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PROGRESS REPORT

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AFTER FRACTURE REDUNDANCY RATING OF TWO-GIRDER STEEL BRIDGES

SUMMARY OF FINDINGS

This progress 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.

In the design and rating by AASHTO of two-girder steel bridges, the two girders are considered in the traditional simplified analytical model of the bridge to be the only load paths available to carry the vertical loads. This model cannot be used to evaluate the capacity and safety of a bridge after fracture of a main load carrying member, such as one of the girders of a simple span two-girder bridge. Viable alternate load paths may be found which bypass the fractured girder, but this suggests a much different and more complex analytical model.

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 progress report proposes a Redundancy Rating level and

concentrates mainly on the related analytical models and procedures which are new to the bridge engineer.

Two-girder bridges are subdivided into different types. Each of the types are determined by the configuration of the components comprising the alternate load path. At this stage of the investigation four types have been identified.

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 suggested in this progress 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. 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.

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. Redundancy Rating equations are developed for the bottom lateral bracing system in terms of the required area of the diagonal members for one type of two-girder bridge.

It is proposed to extend the methods developed in this progress report to other members and connections on the alternate load path and to other types of two-girder steel bridges.

AFTER-FRACTURE REDUNDANCY RATING OF TWO-GIRDER STEEL BRIDGES

BACKGROUND

AASHTO Design and Rating Models

In the design and rating by AASHTO (1,2)* 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

* References begin on page 61 of this report.

conservative (often overly conservative) design or rating is achieved without the need to consider the three-dimensional interaction of all the bridge components.

Need for Operating and Inventory Ratings

An existing bridge is rated as part of a safety program on a regular schedule or whenever it is obvious that the conditions upon which the bridge was designed have significantly changed. For example, these changes can include the following:

1. Deterioration of the structure due to corrosion, etc.
2. Changes in the vehicular loading intensity and frequency.

Rating is also performed as part of a short or long term bridge repair, rehabilitation or replacement plan. The outcome of a rating analysis may be to close a bridge, to post a bridge for maximum vehicle loading and/or to schedule a bridge for repair, rehabilitation or replacement. If a bridge continues in service after a rating analysis has been performed, the bridge is assumed to be able to function continuously in accordance with the rating decision without considering the possibility of an impending disaster until such time that a further rating is scheduled or considered necessary.

Although an AASHTO rating analysis of a steel bridge may be conducted for all vertical load carrying members, the following discussion is confined to the two main girders.

Existing Rating Levels

The bridge girders are rated at two levels (2):

1. Operating Rating Level: Absolute maximum permissible load level for the bridge girders.
2. Inventory Rating Level: The "normal" capacity of the bridge girders, 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 (2).

Existing Rating Methods

The bridge girders are rated using two methods (2):

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 girder is determined such that the maximum stress in the girder does not exceed the specified allowable stress.

For noncomposite bridge girders the RF's for both the Operating and Inventory levels are given by (2),

$$RF = \frac{f_{all} - f_D}{f_L} \tag{1}$$

where f_{all} = Allowable Stress

f_D = Dead Load Stress

f_L = Live plus Impact Load Stress (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 girder is determined such that the load effect (bending moment, for example) does not exceed the strength of the girder (including a strength reduction factor).

For noncomposite bridge girders the RF for the Operating Rating level is given by (2),

$$RF = \frac{\phi S_u - \gamma_D D}{\gamma_L (L+I)} \quad (2)$$

where, ϕ = Strength reduction factor

S_u = Member strength (maximum moment capacity, for example)

D = Dead load effect (bending moment, for example)

L+I = Live plus impact load effect

γ_D = Load factor for dead load = 1.3

γ_L = Load factor for live plus impact loads = 1.3

The corresponding RF for the Inventory Rating level is,

$$RF = \left(\frac{3}{5}\right) \frac{\phi S_u - 1.3D}{1.3(L+I)} \quad (3)$$

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 (3). 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 design and rating analyses(4).

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 (5,6).

There is clearly the need for an additional rating level which would address bridge redundancy with respect to a particular fracture scenario. This report suggests the term Redundancy Rating (RR) level. 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.

Assuming that the probability of maximum design loading occuring in the time interval between girder fracture and fracture detection is low, the proposed RR can be based on elevated allowable stresses or reduced load factors as is currently done for the Operating Rating. The same rating vehicles can be used for the RR. However, the number of traffic lanes loaded might be less than presently required for design and rating and needs to be investigated. In addition, suitable allowable stresses and load factors need to be determined. Since the deflection of the deck after fracture of a girder is expected to be somewhat greater than before fracture, the current AASHTO impact factor may also need reexamination. It is not considered within the scope of this project to do more than suggest the applicable loading conditions, allowable stresses or load and impact factors.

This progress report discusses the concept of Redundancy Rating and concentrates mainly on the analytical procedures which are new to the bridge engineer. These procedures were necessarily developed earlier in this and other projects in order to establish viable analytical models which could be used either in design for redundancy or for Redundancy Rating (3,4). The current phase of this project is concerned with extending these models in order to develop RR procedures, classifications and guidelines for various types of two-girder bridges.

REDUNDANCY RATING

Definition of Redundancy

A slightly updated definition of redundancy developed in the interim report of this project (4) is used:

> 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.

Although this project is concerned with two-girder bridges, this definition is comprehensive enough to apply to virtually any bridge.

Alternate Analytical Approaches to Redundancy Rating

In the RR of two-girder steel bridges, there are two alternate analytical approaches: 1) evaluation from computer analysis, and 2) evaluation using empirically derived equations.

The first approach to RR is to analyze an appropriate 3-dimensional structural model using a comprehensive computer program and to substitute the analysis results into the AASHTO Allowable Stress rating equation. In this approach a problem occurs in deciding how realistic and complicated the structural model has to be. The model has to be simple enough to be easily understood and to lead to a lower bound, or conservative, solution. An appropriate three-dimensional model efficiently utilizing the secondary members can be suggested if any alternate load path is identified. A viable approach may also include isolating the alternate load path as a substructure. This project, after identifying alternate load paths for various types of two-girder steel bridges, will study this approach and include guidelines for computer modeling.

The second approach to RR is to use empirically derived equations. This progress report develops RR concepts and empirical equations for one type of two-girder bridge. The concepts and equations are developed for practical use. The equations will be modified for other types of two-girder bridges later in the project.

<u>Redundancy Rating Model</u>

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

<u>Alternate Load Path Concept</u>: 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.

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 determining redundancy classifications for use in establishing bridge inspection, repair, rehabilitation or 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, and the approach suggested in this investigation, 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. 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.

Redundancy Rating Methods

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 (2). 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 this project in terms of a limiting deflection-to-span-length ratio. The establishment of a serviceability requirement is outside the scope of this project, although reasonable values are suggested.

TWO-GIRDER BRIDGE TYPES

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 steel bridges therefore must contain three basic components.

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. 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, or cross frames or diaphragms.

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 ℓ 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.

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. Figure 7 shows examples of cross frames and diaphragms.

Bridge Type Configurations

There are many configurations of existing two-girder bridges. Some may not contain one or more of the three components required for redundancy. For example, some noncomposite two-girder bridges may not contain a top lateral bracing system, and many bridges with partial depth diaphragms do not have a bottom lateral bracing system. 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.

For the purpose of developing RR concepts in this project, two-girder bridges are subdivided into four different types. Each of the four types are determined by the configuration of the three basic components comprising the alternate load path. At this stage of the investigation, considering only simple spans, the following four bridge types have been identified.

Examples are shown in Fig's. 8 through 11.

Type 1: noncomposite, with top lateral bracing
cross or truss bracing (Fig. 8)
bottom lateral bracing

Type 2: composite, with or without top lateral bracing
cross or truss bracing (Fig. 9)
bottom lateral bracing

Type 3: noncomposite, with top lateral bracing
cross frames or diaphragms (Fig. 10)
bottom lateral bracing

Type 4: composite, with or without top lateral bracing
cross frames or diaphragms (Fig. 11)
bottom lateral bracing

A two-girder steel bridge which does not possess the three basic components required for the alternate load path is considered to be nonredundant.

Top lateral bracing and/or a composite deck is considered a requirement for each of the four bridge types. Similarly each type must have a bottom lateral bracing system. Each bridge type shares the same variations in lateral bracing geometries.

Discussion of Bridge Types

Type 1 - (Fig. 8): Figure 8(a) shows a common example of a Type 1 bridge with cross bracing. This is a common bridge configuration for existing two-girder steel bridges. Figure 8(b) shows an example of a Type 1 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.

Type 2 - (Fig. 9): Figure 9(a) shows a Type 2 bridge with cross bracing and Fig. 9(b) shows an example with truss bracing. The composite deck acting together with the girders may or may not be sufficient to achieve a desired level of redundancy. This depends on the strength of the shear connection between the deck and the girders plus the strength of the deck to carry lateral loads. The contribution of a composite deck is being studied in this project. If the composite deck is insufficient, a top lateral bracing system can be installed to provide the extra lateral stiffness and strength required.

Type 3 - (Fig. 10): Figure 10(a) shows a Type 3 bridge with a cross frame and Fig. 10(b) shows an example with a diaphragm. It is likely that bridges of this type do not have top or bottom lateral bracing systems since it is more common for the lateral bracing to be placed at an intermediate level within the floorbeam. Top and bottom lateral bracing systems need to be installed for redundancy if they are not present. A possible location for these top and bottom laterals is shown with dashed lines in the figure.

Type 4 - (Fig. 11): Similar to the Type 3 bridges, a bottom lateral bracing system needs to be installed if it is absent in the existing bridge. A top lateral bracing system can be added if the composite deck is not sufficient to achieve a desired level of redundancy.

DEVELOPMENT OF REDUNDANCY RATING CONCEPTS FOR TYPE 1 BRIDGES

The remainder of this progress report is devoted to the development of concepts and RR equations for Type 1 bridges, including worked examples. This development is more clearly explained by first considering a bridge with only two symmetrically placed planes of interior cross bracing (or cross frames or diaphragms) as shown in Fig. 12(a). The bottom lateral bracing is therefore confined to a single panel between the cross bracing as shown in the figure. Although this is not likely to be a practical configuration, it is useful for developing the basic equations of redundancy which are then modified for multiple panel bracing.

This progress report considers only midspan fracture of one of the two girders of a Type 1 bridge, as is also shown in Fig. 12(a). Although this is probably the worst case scenario, other fracture scenarios are under investigation. The fracture is assumed to extend through the bottom (tension) flange and through the full web depth. 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.

Single Panel Concept

The single panel concept deals with a bridge with continuous top lateral bracing and one panel of bottom lateral bracing at midspan as shown in Fig. 12. In the figure the girder spacing is S and the girder depth is d. The span length is ℓ. The number of panels of top lateral bracing is n.

The loads and reactions acting on the fractured and unfractured girders are shown in Fig. 13. 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 β. Therefore β(L+I) and (1-β)(L+I) are the equivalent 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 on the fractured girder are found by summing moments about line AB along the unfractured girder and are shown in Fig. 13.

After midspan fracture occurs, the force applied to the bottom flange of the fractured girder by the bottom lateral bracing diagonals is F_1 as shown in the figure. 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 (3). The force F_1 calculated on the condition of zero bending moment at midspan of the fractured girder is,

$$F_1 = \frac{1}{d}\left[\frac{w\ell^2}{8} + \frac{\beta(L+I)\ell}{4}\right] \tag{4}$$

The resulting tension force, F_{BL}, in each of the bottom lateral diagonal members is equal to αF_1, where α is the ratio of the length of a bottom lateral diagonal member to the length of the panel. Substituting this into Eq. 4, the tension force in the bottom lateral diagonal is,

$$F_{BL} = \frac{\alpha}{d}\left[\frac{w\ell^2}{8} + \frac{\beta(L+I)\ell}{4}\right] \tag{5}$$

From Eq. 5, the forces due to dead load, F_{BL_D}, and due to live load plus impact, F_{BL_L}, in the bottom lateral diagonal members are,

$$F_{BL_D} = \frac{\alpha w\ell^2}{8d} \tag{6}$$

$$F_{BL_L} = \frac{\alpha\beta(L+I)\ell}{4d} \tag{7}$$

The Allowable Stress Method is selected as an example of how Redundancy Rating can be developed for the single panel concept as follows. The following Rating Factor for the Allowable Stress Method was previously given in Eq. 1,

$$RF = \frac{f_{all} - f_D}{f_L} \tag{8}$$

The stresses in the bottom lateral diagonals from Eq's. 6 and 7 are,

$$f_D = \frac{\alpha w \ell^2}{8 d A_{BL}}$$

$$f_L = \frac{\alpha \beta (L+I) \ell}{4 d A_{BL}}$$

where A_{BL} = area of bottom lateral diagonal

Substituting the above expressions into Eq. 8, the RRF for both bottom lateral diagonals is,

$$RRF = \frac{f_{all} - \frac{\alpha w \ell^2}{8 d A_{BL}}}{\frac{\alpha \beta (L+I) \ell}{4 d A_{BL}}} \tag{9}$$

As previously indicated, the approach used in this project for the after-fracture evaluation of an existing two-girder bridge is to determine the requirements of the alternate load path in terms of an RRF equal to unity. Setting Eq. 9 equal to one, the required area, A_{BL}, of the two bottom lateral diagonal members is,

$$\text{Required } A_{BL} = \frac{\alpha \ell}{8 d f_{all}} \left[w \ell + 2\beta (L+I) \right] \tag{10}$$

Multiple Panel Concept

The single panel concept is extended to the more practical multiple panel case dealing with two-girder bridges having bottom lateral bracing in more than one panel. Figure 14 shows a two-girder bridge with five panels (n=5) of top and bottom lateral bracing. The areas of all the bottom lateral diagonal members shown in Fig. 14(a) are assumed to be equal.

In the single panel concept the force F_1, as shown in Fig. 13, is developed by only one bottom lateral diagonal in tension. In the multiple panel concept, assuming n = 5, the forces applied to the bottom flange of the fractured girder by the bottom lateral diagonal members are F_1, F_2 and F_3 as shown in Fig's. 14(b) and (c). The cross bracing shear is ignored as was done in the single panel concept. By analogy to Eq. 4,

$$F_1 + F_2 + F_3 = \frac{1}{d}\left[\frac{w\ell^2}{8} + \frac{\beta(L+I)\ell}{4}\right] \qquad (11)$$

Forces F_1 and F_2 are each developed by two members, 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 (3,4). That is $F_1 > F_2 > 2F_3$. 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 panels, 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. 11 multiplied by α and divided by n. To account for the increase in force in this diagonal as discussed above, Eq. 11 can be multiplied again by a coefficient, γ. Since the coefficient γ is different for dead and for live plus impact effects, the coefficient can be separated into a coefficient for dead load, γ_D, and a coefficient for live plus impact, γ_L. Thus the dead load force, F_{BL_D}, and live load plus impact force, F_{BL_L}, in the tension diagonal at midspan are given by,

$$F_{BL_D} = \frac{\alpha w \ell^2}{8d} \cdot \frac{\gamma_D}{n} \tag{12}$$

$$F_{BL_L} = \frac{\alpha \beta (L+I) \ell}{4d} \cdot \frac{\gamma_L}{n} \tag{13}$$

The extreme values of γ_D and γ_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. In this case $\gamma_D = \gamma_L = 1.0$. Similarly if the two girders are assumed to have zero cross sectional areas $\gamma_D = \gamma_L = n$ and the tension force in the diagonals at midspan is the same as in the single panel concept. All other diagonals have zero forces. In what follows, values of V_0 and V_L between these two limits will be established for practical two-girder bridges.

REDUNDANCY RATING EQUATIONS FOR TYPE 1 BRIDGES

Redundancy Rating equations are developed in this section for the Allowable Stress, Load Factor and Serviceability Methods. The equations are developed in terms of the required area of the bottom lateral diagonal members.

Allowable Stress Method

The required area, A_{BL}, of all bottom lateral diagonal members (tension and compression) is given by the following equation,

$$\text{Req'd } A_{BL} = \frac{\alpha \lambda}{8 d n f_{all}} \left[V_0 w \lambda + 2 V_L \beta (L+I) \right] \tag{14}$$

This equation is derived in a similar manner to Eq. 10.

Practical values of V_D and V_L need to be determined by studying existing bridges. A computer study was performed to determine the variation in V_0 and V_L for typical Type 1 bridges. The computer models were based on the bridge in Ref. 7, one of the bridges provided by the project panel. A cross section of the bridge from Ref. 7 showing the noncomposite girders is shown in Fig. 15(a). An elevation view showing the nonprismatic girders is shown in Fig.

15(b). The span length is 150 ft. For this particular span of the bridge in Ref. 7, 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. 15(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. 15(c). The floorbeam spacing is 10 ft. Girders are 10 ft. deep.

Several bridges based on Ref. 7 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. 7. The computer models included bridges with spans of 100, 150 and 200 feet. The span length to girder depth ratio (ℓ/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 Fig. 16(c). Eighteen different cases are modeled as shown in the figure.

The bridges were modeled for computer analysis using the Computer Aided Engineering Laboratory facility at Fritz Engineering Laboratory and the GTSTRUDL finite element analysis program. The top lateral bracing and cross bracing were modeled so that the fractured girder and bottom lateral bracing system would behave as previously assumed.

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 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 $\sqrt{}_0$ and $\sqrt{}_L$ for the tension diagonals at midspan for the above 18 cases.

Values of $\sqrt{}_0$ and $\sqrt{}_L$ were obtained by substituting the values of F_{BL_0} and F_{BL_L} from the computer output into Eq's. 12 and 13. These thirty six values of $\sqrt{}_0$ and $\sqrt{}_L$ are plotted as a function of the stiffness parameter R_K and the number of panels, n, in Fig's. 17 and 18 where,

$$R_K = \frac{A_{BL}}{\alpha^3 \bar{A}_f}$$

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,

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

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. 14 to compute the points plotted in Fig's. 19 and 20. The coefficients γ_D and γ_L are plotted as a function of the three span lengths used in the study for two assumed values of allowable stress. The points in the figure also cover the range of variation of n used in the computer study.

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 γ_D and γ_L for other practical span lengths and allowable stresses. The equations of these straight lines are as follows,

$$\gamma_D = 0.8 + 0.36 \ell / f_{all} \tag{15}$$

$$\gamma_L = 0.8 + 0.18 \ell / f_{all} \tag{16}$$

Where ℓ is in ft. and f_{all} is in ksi.

Table 1 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. 15 and 16. 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 γ_D and γ_L given by Eq's. 15 and 16. A similar procedure was used to determine the required areas in rows *1 and *3. The four levels of allowable stress were chosen to determine if

Eq's 15 and 16 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.

Load Factor Method

Figure 21 shows the model used for the Load Factor Method. It is assumed that all of the bottom lateral diagonals in tension are yielded and that all of the diagonals in compression are buckled. Therefore the number of bottom lateral diagonals subjected to the total force, $F_1 + F_2 + F_3$ (Eq. 11), is $(n+1)/2$. There is no need for a coefficient, γ, because all the tension members have yielded and carry the same load. Therefore the dead load force, F_{BL_D}, and live load plus impact force, F_{BL_L}, in any tension diagonal is given by,

$$F_{BL_D} = \frac{2\alpha}{n+1}\left(\frac{w\ell^2}{8d}\right) \tag{17}$$

$$F_{BL_L} = \frac{2\alpha}{n+1}\left(\frac{\beta(L+I)\ell}{4d}\right) \tag{18}$$

The following Rating Factor for the Load Factor Method was previously given in Eq. 2, (2)

$$RF = \frac{\phi S_u - \gamma_D D}{\gamma_L(L+I)} \tag{19}$$

In this case,

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

where f_y = yield stress level

Therefore, the Redundancy Rating Factor (RRF) for the tension bottom lateral diagonals is found by substituting Eq's. 17 and 18 into Eq. 19,

$$\text{RRF} = \frac{f_y A_{BL} - 2\gamma_D \alpha w \ell^2 / 8d(n+1)}{2\gamma_L \alpha \beta (L+I)\ell / 4d(n+1)}$$

Setting the RRF equal to one and solving for A_{BL} gives,

$$\text{Req'd } A_{BL} = \frac{\alpha \ell}{4d(n+1)f_y}\left[\gamma_D w \ell + 2\gamma_L \beta (L+I)\right] \quad (20)$$

<u>Serviceability Method</u>

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. Figure 22 shows the displacement relationships for the fractured girder and bottom lateral bracing. From Fig. 22(a) it can be seen that,

$$\frac{\Delta}{\ell} = \frac{h}{2d} \quad (21)$$

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

Δ = vertical displacement of fractured girder at midspan

From Fig. 22(b), the strain of the bottom lateral tension diagonal at midspan is,

$$\varepsilon_{BL} = \frac{h/\alpha}{\alpha \ell / n} = \frac{nh}{\alpha^2 \ell}$$

The stress in the bottom lateral diagonal is,

$$f_{BL} = E\varepsilon_{BL} = \frac{Enh}{\alpha^2 \ell} \tag{22}$$

Coefficients, similar to γ_D and γ_L in the Allowable Stress Method, are needed for the Serviceability Method. The coefficients for the Serviceability Method are defined as μ_D for dead load and μ_L for live plus impact. The dead load force, F_{BL_D}, and live load plus impact force, F_{BL_L}, in the bottom lateral tension diagonal at midspan for the Serviceability Method are found by replacing γ with μ in the Allowable Stress Method equations (Eq's. 12 and 13), as follows:

$$F_{BL_D} = \frac{\alpha w \ell^2}{8d} \cdot \frac{\mu_D}{n} \tag{23}$$

$$F_{BL_L} = \frac{\alpha \beta (L+I) \ell}{4d} \cdot \frac{\mu_L}{n} \tag{24}$$

Dividing Eq's. 23 and 24 by A_{BL} and substituting for f_{BL} in Eq. 22 gives,

$$h = \frac{\alpha^2 \ell}{EnA_{BL}} \left[\frac{\alpha w \ell^2}{8d} \cdot \frac{\mu_D}{n} + \frac{\alpha \beta (L+I) \ell}{4d} \cdot \frac{\mu_L}{n} \right]$$

Substituting this value of h into Eq. 21 gives,

$$\frac{\Delta}{\ell} = \frac{\alpha^3 \ell^2}{16 E n^2 d^2 A_{BL}} \left[\mu_D w \ell + 2\mu_L \beta (L+I) \right] \tag{25}$$

In the Serviceability Method, the requirements of the alternate load path are determined by satisfying a Δ/ℓ limit. Solving Eq. 25 for the required area of bottom lateral diagonal,

$$\text{Req'd } A_{BL} = \frac{\alpha^3 \ell^2}{16 E n^2 d^2 (\Delta/\ell)_{lim}} \left[\mu_D w \ell + 2\mu_L \beta (L+I)\right] \tag{26}$$

Suitable values of μ_D and μ_L are found for the Serviceability Method in a similar manner as γ_D and γ_L were found for the Allowable Stress Method. The values of μ_D and μ_L are computed for each of the bridges in the computer study. This is done by obtaining the value of Δ due to dead load from the computer, substituting it into Eq. 25 with the bridge data and the dead loads only and solving for μ_D. The same procedure is used to find μ_L. These thirty six values of μ_D and μ_L are plotted as a function of the stiffness parameter, R_K, and n in Fig's. 23 and 24. Using a trial and error procedure and maintaining an assumed condition that the Δ/ℓ limit equals 1/300, the curves in Fig's. 23 and 24 were used together with Eq. 26 to determine values of μ_D and μ_L for the study bridges. For the 18 cases, μ_D varied from 2.4 to 3.1 and μ_L varied from 1.7 to 2.1. Studies are continuing to determine if functional relationships similar to Eq's. 15 and 16 but in terms of Δ/ℓ can be derived. For purposes of the worked examples shown later in this progress report the following conservative values are used:

$\mu_D = 3.1$

$\mu_L = 2.1$

Substituting these values into Eq. 26 gives the required area of bottom lateral diagonals for the Serviceability Method,

$$\text{Req'd } A_{BL} = \frac{\alpha^3 \ell^2}{16 E n^2 d^2 (\Delta/\ell)_{lim}} \left[3.1 w \ell + 4.2 \beta (L+I) \right] \tag{27}$$

<u>EXAMPLES</u>

The same bridges used in the computer study are used for the worked examples. Figure 16(c) provides details of span lengths, number of panels, etc. A required area of bottom lateral bracing diagonal is calculated for each bridge case using the equations developed in this progress report. Three different values of the required area are calculated using the three Redundancy Rating methods. The following assumptions are used for all examples.

<u>Assumptions</u>

<u>Vehicular Loading</u>: 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 ft. when the truck loading is replaced by an equivalent concentrated load at midspan.

<u>Traffic Lanes Loaded</u>: One traffic lane is loaded.

<u>Allowable Stresses</u>: The allowable stresses for the Operating Rating level are used, i.e. $f_{all} = (0.75) f_y$.

<u>Load Factors</u>: Load factors of 1.1 for dead load and 1.3 for live load are used.

<u>Impact Factor</u>: An impact of 30% is used.

Limiting Deflection: The limiting deflection to span length ratio (Δ/ℓ) is taken as 1/300.

Worked Example: 100 ft. span; n = 5

The first step is to determine the uniform line load w and the equivalent concentrated live load plus impact, $\beta(L+I)$, 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 line load, w, on each girder,

$$w = 1/2(7.16) = 3.58 \text{ k/ft}$$

Figure 25(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 curb (2). The fraction of truck load, β, acting on the fractured girder is found from the influence line shown in Fig. 25(b),

$$\beta = 1/2(1.194 + 0.861) = 1.03$$

Figure 26(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 β 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. 26(b) which will create the same total force F_{L+I}. From Fig. 26(b),

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

or, $\beta(L+I) = 82.0$ kips

The next step is to calculate the term α which is the length of a bottom lateral diagonal divided by the length of the panel.

$$\text{length of panel} = \ell/n = 100/5 = 20 \text{ ft.}$$

For a girder spacing of 18 feet,

$$\text{length of diagonal} = \sqrt{(20)^2 + (18)^2} = 26.91 \text{ ft.}$$

Then, $\alpha = 26.91/20 = 1.35$

Finally, for a yield stress level of $f_y = 36$ ksi, the allowable stress, f_{all}, is $0.75\ f_y = 27$ ksi.

The required area of bottom lateral bracing diagonal can now be determined for each of the Redundancy Rating methods by substituting the above information into the appropriate equations.

<u>Allowable Stress Method</u>: The values of the coefficients $\sqrt{\ }_D$ and $\sqrt{\ }_L$ are found from Eq's. 15 and 16 (λ is in ft.),

$$\sqrt{\ }_D = 0.8 + 0.36(100)/27 = 2.13$$

$$\sqrt{\ }_L = 0.8 + 0.18(100)/27 = 1.47$$

Substituting into Eq. 14, the required area, A_{BL}, of midspan tension bottom lateral diagonal is,

$$\text{Req'd } A_{BL} = \frac{(1.35)(100)}{(8)(6.67)(5)(27)}\left[(2.13)(3.58)(100) + (2)(1.47)(82.0)\right] = 18.8 \text{ in}^2$$

<u>Load Factor Method</u>: The required area, A_{BL}, of bottom lateral diagonal is given by Eq. 20,

$$\text{Req'd } A_{BL} = \frac{(1.35)(100)}{(4)(6.67)(6)(36)}\left[(1.1)(3.58)(100) + 2(1.3)(82.0)\right] = 14.2 \text{ in}^2$$

<u>Serviceability Method</u>: The required area, A_{BL}, of bottom lateral bracing diagonal is obtained from Eq. 27,

$$\text{Req'd } A_{BL} = \frac{(1.35)^3(100)^2}{(16)(29000)(5)^2(6.67)^2(1/300)}\left[(3.1)(3.58)(100) + (4.2)(82.0)\right] = 20.8 \text{ in}^2$$

<u>Additional Examples</u>

The required areas of the bottom lateral bracing diagonal members are found for the other computer study bridges in a similar fashion. The results are shown in Table 2. The areas are calculated using the data provided below the table.

Discussion of Results

The Load Factor Method results in a much lower required area than the Allowable Stress Method for longer spans. For smaller spans, the two strength methods yield similar results. The Serviceability Method yields increasing required areas for decreasing span lengths. For this reason the strength methods control for long and intermediate span lengths, but the Serviceability Method controls for short span lengths.

It is important to note that the results here are influenced by the assumptions made. The results will differ depending on the loading conditions, allowable stresses and deflections, plus load and impact factors used.

Table 1 Comparison of Required Bottom Lateral Bracing Areas, A_{BL}, for RRF=1 (Eq. 14)

Bridge span	100 ft		150 ft		200 ft	
Number of panels	n=5	n=7	n=7	n=9	n=9	n=13
Fy=30 ksi (F_{all}=22.5)						
*1 Computer Analysis	22.5	19.8	32.6	29.4	43.0	35.5
Eq's. 15 and 16	24.5	20.9	33.6	29.5	44.1	36.5
Fy=36 ksi (F_{all}=27.0)						
*2 Computer Analysis	16.8	14.7	23.2	21.5	30.3	25.8
Eq's. 15 and 16	18.2	15.6	24.6	21.6	31.9	26.4
Fy=50 ksi (F_{all}=37.5)						
*3 Computer Analysis	10.5	9.1	13.6	12.4	17.4	14.5
Eq's. 15 and 16	11.0	9.4	14.3	12.5	18.1	15.0
Fy=60 ksi (F_{all}=45.0)						
*4 Computer Analysis	8.2	7.0	10.2	9.4	12.9	10.6
Eq's. 15 and 16	8.4	7.2	10.7	9.4	13.4	11.1

*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 2 Comparison of Area Required for All Bottom Lateral Bracing Diagonal Members (in^2)

Bridge Span	100 ft		150 ft		200 ft	
Number of Panels	n=5	n=7	n=7	n=9	n=9	n=13
Allowable Stress Method (Eq's. 14, 15 and 16)	18.8	16.0	25.2	22.1	32.4	26.8
Load Factor Method (Eq. 20)	14.2	12.8	14.7	13.3	15.4	13.1
Serviceability Method (Eq. 27)	20.8	18.0	14.3	12.4	11.3	9.3

<u>Example Data</u>: $S = 18$ ft., $\lambda/d = 15$

$f_y = 36$ ksi, $f_{all} = 27$ ksi, $E = 29000$ ksi

$\gamma_D = 1.1$, $\gamma_L = 1.3$

<u>Load Data</u>

100 ft. span: $w = 3.58$ k/ft
$\beta(L+I) = 82.01$ kips

150 ft. span: $w = 3.88$ k/ft
$\beta(L+I) = 86.81$ kips

200 ft. span: $w = 4.16$ k/ft
$\beta(L+I) = 89.20$ kips

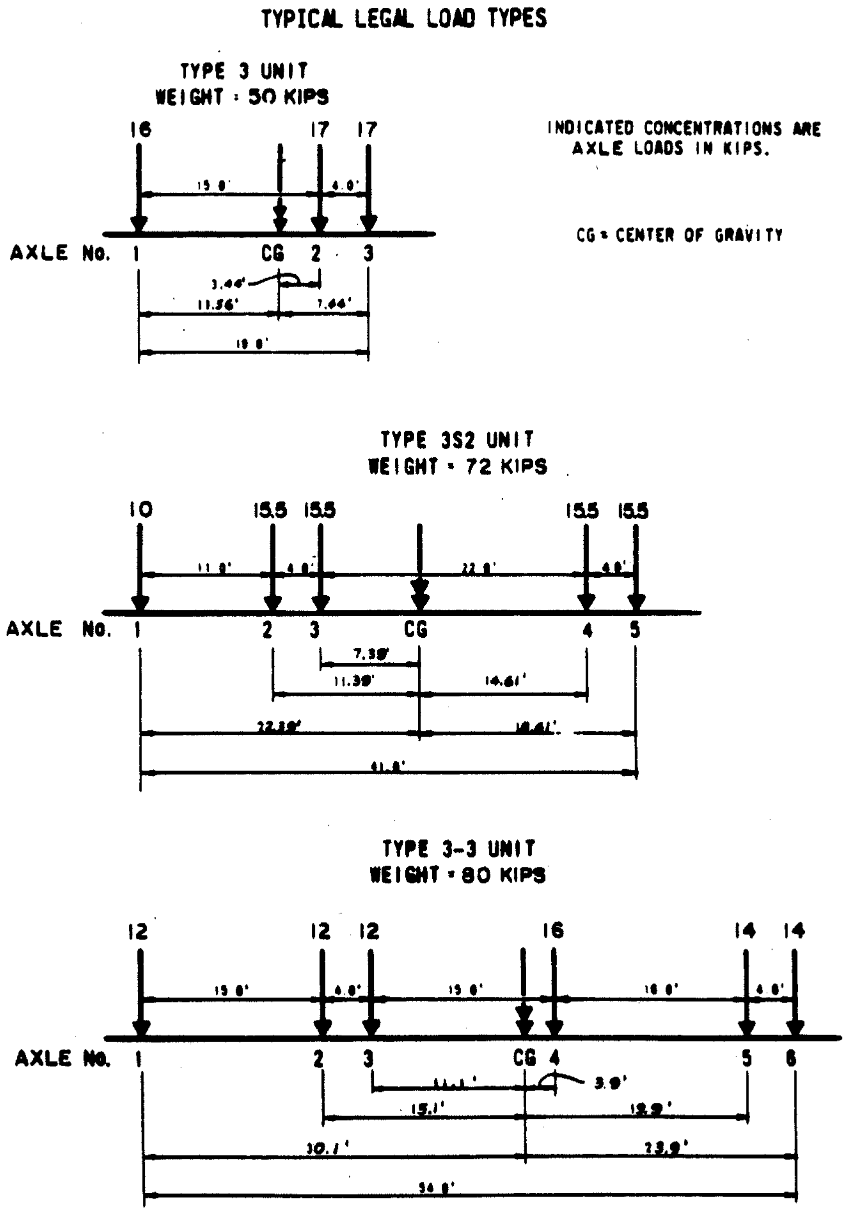


Fig. 1 AASHTO Highway Bridge Rating Vehicles

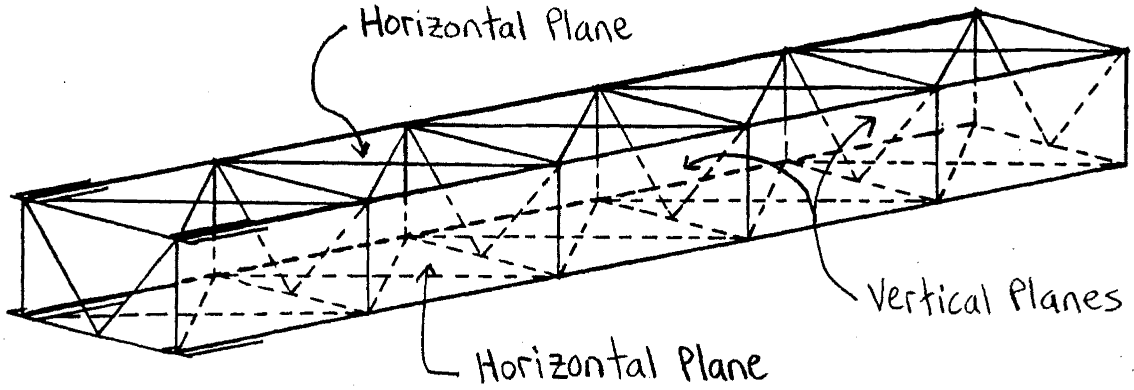


Fig. 2 Three Components of the Alternate Load Path

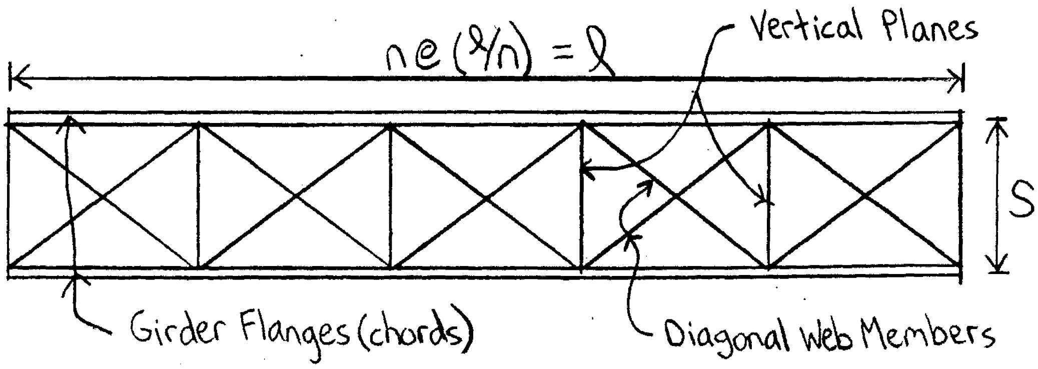


Fig. 3 Typical Top Lateral Bracing System Configuration

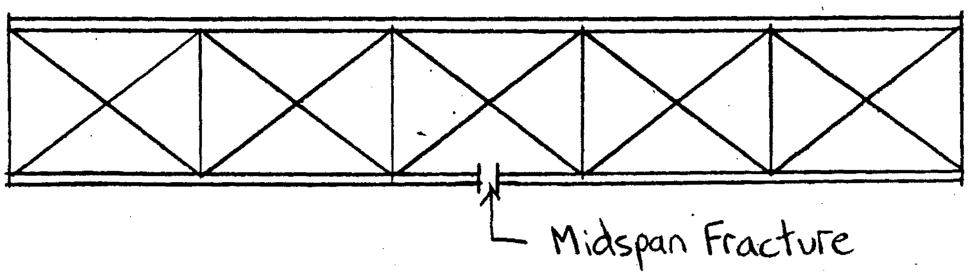


Fig. 4 Typical Bottom Lateral Bracing System Configuration

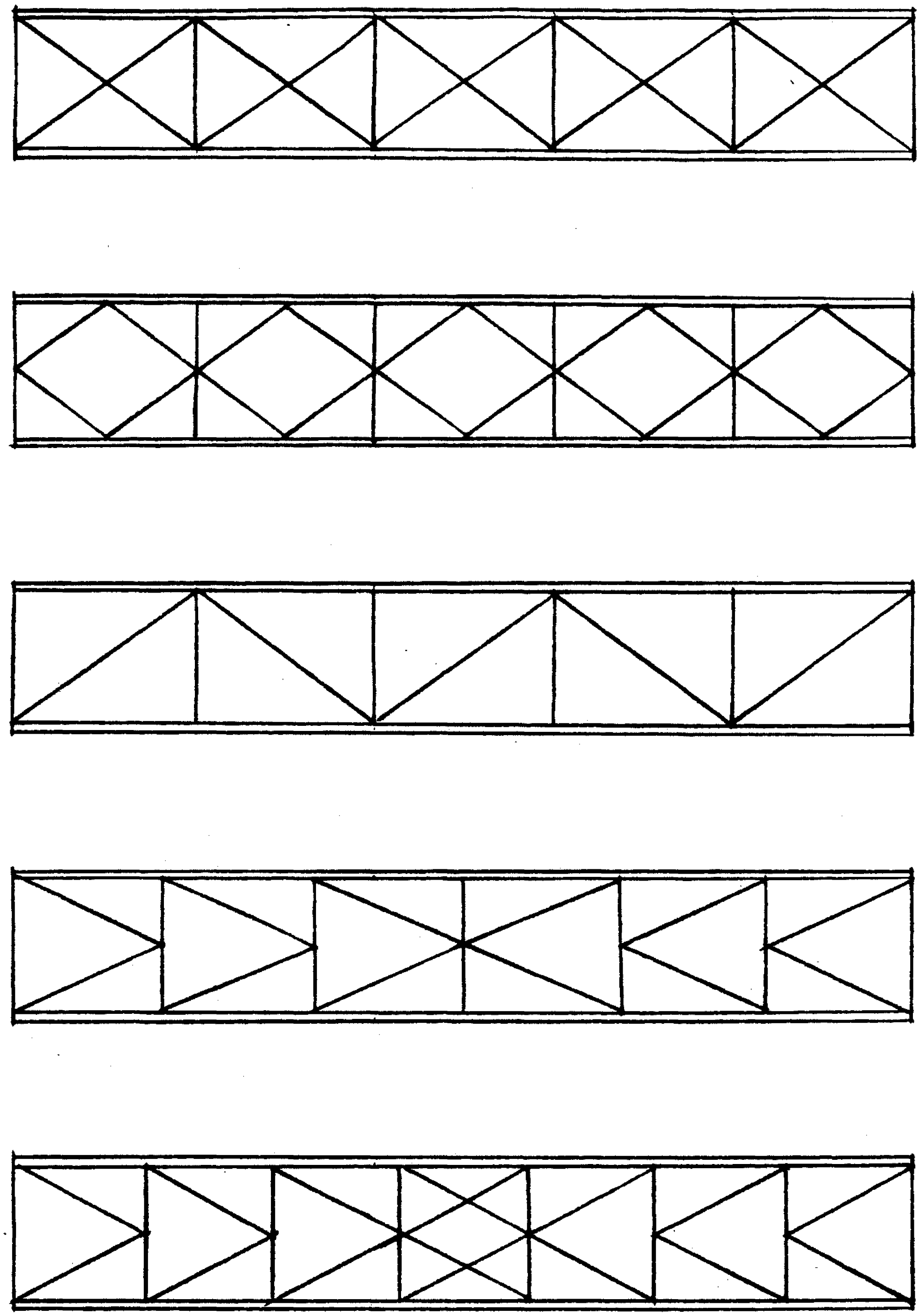


Fig. 5 Typical Variations of Top and Bottom Lateral Bracing Configurations

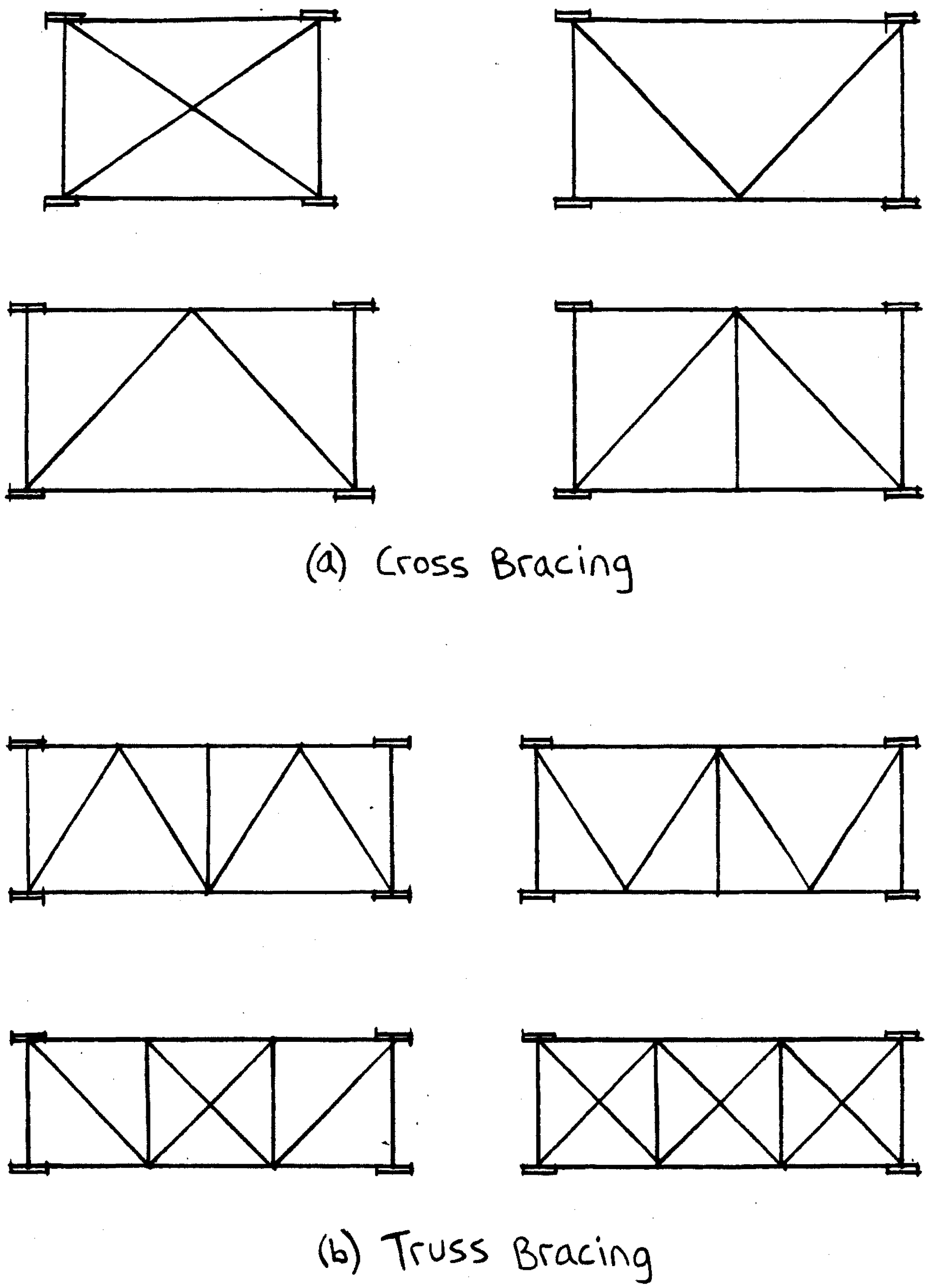
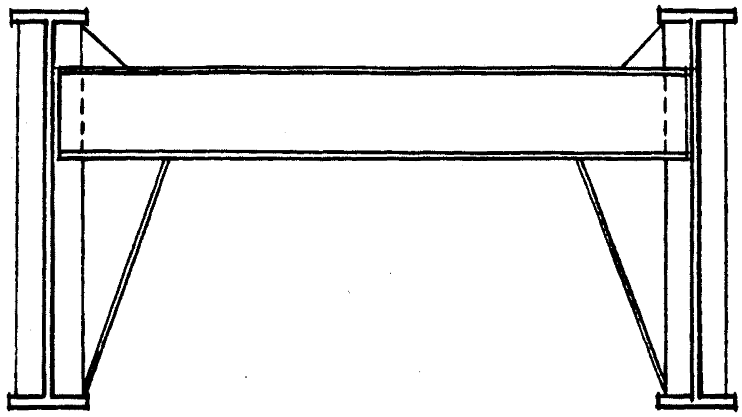
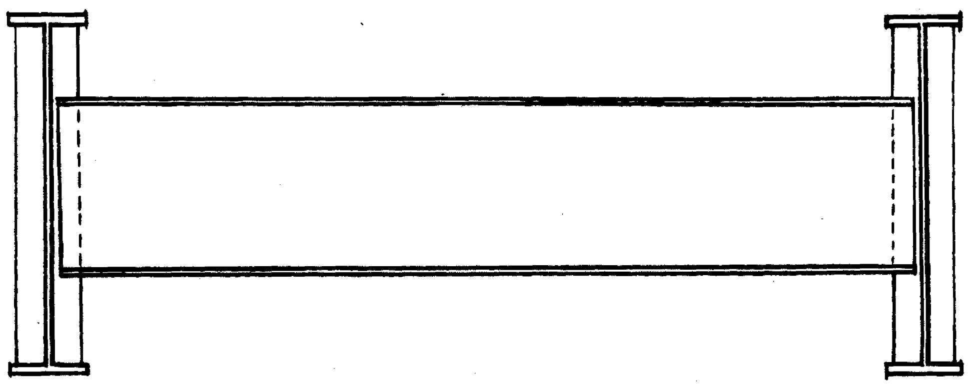


Fig. 6 Typical Cross and Truss Bracing Configurations



(a) Cross Frame



(b) Diaphragm

Fig. 7 Typical Cross Frame and Diaphragm Configurations

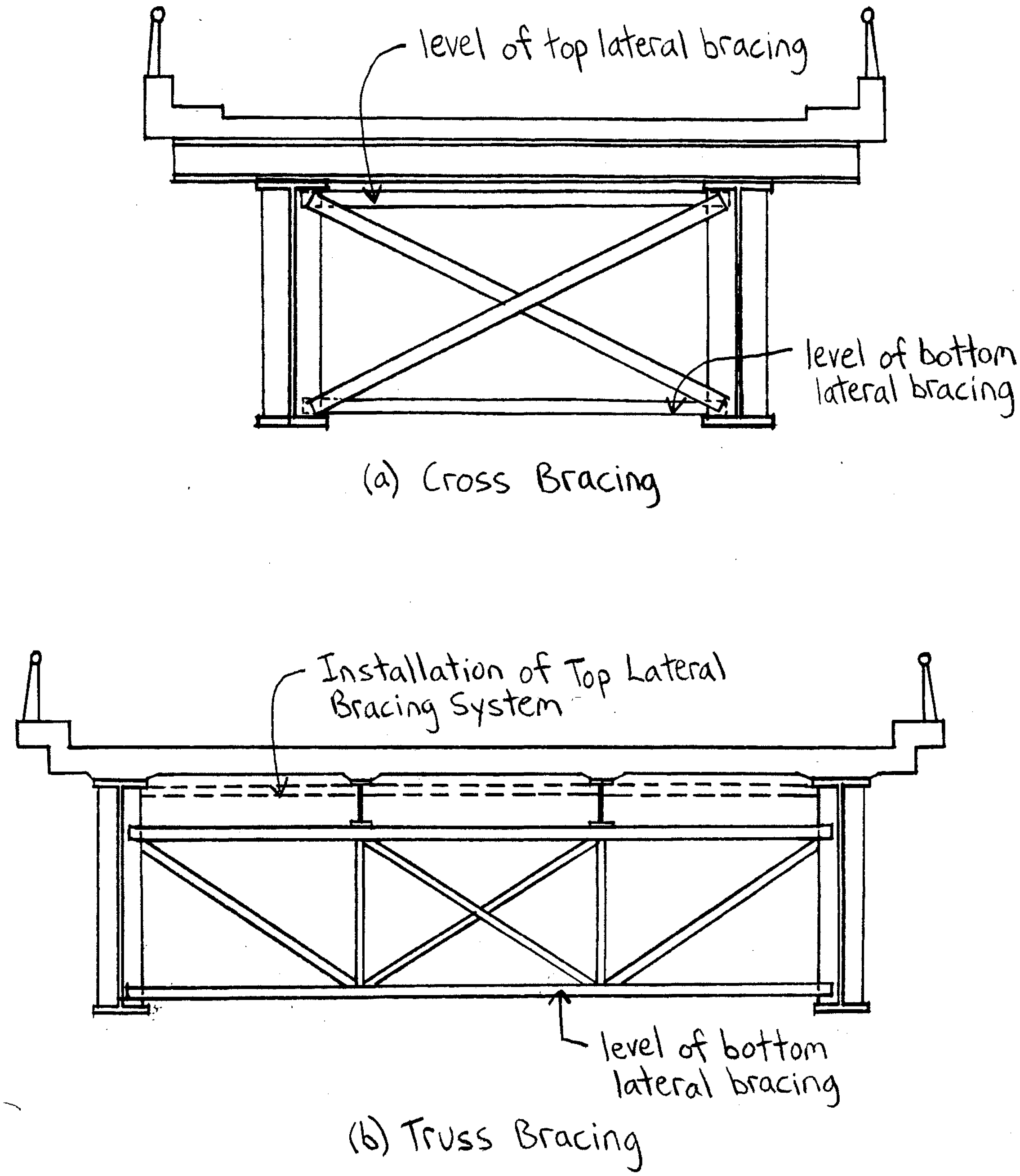


Fig. 8 Examples of Type 1 Bridges

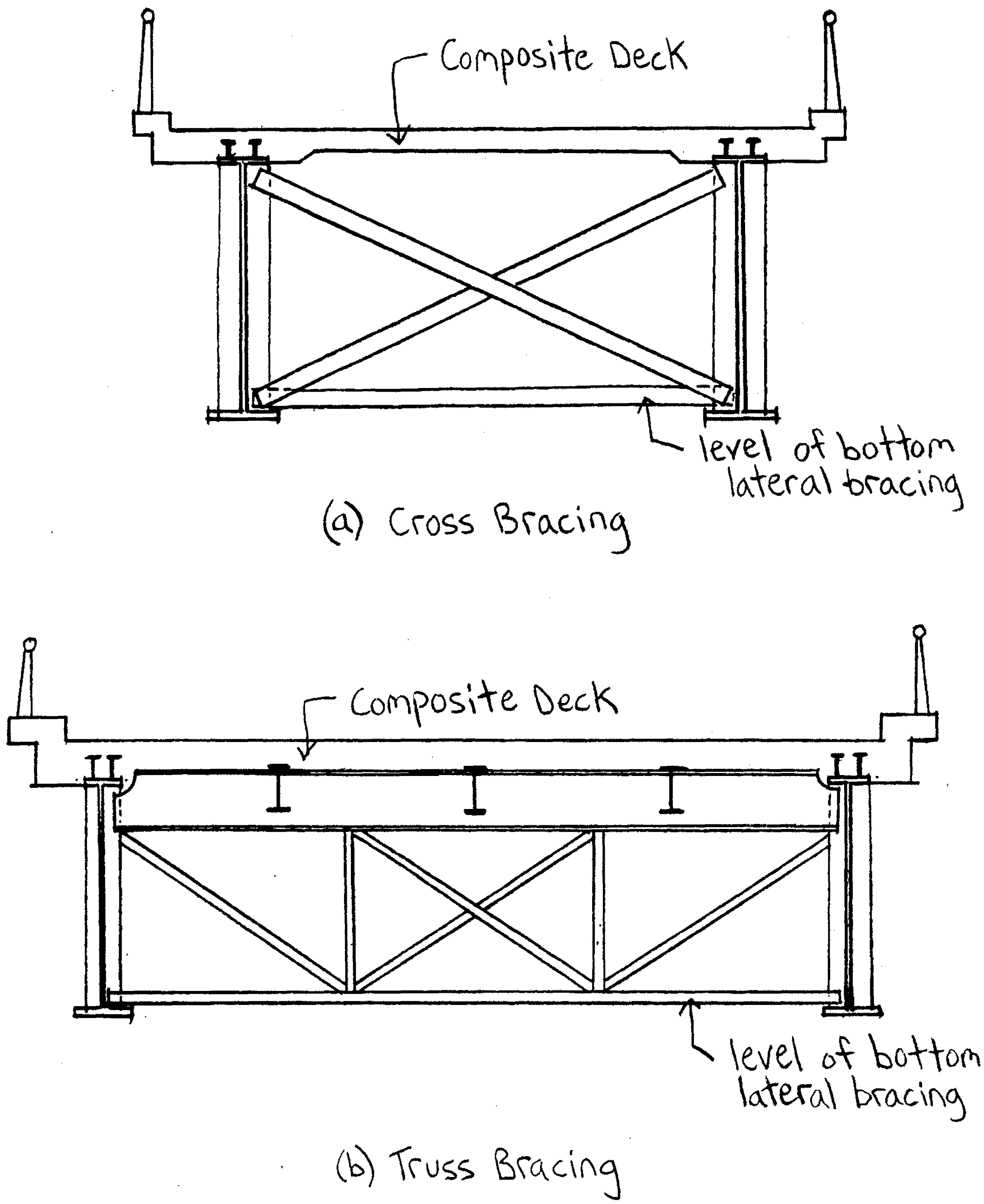
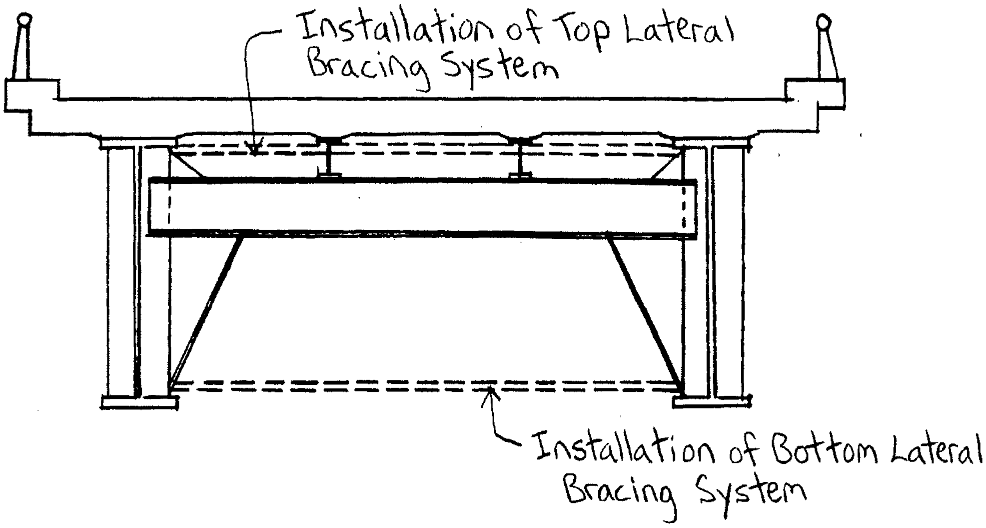
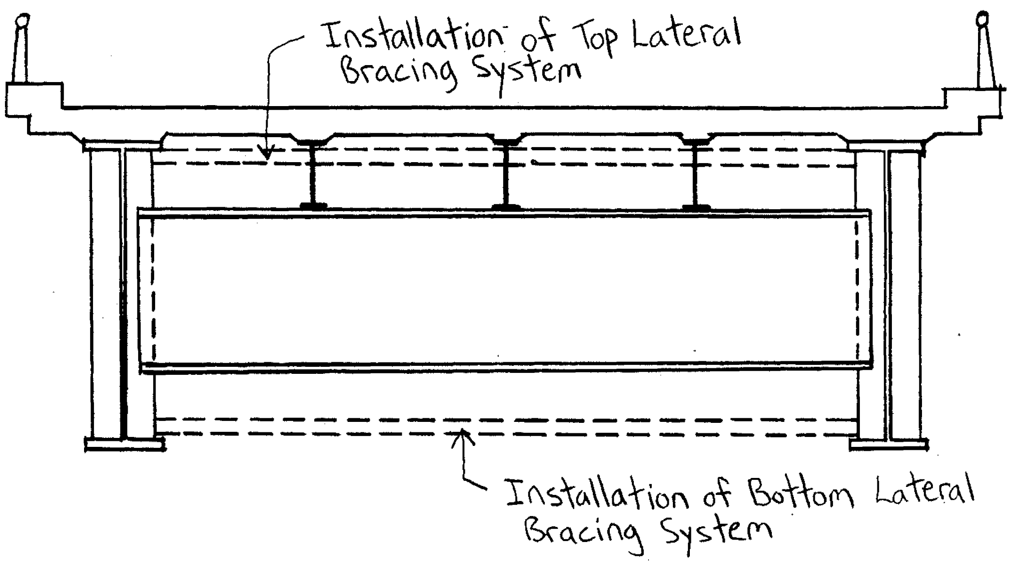


Fig. 9 Examples of Type 2 Bridges

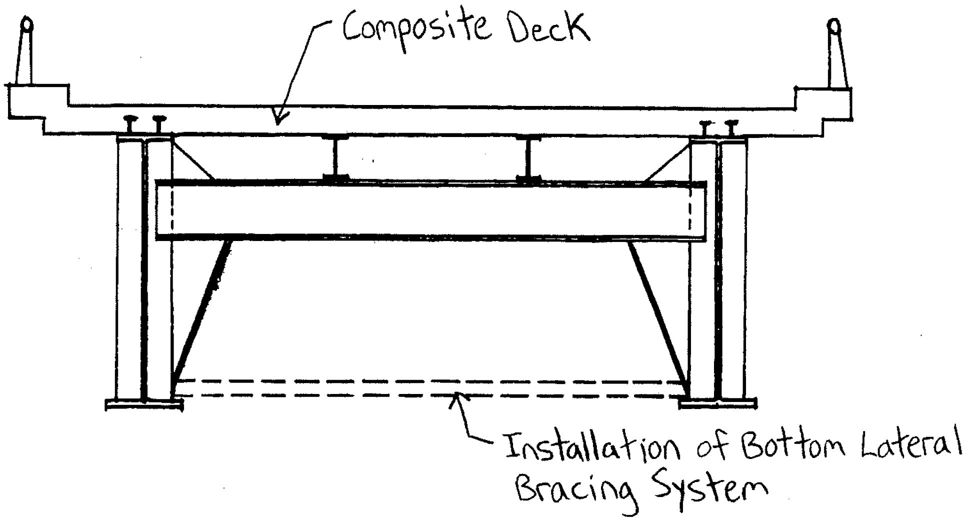


(a) Cross Frame

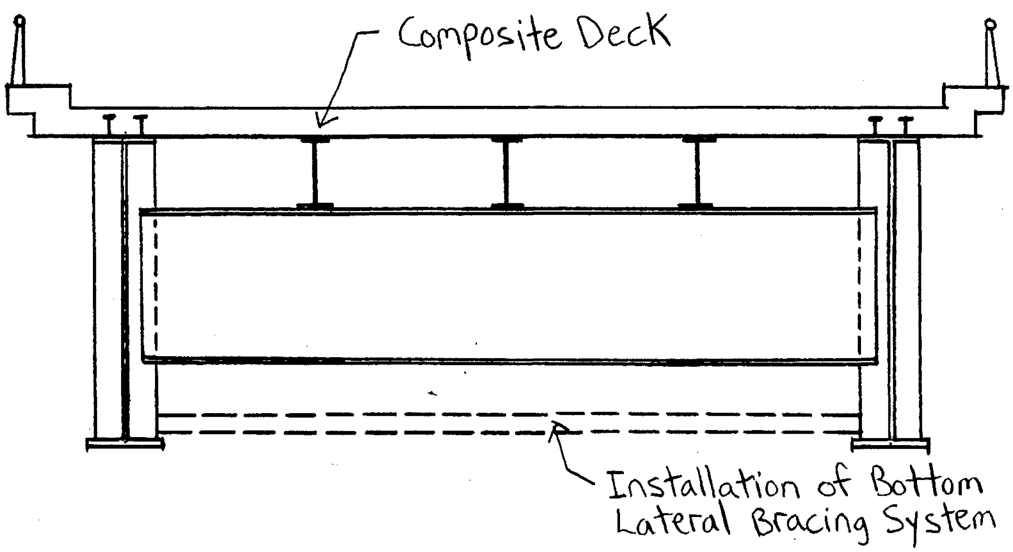


(b) Diaphragm

Fig. 10 Examples of Type 3 Bridges

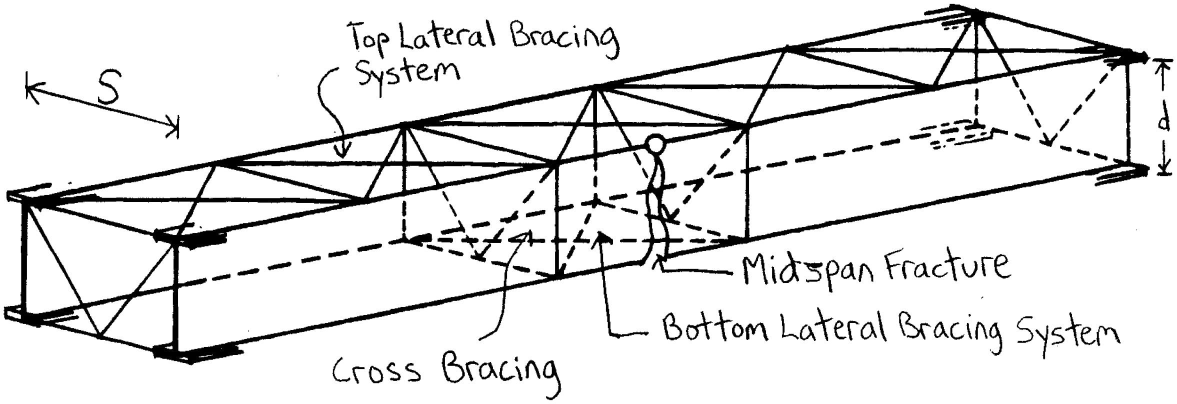


(a) Cross Frame

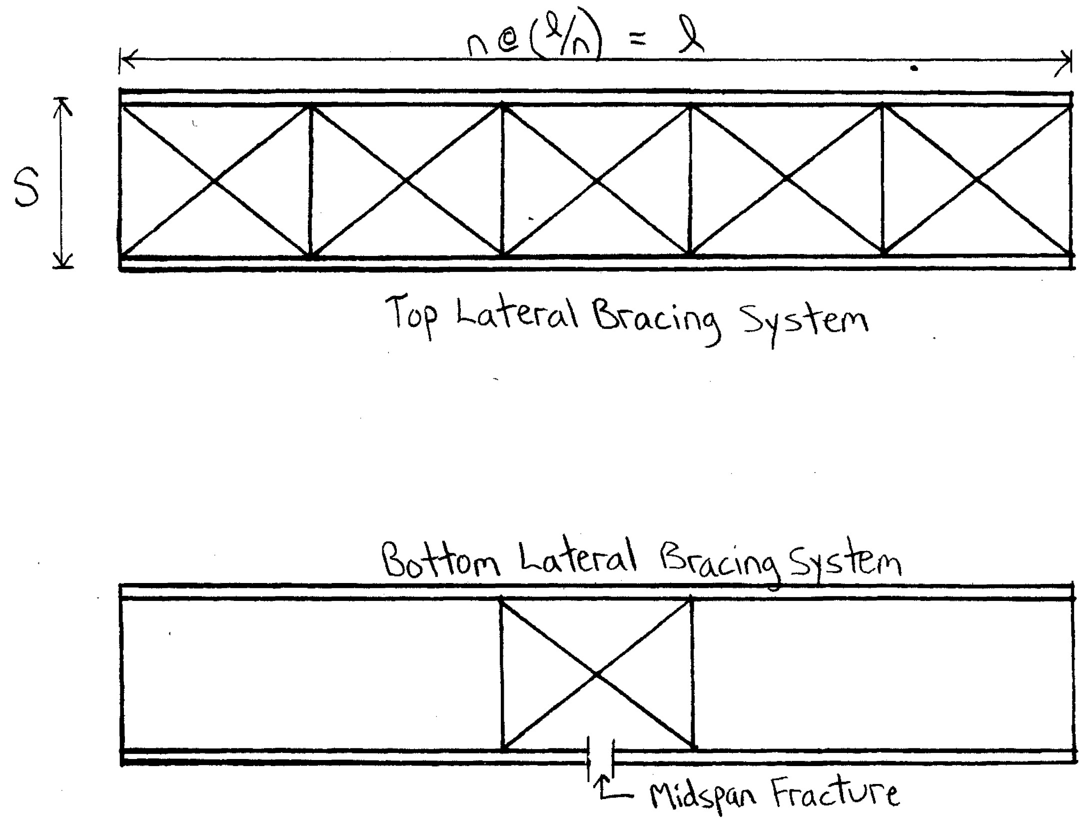


(b) Diaphragm

Fig. 11 Examples of Type 4 Bridges



(a) 3-D bridge with two symmetrically placed planes of interior cross bracing



(b) Top and Bottom Lateral Bracing

Fig. 12 3-D Bridge Model for Single Panel Concept

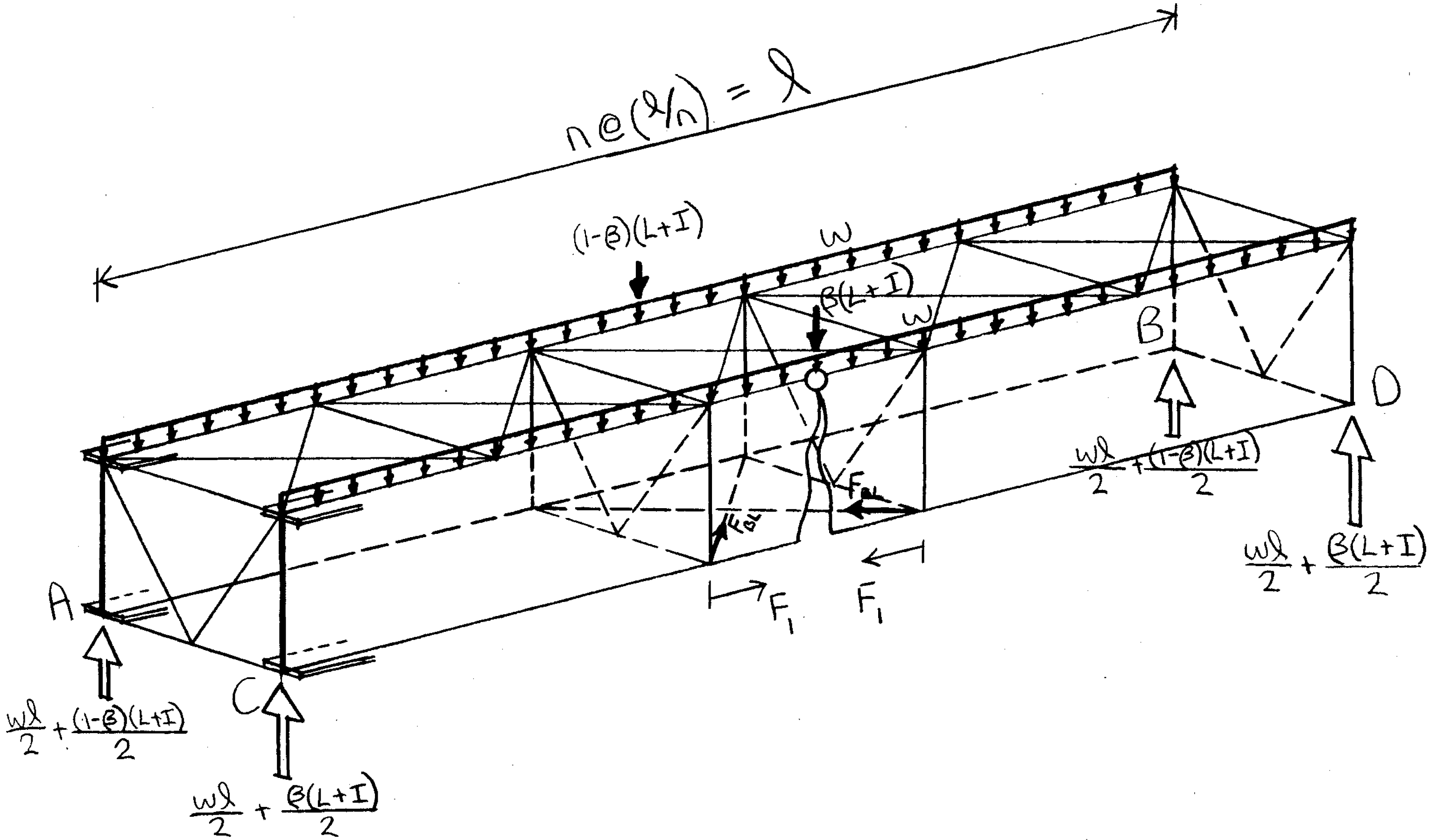


Fig. 13 Loads and Reactions Acting on the 3-D Bridge Structure

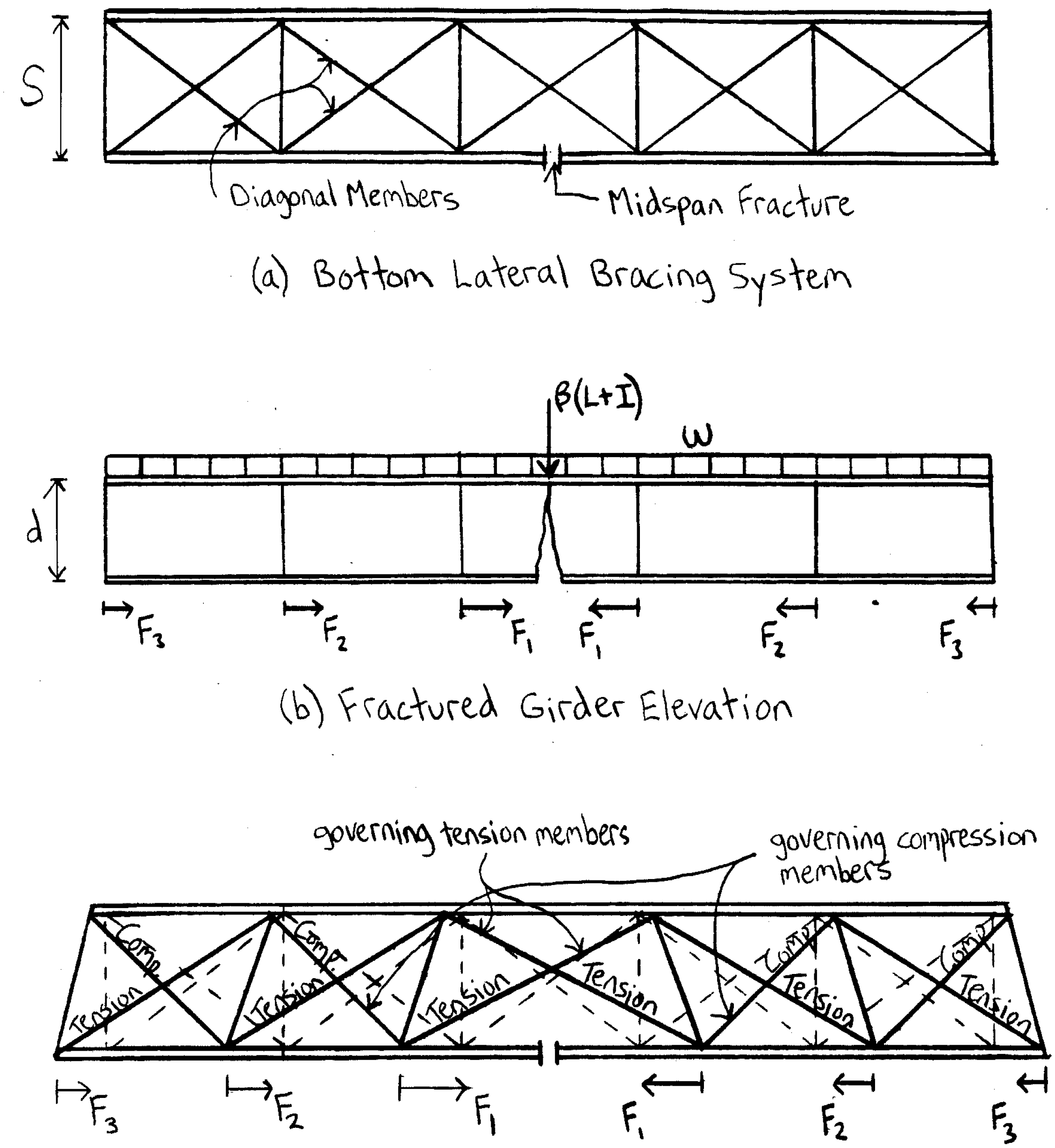
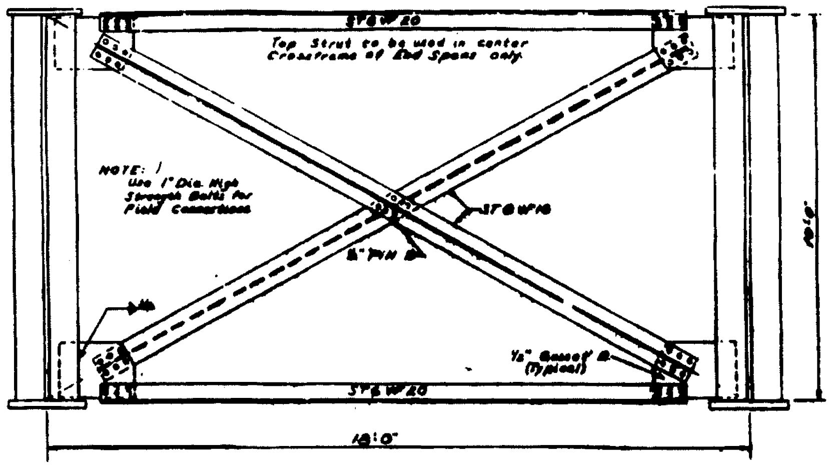
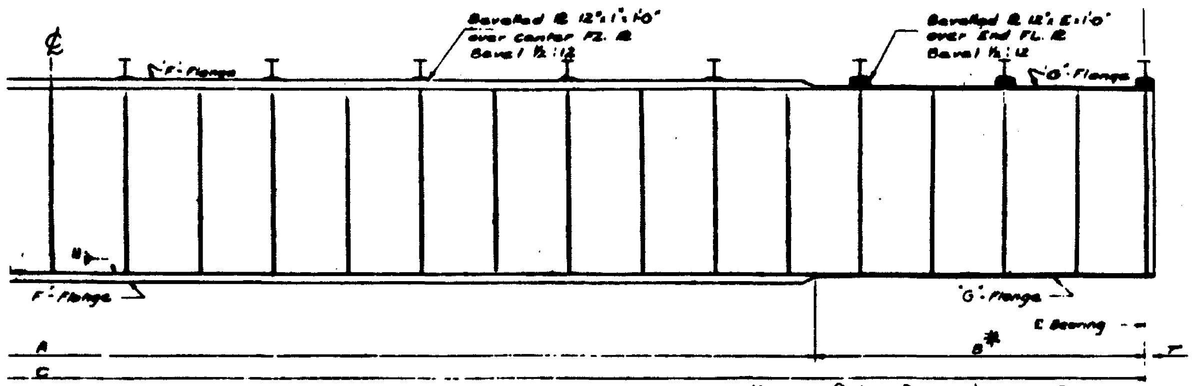


Fig. 14 Bridge with Five Panels of Bottom Lateral Bracing

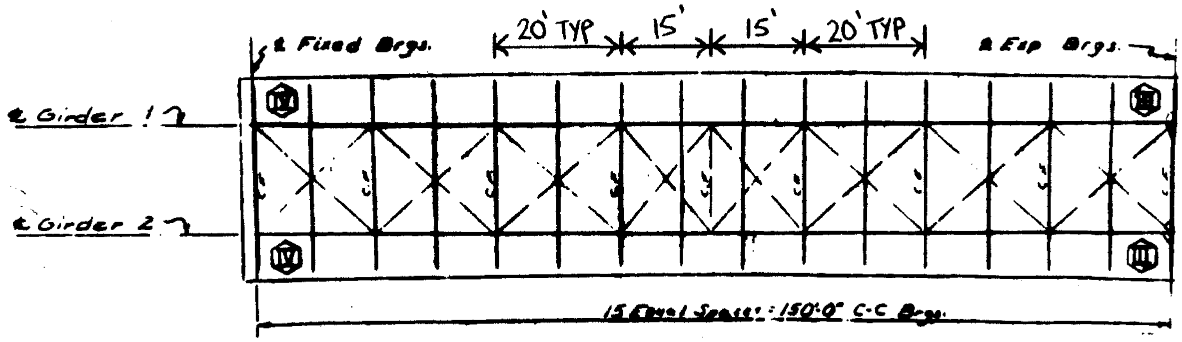


(a) Cross Section



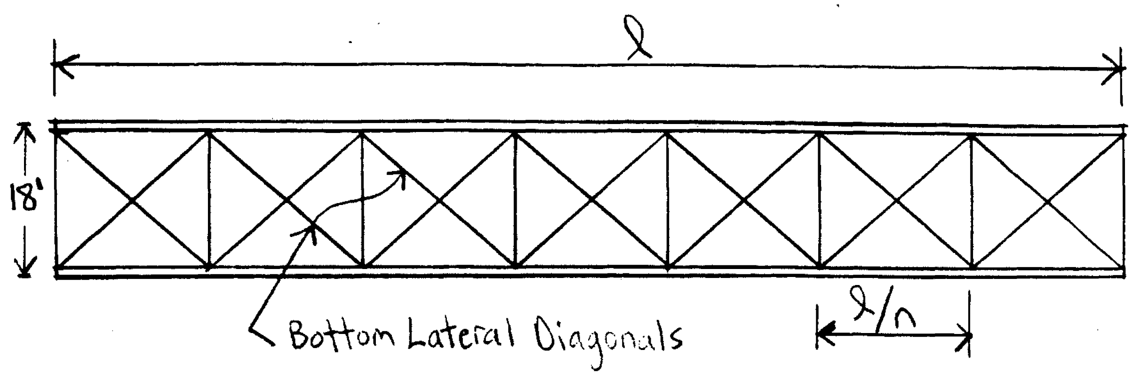
(b) Elevation

*B = 3/4 for the Ref. 7 bridge used in the study

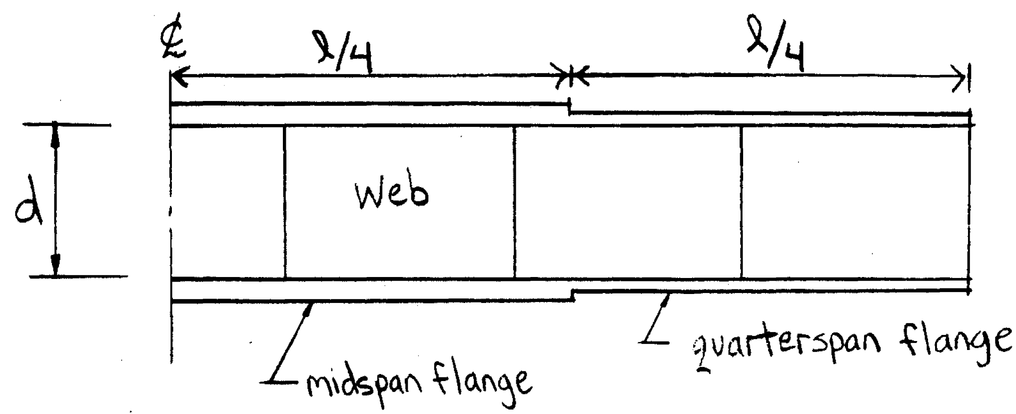


(c) Top and Botom Lateral Bracing

Fig. 15 Details of the Bridge in Ref. 7



(a) Top and Bottom Lateral Bracing



(b) Elevation of Girder

Bridge Span, ℓ	100 ft		150 ft		200 ft	
Number of Panels, n	5	7	7	9	9	13
Girder depth, d	80"		120"		160"	
Flanges, midspan	18"x2.5"		22"x2.75"		25"x3.0"	
quarterspan	18"x1.875"		22"x2.0"		25"x2.25"	
Web	80"x0.5"		120"x0.75"		160"x1.0"	
A_{BLB}, (in2)	9.36	8.00	9.74	8.54	9.95	8.54
	18.72	15.99	19.47	17.07	19.89	17.07
	37.44	31.98	38.94	34.14	39.78	34.14

(c) Member Properties for Computer Study Bridges

Fig. 16 Details of Computer Study Bridges

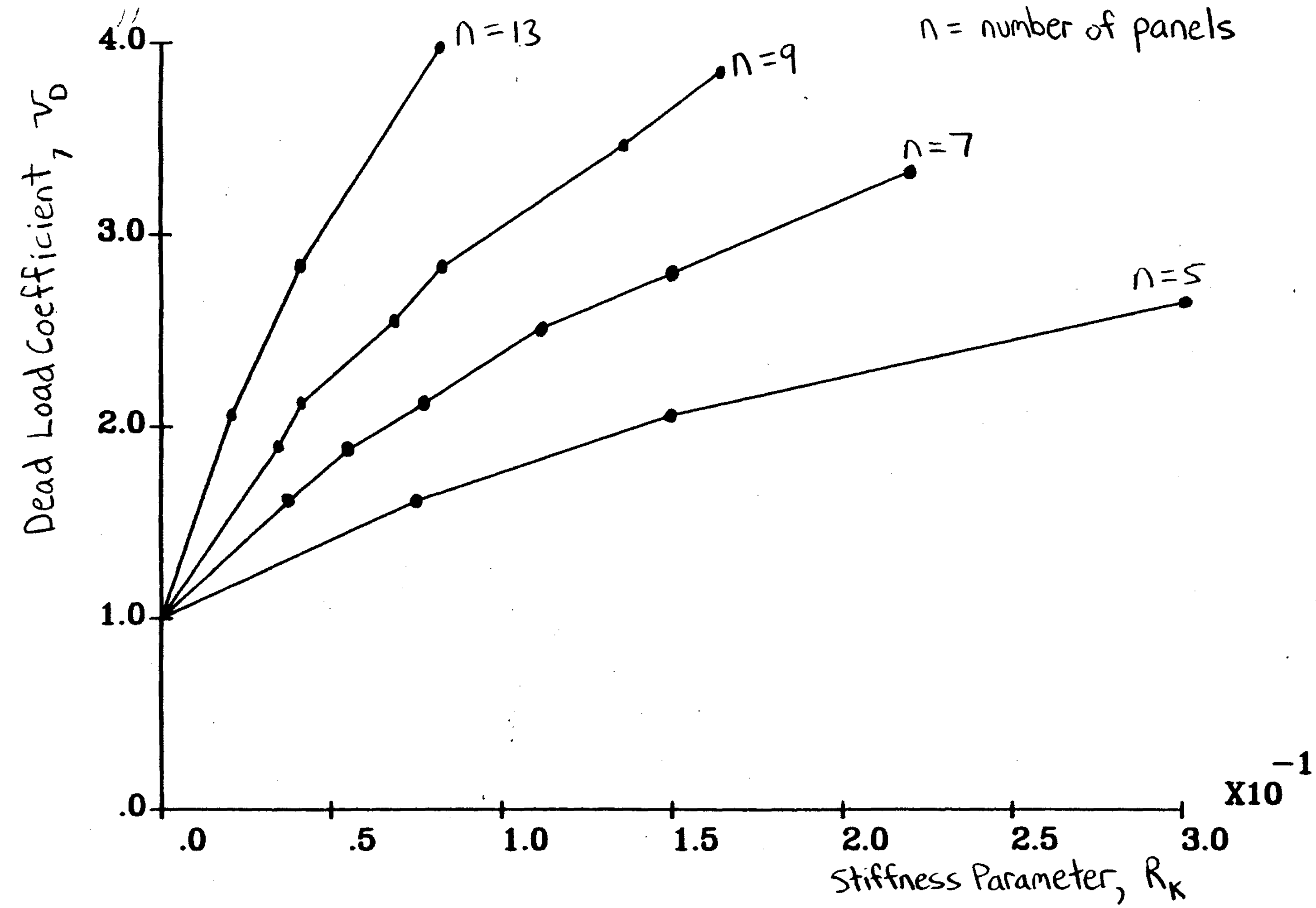


Fig. 17 V_D vs R_k for Different Number of Panels, n.

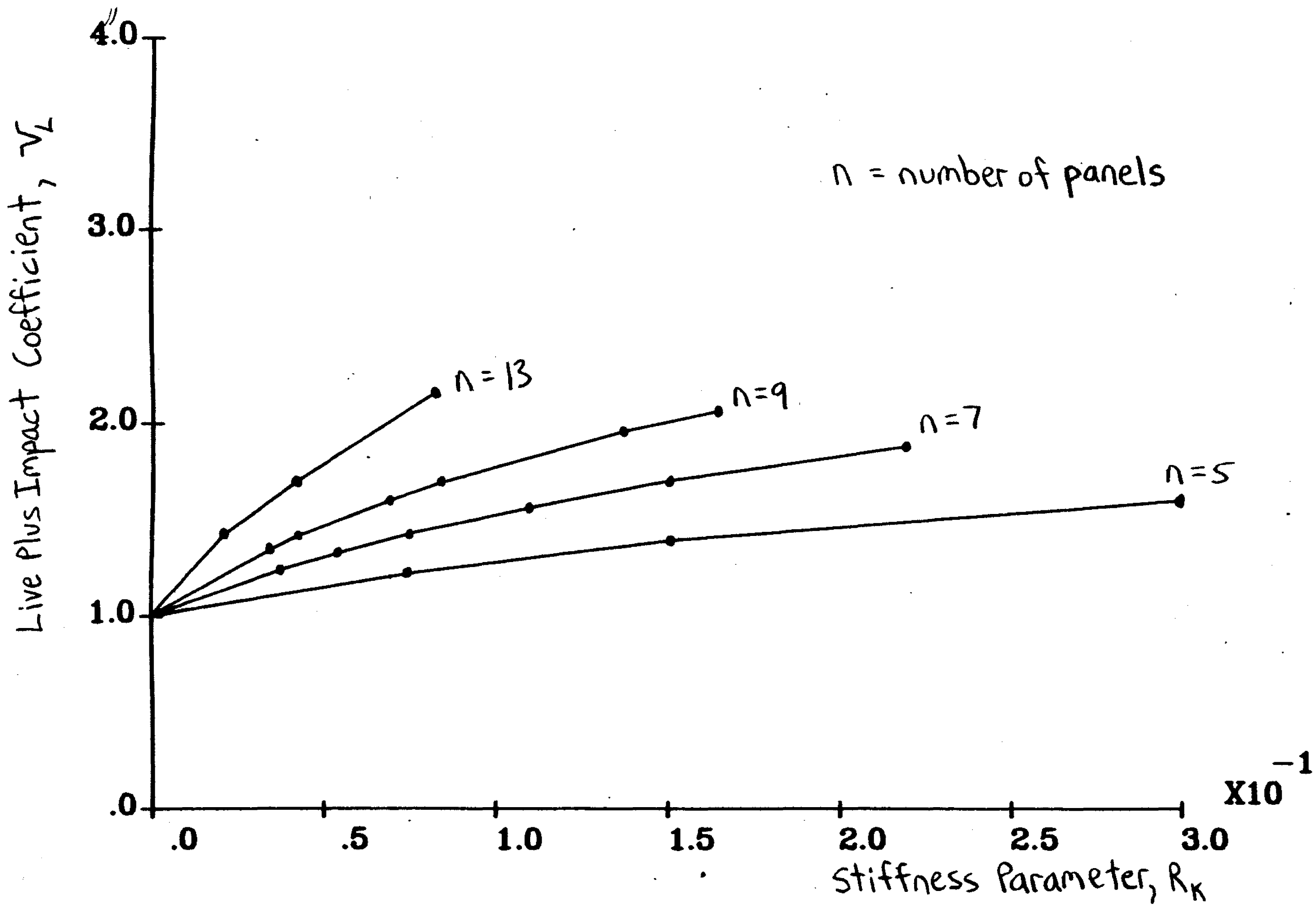


Fig.18 V_L vs R_k for Different Number of Panels, n.

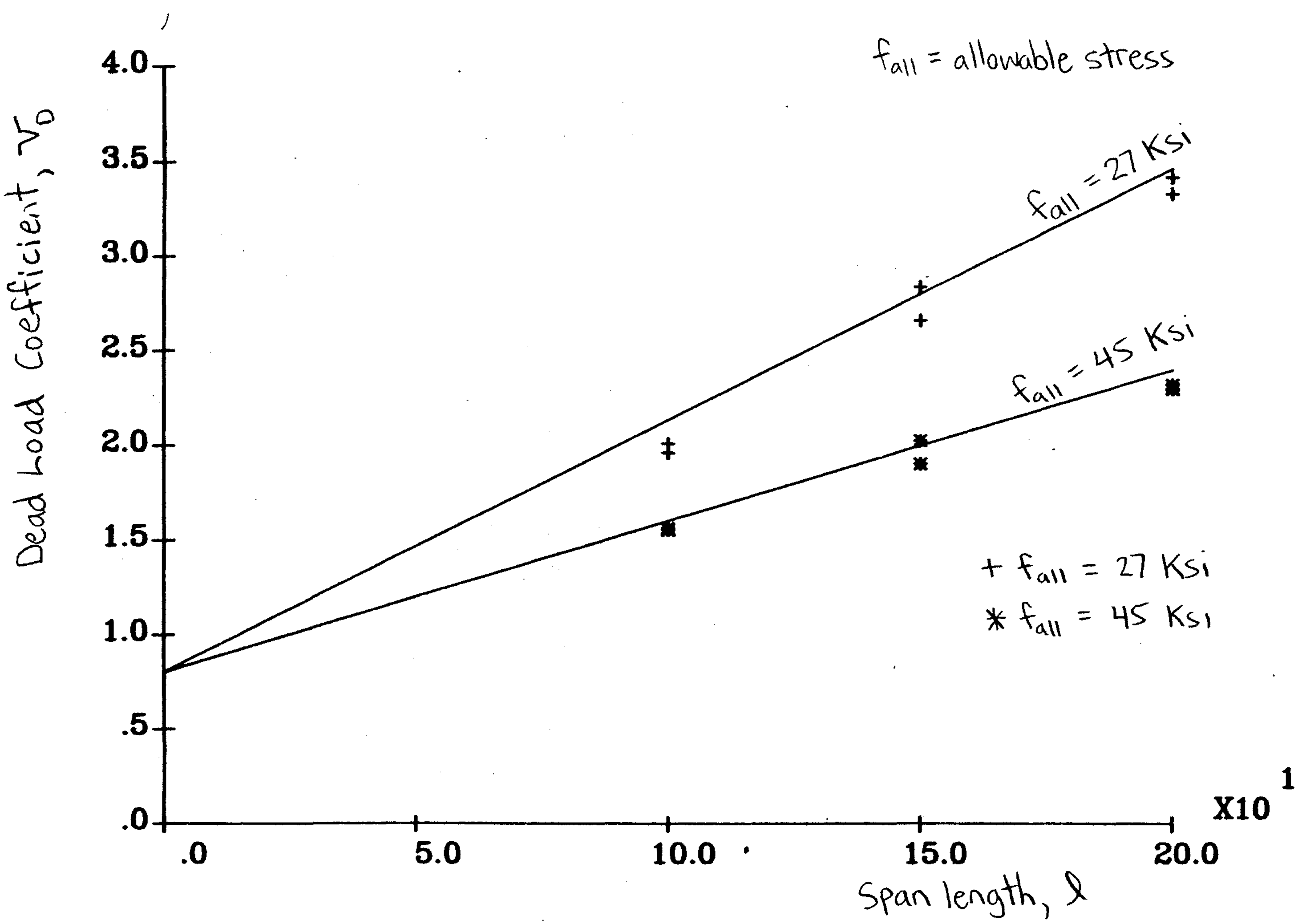


Fig. 19 γ_D vs ℓ for Different Allowable Stresses

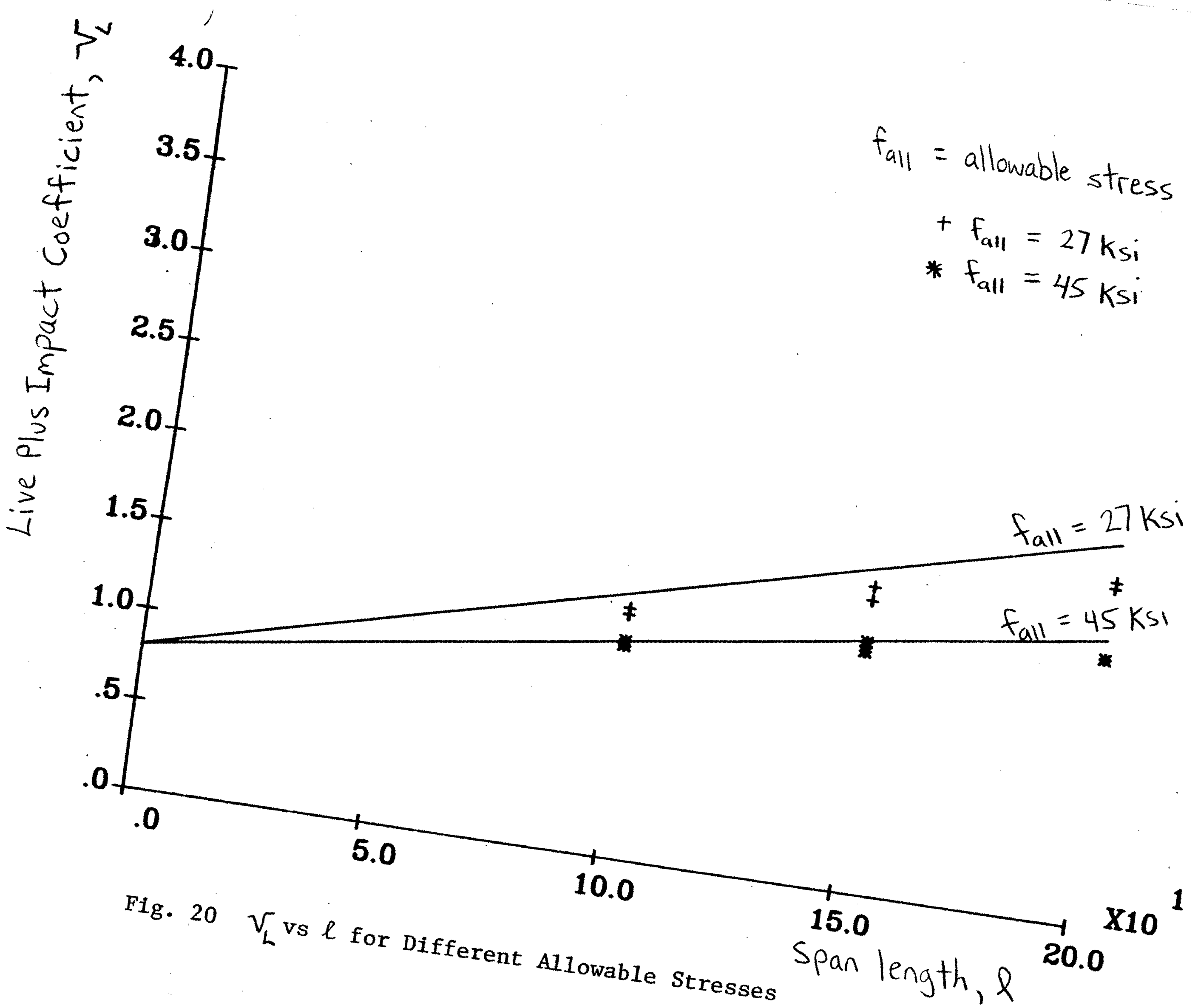
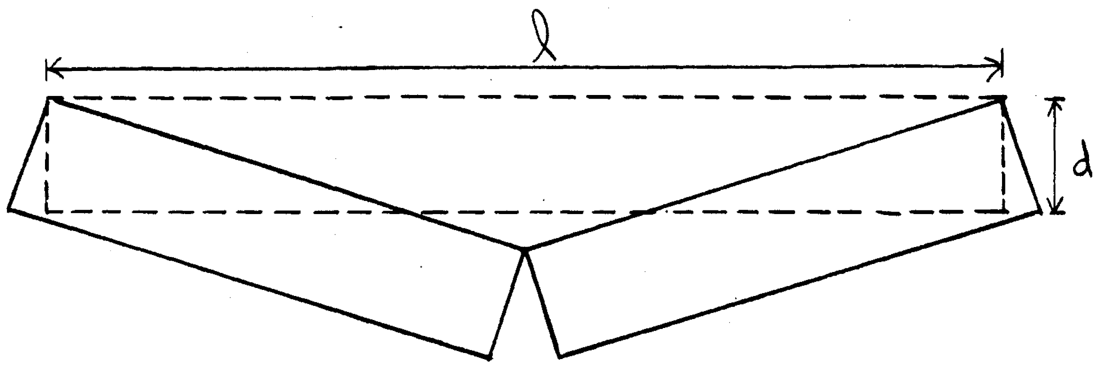
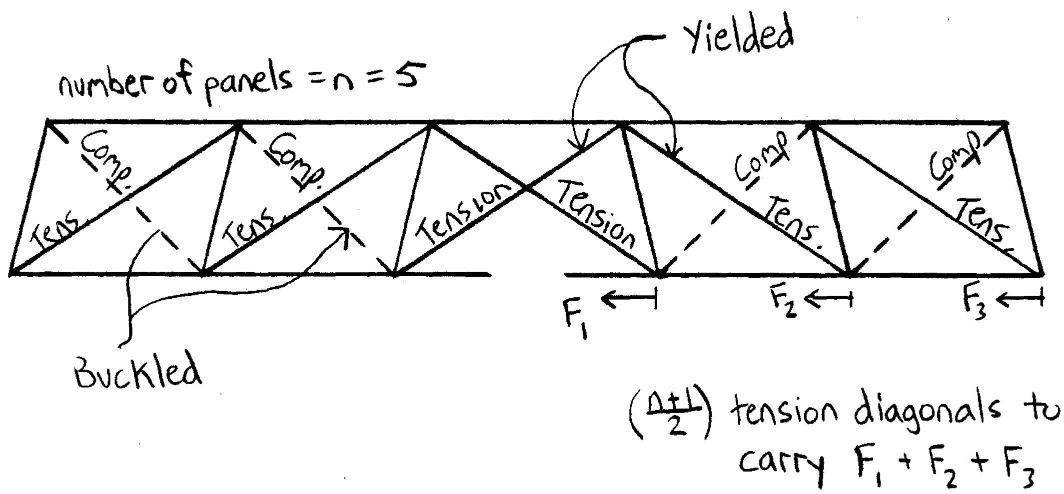


Fig. 20 ν_L vs ℓ for Different Allowable Stresses



(a) Fractured Girder Elevation



(b) Bottom Lateral Bracing

Fig. 21 Model for the Load Factor Method

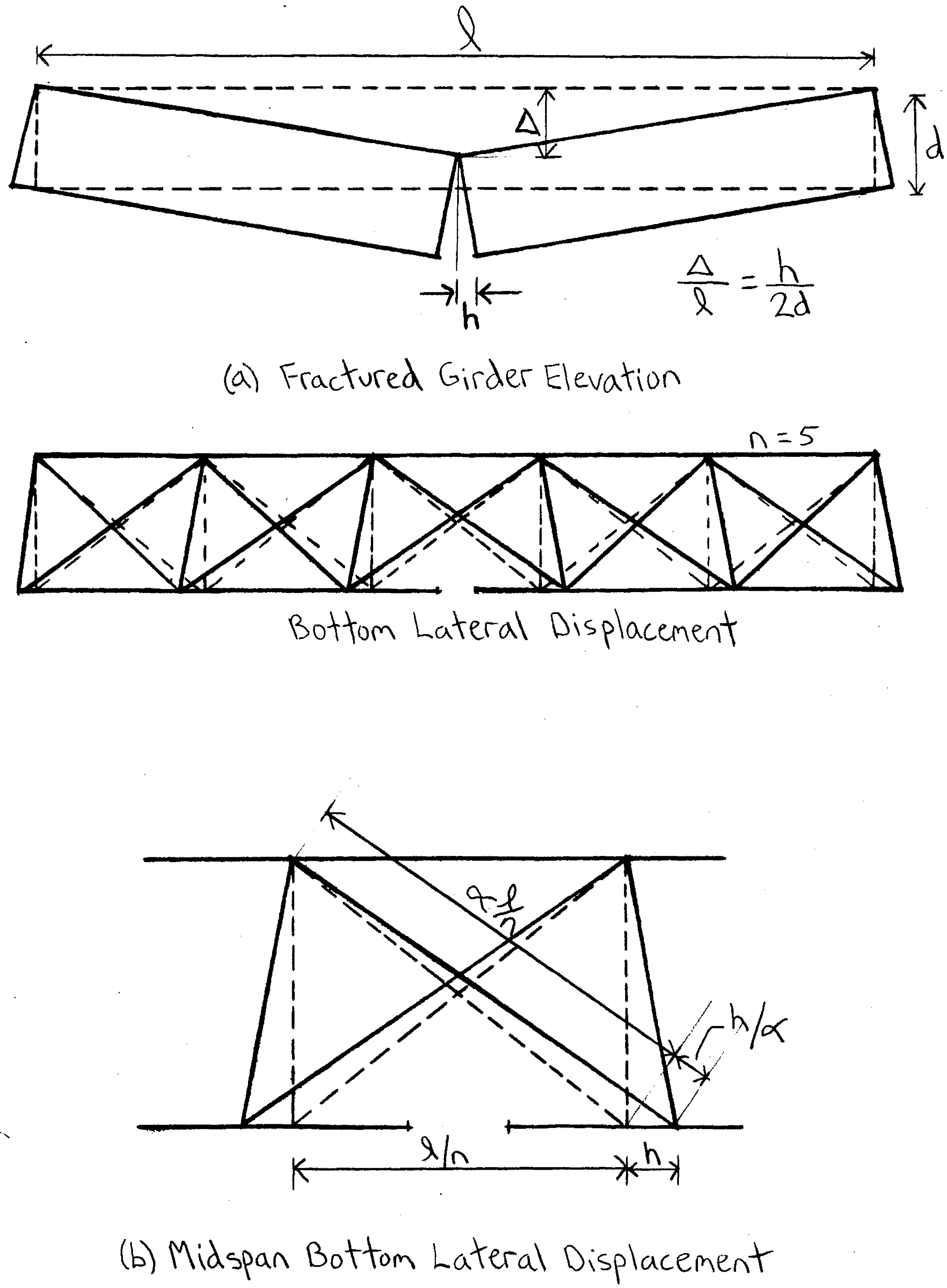


Fig. 22 Displacement Relationships for the Fractured Girder and Bottom Lateral Bracing

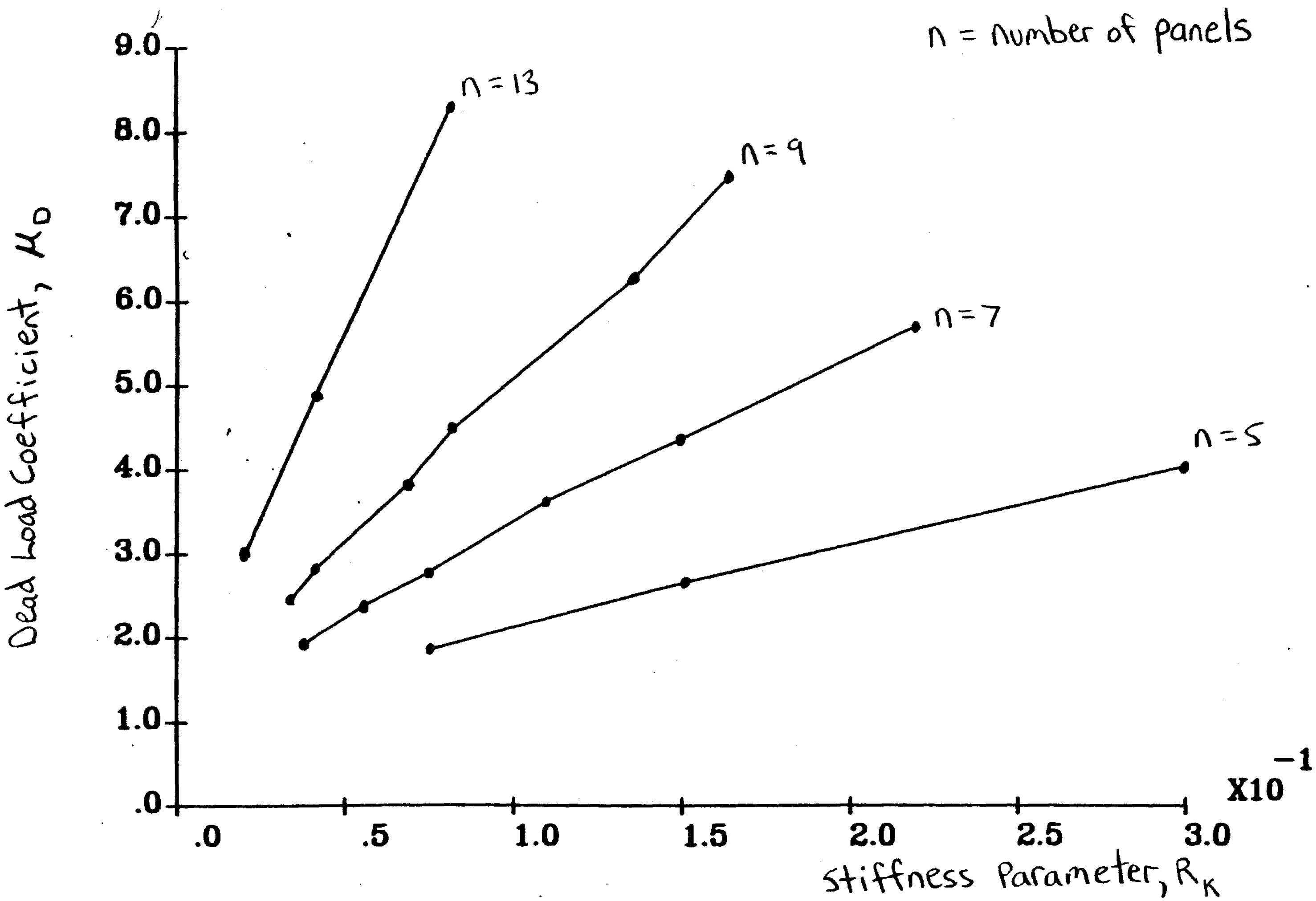


Fig. 23 μ_D vs R_k for Different Number of Panels, n.

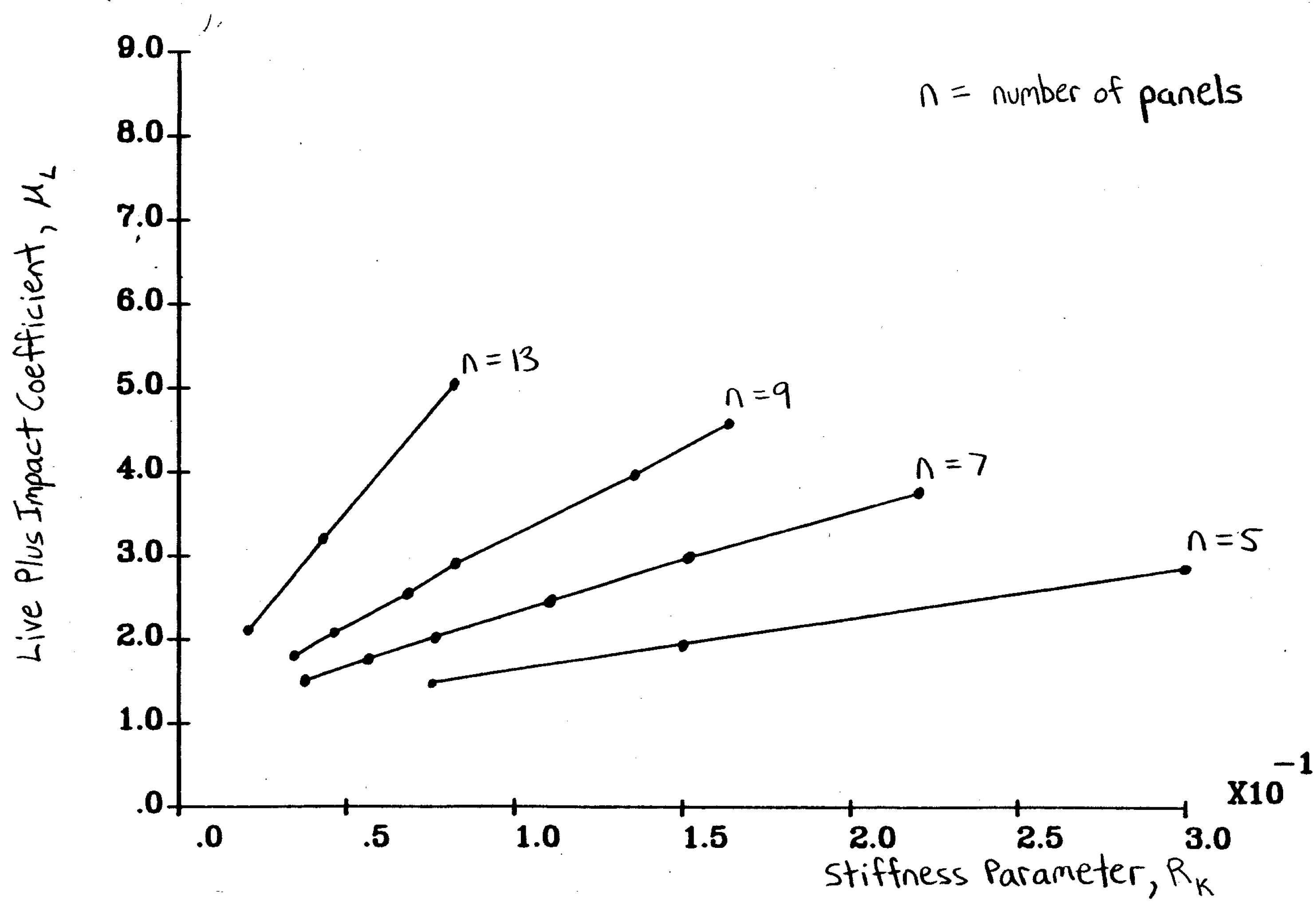


Fig. 24 μ_L vs R_k for Different Number of Panels, n.

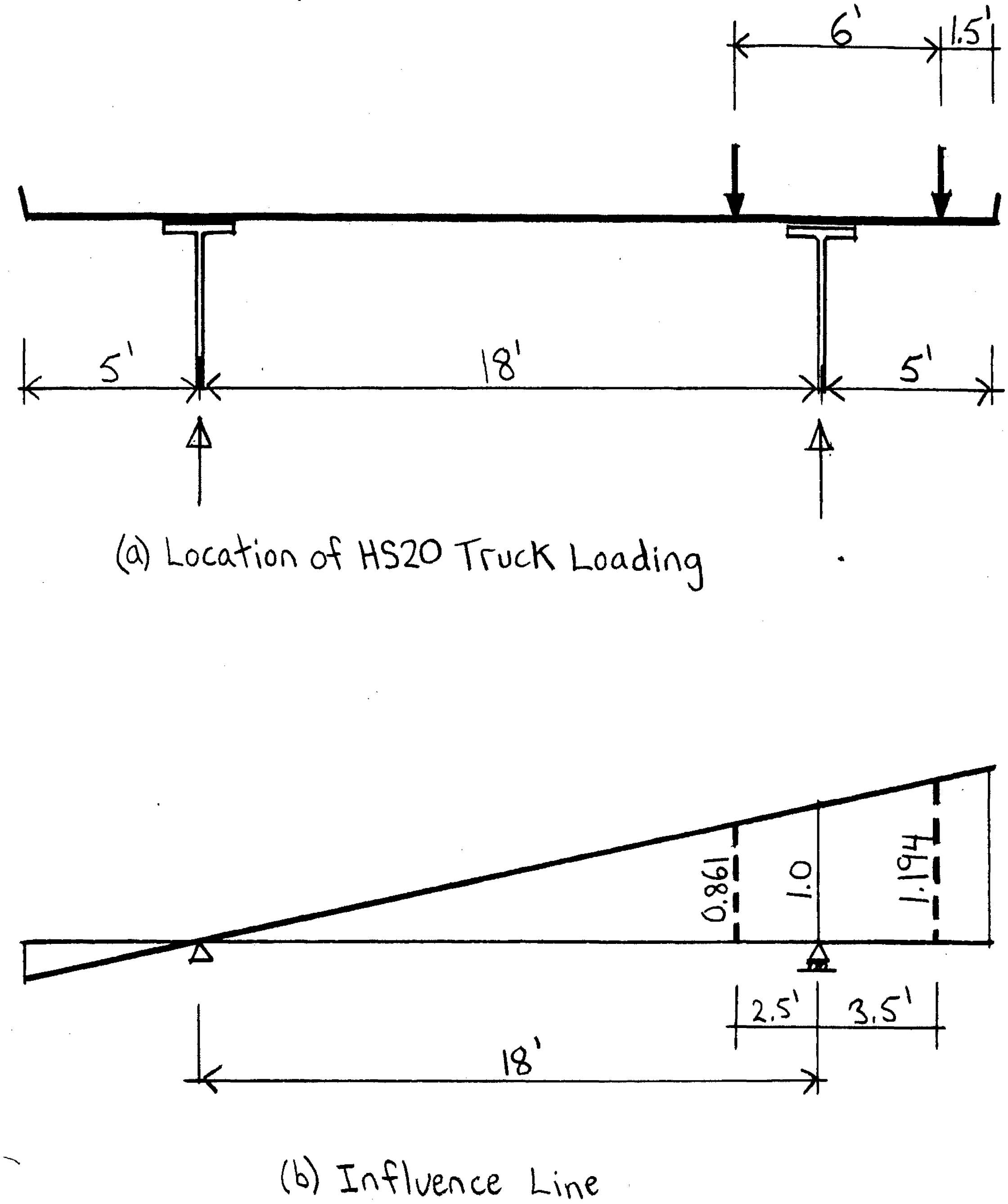
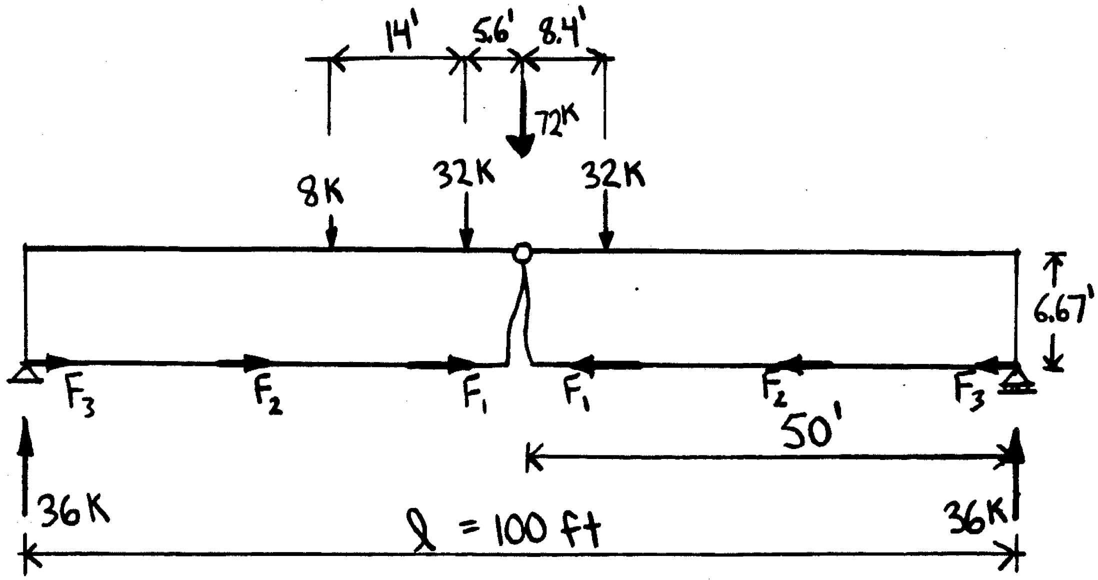
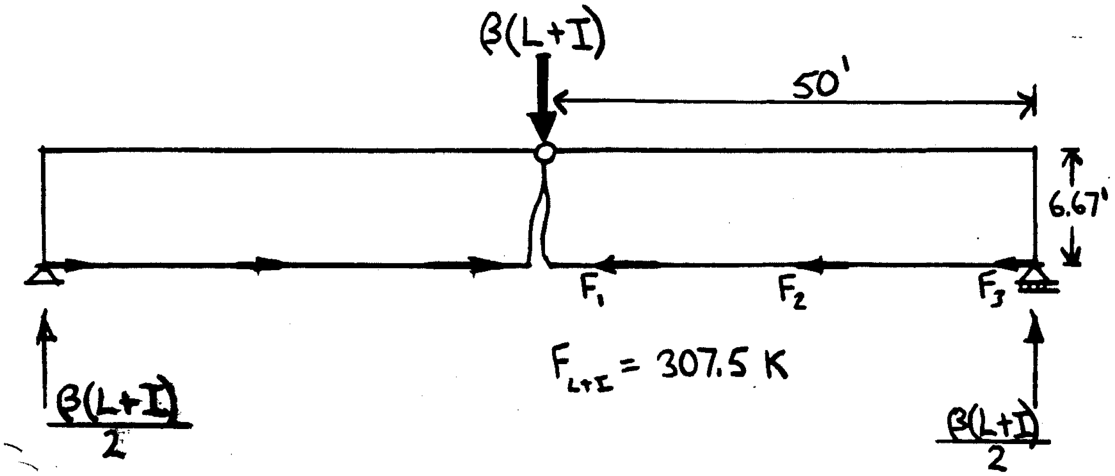


Fig. 25 Fraction of Live Load, β, Acting on the Fractured Girder.



(a) One lane of HS20 truck axle loading applied to the fractured girder



(b) Axle loads replaced by an equivalent concentrated load, β(L+I)

Fig. 26 Equivalent Concentrated Live Plus Impact Load, $\beta(L+I)$.

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