (4.22)

For Type B cross bracing midspan fracture controls if,

$$\bar{\phi}_{\text{max}} + \phi_t \leq 2.0$$  (4.23)

These limits need to be checked for the practical range of number of panels, n. From Fig. 38(a), two yielded tension diagonals on the short side of fracture each carry $\bar{\phi}_{\text{max}} F_{BL}$. There are (n-1) tension diagonals to carry the loads on the long side of fracture. Assume that the tension diagonals on the long side of fracture carry the same total as the two yielded tension diagonals and ignore the compression members on the long side of fracture. Then assuming each tension diagonal on the long side of fracture carries the same force,

$$\phi_t = \frac{2}{n-1} \times \bar{\phi}_{\text{max}}$$

The limits given by Eq's. 4.22 and 4.23 are checked for the practical range of n and the results are summarized in Table 15.

From Table 15, $\bar{\phi}_{\text{max}} + 0.5\phi_t > 1.5$. Therefore the fracture in the first interior panel is critical for the cross bracing horizontal for Type A cross bracing. Substituting a conservative estimate of 1.6 for $\bar{\phi}_{\text{max}} + 0.5\phi_t$ into Eq. 4.19, the maximum force in a cross bracing horizontal for Type A cross bracing is,
From Table 15, $\phi_{\text{max}} + \phi_t < 2.0$. Therefore midspan fracture is critical for the cross bracing horizontal for Type B cross bracing. The maximum force in a cross bracing horizontal for Type B cross bracing is given by Eq. 4.17.

In summary, the maximum compression force in a cross bracing diagonal for Type A and Type B cross bracing is given by,

$$F_{\text{CBD}} = 0.5(k_D F_{\text{BL}} \phi_{\text{max}})$$  \hspace{1cm} (4.20)

where $k_D = \frac{\text{length of cross bracing diagonal}}{\text{length of bottom lateral diagonal}}$

$F_{\text{BL}} = \text{Bottom lateral diagonal force due to midspan fracture (Eq. 4.14)}$

$\phi_{\text{max}} = \text{Amplification factor for fracture in a panel other than midspan (Eq. 3.21)}$

The maximum compression force in a cross bracing horizontal is given by,

Type A: $F_{\text{CH}} = 1.6(k_H F_{\text{BL}})$  \hspace{1cm} (4.24)
Type B: $F_{\text{CH}} = 2.0(k_H F_{\text{BL}})$  \hspace{1cm} (4.17)

where $k_H = \frac{\text{length of a cross bracing horizontal}}{\text{length of a bottom lateral diagonal}}$

$F_{\text{BL}} = \text{From Eq. 4.14}$

The bridge engineer can check if the existing cross bracing members are sufficient for these maximum compression forces. If
retrofitting is being considered, a section can be selected which does not exceed the allowable stress for this compression force.
The same assumption used for the distribution of forces in the cross bracing diagonals is used for the top lateral bracing diagonals. It is assumed that the forces in each of the top lateral bracing diagonals in a panel are equal, one in tension and one in compression.

5.1 Allowable Stress Method

The forces from the cross bracing diagonals are transferred to the top lateral bracing system. As for the cross bracing diagonal, the critical fracture for a top lateral bracing diagonal is midspan fracture. The forces acting on the bottom lateral diagonals of a bridge with seven panels and midspan fracture are shown in Fig. 39(a). The factors shown in the figure decrease from midspan to the end of the girder as previously noted. The free body force diagrams of the cross bracings resulting from these bottom lateral diagonal forces are shown in Fig. 39(b).

The applied loads on the top lateral bracing from the cross bracings are shown in Fig. 40(a). The sum of the forces carried by each panel's top lateral diagonals is found in a manner similar to a shear force diagram of a beam and is shown in Fig. 40(b). The geometric component, $k_H$, from Fig. 40(a) disappears because the sum of the top lateral diagonals must equal $(1/k_H) \times \text{(forces applied to the top lateral bracing)}$ for equilibrium. From this diagram it can be
seen that there are no forces in the top lateral bracing diagonals at midspan. Also, the sum of the forces of top lateral bracing diagonals in a panel other than midspan is identical to the sum of the forces of the bottom lateral diagonals in the same panel.

5.1.1 Maximum Compression Force

From the diagram in Fig. 40(b), the maximum forces carried by two diagonals is in the panel adjacent to midspan and is equal to \((\phi_{t1} + \phi_{c1})F_{BL}\). Since the top lateral diagonal forces are assumed to be equal each diagonal carries \(0.5(\phi_{t1} + \phi_{c1})F_{BL}\), one in tension and one in compression.

In the study done for the cross bracing, it was determined that the maximum value of \((\phi_{t1} + \phi_{c1})\) is 1.3. Therefore the maximum compression force, \(F_{TL}\), in a top lateral diagonal is given by,

\[
F_{TL} = 0.5(\phi_{t1} + \phi_{c1})_{\text{max}}F_{BL} = 0.5(1.3)F_{BL}
\]

or,

\[
F_{TL} = 0.65 F_{BL}
\]  

(5.1)

Where, \(F_{BL}\) = Bottom lateral diagonal force due to midspan fracture (Eq. 4.5)
5.2 Load Factor Method

The forces from the cross bracing diagonals are transferred to the top lateral bracing system. As for the cross bracing diagonal, the critical fracture for a top lateral diagonal is fracture in the first interior panel. This fracture creates the maximum bottom lateral tensile force, \( F_{BL, \text{max}} = \Phi_{\text{max}} F_{BL} \). The forces acting on the bottom lateral diagonals of a bridge with seven panels are shown in Fig. 41(a). The tensile diagonals on the short side of fracture are yielded and the compression diagonals are buckled. It is assumed that none of the diagonals on the long side of fracture are yielded or buckled. The free body diagrams of the cross bracings resulting from these bottom lateral diagonal forces are shown in Fig. 41(b).

The applied loads on the top lateral bracing from the cross bracings are shown in Fig. 42(a). The sum of the forces carried by each panel's top lateral diagonals is shown in Fig. 42(b). From this diagram it can be seen that there are very small top lateral forces in the panel with fracture. Also, the maximum top lateral forces are in the end panel next to the fracture.

5.2.1 Maximum Compression Force

From the diagram in Fig. 42(b), the maximum forces carried by two diagonals are equal to \( \Phi_{\text{max}} F_{BL} \). Since the top lateral diagonal forces are assumed to be equal, each diagonal carries \( 0.5(\Phi_{\text{max}} F_{BL}) \). Therefore the maximum compressive force, \( F_{TL} \), in a top lateral diagonal is given by,
\[ F_{TL} = 0.5(\tilde{\phi}_{\text{max}} F_{BL}) \]

Where,  
\( F_{BL} = \) Bottom lateral diagonal force due to midspan fracture  
(Eq. 4.14)  
\( \tilde{\phi}_{\text{max}} = \) Amplification factor to take into account fracture location (Eq. 3.21)
6. EXAMPLES

The formulas developed in Chapters 3, 4 and 5 are used to determine the requirements of the alternate load path for the bridges used in the computer study. Table 2 provides details of span lengths, number of panels, etc. One worked example of the 100 ft. span with seven panels is presented to show how the formulas are used. The worked example is shown for all three of the Redundancy Rating methods. The results from the other five combinations of span length and number of panels are given and discussed. The following assumptions are used for all examples.

6.1 Assumptions

Vehicular Loading: An HS20 truck is used for live plus impact loads. The HS20 truck is found to be the critical vehicular loading for spans up to 200 feet when the truck loading is replaced by an equivalent concentrated load at midspan.

Traffic Lanes Loaded: One traffic lane is loaded.

Allowable Stresses: The allowable stresses for the Operating Rating level are used,

\[
\text{Tension: } f_{\text{all}} = 0.75f_y \\
\text{Compression: } f_{\text{all},C} = 21180 - 0.67(KL/r)^2
\]

Load Factors: Load factors of 1.1 for dead load and 1.3 for live load are used.

Impact Factor: An impact of 30% is used.
Limiting Deflection: Three different values of \((\Delta/\lambda)_{\text{lim}}\) are investigated. The three values used cover the range of values presented in Art. 3.4.4. The values used are \((\Delta/\lambda)_{\text{lim}} = 1/100, 1/200, \text{ and } 1/300\).

6.2 Worked Example: 100 ft. span; \(n = 7\)

The first step is to determine the uniform line load \(w\) and the equivalent concentrated live load plus impact, \(\beta(I+I)\), acting on the fractured girder. The dead load of the bridge is as follows,

- weight of concrete = 5.40 k/ft
- weight of steel = 1.14 k/ft
- weight of future wearing surface = 0.62 k/ft

Total = 7.16 k/ft

The dead load is assumed to be applied as a uniform live load, \(w\), on each girder,

\(w = 0.5(7.16) = 3.58 \text{ k/ft}\)

Figure 43(a) shows the locations of the lines of wheels on the bridge. One lane of HS20 truck loading is applied 1.5 feet from the face of the curb (4). The fraction of truck load, \(\beta\), acting on the fractured girder is found from the influence line shown in Fig. 43(b),

\(\beta = 0.5(1.194 + 0.861) = 1.03\)

Figure 44(a) shows one lane of HS20 truck loading applied to the fractured girder. The truck is positioned longitudinally so that the center of gravity of the truck is at midspan. Therefore the girder
reactions are identical, as shown in the figure. The total live load force, \( F_1 + F_2 + F_3 = F_L' \), acting at the level of the fractured girder bottom flange is calculated on the condition of zero bending moment at midspan,

\[
(F_L')(6.67) = (36)(50) - (32)(8.4)
\]

\[ F_L = 229.6 \text{ k} \]

Using the \( \beta \) factor computed above, the force \( F_L \) at midspan becomes,

\[ F_L = (1.03)(229.6) = 236.5 \text{ k} \]

The live load plus impact force, \( F_{L+I} \), is found by applying the assumed 30% impact factor,

\[ F_{L+I} = (236.5)(1.3) = 307.5 \text{ k} \]

The truck load is now replaced by an equivalent concentrated load, \( \beta(L+I) \), at midspan as shown in Fig. 44(b) which will create the same total force \( F_{L+I} \). From Fig. 44(b),

\[ [\beta(L+I)/2](50) = (307.5)(6.67) \]

or,

\[ \beta(L+I) = 82.0 \text{ k} \]

The next step is to calculate the length ratio terms \( \alpha \), \( k_H \) and \( k_D \). The term \( \alpha \) is the length of a bottom lateral diagonal divided by the length of the panel.

\[ \text{length of panel} = \frac{\lambda}{n} = \frac{100}{7} = 14.29 \text{ ft} \]

For a girder spacing of 18 feet,

\[ \text{length of bottom lateral diagonal} = \left[ (14.29)^2 + (18)^2 \right]^{1/2} = 22.98 \]

Then,

\[ \alpha = (22.98)/(14.29) = 1.61 \]

The term \( k_H \) is the length of a cross bracing horizontal (girder spacing) divided by the length of a bottom lateral diagonal,
\[ k_H = \frac{18.0}{22.98} = 0.78 \]

The term \( k_D \) is the length of a cross bracing diagonal divided by the length of a bottom lateral diagonal. For a girder depth of 6.67 ft. and spacing of 18 feet,

length of cross bracing diagonal = \( \sqrt{(6.67)^2 + (18)^2} \) = 19.20 ft.

Then,

\[ k_D = \frac{19.20}{22.98} = 0.84 \]

This example data for the 100 ft. span bridge with seven panels, along with the data for the other five combinations of span length and number of panels found in a similar manner, is summarized in Table 16A. This data is used for all three Redundancy Rating methods.

6.2.1 Allowable Stress Method

For a yield stress level of \( f_y = 36 \) ksi, the allowable stresses for the Operating Rating level are,

Tension: \( f_{all} = 0.75f_y = 27 \) ksi

Compression: \( f_{all,c} = 21180 - 0.67(KL/r)^2 \)

6.2.1.1 Midspan Fracture

The required area, \( A_{BL} \), of bottom lateral diagonal for midspan fracture is given by Eq. 3.7. The values of the coefficients \( v_D \) and \( v_L \) are found from Eq's. 3.9 and 3.10 (\( L \) is in ft.),

\[ v_D = 0.8 + 0.36(100)/27 = 2.13 \]

\[ v_L = 0.8 + 0.18(100)/27 = 1.47 \]
Substituting into Eq. 3.7, the required area, $A_{BL}$, of bottom lateral diagonal is,

$$\text{Req'd } A_{BL} = \frac{(1.61)(100)}{(8)(6.67)(7)(27)} [(2.13)(3.58)(100) + 2(1.47)(82.0)]$$

or,  $A_{BL} = 16.0 \text{ in}^2$

The maximum forces in the cross bracing are given by Eq's. 4.11 and 4.13. The force, $F_{BL}$, in the bottom lateral diagonal at midspan is found from Eq. 4.5,

$$F_{BL} = \frac{(1.61)(100)}{(8)(6.67)(7)} [(2.13)(3.58)(100) + (2)(1.47)(82.0)] = 432.6 \text{ kN}$$

Substituting into Eq's. 4.11 and 4.13, the maximum force, $F_{CBH}$, in a cross bracing horizontal and the maximum force, $F_{CBD}$, in a cross bracing diagonal are,

$$F_{CBH} = (1.35)(0.78)(432.6) = 455.5 \text{ kN}$$

$$F_{CBD} = (0.65)(0.84)(432.6) = 236.2 \text{ kN}$$

The maximum compressive force, $F_{TL}$, in a top lateral diagonal is found from Eq. 5.1,

$$F_{TL} = (0.65)(432.6) = 281.2 \text{ kN}$$
6.2.1.2 Critical Fracture Scenario

The critical fracture scenario for maximum force in a bottom lateral diagonal is midspan fracture or fracture in the panel adjacent to midspan. An amplification factor of $\bar{\phi} = 1.1$ was suggested to take into account the possible increase in stress due to fracture in the panel adjacent to midspan. Therefore the required area, $A_{BL}$, of bottom lateral diagonal is the required area for midspan fracture multiplied by $\bar{\phi} = 1.1$,

\[
\text{Req'd } A_{BL} = (1.1)(16.0) = 17.6 \text{ in}^2
\]

The critical fracture for maximum compressive stress in a bottom lateral diagonal is fracture in the first interior panel. An amplification factor, $\bar{\phi}_C$, was developed to account for this. The amplification factor, $\bar{\phi}_C$, from Eq. 3.13 is,

\[
\bar{\phi}_C = 1.61[0.58 - 0.14(100)] = 0.71
\]

Therefore the maximum compressive force, $F_{BLC}$, in a bottom lateral diagonal from Eq. 3.14 is,

\[
F_{BLC} = (0.71)(432.6) = 307.1 \text{ k}
\]

The critical fracture for the cross bracing members and the top lateral bracing diagonals is midspan fracture. Therefore the maximum forces in the cross bracing and top lateral bracing are those given for midspan fracture in Art. 6.2.1.1.
6.2.1.3 Resulting (Δ/λ)

The resulting (Δ/λ) from the Allowable Stress Method is given by Eq. 3.37. The effective area, $\bar{A}_f$, of the bottom flange is found from the data given in Table 2,

$$\bar{A}_f = A_f + 0.3 A_w = (18) \left[ \frac{2.5 + 1.875}{2} \right] + (0.3)(80)(0.5) = 51.4 \text{ in}^2$$

Substituting this into Eq. 3.37 yields,

$$\frac{\Delta}{\lambda} = \frac{(100)^2}{(16)(29000)(6.67)^2} \left[ \frac{(1.61)^3}{(7)^2(16.0)} + \frac{0.55}{51.4} \right] + \frac{2(1.61)^3}{(7)^2(16.0)} + \frac{0.6}{51.4} \right] \left(82.0\right) = \frac{1}{273}$$

The Allowable Stress Method results in a (Δ/λ) of 1/273 for the 100 ft. bridge with seven panels.

6.2.2 Load Factor Method

6.2.2.1 Midspan Fracture

The required area, $A_{BL}$, of bottom lateral diagonal for midspan fracture was given by Eq. 3.18,

$$\text{Req'd } A_{BL} = \frac{(1.61)(100)}{(4)(6.67)(8)(36)} \left[ (1.1)(3.58)(100) + 2(1.3)(82.0) \right]$$

or, $A_{BL} = 12.7 \text{ in}^2$
The maximum forces in the cross bracing are given by Eq's. 4.15 and 4.17. The force, $F_{BL}$, in the bottom lateral diagonal at midspan is found from Eq. 4.14,

$$F_{BL} = \frac{(1.61)(100)}{4(6.67)(8)} \left[ (1.1)(3.58)(100) + 2(1.3)(82.0) \right] = 457.9 \text{ kN}$$

Substituting the above value of $F_{BL}$ into Eq's. 4.15 and 4.17, the maximum force, $F_{CBH}$, in a cross bracing horizontal and the maximum force, $F_{CBD}$, in a cross bracing diagonal are,

$$F_{CBH} = (1.5)(0.78)(457.9) = 535.7 \text{ kN}$$
$$F_{CBD} = (0.5)(0.84)(457.9) = 192.3 \text{ kN}$$

The maximum force, $F_{TL}$, in a top lateral diagonal due to midspan fracture is found from Eq. 5.2 without the amplification factor, $\Phi_{max}$,

$$F_{TL} = (0.50)(457.9) = 229.0 \text{ kN}$$

6.2.2.2 Critical Fracture Scenario

The critical fracture scenario for maximum forces in a bottom lateral diagonal and the cross bracing is fracture in the first interior panel. The value of the amplification factor $\Phi_{max}$ is found from Eq. 3.21,

$$\Phi_{max} = 0.77 \left( 1 + \frac{1}{7} \right) \left( 2 - \frac{3}{7} \right) = 1.38$$
Therefore the required area, $A_{BL}$, of bottom lateral diagonal is the required area for midspan fracture multiplied by $\bar{\phi}_{\text{max}} = 1.38$,

$$\text{Req'd } A_{BL} = (1.38)(12.7) = 17.5 \text{ in}^2$$

The maximum forces, $F_{CBD}$ and $F_{CBH}$, in the cross bracing due to fracture in the first interior panel are found from Eq's. 4.20 and 4.24,

$$F_{CBD} = (0.5)(0.84)(457.9)(1.38) = 265.4 \text{ k}$$

$$F_{CBH} = (1.6)(0.78)(457.9) = 571.5 \text{ k}$$

The maximum force, $F_{TL}$, in a top lateral bracing diagonal is found from Eq. 5.2,

$$F_{TL} = (0.5)(1.38)(457.9) = 316.0 \text{ k}$$

6.2.2.3 Resulting $(\Delta/\lambda)$

The resulting $(\Delta/\lambda)$ from the Load Factor Method is given by Eq. 3.47. The area, $A_g$, and moment of inertia, $I_g$, of the girder are found from the data in Table 2. An average value of $A_g$ and $I_g$ are used,

$$A_g = (80)(0.5) + (2)(18)(2.19) = 118.75 \text{ in}^2$$

$$I_g = (1/12)(0.5)(80)^3 + 2(18)(2.19)(41.1)^2 = 1.54 \times 10^5 \text{ in}^4$$

Substituting into Eq. 3.47 yields,
\[ \frac{\Delta}{\lambda} = \frac{(36)(100)}{2(29000)(6.67)(7)} \left[ (1.61)^2 + \frac{(8)(5)}{4(1.61)}(12.7) \left[ \frac{1}{118.75} + \frac{(82.2)^2}{4(1.54 \times 10^5)} \right] \right] \]

or,
\[ \frac{\Delta}{\lambda} = \frac{1}{183} \]

The ratio of the midspan diagonals strain, \( \epsilon_{md} \), to the yield strain \( \epsilon_y \), is given by Eq. 3.50,
\[ \frac{\epsilon_{md}}{\epsilon_y} = 1 + \frac{1}{4(1.61)^3}(49 - 7 - 5)(12.7) \left[ \frac{1}{118.75} + \frac{(82.2)^2}{4(1.54 \times 10^5)} \right] \]

or,
\[ \frac{\epsilon_{md}}{\epsilon_y} = 1.55 \]

6.2.3 Serviceability Method

If the strength methods result in a larger \( (\Delta/\lambda) \) for midspan fracture than the chosen \( (\Delta/\lambda)_{lim} \) serviceability controls. When serviceability controls, the required area, \( A_{BL} \), of the bottom lateral diagonals is determined from the \( (\Delta/\lambda)_{lim} \) using the Serviceability Method equations (Eq's. 3.30, 3.33 and 3.34). The requirements of the cross bracing and top lateral bracing systems are determined from the Allowable Stress Method equations using the value of \( F_{BL} \) from the Serviceability Method. This is because the Allowable Stress and Serviceability Methods use the same model considering all
of the bottom lateral diagonals.

The required \( A_{BL} \) from the Serviceability Method considers only midspan fracture. This value of \( A_{BL} \) must be compared to the required area for the critical fracture scenario in the Allowable Stress Method (Eq. 3.11). The larger of these two values determines the requirements of the bottom lateral diagonals.

Three different values of \( (\Delta/\lambda)_{\text{lim}} \) are chosen to illustrate how the Serviceability Method is used with the strength methods to determine the requirements of the bottom lateral bracing system. The three values cover the limits of \( (\Delta/\lambda)_{\text{lim}} \) established in Art. 3.4.4.

The resulting \( (\Delta/\lambda) \) for the strength methods were presented in Art. 3.5.1 for the Allowable Stress Method and Art. 3.5.2 for the Load Factor Method. For the 100 ft span bridge with seven panels the strength methods resulted in the following values of \( (\Delta/\lambda) \):

\[
\text{Allowable Stress Method: } \frac{\Delta}{\lambda} = \frac{1}{273} \quad (6.1)
\]
\[
\text{Load Factor Method: } \frac{\Delta}{\lambda} = \frac{1}{183} \quad (6.2)
\]

6.2.3.1 \( (\Delta/\lambda)_{\text{lim}} = 1/100 \)

This is a lower bound of \( (\Delta/\lambda)_{\text{lim}} \) and is about as much deflection as should be tolerated. In this case the maximum slope due to fracture in the end panel from Eq. 3.35 is,

\[
\theta_{cr} = \left[3.6 - (1.6)/7\right] (1/100) = 0.034 \text{ rad} = 1.93^\circ
\]
From Eq's. 6.1 and 6.2, both the Load Factor and Allowable Stress Methods result in less deflection than \((\Delta/\lambda)_{\text{lim}} = 1/100\). Therefore either method may be used to determine the requirements of the bottom lateral bracing. The following required areas, \(A_{BL}\) for midspan fracture have been found,

- **Allowable Stress Method:** \(A_{BL} = 16.0 \text{ in}^2\)
- **Load Factor Method:** \(A_{BL} = 12.7 \text{ in}^2\)

Therefore the smaller \(A_{BL}\) (12.7 in\(^2\)) from the Load Factor Method controls. From Art. 6.2.2.2, the required area for the critical fracture in the first interior panel is 17.5 in\(^2\).

6.2.3.2 \((\Delta/\lambda)_{\text{lim}} = 1/200\)

The maximum slope due to fracture in the end panel is given by Eq. 3.35,

\[ \theta_{\text{cr}} = \left[ 3.6 - \frac{(1.6)}{7} \right] (1/200) = 0.017 \text{ rad} = 0.97^\circ \]

In this case the deflection (1/183) resulting from the Load Factor Method (Eq. 6.2) is greater than the \((\Delta/\lambda)_{\text{lim}} = 1/200\). The deflection (1/273) resulting from the Allowable Stress Method (Eq. 6.1) is less than the \((\Delta/\lambda)_{\text{lim}}\).

Therefore, with \((\Delta/\lambda)_{\text{lim}} = 1/200\), the Allowable Stress Method controls and the required area for midspan fracture is 16.0 in\(^2\). From Art. 6.2.1.2, the required area for the critical fracture scenario was found to be 17.6 in\(^2\).
6.2.3.3 \( \left( \frac{\Delta}{\lambda} \right)_{\text{lim}} = 1/300 \)

This is the upper bound of \( \left( \frac{\Delta}{\lambda} \right)_{\text{lim}} \) and is the most restrictive \( \left( \frac{\Delta}{\lambda} \right)_{\text{lim}} \). The maximum end slope due to fracture in the end panel from Eq. 3.35 is,

\[
\Theta_{\text{cr}} = \left[ 3.6 - (1.6)/7 \right] (1/300) = 0.011 \text{ rad} = 0.6^\circ
\]

In this case the deflections resulting from the Load Factor Method (1/183) and the Allowable Stress Method (1/273) are greater than the \( \left( \frac{\Delta}{\lambda} \right)_{\text{lim}} \). Therefore, serviceability controls.

The required area of bottom lateral diagonal is found from Eq. 3.30. The values of the coefficients \( u_D \) and \( u_L \) are found from Eq's. 3.33 and 3.34,

\[
u_D = \frac{100}{50 + (100)[3 - 0.007 / (1/300)]} = 0.71
\]

\[
u_L = \frac{100}{50 + (100)[3.5 - 0.007 / (1/300)]} = 0.53
\]

Substituting these values of \( u_D \) and \( u_L \) into Eq. 3.30, the required area, \( A_{BL} \), of bottom lateral diagonal is,

\[
A_{BL} = \frac{(1.61)^2 (100)^2}{16(29000)(7)(6.67)^2(1/300)} \frac{[(0.71)(3.58)(100) + 2(0.53)(82.0)]}{[(0.71)(3.58)(100) + 2(0.53)(82.0)]}
\]

or,

\[
A_{BL} = 18.4 \text{ in}^2
\]

This is the required area for midspan fracture. This area must be compared to the required area for the critical fracture scenario.
in the Allowable Stress Method (17.6 in\(^2\)). Since 18.4 in\(^2\) > 17.6 in\(^2\), the area required for the Serviceability Method is also satisfactory for strength considerations with fractures other than at midspan. Therefore, with \((\Delta/l)_{\text{lim}} = 1/300\), the Serviceability Method controls and the required \(A_{BL}\) is 18.4 in\(^2\).

For this case where serviceability controls, the requirements of the cross bracing and top lateral bracing systems are determined from the Allowable Stress Method equations. The force, \(F_{BL}\) is found by multiplying \(A_{BL}\) by the allowable stress, \(f_{\text{all}}\),

\[
F_{BL} = (18.4 \text{ in}^2) (27 \text{ ksi}) = 496.8 \text{ k}
\]

The maximum forces in the cross bracing are given by Eq's. 4.11 and 4.13,

\[
F_{CBH} = (1.35)(0.78)(496.8) = 523.1 \text{ k}
\]

\[
F_{CBD} = (0.65)(0.84)(496.8) = 271.3 \text{ k}
\]

The maximum force, \(F_{TL}\), in a top lateral diagonal is found from Eq. 5.1,

\[
F_{TL} = (0.65)(496.8) = 322.9 \text{ k}
\]

6.3 Additional Examples

The formulas developed in Chapters 3, 4 and 5 are applied to the other five combinations of span length and number of panels in addition to the 100 ft span bridge with seven panels. The data for each of the six bridges is summarized in Table 16.
The results of the required $A_{BL}$ and the forces in the alternate load path for each bridge are summarized in Tables 17 and 18 for the Allowable Stress and Load Factor Methods respectively.

6.3.1 Discussion of Results

Examination of Tables 17 and 18 reveals that the Load Factor Method results in a lower required $A_{BL}$ for all six bridges. The cross bracing and top lateral bracing forces for the 100 ft. spans are very close for both strength methods. For longer spans, the Load Factor Method results in much lower cross bracing and top lateral bracing forces.

Table 19A summarizes the required $A_{BL}$ and resulting $(\Delta/\lambda)$ for midspan fracture for both the strength methods.

6.3.1.1 $(\Delta/\lambda)_{\text{lim}} = 1/100$

Examination of Table 19A shows that all of the resulting values of $(\Delta/\lambda)_{\text{lim}}$ are below 1/100. Therefore serviceability is not a factor if $(\Delta/\lambda)_{\text{lim}}$ is chosen as 1/100. Table 19B shows the required $A_{BL}$ for the critical fracture scenario. Examination of the table shows that the Load Factor Method governs in all six cases with a smaller $A_{BL}$.

6.3.1.2 $(\Delta/\lambda)_{\text{lim}} = 1/200$

All of the deflections are less than 1/200 except for the Load Factor Method with the 100 ft. span with seven panels as shown in Table 19A. In this case the Load Factor Method controls for all of
the bridges except the 100 ft span with seven panels. This bridge is controlled by the Allowable Stress Method because it results in acceptable deflections.

6.3.1.3 \((\Delta/l)_{\text{lim}} = 1/300\)

Table 19A shows that the Load Factor Method results in unacceptable deflections in all six cases. The Allowable Stress Method governs for the 150 ft and 200 ft spans.

Serviceability controls for the 100 ft spans when this restrictive \((\Delta/l)_{\text{lim}}\) is used. The required \(A_{BL}\) from the Serviceability Method is shown in the bottom row of Table 19A. These values of \(A_{BL}\) must be compared to the value of \(A_{BL}\) for the critical fracture scenario from the Allowable Stress Method.

For the 100 ft span bridge with five panels the critical fracture for the Allowable Stress Method controls because 20.7 in\(^2\) > 18.1 in\(^2\) as shown in the table. For the 100 ft span bridge with seven panels the Serviceability Method controls because 18.4 in\(^2\) > 17.6 in\(^2\) as shown in the table.
7. VERIFICATION OF THE ALTERNATE LOAD PATH REQUIREMENTS

The equations developed for the requirements of the alternate load path in Chapters 3, 4 and 5 need to be checked on a three-dimensional bridge model. The model used to develop the equations considered the bottom lateral diagonals to be the only system available to develop the forces released at the fracture. The areas of the cross bracing diagonals and the top lateral bracing were reduced to nearly zero (0.001 in$^2$). The model was then adjusted to prevent any relative movement between the two girders so that the forces in the bottom lateral diagonals could be developed.

7.1 Computer Model

The computer model for the verification study must more closely approximate the real behavior of the bridge. Therefore the moment of inertia of the unfractured girder bottom flange is reduced from practically infinite ($10^6$ in$^4$) to its actual value. The areas of the bottom lateral diagonals, cross bracing horizontals, cross bracing diagonals and top lateral diagonals from the equations developed in Chapters 3, 4 and 5 are inserted into the computer model. This is done for each of the three methods. The resulting forces and deflections are examined to see if the equations developed for the requirements of the alternate load path are satisfactory.
7.2 Allowable Stress Method

The forces which must be carried by the members of the alternate load path are summarized in Table 17. Members are chosen which are capable of carrying these forces without exceeding the allowable stresses. The areas of these members are input into the computer model.

The same assumptions in Art. 6.1 used for the examples are used for the verification study. It is assumed that the steel has a yield stress of 36 ksi and the allowable stresses of the Operating Rating level are used. For the Operating Rating level with steel of 36 ksi yield strength the allowable stresses are,

\[
\text{Tension: } f_{\text{all}} = 0.75 f_y = 27 \text{ ksi} \tag{7.1}
\]

\[
\text{Compression: } f_{\text{all,c}} = 21180 - 0.67(KL/r)^2 \tag{7.2}
\]

7.2.1 Midspan Fracture

The required area, \( A_{BL} \) of bottom lateral bracing and the forces which must be carried by the alternate load path members for midspan fracture are summarized in Table 17A. The members chosen to carry these forces without exceeding the allowable stresses given by Eq's. 7.1 and 7.2 are summarized in Table 20. The areas of these members are input into the computer model and midspan fracture is introduced.

The results of the computer output are summarized in Table 21A. In the table \( f_{t,\text{max}} \) is the maximum tensile stress in a member and \( f_{c,\text{max}} \) is the maximum compressive stress. The allowable compressive
stress, $f_{all,c}$ is found from Eq. 7.2. There is a slight overstress in the bottom lateral tension diagonals at midspan for the 150 and 200 ft spans. The stresses in all the other members of the alternate load path are below their allowable stress as shown in the table. The stresses in the cross bracing diagonals and top lateral diagonals are significantly below $f_{all,c}$ as shown in the table.

7.2.2 Critical Fracture Scenarios

The only members affected by movement of the fracture location is the bottom lateral diagonals. Midspan fracture is the critical fracture scenario for all other members on the alternate load path. The required area, $A_{BL'}$ of the bottom lateral diagonals for fracture in the panel adjacent to midspan are shown in Table 17B. Also, the maximum compression force, $F_{BLC'}$ in a bottom lateral diagonal due to fracture in the first interior panel are shown in the table. Members are chosen to satisfy the required $A_{BL}$ without exceeding the allowable compressive stress.

The areas of these members are input into the computer model. Both critical fracture scenarios are imposed. First a fracture is imposed in the panel adjacent to midspan to see if the maximum tensile stress, $f_{t,\text{max}}'$ in a bottom lateral diagonal exceeds the allowable tensile stress, $f_{all}$. Second, a fracture is imposed in the first interior panel to see if the maximum compressive stress, $f_{c,\text{max}}'$ exceeds the allowable compressive stress, $f_{all,c}$.

The results of the computer output are summarized in Table 21B.
For the case of fracture in the panel adjacent to midspan, there is a very slight overstress in a bottom lateral diagonal in tension for the 200 ft span bridge with 13 panels. It appears that the amplification factor, $\phi = 1.1$, will result in adequate values of $A_{BL}$ for any fracture scenario.

The maximum compressive stress, $f_{c,max}'$ in a bottom lateral diagonal due to fracture in the first interior panel is much less than the allowable compressive stress, as shown in the table.

The maximum compressive stress, $f_{c,max}'$ in the cross bracing horizontals are all below the allowable compressive stress as shown in the table. The stresses in the cross bracing diagonals and top lateral diagonals are not shown in Table 21B because they are significantly below $f_{all,c}$ as was the case for midspan fracture.

7.2.3 Discussion of Results

The Allowable Stress Method equations developed in Chapters 3, 4 and 5 for the requirements of the alternate load path gave satisfactory results in the verification study. The stresses in all of the alternate load path members are below the allowable, except for a slight overstress in the midspan diagonal in the 150 and 200 ft spans.

One interesting outcome of the verification study is the resulting stress level in the cross bracing and top lateral diagonals. The equations developed in Chapters 4 and 5 for the requirements of these members result in stresses significantly below
as shown in Table 21A. The low stresses in these members is
the result of neglecting the cross bracing forces in the derivation
of the equations.

Figure 45(a) shows the forces and reactions acting on the
fractured girder including the forces from the cross bracing. The
downward cross bracing shears in the panels without fracture tend to
reduce the total force, \( \sum F = F_1 + F_2 + F_3 + F_4 \), acting at the
fractured girder bottom flange.

The forces \( F_2, F_3 \) and \( F_4 \) shown in Fig. 45(a) are greatly reduced
by the downward cross bracing shears. The force \( F_1 \) remains
approximately the same because the upward cross bracing shears in the
panel with fracture counteracts the reduction in \( \sum F \) due to the
downward cross bracing forces in the other panels.

Therefore, the bottom lateral diagonal forces in the panel with
fracture are approximately the same whether the cross bracing forces
are included or not. However, the bottom lateral diagonal forces in
the panel with fracture are greatly reduced by the downward cross
bracing shears. This explains why the equations for the midspan
tension diagonals give reasonable results while the equations for the
maximum compressive stress in the end panel diagonal due to fracture
in the first interior panel greatly overestimate the force.

The forces in the bottom lateral bracing diagonals are shown in
Fig. 45(b). The force \( \phi_1 F_{BL} \) is greatly reduced because of the
downward cross bracing shears. The forces from the bottom lateral
diagonals acting on the cross bracing are shown in Fig. 45(c).
force acting on the unfractured girder, \( U \), is greatly reduced because of the reduction in \( \phi_{t1} F_{BL} \).

The assumed force distribution between the cross bracing members was given by Eq's. 4.1 and 4.2,

\[
F_{CBH} = k_d (U+F)/2
\]
\[
F_{CBD} = k_d (U-F)/2
\]

The cross bracing horizontal force, \( F_{CBH} \), is a function of \( (U+F) \). The cross bracing diagonal force, \( F_{CBD} \), is a function of \( (U-F) \).

The values of \( (U+F) \) and \( (U-F) \) are compared for the two different computer models used. The simplified model described in Art. 3.1, which was used in the development of all the equations, and the verification study model described in Art. 7.1 which included the cross bracing and top lateral bracing. The results are summarized in Table 22.

As expected, there is a great reduction in \( U \) from the simplified model to the verification study model. There is also a slight increase in \( F \) as shown in the table.

From Table 22, \( (U+F) \) is decreased slightly (4-16%) while \( (U-F) \) is reduced significantly (37-43%). This explains why the equation for the force in the cross bracing horizontal (Eq. 4.24) gives good results while the equation for the force in the cross bracing diagonals (Eq. 4.20) is very conservative.
7.3 Load Factor Method

Verification of the Load Factor Method is performed on the 150 ft span bridge with seven panels of lateral bracing. For verification of the Load Factor Method, the bottom lateral diagonals in compression are assumed to be buckled. Therefore only the tension diagonals are included in the computer model. For the tension diagonals assumed to be yielded, the yield force is applied in place of the member.

7.3.1 Midspan Fracture

The required area, $A_{BL}$, of bottom lateral bracing and the forces which must be carried by the alternate load path members for the case of midspan fracture are summarized in Table 18A. Bottom lateral diagonal members are chosen to satisfy the required $A_{BL}$ (14.8 in$^2$). Cross bracing and top lateral diagonal members are chosen which carry the forces shown in Table 18A without exceeding the maximum axial compression stress,

$$f_a < (0.85)f_y \left[ 1 - \frac{(KL/r)^2 f_y}{4\pi^2 E} \right]$$  \hspace{1cm} (7.3)

The members chosen are shown in Table 23A.

The areas of these members are input into the computer model and midspan fracture is introduced. It is assumed that only the midspan tension diagonals are yielded. Therefore, the yield force, $F_y$, is introduced in place of the midspan diagonals at these locations,
The results of the computer output are shown in Fig. 46. All of the bottom lateral forces shown in Fig. 46(a) are less than the yield force, $F_Y = 536.4$ k. The assumption that the midspan tension diagonals yielded must now be checked. The deflections of the bottom lateral system at the midspan panel are shown in Fig. 46(b).

From Fig. 46(b) the elongation, $e_{md}$, of the midspan tension diagonals is,

$$e_{md} = (0.84 + 0.11) \frac{21.43}{27.99} + (0.99 - 1.17) \frac{18}{27.99} = 0.612 \text{ in}$$

Therefore the strain, $\varepsilon_{md}$, in the midspan diagonals is,

$$\varepsilon_{md} = \frac{(0.612)}{(27.99)(12)} = 1.82 \times 10^{-3}$$

The strain, $\varepsilon_{md}$, in the midspan diagonal should be greater than the yield strain, $\varepsilon_Y = (f_Y)/E$, if the assumption that the midspan diagonals are yielded is correct.

$$\varepsilon_Y = \frac{f_Y}{E} = \frac{36}{29000} = 1.24 \times 10^{-3}$$

$$\varepsilon_{md} = 1.82 \times 10^{-3} > \varepsilon_Y = 1.24 \times 10^{-3}$$

Therefore the assumption is correct and the Load Factor Method is
verified for the requirements of the bottom lateral diagonals.

The forces in the cross bracing and top lateral bracing are shown in Fig's. 46(c) and (d). The maximum forces in all these members are all below the forces shown in Fig. 18A from the Load Factor Method equations for midspan fracture. The results are summarized in Table 23B.

7.3.2 Critical Fracture Scenario

The critical fracture scenario for the Load Factor Method is fracture in the first interior panel. The required area, $A_{BL}$, of bottom lateral diagonal and the forces which must be carried by the members of the alternate load path are summarized in Table 18B. Bottom lateral diagonal members are chosen to satisfy the required $A_{BL}$ (20.4 in$^2$). Cross bracing and top lateral bracing members are chosen which carry the forces shown in Table 18B without exceeding the maximum axial compression stress given by Eq. 7.3. The members chosen are summarized in Table 24A.

The areas of these members are input into the computer model and fracture is introduced in the first interior panel. It is first assumed that the tension diagonal in the first interior panel has yielded and that the rest of the bottom lateral tension diagonals have not reached yield. The computer output proved this assumption to be wrong as the strain, $\varepsilon_{md}$, of the diagonal in the first interior panel was less than the yield strain.

It is now assumed that none of the bottom lateral tension
diagonals are yielded. The results of the computer output are shown in Fig. 47. The yield force, $F_y$, for the bottom lateral diagonal is,

$$F_y = (A_{BL})(f_y) = (20.8)(36) = 748.8 \text{ k}$$

All of the bottom lateral diagonal forces shown in Fig. 47(a) are less than $F_y = 748.8 \text{ k}$. Therefore, none of the bottom lateral diagonals have reached yield.

The forces in the cross bracing and top lateral bracing are shown in Fig's. 47(b) and (c). The maximum forces in all these members are all below the forces shown in Table 18B from the Load Factor Method equations for the critical fracture scenario. The results are summarized in Table 24B.

7.3.3 Discussion of Results

The model used in the development of the Load Factor Method equations assumes that all of the compression diagonals are buckled and the tension diagonals are yielded. The verification study on the 150 ft span bridge with seven panels showed that the equations developed for the requirements of the alternate load path in Chapters 3, 4 and 5 are conservative.

In the verification study for midspan fracture, only the midspan tension diagonals reach the yield point. For the case of fracture in the first interior panel, none of the bottom lateral diagonals reached yield. Therefore, the equations for the Load Factor Method result in conservative values for the requirements of the alternate
7.4 Serviceability Method

The deflections from the Allowable Stress Method computer studies are examined to determine the effect of the two different computer models on the magnitude of deflections. Table 25 summarizes the fractured girder deflection at midspan for the case of midspan fracture. The first row shows the deflection resulting from the simplified model (Art. 3.1) used in the development of the alternate load path requirements in Chapters 3, 4 and 5. The second row shows the deflection resulting from the computer model used in the verification study (Art. 7.1).

Examination of Table 25 reveals that the deflections from the verification study are extremely close to the values from the simplified computer model. The deflections in the verification study are only 4-8% higher. Therefore, this verifies the equations for serviceability developed in Art. 3.4.
8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

This paper presents the concept of Redundancy Rating of two-girder steel bridges. Three different Redundancy Rating methods are presented. The strength methods are the Allowable Stress Method and the Load Factor Method. The third method is the Serviceability Method which is based on a limiting deflection-to-span-length ratio. The requirements of the alternate load path are determined using each of the three methods.

The bridge configuration studied in this research is based on the bridge in Ref. 10 as shown in Fig. 8. Six combinations of span length and number of panels as shown in Table 2 are investigated. Equations for the requirements of the alternate load path members are developed by each of the Redundancy Rating methods.

It is found that the Load Factor Method results in the lowest required area, \( A_{BL} \), of bottom lateral diagonals for all cases. The Allowable Stress Method results in a higher required \( A_{BL} \) and less deflection. Therefore, the method which controls the Redundancy Rating of a given bridge depends on the limiting deflection-to-span-length ratio, \( (\Delta/l)_{\text{lim}} \).

When a large amount of deflection is tolerated, \( (\Delta/l)_{\text{lim}} = 1/100 \), the Load Factor Method governs for each combination of span length and number of panels. When less deflection is allowed, \( (\Delta/l)_{\text{lim}} = 1/200 \), the Load Factor Method still controls for five of the six
combinations of $\lambda$ and $n$. The Allowable Stress Method controls for the 100 ft span bridge with seven panels because the Load Factor Method results in deflections greater than $(\Delta/\lambda)_{\text{lim}} = 1/200$.

When a very restrictive $(\Delta/\lambda)_{\text{lim}}$ of 1/300 is chosen, the Load Factor Method results in excessive deflections for all cases. The Allowable Stress Method controls for the 150 and 200 ft spans. The Serviceability Method becomes a factor for the 100 ft spans when this restrictive $(\Delta/\lambda)_{\text{lim}}$ is used.

For the 100 ft span bridge with five panels, the critical fracture for the Allowable Stress Method controls. For the 100 ft span bridge with seven panels, the Serviceability Method controls. Therefore, the Serviceability Method controls for only one bridge, even with the very strict limitation of $(\Delta/\lambda)_{\text{lim}} = 1/300$.

It is concluded that the Serviceability Method is not a factor for the bridges studied in this research. The requirements of the alternate load path can be found by each of the strength methods. If the resulting $(\Delta/\lambda)$ from the Load Factor Method is satisfactory to the bridge engineer, the Load Factor Method controls. If the Load Factor Method results in a $(\Delta/\lambda)$ which is more than the bridge engineer can tolerate, the Allowable Stress Method determines the requirements of the alternate load path members.
8.2 Recommendations

This report presented the concept of Redundancy Rating of two-girder steel bridges. Only one bridge configuration was studied in this research. The study was limited to simple span, noncomposite two-girder bridges with bottom lateral bracing, cross bracing, and top lateral bracing. The bottom and top lateral bracing are assumed to be X-shaped. Equations are developed for the requirements of these members for the practical range of existing two-girder bridges with this configuration.

More research is needed to develop the requirements of all two-girder bridges with an alternate load path consisting of the bottom lateral bracing, cross bracing, and top lateral bracing. For instance, a bridge with K-shaped bracing.

Research is needed to develop the requirements of two-girder bridges with different configurations. For example, two-girder bridges with a composite deck, cross frames or diaphragms, etc. The alternate load path available for these bridges needs to be identified. The requirements of this alternate load path must then be determined.

A Redundancy Rating procedure must also be established for continuous two-girder bridges. The alternate load path in this case can make use of the negative moment capacity and stiffness of the fractured girder (6).

Bridges, such as two-girder through bridges, which do not have the bracing systems necessary for a reliable alternate load path must
also be examined. One possible alternative would be to investigate the use of cable support techniques to provide the alternate load path.

Extension of research is needed to consider two-girder steel bridges which are horizontally curved, straight and curved articulated and straight, curved and continuous skewed bridges.

Finally, more research is needed to establish appropriate loading conditions, load factors, allowable stresses and impact factors for Redundancy Rating. Suitable values are suggested in Art. 6.1 for the examples. More research is needed to determine what these values should be.
<table>
<thead>
<tr>
<th>Type</th>
<th>Designation</th>
<th>Shape</th>
<th>Degrees of Freedom</th>
<th>Bridge Components Modeled</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-D Truss</td>
<td>SPACE TRUSS</td>
<td></td>
<td>3 translations at each node</td>
<td>Cross bracing: diagonals</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>horizontals</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>bottom laterals</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>top laterals</td>
</tr>
<tr>
<td>3-D Beam</td>
<td>SPACE FRAME</td>
<td></td>
<td>3 translations and</td>
<td>girder flanges</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3 rotations at each node</td>
<td>girder stiffeners</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>deck link</td>
</tr>
<tr>
<td>Plane Stress</td>
<td>PSHQ</td>
<td></td>
<td>2 translations at each node</td>
<td>girder web</td>
</tr>
<tr>
<td>Flat Shell</td>
<td>SBHQ6</td>
<td></td>
<td>3 translations and</td>
<td>deck</td>
</tr>
<tr>
<td>(Plate)</td>
<td></td>
<td></td>
<td>3 rotations at each node</td>
<td></td>
</tr>
</tbody>
</table>

Table 1 Summary of Finite Elements Used
### Table 2: Details of Computer Study Bridges

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5 n=7</td>
<td>n=7 n=9</td>
<td>n=9 n=13</td>
</tr>
<tr>
<td>Girder Depth, (d) (in)</td>
<td>80</td>
<td>120</td>
<td>160</td>
</tr>
<tr>
<td>Flanges</td>
<td>midspan</td>
<td>quarterspan</td>
<td>midspan</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Web</td>
<td>80&quot; x 0.5&quot;</td>
<td>120&quot; x 0.75&quot;</td>
<td>160&quot; x 1.0&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>(A_{BL}) (in(^2))</td>
<td>9.36 8.00 9.74 8.54 9.95 8.54</td>
<td>18.72 15.99 19.47 17.07 19.89 17.07</td>
<td>37.44 31.98 38.94 34.14 39.78 34.14</td>
</tr>
</tbody>
</table>
Table 3 Comparison of Required Bottom Lateral Bracing Areas, $A_{NL}$, for RRF = 1 (Eq. 3.7)

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>$f_y=30$ ksi ($f_{all}=22.5$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>*1 Computer Analysis</td>
<td>22.5</td>
<td>19.8</td>
<td>32.6</td>
</tr>
<tr>
<td>Eq's 3.9 and 3.10</td>
<td>24.5</td>
<td>20.9</td>
<td>33.6</td>
</tr>
<tr>
<td>$f_y=36$ ksi ($f_{all}=27.0$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>*2 Computer Analysis</td>
<td>16.8</td>
<td>14.7</td>
<td>23.2</td>
</tr>
<tr>
<td>Eq's 3.9 and 3.10</td>
<td>18.2</td>
<td>15.6</td>
<td>24.6</td>
</tr>
<tr>
<td>$f_y=50$ ksi ($f_{all}=37.5$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>*3 Computer Analysis</td>
<td>10.5</td>
<td>9.1</td>
<td>13.6</td>
</tr>
<tr>
<td>Eq's 3.9 and 3.10</td>
<td>11.0</td>
<td>9.4</td>
<td>14.3</td>
</tr>
<tr>
<td>$f_y=60$ ksi ($f_{all}=45.0$)</td>
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<td></td>
</tr>
<tr>
<td>*4 Computer Analysis</td>
<td>8.2</td>
<td>7.0</td>
<td>10.2</td>
</tr>
<tr>
<td>Eq's 3.9 and 3.10</td>
<td>8.4</td>
<td>7.2</td>
<td>10.7</td>
</tr>
</tbody>
</table>

Allowable Stresses for Operating Rating level (Ref. 4):

*1 For steel unknown, built in 1905 to 1936

*2 For steel unknown, built after 1963

*3 For steel A94 (1-1/8" and under), A242, A440 and A441 (3/4" and under), and A588 (4" and under)

*4 For steel A572 (1" max.)
Table 4  Maximum Stresses in a Bottom Lateral Diagonal for Midspan Fracture

4A. Maximum Tensile Stress (ksi)

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>$f_{all} = 27$ ksi</td>
<td>25.2</td>
<td>25.8</td>
<td>25.9</td>
</tr>
</tbody>
</table>

4B. Ratio of Maximum Compressive Stress to Maximum Tensile Stress ($f_c/f_t$)

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>$\frac{f_c}{f_t}$</td>
<td>0.27</td>
<td>0.41</td>
<td>0.17</td>
</tr>
</tbody>
</table>
Table 5: Maximum Stresses in a Bottom Lateral Diagonal for each Fracture Scenario

5A. Maximum Tensile Stress (ksi)

<table>
<thead>
<tr>
<th>Fracture Scenario</th>
<th>Bridge</th>
<th>100 ft, n=5</th>
<th>100 ft, n=7</th>
<th>200 ft, n=9</th>
</tr>
</thead>
<tbody>
<tr>
<td>midspan</td>
<td>25.2</td>
<td>25.8</td>
<td>26.6</td>
<td></td>
</tr>
<tr>
<td>one panel from midspan</td>
<td>25.6</td>
<td>26.7</td>
<td>26.0</td>
<td></td>
</tr>
<tr>
<td>two panels from midspan</td>
<td>24.9</td>
<td>26.1</td>
<td>23.5</td>
<td></td>
</tr>
<tr>
<td>three panels from midspan</td>
<td></td>
<td>25.6</td>
<td>19.4</td>
<td></td>
</tr>
<tr>
<td>four panels from midspan</td>
<td></td>
<td></td>
<td>15.7</td>
<td></td>
</tr>
</tbody>
</table>

5B. Ratio of Maximum Compressive Stress to Maximum Tensile Stress for Midspan Fracture (f_c/f_t)

<table>
<thead>
<tr>
<th>Fracture Scenario</th>
<th>Bridge</th>
<th>100 ft, n=5</th>
<th>100 ft, n=7</th>
<th>200 ft, n=9</th>
</tr>
</thead>
<tbody>
<tr>
<td>midspan</td>
<td>0.27</td>
<td>0.41</td>
<td>0.12</td>
<td></td>
</tr>
<tr>
<td>one panel from midspan</td>
<td>0.51</td>
<td>0.50</td>
<td>0.16</td>
<td></td>
</tr>
<tr>
<td>two panels from midspan</td>
<td></td>
<td>0.68</td>
<td>0.19</td>
<td></td>
</tr>
<tr>
<td>three panels from midspan</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 6 Effect of $A_{DL}$ on the Maximum Tensile Stress for Different Fracture Scenarios

<table>
<thead>
<tr>
<th>$A_{DL}$</th>
<th>Lower Bound Area (12.0 in$^2$)</th>
<th>Area for RRF=1.0 (16.0 in$^2$)</th>
<th>Upper Bound Area (20.0 in$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Midspan Fracture</td>
<td>31.0</td>
<td>25.8</td>
<td>22.4</td>
</tr>
<tr>
<td>Fracture in panel adjacent to midspan</td>
<td>33.0 (+6.3%)</td>
<td>26.7 (+3.5%)</td>
<td>22.8 (+1.8%)</td>
</tr>
</tbody>
</table>

Table 7 Ratio of Maximum Compressive Stress for Fracture in the First Interior Panel to Maximum Tensile Stress for Midspan Fracture ($f_c/f_t$)

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=9</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>1.345</td>
<td>1.609</td>
<td>1.306</td>
</tr>
<tr>
<td>$\frac{f_c}{f_t}$</td>
<td>0.51</td>
<td>0.68</td>
<td>0.40</td>
</tr>
</tbody>
</table>
Table 8 Amplification Factor, \( \phi \), to take into Account the Location of Fracture for the Practical Range of \( n \)

<table>
<thead>
<tr>
<th>( n )</th>
<th>0 (Midspan Fracture)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>n = 5</td>
<td>1.0</td>
<td>1.26</td>
<td>1.08</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>n = 7</td>
<td>1.0</td>
<td>1.22</td>
<td>1.35</td>
<td>1.06</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>n = 9</td>
<td>1.0</td>
<td>1.19</td>
<td>1.34</td>
<td>1.39</td>
<td>1.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>n = 11</td>
<td>1.0</td>
<td>1.16</td>
<td>1.30</td>
<td>1.40</td>
<td>1.41</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>n = 13</td>
<td>1.0</td>
<td>1.14</td>
<td>1.27</td>
<td>1.38</td>
<td>1.45</td>
<td>1.43</td>
<td>1.04</td>
</tr>
<tr>
<td>n = 15</td>
<td>1.0</td>
<td>1.12</td>
<td>1.24</td>
<td>1.34</td>
<td>1.43</td>
<td>1.48</td>
<td>1.44</td>
</tr>
</tbody>
</table>
Table 9  Values of $u_D$ and $u_L$ for Different Values of $(\delta/\lambda)_{lim}$

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>$u_D$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$(\delta/\lambda)_{lim} = 1/200$</td>
<td>0.46</td>
<td>0.41</td>
<td>0.33</td>
</tr>
<tr>
<td>$(\delta/\lambda)_{lim} = 1/300$</td>
<td>0.55</td>
<td>0.64</td>
<td>0.49</td>
</tr>
<tr>
<td>$u_L$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$(\delta/\lambda)_{lim} = 1/200$</td>
<td>0.38</td>
<td>0.33</td>
<td>0.27</td>
</tr>
<tr>
<td>$(\delta/\lambda)_{lim} = 1/300$</td>
<td>0.47</td>
<td>0.46</td>
<td>0.35</td>
</tr>
</tbody>
</table>
Table 10 Deflection Data for Critical Fracture in the End Panel for Serviceability Method

<table>
<thead>
<tr>
<th>λ (ft)</th>
<th>(Δ/λ)_lim</th>
<th>n</th>
<th>(Δ/λ)_lim</th>
<th>n</th>
<th>(Δ/λ)_lim</th>
<th>n</th>
<th>Midspan Fracture</th>
<th>End Panel Fracture</th>
<th>Midspan Fracture</th>
<th>End Panel Fracture</th>
<th>Midspan Fracture</th>
<th>End Panel Fracture</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1/100</td>
<td>5</td>
<td>1/200</td>
<td>9</td>
<td>1/300</td>
<td>9</td>
<td>11.41&quot;</td>
<td>3.92&quot;</td>
<td>5.79&quot;</td>
<td>1.89&quot;</td>
<td>3.80&quot;</td>
<td>1.19&quot;</td>
</tr>
<tr>
<td></td>
<td>1/105</td>
<td></td>
<td>1/207</td>
<td></td>
<td>1/316</td>
<td></td>
<td>1.0</td>
<td>0.34</td>
<td>1.0</td>
<td>0.33</td>
<td>1.0</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td></td>
<td>1.72</td>
<td></td>
<td>1.63</td>
<td></td>
<td>1.0</td>
<td>1.72</td>
<td>1.0</td>
<td>1.63</td>
<td>1.0</td>
<td>1.57</td>
</tr>
<tr>
<td>150</td>
<td>1/115</td>
<td>9</td>
<td>1/218</td>
<td>9</td>
<td>1/316</td>
<td>9</td>
<td>15.59&quot;</td>
<td>3.06&quot;</td>
<td>8.27&quot;</td>
<td>1.53&quot;</td>
<td>5.69&quot;</td>
<td>1.00&quot;</td>
</tr>
<tr>
<td></td>
<td>1/115</td>
<td></td>
<td>1/218</td>
<td></td>
<td>1/316</td>
<td></td>
<td>1.0</td>
<td>0.20</td>
<td>1.0</td>
<td>0.19</td>
<td>1.0</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td></td>
<td>1.77</td>
<td></td>
<td>1.67</td>
<td></td>
<td>1.0</td>
<td>1.77</td>
<td>1.0</td>
<td>1.67</td>
<td>1.0</td>
<td>1.58</td>
</tr>
<tr>
<td>200</td>
<td>1/121</td>
<td>9</td>
<td>1/223</td>
<td>9</td>
<td>1/320</td>
<td>9</td>
<td>19.90&quot;</td>
<td>2.74&quot;</td>
<td>10.74&quot;</td>
<td>1.40&quot;</td>
<td>7.51&quot;</td>
<td>0.94&quot;</td>
</tr>
<tr>
<td></td>
<td>1/121</td>
<td></td>
<td>1/223</td>
<td></td>
<td>1/320</td>
<td></td>
<td>1.0</td>
<td>0.14</td>
<td>1.0</td>
<td>0.13</td>
<td>1.0</td>
<td>0.13</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td></td>
<td>1.79</td>
<td></td>
<td>1.69</td>
<td></td>
<td>1.0</td>
<td>1.79</td>
<td>1.0</td>
<td>1.69</td>
<td>1.0</td>
<td>1.63</td>
</tr>
</tbody>
</table>
Table 11 Resulting (4/%) for the Allowable Stress Method

<table>
<thead>
<tr>
<th></th>
<th>Equation 3.37</th>
<th>Trial and Error Procedure</th>
<th>Computer</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 ft., n=5</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>275</td>
<td>305</td>
<td>321</td>
</tr>
<tr>
<td>100 ft., n=7</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>273</td>
<td>282</td>
<td>301</td>
</tr>
<tr>
<td>150 ft., n=7</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>340</td>
<td>364</td>
<td>390</td>
</tr>
<tr>
<td>150 ft., n=9</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>341</td>
<td>353</td>
<td>374</td>
</tr>
<tr>
<td>200 ft., n=9</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>388</td>
<td>393</td>
<td>428</td>
</tr>
<tr>
<td>200 ft., n=13</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>388</td>
<td>381</td>
<td>406</td>
</tr>
</tbody>
</table>
Table 12 Resulting ($\Delta/\lambda$) and ($\epsilon_{md}/\epsilon_y$) for the Load Factor Method

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>$n=5$</td>
<td>$n=7$</td>
<td>$n=9$</td>
</tr>
<tr>
<td>$\frac{\Delta}{\lambda}$ (Eq. 3.47)</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>$\frac{\epsilon_{md}}{\epsilon_y}$ (Eq. 3.50)</td>
<td>1.42</td>
<td>1.55</td>
<td>1.77</td>
</tr>
</tbody>
</table>
Table 13 Comparison of Resulting ($\Delta/\chi$) for Allowable Stress and Load Factor Methods

<table>
<thead>
<tr>
<th></th>
<th>Allowable Stress Method (Eq. 3.37)</th>
<th>Load Factor Method (Eq. 3.47)</th>
<th>Eq. 3.47 Eq. 3.37</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 ft., $n=5$</td>
<td>$\frac{1}{275}$</td>
<td>$\frac{1}{197}$</td>
<td>1.40</td>
</tr>
<tr>
<td>100 ft., $n=7$</td>
<td>$\frac{1}{273}$</td>
<td>$\frac{1}{183}$</td>
<td>1.50</td>
</tr>
<tr>
<td>150 ft., $n=7$</td>
<td>$\frac{1}{340}$</td>
<td>$\frac{1}{240}$</td>
<td>1.42</td>
</tr>
<tr>
<td>150 ft., $n=9$</td>
<td>$\frac{1}{341}$</td>
<td>$\frac{1}{232}$</td>
<td>1.47</td>
</tr>
<tr>
<td>200 ft., $n=9$</td>
<td>$\frac{1}{388}$</td>
<td>$\frac{1}{275}$</td>
<td>1.41</td>
</tr>
<tr>
<td>200 ft., $n=13$</td>
<td>$\frac{1}{388}$</td>
<td>$\frac{1}{264}$</td>
<td>1.47</td>
</tr>
</tbody>
</table>
Table 14 Variation of $\phi_{t1}$, $(\phi_{t1} - \phi_{c1})$ and $(\phi_{t1} + \phi_{c1})$

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100 ft</td>
<td></td>
<td></td>
<td>\n</td>
<td>Number of panels</td>
</tr>
<tr>
<td></td>
<td>Dead Load</td>
<td>RRF &gt; 1.0</td>
<td>0.70</td>
<td>0.72</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF = 1.0</td>
<td>0.73</td>
<td>0.76</td>
<td>0.73</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF &lt; 1.0</td>
<td>0.77</td>
<td>0.74</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>Live Load</td>
<td>RRF &gt; 1.0</td>
<td>0.82</td>
<td>0.83</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF = 1.0</td>
<td>0.85</td>
<td>0.86</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF &lt; 1.0</td>
<td>0.86</td>
<td>0.87</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td>Dead Load</td>
<td>RRF &gt; 1.0</td>
<td>0.75</td>
<td>0.59</td>
<td>0.74</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF = 1.0</td>
<td>0.55</td>
<td>0.42</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF &lt; 1.0</td>
<td>0.54</td>
<td>0.45</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>Live Load</td>
<td>RRF &gt; 1.0</td>
<td>0.43</td>
<td>0.38</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF = 1.0</td>
<td>0.28</td>
<td>0.23</td>
<td>0.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF &lt; 1.0</td>
<td>0.31</td>
<td>0.28</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>Dead Load</td>
<td>RRF &gt; 1.0</td>
<td>0.65</td>
<td>0.85</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF = 1.0</td>
<td>0.91</td>
<td>1.10</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF &lt; 1.0</td>
<td>1.20</td>
<td>0.94</td>
<td>1.07</td>
</tr>
<tr>
<td></td>
<td>Live Load</td>
<td>RRF &gt; 1.0</td>
<td>1.31</td>
<td>1.28</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF = 1.0</td>
<td>1.42</td>
<td>1.49</td>
<td>1.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RRF &lt; 1.0</td>
<td>1.60</td>
<td>1.37</td>
<td>1.44</td>
</tr>
</tbody>
</table>
Table 15 Limits of \((\bar{\phi}_{\text{max}} + 0.5 \phi_t)\) and \((\bar{\phi}_{\text{max}} + \phi_t)\)

<table>
<thead>
<tr>
<th>n</th>
<th>5</th>
<th>7</th>
<th>9</th>
<th>11</th>
<th>13</th>
<th>15</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\bar{\phi}_{\text{max}})</td>
<td>1.26</td>
<td>1.35</td>
<td>1.39</td>
<td>1.41</td>
<td>1.45</td>
<td>1.48</td>
</tr>
<tr>
<td>(\phi_t)</td>
<td>0.63</td>
<td>0.45</td>
<td>0.35</td>
<td>0.28</td>
<td>0.26</td>
<td>0.22</td>
</tr>
<tr>
<td>(\bar{\phi}_{\text{max}} + 0.5\phi_t)</td>
<td>1.58</td>
<td>1.58</td>
<td>1.57</td>
<td>1.55</td>
<td>1.58</td>
<td>1.59</td>
</tr>
<tr>
<td>(\bar{\phi}_{\text{max}} + \phi_t)</td>
<td>1.89</td>
<td>1.80</td>
<td>1.74</td>
<td>1.69</td>
<td>1.71</td>
<td>1.70</td>
</tr>
</tbody>
</table>
Table 16  Example Data

A. Data Common For all Three Methods

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>1.35</td>
<td>1.61</td>
<td>1.31</td>
</tr>
<tr>
<td>( k_H )</td>
<td>0.67</td>
<td>0.78</td>
<td>0.64</td>
</tr>
<tr>
<td>( k_D )</td>
<td>0.71</td>
<td>0.84</td>
<td>0.74</td>
</tr>
<tr>
<td>( w ) (k/ft)</td>
<td>3.58</td>
<td>3.58</td>
<td>3.88</td>
</tr>
<tr>
<td>( \theta (L-I) ) (k)</td>
<td>82.0</td>
<td>82.0</td>
<td>86.8</td>
</tr>
<tr>
<td>( d = \sqrt{v_0/15} ) (ft)</td>
<td>6.67</td>
<td>6.67</td>
<td>10.0</td>
</tr>
</tbody>
</table>

B. Additional Data

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>( v_D )</td>
<td>2.13</td>
<td>2.8</td>
<td>3.47</td>
</tr>
<tr>
<td>( v_L )</td>
<td>1.47</td>
<td>1.8</td>
<td>2.13</td>
</tr>
<tr>
<td>( A_f ) (in(^2))</td>
<td>51.4</td>
<td>79.3</td>
<td>113.6</td>
</tr>
<tr>
<td>( A_g ) (in(^2))</td>
<td>118.75</td>
<td>194.5</td>
<td>291.3</td>
</tr>
<tr>
<td>( I_g ) (in(^4) x 10(^5))</td>
<td>12.1</td>
<td>5.0</td>
<td>1.54</td>
</tr>
</tbody>
</table>
Table 17  Required $A_{BL}$ and Forces to be Carried by the Alternate Load Path Members (Allowable Stress Method, $f_{all} = 27$ ksi)

A. Midspan Fracture

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=9</td>
</tr>
<tr>
<td>$A_{BL}$ (in$^2$)</td>
<td>18.8</td>
<td>16.0</td>
<td>25.2</td>
</tr>
<tr>
<td>$F_{BL}$ (k)</td>
<td>507.8</td>
<td>432.6</td>
<td>681.5</td>
</tr>
<tr>
<td>$F_{CCH}$ (k)</td>
<td>459.3</td>
<td>455.5</td>
<td>588.8</td>
</tr>
<tr>
<td>$F_{CRD}$ (k)</td>
<td>234.3</td>
<td>236.2</td>
<td>327.8</td>
</tr>
<tr>
<td>$F_{TL}$ (k)</td>
<td>330.1</td>
<td>281.2</td>
<td>443.0</td>
</tr>
<tr>
<td>$(\Delta/l)$</td>
<td>1/275</td>
<td>1/273</td>
<td>1/340</td>
</tr>
</tbody>
</table>

B. Fracture Scenario

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=9</td>
</tr>
<tr>
<td>$A_{BL}$ (in$^2$)</td>
<td>20.7</td>
<td>17.6</td>
<td>27.7</td>
</tr>
<tr>
<td>$F_{BLC}$ (k)</td>
<td>299.6</td>
<td>307.1</td>
<td>327.1</td>
</tr>
</tbody>
</table>
Table 18  Required $A_{BL}$ and Forces to be Carried by the Alternate Load Path Members (Load Factor Method, $f_y = 36$ ksi)

A. Midspan Fracture

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>$A_{BL}$ (in$^2$)</td>
<td>14.2</td>
<td>12.7</td>
<td>14.8</td>
</tr>
<tr>
<td>$F_{BL}$ (k)</td>
<td>511.9</td>
<td>457.9</td>
<td>531.7</td>
</tr>
<tr>
<td>$F_{CHE}$ (k)</td>
<td>514.5</td>
<td>535.7</td>
<td>510.4</td>
</tr>
<tr>
<td>$F_{CDD}$ (k)</td>
<td>181.7</td>
<td>192.3</td>
<td>196.7</td>
</tr>
<tr>
<td>$F_{TL}$ (k)</td>
<td>256.0</td>
<td>229.0</td>
<td>265.9</td>
</tr>
<tr>
<td>$(\Delta/\lambda)$</td>
<td>1/197</td>
<td>1/183</td>
<td>1/240</td>
</tr>
</tbody>
</table>

B. Fracture in the First Interior Panel

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>$\delta_{max}$</td>
<td>1.29</td>
<td>1.38</td>
<td>1.38</td>
</tr>
<tr>
<td>$A_{BL}$ (in$^2$)</td>
<td>18.3</td>
<td>17.5</td>
<td>20.4</td>
</tr>
<tr>
<td>$F_{CHE}$ (k)</td>
<td>548.8</td>
<td>571.5</td>
<td>544.5</td>
</tr>
<tr>
<td>$F_{CDD}$ (k)</td>
<td>234.4</td>
<td>265.4</td>
<td>271.5</td>
</tr>
<tr>
<td>$F_{TL}$ (k)</td>
<td>330.2</td>
<td>316.0</td>
<td>366.9</td>
</tr>
</tbody>
</table>
Table 19 Required $A_{BL}$ and the Resulting $(\Delta/\lambda)$ for each Method

A. Midspan Fracture

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>Allowable Stress Method</td>
<td>$A_{BL}$ (in²)</td>
<td>18.8</td>
<td>16.0</td>
</tr>
<tr>
<td></td>
<td>$\Delta$</td>
<td>1/275</td>
<td>1/273</td>
</tr>
<tr>
<td>Load Factor Method</td>
<td>$A_{BL}$ (in²)</td>
<td>14.2</td>
<td>12.7</td>
</tr>
<tr>
<td></td>
<td>$\Delta$</td>
<td>1/197</td>
<td>1/183</td>
</tr>
<tr>
<td>Serviceability Method</td>
<td>$A_{BL}$</td>
<td>18.1</td>
<td>18.4</td>
</tr>
<tr>
<td>$(\Delta/\lambda)_{lim} = 1/300$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

B. Critical Fracture Scenario ($A_{BL}$)

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>Allowable Stress Method</td>
<td>$A_{BL}$ (in²)</td>
<td>20.7</td>
<td>17.6</td>
</tr>
<tr>
<td>Load Factor Method</td>
<td>$A_{BL}$ (in²)</td>
<td>18.3</td>
<td>17.5</td>
</tr>
<tr>
<td></td>
<td>100 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------------</td>
<td>--------</td>
<td>------------</td>
<td>------------</td>
</tr>
<tr>
<td><strong>n</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Bottom</strong></td>
<td>6.5x65</td>
<td>7x54.5</td>
<td>15x86.5</td>
</tr>
<tr>
<td>Lateral</td>
<td>A=19.2</td>
<td>A=16.0</td>
<td>A=25.4</td>
</tr>
<tr>
<td>Diagonal</td>
<td>r=2.39</td>
<td>r=1.68</td>
<td>r=3.43</td>
</tr>
<tr>
<td><strong>Cross Bracing</strong></td>
<td>13.5x80.5</td>
<td>13.5x80.5</td>
<td>15x105.5</td>
</tr>
<tr>
<td>Horizontal</td>
<td>A=23.7</td>
<td>A=23.7</td>
<td>A=31.0</td>
</tr>
<tr>
<td></td>
<td>r=3.24</td>
<td>r=3.24</td>
<td>r=3.49</td>
</tr>
<tr>
<td><strong>Cross Bracing</strong></td>
<td>13.5x42</td>
<td>13.5x42</td>
<td>12x58.5</td>
</tr>
<tr>
<td>Horizontal</td>
<td>A=12.4</td>
<td>A=12.4</td>
<td>A=17.2</td>
</tr>
<tr>
<td></td>
<td>r=2.07</td>
<td>r=2.07</td>
<td>r=2.91</td>
</tr>
<tr>
<td><strong>Top Lateral</strong></td>
<td>10.5x61</td>
<td>10.5x60.5</td>
<td>13.5x80.5</td>
</tr>
<tr>
<td>Diagonal</td>
<td>A=17.9</td>
<td>A=14.9</td>
<td>A=23.7</td>
</tr>
<tr>
<td></td>
<td>r=2.92</td>
<td>r=2.89</td>
<td>r=3.24</td>
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</table>
### Table 21 Maximum Stresses (ksi) in Verification of Allowable Stress Method

#### A. Midspan Fracture

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>$n=5$</td>
<td>$n=7$</td>
<td>$n=7$</td>
</tr>
<tr>
<td>Bottom Lateral Diagonal</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_t, \text{max}$</td>
<td>25.2</td>
<td>25.3</td>
<td>27.8</td>
</tr>
<tr>
<td>$f_c, \text{max}$</td>
<td>4.9</td>
<td>8.8</td>
<td>3.8</td>
</tr>
<tr>
<td>$f_{all,c}$</td>
<td>18.1</td>
<td>16.7</td>
<td>19.6</td>
</tr>
<tr>
<td>Cross Bracing Horizontal</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_c, \text{max}$</td>
<td>15.4</td>
<td>13.1</td>
<td>16.9</td>
</tr>
<tr>
<td>$f_{all,c}$</td>
<td>19.5</td>
<td>19.5</td>
<td>19.7</td>
</tr>
<tr>
<td>Cross Bracing Diagonal</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_c, \text{max}$</td>
<td>7.0</td>
<td>10.8</td>
<td>6.4</td>
</tr>
<tr>
<td>$f_{all,c}$</td>
<td>19.1</td>
<td>19.1</td>
<td>20.0</td>
</tr>
<tr>
<td>Top Lateral Diagonal</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_c, \text{max}$</td>
<td>8.7</td>
<td>9.7</td>
<td>8.4</td>
</tr>
<tr>
<td>$f_{all,c}$</td>
<td>19.1</td>
<td>19.7</td>
<td>19.4</td>
</tr>
</tbody>
</table>

#### B. Critical Fracture Scenario

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Panels</td>
<td>$n=5$</td>
<td>$n=7$</td>
<td>$n=7$</td>
</tr>
<tr>
<td>Bottom Lateral Diagonal</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_t, \text{max}$</td>
<td>22.2</td>
<td>23.2</td>
<td>24.8</td>
</tr>
<tr>
<td>$f_c, \text{max}$</td>
<td>6.2</td>
<td>9.1</td>
<td>4.8</td>
</tr>
<tr>
<td>$f_{all,c}$</td>
<td>18.9</td>
<td>17.8</td>
<td>16.1</td>
</tr>
<tr>
<td>Cross Bracing Horizontal</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_c, \text{max}$</td>
<td>14.6</td>
<td>13.0</td>
<td>16.8</td>
</tr>
<tr>
<td>$f_{all,c}$</td>
<td>19.5</td>
<td>19.5</td>
<td>19.7</td>
</tr>
</tbody>
</table>
Table 22  Comparison of the Forces Acting on the Cross Bracing for each of the Computer Models

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=9</td>
</tr>
<tr>
<td>U (k)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simplified Model (Art. 3.1)</td>
<td>555.2</td>
<td>732.0</td>
<td>947.9</td>
</tr>
<tr>
<td>Verification Model (Art. 7.1)</td>
<td>447.2</td>
<td>623.9</td>
<td>829.5</td>
</tr>
<tr>
<td>F (k)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simplified Model (Art. 3.1)</td>
<td>230.8</td>
<td>348.8</td>
<td>477.4</td>
</tr>
<tr>
<td>Verification Model (Art. 7.1)</td>
<td>260.8</td>
<td>391.1</td>
<td>541.9</td>
</tr>
<tr>
<td>(U+F)/2 (k)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simplified Model (Art. 3.1)</td>
<td>393.0</td>
<td>540.0</td>
<td>712.7</td>
</tr>
<tr>
<td>Verification Model (Art. 7.1)</td>
<td>354.0</td>
<td>507.5</td>
<td>685.7</td>
</tr>
<tr>
<td>(U-F)/2 (k)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simplified Model (Art. 3.1)</td>
<td>162.2</td>
<td>224.6</td>
<td>235.3</td>
</tr>
<tr>
<td>Verification Model (Art. 7.1)</td>
<td>93.2</td>
<td>140.2</td>
<td>143.8</td>
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</table>
Table 23 Verification Study for the Load Factor Method
(Midspan Fracture)

A. Properties of Members Chosen

<table>
<thead>
<tr>
<th>Section</th>
<th>Bottom Lateral Diagonal</th>
<th>Cross Bracing Horizontal</th>
<th>Cross Bracing Diagonal</th>
<th>Top Lateral Diagonal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (in²)</td>
<td>14.9</td>
<td>19.3</td>
<td>7.81</td>
<td>11.2</td>
</tr>
<tr>
<td>Mini:LIum Radius of Gyration (in)</td>
<td>——</td>
<td>2.97</td>
<td>1.88</td>
<td>2.54</td>
</tr>
</tbody>
</table>

B. Maximum Forces From Computer Output Compared to Maximum Forces Predicted From Equations

<table>
<thead>
<tr>
<th></th>
<th>Bottom Lateral Diagonal</th>
<th>Cross Bracing Horizontal</th>
<th>Cross Bracing Diagonal</th>
<th>Top Lateral Diagonal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Computer Force</td>
<td>394 k</td>
<td>471 k</td>
<td>145 k</td>
<td>195 k</td>
</tr>
<tr>
<td>Maximum Force Predicted</td>
<td>458 k</td>
<td>536 k</td>
<td>192 k</td>
<td>229 k</td>
</tr>
</tbody>
</table>
Table 24 Verification Study for the Load Factor Method
(Fracture in the First Interior Panel)

A. Properties of Members Chosen

<table>
<thead>
<tr>
<th>Section</th>
<th>Bottom Lateral Diagonal</th>
<th>Cross Bracing Horizontal</th>
<th>Cross Bracing Diagonal</th>
<th>Top Lateral Diagonal</th>
</tr>
</thead>
<tbody>
<tr>
<td>WT 16.5x70.5</td>
<td>WT 12x73</td>
<td>WT 7x34</td>
<td>WT 9x48.5</td>
<td></td>
</tr>
<tr>
<td>Area (in²)</td>
<td>20.8</td>
<td>21.5</td>
<td>9.99</td>
<td>14.3</td>
</tr>
<tr>
<td>Minimum Radius of Gyration (in)</td>
<td>----</td>
<td>3.01</td>
<td>1.81</td>
<td>2.56</td>
</tr>
</tbody>
</table>

B. Maximum Forces From Computer Output Compared to Maximum Forces Predicted From Equations

<table>
<thead>
<tr>
<th></th>
<th>Bottom Lateral Diagonal</th>
<th>Cross Bracing Horizontal</th>
<th>Cross Bracing Diagonal</th>
<th>Top Lateral Diagonal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Computer Force</td>
<td>612 k</td>
<td>460 k</td>
<td>126 k</td>
<td>138 k</td>
</tr>
<tr>
<td>Maximum Force</td>
<td>734 k</td>
<td>545 k</td>
<td>272 k</td>
<td>367 k</td>
</tr>
</tbody>
</table>
Table 25 Fractured Girder Deflection at Midspan for Midspan Fracture (in)

<table>
<thead>
<tr>
<th>Bridge Span</th>
<th>100 ft</th>
<th>150 ft</th>
<th>200 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels</td>
<td>n=5</td>
<td>n=7</td>
<td>n=7</td>
</tr>
<tr>
<td>Simplified Model (Art. 3.1)</td>
<td>3.73</td>
<td>3.99</td>
<td>4.62</td>
</tr>
<tr>
<td>Verification Model (Art. 7.1)</td>
<td>3.86</td>
<td>4.14</td>
<td>4.93</td>
</tr>
</tbody>
</table>
TYPICAL LEGAL LOAD TYPES

TYPE 3 UNIT
WEIGHT = 50 KIPS

AXLE NO. 1 2 3

AXLE LOADS IN KIPS.

CG = CENTER OF GRAVITY

TYPE 352 UNIT
WEIGHT = 72 KIPS

AXLE NO. 1 2 3 4 5

TYPE 3-3 UNIT
WEIGHT = 80 KIPS

AXLE NO. 1 2 3 4 5 6

Fig. 1 AASHTO Highway Bridge Rating Vehicles (Ref. 4)
Fig. 2 Three Components of the Alternate Load Path

Fig. 3 Typical Top Lateral Bracing System Configuration

Fig. 4 Typical Bottom Lateral Bracing System Configuration
Fig. 5 Typical Variations of Top and Bottom Lateral Bracing Configurations
Fig. 6 Typical Cross and Truss Bracing Configurations
Fig. 7  Typical Configurations of Existing Two-Girder Bridges
Fig. 8 Details of the Bridge in Ref. 10
Fig. 9 Support Boundary Conditions for the Computer Model
Fig. 10 Adjustments in Computer Model to Prevent any Relative Movement Between the Two Girders

Fig. 11 Deck Link Members to Transfer Dead Load to Girders
### Number of members and elements

<table>
<thead>
<tr>
<th>Element</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>75</td>
</tr>
<tr>
<td>Floorbeam</td>
<td>80</td>
</tr>
<tr>
<td>Deck Link</td>
<td>32</td>
</tr>
<tr>
<td>Girder Top Flange</td>
<td>60</td>
</tr>
<tr>
<td>Top Lateral Diagonal</td>
<td>16</td>
</tr>
<tr>
<td>X-Bracing Top Horizontal</td>
<td>3</td>
</tr>
<tr>
<td>Girder Stiffener</td>
<td>124</td>
</tr>
<tr>
<td>Girder Web</td>
<td>120</td>
</tr>
<tr>
<td>X-Bracing Diagonal</td>
<td>18</td>
</tr>
<tr>
<td>Girder Bottom Flange</td>
<td>60</td>
</tr>
<tr>
<td>Bottom Lateral Diagonal</td>
<td>16</td>
</tr>
<tr>
<td>X-Bracing Bottom Horizontal</td>
<td>9</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>613</strong></td>
</tr>
</tbody>
</table>

**Fig. 12 Finite Element Mesh for Computer Model**
Fig. 13 Computer Model of Fracture
Fig. 14 Bridge With Five Panels of Bottom Lateral Bracing
Fig. 15  Loads and Reactions Acting on the 3-D Bridge Structure
Fig. 16 Details of Computer Study Bridges
Fig. 17 $v_D$ vs. $R_k$ for Different Number of Panels, $n$
Fig. 18 $v_L$ vs. $R_K$ for Different Number of Panels, $n$
Fig. 19 $v_D$ vs $\lambda$ for Different Allowable Stresses

$f_{\text{all}} = \text{allowable stress}$

$f_{\text{all}} = 27$ ksi

$f_{\text{all}} = 45$ ksi
Fig. 20 $v_L$ vs. $l$ for Different Allowable Stresses

$f_{all} = \text{allowable stress}$

$+ f_{all} = 27 \text{ ksi}$

$* f_{all} = 45 \text{ ksi}$

Span length, $l$
Fig. 21 Bottom Lateral Forces (kips) for Midspan Fracture with $A_{BL} = \text{Area}$ [Eq's. 3.7, 3.9 and 3.10]
Fig. 21 Bottom Lateral Forces (kips) for Midspan Fracture with $A_{BL} = \text{Area}$ [Eq's. 3.7, 3.9 and 3.10]
Fig. 22 Buckling Model of the Compression Diagonal Assuming it is Braced at Mid-Length by the Tension Diagonal

Fig. 23 Bottom Lateral Diagonal Forces (kips) for Fracture in the First Interior Panel
Fig. 24 \( \frac{f_C}{f_t} \) vs. Span Length for Different Values of \( \alpha \)
(a) Fractured Girder Elevation

(b) Bottom Lateral Bracing

\[ (n+1)/2 \text{ tension diagonals to carry } \sum F = F_1 + F_2 + F_3 \]

Fig. 25 Model for the Load Factor Method
\[
\left( \frac{n+1}{2} - 1 \right) \text{ tension diagonals to carry } \sum F
\]

(a) Bottom Lateral Bracing

(b) Forces and Reactions Acting on the Fractured Girder

Fig. 26 Model for the Load Factor Method for Fracture in a Panel Other Than Midspan
Fig. 27 Displacement Relationships for the Fractured Girder and Bottom Lateral Bracing
Fig. 28 $\zeta_D$ vs $R_k$ for Different Number of Panels, $n$
Fig. 29 $\zeta_L$ vs $R_k$ for Different Number of Panels, $n$

Stiffness Parameter, $R_k$
Fig. 30 ($\frac{1}{\nu_D}$) vs. $\chi$ for Different ($\frac{\Delta}{\chi}$)$_{\text{lim}}$

$\frac{1}{\nu_D} = 0.5 + C_D$

$\left( C_D = 0.017 \right)$

$\lim_{\chi \to 0} \frac{\Delta}{\chi} = \frac{1}{200}$

$\lim_{\chi \to 0} \frac{\Delta}{\chi} = \frac{1}{300}$

$\chi$

$\frac{1}{\nu_D}$

5.0  4.0  3.0  2.0  1.0  0.0
\[
\frac{1}{u_L} = 0.5 + c_L
\]

\[
\frac{\Delta}{l}\lim = \frac{1}{200}
\]

\[
(C_L = 0.0215)
\]

\[
\frac{\Delta}{l}\lim = \frac{1}{300}
\]

\[
(C_L = 0.016)
\]

Fig. 31 \((1/u_L)\) vs. \(l\) for Different \((\Delta/l)\) lim
Fig. 32 Fracture Scenario for Serviceability Method
(a) Deflection of the Unfractured Bridge

\[ \frac{\Delta_L}{\lambda} = \frac{1}{800} \]

(b) Deflection of the Fractured Bridge

Fig. 33 Existing Deflections in Bridges
Fig. 34 Displacements of Girder and Bottom Lateral Bracing System After Fracture [Load Factor Method]
Fractured Girder

(a) Forces from the bottom lateral diagonals acting on the unfractured and fractured girders

\[ F_{CBH} = k_d \frac{(U+F)}{2} \]

\[ F_{CBD} = k_d \frac{(U-F)}{2} \]

(b) Type A Cross Bracing

\[ F_{CBH} = U \]

\[ F_{CBD} = k_d \frac{(U-F)}{2} \]

(c) Type B Cross Bracing

Fig. 35 Transfer of Forces to the Cross Bracing
(a) Forces in the Bottom Lateral Diagonals

\[ F = F_{BL}[1 - \phi_{cl}]k_H \]

(b) Components of the Forces in the Bottom Lateral Diagonals Acting at the Critical Cross Bracing Location

\[ U = F_{BL}(1 + \phi_{tl})k_H \]

Fig. 36 Forces Developed by the Critical Cross Bracing System for Midspan Fracture [Allowable Stress Method]
(a) Bottom Lateral Diagonal Forces for the Load Factor Model

(b) Components of the Forces in the Bottom Lateral Diagonals Acting at the Critical Cross Bracing Location

Fig. 37 Forces Developed by the Critical Cross Bracing System for Midspan Fracture (Load Factor Method)
(a) Bottom Lateral Diagonal Forces for the Load Factor Method with Fracture in the First Interior Panel

\[ U = F_{BL}[s_{\text{max}} + \phi_t]k_H \]

\[ F = F_{BL}[s_{\text{max}} + \phi_t]k_H \]

\[ F_{CEH} = k_H[s_{\text{max}} + 0.5 \phi_t]F_{BL} \]

\[ F_{CBD} = 0.5(k_d \phi_t)F_{BL} \]

(b) Components of the Forces in the Bottom Lateral Diagonals Acting on the First Interior Cross Bracing System

\[ U = 0 \]

\[ F = F_{BL}[s_{\text{max}} + \phi_t]k_H \]

\[ F_{CEH} = 0.5(k_h \phi_t)F_{BL} \]

\[ F_{CBD} = 0.5(k_d \phi_t)F_{BL} \]

(c) Components of the Forces in the Bottom Lateral Diagonals Acting on the End Cross Bracing System

\[ U = 0 \]

\[ F = F_{BL}[s_{\text{max}} + \phi_t]k_H \]

\[ F_{CEH} = 0 \]

\[ F_{CBD} = 0.5(k_d \phi_t)F_{BL} \]

Fig. 38 Forces Developed by Critical Cross Bracing Locations for Fracture in the First Interior Panel
(a) Forces acting on the bottom lateral diagonals after midspan fracture

\[ 0.5 F_{HL}(\phi_{c1} + \phi_{c2})k_H \]

\[ F_{HL}(1 + \phi_{c1})k_H \]

\[ F_{HL}(1 - \phi_{c1})k_H \]

\[ 0.5 F_{HL}(\phi_{c1} + \phi_{c2} - \phi_{c3})k_H \]

\[ F_{HL}(\phi_{c1} - \phi_{c2})k_H \]

\[ F_{HL}(\phi_{c1} - \phi_{c3})k_H \]

\[ 0.5 F_{HL}(\phi_{c2} + \phi_{c3} - \phi_{c4})k_H \]

\[ F_{HL}(\phi_{c2} - \phi_{c3})k_H \]

\[ F_{HL}(\phi_{c2} - \phi_{c4})k_H \]

(b) Free Body Force Diagrams of the Cross Bracings

Fig. 39 Forces Transferred to the Top Lateral Bracing System (Allowable Stress Method)
Fig. 40  Forces Carried by the Top Lateral Diagonals  
(Allowable Stress Method)
(a) Forces acting on the bottom lateral diagonals after fracture in the first interior panel

\[ 0.5 F_{BL} \bar{\phi}_{max} k_H \]

\[ 0.5 F_{BL} \phi_{to} k_H \]

\[ F_{BL}(\bar{\phi}_{max} + \phi_{to}) k_H \]

\[ 0.5 F_{BL}(\bar{\phi}_{max} + \phi_{cl} - \phi_{to}) k_H \]

\[ F_{BL}(\bar{\phi}_{max} + \phi_{cl}) k_H \]

\[ F_{BL}(\phi_{to} - \phi_{cl}) k_H \]

(b) Free Body Diagrams of Cross Bracings

Fig. 41 Forces Transferred to the Top Lateral Bracing System (Load Factor Method)
Identical loads as below

(a) Applied loads on the top lateral bracing

\[ 0.5 F_{EL}(\phi_{c5} + \phi_{c4}) y_H \]
\[ 0.5 F_{EL}(\phi_{c4} + \phi_{c3}) y_H \]
\[ 0.5 F_{EL}(\phi_{c3} + \phi_{c2}) y_H \]
\[ 0.5 F_{EL}(\phi_{c2} + \phi_{c1}) y_H \]
\[ 0.5 F_{EL}(\phi_{c1} + \phi_{c0}) y_H \]
\[ 0.5 F_{EL}(\phi_{max} + \phi_{c0}) y_H \]

(b) Forces carried by each panels top lateral diagonals

Fig. 42 Forces Carried by the Top Lateral Diagonals (Load Factor Method)
Fig. 43 Fraction of Live Load, $\theta$, Acting on the Fractured Girder
(a) One lane of HS20 truck axle loading applied to the fractured girder

(b) Axle loads replaced by an equivalent concentrated load, $\frac{\beta(L+I)}{2}$

$F_{L+I} = 307.5 \text{ k}$

Fig. 44 Equivalent Concentrated Live Plus Impact Load, $\frac{\beta(L+I)}{2}$
(a) Forces and reactions acting on the fractured girder including the forces from the cross bracing

\[ \frac{w}{2} + \frac{\beta(L+I)}{2} \]

(b) Forces in the bottom lateral diagonals

\[ U = F_{BL}(1 + \phi_{c1})k_d \]
\[ F = F_{BL}(1 - \phi_{c1})k_d \]

(c) Forces from the bottom lateral diagonals acting on the cross bracing

Fig. 45 Forces Resulting when Cross Bracing Forces are Included
Fig. 46 Results of Computer Output for Load Factor Method Verification for Midspan Fracture [150 ft, n=7]
Fig. 47 Results of Computer Output for Load Factor Method Verification, Fracture in the First Interior Panel [150 ft, n=7]
REFERENCES


10. Bridge S.H. 613 So. Cairo to Catskill over Catskill Creek, New York State.

APPENDIX A: Nomenclature

\[ \alpha \] length of a bottom lateral diagonal

\[ \beta \] length of the panel

\[ \gamma \] The fraction of total live, \( L \), plus impact, \( I \), load on the fractured girder

\[ f_D \] Load factor for dead load

\[ f_L \] Load factor for live plus impact loads

\[ \Delta \] Vertical displacement of fractured girder at midspan due to midspan fracture

\[ \Delta_{\text{lim}} \] Limiting deflection-to-span-length ratio for midspan fracture

\[ \epsilon_{\text{ed}} \] Strain of the bottom lateral tension diagonal in the end panel

\[ \epsilon_{\text{md}} \] Strain of the midspan bottom lateral diagonals

\[ \epsilon_Y \] Yield strain \([(f_y)/E]\)

\[ \epsilon_{\text{cr}} \] Critical end slope for fracture in the end panel [Eq. 3.35]

\[ \Sigma F \] Force applied to the bottom flange of the fractured girder on half the span by the bottom lateral bracing diagonals

\[ \varphi \] Strength reduction factor

\[ \delta_c \] Ratio of maximum compressive stress in a bottom lateral diagonal due to fracture in the first interior panel to the maximum tensile stress due to midspan fracture

\[ \delta_{\text{max}} \] Amplification factor to account for the increase in the maximum force in a bottom lateral diagonal [Load Factor Method]

\[ A_{BL} \] Area of one bottom lateral diagonal

\[ A_f \] Average area of one girder bottom flange

\[ \bar{A}_f \] Effective area of one girder bottom flange \([A_f + 0.3\ A_w]\)
\( A_g \)  Average area of a girder  
\( A_w \)  Area of girder web  
\( d \)  Depth of girder  
\( D \)  Dead load effect  
\( e_{ed} \)  Elongation of the bottom lateral tension diagonal in the end panel  
\( e_{md} \)  Elongation of the midspan bottom lateral diagonals  
\( E \)  Young's Modulus  
\( f_a \)  Maximum axial compression stress  
\( f_{all} \)  Allowable tensile stress  
\( f_{all,c} \)  Allowable compressive stress  
\( f_{c,max} \)  Maximum compressive stress in a member  
\( f_D \)  Dead load stress  
\( f_L \)  Live load plus impact stress  
\( f_{t,max} \)  Maximum tensile stress in a member  
\( f_Y \)  Yield stress  
\( F \)  Forces from the bottom lateral diagonals acting on the fractured girder  
\( F_{BL} \)  Force in the midspan bottom lateral diagonals due to midspan fracture  
\( F_{CBD} \)  Force in a cross bracing diagonal  
\( F_{CBH} \)  Force in a cross bracing horizontal  
\( F_{TL} \)  Force in a top lateral bracing diagonal  
\( F_Y \)  Yield force \( \left[ (A_{BL})(f_Y) \right] \)  
\( h \)  Horizontal displacement of the fractured girder at midspan due to midspan fracture  
\( I_g \)  Average moment of inertia of a girder
\( k_D \) length of a cross bracing diagonal

\( k_H \) length of a bottom lateral diagonal

\( k_H \) length of a cross bracing horizontal

\( k_H \) length of a bottom lateral diagonal

\( K \) Effective length factor

\( \lambda \) Span length

\( \beta (L+I) \) Live load plus impact effect

\( n \) Number of panels of bottom lateral bracing

\( r \) radius of gyration

\( R_k \) Stiffness parameter which is a function of the ratio of the axial stiffness of a bottom lateral diagonal member to the axial stiffness of the effective area of the bottom flange

\( RF \) Rating Factor

\( RRF \) Redundancy Rating Factor

\( S \) Girder spacing

\( S_u \) Member strength

\( u_D \) Coefficient which accounts for the bottom lateral diagonal force distribution for dead load [Serviceability Method]

\( u_L \) Coefficient which accounts for the bottom lateral diagonal force distribution for live plus impact loads [Serviceability Method]

\( U \) Forces from the bottom lateral diagonals acting on the unfractured girder

\( v_D \) Coefficient which accounts for the bottom lateral diagonal force distribution for dead load [Allowable Stress Method]

\( v_L \) Coefficient which accounts for the bottom lateral diagonal force distribution for live plus impact loads [Allowable Stress Method]

\( w \) Weight of the structure as a uniform line load on each girder