Computer Analysis of AASHTO Plate Girders in Pure Bending with Load-Factor Design

PAUL G. NORTON

ABSTRACT

A computer program has been developed to perform the calculations to analyze AASHTO plate-girder cross sections in pure bending with load-factor design. The program is based on an extensive search of current design specifications and will analyze a straight girder or curved girder with hybrid or homogeneous steel elements. The procedure used by this computer program to analyze plate girders is described. Several interpretations of the AASHTO specifications made during the development of this computer program are described.

A computer program has been developed to calculate the stresses and allowable stresses to analyze steel plate-girder cross sections in pure bending by using the AASHTO load-factor design method. The program module, located in an IBM 4341 computer, operates in the interactive mode under the IBM conversational-mode system environment. Currently plans are under way to incorporate the program to run on an IBM PC/XT microcomputer.

The program listing is based on an extensive search for current design considerations (1-6). It was necessary to refer to these reports to determine the assumptions made in the AASHTO allowable-stress formulas. For example, the basic allowable compression-flange stress for a straight girder is based on a prismatic flange within the unbraced length. In continuous bridges, the compression flange is often nonprismatic within the unbraced length. A U.S. Steel technical report (1) addresses this problem and proposes a design procedure. Discrepancies between the specifications and the research reports have been corrected. Specification modifications have been incorporated into the program where engineering judgement required them.

Input for the program includes basic geometric properties such as diaphragm spacing, radius of curvature, width and thickness of top and bottom flanges, depth and thickness of web, and material properties such as ultimate strength of the concrete slab and yield strength of the web and of each flange. The program's analysis is for pure bending, so all the moments (nonfactored) at the cross section are needed, including not only the normal moments but also the lateral bending moments and the fatigue moments. The allowable fatigue stress and the distance from the extreme tension fiber in the web to the fatigue point under consideration are also required.

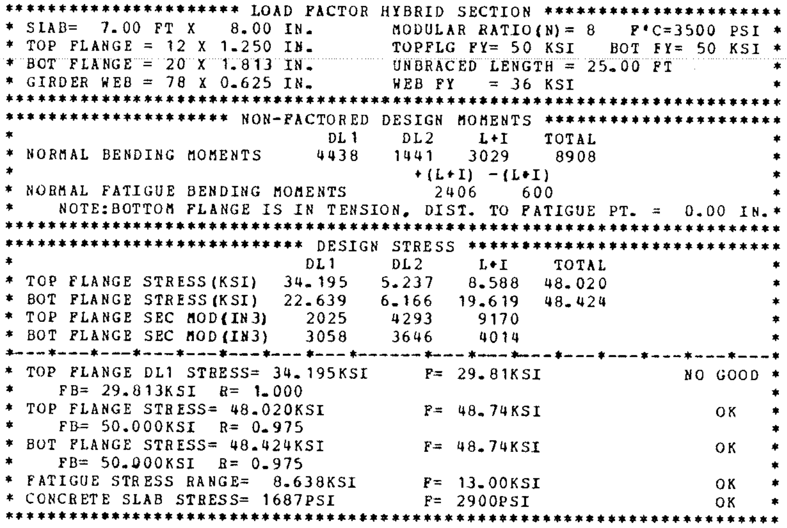
Normal and lateral bending stresses are computed for the dead-load-1 (DL1) and total stress conditions. Normal and lateral fatigue stresses are computed for the live loads.

Special features in the program include (a) identification of compact compression flanges for curved girder sections, (b) specification limits of compression flange width-to-thickness ratios, (c) different yield strengths of the flanges, and (d) an option of composite action in the negative moment regions of continuous bridges.

All the results of the analysis for one cross section are printed on a single 8.5 x 11-in. sheet (Figure 1). Included are all input information, normal and lateral bending stresses, and the allowable bending stresses. Determination of the allowable bending stresses requires calculating the basic allowable bending stress for each flange and, when appropriate, the hybrid reduction factor or the three curvature-reduction factors or both.

PURPOSE AND SCOPE

The AASHTO specifications have evolved from liter-



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********************** LOAD FACTOR HYBRID SECTION **********************
* SLAB=  7.00 FT X   8.00 IN.        MODULAR RATIO(N)= 8   F'C=3500 PSI *
* TOP FLANGE = 12 X 1.250 IN.        TOPFLG FY= 50 KSI   BOT FY= 50 KSI *
* BOT FLANGE = 20 X 1.813 IN.        UNBRACED LENGTH = 25.00 FT         *
* GIRDER WEB = 78 X 0.625 IN.        WEB FY    = 36 KSI                 *
************************************************************************
********************* NON-FACTORED DESIGN MOMENTS **********************
*                                DL1    DL2    L+I   TOTAL              *
* NORMAL BENDING MOMENTS       4438   1441   3029    8908               *
*                                          +(L+I) -(L+I)                *
* NORMAL FATIGUE BENDING MOMENTS             2406    600                *
*    NOTE:BOTTOM FLANGE IS IN TENSION, DIST. TO FATIGUE PT. =  0.00 IN.*
************************************************************************
*************************** DESIGN STRESS ******************************
*                             DL1      DL2      L+I    TOTAL            *
* TOP FLANGE STRESS(KSI)   34.195   5.237   8.588   48.020              *
* BOT FLANGE STRESS(KSI)   22.639   6.166  19.619   48.424              *
* TOP FLANGE SEC MOD(IN3)    2025    4293    9170                       *
* BOT FLANGE SEC MOD(IN3)    3058    3646    4014                       *
*---*---*---*---*---*---*---*---*------*---*---*---*---*---*---*---*---*
* TOP FLANGE DL1 STRESS= 34.195KSI      F= 29.81KSI          NO GOOD *
*    FB= 29.813KSI  R= 1.000                                         *
* TOP FLANGE STRESS= 48.020KSI          F= 48.74KSI               OK *
*    FB= 50.000KSI  R= 0.975                                         *
* BOT FLANGE STRESS= 48.424KSI          F= 48.74KSI               OK *
*    FB= 50.000KSI  R= 0.975                                         *
* FATIGUE STRESS RANGE=  8.638KSI       F= 13.00KSI               OK *
* CONCRETE SLAB STRESS= 1687PSI         F= 2900PSI                OK *
************************************************************************
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FIGURE 1 Sample output.

ally hundreds of research reports published by scholars from industry and from prestigious engineering schools. The AASHTO specifications are guide specifications and are meant to serve as a reference for bridge engineers (1,2). They are analogous to the laws of the United States in that bridge engineers must interpret the intent of the words that make up the specification just as lawyers and judges must interpret the laws. As stated in NCHRP Synthesis 23 (8, p. 13): "Squanto showed the Pilgrims how to plant corn. The Pilgrims survived. Had Squanto written them a set of instructions [specifications], the outcome may have been quite different." Attempting to correctly assess the intent of the AASHTO specifications is not a life-or-death struggle, but the specifications can be misinterpreted by the practicing bridge engineer.

Unlike the codes of the American Institute of Steel Construction and the American Concrete Institute, the AASHTO code does not have instructive textbooks, engineering handbooks, and expanded-commentary books dealing with it. Both industry and university researchers are busy developing new concepts and ideas. The intent of the AASHTO specifications is left to the interpretation of the bridge engineer.

The intent in this paper is to share some ideas on what are perceived to be the intent of the specifications and to describe the procedure used by a specific computer program to analyze plate girders. The program's scope is limited to welded plate girders. The program assumes an AASHTO group 1 loading. Plates that are grade 36 or grade 50 are the acceptable materials. In addition, the analysis is independent of the action of shear.

COMPUTATION OF STRESS

DL1 Normal Stress

The stress due to the DL1 moment is computed for the compression flange by using the moment-of-inertia method. The tension flange is not a design consideration at this loading stage, because the total normal or total tip stress would control. However, a check of the DL1 normal stress is necessary for the top compression flange of a composite section. The fullest use of the flange steel will yield a final stress close to the allowable stress for each flange. Because the partly composite and the fully composite sections have their neutral axes closer to the top flange than the noncomposite section, most of the stress in the top flange is from DL1. The combination of a high DL1 stress and low allowable stress can control the size of the top flange. For a curved girder the top flange stress for the DL1 loading stage is even more critical. Therefore, the calculation of the DL1 stress is important for the proper design of the top flange of a composite section.

Total Normal Stress

The stress due to the group 1 loading combination 1.3 [DL1 + DL2 + 5/3(L + I)] is computed for the top flange and the bottom flange by using the moment-of-inertia method. Calculation of the total normal stress in the top and bottom flanges is necessary to properly design each flange in composite or noncomposite straight girders or curved girders.

Total Tip Stress

The stress due to nonuniform torsion, lateral flange bending, in a horizontally curved girder is computed for the group 1 loading combination by using the moment-of-inertia method. The total lateral bending stress at the tips of the flange added algebraically to the total normal bending stress results in the maximum and minimum values of the nonuniform stress distribution experienced by the flange. The maximum value is defined as the total tip stress.

The top flange of a composite section must have its lateral bending stress calculated by adding the individual stresses experienced at each loading stage. The flange is attached to the deck slab by shear connectors. This connection results in the partly composite and fully composite horizontal inertia of the slab resisting the lateral bending moments along with the flange for the dead-load-2 (DL2) and live-load loading stages. This composite action significantly reduces the lateral bending stress experienced by this flange.

The total tip stress is crucial for the design of horizontally curved girders. It will tend to control the size of the flange for sharp radii (large lateral bending moments) or narrow flange choices (small horizontal section modulus).

Fatigue Stress Range

The fatigue stress range is computed for the fatigue point under consideration. The stress range is calculated at a designated point on the girder. For a curved girder design, the lateral fatigue stress range is added to the normal fatigue stress range to give the total fatigue stress range.

When shear connectors are provided in the negative-moment region of a continuous girder, it is the longitudinal reinforcing bars, not the concrete deck in tension, that act compositely. The fatigue stress range is computed in the reinforcing bars by extrapolating the straight-line stresses on the girder to the center of the top layer of reinforcement.

The fatigue stress range at the critical fatigue point is an important design consideration. For imposed loadings larger than HS20, the fatigue stress range can control the size of the flanges at almost every point of a horizontally curved girder with welded diaphragm connection plates.

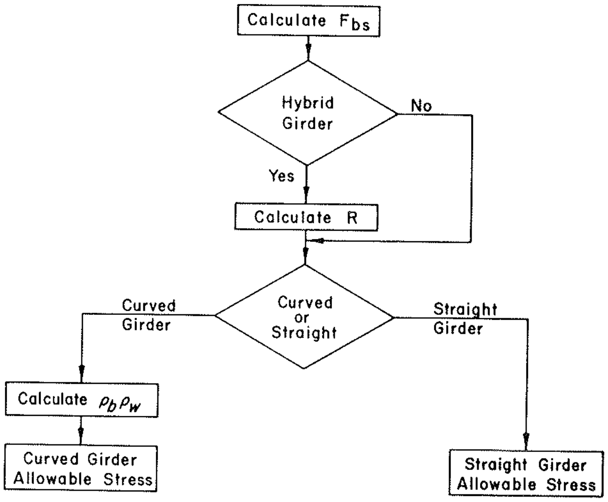
COMPUTATION OF ALLOWABLE STRESS

The allowable stresses needed to properly analyze a girder cross section include the following:

1. Compression-flange allowable normal stress at the DL1 loading stage,
2. Top-flange allowable normal stress at maximum load,
3. Top-flange allowable tip stress (for a curved girder),
4. Bottom-flange allowable normal stress at maximum load,
5. Bottom-flange allowable tip stress (for a curved girder), and
6. Allowable fatigue stress range.

The computation of the allowable stresses includes calculation of the basic allowable compression-flange normal stress, calculation of the hybrid reduction factor for a hybrid girder, and calculation of the curvature correction factors for a curved girder. These values in combination with the minimum yield point of the flange (F_y) equal the allowable stresses for a straight or curved girder. The allowable fatigue stress range is specified by the engineer.

The process of calculating the allowable stress is represented in terms of a flowchart in Figure 2.



Legend

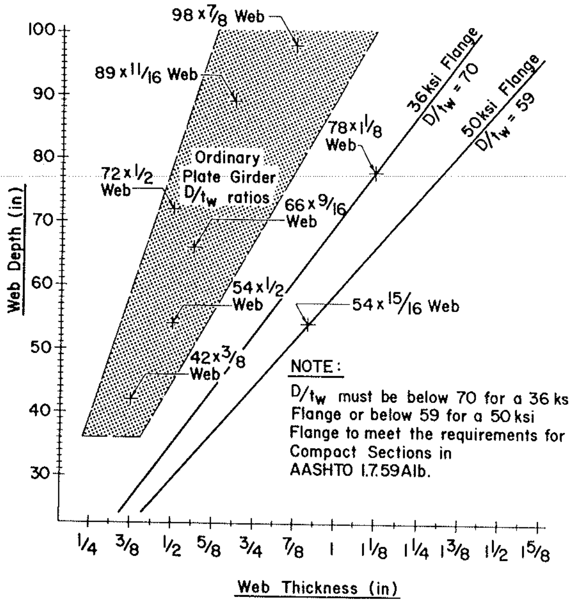
F_{bs} = Basic Allowable Compression Flange Normal Stress
R = Hybrid Reduction Factor
$\rho_b \rho_w$ = Curvature Correction Factors

FIGURE 2 Computation of allowable stress.

Basic Allowable Compression-Flange Normal Stress

The basic allowable compression-flange normal stress is computed for a straight girder from AASHTO 1.7.59B or AASHTO 1.7.59D (1,2) and for a curved girder from CURVED AASHTO 2.12B (3).

It is appropriate at this point to note that an ordinary plate-girder web will not conform to the severe D/t_w constraint specified in AASHTO 1.7.59A1b (Figure 3). Accordingly, the program will not analyze a straight girder as a compact section or a straight girder in transition as defined by AASHTO 1.7.59C.



NOTE: D/t_w must be below 70 for a 36 ksi Flange or below 59 for a 50 ksi Flange to meet the requirements for Compact Sections in AASHTO 1.7.59A1b.

FIGURE 3 Compact section web requirement for straight girder.

The formula for the maximum strength of an unbraced section for a straight girder is given in AASHTO 1.7.59D. It is the same formula proposed by Vincent (4). Note that it is similar to the allowable stress design formula given in AASHTO Table 1.7.1A in that both formulas have the constant F_y/E. However, the allowable stress design specification also lists simplified formulas for each value of F_y.

The computer program described here uses simplified formulas based on the equation given in AASHTO 1.7.59D. The simplified formulas are as follows:

1. If F_y = 36,000 psi and the section is symmetrical, $F_{bs} = 36{,}000 - 13.6(L/b)^2$.
2. If F_y = 36,000 psi and the section is unsymmetrical, $F_{bs} = 36{,}000 - 16.8(L/b)^2$.
3. If F_y = 50,000 psi and the section is symmetrical, $F_{bs} = 50{,}000 - 26.2(L/b)^2$.
4. If F_y = 50,000 psi and the section is unsymmetrical, $F_{bs} = 50{,}000 - 32.3(L/b)^2$.

Note that all these formulas are similar to the practical and familiar formula, $F_b = 20{,}000 - 7.5(L/b)^2$, listed in AASHTO Table 1.7.1A. Also note that the term F_{bs}, the basic allowable compression-flange normal stress, is expressed in pounds per square inch. The original formula (1.7.59D), defined as the maximum strength, is expressed as the resisting moment (M_u). In order to analyze a composite section, the magnitudes of the stresses experienced at each loading stage by the extreme fibers of each flange are required. The stresses, not the individual resisting moments, are of practical use. This concept is similar to the curved-girder specification, CURVED AASHTO 2.12B, which describes the resistance in terms of maximum flexural stress. The simplified formulas are different for a symmetrical and a nonsymmetrical girder; the nonsymmetrical girder conforms to AASHTO 1.7.60A, in which the term b is replaced by 0.9b.

The derivation of the original formula (AASHTO 1.7.59D) assumed a prismatic compression flange within its unbraced length. In continuous bridges the compression flange at an interior support is often nonprismatic within the unbraced length. A U.S. Steel technical report (7) addresses this condition and proposes a design procedure that involves calculating F_{bs} by using the flange width at the low-moment side (assumes narrower flange) of the unbraced length. The program can analyze a nonprismatic compression flange. The design flange width at the low-moment side is input and F_{bs} is calculated, substituting the design flange width for the flange width at the section under consideration.

For a straight girder design, AASHTO 1.7.59D allows an increase of 20 percent of the resistance at any point along the length of the girder when the ratio of stresses at the two ends of the unbraced length is less than 0.7. Because the possibility of a nonprismatic compression flange exists, the 20 percent increase should be based on the ratio of forces (not stresses) in the flange at each end of the unbraced length. The decision to increase the allowable stress by 20 percent is made by the engineer. It is believed that the 20 percent increase would be best applied within the unbraced length at the interior supports only.

If the unbraced length of the compression flange is less than that specified in AASHTO 1.7.59B1c, the section is braced.

A flowchart of the calculation of the basic allowable compression-flange normal stress is shown in Figure 4.

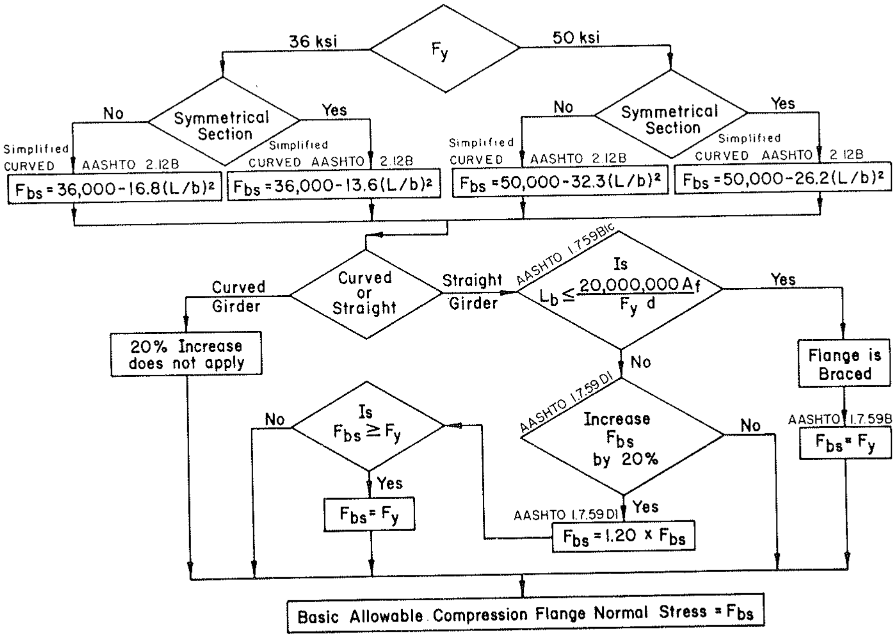


FIGURE 4 Computation of basic allowable compression-flange normal stress (F_{bs}).

Hybrid Reduction Factor

The hybrid reduction factor is calculated for a girder that uses a 36-ksi web and one or two 50-ksi flanges. For a homogeneous girder, the reduction factor is set equal to 1.000.

At this point in the program, the local buckling requirement for the unbraced compression flange is checked. If the requirement is not met, the flange is defined as illegal and the reduction factor is set equal to zero. For a composite section the top flange is checked at the DL1 loading stage only. For a curved girder the local buckling requirements in CURVED AASHTO 2.12B1 and 2.12B2 are compared with the compression-flange ratio of width to thickness.

The hybrid reduction factor is calculated for a straight girder by using the formulas in AASHTO 1.7.67B. These are the original formulas proposed by the ASCE subcommittee on hybrid beams and girders (9).

For the case of the nonsymmetrical girder, the formula is similar to AASHTO 1.7.50 (service load design method), in which the ratio of web yield strength to tension flange yield strength is designated "alpha." The same variable is designated "rho" in AASHTO 1.7.67B. In the figures for this paper "alpha" is used for this variable to make the specification terminology consistent.

The hybrid reduction factor is calculated for a curved girder by using the formulas in CURVED AASHTO 2.19Aa and 2.19Ab. The formulas, derived by Culver (6), are for a compact section and a noncompact section as defined by the compression-flange ratio of width to thickness.

A flowchart of the calculation of the hybrid reduction factor is shown in Figure 5.

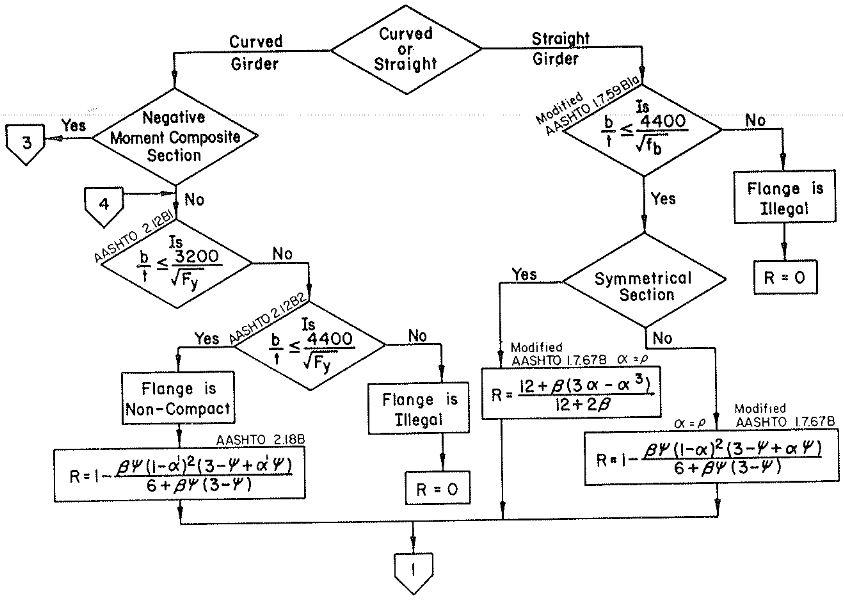


FIGURE 5 Computation of hybrid reduction factor (R).

After the reduction factor has been calculated, the program determines whether the reduction factor applies to each flange. If the stress in the extreme tension fiber of the web is larger than the minimum yield point of the web, the hybrid reduction factor applies to the tension flange. A similar check is made at the extreme compression fiber of the web to determine whether the hybrid reduction factor applies to the compression flange. It is believed that the reduction factor can properly be applied to the tension flange and not to the compression flange. This condition often occurs at a composite section.

The derivation of the hybrid reduction formula for a composite section is based on the tension capacity of the girder (5). It does not consider whether the compression flange yields or not because of normal bending moments. Because the fullest use of the top flange of a composite section will result in a high magnitude of flange stress, the application of the reduction factor to the top flange of a composite section will affect the size of the flange.

For a curved girder design, the allowable normal stress is usually reduced because of curvature correction factors. If a 36-ksi flange is chosen for the top flange of a composite curved girder, for example, the application of the curvature correction factors could reduce the allowable normal stress to 30 ksi. It is now necessary to decide whether the top flange, which has the same F_y as the web and a stress at least 20 percent less than the F_y of the web, should have the hybrid reduction factor applied to it. It is believed that the hybrid reduction factor should not be applied for this case.

After determining whether to apply the reduction factor to each flange, the program then checks whether the compression-flange area is greater than or equal to the tension-flange area. If the compression-flange area is less than the tension-flange area, the reduction factor is set equal to zero.

A flowchart showing the process of deciding whether the reduction factor applies to each flange and checking of the compression-flange area is given in Figure 6.

For a curved girder, the hybrid reduction factor is adjusted, depending on the ratio of lateral bending stress to normal bending stress in the tension flange, in accordance with CURVED AASHTO 2.19A. If the ratio of lateral bending stress to normal bending stress is high, the flange is controlled by the tip stress. In this case the hybrid reduction factor does not apply, and the program sets the reduction factor equal to 1.000.

Finally, for a curved hybrid girder that uses the deck-slab reinforcement to achieve composite action in the negative-moment region, the ratio of lateral bending stress to normal bending stress is checked for the compression flange in accordance with CURVED AASHTO 2.19B. This check results in either an adjustment in α' or the recognition that the lateral bending stress controls the design of the flange (6), in which case the hybrid reduction factor does not apply and the program sets the reduction factor equal to 1.000.

A flowchart of the adjustment of the hybrid reduction factor is given in Figure 7.

Curvature Correction Factors

The curvature correction factors are computed for a curved girder from CURVED AASHTO 2.12B. The compression-flange ratio of width to thickness is compared with the specification limit in CURVED AASHTO 2.12B1. If the ratio does not exceed this limit, the flange is defined as compact and the curvature correction factors are calculated by using the formulas in CURVED AASHTO 2.12B1. Note that the severe web constraint (D/t_w) for compactness, as specified in AASHTO 1.7.59A1b (Figure 3), does not apply for a curved girder (6). However, it should be noted that a compact straight girder is not the same as a compact curved girder.

If the compression flange does not meet the compact-flange requirements, the program checks its ratio of width to thickness against the noncompact requirement as defined in CURVED AASHTO 2.12B2. If the flange does not meet this requirement, it is defined as illegal and both correction factors are set equal to zero. For a flange that is noncompact, the curvature correction factors are calculated by using the formulas in CURVED AASHTO 2.12B2.

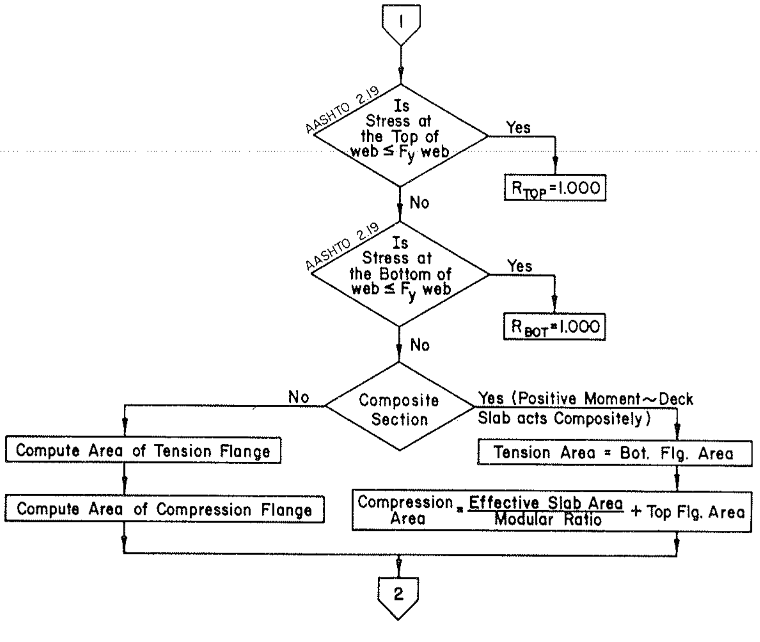


FIGURE 6 Application of hybrid reduction factor (R).

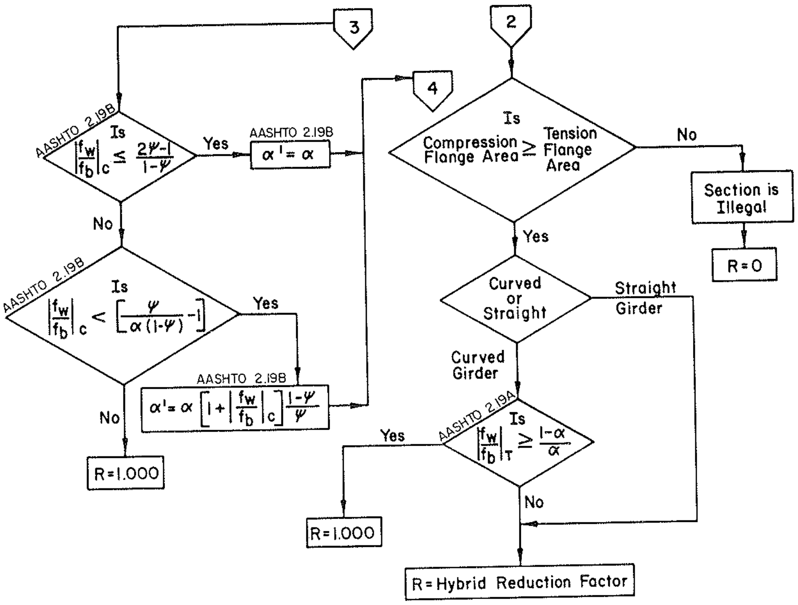


FIGURE 7 Adjustment of hybrid reduction factor (R).

The correction factors are based on the length of the unsupported compression flange between cross frames or diaphragms. They reduce the allowable compression-flange stress because the flange is unstable and will buckle laterally, torsionally, or locally under the influence of high stress. The top flange of a composite section is attached to the deck slab by shear connectors and partly encased in concrete. It is believed that the curvature correction factors should not be applied to the top flange of a composite section. However, CURVED AASHTO 2.16 uses the conservative approach and applies the curvature correction factors to the compression flange regardless of the presence of composite action. The program uses this conservative AASHTO approach.

The flowchart for calculation of the curvature correction factors is given in Figure 8.

Straight-Girder Allowable Stress

The compression-flange allowable stress at the DL1 loading stage and the compression-flange allowable stress at maximum load are calculated by using combinations of the basic allowable compression-flange normal stress and the hybrid reduction factor. The tension-flange allowable stress is calculated by using combinations of the minimum yield point of the flange material and the hybrid reduction factor.

The flowchart for calculation of the straight-girder allowable stress is given in Figure 9.

Curved-Girder Allowable Stress

The compression-flange allowable normal stress at the DL1 loading stage and the compression-flange

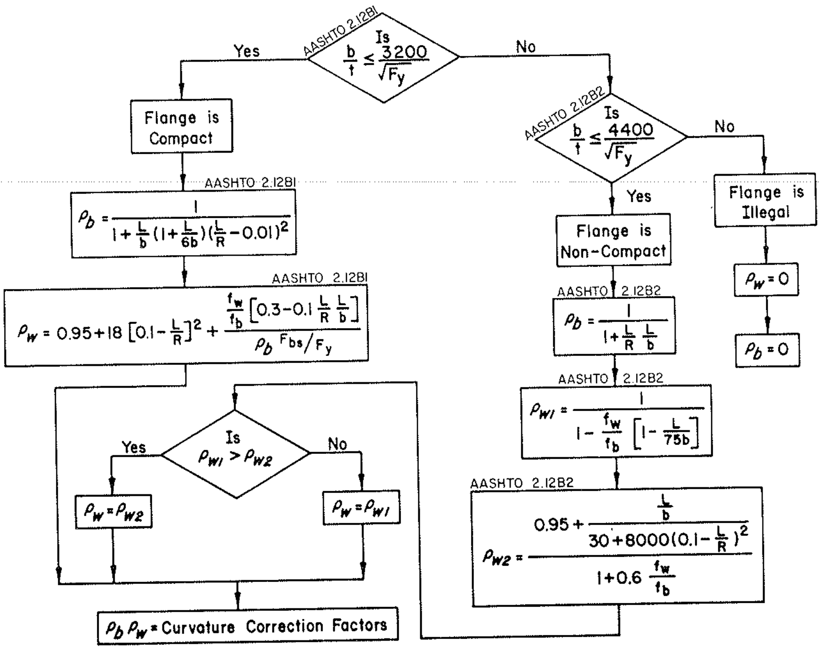


FIGURE 8 Computation of curvature correction factors (P_bP_w).

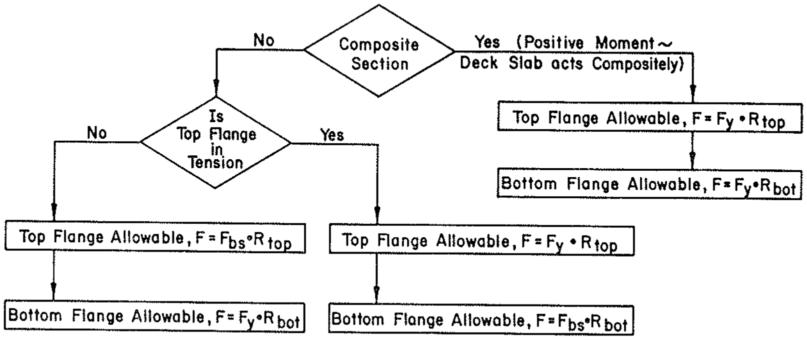


FIGURE 9 Computation of straight-girder allowable stresses.

allowable normal stress at maximum load are calculated by using combinations of the basic allowable compression-flange normal stress, the hybrid reduction factor, and the curvature correction factors. The tension-flange allowable normal stress is calculated by using combinations of the minimum yield point of the flange material and the hybrid reduction factor.

Curvature correction factors are not applied to the tension flange because it will not buckle. However, the AASHTO 2.12B specification is written in such a way that it could easily be interpreted that the rho factors are applied to the tension flange. This interpretation can be shown in three steps, as follows:

1. $F_{by} = F_{bs}P_bP_w$,
2. For the tension flange, $F_{bs} = F_y$, and
3. Therefore, for the tension flange, $F_{by} = F_yP_bP_w$.

It is believed that compact and noncompact curved-girder flanges should have the maximum flexural stress for the tension flange defined as follows:

1. $F_{by} = F_y$ (noncompact) and
2. $F_{bu} = F_y$ (compact).

This judgment is based on the service load design method, which defines the allowable tension-flange normal stress, in CURVED AASHTO 1.10B, in relation to F_y.

The flowchart for calculation of the curved-girder allowable stress is given in Figure 10.

The maximum allowable tip stress experienced by a tension or compression flange for a homogeneous or hybrid girder is equal to the minimum yield point of the flange material.

Allowable Range of Fatigue Stress

The allowable range of fatigue stress is given in AASHTO Table 1.7.2A1 for redundant load-path (multi-girder) structures and for nonredundant load-path (fracture-critical) structures. The location of the critical fatigue point and the allowable range of fatigue stress are input to the program. However, at any girder cross section with case 1 roadway load cycles, there are three choices of fatigue moments: lane fatigue moments, truck fatigue moments, and truck fatigue moments at a live-load distribution of S/7.0. At any girder cross section there may be numerous locations of fatigue design points.

For a girder with case 1 roadway load cycles, the determination of which live-load distribution to use for the truck fatigue moments is necessary. The stress range due to the distribution S/7.0 is 21.4

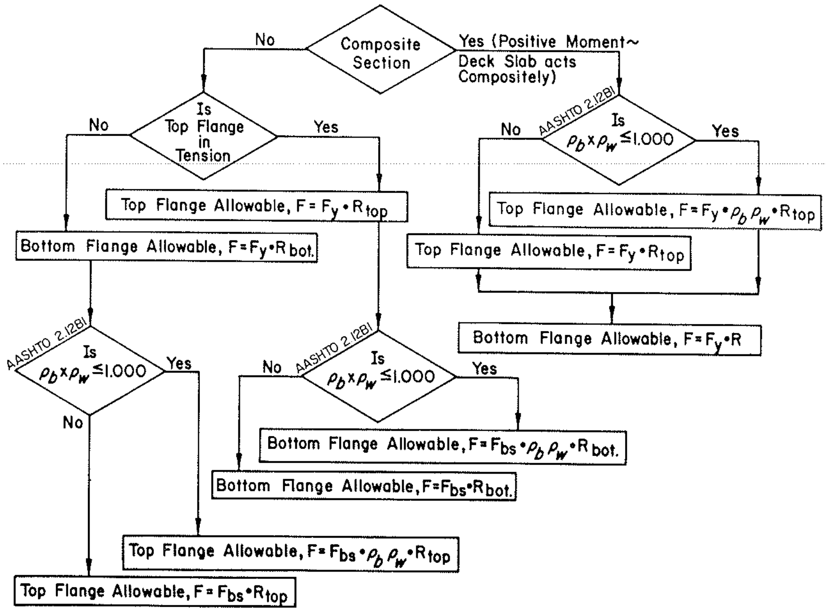


FIGURE 10 Computation of curved-girder allowable normal stresses.

percent less than the stress range due to the distribution S/5.5. The allowable range of fatigue stress for more than 2 million cycles is equal to or less than the allowable range for 2 million cycles for all fatigue categories, as shown in AASHTO Table 1.7.2A1. If the allowable range of fatigue stress for more than 2 million cycles is more than 21.4 percent of the allowable range for 2 million cycles, the truck fatigue moments at a distribution of S/7.0 will control the truck fatigue-stress range. This will occur for fatigue categories D and E** for a fracture-critical girder and for fatigue categories C, D, and E' for a multigirder system (Table 1).

TABLE 1 Case 1 Roadway: Allowable Fatigue Stresses

	Multi-Girder Structures			
Category	Lane Allowable (Ksi)	Ratio	Truck Allowable (Ksi)	Controlling Truck Distribution
A	36	1.50	24	S/5.5
B	27.5	1.72	16	S/5.5
C	19	1.90	10	S/7.0 ←
C*	19	1.58	12	S/5.5
D	16	2.29	7	S/7.0 ←
E	12.5	2.50	5	S/7.0 ←
E'	9.4	3.62	2.6	S/7.0 ←
F	12	1.50	8	S/5.5

	Fracture Critical Structures			
Category	Lane Allowable (Ksi)	Ratio	Truck Allowable (Ksi)	Controlling Truck Distribution
A	24	1.00	24	S/5.5
B	18	1.13	16	S/5.5
C	13	1.44	9	S/5.5
C*	13	1.18	11	S/5.5
D	10	2.00	5	S/7.0 ←
E**	8	3.20	2.5	S/7.0 ←
F	9	1.29	7	S/5.5

Legend

← Indicates that one truck at a distribution of S/7.0 for "over 2 million" stress cycles controls over a truck distribution of S/5.5 at "2 million" stress cycles.

Next the determination of whether the truck fatigue moments or the lane fatigue moments will control is necessary. This comparison is made in the same manner as the determination of the controlling truck fatigue moments. The ratio of lane fatigue moments to truck fatigue moments must be larger than the ratio of allowable stresses for the lane fatigue moments to control. The ratio of allowable stresses and appropriate controlling truck distribution are given in Table 1.

In addition to the considerations cited, the location of the fatigue point is also important. The fatigue-point location is input to the program.

Common controlling fatigue design points for a fracture-critical or multigirder system are shown in Figure 11.

SUMMARY AND COMMENTS

During the development of the computer program, several interpretations of the AASHTO specifications had to be made. The key ideas in this paper may be summarized as follows:

1. Ordinary plate-girder webs will not conform to the severe D/t_w constraint for a compact straight girder. Therefore, ordinary straight plate girders will not be compact.
2. Because ordinary straight plate girders will not be compact, the stress and allowable stress are of practical use but not the moment and resisting moment.
3. The design of nonprismatic compression flanges can be based on the narrower flange width at the low-moment side of the unbraced length.
4. The hybrid reduction factor is not applied to a flange until its adjacent extreme web fiber stress becomes larger than the web F_y.
5. It is believed that the curvature correction factors should not be applied to the top flange of a composite section. However, unless AASHTO clearly adopts this concept into the specification, the factors will be applied to the flange.
6. It is believed that the tension flange should not have the curvature correction factors applied to it, because it will not buckle.

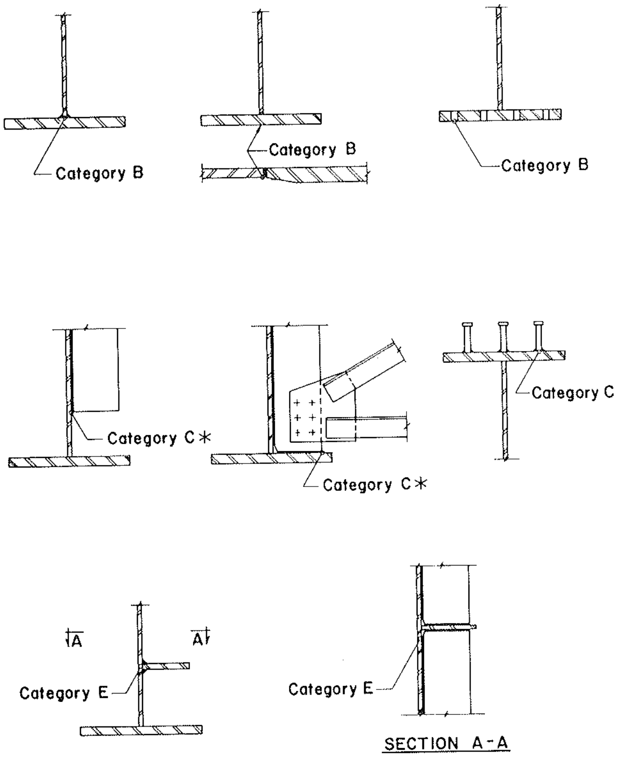


FIGURE 11 Common fatigue design points.

This program enables the engineer to evaluate AASHTO plate-girder cross sections quickly, which in turn minimizes the actual design time. However, it is only the first step of a much larger program that would make the decisions that the engineer must now make.

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Finite-Element Program for Analysis of Folded-Plate Bridge Superstructures

FAHIM A. BATLA, PATRICK R. REISNOUR, and DIVAKAR V. PATHAK

ABSTRACT

The behavior of bridge superstructures such as box girders and T-beams is similar to that of a folded-plate structure. A simplified finite-element program, FAP, specifically developed for the elastic analysis of constant depth and straight folded-plate type structures is presented. Being a specific-purpose program, it can be used by a bridge engineer without the extensive training, knowledge, and effort that may be required for finite-element programs developed for the analysis of a wide range of structural types. Most of the data for FAP analysis is generated by the program from minimal and straightforward input information. FAP has been developed with particular emphasis on practical design and construction considerations. It has already been used for the design of several bridge superstructures. FAP can facilitate the design of many folded-plate type bridge superstructures, especially in those cases in which the design may otherwise be a difficult and time-consuming effort because of the complex geometrical, loading, support, or construction conditions. The illustrative examples presented indicate that the results of analyses using FAP are in good agreement with those based on more exact theories and experimental data.

The finite-element method of structural analysis has become progressively more practical and economical as the availability and use of digital computers have increased. In the finite-element method complicated geometric forms, arbitrary loading and support conditions, and other structural parameters can be accurately and readily represented without extensive use of simplifying assumptions. This method, therefore, offers several advantages over conventional methods of structural analysis.

In recent years several computer programs based on the finite-element method have been developed for structural analysis and design. These programs have been developed for the analysis of a wide range of structural types and usually involve a large number of variables and complicated and extensive input data. This in turn requires a substantial amount of user effort, computational time, and computer capacity, which may not be necessary for the analysis of certain types of structures for which the modeling of the structural behavior can be simplified without affecting the acceptability of results for the purpose of design.

A large number of structures can be categorized as folded plates because of their behavior under loads and their cross-sectional shapes. The spatial rigidity of a folded-plate structure is provided by the out-of-plane (plate bending) and in-plane (membrane) behavior of its component plates, which join at folds (1-6). The width of these plates between folds in the transverse direction of the structure is small in comparison with their respective lengths between supports of the structure. As a result, the bending of these plates is predominantly a one-way behavior in the transverse direction of the structure.

The behavior of box girders, T-beams, and similar types of bridge superstructures is similar to that of a folded-plate structure. A finite-element computer program, FAP, for the elastic analysis of