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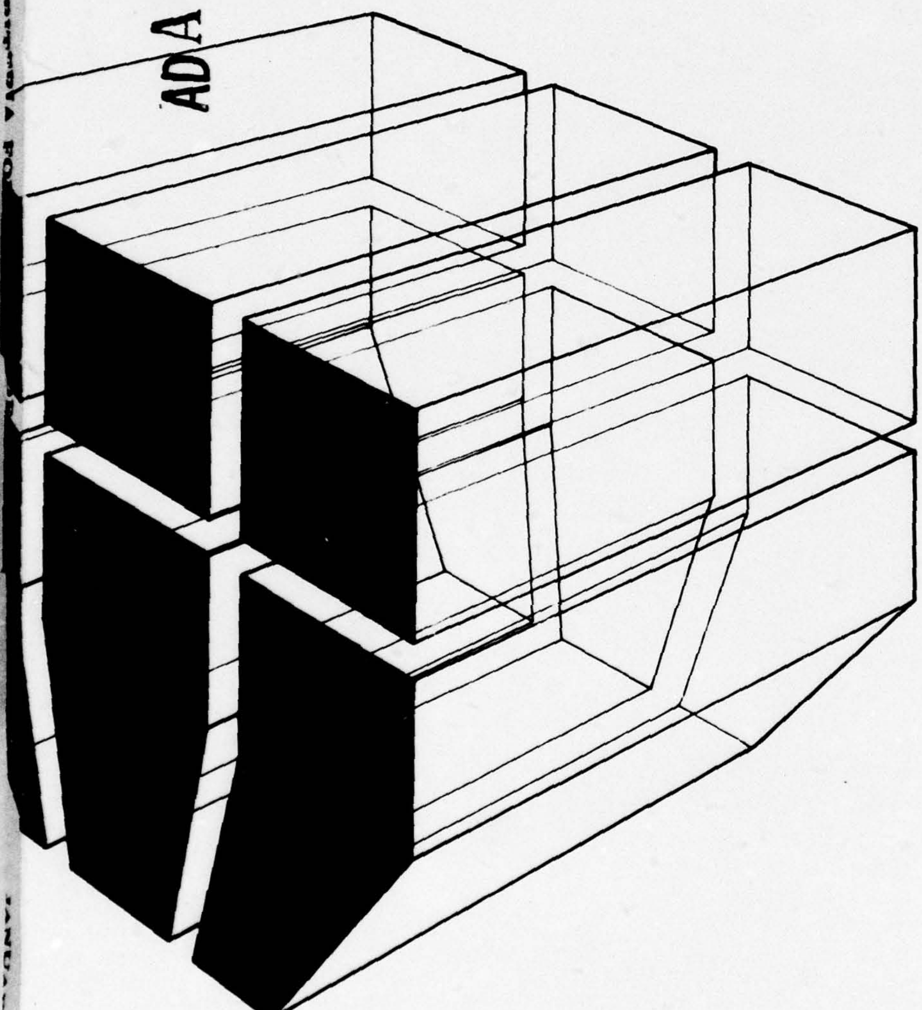
VOLUME II: APPENDICES A-I



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This volume details the development and justification of the design criteria, procedures, and material specifications for Theater of Operations (T/O) temporary steel highway bridges recommended in Volume I. Appendices A through D describe the development of the static load case criteria. The brittle fracture criteria are treated in Appendix E and the fatigue criteria are developed in Appendix F. Appendix G provides the background for the fastener and connection resistance for the static load case. Appendices H and I contain the development of the lateral load distribution and shear formulas.

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Block 20 continued.

It should be emphasized that this volume does not provide recommendations for design criteria, procedures, and material specifications; it provides only their development and justification. Volume I provides the recommendations.

FOREWORD

This study was conducted for the Directorate of Facilities Engineering, Office of the Chief of Engineers (OCE) under Project 4A763734D734 "Development of Engineer Support to the Field Army"; Task 04, "Base Development"; Work Unit 002, "AFCS Design Parameters for T/O Material Applications." The OCE Technical Monitor is Mr. R. H. Barnard.

The work was performed by the Construction Materials Branch (MSC) of the Materials and Science Division (MS), U. S. Army Construction Engineering Research Laboratory (CERL). Dr. L. I. Knab was the CERL Principal Investigator for the project. Mr. P. A. Howdyshell is Chief of MSC and Dr. G. R. Williamson is Chief of MS.

The fracture control plan for brittle fracture (Appendix E) was developed by S. T. Rolfe of the University of Kansas, Lawrence, KA. The fatigue criteria (Appendix F) and the background for fastener and connection resistance for the static load case (Appendix G) were developed by W. H. Munse and A. H-S. Ang of the University of Illinois at Urbana-Champaign, IL; W. H. Munse also reviewed Volume II in its entirety. W. W. Sanders, Jr. and H. A. Elleby of Iowa State University, Ames, IA, developed the lateral load distribution and shear formulas (Appendices H and I). T. V. Galambos of Washington University, St. Louis, MO, served as a consultant on the static load case criteria development.

COL J. E. Hays is Commander and Director of CERL and Dr. L. R. Shaffer is Technical Director.

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APPENDIX A:

DESCRIPTION OF RELIABILITY METHOD
USED TO DEVELOP CRITERIA FOR
STATIC LOAD CASE

Theoretical Framework

Studies of existing traditional structural codes have shown that the conventional safety factor approach and corresponding design criteria are not entirely rational and can lead to inconsistencies in structural reliability. Reliability-based design principles offer a more rational approach to design criteria development. Rational criteria development should explicitly account for the many underlying uncertainties present in the structural resistance and load functions. The framework adopted for use in this study was the second moment reliability analysis, which considers both the mean and variability of the resistance and loading random variables.

The resistance R and load effect Q (a load effect is a force such as a moment or shear) are random variables assumed to be statistically independent and to follow a log-normal distribution.* The limit state occurs when $R \le Q$, or, equivalently when $\ell n R \le \ell n Q$. The corresponding approximate limit state probability p_f is[1]

$$p_f = 1 - \Phi\left[\frac{\ell n(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}}\right] \qquad \text{[Eq A1]}$$

where $\Phi(X)$ = cumulative probability distribution of the standard normal distribution evaluated at X

R_m = mean resistance

* Limit state probabilities which are relatively large ($\ge$ 0.001) are not sensitive to the assumed distribution type. See A. H-S. Ang, "Structural Risk Analysis and Reliability-Based Design," *Journal of the Structural Division*, American Society of Civil Engineers (ASCE), Vol 99, No. ST9 (September 1973).

[1] A. H-S. Ang.

Q_M = mean load effect

V_R = coefficient of variation (COV) of resistance

V_Q = COV of the load effect.

The safety index β (beta) defined by:

$$\beta = \Phi^{-1}(1 - p_f) = \left[\frac{\ell n(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}}\right] \qquad \text{[Eq A2]}$$

is the value of the standard normal variate at a cumulative probability of $(1 - p_f)$. As shown in Figure A1, beta can be interpreted as the number of standard deviations between the mean of $\ell n(R/Q)$ and the point at which the limit state is reached.

To estimate the total variability, the member resistance R can be expressed as:

$$R = R_n MFP \qquad \text{[Eq A3]}$$

where M, F, and P = factors accounting for the uncertainties in material strength, fabrication (tolerances, geometry, etc.), and strength prediction (variation in predicted versus actual strength results caused by using approximate instead of exact formulas, etc.), respectively. R_n = the nominal resistance. The COV of R is

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \qquad \text{[Eq A4]}$$

where V_M, V_F, and V_P = COVs of M, F, and P, respectively.

The load effect Q for the dead plus live (D + L) load case is assumed to be of the form

$$Q = E(D + L) \qquad \text{[Eq A5]}$$

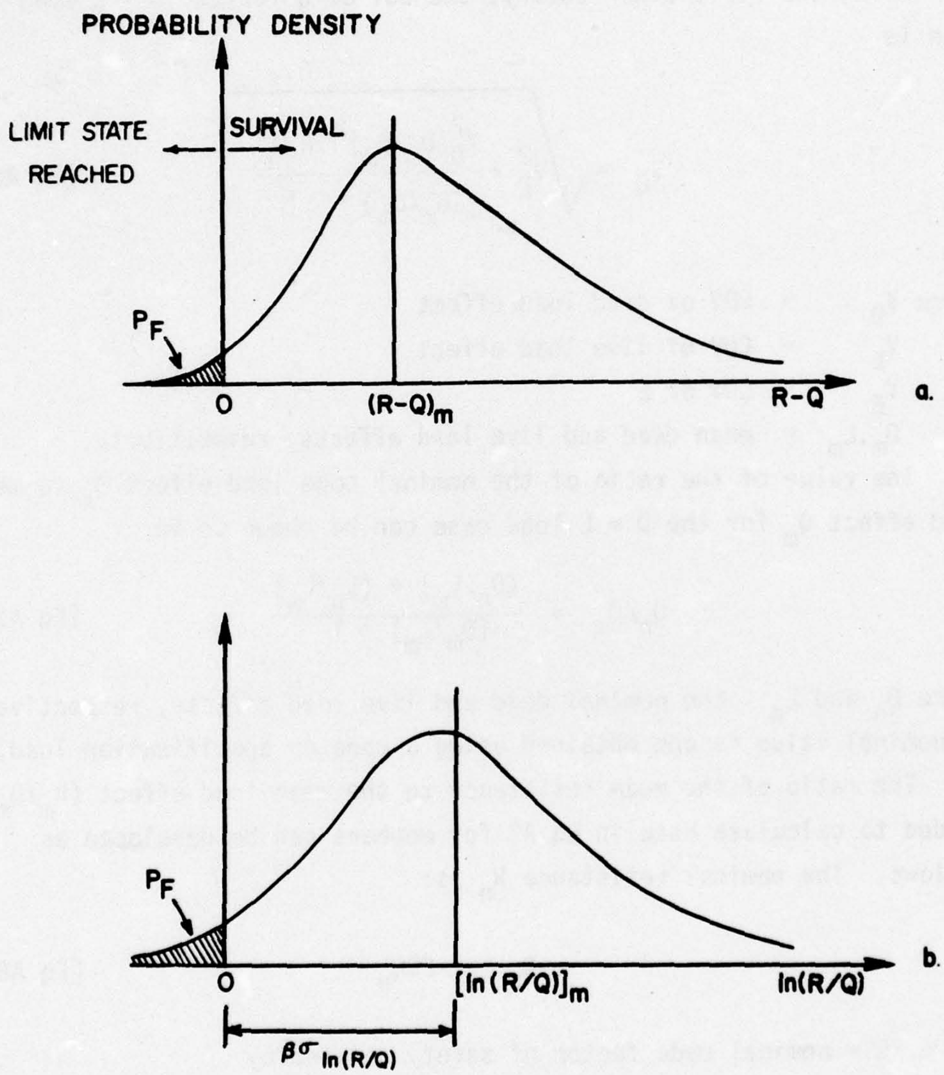

Figure A1. Definition of safety index β.

where E^* = a factor accounting for the uncertainties in structural analysis (including assumptions made)

D = the dead load effect

L = the live load effect caused by vehicles and their payloads.

Using the first order theory, the COV of Q for the D + L load case is

$$V_Q = \sqrt{V_E^2 + \frac{V_D^2(D_m/L_m)^2 + V_L^2}{[(D_m/L_m) + 1]^2}} \qquad \text{[Eq A6]}$$

where V_D = COV of dead load effect

V_L = COV of live load effect

V_E = COV of E

D_m, L_m = mean dead and live load effects, respectively.

The value of the ratio of the nominal code load effect Q_n to mean load effect Q_m for the D + L load case can be shown to be

$$Q_n/Q_m = \frac{(D_n/L_m) + (L_n/L_m)}{(D_m/L_m) + 1} \qquad \text{[Eq A7]}$$

where D_n and L_n = the nominal dead and live load effects, respectively. (A nominal value is one obtained using a code or specification load.)

The ratio of the mean resistance to the mean load effect (R_m/Q_m) needed to calculate beta in Eq A2 for members can be developed as follows. The nominal resistance R_n is:

$$R_n \geq FSQ_n \qquad \text{[Eq A8]}$$

where FS = nominal code factor of safety defined by

* The mean of E is assumed to be one.

$$FS = R_n/Q_n = f_{nu}/F_a \qquad \text{[Eq A9]}$$

f_{nu} = nominal ultimate stress

F_a = base allowable stress for permanent bridges

Q_n = nominal code load effect.

The mean resistance R_m can be written as

$$R_m = (R_m/R_n)\ R_n \qquad \text{[Eq A10]}$$

Using Eq A8 in A10 and dividing the result by Q_m results in the expression for R_m/Q_m for members:

$$R_m/Q_m = (R_m/R_n)FS(Q_n/Q_m) \qquad \text{[Eq A11]}$$

The ratio R_m/Q_m can be expressed in terms of a base allowable stress for permanent bridges F_a and the allowable stress factor Y, which is used as a multiplier to increase F_a. The result using Eq A9 in A11 and applying Y is:

$$R_m/Q_m = (R_m/R_n)(f_{nu}/[YF_a])(Q_n/Q_m) \qquad \text{[Eq A12]}$$

where Y = allowable stress factor defined as the ratio of the allowable stress used to the base allowable stress for permanent bridges F_a. (Note that YF_a is a modified allowable stress.)

An alternate form of R_m/Q_m used for fasteners and connections can be found in terms of the ratio of the mean ultimate stress f_m to the base allowable stress F_a as follows. The mean resistance R_m is:

$$R_m = Gf_m \qquad \text{[Eq A13]}$$

where f_m = mean ultimate stress

G = geometrical shape property, such as section modulus or cross-sectional area; for conventional permanent design, it is defined as

$$G \geq Q_n/F_a \qquad \text{[Eq A14]}$$

Substituting Eq A14 into A13, dividing by the mean load effect Q_m and using Y results in

$$R_m/Q_m = (f_m/[YF_a])(Q_n/Q_m) \qquad \text{[Eq A15]}$$

Method for Developing Allowable Stresses for Temporary Bridges

Using the procedure described above and the information on resistance and load assumptions in Appendices B and C, beta values (called "permanent" beta) were computed for permanent bridge allowable stresses (referred to as "permanent" stresses). Beta values (called "temporary" beta) were then computed for temporary bridges, based on an increase in the permanent allowable stress. This increase in permanent stress is represented by the allowable stress factor Y (Eq A12 and A15), which is defined as the ratio of the increased (modified) allowable stress to the base permanent allowable stress F_a. For a given load case and resistance, the permanent allowable stresses were increased (Y-increased) until the permanent and temporary beta values agreed within acceptable limits, while a minimum temporary beta level was maintained. The permanent and temporary beta values were tabulated and graphed to provide easier interpretation. To expedite the graphing, a digital computer-plotter combination was used.

Care was taken to insure that adequate minimum beta levels were maintained and that these levels were relatively close to the permanent criteria beta levels. Thus both the *magnitude* of beta and the *difference* between the permanent and temporary beta values were of key importance in establishing meaningful reliability levels and corresponding design criteria.

The question of the sensitivity of the design criteria recommendations to the probabilistic assumptions used to calculate beta is significant and deserves discussion.

The beta values are sensitive to the assumptions of the nominal to mean load effect ratios Q_n/Q_m, D_n/D_m,* and L_n/L_m as well as the load effect COVs V_Q, V_D, and V_L. Beta increases as the nominal to mean load effect ratio increases and decreases as the COV of the load effect increases. The assumptions concerning a given load effect become particularly significant when that load effect is predominant compared to the other types of load effects in a given load case. To illustrate, the beta values for the D + L load case become particularly sensitive to live load assumptions when the D_m/L_m is very small. That is, the live load effect is predominant over the dead load effect.

Beta values are also sensitive to the resistance assumptions for the mean R_m and COV V_R. Beta values increase when larger values of R_m and lower values of V_R are used.

The sensitivity of beta to the probabilistic assumptions can be reduced in several ways. "Base" betas for key or significant structural elements for permanent criteria can be established through a process called calibration. These "base" betas reflect the reliability levels of permanent structural elements which have been in use and have been structurally adequate for a long period of time. The "base" betas can be used as a basis for comparing beta values for temporary criteria. Hence, the comparison is made on a *relative* basis, resulting in a partial reduction in the sensitivity of beta to the probabilistic assumptions. In addition to using temporary beta values, the difference between permanent and temporary beta is used. Differences in beta will not be as sensitive to the assumptions, since a given

---

* In this study, D_n was assumed to be equal to D_m (see Appendix C).

change in assumptions affects permanent and temporary beta values similarly.

Several rough checks on the validity of the assumptions can be performed. One check is to determine whether the assumptions result in permanent beta values which appear to be reasonable in terms of their associated reliability level or limit state probabilities (Table A1).

Recent work in steel and concrete has resulted in guidelines for ranges of beta. For example, if for a given load case a computed beta value is significantly lower for one element than other similar structural elements, the assumptions should be questioned.

When using the beta analysis, trends and situations which result in low or unusual reliability levels can be identified. The recommended design criteria can then be chosen to account for these trends.

The method is realistic because it considers load and resistance together. Nominal loads and allowable stresses for the design of temporary bridges can be chosen which account for their temporary nature and yet result in reliability levels reasonably close to levels in temporary and permanent bridges which have performed adequately in the past.

Thus, the reliability method presented provides a quantitative tool for identifying permanent and temporary criteria which result in unusually low or high reliability levels. The method, when combined with traditional engineering judgment and experience, can be used to systematically develop consistent design criteria for temporary bridges. Use of the criteria should result in safe and satisfactory performance of temporary bridges, provided that adequate fabrication, erection, and construction practices are used.

Format

Both load and resistance factor design (LRFD) and working stress design (WSD) criteria are possible. The WSD format was chosen because

Table A1

Limit State Probabilities for Beta* Values from 0.5 to 3.0

β	Limit State Probability	Reliability
0.00	0.5000	0.5000
0.50	0.3085	0.6915
0.60	0.2743	0.7257
0.70	0.2420	0.7580
0.80	0.2119	0.7881
0.90	0.1841	0.8159
1.00	0.1587	0.8413
1.25	0.1056	0.8944
1.50	0.0668	0.9332
1.75	0.0401	0.9599
2.00	0.0228	0.9772
2.25	0.0122	0.9878
2.50	0.0062	0.9938
2.75	0.0030	0.9970
3.00	0.0013	0.9987

* Fcr the development of beta see C. A. Cornell, "A Probability-Based Structural Code," *Journal of the American Concrete Institute,* Vol 66, No. 12 (1969), pp 974-985.

the existing permanent (AASHTO and AISC) codes could be used by modifying them in those areas which would result in increased uniformity, decreased cost, and/or improved performance. The remaining criteria can remain unchanged. The WSD format is currently used in military bridge criteria and hence requires a minimum of change in current procedures. *In contrast, an LRFD format would require assignment of* resistance factors to all resistances and load factors to all load types.

APPENDIX B:

RESISTANCE ASSUMPTIONS USED IN RELIABILITY METHOD FOR STATIC LOAD CASE

This appendix describes the development of the resistance assumptions needed for the reliability analyses described in Appendix A. Table B1 shows the ratio of mean to nominal resistance R_m/R_n, the COV of resistance V_R, and the nominal factor of safety FS used in the equations in Appendix A for members. The ratios of the mean ultimate stress to the allowable stress f_m/F_a (Eq A15) and V_R are given in Table B2 for fasteners and connections. The information in Table B2 is based on Appendix G.

To aid in the analysis of the fasteners and connections, the f_m/F_a values were organized into four groups based on their V_R values. Instead of treating every resistance in Table B2, representative resistances were analyzed for each group; these resistances are designated by daggers in Table B2. The relationships for the mean strength of fasteners in combined tension and shear and corresponding V_R values are given in Table G14 of Appendix G.

Table B1

Resistance Assumptions for Members

Resistance Type	Mean/Nominal Resistance R_m/R_n*	COV Resistance V_R*	Factor of Safety $FS = f_{nu}/F_a$		Nominal Ultimate Stress f_{nu}	Base Allowable Stress (Based on AASHTO, except as noted otherwise) F_a	
Flexure							
Fully braced beam (plastic moment)	1.07	0.13	2.04		$1.12\ F_y$	$0.55\ F_y$	
Lateral torsional buckling	1.06	0.15	1.82**		**	**	
Shear	1.10	0.15	1.75		$F_y/\sqrt{3}$	$0.33\ F_y$	
Compression members							
Slenderness Parameter††			(AASHTO)	(TM 5-312+)		(AASHTO pinned ends)	(TM 5-312+)
$\lambda = 0.2$	1.05	0.12	2.27	1.68†	$(1 - 0.25\lambda^2)F_y$+	$(0.55/1.25)(1 - 0.25\lambda^2)F_y$	$0.592 - 0.0828\lambda^2$
0.4	1.03	0.13	2.27	1.66†	"	"	"
0.6	0.97	0.15	2.27	1.62†	"	"	"
0.8	0.94	0.17	2.27	1.56†	"	"	"
1.0	0.89	0.18	2.27	1.47†	"	"	"
1.2	0.86	0.17	2.27	1.35†	"	"	"
Tension Members	Covered under Connected Material--Tension Members in Fasteners and Connections.						

* Taken from T. V. Galambos and M. K. Ravindra, *Tentative Load and Resistance Factor Design Criteria for Steel Buildings*, Research Report No. 18 (Structural Division, Washington University, 1973); and T. V. Galambos and M. K. Ravindra, *Load and Resistance Factor Criteria for Steel Beams*, Research Report No. 27 (Structural Division, Washington University, 1974).

** See E. H. Gaylord and C. N. Gaylord, *Design of Steel Structures*, 2nd Ed. (McGraw Hill Book Co., 1972). Determination of FS = 1.82 is discussed on p 285 of this reference.

† Exceptions to the definition of F_a, since F_a is based on TM 5-312 rather than AASHTO; F_a is adjusted for F_y = 36 ksi.

†† $\lambda = (K\ell/r)(\sqrt{F_y/E})$

where K = effective length factor
ℓ = unsupported length
r = radius of gyration
F_y = nominal yield stress
E = elastic modulus

\+ Given by the Column Research Council's base column curve in Guide to Design Criteria for Metal Compression Members, 2nd Ed., B. G. Johnston, ed. (John Wiley and Sons, Inc., 1966).

Table B2

Fastener and Connection Information

Group	Identification Number (See Table G13)	Description*	Mean Ultimate Stress, f_m** (ksi except where noted)	Allowable Stress F_a** (ksi except where noted)	f_m/F_a
Group I, V_R = 0.12 bolts and rivets	1	A 490 tension†	122.0	54.0††	2.26
	2	A 490 S, S, TR	76.0	32.0††	2.38
	3	A 490 S, T, TR	58.1	22.5††	2.58
	4	A 307 S, T, TR	26.0	10.0	2.60
	5	A 502 Grade 2 S, TR	53.0	20.0	2.65
	6	A 325 Tension	100.3	36.0	2.79
	7	A 325 S, S, TR	56.2	20.0	2.81
	8	A 490 S, S, PL	90.0	32.0††	2.81
	9	A 325 S, T, TR	43.0	15.0††	2.87
	10	A 502 Grade 2 S, PL	58.0	20.0	2.90
	11	A 307 S, T, PL	29.1	10.0††	2.91
	12	A 502 Grade 1 S, TR	41.3	13.5	3.06
	13	A 490 S, T, PL	68.8	22.5††	3.06
	14	A 307 S, S, TR†	34.0	11.0	3.09
	15	A 307 Tension	50.0	15.3††	3.27
	16	A 502 Grade 2, tension	89.0	27.0††	3.30
	17	A 307 S, S, PL	38.0	11.0	3.45
	18	A 325 S, S, PL	69.5	20.0	3.47
	19	A 325 S, T, PL	53.2	15.0††	3.55
	20	A 502 Grade 1 tension	71.0	20.0††	3.55
	21	Bearing	4.35 F_y	1.22 F_y	3.57
	22	A 502 Grade 1, S, PL†	49.7	13.5	3.68
Group II, V_R = 0.14 Connected Material (tension members)	23	Double-plane truss type tension connection†	1.50 F_y	0.55 F_y	2.73
	24	Flat-plate type connection	1.62 F_y	0.55 F_y	2.95
Group III, V_R = 0.18 shop welds (shear)	25	E110†	86.6	33.0††	2.62
	26	E100	82.6	30.0††	2.75
	27	E90	77.6	27.0††	2.87
	28	E80	72.1	24.0††	3.00
	29	E70	65.6	21.0††	3.12
	30	E60†	58.7	18.0††	3.26
Group IV, V_R = 0.27 field welds (shear)	31	E110†	86.6	33.0††	2.62
	32	E100	82.6	30.0††	2.75
	33	E90	77.6	27.0††	2.87
	34	E80	72.1	24.0††	3.00
	35	E70	65.6	21.0††	3.12
	36	E60†	58.7	18.0††	3.26

* S = shear; S, T = shear, threads in shear plane; S, S, = shear, threads not in shear plane; PL = flat-plate type connection L < 50 in.; TR = double-plane truss type connection.
** f_m and F_a values for bolts and rivets based on nominal area.
† Representative resistance.
†† F_a based on AISC; values of F_a without †† based on AASHTO.

APPENDIX C:

DEVELOPMENT OF STATIC LOAD ASSUMPTIONS FOR RELIABILITY METHOD

This appendix describes the load assumptions needed for the reliability analysis described in Appendix A.

The value of the coefficient of variation (COV) accounting for uncertainties in structural analysis V_E was assumed to be 0.05.[2]

Dead Load (D)

It is reasonable to assume that the nominal dead load effect D_n based on the code specifications is approximately equal to the mean dead load effect D_m. Hence,

$$D_n = D_m \quad \text{[Eq C1]}$$

The COV of the dead load effect V_D can be treated as including the uncertainty of the dead load (e.g., load, psf) V_{D1}, and the uncertainty in the transformation of the dead load into a dead load effect (e.g., moment, axial force, etc.) V_{D2}. Galambos and Ravindra[3] estimate that V_{D1} and V_{D2} are equal to 0.04. Based on their values

$$V_D = \sqrt{V_{D1}^2 + V_{D2}^2} = 0.06 \quad \text{[Eq C2]}$$

Allen[4] estimates that V_D equals 0.07 for the Canadian code development. A value of 0.06 was chosen for V_D for this study.

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[2] T. V. Galambos and M. K. Ravindra, *Load and Resistance Factor Design Criteria for Steel Buildings*, Research Report No. 18 (Structural Division, Washington University, 1973).

[3] T. V. Galambos and M. K. Ravindra, *Load and Resistance Factor Design Criteria for Steel Buildings*.

[4] D. E. Allen, "Limit States Design--A Probabilistic Study," *Canadian Civil Engineering Journal* (March 1975).

Live Vehicle Load and Payload (L)

Permanent (AASHTO) Bridge Assumptions--
Loads, Allowable Stresses, and L_n/L_m
Values

Normal and overload crossings for permanent (American Association of State Highway and Transportation Officials [AASHTO]) bridges were analyzed and used to compute "base" betas to compare with the betas for temporary criteria. Vehicle loads for normal crossings are based on hypothetical loads given in Article 1.2.5 of AASHTO.[5] The allowable stresses for the normal crossings correspond to the current AASHTO allowable stresses for permanent bridges (Y = 1.0, where Y is the ratio of the allowable stress used to the base permanent bridge [AASHTO] allowable stress). Overload crossings covered under Article 1.2.4 of AASHTO require that any H or HS truck load (except H20 and HS20) shall be increased 100 percent and applied in any single lane without concurrent loading from other lanes; the combined dead, live, and impact stresses shall not be greater than 150 percent (Y = 1.50) of the allowable stress for normal crossings. Article 1.11.1 of AASHTO, which covers overload under permit, specifies that the allowable tensile stress shall not exceed 75 percent of the yield point of structural steel members and the compressive stresses shall be checked on a corresponding basis.

Illegal overloads which occur can, in certain situations, cause a more severe overload effect than permitted overloads. For example, the force in a compression or tension member with permitted overloads for single vehicles can be less than that produced by two vehicles side by side, one at the legal weight limit and the other exceeding the legal limit. A representative hypothetical illegal overload consisting of a vehicle 50 percent above the legal limit weight alongside a legal limit

[5] Unless otherwise specified, all references to AASHTO refer to *Standard Specifications for Highway Bridges*, 11th Ed. (American Association of State Highway and Transportation Officials, 1973).

weight vehicle on a two-lane bridge was chosen for analysis in this report.

In summary, for permanent (AASHTO) bridges, a value of 1.0 was used for Y for normal crossings. In the case of the overload design condition of Article 1.2.4 of AASHTO, a value of 1.50 was used for Y. For the AASHTO overload under permit provision of Article 1.11.1, Y equals 1.36 (= 0.75 F_y/0.55 F_y) was used for plastic moment M_p development in a fully braced stringer (0.75 F_y was assumed for compressive stress for M_p). A Y value of 1.0 was used for the chosen hypothetical illegal overload, which assumes the bridge was designed based on AASHTO stresses for normal loading and then illegally overloaded.

To determine the ratio of the nominal code live load effect to the mean maximum lifetime live load effect L_n/L_m, which is used in Eq A7 of Appendix A, the mean maximum lifetime live load LL_m(e.g., axle load in tons) must be estimated. In reality there is only one LL_m value for all the load cases. For comparative purposes, however, it is convenient to calculate beta values for each load case considered by assuming a reasonable LL_m value. As a result, the following assumptions for LL_m were made. For the overload crossing case, based on AASHTO Article 1.2.4, LL_m was assumed to be equal to the truck load corresponding to a normal crossing, increased by 100 percent. For the case of overload under permit (AASHTO Article 1.11.1), LL_m was assumed to be equal to the truck load causing the maximum allowable stress permitted (75 percent of the yield point in tension). For normal crossings, LL_m was assumed to be equal to the truck load corresponding to normal crossings. For the illegal overload case, LL_m was assumed equal to the bridge loaded with a vehicle 50 percent over the legal limit alongside of a legal weight vehicle.

For a typical two-lane, concrete deck, steel stringer AASHTO bridge, the AASHTO lateral load distribution formulas result in stringer moments which are, on the average, about 10 percent conservative.[6] Thus, for crossings covered under AASHTO provisions (normal and overload of Articles 1.2.4 and 1.11.1), an L_n/L_m value of 1.10 is reasonable for stringer moment. For the illegal overload case chosen, an L_n/L_m value of 0.73 (= 1.10/1.50) is reasonable for stringer moment. For shear, tension, and compression members, it is assumed that the AASHTO procedure for determining the member force is, on the average, the true member force. Thus, for crossings covered under AASHTO provisions, L_n/L_m was assumed to be 1.0. For the illegal overload case, L_n/L_m values of 0.67 (= 1.0/1.50) for shear and 0.80 (= 1.0/[(1.0 + 1.5)/2]) for tension and compression members were assumed.

Table C1 summarizes the resistance, crossing type, Y, and L_n/L_m assumptions for the AASHTO bridges.

*Temporary (Military) Bridge Assumptions--
Loads, Allowable Stresses, and* L_n/L_m
Values

For temporary bridges, the normal and caution (overload) crossings (described in TM 5-312[7]) were analyzed. The normal crossing consists of convoy(s) of vehicles not exceeding the posted bridge class. A caution crossing consists of a single line of vehicles, crossing a one- or two-lane bridge on the bridge centerline; the vehicles are spaced at 150 ft* or more and shall not exceed 1.25 times the normal posted class. The caution crossing is recommended as the largest overload to be permitted. All discussion, analyses, and criteria recommendations assume that the normal and caution crossings are the only two types of crossings permitted.

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[6] W. W. Sanders and H. A. Elleby, *Distribution of Wheel Loads on Highway Bridges,* Report No. 83 (National Cooperative Highway Research Program, 1970).

[7] *Military Fixed Bridges,* TM 5-312 (Department of the Army, December 1968).

* SI conversion factors for all units of measure used in this report are given at the end of the report.

Table C1

Summary of Load Assumptions Used

Load Case No.	Bridge Type*	Specification	Crossing Type (Loading Used)	Resistance (Failure Mode)**	L_n/L_m	γ
1	P	AASHTO	Overload (Article 1.2.4)	M_p, LTB, FLSPL	1.10†	1.50
2	P	AASHTO	Overload with Permit (Article 1.11.1)	M_p	1.10†	1.36††
3	P	AASHTO	Illegal Overload+	M_p,LTB	0.73† (1.1/1.5)	1.00
4	P	AASHTO	Normal (civilian) crossing	M_p,LTB,FLSPL	1.10†	1.00
5	T	Military TM 5-312	Normal and caution++ military crossings	M_p,LTB, FLSPL	1.35#	≥1.00
6	P	AASHTO	Overload (Article 1.2.4)	Shear, Tens. Comp.	1.00	1.50
7	P	AASHTO	Illegal overload+	Shear	0.67 (1.0/1.5)	1.00
8	P	AASHTO	Illegal overload+	Tens., Comp.	0.80 (1/([1.5 + 1.0]/2))	1.00
9	P	AASHTO	Normal (civilian) crossing	Shear, Tens. Comp.	1.00	1.00
10	T	Military##	Normal or caution military crossing used--choose whichever produces larger force	Shear, Tens. Comp.	1.00	≥1.00
11	T	Military##	Force due to normal military crossing used	Shear, Tens., Comp.	0.80	≥0.75

* P = permanent (AASHTO) bridge; T = temporary military bridge.

** M_p = plastic moment, fully braced beam; LTB = lateral torsional buckling; FLSPL = flexural splice; Tens. = tension members and associated fasteners and connections; Comp. = compression members and associated fasteners and connections.

† Based on typical two-lane concrete deck, AASHTO steel stringer bridge.

†† γ=1.36 actually applies only to tensile stress, but was used for tension and compression for M_p.

\+ Corresponds to a two-lane bridge loaded with a vehicle 50 percent over the legal limit alongside a legal-weight vehicle.

++ Both normal and caution crossings assumed to produce the same maximum moment; see Appendix H.

\# Based on solid deck (concrete or glued-laminated panel); see Appendix H.

\#\# Assumes use of recommended procedure to determine shear force given in Appendix I.

The allowable stress factor Y for the temporary bridge criteria was varied until reasonable agreement was attained between the beta values for permanent and temporary bridges (see Eq A2, A12, and A15 of Appendix A).

In the case of military temporary bridges, the mean maximum lifetime load (not load effect) was assumed to be that corresponding to the critical crossing load (not load effect). By critical is meant that crossing, either normal or caution, which results in the largest load effect.

For a stringer moment, Appendix H shows that L_n/L_m for a wide variety of temporary single-lane military bridges ranges between 1.10 and 1.61 with an average of 1.35; for double-lane bridges L_n/L_m ranges from 1.00 to 1.75 with an average of 1.38. The major difference between single- and double-lane bridges is that for single-lane bridges the caution overload crossing is the critical load case, whereas for double-lane bridges the normal crossing, with more than one vehicle in adjacent lanes on the bridge, is the most critical. To use a single allowable stress for both single- and double-lane bridges, an average value of $L_n/L_m = 1.35$ was chosen. It should be noted that this value of L_n/L_m was based on a "solid" deck assumption. Concrete or glued-laminated timber panel decks are considered solid; nailed-laminated timber, plank, or multiple-layered decks are not. Appropriate reductions in L_n/L_m for decks which are not solid are given in Appendix H. The L_n/L_m value of 1.35 was also based on a width of bridge floor to span length ratio (W/L) of less than one. Appropriate reductions in L_n/L_m when $W/L > 1.0$ are given in Appendix H.

The current military procedure for determining shear force (given in TM 5-312 and discussed in Appendix I) can significantly underestimate the mean shear load effect. To prevent this, a recommended procedure for determining shear force is given in Appendix I. Use of the recommended procedure was assumed in all of the analyses and discussions in this report. If the recommended shear procedure of Appendix I is used, L_n/L_m is assumed to be 1.0; this value corresponds to using the larger of the shear forces caused by the normal or caution crossings. For comparison, an L_n/L_m value of 0.80 was also used. The 0.80 value could occur if the "1.25" coefficient in Table I2 of the shear procedure in Appendix I were reduced to 1.00. Thus, the 0.80 value would result when the shear force for a normal crossing was used, but actually a caution crossing produced a shear force 25 percent greater than the normal crossing. Considerable reliability differences can result for L_n/L_m = 0.80 or 1.0.

Determination of the force in tension or compression members is primarily based on statics. Hence, depending on many factors including the number of lanes and their corresponding classes, either the caution or the normal load cases--whichever results in the largest tensile or compressive force in the member--can be critical. For example, in a single-lane bridge, the caution crossing is usually critical, whereas for a bridge having two lanes of equal class, the normal load of two vehicle convoys would be critical. An L_n/L_m value of 0.80 represents a tension or compression member design based on normal crossing vehicle forces; the member, however, is actually subjected to a caution loading which results in a tensile or compressive force in the member 25 percent higher than the normal crossing force. A tension or compression member design based on the larger force caused by either the caution or normal crossings, however, results in an L_n/L_m value of 1.0. If, as in the current design procedure of TM 5-312, the normal load case is used to proportion the compression members, then L_n/L_m can range between 0.80 and 1.0. Since the allowable stress is currently not reduced for an L_n/L_m value of 0.80, considerable realiability differences can result for L_n/L_m = 0.80 or 1.0. Hence, both L_n/L_m = 0.80 and 1.0 are analyzed.

Table C1 summarizes the resistance, crossing type, Y, and L_n/L_m assumptions for temporary bridges.

Coefficient of Variation Assumptions for Live Load Effect for Permanent and Temporary Bridges

The COV of the live load effect V_L can be treated as consisting of the uncertainty V_{L1} in the vehicle load (e.g., axle weight in tons) and the uncertainty V_{L2} in transforming the live load into a load effect (e.g., moment in ft-kips). That is:

$$V_L = \sqrt{V_{L1}^2 + V_{L2}^2} \qquad \text{[Eq C3]}$$

Table C2 gives values of V_{L1} for temporary bridges for various nominal classes of military vehicles as defined in TM 5-312. The calculations are based on a normal distribution for the maximum lifetime vehicle class* (assumed equal to 1.25 x nominal vehicle class*) and that 95 percent of the maximum class* vehicles fall within 1.25 x upper class** boundary and the lower class** boundary. The vehicle class** boundaries were chosen to include any variation in the classification* process, including combination of vehicle classes,* temporary classification,* overload (1.25 x normal class) determination, and discrepancies between the hypothetical and actual vehicle loads, including vehicle spacing (longitudinal) and vehicle dimensions. Except for very low vehicle classes, V_{L1} ranges from 0.10 to 0.12. A value of $V_{L1} = 0.11$ was assumed representative of the uncertainty in the temporary as well as permanent bridge vehicle loads.

The value of V_{L2}, which accounts for the transformation of the live load into a live load effect can be considered to be composed of (1) the actual observed (i.e., measured in the field) variation in the transformation of the vehicle load into a load effect, called V_{L2A}, which includes the effects of impact, vehicle position including passing patterns, or roadway, and (2) the error introduced in predicting

* Refers to vehicle class (load) as given in TM 5-312.

** Class is used in its statistical sense, rather than as a vehicle class of TM 5-312.

Table C2

Basis for Calculation of V_{L1}--Temporary Military Bridges

Nominal Class	Lower Class Boundary	Upper Class Boundary	Class Mark	1.25 x Nominal Class	Standard Deviation*	Coefficient of Variation** V_{L1}
4	0	5.9	2.95	3.69+	1.84	0.500+
8	6.0	9.9	7.95	10	1.59	0.159
12	10.0	13.9	11.95	15	1.84	0.123
16	14.0	17.9	15.95	20	2.09	0.105
20	18.0	21.9	19.95	25	2.34	0.094
24	22.0	26.9	24.45	30	2.91	0.097
30	27.0	34.9	30.95	37.5	4.16	0.111
40	35.0	44.9	39.95	50	5.28	0.106
50	45.0	54.9	49.95	62.5	5.91	0.095
60	55.0	64.9	59.95	75	6.53	0.087
70	65.0	74.9	69.95	87.5	7.16	0.082
80	75.0	84.9	79.95	100	7.78	0.078
90	85.0	94.9	89.95	112.5	8.41	0.075
100	95.0	109.9	102.45	125	10.59	0.085
120	110.0	124.9	117.45	150	11.53	0.077
150	125.0	164.9	144.95	187.5	20.28	0.108

* Standard Deviation $\simeq \frac{1.25 \times \text{Upper Class Boundary} - \text{Lower Class Boundary}}{4}$

** Coefficient of Variation ≃ Standard Deviation/(1.25 x Nominal Class)

\+ Based on use of class mark rather than nominal class.

the transformation of the vehicle load into a load effect, called V_{L2B}. Included in V_{L2B} is the error introduced in estimating the forces such as moment, shear, and axial load caused by the vehicle load. For example (see Appendix H for details) one of the main sources of uncertainty in flexure is the prediction of the lateral load distribution, which depends on many variables, including vehicle lateral spacing, number and spacing of stringers, thickness and type of deck, whether tracked or wheeled vehicles, and vehicle length and width. The value of V_{L2} can be expressed as

$$V_{L2} = \sqrt{V_{L2A}^2 + V_{L2B}^2} \qquad \text{[Eq C4]}$$

Rohl and Walker[8] found that V_{L2A} for flexure for permanent highway bridges ranges between 0.05 and 0.19, with an average of about 0.12. The value of 0.12 represents the COV of the percent of the moment carried by a beam. The percent of moment (M%) was found by:

$$M\% = \frac{\text{observed maximum stringer strain on bottom fiber}}{\text{observed mean total maximum stringer strains (sum of bottom strains for all stringers)}}$$

The V_{L2A} value of 0.12 represents a mixture of components and includes lateral distribution of load into stringers, vehicle speed and impact, vehicle position (laterally), and vehicle weight and size. The results are for normal heavy truck traffic and normal passing maneuvers for two-lane, concrete deck, steel stringer interstate bridges.

A part of V_{L2A} is the COV for impact stresses. Analyses of field data on the impact for railroad bridges for various railway bridge span lengths and train speeds[9] showed that the average variability of the impact coefficient is approximately 0.08.

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[8] J. A. Rohl and W. H. Walker, *Stress Histories for Highway Bridges Subjected to Traffic Loading,* Structural Research Series 416 NTIS-UILU-ENG-75-2004 (University of Illinois, April 1975).

[9] W. G. Byers, "Impact from Railway Loading or Steel Girder Spans," *Journal of the Structural Division,* ASCE, Vol 96, No. ST6 (June 1970), pp 1093-1103.

An estimate of V_{L2B} based on the uncertainty in predicting lateral load distribution can be obtained from Appendix H. Appendix H presents an assessment of the ratio of the effective number of stringers, N, computed by theory to N comouted by the current TM 5-312 procedure and provides values that range between 1.10 and 1.61 for single-lane military bridges and 1.00 and 1.75 for double-lane military bridges. For typical two-lane, concrete deck, AASHTO bridges[10] the ratio of N by theory to N by AASHTO ranges between 0.85 and 1.16. Conservatively, assuming a rectangular distribution, the resulting coefficient of variation V_{L2B} values for military bridges are 0.11 and 0.16 for single- and double-lanes, respectively. For permanent AASHTO bridges the coefficient of variation is 0.09. Reasonable estimates of V_{L2A} and V_{L2B} based on the previous analyses are

$$V_{L2A} = 0.10 \qquad \text{[Eq C6]}$$

$$V_{L2B} = \begin{cases} 0.20 \text{ stringer moment, military bridges} & \text{[Eq C7a]} \\ 0.15 \text{ stringer moment, AASHTO bridges} & \text{[Eq C7b]} \end{cases}$$

Thus, the values of V_{L2B} for stringer moment depend on what lateral load distribution formulas are used to determine the stringer moment.

Clearly, the values of V_{L2A} and V_{L2B} depend on the type of structural element such as beam, column, fastener, etc. Sufficient information to establish this dependency is not available. For this study, a V_{L2A} value of 0.10 was assumed for all structural elements for both military and AASHTO bridges. With the exception of stringer moment in

[10] W. W. Sanders and H. A. Elleby, *Distribution of Wheel Loads on Highway Bridges*, Report No. 83 (National Cooperative Highway Research Program, 1970).

AASHTO bridges, a value of V_{L2B} of 0.20 was assumed for all structural elements for both military and AASHTO bridges. Due to the lower COV value associated with the prediction of stringer moment for AASHTO bridges, a V_{L2B} value of 0.15 was assumed for AASHTO stringer moment.

Table C3 summarizes the assumptions used for the COV values for live load, including the V_L values. Use of these V_L values results in reasonable beta values for the representative permanent and temporary bridge criteria for members and fasteners (see Appendix D).

Table C3

Summary of Coefficients of Variation Assumptions Used for Live Load Effects

Description	V_{L1}	V_{L2A}	V_{L2B}	$V_{L2} = \sqrt{V_{L2A}^2 + V_{L2B}^2}$	$V_L = \sqrt{V_{L1}^2 + V_{L2}^2}$
All structural elements for military and AASHTO bridges, except stringer moment in AASHTO bridges	0.11	0.10	0.20	0.22	0.25
Stringer moment in AASHTO bridges	0.11	0.10	0.15	0.18	0.21

APPENDIX D:

DEVELOPMENT OF DESIGN CRITERIA FOR
STATIC LOAD CASE

The information in Appendices A, B, and C was used to calculate safety index (beta) values. Beta values were computed for mean dead to live load effect ratios D_m/L_m for members (Table B1) and fasteners (Table B2) for the 11 load cases analyzed (Table C1). Beta values for members and representative fasteners for selected load cases were tabulated for D_m/L_m values of 0.0, 0.1, 1.0, and 25.0. The D_m/L_m values of 0.1 and 1.0 are considered to represent the design range of actual brdiges; D_m/L_m values of 0.0 and 25.0 were included to show the effects of extreme load effect ratios. Throughout this appendix, graphs for selected members and fasteners illustrating beta as a function of D_m/L_m for load cases in Table C1 are provided. D_m/L_m on the graphs ranges from 0.0 to 500.0 to illustrate the effects of extreme load effects (the common D_m/L_m range is estimated to be from 0.1 to 1.0). The D_m/L_m scales on the graphs consist of a series of linear portions to illustrate the extreme D_m/L_m values.

Members Criteria

The following sections provide recommended Y values for members. Recall that Y, the allowable stress factor, is the ratio of the recommended allowable stress for temporary bridges to the base allowable stress for permanent bridges (for members, AASHTO allowable stress).

Flexure, Plastic Moment

Beta values for plastic moment are provided in Table D1 and graphically shown in Figure D1. A value of Y of 1.50 for temporary bridges appears reasonable and is recommended, provided the moments caused by a normal military crossing are used with the lateral load distribution formulas recommended in Appendix H.

To justify this recommendation, beta values corresponding to a D_m/L_m of 0.1, which is a representative lower value, were compared for various

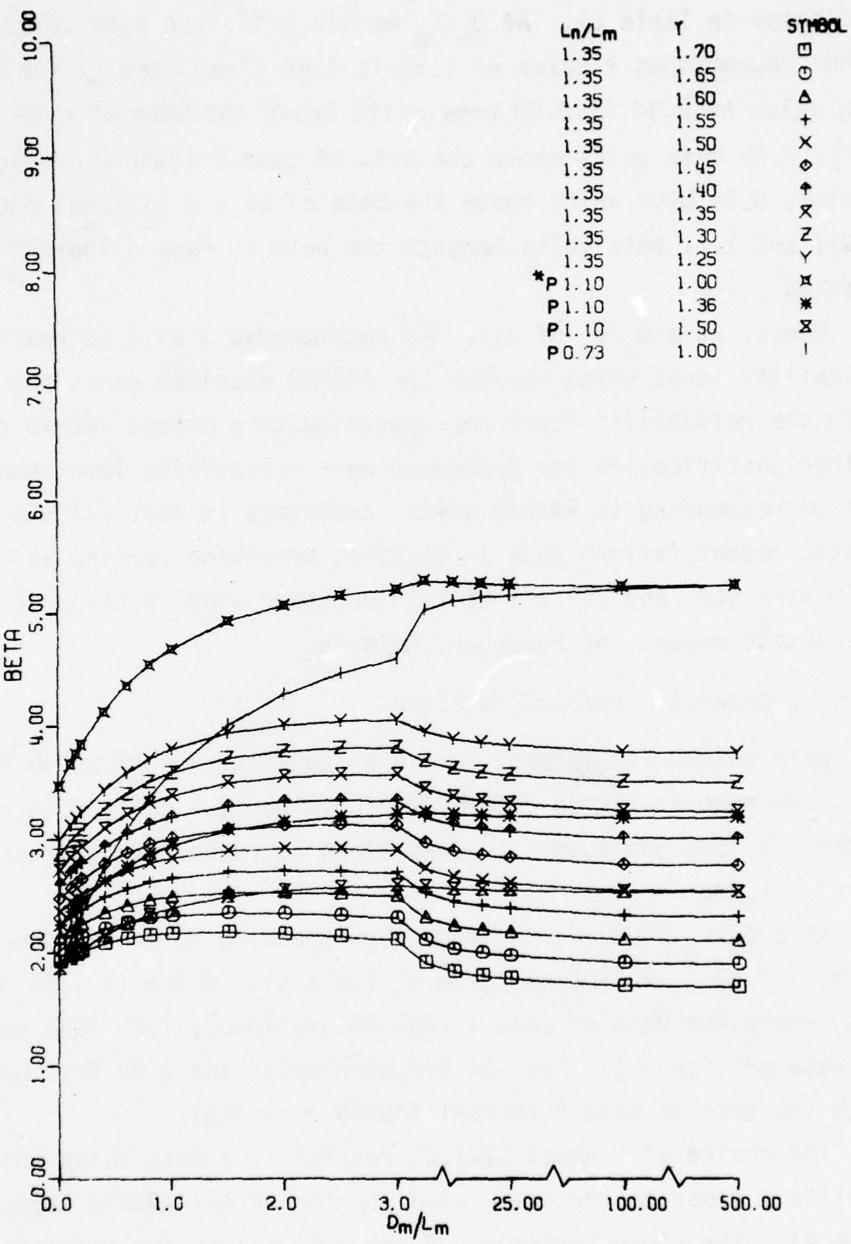
Table D1

Beta Comparison for Flexure -- Plastic Moment

Criteria Type *	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P	1	1.50**	1.10	1.85	1.94	2.39	2.52
P	2	1.36**	1.10	2.24	2.36	2.94	3.17
P	3	1.00**	0.73	1.84	2.10	3.58	5.13
P	4	1.00**	1.10	3.46	3.65	4.68	5.23
T	5	1.20	1.35	3.13	3.27	4.00	4.07
T	5	1.25	1.35	2.99	3.12	3.79	3.80
T	5	1.30	1.35	2.85	2.97	3.58	3.54
T	5	1.35	1.35	2.72	2.83	3.38	3.28
T	5	1.40	1.35	2.59	2.70	3.19	3.04
T	5	1.45	1.35	2.47	2.56	3.00	2.81
T	5	1.50	1.35	2.35	2.44	2.82	2.58
T	5	1.55	1.35	2.24	2.31	2.65	2.36
T	5	1.60	1.35	2.13	2.19	2.48	2.15
T	5	1.65	1.35	2.02	2.08	2.32	1.94
T	5	1.70	1.35	1.92	1.97	2.16	1.75
T	5	1.75	1.35	1.81	1.86	2.01	1.55
T	5	1.80	1.35	1.72	1.75	1.86	1.36

* P = permanent (AASHTO) bridges; T = temporary (military) bridge.

** These cases based upon V_{L2} = 0.18 (AASHTO); all other cases upon V_{L2} = 0.22.



* P refers to permanent bridge criteria; all other Y and L_n/L_m combinations are for temporary bridge criteria.

Figure D1. Beta versus D_m/L_m for flexure-plastic moment for selected L_n/L_m and Y combinations.

load cases in Table C1. At D_m/L_m equals 0.10, the beta value corresponding to the recommended Y value of 1.50 is 2.44 (load case 5, Table C1). This beta value of 2.44 is 0.50 beta units above the beta of case 1 (AASHTO overload); 0.08 beta units above the beta of case 2 (AASHTO overload with permit); 0.34 beta units above the beta of case 3 (illegal AASHTO overload); and 1.21 beta units beneath the beta of case 4 (normal AASHTO crossing).

Hence, at a D_m/L_m of 0.1, the recommended Y of 1.50 results in a reliability level which exceeds the AASHTO overload cases and is beneath the reliability level corresponding to a normal AASHTO crossing. Further justification for recommending a reliability level beneath that corresponding to AASHTO normal crossings is that (1) the flexure-plastic moment failure mode is ductile, providing warning and post yield strength, and (2) a Y of 1.50 has been used in the past for flexure-plastic moment for temporary bridges.

Flexure, Lateral Torsional Buckling

Beta values for lateral torsional buckling are given in Table D2 and shown graphically in Figure D2. A value of Y of 1.20 is recommended provided the moment is determined as recommended for plastic moment design.

At a D_m/L_m of 0.10, the beta corresponding to the recommended Y value of 1.20 is 2.71 (load case 5, Table C1), which is 1.35 beta units above the beta of case 1 (AASHTO overload); 1.20 beta units above the beta of case 3 (illegal AASHTO overload); and 0.28 beta units beneath the beta of case 4 (normal AASHTO crossing).

The choice of Y equal to 1.20 results in a beta value which is relatively close to the beta value for the normal AASHTO crossing (case 4). The close agreement of the beta values for temporary and permanent criteria reflects the nature of the lateral torsional buckling failure mode, which is often unstable and gives little warning.

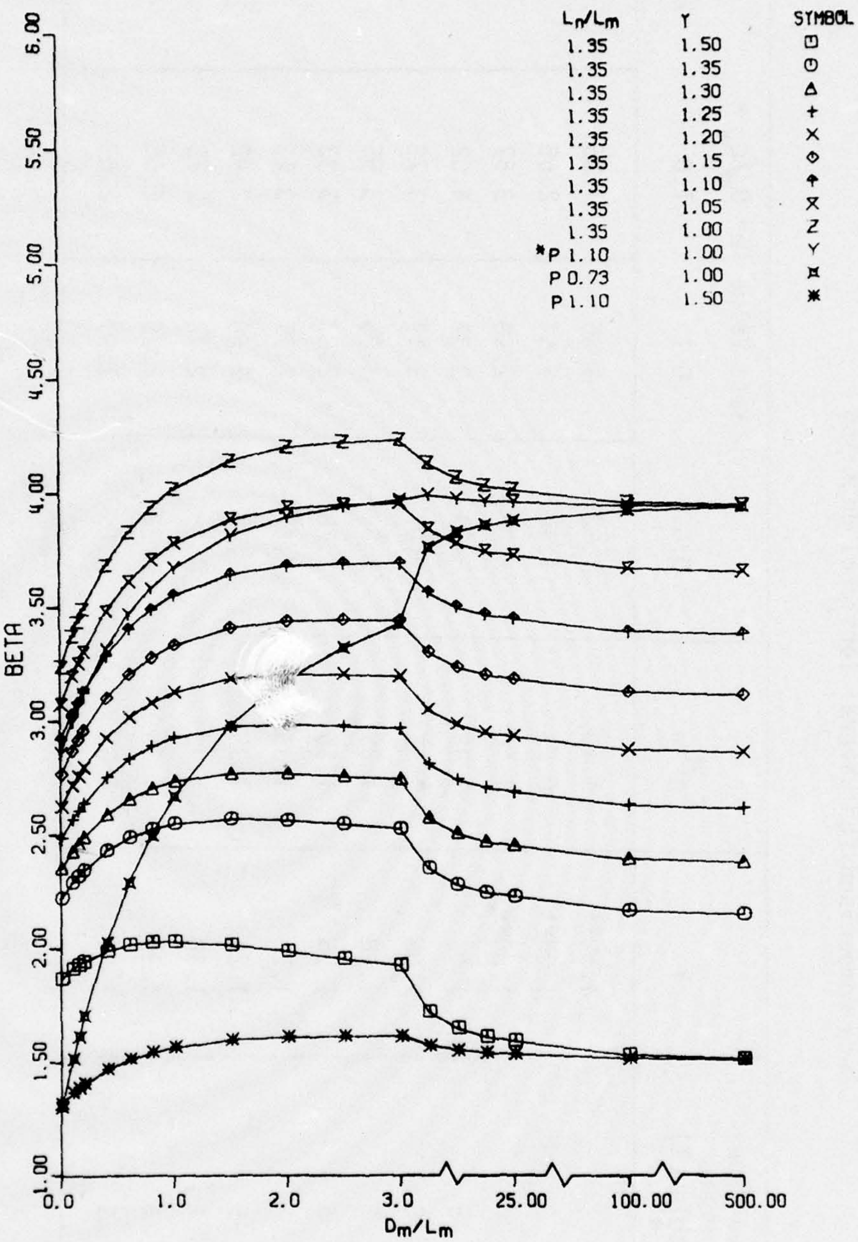
Table D2

Beta Comparison for Lateral Torsional Buckling

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m =			
				0.0	0.1	1.0	25.0
P	1	1.50**	1.10	1.32	1.36	1.56	1.53
P	3	1.00**	0.73	1.30	1.51	2.66	3.86
P	4	1.00**	1.10	2.85	2.99	3.67	3.95
T	5	1.00	1.35	3.24	3.37	4.02	4.00
T	5	1.05	1.35	3.07	3.19	3.78	3.71
T	5	1.10	1.35	2.91	3.03	3.55	3.43
T	5	1.15	1.35	2.76	2.87	3.33	3.17
T	5	1.20	1.35	2.62	2.71	3.13	2.91
T	5	1.25	1.35	2.48	2.56	2.92	2.67
T	5	1.30	1.35	2.35	2.42	2.73	2.44
T	5	1.35	1.35	2.22	2.29	2.55	2.21
T	5	1.50	1.35	1.87	1.91	2.03	1.58

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.

** These cases based on V_{L2} = 0.18 (AASHTO); all other cases based on V_{L2} = 0.22.



* P refers to permanent bridge criteria; all other Y and L_n/L_m combinations are for temporary bridge criteria.

Figure D2. Beta versus D_m/L_m for lateral torsional buckling for selected L_n/L_m and Y combinations.

Shear

Beta values for shear are shown in Table D3 and depicted graphically in Figure D3. A Y value of 1.10 is recommended, provided the recommended procedure to determine the shear force given in Appendix I is used.

At D_m/L_m = 0.10, the beta corresponding to the recommended Y of 1.10 is 2.02 (case 10), which is 1.12 beta units above the beta of case 6 (AASHTO overload); 0.94 beta units above the beta of case 7 (illegal AASHTO overload); and 0.34 beta units beneath the beta of case 9 (normal AASHTO crossing).

It should be noted that the shear force determination procedure given in Appendix I results in using the larger of the shear forces resulting from normal or caution military crossings. The "1.25" coefficient in Table I2 accounts for the caution military crossing. If the shear force were based only on the normal military crossing, which is equivalent to removing the "1.25" coefficient in Table I2, then a Y of 0.90 in Table D3 (load case 11, Table C1) would be recommended.

As Table D3 shows, the beta values for Y equals 1.10 (case 10) and Y equals 0.90 (case 11) are similar. Thus, a 22 percent reduction (1.10/0.90 ≈ 1.22) in the allowable shear stress would be required if only the normal military corssing case is used to determine the shear force.

Table D3 also shows that beta values for the current TM 5-312 allowable shear stress (Y = 1.50) are very low. If the current procedure to determine the shear force in TM 5-312 were used (as opposed to Appendix I recommendations) the beta values could be even lower. Hence the current TM 5-312 shear criteria are judged unsafe and are not recommended.

Compression in Axially Loaded Columns
(Compression Members)

Beta values for compression members are shown in Tables D4 to D9 for values of λ equal to 0.2, 0.4, 0.6, 0.8, 1.0, and 1.2 (λ is a

Table D3

Beta Comparison for Shear

Criteria Type*	Load Case Number (Table C1)	γ	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	0.84	0.90	1.22	1.49
T	10	1.25	1.00	1.46	1.56	2.12	2.58
T	10	1.20	1.00	1.60	1.71	2.32	2.82
T	10	1.15	1.00	1.74	1.86	2.53	3.07
T	10	1.10	1.00	1.89	2.02	2.75	3.34
T	10	1.05	1.00	2.05	2.19	2.98	3.62
P, T	9, 10	1.00	1.00	2.21	2.36	3.22	3.91
P	7	1.00	0.67	0.86	1.08	2.33	3.83
T	11	1.50	0.80	0.09	0.18	0.71	1.44
T	11	1.20	0.80	0.84	0.98	1.80	2.77
T	11	1.15	0.80	0.99	1.13	2.01	3.03
T	11	1.10	0.80	1.14	1.29	2.23	3.29
T	11	1.05	0.80	1.29	1.46	2.46	3.57
T	11	1.00	0.80	1.46	1.64	2.70	3.86
T	11	0.95	0.80	1.63	1.82	2.95	4.17
T	11	0.90	0.80	1.82	2.02	3.22	4.49
T	11	0.85	0.80	2.01	2.23	3.50	4.83
T	11	0.80	0.80	2.21	2.45	3.80	5.20

*P = permanent (AASHTO) bridges; T = temporary (military) bridges.

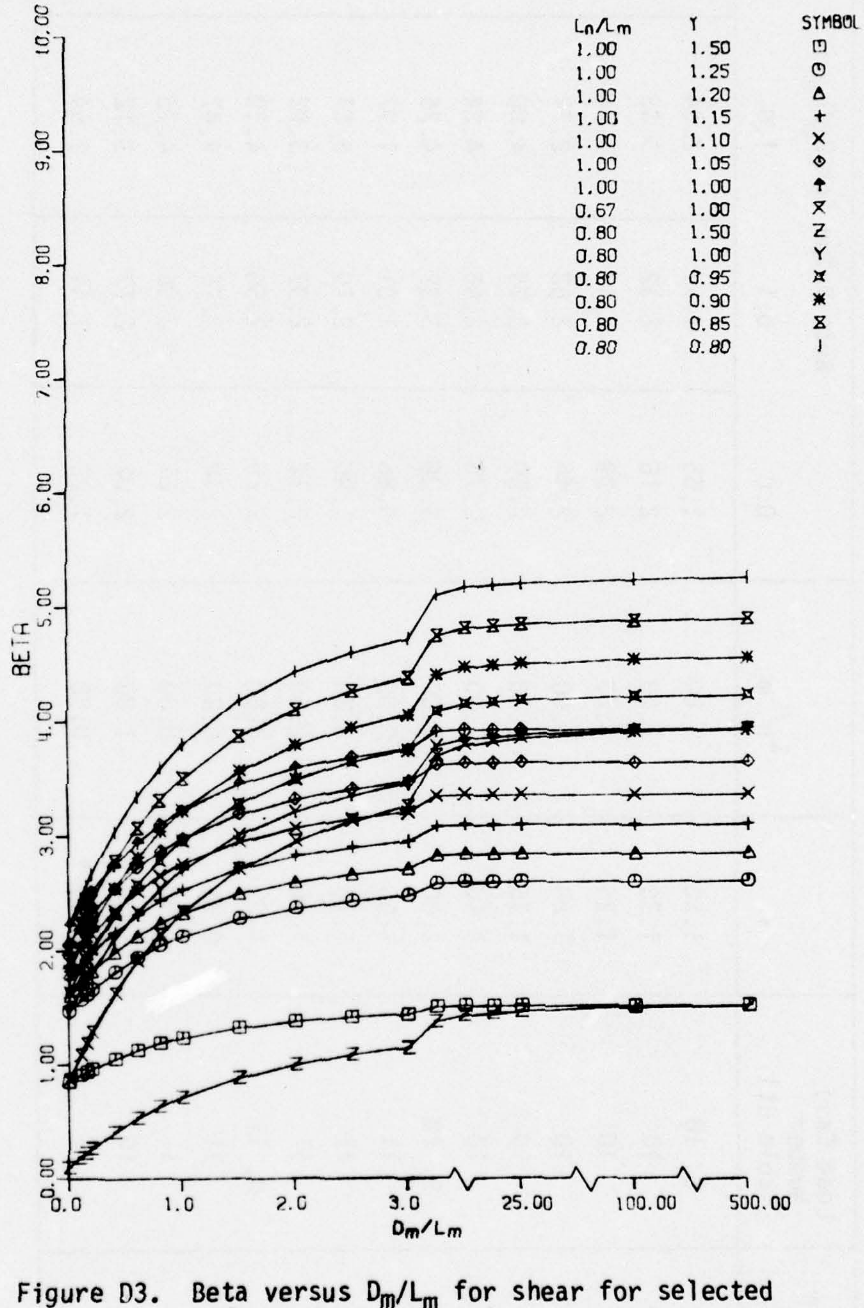


Figure D3. Beta versus D_m/L_m for shear for selected L_n/L_m and Y combinations.

Table D4

Beta Comparison for Compression Members, $\lambda = 0.2$

Criteria Type*	Load Case Number (Table C1)	γ	L_n/L_m	Beta Value for D_m/L_m 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	1.65	1.77	2.54	3.29
T	10	1.30	1.00	2.16	2.32	3.33	4.30
T	10	1.25	1.00	2.29	2.47	3.54	4.58
T	10	1.20	1.00	2.44	2.63	3.77	4.87
T	10	1.15	1.00	2.59	2.79	4.00	5.17
T	10	1.10	1.00	2.75	2.96	4.24	5.48
P, T	9, 10	1.00	1.00	3.09	3.32	4.76	6.16
T	11	1.50	0.80	0.86	1.01	1.97	3.23
T	11	1.10	0.80	1.96	2.19	3.67	5.43
T	11	1.05	0.80	2.12	2.37	3.92	5.76
P, T	8, 11	1.00	0.80	2.29	2.56	4.19	6.10
T	11	0.95	0.80	2.48	2.75	4.47	6.47
T	11	0.90	0.80	2.67	2.96	4.76	6.85
T	10	1.00**	1.00	2.02	2.17	3.12	4.03
T	11	1.00**	0.80	1.23	1.41	2.54	3.97

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.

** These cases are for compressive stress given by TM 5-312 as $F_a = 21300 - 3/8(\ell/r)^2$ adjusted for $F_y = 36$ ksi, where ℓ = unsupported length in inches, r = least radius gyration; F_a in terms of $F_y = 36$ ksi is $F_a = F_y[0.592 - 0.0828\lambda^2]$.

Table D5

Beta Comparison for Compression Members, $\lambda = 0.4$

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	1.56	1.67	2.35	2.97
P, T	9, 10	1.00	1.00	2.97	3.19	4.49	5.68
T	10	1.25	1.00	2.19	2.35	3.31	4.19
T	10	1.20	1.00	2.33	2.51	3.53	4.46
T	10	1.15	1.00	2.48	2.67	3.76	4.74
T	10	1.10	1.00	2.64	2.83	3.99	5.04
T	10	1.05	1.00	2.80	3.01	4.24	5.35
T	11	1.50	0.80	0.78	0.92	1.79	2.92
T	11	1.05	0.80	2.02	2.25	3.68	5.30
P, T	8, 11	1.00	0.80	2.19	2.44	3.94	5.62
T	11	0.95	0.80	2.37	2.63	4.21	5.97
T	11	0.90	0.80	2.56	2.83	4.49	6.33
T	10	1.00**	1.00	1.87	2.01	2.83	3.58
T	11	1.00**	0.80	1.09	1.26	2.28	3.53

*P = permanent (AASHTO) bridges; T = temporary (military) bridges.
**Based on TM 5-312 allowable stress. See footnote in Table D4.

Table D6

Beta Comparison for Compression Members, $\lambda = 0.6$

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	1.30	1.39	1.89	2.30
T	10	1.25	1.00	1.92	2.05	2.79	3.39
T	10	1.20	1.00	2.06	2.20	2.99	3.63
T	10	1.15	1.00	2.20	2.35	3.20	3.89
T	10	1.10	1.00	2.35	2.51	3.42	4.15
T	10	1.05	1.00	2.51	2.68	3.64	4.43
P, T	9, 10	1.00	1.00	2.67	2.85	3.88	4.72
T	11	1.50	0.80	0.55	0.67	1.37	2.25
T	11	1.15	0.80	1.45	1.63	2.68	3.84
T	11	1.10	0.80	1.60	1.79	2.90	4.10
T	11	1.05	0.80	1.75	1.95	3.13	4.38
P, T	11	1.00	0.80	1.92	2.13	3.37	4.67
T	11	0.95	0.80	2.09	2.32	3.62	4.98
T	11	0.90	0.80	2.27	2.51	3.88	5.30
T	11	0.85	0.80	2.47	2.72	4.16	5.64
T	11	0.80	0.80	2.67	2.94	4.46	6.01
T	10	1.00**	1.00	1.53	1.63	2.22	2.70
T	11	1.00**	0.80	0.77	0.91	1.70	2.65

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** Based on TM 5-312 allowable stresses. See footnote in Table D4.

Table D7

Beta Comparison for Compression Members, $\lambda = 0.8$

Criteria Type*	Load Case Number (Table C1)	γ	L_n/L_m	Beta Value for D_m/L_m 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	1.15	1.23	1.62	1.91
T	10	1.15	1.00	2.02	2.15	2.83	3.34
T	10	1.10	1.00	2.17	2.30	3.04	3.58
T	10	1.05	1.00	2.32	2.46	3.25	3.83
P, T	9, 10	1.00	1.00	2.48	2.63	3.47	4.09
T	10	0.95	1.00	2.64	2.81	3.71	4.37
T	11	1.50	0.80	0.43	0.53	1.14	1.86
T	8, 11	1.00	0.80	1.75	1.94	2.99	4.05
T	11	0.95	0.80	1.92	2.12	3.22	4.32
T	11	0.90	0.80	2.09	2.30	3.47	4.62
T	11	0.85	0.80	2.28	2.50	3.73	4.92
T	11	0.80	0.80	2.48	2.71	4.01	5.25
T	10	1.00**	1.00	1.25	1.33	1.75	2.06
T	11	1.00**	0.80	0.52	0.63	1.27	2.02

*P = permanent (AASHTO) bridges; T = temporary (military) bridges.
**Based on TM 5-312 allowable stresses. See footnote in Table D4.

Table D8

Beta Comparison for Compression Members, $\lambda = 1.0$

Criteria Type*	Load Case Number (Table C1)	γ	L_n/L_m	Beta Value for D_m/L_m			
				0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	0.96	1.02	1.32	1.53
T	10	1.25	1.00	1.54	1.64	2.12	2.47
T	10	1.15	1.00	1.81	1.92	2.49	2.90
T	10	1.10	1.00	1.95	2.07	2.69	3.13
T	10	1.05	1.00	2.10	2.23	2.89	3.37
P, T	9, 10	1.00	1.00	2.26	2.39	3.11	3.62
T	10	0.95	1.00	2.42	2.57	3.34	3.88
T	10	0.90	1.00	2.59	2.75	3.57	4.16
T	11	1.50	0.80	0.24	0.33	0.85	1.50
P, T	8, 11	1.00	0.80	1.54	1.71	2.64	3.58
T	11	0.95	0.80	1.71	1.89	2.87	3.84
T	11	0.90	0.80	1.88	2.07	3.11	4.12
T	11	0.85	0.80	2.06	2.26	3.36	4.41
T	11	0.80	0.80	2.26	2.47	3.63	4.72
T	11	0.75	0.80	2.46	2.69	3.91	5.05
T	10	1.00**	1.00	0.87	0.92	1.20	1.39
T	11	1.00**	0.80	0.15	0.24	0.73	1.35

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** Based on TM 5-312 allowable stresses. See footnote in Table D4.

Table D9

Beta Comparison for Compression Members, $\lambda = 1.2$

Criteria Type*	Load Case Number (Table C1)	γ	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	0.86	0.92	1.21	1.43
T	10	1.10	1.00	1.88	1.99	2.63	3.10
T	10	1.05	1.00	2.03	2.16	2.84	3.35
P, T	9, 10	1.00	1.00	2.19	2.33	3.06	3.61
T	10	0.95	1.00	2.35	2.50	3.30	3.89
T	10	0.90	1.00	2.53	2.69	3.55	4.18
T	11	1.50	0.80	0.14	0.22	0.73	1.38
P,T	8, 11	1.00	0.80	1.46	1.63	2.58	3.57
T	11	0.85	0.80	1.99	2.19	3.33	4.44
T	11	0.80	0.80	2.19	2.40	3.60	4.77
T	11	0.75	0.80	2.40	2.63	3.90	5.12
T	11	0.70	0.80	2.62	2.87	4.21	5.49
T	10	1.00**	1.00	0.50	0.53	0.70	0.82
T	11	1.00**	0.80	-0.23	-0.17	0.22	0.78

*P = permanent (AASHTO) bridges; T = temporary (military) bridges.
**Based on TM 5-312 allowable stresses. See footnote in Table D4.

column slenderness parameter, defined in Table B1). Beta is graphically shown for values of λ equal to 0.4, 0.8, and 1.2 in Figures D4 to D6.

Table D10 shows two sets of recommended Y values. The larger Y values (1.20 to 0.95) are recommended when the larger of the compressive forces resulting from the normal or caution military crossings (load case 10, Table C1) is used in design. The smaller Y values (1.00 to 0.75) are recommended when the force resulting from the normal military crossing only (load case 11) is used for design.

The Y values in table D10 are given as functions of the slenderness parameter λ and the slenderness ratio $K\ell/r$ (where $K\ell/r$ is based on F_y = 36 ksi minimum yield point).

The larger Y values in Table D10, corresponding to using the larger force from the normal or caution military crossing, can be expressed as a continuous linear function of λ:

$$Y = 1.23 - 0.222\lambda \quad \text{for } \lambda \leq 1.2 \qquad \text{[Eq D1]}$$

or equivalently, Y in terms of F_y and $K\ell/r$ is

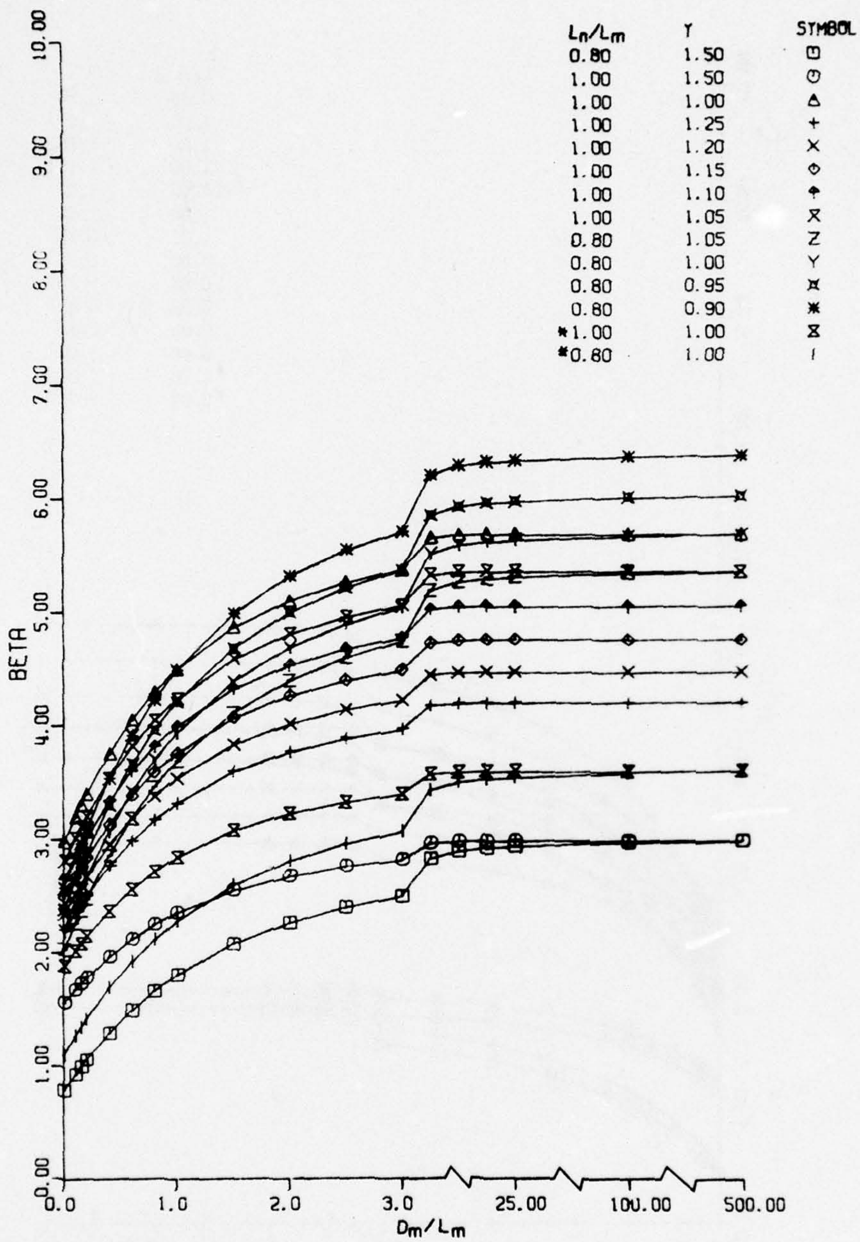
$$Y = 1.23 - [(K\ell/r)0.000415\sqrt{F_y}] \qquad \text{[Eq D2]}$$

provided $K\ell/r \leq 91$ for F_y = 50 ksi and $K\ell/r \leq 107$ for F_y = 36 ksi. For F_y = 36 ksi, Eq D2 becomes

$$Y = 1.23 - [0.0025(K\ell/r)] \quad \text{for } K\ell/r < 107 \qquad \text{[Eq D3]}$$

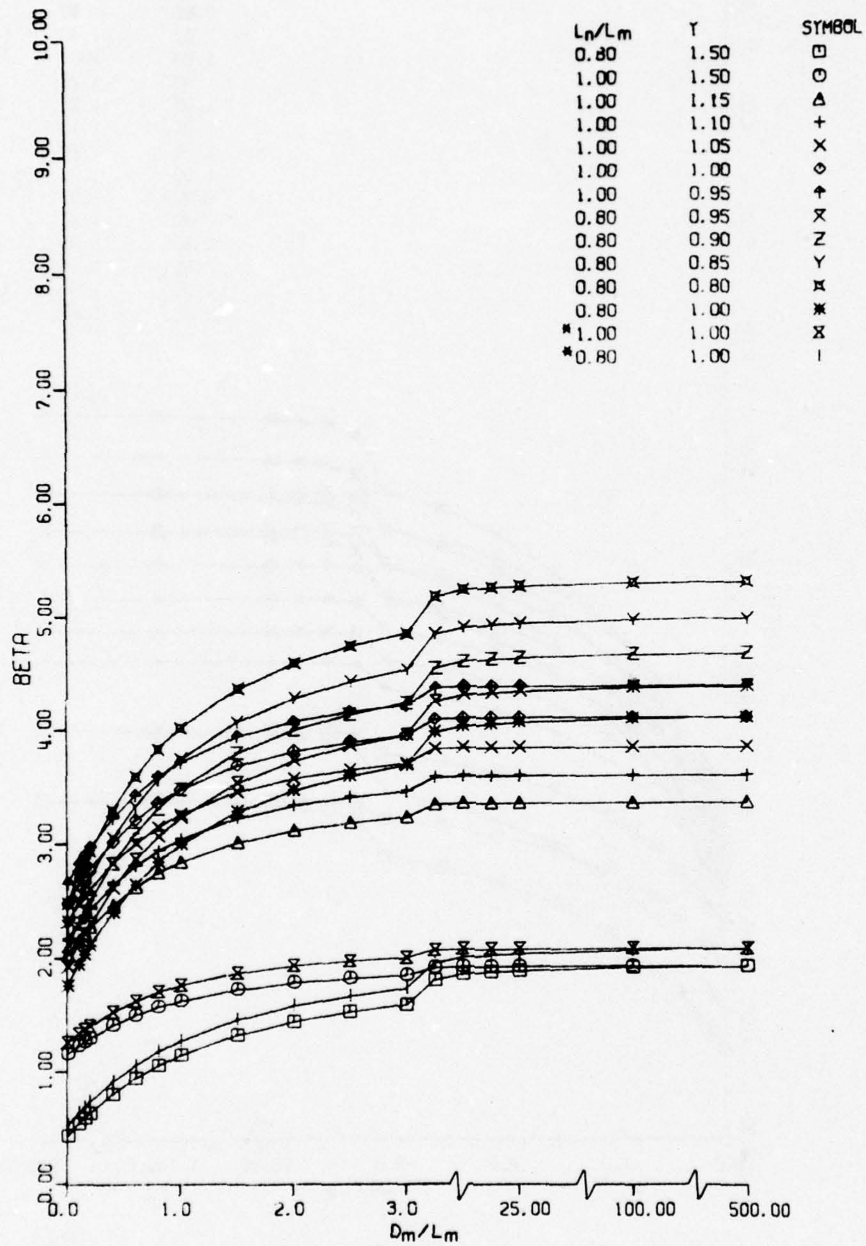
In a similar manner, the smaller Y values in Table D10 which correspond to using the force from the normal military crossing only can be expressed as

$$Y = 1.03 - 0.222\lambda \quad \text{for } \lambda \leq 1.2 \qquad \text{[Eq D4]}$$



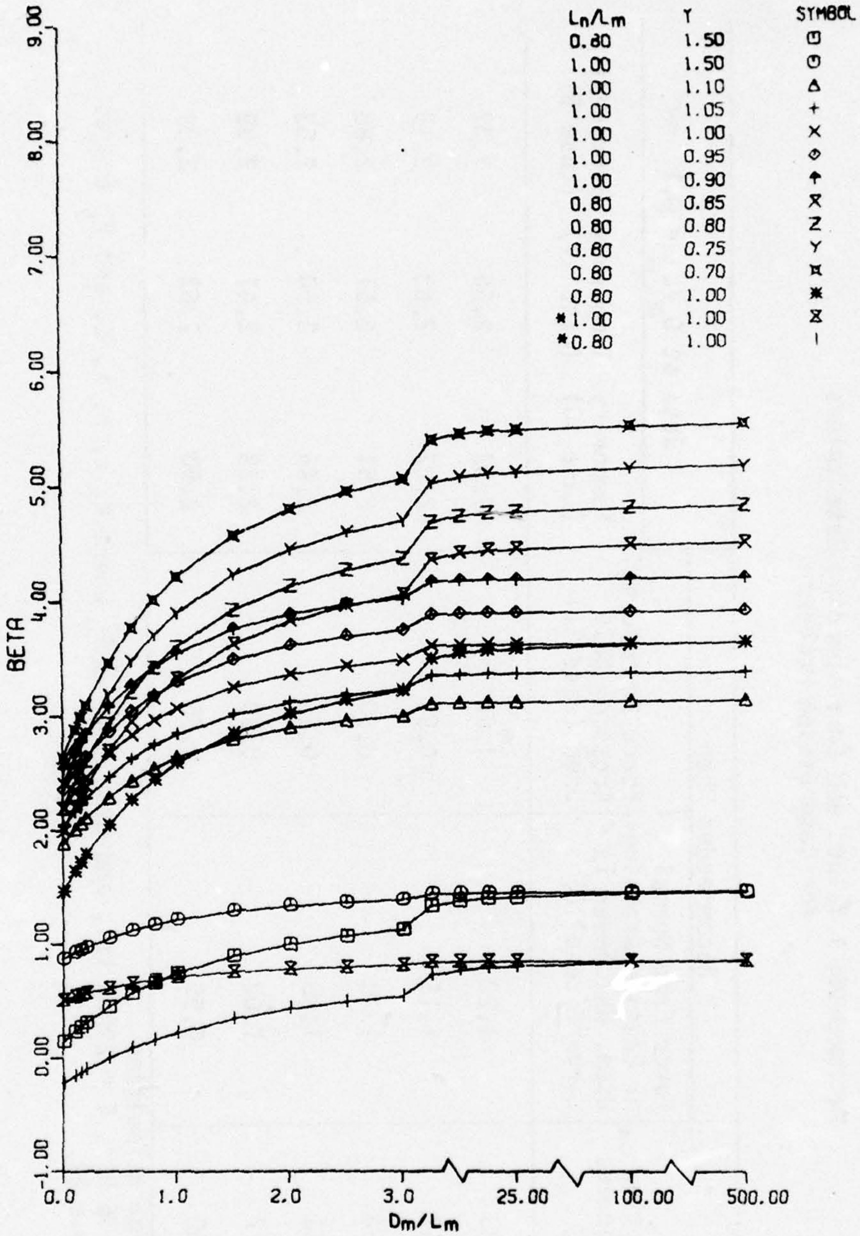
* These cases are based on the allowable stress as given by TM 5-312--see footnote in Table D4.

Figure D4. Beta versus D_m/L_m for compression members, λ = 0.4, for selected L_n/L_m and Y combinations.



* These cases are based on the allowable stress as given by TM 5-312--see footnote in Table D4.

Figure D5. Beta versus D_m/L_m for compression members, λ = 0.8, for selected L_n/L_m and Y combinations.



* These cases are based on the allowable stress as given by TM 5-312--see footnote in Table D4.

Figure D6. Beta versus D_m/L_m for compression members, $\lambda = 1.2$, for selected L_n/L_m and Y combinations.

Table D10

Recommended Y Values and Corresponding Beta Values for Compression Members

Slenderness Parameter* λ	Effective Slenderness** $K\ell/r$	Y Recommended When: Force From Normal or Caution Crossings Used, Whichever Is Larger (case 10)	Y Recommended When: Force From Normal Crossing Only Used (case 11)	Beta at $D_m/L_m = 0.1$: Temporary (case 10)	Beta at $D_m/L_m = 0.1$: Temporary (case 11)	Beta at $D_m/L_m = 0.1$: Permanent (case 9)
0.2	17.83	1.20	1.00	2.63	2.56	3.32
0.4	35.67	1.15	0.95	2.67	2.63	3.19
0.6	53.50	1.10	0.90	2.51	2.51	2.85
0.8	71.33	1.05	0.85	2.46	2.50	2.63
1.0	89.17	1.00	0.80	2.39	2.47	2.39
1.2	107.00	0.95	0.75	2.50	2.63	2.33

* See Table B1 for definition

** Based on F_y = 36 ksi, E = 29000 ksi, and $K\ell/r = \pi\lambda\sqrt{E/F_y}$ where K, ℓ, r, λ, E, and F_y are as defined in Table B1.

or equivalently, Y in terms of F_y and $K\ell/r$ is

$$Y = 1.03 - [(K\ell/r)0.000415\sqrt{F_y}] \qquad \text{[Eq D5]}$$

provided $K\ell/r \leq 91$ for $F_y = 50$ ksi and $K\ell/r \leq 107$ for $F_y = 36$ ksi.

For $F_y = 36$ ksi Eq D5 becomes

$$Y = 1.03 - [0.0025(K\ell/r)] \quad \text{for } K\ell/r < 107 \qquad \text{[Eq D6]}$$

The following justification is provided for the recommended Y values in Table D10, which shows the beta values at $D_m/L_m = 0.1$ for the recommended criteria for temporary bridges (cases 10 and 11) and the AASHTO criteria for a normal AASHTO crossing (case 9). The recommended Y values correspond to beta values of about 2.50. For $\lambda > 0.4$, the beta of 2.50 is relatively close to, or exceeds, the beta for permanent criteria. The Y values were chosen on the conservative side to account for the column buckling failure mode, which is often unstable and gives little warning.

It should be noted that to maintain comparable beta values (Table D10, case 10 versus case 11), the Y values are reduced 20 to 27 percent (Y for case 10/Y for case 11) to account for the caution military crossing force controlling when only the normal military crossing force was used in design.

It is also interesting to note that, as shown in Tables D4 to D9, the beta corresponding to the current TM 5-312 criteria (denoted by double asterisks) are very low for the caution military crossing (case 11) for all λ values and for the normal military crossing (case 10) for $\lambda > 0.4$. Hence, use of the TM 5-312 criteria is judged unsafe and is not recommended.

Tension Members

The analysis of tension members (connected material in tension) is given in the Fastener and Connection Criteria section of this appendix.

The recommendations are summarized here. A Y value of 1.33 is recommended when the larger tensile force resulting from the normal or caution military crossings is used. If only the tensile force resulting from the normal military crossing is used, a Y value of 1.07 is recommended. Hence, a 24 percent (1.33/1.07) reduction in Y is recommended if only the normal military crossing force is used in design. The reduction in allowable stress provides increased load capacity under caution crossings.

Summary

Table D11 summarizes the recommended criteria for members.

Fasteners and Connection Criteria

This section describes the development of recommended Y values for fasteners and connections.

As discussed in Appendix B, although criteria were developed for 36 resistances, only eight representative resistances, representing the range of values in the four groups, were analyzed (Table B2). Tables D12 through D19 give the beta values for the eight representative resistances. Beta values for five of the representative resistances are shown in Figures D7 through D11. The forces used to design fasteners and connections are directly related to their corresponding member forces. Hence, in Tables D12 through D19, values of L_n/L_m for stringer end connections of 0.67 (load case 7, Table C1), 0.80 (case 11), and 1.0 (cases 9 and 10) were used. For fasteners in connections of tension and compression members, values of L_n/L_m of 1.0 (cases 6, 9, and 10) and 0.80 (cases 8 and 11) were used. For fasteners in flexural splices, L_n/L_m values of 1.10 (cases 1 and 4), and 1.35 (case 5) were used.

Table D20 presents the recommended Y values for L_n/L_m values of 0.8 and 1.0 for temporary bridges. The recommended Y values were determined by choosing Y values such that their corresponding beta values agreed with the beta values of Table D21 for L_n/L_m = 0.80 or 1.0.

Table D11

Summary of Recommended Criteria for Members

Member Type	Y	Recommended Allowable Stress	Additional Requirements
Flexure--plastic moment	1.50	0.83 F_y provided the member meets the requirements of Section 1.5.1.4.1 of AISC*	Use recommended lateral load distribution formulas (effective number of stringers) in Appendix H of this report. Use moment corresponding to a normal military crossing in Appendix D of TM 5-312.
Flexure--lateral torsional buckling	1.20	1.20 F_b where F_b is the AASHTO (1973) allowable stress	
Shear	1.10	0.36 F_y	Use recommended procedure in Appendix I of this report to determine shear force
Compression members	See Eq D2 and D3	Y F_a Where F_a is the AASHTO (1973) allowable stress for concentrically loaded columns	Use the larger compressive force resulting from the normal or caution military crossings
Compression members	See Eq D5 and D6	Y F_a Where F_a is AASHTO (1973) allowable stress for concentrically loaded columns	Compressive force determination based only on normal military crossing case
Tension members	1.33	0.73 F_y	Use the larger tensile force resulting from the normal or caution military crossings
Tension members	1.07	0.59 F_y	Tensile force determination based only on normal military crossing case

* *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* (American Institute of Steel Construction [AISC], 1969), with supplements 1 (1970), 2 (1971), and 3 (1974).

Table D12

Beta Comparison for Fasteners and Connections
Group I, Identification No. 1 (Table B2), $f_m/F_a = 2.26$, $V_R = 0.12$

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	1.45	1.57	2.25	2.90
T	10	1.20	1.00	2.25	2.42	3.47	4.48
T	10	1.15	1.00	2.40	2.58	3.70	4.78
T	10	1.10	1.00	2.56	2.75	3.94	5.10
T	10	1.05	1.00	2.72	2.93	4.20	5.43
P, T	9, 10	1.00	1.00	2.89	3.11	4.47	5.77
P	7	1.00	0.67	1.47	1.75	3.48	5.68
T	11	1.50	0.80	0.66	0.80	1.67	2.85
P, T	8, 11	1.00	0.80	2.10	2.35	3.89	5.72
T	11	0.95	0.80	2.28	2.54	4.17	6.08
T	11	0.90	0.80	2.48	2.75	4.47	6.46
T	11	0.85	0.80	2.68	2.97	4.78	6.87
T	11	0.80	0.80	2.89	3.20	5.11	7.30
P	4**	1.00	1.10	3.68	3.89	5.09	5.80
T	5	1.10	1.35	3.62	3.80	4.83	5.19
T	5	1.15	1.35	3.46	3.63	4.58	4.88
T	5	1.20	1.35	3.31	3.47	4.35	4.58
T	5	1.25	1.35	3.17	3.32	4.13	4.29
T	5	1.30	1.35	3.03	3.17	3.81	4.01

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** This case based on $V_{L2} = 0.18$; all other cases based on $V_{L2} = 0.22$.

Table D13

Beta Comparison for Fasteners and Connections
Group I, Identification No. 15 (Table B2), $f_m/F_a = 3.09$, $V_R = 0.12$

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	2.56	2.76	3.96	5.12
T	10	1.55	1.00	2.45	2.63	3.78	4.88
T	10	1.50	1.00	2.56	2.76	3.96	5.12
T	10	1.45	1.00	2.69	2.89	4.14	5.36
T	10	1.40	1.00	2.81	3.02	4.34	5.61
T	10	1.35	1.00	2.94	3.16	4.54	5.86
P	9, 10	1.00	1.00	4.00	4.31	6.18	7.99
P	7	1.00	0.67	2.58	2.95	5.19	7.90
T	11	1.50	0.80	1.77	1.99	3.38	5.06
T	11	1.30	0.80	2.28	2.54	4.17	6.08
T	11	1.25	0.80	2.42	2.69	4.38	6.35
T	11	1.20	0.80	2.56	2.85	4.60	6.64
T	11	1.15	0.80	2.72	3.01	4.84	6.94
T	11	1.10	0.80	2.87	3.18	5.08	7.26
P, T	8, 11	1.00	0.80	3.21	3.54	5.60	7.93
P	4**	1.00	1.10	4.94	5.24	6.93	8.02
T	5	1.70	1.35	3.19	3.34	4.16	4.33
T	5	1.65	1.35	3.29	3.45	4.32	4.54
T	5	1.60	1.35	3.40	3.57	4.49	4.75
T	5	1.55	1.35	3.51	3.69	4.66	4.98
T	5	1.50	1.35	3.63	3.81	4.84	5.21

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** This case based on $V_{L2} = 0.18$; all other cases based on $V_{L2} = 0.22$.

Table D14

Beta Comparison for Fasteners and Connections
Group I, Identification No. 22 (Table B2), $f_m/F_a = 3.68$, $V_R = 0.12$

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	3.18	3.43	4.92	6.35
T	10	1.80	1.00	2.54	2.73	3.92	5.06
T	10	1.75	1.00	2.64	2.84	4.07	5.26
T	10	1.70	1.00	2.74	2.95	4.23	5.47
T	10	1.65	1.00	2.85	3.06	4.39	5.68
T	10	1.60	1.00	2.96	3.18	4.56	5.90
P, T	9, 10	1.00	1.00	4.62	4.98	7.14	9.22
P	7	1.00	0.67	3.20	3.61	6.15	9.13
T	11	1.50	0.80	2.39	2.66	4.34	6.30
T	11	1.45	0.80	2.51	2.79	4.52	6.54
T	11	1.40	0.80	2.64	2.92	4.72	6.79
T	11	1.35	0.80	2.77	3.06	4.92	7.04
T	11	1.30	0.80	2.90	3.21	5.12	7.31
P, T	8, 11	1.00	0.80	3.83	4.21	6.56	9.17
P	4**	1.00	1.10	5.64	6.00	7.96	9.26
T	5	2.05	1.35	3.14	3.29	4.09	4.24
T	5	2.00	1.35	3.23	3.38	4.22	4.41
T	5	1.95	1.35	3.32	3.48	4.36	4.59
T	5	1.90	1.35	3.41	3.58	4.50	4.77
T	5	1.85	1.35	3.51	3.68	4.65	4.96

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** This case based on $V_{L2} = 0.18$; all other cases based on $V_{L2} = 0.22$.

Table D15

Beta Comparison for Fasteners and Connections
Group II, Identification No. 23 (Table B2), $f_m/F_a = 2.73$, $V_R = 0.14$

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	2.06	2.20	3.05	3.78
T	10	1.40	1.00	2.30	2.46	3.40	4.21
T	10	1.35	1.00	2.42	2.59	3.59	4.44
T	10	1.30	1.00	2.55	2.73	3.78	4.68
T	10	1.25	1.00	2.69	2.88	3.98	4.93
T	10	1.20	1.00	2.83	3.03	4.19	5.18
P, T	9, 10	1.00	1.00	3.45	3.70	5.12	6.33
P	7	1.00	0.67	2.08	2.38	4.20	6.25
T	11	1.50	0.80	1.29	1.47	2.51	3.73
T	11	1.10	0.80	2.36	2.61	4.09	5.68
T	11	1.05	0.80	2.52	2.78	4.33	5.98
P, T	8, 11	1.00	0.80	2.69	2.96	4.58	6.28
T	11	0.95	0.80	2.86	3.15	4.84	6.61
T	11	0.90	0.80	3.05	3.35	5.11	6.95
P	4**	1.00	1.10	4.26	4.50	5.71	6.36
T	5	1.30	1.35	3.58	3.75	4.60	4.76
T	5	1.35	1.35	3.45	3.61	4.41	4.52
T	5	1.40	1.35	3.33	3.48	4.22	4.30
T	5	1.45	1.35	3.21	3.35	4.05	4.07
T	5	1.50	1.35	3.09	3.22	3.87	3.86

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** This case based on $V_{L2} = 0.18$; all others based on $V_{L2} = 0.22$.

Table D16

Beta Comparison for Fasteners and Connections
Group III, Identification No. 25 (Table B2), $f_m/F_a = 2.62$, $V_R = 0.18$

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	1.79	1.90	2.46	2.86
P, T	9, 10	1.00	1.00	3.09	3.27	4.25	4.94
T	10	0.95	1.00	3.25	3.45	4.48	5.21
T	10	0.90	1.00	3.42	3.63	4.72	5.49
T	10	0.85	1.00	3.61	3.83	4.97	5.78
T	10	0.80	1.00	3.80	4.03	5.24	6.09
P, T	7	1.00	0.67	1.80	2.06	3.46	4.88
P, T	8, 11	1.00	0.80	2.37	2.59	3.79	4.90
T	11	0.85	0.80	2.89	3.14	4.50	5.74
T	11	0.80	0.80	3.09	3.35	4.77	6.05
T	11	0.75	0.80	3.29	3.57	5.06	6.38
T	11	0.70	0.80	3.51	3.80	5.36	6.74
T	11	0.65	0.80	3.75	4.06	5.69	7.12
P	4**	1.00	1.10	3.76	3.92	4.68	4.97
T	5	1.10	1.35	3.74	3.89	4.54	4.52
T	5	1.05	1.35	3.89	4.05	4.75	4.76
T	5	1.00	1.35	4.05	4.21	4.96	5.01
T	5	0.95	1.35	4.21	4.39	5.19	5.28
T	5	0.90	1.35	4.39	4.57	5.43	5.55

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** This case based on $V_{L2} = 0.18$; all others based on $V_{L2} = 0.22$.

Table D17

Beta Comparison for Fasteners and Connections
Group III, Identification No. 30 (Table B2), f_m/F_a = 3.26, V_R = 0.18

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	2.49	2.64	3.43	3.98
T	10	1.25	1.00	3.07	3.26	4.23	4.92
T	10	1.20	1.00	3.20	3.40	4.41	5.13
T	10	1.15	1.00	3.34	3.54	4.60	5.35
T	10	1.10	1.00	3.48	3.69	4.80	5.58
T	10	1.05	1.00	3.63	3.85	5.00	5.82
P, T	9, 10	1.00	1.00	3.79	4.02	5.22	6.07
P	7	1.00	0.67	2.50	2.80	4.42	6.00
T	11	1.05	0.80	2.92	3.17	4.54	5.78
P, T	8, 11	1.00	0.80	3.07	3.33	4.75	6.03
T	11	0.95	0.80	3.24	3.51	4.98	6.29
T	11	0.90	0.80	3.41	3.69	5.22	6.57
T	11	0.85	0.80	3.59	3.89	5.47	6.86
P	4**	1.00	1.10	4.53	4.74	5.69	6.09
T	5	1.30	1.35	3.91	4.06	4.77	4.79
T	5	1.35	1.35	3.79	3.94	4.60	4.59
T	5	1.25	1.35	4.03	4.20	4.94	4.99
T	5	1.20	1.35	4.16	4.34	5.12	5.20
T	5	1.15	1.35	4.30	4.48	5.31	5.42

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** This case based on V_{L2} = 0.18; all others based on V_{L2} = 0.22.

Table D18

Beta Comparison for Fasteners and Connections
Group IV, Identification No. 31 (Table B2), f_m/F_a = 2.62, V_R = 0.27

Criteria Type*	Load Case Number (Table C1)	γ	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	1.50	1.56	1.84	1.99
P, T	9, 10	1.00	1.00	2.59	2.70	3.18	3.44
P	7	1.00	0.67	1.52	1.70	2.58	3.39
T	10	0.60	1.00	3.97	4.13	4.86	5.26
T	10	0.65	1.00	3.75	3.91	4.60	4.98
T	10	0.55	1.00	4.20	4.38	5.15	5.57
T	10	0.50	1.00	4.46	4.65	5.47	5.91
T	10	0.45	1.00	4.74	4.94	5.81	6.29
T	11	0.55	0.80	3.60	3.82	4.80	5.55
T	11	0.50	0.80	3.86	4.08	5.12	5.89
T	11	0.45	0.80	4.14	4.38	5.47	6.26
T	11	0.40	0.80	4.46	4.71	5.85	6.68
T	11	0.35	0.80	4.82	5.08	6.30	7.16
P	4**	1.00	1.10	3.06	3.14	3.42	3.45
T	5	0.70	1.35	4.36	4.48	4.89	4.26
T	5	0.65	1.35	4.56	4.69	5.13	5.02
T	5	0.60	1.35	4.78	4.91	5.40	5.31
T	5	0.55	1.35	5.01	5.15	5.68	5.62
T	5	0.50	1.35	5.27	5.42	6.00	5.96
P, T	8, 11	1.00	0.80	1.99	2.14	2.83	3.41

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** This case based on V_{L2} = 0.18; all others based on V_{L2} = 0.22.

Table D19

Beta Comparison for Fasteners and Connections
Group IV, Identification No. 36 (Table B2), $f_m/F_a = 3.26$, $V_R = 0.27$

Criteria Type*	Load Case Number (Table C1)	Y	L_n/L_m	Beta Value for D_m/L_m = 0.0	0.1	1.0	25.0
P, T	6, 10	1.50	1.00	2.09	2.18	2.56	2.77
P, T	9, 10	1.00	1.00	3.18	3.31	3.90	4.22
T	10	0.80	1.00	3.78	3.94	4.64	5.02
T	10	0.75	1.00	3.96	4.12	4.85	5.25
T	10	0.70	1.00	4.14	4.32	5.08	5.49
T	10	0.65	1.00	4.34	4.52	5.32	5.76
T	10	0.60	1.00	4.56	4.75	5.59	6.04
P, T	8, 11	1.00	0.80	2.58	2.75	3.55	4.19
T	11	0.65	0.80	3.74	3.96	4.97	5.73
T	11	0.60	0.80	3.96	4.19	5.24	6.02
T	11	0.55	0.80	4.19	4.43	5.52	6.33
T	11	0.50	0.80	4.45	4.70	5.84	6.67
P	4**	1.00	1.10	3.69	3.79	4.16	4.23
P	7	1.00	0.67	2.10	2.31	3.30	4.17
T	5	0.85	1.35	4.43	4.55	4.97	4.85
T	5	0.80	1.35	4.59	4.72	5.17	5.06
T	5	0.75	1.35	4.77	4.90	5.38	5.29
T	5	0.70	1.35	4.95	5.09	5.61	5.54
T	5	0.65	1.35	5.15	5.30	5.85	5.80

* P = permanent (AASHTO) bridges; T = temporary (military) bridges.
** This case based on $V_{L2} = 0.18$; all others based on $V_{L2} = 0.22$.

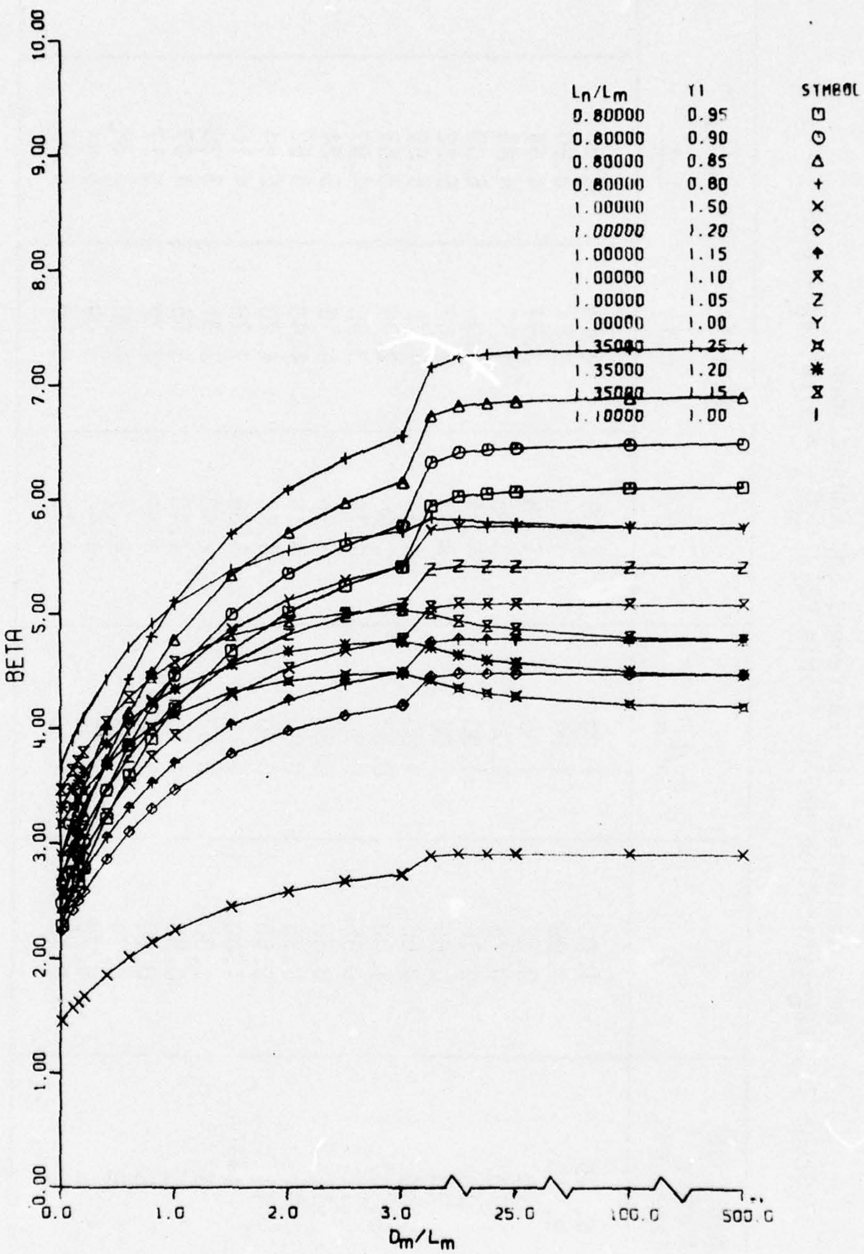


Figure D7. Beta versus D_m/L_m for fasteners, f_m/F_a = 2.26, Group I, for selected L_n/L_m and Y combinations.

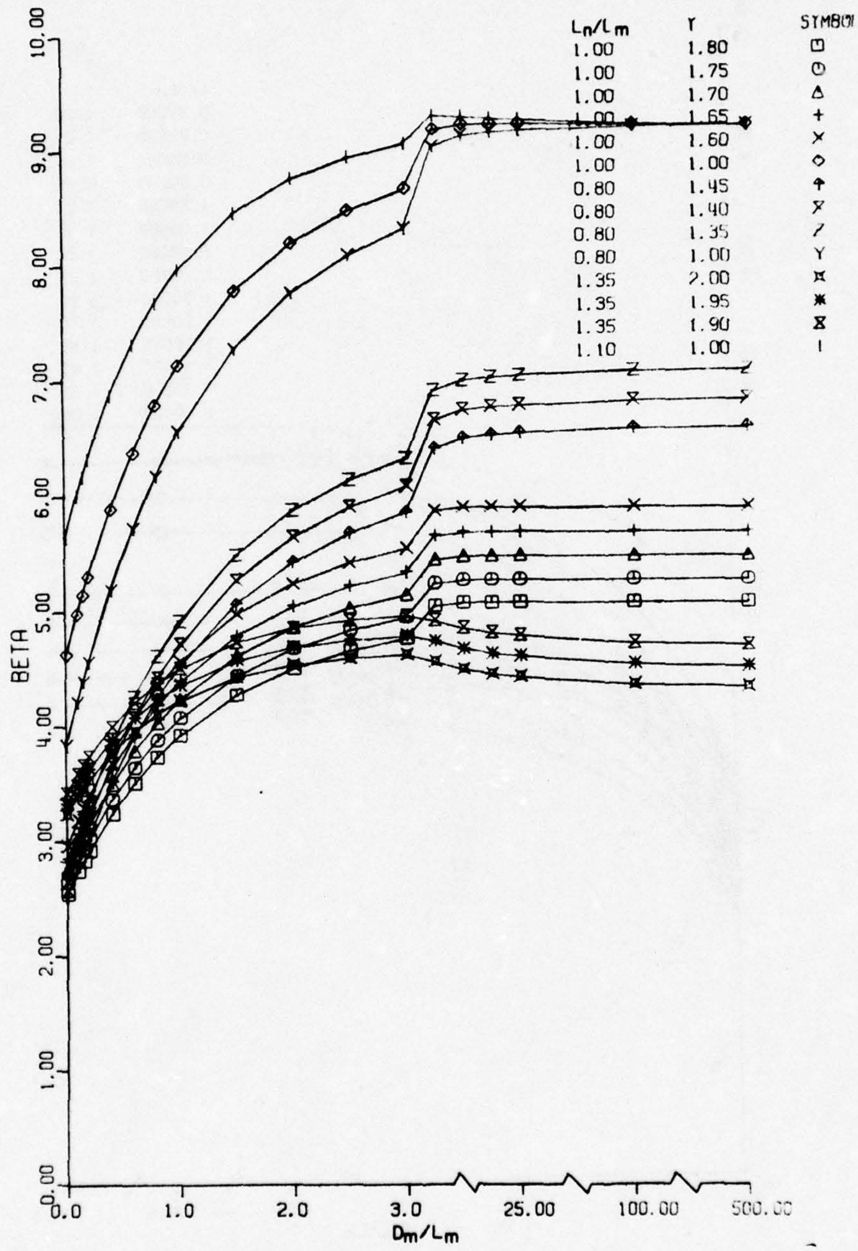


Figure D8. Beta versus D_m/L_m for fasteners, f_m/F_a = 3.68, Group I, for selected L_n/L_m and Y combinations.

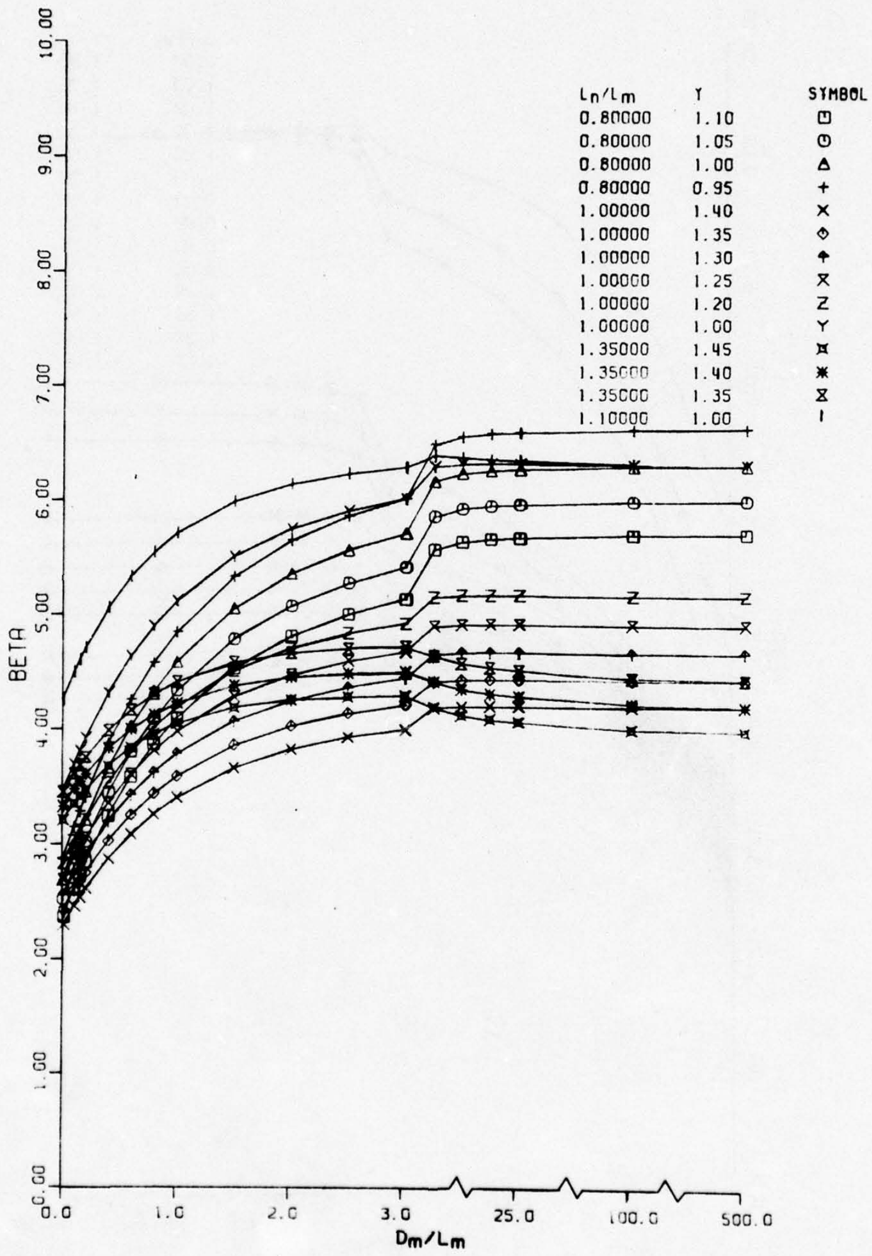


Figure D9. Beta versus D_m/L_m for fasteners, f_m/F_a = 2.73, Group II, for selected L_n/L_m and Y combinations.

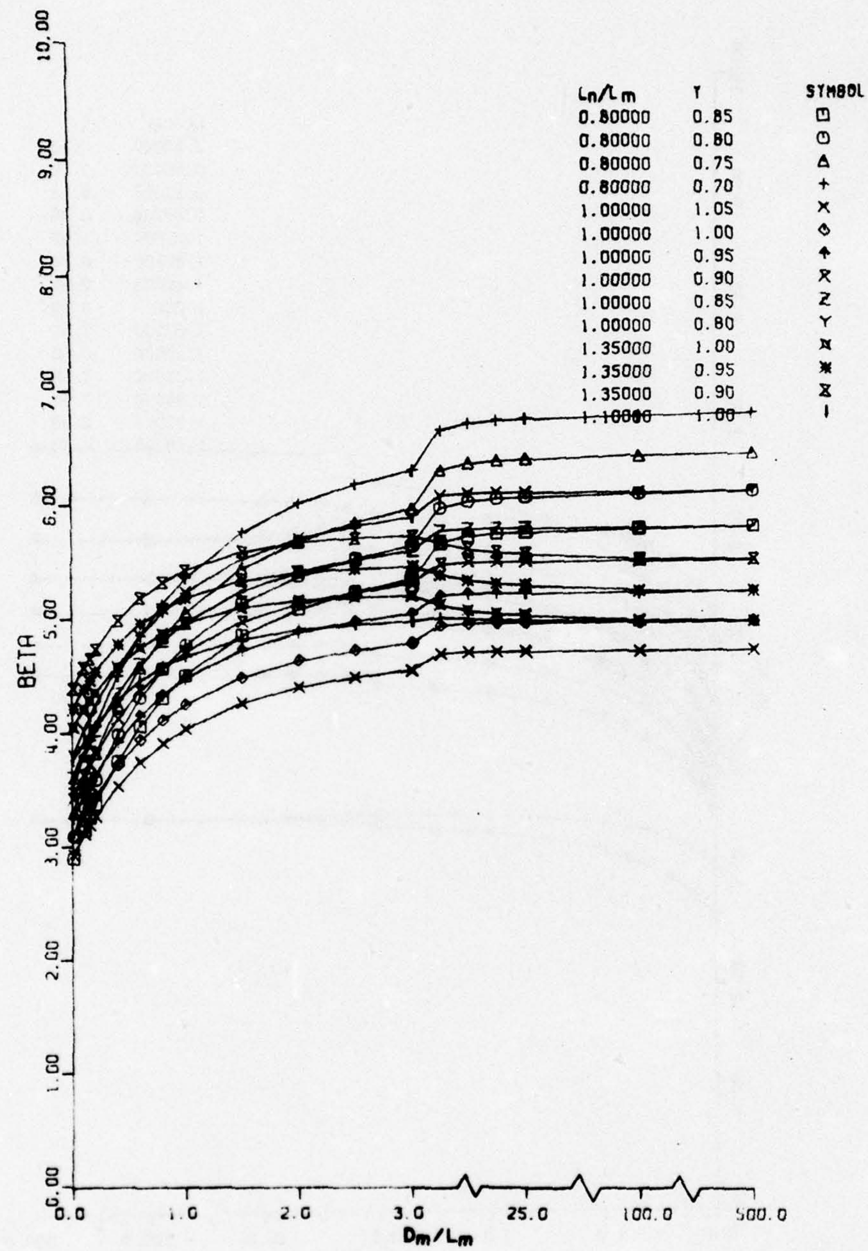


Figure D10. Beta versus D_m/L_m fasteners, f_m/F_a = 2.62, Group III, for selected L_n/L_m and Y combinations.

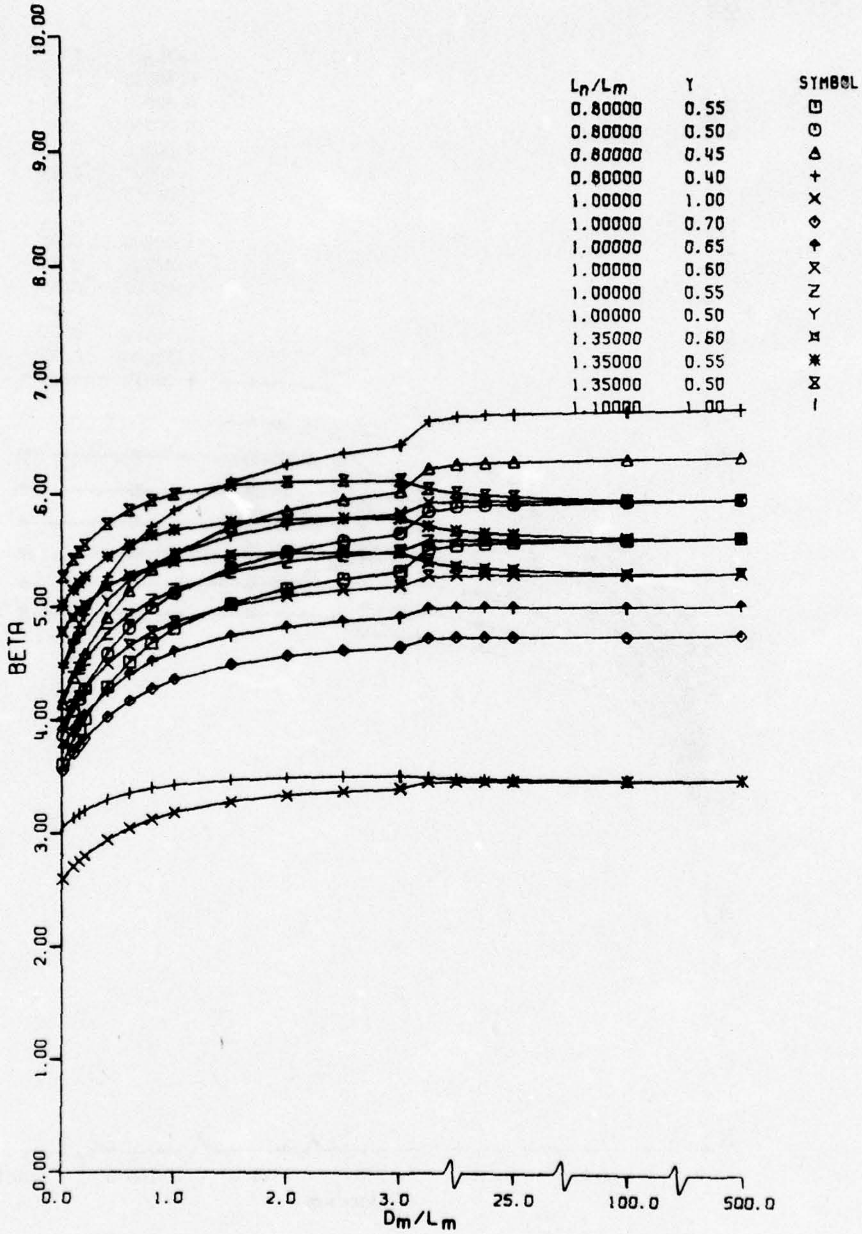


Figure D11. Beta versus D_m/L_m for fasteners, f_m/F_a = 2.62, Group IV, for selected L_n/L_m and Y combinations.

Table D20

Recommended Y Values for Fasteners and Connections

Identification No. (Table B2 and Table G13)	Group	f_m/F_a*	L_n/L_m = .8: β at D_m/L_m = .1	L_n/L_m = .8: Recommended Y	L_n/L_m = 1.0: β at D_m/L_m = .1	L_n/L_m = 1.0: Recommended Y
Fasteners						
1	I	2.26	2.75	0.90	2.75	1.10
2	I	2.38	2.74	0.95	2.78	1.15
3	I	2.58	2.85	1.00	2.77	1.25
4	I	2.60	2.88	1.00	2.80	1.25
5	I	2.65	2.77	1.05	2.72	1.30
6	I	2.79	2.79	1.10	2.77	1.35
7,8	I	2.81	2.82	1.10	2.80	1.35
9	I	2.87	2.73	1.15	2.74	1.40
10	I	2.90	2.77	1.15	2.78	1.40
11	I	2.91	2.78	1.15	2.79	1.40
12,13	I	3.06	2.81	1.20	2.85	1.45
14	I	3.09	2.85	1.20	2.89	1.45
15	I	3.27	2.91	1.25	2.85	1.55
16	I	3.30	2.94	1.25	2.89	1.55
17	I	3.45	2.96	1.30	2.93	1.60
18	I	3.47	2.98	1.30	2.96	1.60
19,20	I	3.55	2.93	1.35	2.93	1.65
21	I	3.57	2.94	1.35	2.94	1.65
22	I	3.68	2.92	1.40	2.95	1.70
Connected material**						
23	II	2.73	2.78	1.05	2.73	1.30
24	II	2.95	2.89	1.10	2.88	1.35
Shop welds						
25	III	2.62	3.57	0.75	3.63	0.90
26	III	2.75	3.51	0.80	3.61	0.95
27	III	2.87	3.66	0.80	3.58	1.00
28	III	3.00	3.60	0.85	3.57	1.05
29	III	3.12	3.54	0.90	3.54	1.10
30	III	3.26	3.51	0.95	3.54	1.15
Field welds						
31	IV	2.62	4.38	0.45	4.38	0.55
32	IV	2.75	4.51	0.45	4.27	0.60
33	IV	2.87	4.34	0.50	4.39	0.60
34	IV	3.00	4.46	0.50	4.29	0.65
35	IV	3.12	4.31	0.55	4.40	0.65
36	IV	3.26	4.43	0.55	4.32	0.70

* f_m/F_a is the ratio of the mean ultimate fastener stress to the permanent specification allowable stress.
** Tension members.

Table D21

Summary of Beta Values Used for Fasteners and Connections

Fasteners or Connections Used With	Group (Table B2)	Recommended Beta for Fasteners and Connections for Temporary Bridges at D_m/L_m = 0.10	Average AASHTO Beta Value for Fasteners and Connections Used as Comparison, at D_m/L_m = 0.10
Stringer end connections and compression and tension members (L_n/L_m = 0.80 or 1.00 for temporary bridges)	I, II	2.75 - 3.00	4.00
	III	3.50	4.00
	IV	4.25	4.00
Flexural splices (L_n/L_m = 1.35 for temporary bridges)	I, II	3.50	4.75
	III	4.25	4.75
	IV	5.00	4.75

The Y values for L_n/L_m = 1.0 are recommended (Table D20) for fasteners and connections on stringer end connections, provided the procedure recommended in Appendix I for determining the shear force is used. The Y values for L_n/L_m values of 1.0 are also recommended for fasteners and connected material in connections of tension and compression members, provided the larger design force resulting from the normal or caution military crossings is used.

The recommended Y values for fasteners and materials used in flexural splices were found by choosing Y values such that their corresponding beta values agreed with the beta values in Table D21 for an L_n/L_m value of 1.35. The ratio of the recommended Y for an L_n/L_m of 1.35 to Y for an L_n/L_m of 1.0 (Table D20) ranged from 1.00 to 1.13. To use one set of Y values (and allowable stresses) for both L_n/L_m values, it is recommended that the Y values for an L_n/L_m value of 1.35 be reduced to match the Y values for an L_n/L_m value of 1.0, provided the moment and resulting forces are determined as recommended for flexure (plastic moment and lateral torsional buckling).

The recommended allowable stresses corresponding to L_n/L_m = 1.0 are given in Tables 4 and 5, Chapter 3, of Volume I.

If design procedures* to determine forces in fasteners and connection materials are used which result in L_n/L_m values of 0.80 to 1.0, then the Y values for L_n/L_m values of 0.80 are recommended for all fasteners and connections. The allowable stresses corresponding to L_n/L_m values of 0.80 are about 80 percent of the values of the allowable stresses based on L_n/L_m values of 1.0. (The allowable stresses for L_n/L_m = 1.0 are given in Chapter 3 of Volume I--see Tables 4 and 5 and the Tension Members and Bearing on Projected Area of Bolts and Rivets sections.)

The following justification for the recommended Y values is provided. It can be argued that fasteners and connections should have at least as high and probably higher reliability levels as members because

* An example of such a design procedure would be the use of a force resulting from a normal crossing when the caution crossing actually produces a larger force.

of the uncertainty associated with field fabrication and/or installation, especially in temporary T/O facilities. Traditionally, for permanent criteria, fasteners and connections have had higher factors of safety than members.

For fasteners and connection materials used with stringer end connections as well as compression and tension member connections, the normal AASHTO load case of $Y = 1.0$ and $L_n/L_m = 1.0$ (load case 9, Table C1) was used as a basis of comparison. Tables D12 through D19 show that the beta values corresponding to a D_m/L_m value of 0.1 (a representative lower value) for case 9 range, for the most part, from 3 to 5. An average beta value of 4 appears reasonable for the normal AASHTO load case. On this basis, a beta range of 2.75 to 3.00 at D_m/L_m of 0.1 was chosen for fasteners and connection materials for Groups I (bolts and shop rivets) and II (connected material-tension members) in Table B2 for temporary bridges. This range appears reasonable when compared to beta values for recommended Y values for temporary bridges for shear (beta = 2.02 at $D_m/L_m = 0.1$) and compression members (beta ranges from 2.39 to 2.67 at $D_m/L_m = 0.1$). Beta values of 3.5 and 4.25 were chosen for Group III (shop welds) and Group IV (field welds), respectively, for temporary bridges. The higher beta values reflect the greater susceptibility of welds (as opposed to bolting) to fabrication and erection error in the T/O.

For fasteners and material used in flexural splices, the normal crossing AASHTO load case of $Y = 1.0$ and $L_n/L_m = 1.10$ (case 4) was chosen as a basis of comparison. Based on Tables D12 through D19, a representative beta value of 4.75 at $D_m/L_m = 0.1$ for case 4 was chosen. As a result, a beta value of 3.5 at $D_m/L_m = 0.1$ was chosen as a reasonable guide for fasteners and material used in flexural splices for temporary bridges ($L_n/L_m = 1.35$) for Group I (bolts and rivets) and II (connected material-tension members). Moreover, a beta value of 3.5 appears reasonable when compared to the beta values for the recommended Y values for temporary bridges for flexure (plastic moment) and lateral torsional buckling, which are 2.44 and 2.71 respectively at $D_m/L_m = 0.1$. Beta values of 4.25 and 5.0 were chosen for Group III (shop welds) and Group IV (field welds), respectively, for temporary bridges. Again, the higher beta values reflect the greater susceptibility of welds (as opposed to bolting) to fabrication and erection error in the T/O.

Table D21 summarizes the beta values chosen for fasteners and connection materials. The beta values recommended in Table D21 for fasteners and connection materials for temporary bridges were used to determine the recommended Y values shown in Table D20. These allowable stress factors (Y values) were then used to determine the recommended allowable stresses shown* in Tables 4 and 5 of Chapter 3 of Volume I.

The recommended allowable stresses for the combined tension and shear case for bolts and shop rivets are given in Table 6, Chapter 3, Volume I. The recommended stresses may be used provided that the recommendations given in the <u>Loads, Moments, and Forces</u> section in Chapter 3 of Volume I are used.

The following justification for the allowable stresses for the combined tension and shear is provided. For the combined tension and shear stress cases for bolts and rivets, approximate mean stress relationships for bolts and rivets are given in Table G14, Appendix G and can be written as

for bolts: $f_{tsm} = G = 1.9F_s \le f_{tm}; \; F_s \le f_{sm}$ [Eq D7a]

for rivets: $f_{tsm} = G - 1.6F_s \le f_{tm}; \; F_s \le f_{sm}$ [Eq D7b]

where,** f_{tsm} = mean ultimate tensile stress in the presence of the shear stress, F_s

f_{tm} = mean ultimate tensile stress in the absence of shear stress

* The recommended allowable stresses for connected material (tension members) and bearing are given in the <u>Tension Members</u> and <u>Bearing on Projected Area of Bolts and Rivets</u> sections in Chapter 3, Volume I.

** Note that in Appendix G, $f_{tsm} = \sigma_t$ and $F_s = \sigma_s$.

f_{sm} = mean ultimate shear stress in the absence of tensile stress, f_t

G = intercept of straight line approximation on tension axis (Figure D12)

F_s = shear stress on fastener.

Figure D12 shows a general representation of the mean stress equation and interaction diagram.

The mean relationship, shown as the solid line BCDE in Figure D12, is represented by Eq D7a and b. The design relationship, shown as the dotted line B'C'D'E', can be obtained by reducing f_{tm}, f_{sm}, and G, while maintaining the same slope. The design relationship can be represented by

$$F_t = G' - 1.9F_s \leq F_{at};\ F_s \leq F_{as} \qquad \text{[Eq D8]}$$

where G' = $G/(f_m/F_a')$

f_m/F_a' = representative value by which to reduce G; should be close to f_{tm}/F_{at} and f_{sm}/F_{as}

F_{at} = temporary allowable tensile stress in the absence of shear stress = $f_{tm}/(f_{tm}/F_{at})$

F_{as} = temporary allowable shear stress in the absence of tensile stress = $f_{sm}/(f_{sm}/F_{as})$

F_t = temporary allowable tensile stress in the presence of shear stress.

The values of F_{at} and F_{as} are found by reducing (dividing) f_{tm} and f_{sm} by (f_{tm}/F_{at}) and (f_{sm}/F_{as}), respectively. The value of G', however, requires use of a reduction factor on G, namely f_m/F_a. To maintain consistency, the value of f_m/F_a should be close to both f_{tm}/F_{at} and f_{sm}/F_{as}. Table D22 gives the information used to determine f_m/F_a', including the values of f_{tm}/F_{at} and f_{sm}/F_{as} for bolts and rivets. As the table shows, f_{tm}/F_{at} and f_{sm}/F_{as} do not differ greatly. In all cases the larger (conservative) value of f_{tm}/F_{at} and f_{sm}/F_{as} was chosen for use as f_m/F_a'.

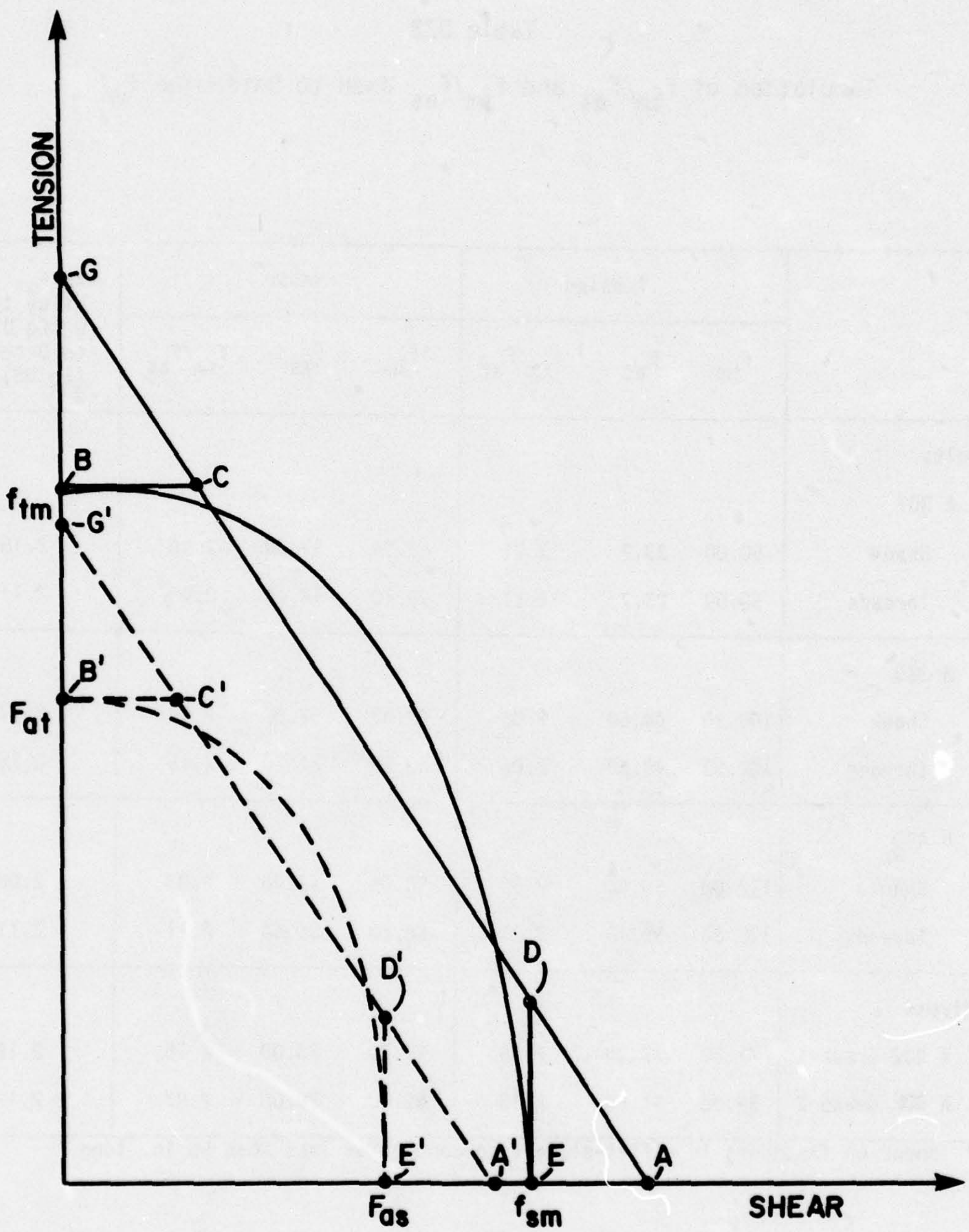


Figure D12. Schematic representation of interaction diagram; mean and design relationships for fasteners subjected to tension and shear.

Table D22

Tabulation of f_{tm}/F_{at} and f_{sm}/F_{as} Used to Determine f_m/F_a'

	Tension			Shear*			f_m/F_a', Value to Divide G (Eq D7) by to Determine G' (Eq D8)
	f_{tm}	F_{at}	f_{tm}/F_{at}	f_{sm}	F_{as}	f_{sm}/F_{as}	
Bolts							
A 307							
Shank	50.00	23.7	2.11	38.00	17.60	2.16	2.16
Threads	50.00	23.7	2.11	29.10	14.00	2.08	2.11
A 325							
Shank	100.30	48.60	2.06	69.50	32.00	2.17	2.17
Threads	100.30	48.60	2.06	53.20	24.70	2.15	2.15
A 490							
Shank	122.00	59.40	2.05	90.00	43.20	2.08	2.08
Threads	122.00	59.40	2.05	68.80	32.60	2.11	2.11
Rivets							
A 502 Grade 1	71.00	33.00	2.15	49.70	23.00	2.16	2.16
A 502 Grade 2	89.00	41.80	2.13	58.00	28.00	2.07	2.13

* Shear on fasteners in a flat-plate type connection less than 50 in. long.

The recommended allowable stress relationships, based on Eq D8, Table D22, and the mean stress relationships (Eq D7 and Table G14) are given in Table 6, Chapter 3, Volume I.

APPENDIX E:

DESCRIPTION OF FRACTURE-CONTROL PLAN AND DEVELOPMENT OF MATERIAL SPECIFICATIONS, DESIGN CRITERIA, AND DESIGN PROCEDURES TO PREVENT BRITTLE FRACTURE IN T/O BRIDGES

Background

This appendix summarizes the technical development of a fracture-control plan designed to prevent brittle fracture or fatigue growth leading to brittle fracture in T/O structures. Brittle fracture is a type of catastrophic failure that usually occurs without prior plastic deformation and at extremely high speeds of crack propagation (as high as 5000 ft/sec). The fracture is usually characterized by a flat fracture surface (cleavage) with little or no plastic deformation; it generally occurs at average stress levels below those of general yielding. Although brittle fracture failures are not as common as fatigue, yielding, or buckling failures, when they do occur, they are usually more costly in terms of human life and/or property damage.

Accordingly, they can lead to catastrophic failures of structures with little or no prior warning and a consequent loss of load-carrying ability. In contrast, failures by general yielding in tension are preceded by considerable deformation and the members can still carry loads. Thus, since the consequences of brittle fracture are much greater than those of general yielding, specific design precautions should be taken in designing T/O structures to avoid this type of structural failure.

While the number of brittle fractures in structures such as bridges or buildings is small, the overall safety and reliability of structures can be improved significantly by rather small changes in material specifications and design. Because of the consequences of this catastrophic mode of failure, fracture control is an important design consideration for T/O bridges.

A fracture control plan is a detailed procedure that:

1. Identifies the known factors that may contribute to the

brittle fracture of a structural detail or to the failure of an entire structure

2. Establishes the contribution of each of these factors and the synergistic contribution of the factors to the fracture or failure process

3. Determines the relative efficiency and trade-offs of various methods to minimize the probability of fracture or failure

4. Recommends specific design considerations (including material selection, design-stress levels, and fabrication) to insure the safety and reliability of a structure.

Numerous factors can contribute to brittle fractures, including material toughness, temperature, flaw size, fabrication and inspection, tensile stresses, loading rate and cycles, constraint, residual stresses, redundancy of load path, and fatigue-crack-growth behavior. Because some or even all of these factors can contribute to fractures of structural members, merely specifying that either a material with a particular notch toughness be used, or that all welds be inspected, or that the design stress be low will not insure that fractures will not occur. The relative importance of each of these factors and their synergistic effect must be defined.

Of these factors, fracture mechanics has shown that three *primary* factors control the susceptibility of a structure to brittle fracture: (1) material toughness, (2) flaw size, and (3) stress level.

1. Material Toughness. Material toughness is the resistance to crack propagation in the presence of a notch. For linear-elastic behavior, material toughness is measured in terms of a static critical stress-intensity factor under conditions of plane stress (K_c), plane strain (K_{Ic}), or dynamic loading (K_{Id}). For elastic-plastic fracture behavior, the material toughness may be measured in terms of ductility-related parameters as in the J-integral, resistance curve, crack-opening displacement, and equivalent energy approaches.

The J-integral (J_{Ic}) is a path-independent integral which is an average measure of the elastic-plastic stress/strain field ahead of a crack. For elastic conditions, J_{Ic} equals $K_{Ic}^2/E(1-\nu^2)$. A test method for this approach is currently under development.

The resistance-curve (R-Curve) analysis is a procedure used to characterize materials' resistance to fracture during incremental slow-stable crack extension, K_R. At instability, K_R is equal to K_c, the plane stress fracture-toughness which depends on specimen thickness, as well as temperature and loading rate.

The crack-opening displacement (COD) technique evaluates toughness in terms of the prefracture deformation at the tip of a sharp crack.

The equivalent energy approach is based on using test results to predict failure, primarily of thick-walled pressure vessels.

2. Flaw Size. Brittle fractures initiate from flaws or discontinuities of various kinds, such as porosity, inclusions, lack of fusion, toe cracks, and mismatch. These discontinuities can vary from extremely small cracks within a weld arc strike to much larger weld or fatigue cracks, or cracks growing from rivet or bolt holes. Although good fabrication practice and inspection can minimize the original size and number of flaws, discontinuities are present in all complex welded structures, even after all inspections and weld repairs are finished. Cracks or discontinuities can also be present in bolted structures, although the initial flaw sizes may be smaller or less severe than in welded structures. However, even though only "small" flaws may be present initially, fatigue stressing can cause them to enlarge, possibly to a critical size.

3. Stress Level. Tensile stresses (nominal, residual, or both) are necessary for brittle fractures to occur. Stress is elevated in the vicinity of stress concentrations or discontinuities.

Controlling these three factors can reduce the susceptibility of a structure to brittle fracture. All other factors such as temperature, loading rate, and residual stresses, merely affect these *primary* factors.

Based on these facts engineers have been reducing susceptibility of structures to brittle fractures for many years by applying these concepts to their structures *qualitatively*. That is, good design (use of appropriate stress levels and minimizing of discontinuities) and fabrication practices (using proper welding control to decrease flaw size), as well as use of materials with good notch-toughness levels (e.g., as measured by a Charpy V-notch (CVN) impact test) will and *have* minimized the probability of brittle fractures in structures. However, the engineer has not had specific design guidelines to evaluate the relative performance and economic trade-offs between design, fabrication, and materials quantitatively.

Recent nonmilitary bridge failures, a growing concern about the possibility of future failures of nonmilitary bridges, and the realization that the structural engineer needs guidelines have led to AASHTO's adoption of material toughness requirements for bridge steels.[11] These requirements specify that the structural steels have particular values of Charpy V-notch impact energy depending on material strength level, service temperature, and plate thickness. However, these requirements apply to nonmilitary bridges designed for AASHTO loadings and are not necessarily directly applicable to bridges in the T/O, where service conditions and material availability are different. The fracture control plan presented in this appendix is designed to develop quantitative design guidelines for T/O bridges.

The fundamental concept of linear-elastic fracture mechanics is that the stress field ahead of a sharp crack can be characterized in terms of a single parameter--K_I, the stress intensity factor for flat crack propagation (usually referred to as opening mode), expressed in ksi $\sqrt{in.}$ The term K_I is related to both the stress level (σ) and the flaw size (a). When the particular combination of σ and a leads to a critical

[11] "Material Toughness Requirements," *Standard Specifications for Highway Bridges* (American Association of State Highway and Transportation Officials, 1973).

of K_I, called K_{Ic} or K_c, unstable crack growth occurs. Figure E1 presents the equations that describe the elastic stress field in the vicinity of a crack tip in a body subjected to tensile stresses normal to the plane of a simple crack. These stress field equations define the distribution of the elastic stress field in the vicinity of the crack tip and can be used to establish the relation between K_I, σ, and a for different structural configurations (Figure E2). Other crack geometries have been analyzed for different structural configurations and are published in the literature.

If the critical value of K_I at failure (K_c, K_{Ic}, or K_{Id}) can be determined for a given metal of a particular thickness at a specific temperature and loading rate, the designer can determine theoretically the flaw size that can be tolerated in structural members for a given design stress level. Conversely, the designer can determine the design stress level that can safely be used for a flaw size that may be present in a structure.

Figure E3 shows schematically this general (conceptual) relationship between material toughness (K_{Ic} or K_c), nominal stress (σ), and flaw size (a). If a particular combination of stress and flaw size in a structure (K_I) reaches the K_{Ic} or K_c level, fracture can occur. Thus, there are many combinations of stress and flaw size (e.g., σ_f and a_f) that may cause fracture in a structure fabricated from a steel having a particular K_{Ic} or K_c value at a particular service temperature, loading rate, and plate thickness. Conversely, many combinations of stress and flaw size (e.g., σ and a) will not cause failure of a particular steel, i.e., below the K_{Ic} or K_c line.

A useful analogy for the designer is the relationship between applied load (P), nominal tensile stress (σ), and yield or limit stress (σ_y) in an unflawed structural member, and between applied load (P), stress intensity (K_I), and critical stress intensity for fracture (K_c, K_{Ic}, or K_{Id}) in a structural member with a flaw. In an unflawed structural member, as the load is increased, the nominal stress

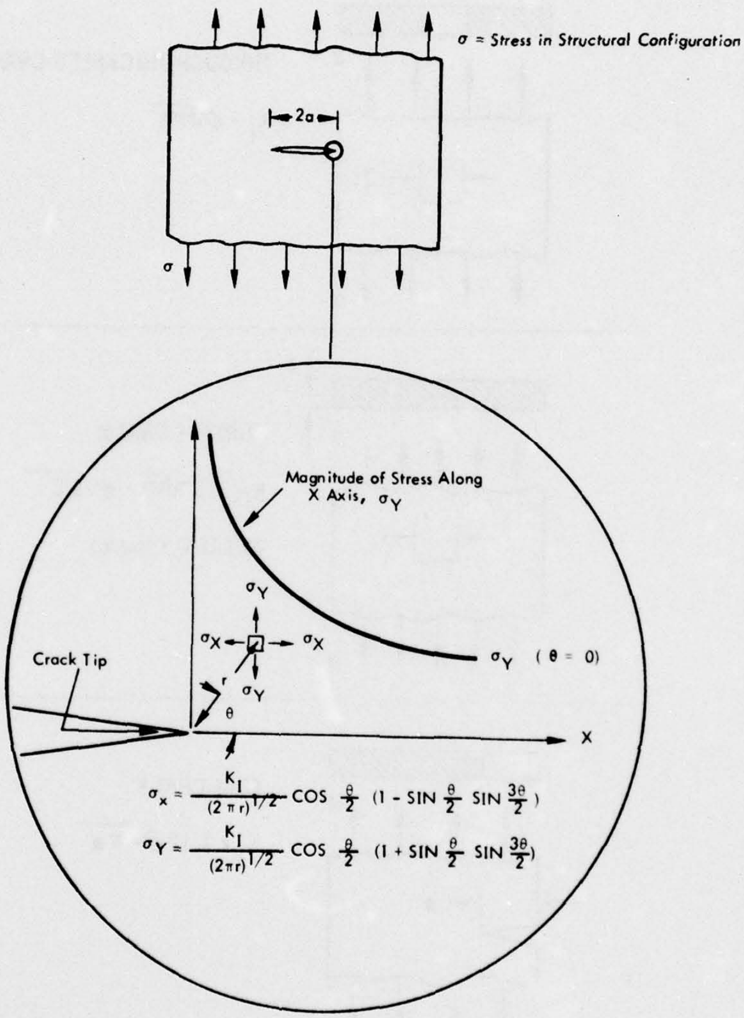


Figure E1. Elastic stress field distribution ahead of a crack.

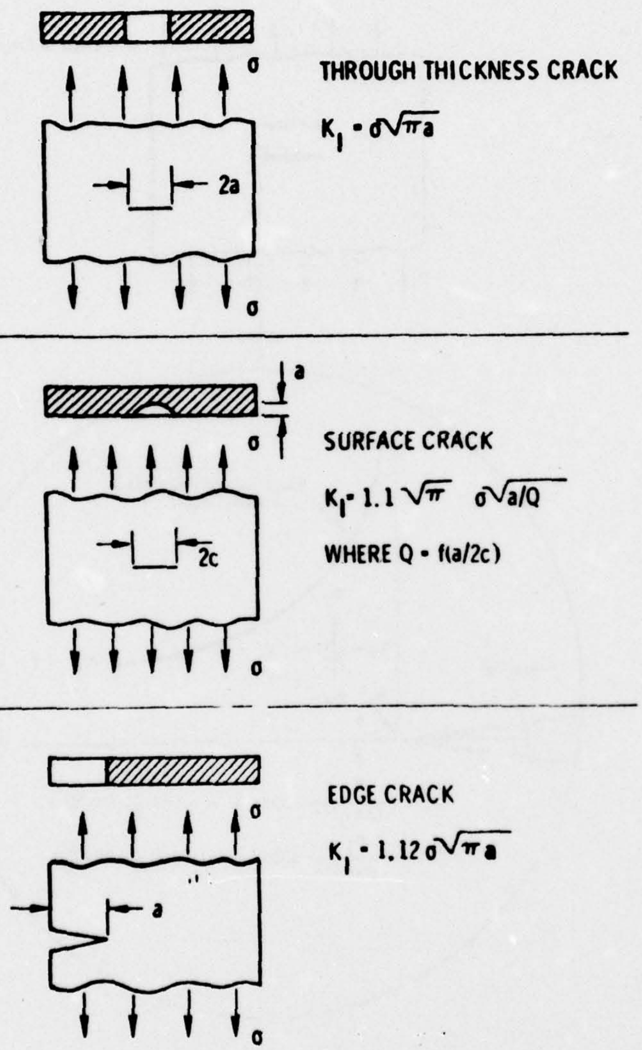


Figure E2. K_I values for various crack geometries.

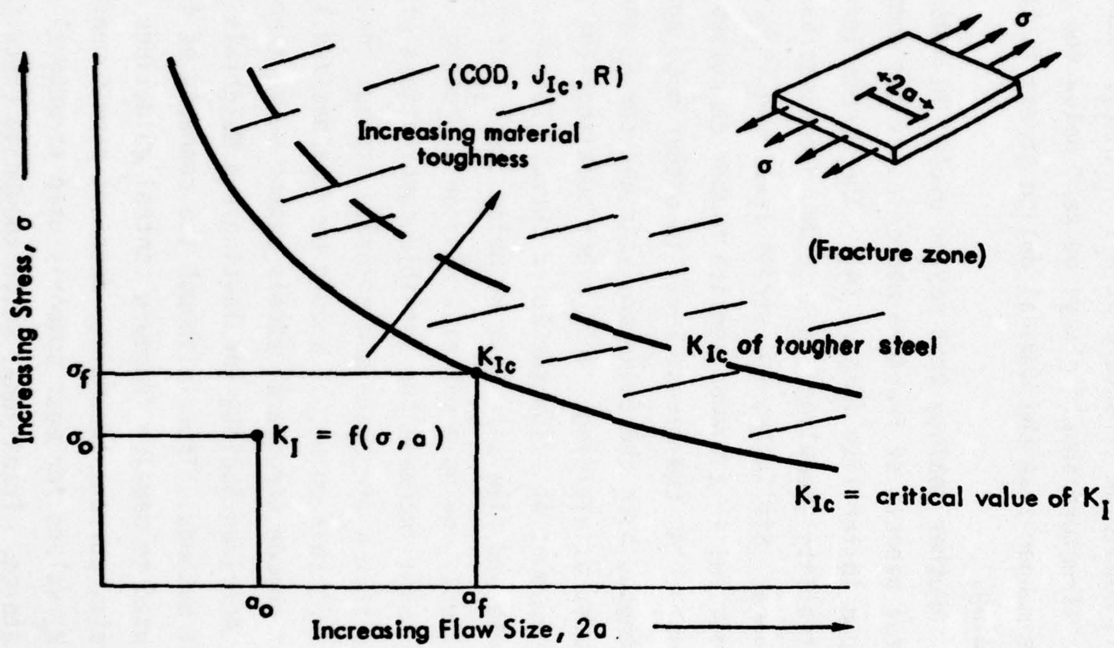


Figure E3. Schematic of relationship between stress, flaw size, and material toughness.

increases until a limit loading (yielding) occurs. As the load is increased in a structural member with a flaw (or as the size of the flaw grows by fatigue or stress corrosion), K_I increases until a limit condition (fracture at K_c, K_{Ic}, K_{Id}) occurs. Thus, the K_I level in a structure should always be kept below the critical value in the same manner that the nominal design stress is kept below the limit loading.

Another analogy that may be useful in understanding the fundamental aspects of fracture mechanics is the comparison with the Euler column instability (Figure E4). The stress level required to cause instability (buckling) in a column decreases as the L/r ratio increases. Similarly, the stress level required to cause instability (fracture) in a flawed tension member decreases as the flaw size increases. As the stress level in either case approaches the yield strength, both the Euler analysis and the K_c analysis are invalidated because of yielding. To prevent buckling, the actual stress and L/r values must be below the Euler curve. To prevent fracture, the actual stress and flaw size must be below the K_{Ic} or K_c level (Figure E4). Obviously, using a material with an increased level of notch toughness will increase the possible combinations of design stress and flaw size that a structure can tolerate without fracturing.

At this point, it should be reemphasized that the K_c levels for most common structural steels cannot be measured directly using existing American Society for Testing and Materials (ASTM) standardized test methods. Thus, although the concepts of fracture mechanics can be used to develop fracture control guidelines and desirable toughness levels, current technology does not permit measurement of actual K_{Ic} or K_c values for most commonly used structural metals at service temperatures. Traditional notch toughness tests (e.g., CVN, nil ductility transition (NDT), etc.) are therefore widely used at the present time to specify the notch toughness requirements for various structural applications. The recently developed AASHTO material toughness

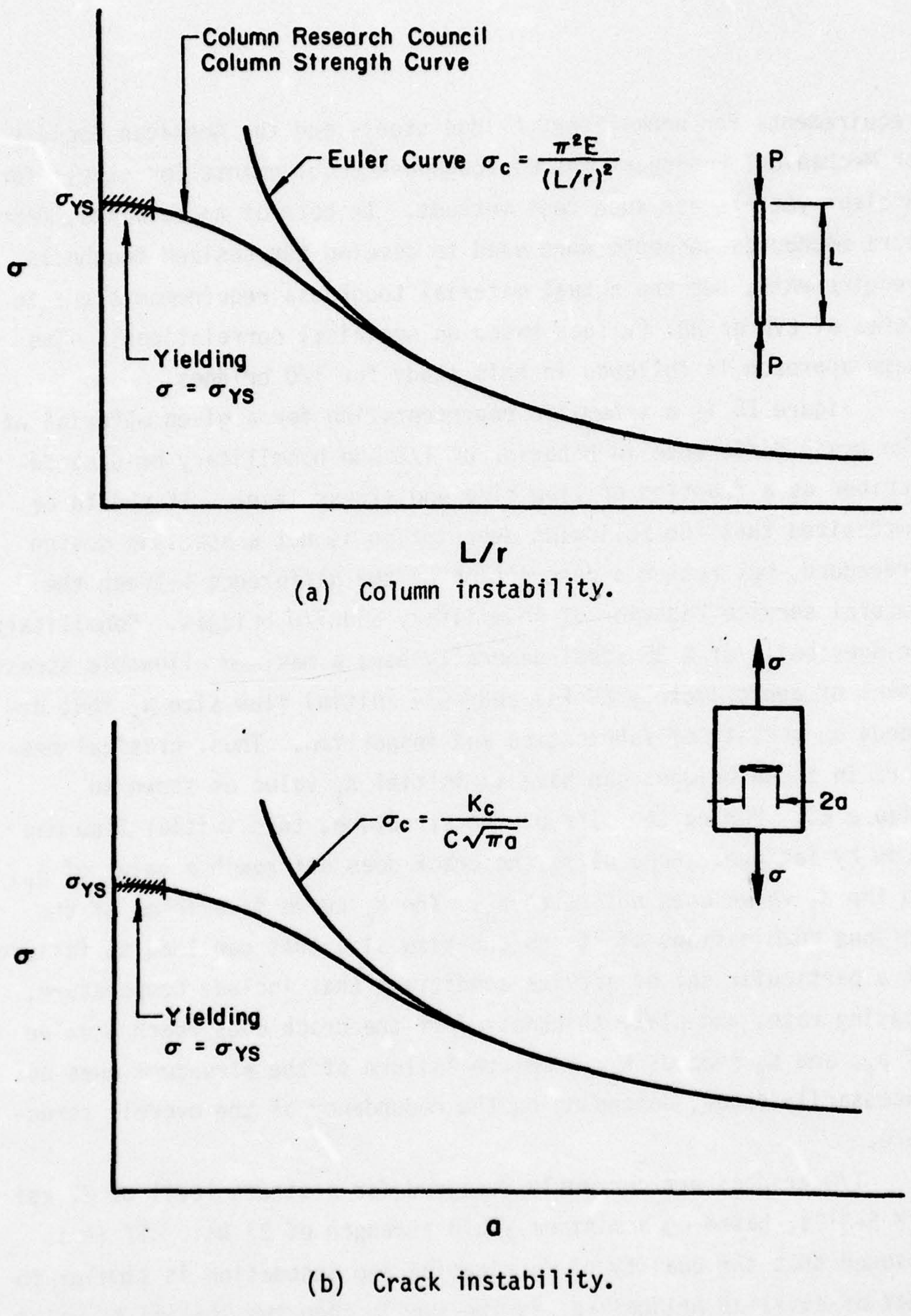


Figure E4. Column and crack instability.

requirements for nonmilitary bridge steels and the American Society of Mechanical Engineers (ASME) toughness requirements for steels for nuclear vessels use such test methods. In both of these cases, fracture mechanics concepts were used to develop the desired toughness requirements, but the actual material toughness requirements are in terms of CVN or NDT (values based on empirical correlations). The same approach is followed in this study for T/O bridges.

Figure E5 is a schematic representation for a given material of the basic difference in behavior of T/O and nonmilitary bridges described as a function of flaw size and stress level. It should be emphasized that the following description is not a specific design procedure, but rather a description of the difference between the general service behavior of nonmilitary and T/O bridges. Nonmilitary bridges built of A 36 steel generally have a maximum allowable stress level of approximately 20 ksi and some initial flaw size a_o that depends on quality of fabrication and inspection. Thus, critical members in these bridges can have an initial K_I value as shown in Figure E5. During the life of the structure, this initial flaw may grow by fatigue. Hopefully, the crack does not reach a value of a_f, so the K_I value does not reach K_c. The K_c curve is a locus of the various combinations of stress and flaw size that can lead to failure* at a particular set of service conditions that include temperature, loading rate, and plate thickness. If the crack does reach a value of a_f, and K_I reaches K_c, complete failure of the structure does not necessarily occur, depending on the redundancy of the overall structure.

T/O bridges are currently designed for a stress level of 27 ksi (TM 5-312), based on a minimum yield strength of 33 ksi. If it is assumed that the quality of fabrication and inspection is similar to that of civilian bridges (a_o is the same), then the initial K_I value

* The fracture condition provided in the design specifications includes an adjustment for factor of safety.

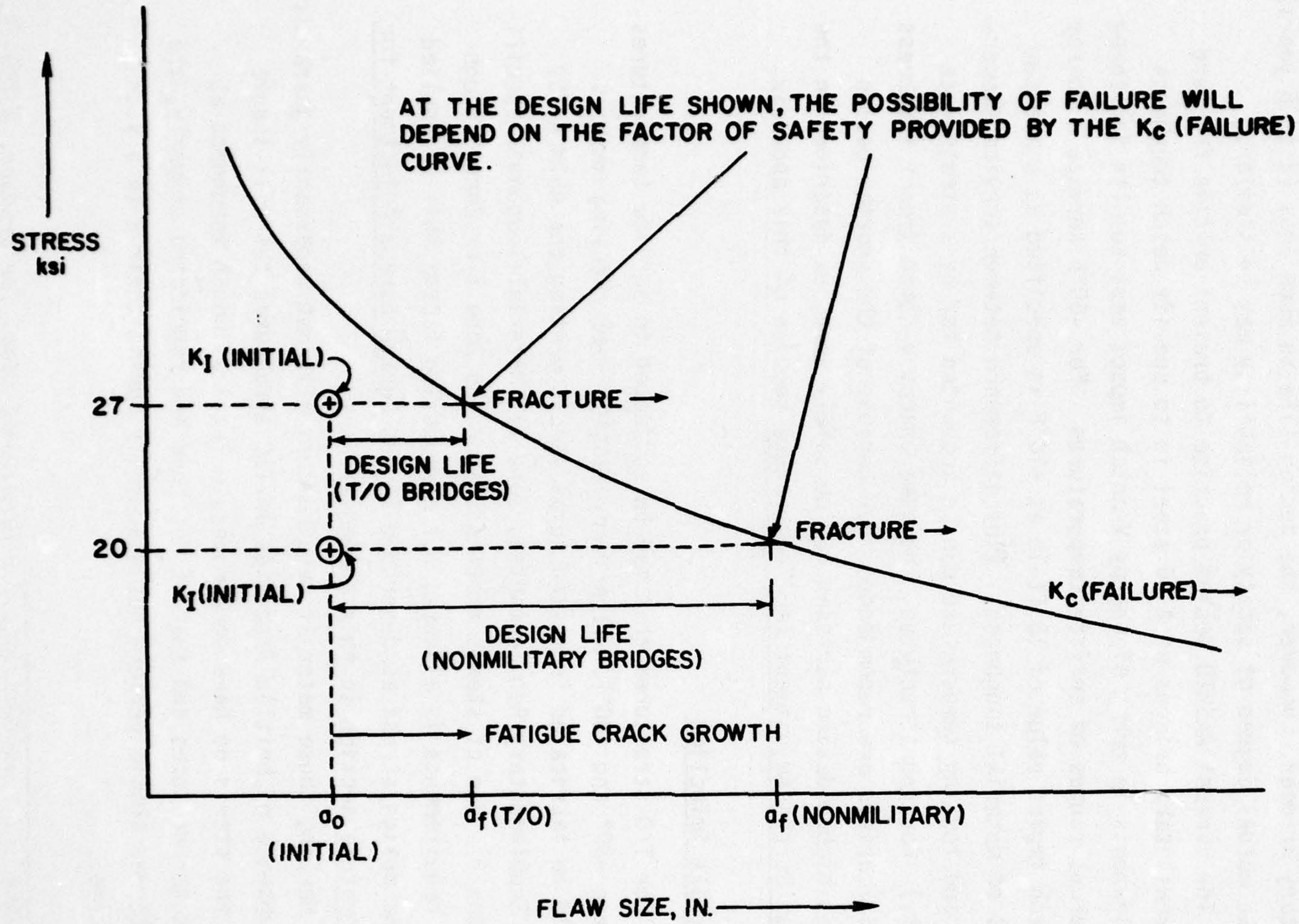


Figure E5. Schematic diagram showing general difference in service behavior of nonmilitary and T/O bridges.

for T/O bridges is much closer to the K_c value than it is for nonmilitary bridges. However, the design life is much less (2 to 5 years) so the actual degree of safety or reliability may be similar.

The current AASHTO design practice to prevent brittle fracture in nonmilitary bridges of A 36 steel is to specify notch toughness requirements in terms of Charpy V-notch impact test results for three different ranges of service temperatures. For -60°F service a Charpy V-notch impact value of 15 ft-lb at +10°F is specified as a minimum level of material toughness. (The difference between service temperature and testing temperature can be accounted for by a strain-rate shift.) For T/O structures, the same Charpy V-notch impact toughness specifications are recommended, but because of the shorter design lives, higher design stresses can be safely used, as described in the Design Stress Adjustment for Temperature section of this appendix.

Materials Selection

For T/O structures that may be subjected to service temperatures between -30° and -60°F, all primary tension load-carrying members should be fabricated from structural steels meeting the ASTM A 709 (S4, Supplementary Requirements), Zone III Material-Toughness Specifications.[12] Use of steels meeting either the Zone I or Zone 2 toughness requirements is allowed, but a reduction factor must be applied to the design stress as described in the Design Stress Adjustment for Temperature section in this appendix.

Meeting these material requirements will not necessarily guarantee the absence of brittle fractures in T/O structures but will insure that the steels do have some moderate level of notch toughness at these temperatures and that if designed and fabricated properly, the structures should perform satisfactorily during their 2 to 5 year lifetime.

[12] *Standard Specifications for Structural Steel for Bridges*, A 709-74, S4 Supplementary Requirements (American Society for Testing and Materials, 1974).

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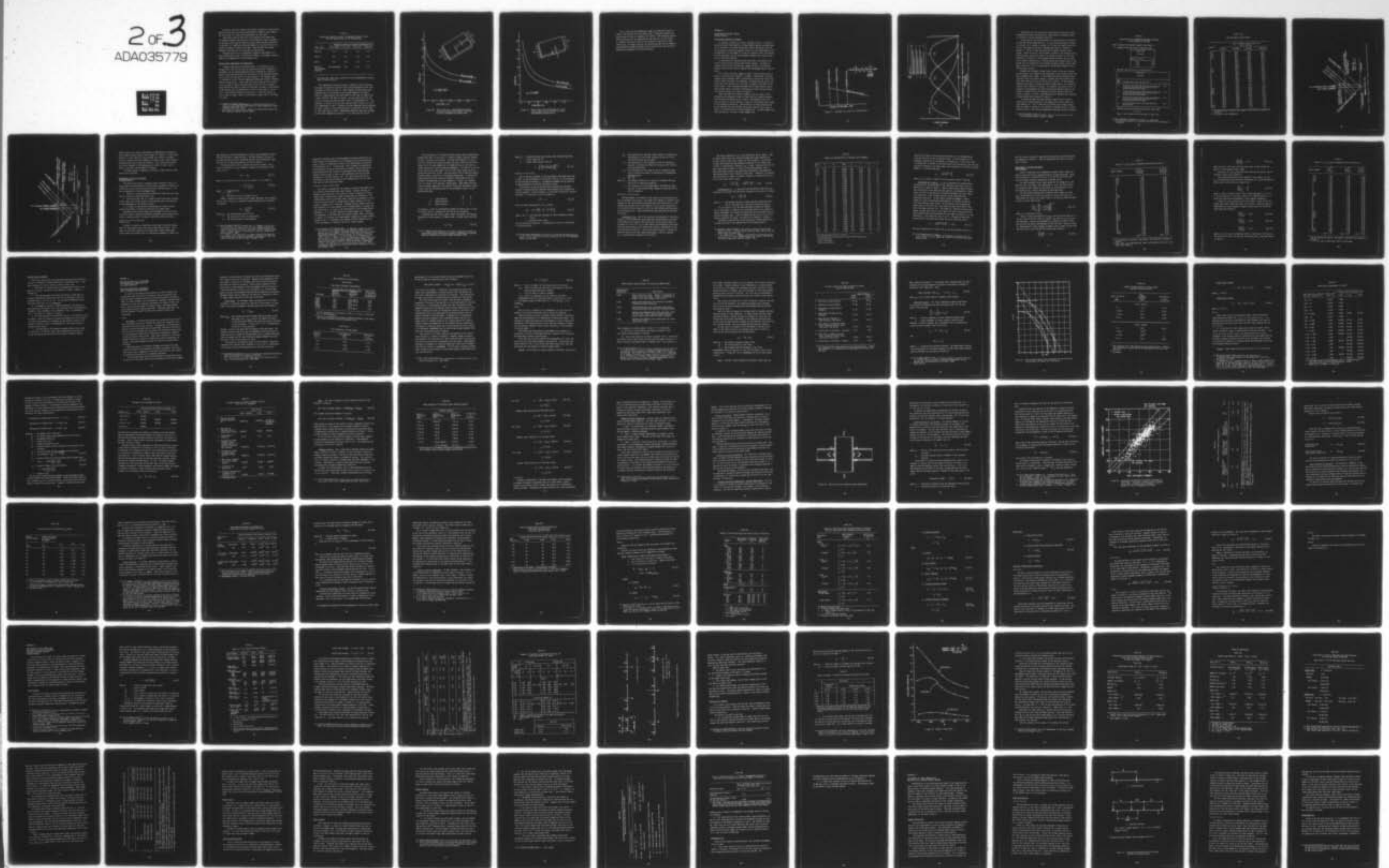

Secondary structural members and primary compression load-carrying members do not need to meet the ASTM material toughness requirements, but should meet all other ASTM requirements for the steel.

Weld metals used to fabricate T/O structures also should meet the AASHTO toughness requirements[13] using weld-metal impact specimens in accordance with American Welding Society (AWS) testing procedures.[14] In addition, all applicable AWS requirements for the qualification of welding procedures should be followed. Heat-affected zone notch-toughness specimens are not required for these steels.

Because there are no corresponding toughness requirements for high-strength bolts used in nonmilitary bridges, no toughness requirements are recommended for high-strength bolts.

Design Stress Adjustment for Temperature

Because there may be situations where it is necessary to use bridge steels that do not meet the appropriate material toughness requirements for the minimum expected service temperature, reduction factors have been established for application to the maximum allowable static tensile design stress in such instances. The reduced stress is referred to as the "service-temperature-adjusted" maximum allowable tensile design stress. These reduction factors (Table E1) are based on a K_{Ic} value at -60°F of no more than 33 ksi $\sqrt{in.}$ for the Zone I steels and 43 ksi $\sqrt{in.}$ or higher for the Zone III steels. Accordingly, if the minimum service temperature is to be in the range of -30° to -60°F, lower design stresses are necessary: a reduction factor of 0.6 is used for Zone I steel, 0.8 for Zone II steel, and 1.0 for Zone III steel.

[13] "Material Toughness Requirements," *Standard Specifications for Highway Bridges* (American Association of State Highway and Transportation Officials, 1973).

[14] *Specification for Mild Steel Covered Arc-Welding Electrodes*, AWS A5.1 (American Welding Society [AWS]).

Table E1

Temperature Reduction Factor for Maximum Allowable Static Tensile Design Stress at Low Temperatures

	Reduction Factors for Service Temperatures of			
Steel Type*	-31 to -60°F	-1 to -30°F	+32 to 0°F	Above 32°
Zone III	1.0	1.0	1.0	1.0
Zone II	0.8	1.0	1.0	1.0
Zone I	0.6	0.8	1.0	1.0
General (no toughness control)	Not permitted	0.6	0.8	1.0

* The three zone steel types conform to the S4 supplementary requirements of ASTM A 709-74.

The temperature reduction factors in Table E1 can be related to the material properties and flaw sizes using information of the type presented in Figures E6 and E7. These figures include curves that represent what is considered to be an upper bound for the Zone I steels (or approximately the lower bound for the Zone II steels) and a lower bound for the Zone III steels. At any given flaw size, the ratio of stresses provided by these curves is approximately 0.8. Consequently, a factor of 0.8 has been used for Zone II steels at a service temperature of -60°F. Because a similar reduction can be expected for Zone I steels, a 0.6 reduction factor is specified for this type of steel when used at -60°F. Comparable reductions have been included for the other temperature ranges of -1° to -30°F and +32° to 0°F.

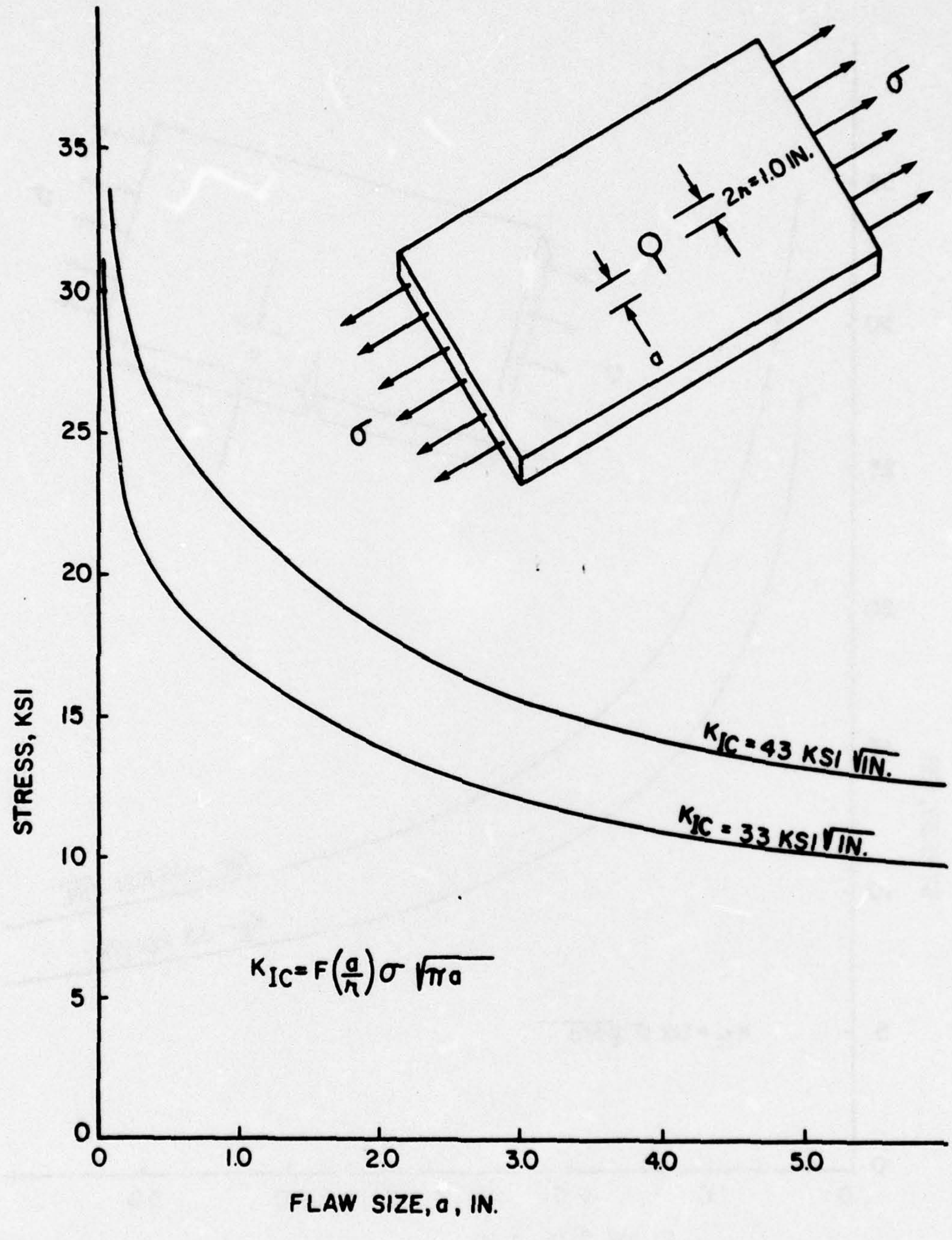

Figure E6. Stress-flaw size relationships for plate with crack growing from a hole--A 36 steel at -60°F (intermediate loading rate).

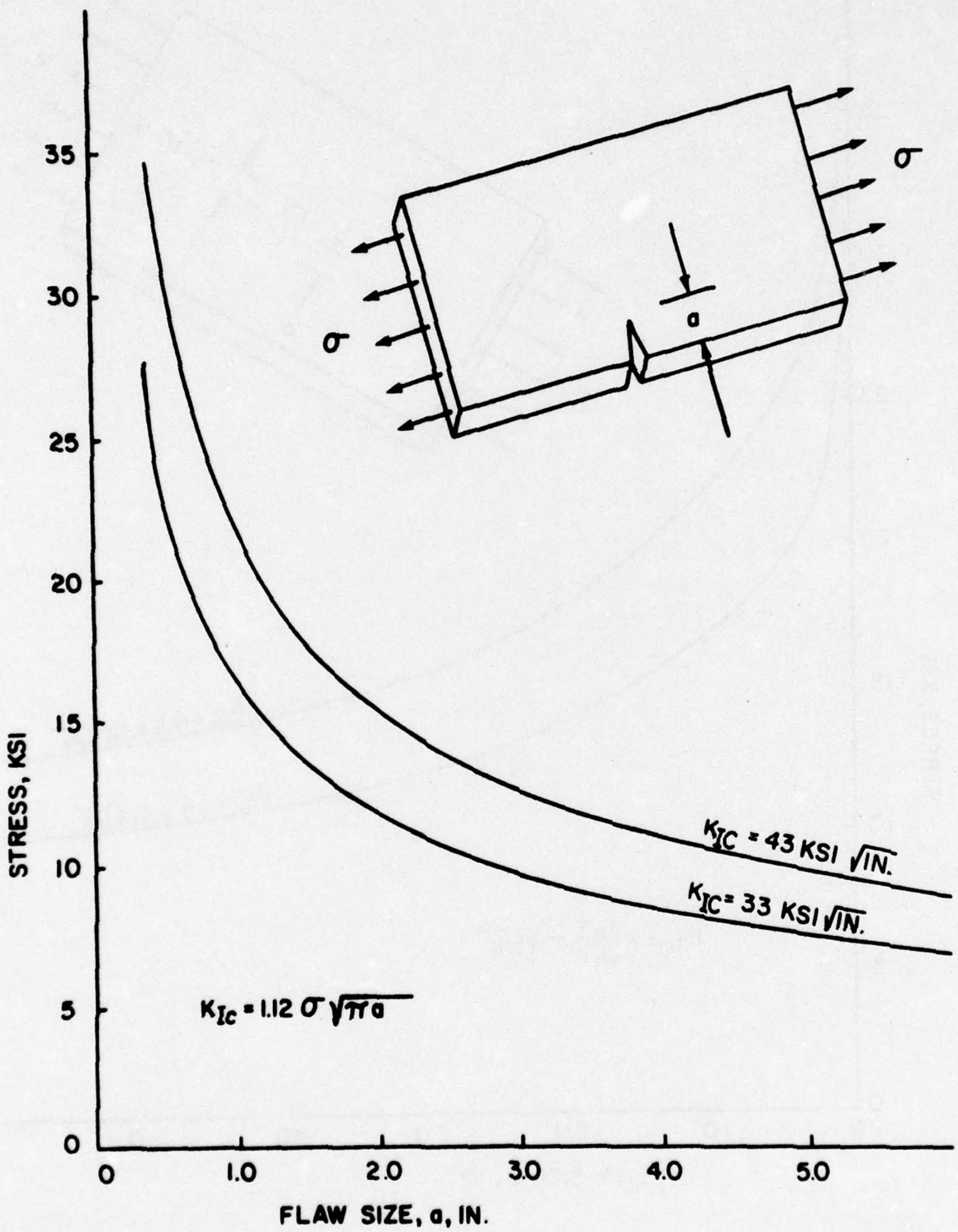

Figure E7. Stress-flaw size relationships for plate with an edge crack--A 36 steel at -60°F (intermediate loading rate).

For a given service temperature range, the reduction factors in Table E1 are based on the requirement that if repeated loadings (fatigue) cause a crack to propagate, the safety of the structure against brittle fracture will be approximately the same for all the steels. If, however, the stress range at the location in question is sufficiently low, a fatigue crack may not develop and the "service-temperature-adjusted" maximum allowable tensile design stress will provide a design less susceptible to brittle fracture.

APPENDIX F:

DEVELOPMENT OF BRIDGE FATIGUE DESIGN CRITERIA

Introductory Remarks on Fatigue

In contrast to design for static loadings, which is in terms of loads and static load capacity, fatigue design requires consideration of (1) the details at which fatigue may control the design, (2) the stress ranges or loading frequency history to which these details will be subjected, (3) the number of cycles of loading to which the details will be subjected, and (4) the allowable fatigue stress range, based on the first three factors.

In developing the fatigue design provisions, consideration was given to the principal factors that affect fatigue. In addition, suitable statistical distributions, damage criteria, and selected loading histories were used to develop the recommended design requirements.

For a given structural member or detail, such as those shown in Figure 1 and Table 10 of Chapter 3, Volume I, the fatigue life under constant-cycle repeated loads is principally a function of the stress range to which the detail is subjected. Although the mean stress can also affect the fatigue life, its effect is smaller than that of the stress range. Since including the mean stress would greatly complicate fatigue design, the basic relationship on which fatigue designs are currently based is as shown in Figure F1.

Under more realistic fatigue loadings, such as those to which bridges may be subjected (Figure F2 shows the four loading frequency distributions considered in this study), relationships similar to that in Figure F1 can be obtained in terms of the maximum stress range in the loading distribution. Approximations of these relationships can be obtained from the constant cycle data (such as those shown in Figure F1) and the use of a fatigue damage rule. The most common rule, that used herein, is Miner's linear damage rule.

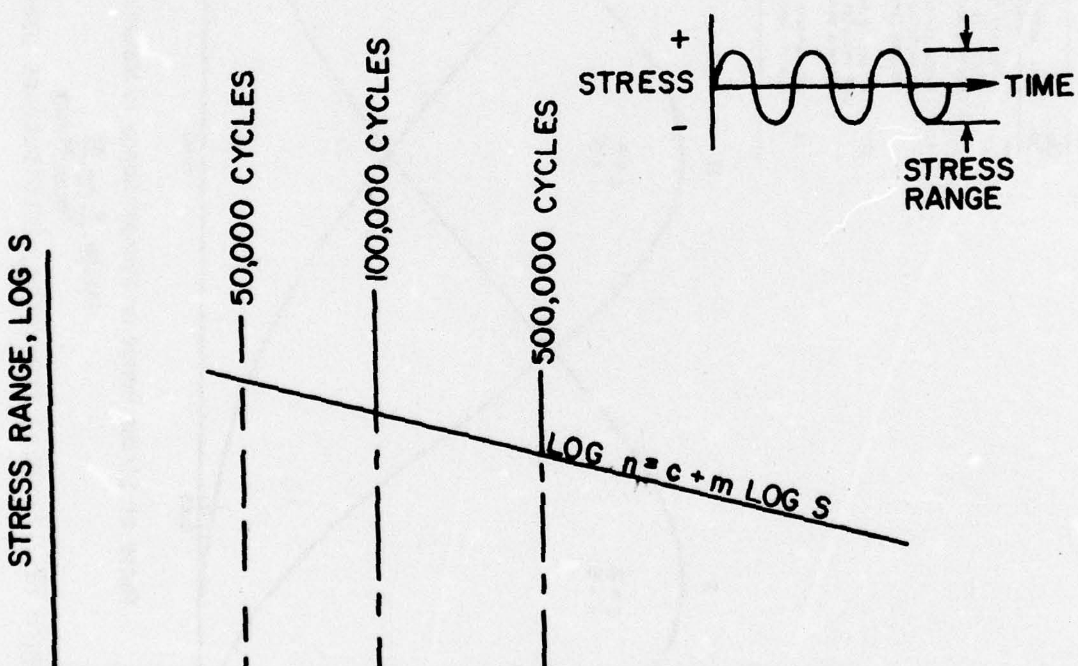

Figure F1. Schematic S-n curve for a given detail.

Load Type	Load Description	C_L
I	Primarily light weight vehicle crossings--50 percent of vehicles crossing weigh less than 0.3 of the maximum permitted vehicle weight.	1.90
II	Primarily medium weight vehicle crossings--50 percent of vehicles crossing weigh more than 0.5 of the maximum permitted vehicle weight.	1.35
III	Primarily heavy weight vehicle crossings--50 percent of vehicles crossing weigh more than 0.7 of the maximum permitted vehicle weight.	1.00
IV	All vehicle crossings are the maximum permitted vehicle weight.	0.75

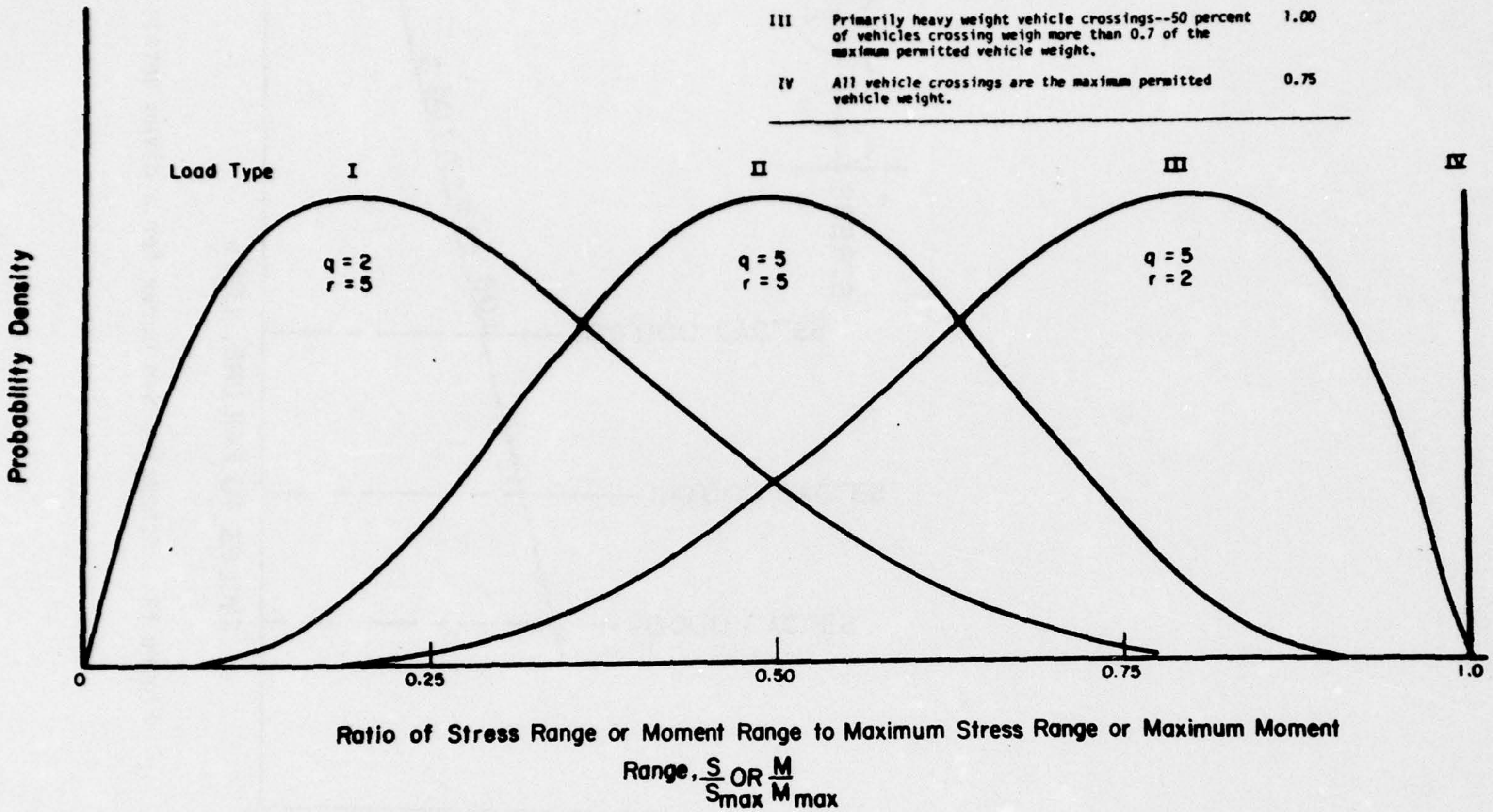

Figure F2. Loading frequency distributions used in fatigue design.

Considering also the statistical distribution of the basic data, recommended allowable maximum-stress ranges based on selected levels of reliability can be developed (Table F1).

Fatigue design can then be based on a selected level of reliability and the factors noted earlier: (1) the structural detail (Figure 1 and Table 10 of Chapter 3, Volume I), (2) the load type (Figure F2), and (3) the number of cycles of loading (see Table 11, Chapter 3, Volume I). Table F1 shows the resulting allowable fatigue stress ranges. These values can also be shown in terms of the minimum stress, maximum stress, and stress range in a fatigue diagram[15] of the type presented in Figure F3. This diagram shows that under certain conditions (various combinations of minimum stress and maximum stress), the design will be governed by the allowable fatigue stresses, while under other conditions it will be governed by the maximum allowable static tensile design stress.

Whether the allowable fatigue stress range or the static design stress controls will depend on the magnitude of the minimum stress and the allowable stress range. If the resulting maximum fatigue stress (minimum stress plus stress range) is greater than the maximum allowable static design stress, the latter will control. If the maximum repeated load stress is below the static design stress, the allowable fatigue stress range will control. For example, at points A and B in Figure F3 (corresponding to certain minimum and maximum stresses and 50,000 or 100,000 cycles of loading) the static design stress will control; however, at point C (500,000 cycles of loading) the allowable fatigue design stress range will control.

It should be noted that the maximum allowable static tensile stress may be reduced because of low temperature service conditions (see Appendix E). Such a reduction will modify the fatigue requirements as shown in Figure F4. For 50,000 or 100,000 cycles of loading (points D and E), the "service-temperature-adjusted" maximum allowable

[15] For more detail see W. H. Munse, *Fatigue of Welded Steel Structures* (Welding Research Council, 1964).

Table F1

Determination of Recommended Maximum Allowable Stress Ranges for Fatigue

Steps to determine the maximum allowable stress range, S_r:

1. **Determine reliability factor R^* from Table F1(a).**

Table F1(a)

R Factors

Reliability Level	R
0.90	1.15
0.95	1.00
0.99	0.76

2. **Determine load-type factor C_L** from Table F1(b).**

Table F1(b)

C_L Factors

Load Type	Load Description (Also see Figure F2.)	C_L
I	**Primarily light weight vehicle crossings--50 percent of vehicles crossing weigh less than 0.3 of the maximum permitted vehicle weight.**	1.90
II	**Primarily medium weight vehicle crossings--50 percent of vehicles crossing weigh more than 0.5 of the maximum permitted vehicle weight.**	1.35
III	**Primarily heavy weight vehicle crossings--50 percent of vehicles crossing weigh more than 0.7 of the maximum permitted vehicle weight.**	1.00
IV	**All vehicle crossings are the maximum permitted vehicle weight.**	0.75

3. **Determine the maximum allowable fatigue design stress range**

$$S_r = RC_L S$$

where S = base allowable stress range given in Table F1(c).

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* **For T/O bridges, a 0.95 level of reliability is recommended.**

** **If load type information is not available, load Type III is recommended for T/O bridges.**

Table F1(c)

Base Allowable Stress Range S

Detail*	Base Allowable Stress Range (ksi) No. of Cycles			
	50,000	100,000	500,000	2,000,000
1(1)	45.3	42.2	35.8	31.0
1(2)	59.1	54.3	44.5	37.5
2(1)	46.8	42.1	32.8	26.5
2(2)	52.8	47.1	36.2	28.8
3	43.0	38.0	28.4	22.1
4	56.1	43.6	24.3	14.7
5	25.3	20.3	12.2	7.9
6	56.1	43.6	24.3	14.7
7	36.4	29.9	18.9	12.7
8	47.7	43.6	35.3	29.4
9(1)	27.1	24.7	19.8	16.5
9(2)	37.5	34.1	27.5	22.8
10	37.8	30.8	19.2	12.7
11	40.6	33.9	22.3	15.5
12	35.2	27.7	15.9	9.8
13	42.1	36.1	25.3	18.6
14	34.2	27.9	17.5	11.7
15	25.0	20.5	12.9	8.6
16	**	**	**	**
17	25.2	20.6	12.9	8.6
18	17.0	12.9	6.7	3.9
19(1)	23.9	21.3	16.3	12.9
19(2)	23.5	21.0	16.0	12.7
20(1)	36.5	29.1	17.2	10.9
20(2)	16.7	14.3	10.1	7.4
21	36.2	32.6	25.6	20.8
22	44.7	34.6	19.1	11.5
23	35.9	29.0	17.7	11.5
24	35.9	29.0	17.7	11.5
25	40.6	30.8	16.3	9.4
26	26.7	22.2	14.4	10.0
27(1)	20.1	17.4	12.3	9.1
27(2)	21.8	18.7	13.0	9.6

* For description of details see Figure 1 and Table 10 of Chapter 3, Volume I.

** This detail not recommended.

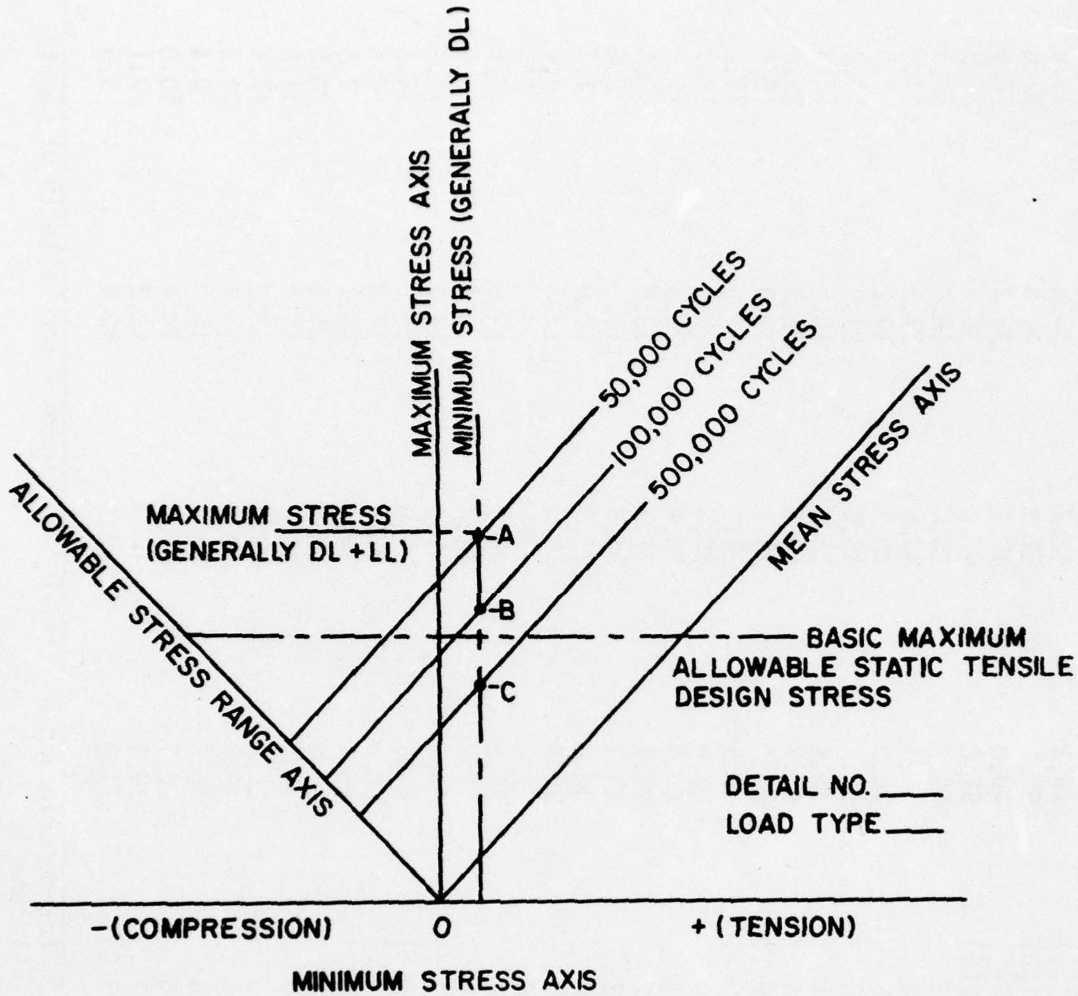

Figure F3. Fatigue diagram of allowable fatigue stresses and static design stress.

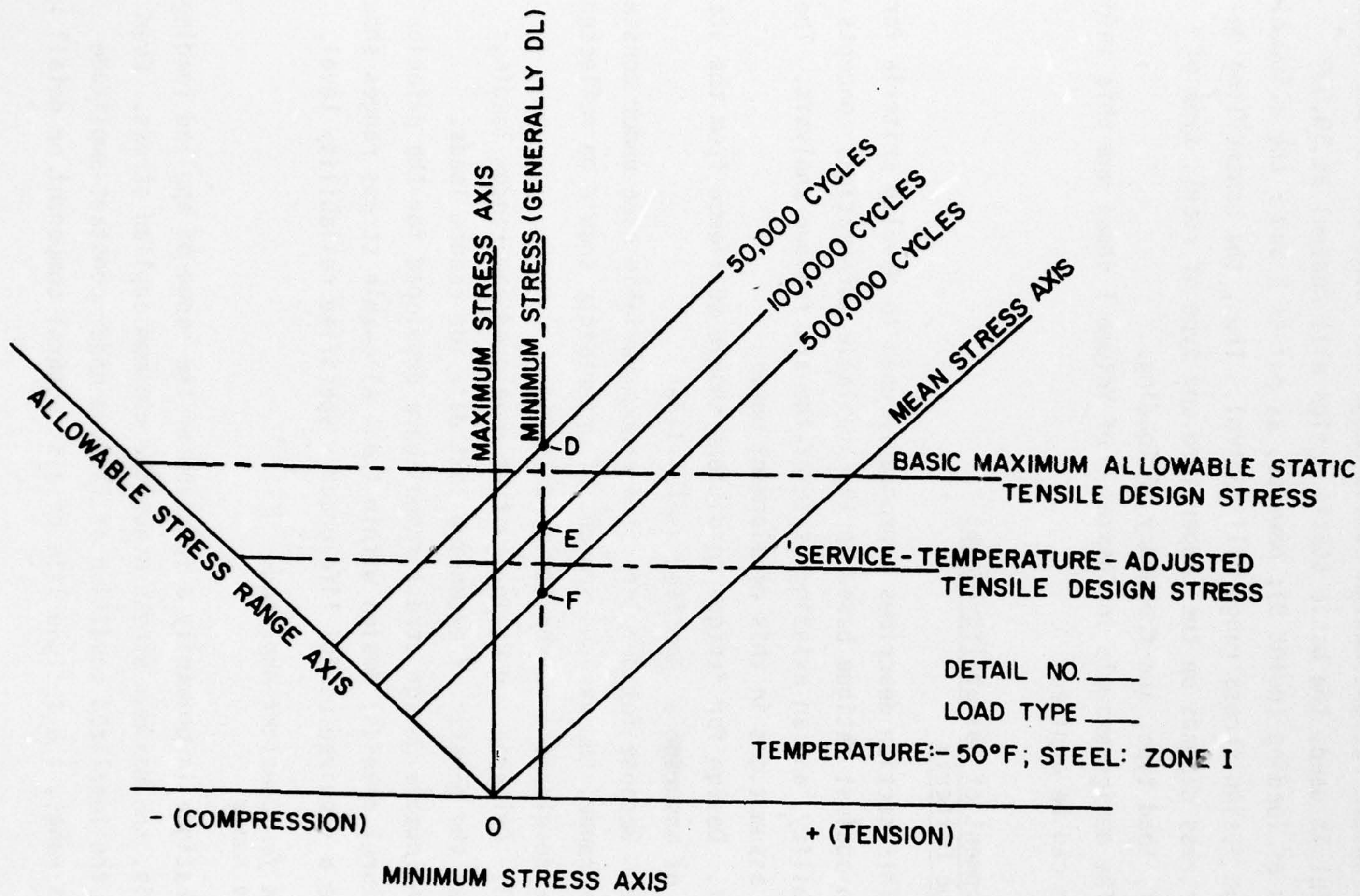

Figure F4. Fatigue diagram of allowable fatigue stresses reduced for low temperature and type of steel.

tensile stress will control the design; at 500,000 cycles (point F), the allowable fatigue design stress range will control. If the proper material is used, the basic static design will control at 50,000 cycles of loading (point D); however, at points E and F the allowable fatigue design stress range will control. Thus, the controlling design stress depends on the temperature and type of steel, type of detail, load type, and frequency of loading.

The design example in Chapter 5 of Volume I shows how this information can be applied.

Development of Reliability-Based Fatigue Criteria

This section describes a procedure used to develop criteria for design against fatigue based on the available information, concepts of reliability, and an existing state-of-the-art fatigue analysis. The basic assumptions in this development were:

1. Design for fatigue conditions should be viewed from the standpoint of assuring a specified useful life
2. Because fatigue life is a random variable even under constant stress range, the various sources of uncertainty should be reflected in the development of the design criteria
3. Criteria for design must be developed for random loadings without the benefit of extensive test data for random loads.

Allowable design stress ranges were developed for the principal structural details; design within these allowable stress ranges should assure a required useful life under a specified reliability level.

Design for Constant-Amplitude Stress Range

Fatigue is primarily a function of the range of applied loading, that is, the maximum stress minus the minimum applied stress. Even under the idealized condition of fatigue under constant-amplitude stress range, the fatigue life of a structural component or detail has been observed to have considerable variability and, therefore, should be described with a random variable. The mean fatigue life and associated variability can be evaluated directly from experimental data available for a specified material and type of detail.[16]

The required mean life $\overline{n}_D$ necessary to insure a useful life n_o with a reliability of $L(n_o)$, based on a Weibull distribution for fatigue life is[17]

$$\overline{n}_D = n_o \gamma_L \qquad \text{[Eq F1]}$$

where γ_L is the *fatigue life factor*, or *scatter factor*, given by

$$\gamma_L = \frac{\Gamma(1 + a)}{[1 - L(n_o)]^a} \qquad \text{[Eq F2]}$$

where Γ = gamma function

$a = \Omega_n^{1.08}$

Ω_n = uncertainty level in fatigue life n.

Under a constant-amplitude stress range, therefore, the allowable design stress range S_D is obtained from the appropriate S-n equation as follows:

$$S_D = \left(\frac{c}{\overline{n}_D}\right)^{1/m} \qquad \text{[Eq F3]}$$

where $\overline{n}_D$ = the required mean life of Eq F1

c = the intercept of the S-n relationship

m = the slope of the S-n relationship.

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[16] For an example, see T. R. Gurney and S. J. Maddox, *A Re-Analysis of Fatigue Data for Welded Joints in Steel*, Research Report No. E-44/72 (The Welding Institute, Cambridge, England, January 1972); and W. H. Munse, *Fatigue of Welded Steel Structures* (Welding Research Council, 1964).

[17] L. I. Knab, A. H-S. Ang, and W. H. Munse, "Reliability Based Fatigue Design Code for Military Bridges," Proceedings of ASCE Specialty Conference on Metal Bridges (November 1974).

Equation F2 indicates that the development of design criteria for fatigue requires the determination or assessment of the uncertainty measure Ω_n, which is expressed in terms of the coefficients of variation of the fatigue life n. In particular, this includes the uncertainty associated with the basic variability of fatigue life as reflected in and estimated from the scatter of experimental data obtained under constant stress range, as well as the uncertainty arising from the estimation of the mean fatigue life. The latter would include uncertainties in the specification of the loading and in the S-n equation used for the particular detail.

Design for Random Stress Range

The results presented thus far pertain to and are applicable only for uniform constant-amplitude stress ranges. Because stresses induced by actual vehicular live loads will cover a spectrum of stress-range values, fatigue under random stresses must be considered. The relationships given for constant stress range can be extended to situations involving variable or random stress ranges; however, for this purpose a damage rule is required. Although various damage rules have been proposed, the linear damage rule of Palmgren-Miner is perhaps the most widely used. In spite of the shortcomings of the Palmgren-Miner hypothesis, it is the most workable available damage rule. Despite the weaknesses of the Miner Rule for random fatigue, there is evidence to support its validity for bridge members subjected to random traffic-induced loads.[18] Accordingly, this damage rule was adapted for use in the development of fatigue criteria for random loading.

[18] W. H. Munse, J. R. Fuller, and K. S. Petersen, *Cumulative Damage in Structural Joints*, AREA Bulletin 544 (June-July 1958), p 67; G. Welter, and J. A. Choquet, "Variable Stress Cycle Fatigue of Large Butt-Welded Specimens," *Welding Journal*, Vol 46, No. 1 (January 1967), pp 39-s to 48-3; C. G. Schilling, H. H. Klippstein, J. M. Barsom, and G. T. Blake, *Fatigue of Welded Steel Bridge Members Under Variable-Amplitude Loadings*, Research Results Digest, Highway Research Board Digest 60 (April 1974); and Harold S. Reemsnyder, *Fatigue Life Extension of Riveted Structural Connections*, paper presented at ASCE Specialty Conference on Metal Bridges, St. Louis, MO (November 1974).

The distribution of the applied stress range S may be conveniently modeled with the *beta-distribution;* Figure F2 shows a typical example of such a frequency distribution. The beta distribution is a versatile distribution with specified upper and lower limits for the stress range; it can be made symmetrical or skewed one way or the other by proper selection of the distribution parameters. In light of its versatility and the fact that upper limits in the stress range can be expected from vehicular traffic, this particular form of the distribution function shown in Figure F2 is considered appropriate. In particular, three beta-distribution types of loading or load patterns are prescribed, corresponding to a high frequency of "light" (I), "medium" (II), or "heavy" (III) vehicles, respectively. These three load patterns are shown graphically in Figure F2 and can be described, respectively, with the following values of the parameters q and r of the beta-distribution:

		q	r
I	light vehicles	2	5
II	medium vehicles	5	5
III	heavy vehicles	5	2

In addition, a type IV distribution provides for those cases in which the stress range is constant for the life of the structure.

Assuming that the stress range is beta distributed, the maximum permissible stress-range S_0 in design under a random stress condition is[19]

$$S_0 \leq \xi S_D \qquad \text{[Eq F4]}$$

[19] L. I. Knab, A. H-S. Ang, and H. W. Munse, "Reliability Based Fatigue Design Code for Military Bridges," Proceedings of the ASCE Specialty Conference on Metal Bridges (November 1974).

where S_D = allowable design stress range under constant-amplitude stress range (Eq F3)

ξ = random stress factor given by

$$\xi = \left[\frac{\Gamma(m + q + r)\Gamma(q)}{\Gamma(m + q)\Gamma(q + r)}\right]^{1/m} \qquad \text{[Eq F5]}$$

Analysis of Uncertainty

One of the main problems in the developments described above and, in fact, in the development of any design criteria, is the assessment of the uncertainty measure Ω_n. Engineering judgment may be required in the assessment of realistic uncertainty levels.

The value of Ω_n may be evaluated by assessing the individual sources of uncertainty and combining them systematically through statistical methods. For this purpose, a first-order statistical analysis was made.[20] On the basis of the S-n equation,

$$nS^m = c \qquad \text{[Eq F6]}$$

the first-order approximation for Ω_n yields,

$$\Omega_n^2 = \Omega_f^2 + \bar{m}^2\Omega_{\bar{S}}^2 + \Omega_c^2 + (\bar{m}\,\ln\bar{S})^2\Omega_m^2 \qquad \text{[Eq F7]}$$

where m and c = the slope and intercept of the S-n equation, respectively

$\bar{S}$ = the mean stress range

and the uncertainty measures are expressed in terms of coefficient of variation (COV):

[20] A. H-S. Ang, "Structural Risk Analysis and Reliability-Based Design," *Journal of the Structural Division*, ASCE, Vol 99, No. ST9 (September 1973), pp 1891-1910.

$\Omega_{\overline{S}}$ = the uncertainty in the mean stress range $\overline{S}$, including the uncertainties in the stress analysis as well as in the load amplification due to impact

Ω_c = the uncertainty in the intercept of the S-n regression equation, including the effect of the quality of fabrication and workmanship

Ω_m = the uncertainty in the slope of the S-n regression equation, including the effect of the quality of workmanship and fabrication

$\Omega_f = \sqrt{\delta_f^2 + \Delta_f^2}$

where δ_f = the average variability or scatter of fatigue life data about the S-n regression equation

Δ_f = the inaccuracy of the fatigue model, including the imperfections in the Palmgren-Miner damage rule and the form of the S-n equation.

Evaluation of Uncertainty

The development of allowable stress ranges for the structural details considered (see Figure 1 and Table 10 of Chapter 3, Volume I) must account for the uncertainties associated with the variabilities in the loading and fatigue life of each detail, as well as the inaccuracies in the definition and analysis of the live load effects and the prediction of fatigue life. These uncertainties were analyzed and assessed as follows.

Evaluation of $\Omega_{\overline{S}}$. One of the sources of uncertainty in the stress range $\Omega_{\overline{S}}$ is the uncertainty associated with the estimation or analysis of the mean stress range $\overline{S}$ (the variability in the stress range $\overline{S}$ is accounted for in Eq F7), or in other words, the uncertainty associated with the specification of the mean stress range as represented in the load patterns of Figure F2. It was felt that the error in structural analysis would not be very large; accordingly, a COV of 5 percent (Δ_S = 5 percent) was assigned for this error.

The other component of $\Omega_{\bar{S}}$ is the uncertainty due to impact. Data on impact coefficients for railroad bridges have been obtained for various railway bridge span lengths and train speeds.[21] An analysis of these data shows that the average variability of the impact coefficient is approximately 8 percent ($\delta_{Im} = 0.08$). Assuming that the impact factor used in design is based on field measurements such as those described by Byers, any additional uncertainty associated with the average impact factor should be negligible. Therefore, considering that the effect of impact is normally treated as a multiplicative factor on the stress range S gives

$$\Omega_{\bar{S}} = \sqrt{\Delta_S^2 + \delta_{Im}^2} = \sqrt{.05^2 + .08^2} = 0.09 \qquad \text{[Eq F8]}$$

<u>Evaluation of Ω_f</u>. Ω_f represents the uncertainty underlying the prediction of fatigue life and may be analyzed in two parts as follows:

$$\Omega_f^2 = \sqrt{\delta_f^2 + \Delta_f^2} \qquad \text{[Eq F9]}$$

where δ_f = the average variability about the mean regression equation of test data for a particular structural detail.

This variability (δ_f) has been analyzed by Gurney and Maddox[22] for a large number of details; for structural details not covered by Gurney and Maddox, the required variabilities were evaluated using extensive available data. Table F2 summarizes the results for all the details presented in Figure 1 and Table 10 of Chapter 3, Volume I.

Additional uncertainties in the prediction of mean fatigue life include those due to the imperfection of the fatigue models,

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[21] William G. Byers, "Impact from Railway Loading on Steel Girder Spans," *Journal of the Structural Division*, ASCE, Vol 96, No. ST6 (June 1970), pp 1093-1103.

[22] T. R. Gurney and S. J. Maddox, *A Re-Analysis of Fatigue Data for Welded Joints in Steel*, Research Report No. E/44/72 (The Welding Institute, Cambridge, England, January 1972).

Table F2

Summary of Uncertainties in Fatigue Life Parameters

Detail No.	δ_f	Δ_f	$\log_{10} c$	Ω_c	m	Ω_S	Ω_N
1(1)*	0.56	0.15	21.5082	0.40	-9.778	0.09	1.13
1(2)**	0.63	0.15	19.6140	0.40	-8.080	0.09	1.05
2(1)*	0.56	0.15	16.0157	0.40	-6.484	0.09	0.91
2(2)**	0.63	0.15	15.7611	0.40	-6.102	0.09	0.94
3	0.35	0.15	14.0231	0.40	-5.524	0.09	0.74
4	0.35	0.15	9.8599	0.40	-2.750	0.09	0.60
5	0.21	0.15	9.3838	0.40	-3.168	0.09	0.55
6	0.35	0.15	9.8599	0.40	-2.750	0.09	0.60
7	0.49	0.15	10.6089	0.40	-3.500	0.09	0.72
8	0.86	0.15	18.3252	0.40	-7.618	0.09	1.18
9(1)†	0.85	0.15	16.1598	0.40	-7.427	0.09	1.16
9(2)††	0.77	0.15	17.1006	0.40	-7.419	0.09	1.10
10	0.62	0.15	10.6335	0.40	-3.388	0.09	0.81
11	0.42	0.15	11.2471	0.40	-3.843	0.09	0.69
12	0.55	0.15	9.7112	0.40	-2.895	0.09	0.74
13	0.41	0.15	12.4019	0.40	-4.530	0.09	0.72
14	0.46	0.15	10.3885	0.40	-3.437	0.09	0.70
15	0.40	0.15	9.9206	0.40	-3.478	0.09	0.66
16	0.67	0.15	10.8316	0.40	-3.721	0.09	0.86
17	0.47	0.15	9.9313	0.40	-3.430	0.09	0.71
18	0.47	0.15	8.2372	0.40	-2.488	0.09	0.67
19(1)†	0.83	0.15	13.7474	0.40	-5.997	0.09	1.08
19(2)+	0.86	0.15	13.7474	0.40	-5.997	0.09	1.10
20(1)†	0.46	0.15	9.9037	0.40	-3.054	0.09	0.69
20(2)+	0.67	0.15	10.8804	0.40	-4.559	0.09	0.89
21	0.80	0.15	15.8602	0.40	-6.681	0.09	1.09
22	0.30	0.15	9.4933	0.40	-2.714	0.09	0.58
23	0.30	0.15	10.0404	0.40	-3.246	0.09	0.60
24	0.30	0.15	10.0404	0.40	-3.246	0.09	0.60
25	0.49	0.15	9.2560	0.40	-2.526	0.09	0.69
26	0.39	0.15	10.3807	0.40	-3.742	0.09	0.67
27(1)†	0.58	0.15	11.2706	0.40	-4.652	0.09	0.83
27(2)+	0.46	0.15	11.0889	0.40	-4.485	0.09	0.75

* For mild steels such as A 36, A 7, A 373, etc.

** For high-strength low alloy steels such as A 242, A 588, A572 Grade 50, etc.

† Stress on base metal.

†† Shear on the fasteners.

+ Stress on throat of weld.

specifically those arising from the deficiency of the Palmgren-Miner rule and the form of the S-n equation used. In this connection, an aggregate uncertainty of 15 percent is assigned (Δ_f = 0.15); this level of uncertainty is perhaps on the high (safe) side considering that the Palmgren-Miner hypothesis has been shown to be reasonable for fatigue of bridge members. It follows then that

$$\Omega_f = \sqrt{(0.15)^2 + \delta_f^2} \qquad \text{[Eq F10]}$$

where δ_f is as given in Table F2 for the specific detail involved.

Evaluation of Ω_c and Ω_m. Ω_c and Ω_m represent the uncertainties associated with the estimation of the intercept and slope of the S-n equation for a specific detail. Both of these factors should reflect the uncertainties associated with the quality of workmanship in fabrication, as well as the possible differences between field and laboratory conditions, since fatigue data are largely obtained from laboratory specimens. In this regard, Gurney and Maddox[23] report ranges of c and m representing approximately ±2 standard deviations from the respective mean values. On this basis, the corresponding coefficient of variation for c was found to range from 0.08 to 0.89, with an average value of 0.37. The range of c reported by Gurney and Maddox includes the effect of the slope m of the S-n regression equations, meaning that the uncertainty in m has been reflected in Ω_c, and therefore Ω_m = 0.0 should be used in Eq F7. Allowing an additional uncertainty of 15 percent to account for field conditions gives

$$\Omega_c = \sqrt{0.37^2 + 0.15^2} = 0.40 \qquad \text{[Eq F11]}$$

The total uncertainty in fatigue life Ω_n can be obtained using Eq F7

[23] T. R. Gurney and S. J. Maddox, *A Re-Analysis of Fatigue Data for Welded Joints in Steel*, Report No. E/44/72 (The Welding Institute, January 1972).

with the various sources of uncertainties evaluated as described above and tabulated in Table F2. Table F2 summarizes the results for each type of detail.

Development of Fatigue Allowable Stress Ranges

Table F1 summarizes the recommended allowable stress ranges for the 27 structural details commonly found in bridge structures. These were developed for reliability levels of 90, 95 and 99 percent and lives of 50,000, 100,000, 500,000 and 2,000,000 cycles.

The 1680 allowable stress range values for 33 categories in the details, four load types, four magnitudes of life, and three levels of reliability were reduced to 140 values by using reliability (R) and load-type (C_L) factors developed as described below.

The relationship between two allowable stress ranges, S_{r1} and S_{r2}, corresponding to reliability levels of $L_1(n_o)$ and $L_2(n_o)$, for a given number of cycles (n_o), detail type (c, m, and Ω_n), and load type (q and r) can be shown to be:

$$\left(\frac{S_{r1}}{S_{r2}}\right)_R = \left\{\frac{\left[1 - L_1(n_o)\right]^a}{\left[1 - L_2(n_o)\right]^a}\right\}^{1/m} \qquad \text{[Eq F12]}$$

where a is as defined in Eq F2.

Table F3 gives the values of S_{r1}/S_{r2}, for the various details using the 0.95 reliability level as a base. Two sets of S_{r1}/S_{r2} ratios are presented: one set corresponding to reliability levels of 0.90 and 0.95 and the other corresponding to 0.99 and 0.95. Based on Table F3, representative ratios between allowable stress ranges for the 0.90 and 0.95 levels, and 0.99 and 0.95 levels were chosen as:

$$\frac{(S_r)_{.90}}{(S_r)_{.95}} = 1.15 \qquad \text{[Eq F13]}$$

Table F3

Values of S_{r1}/S_{r2} Used to Determine Reliability Factor, R

Detail Number*	$\frac{(S_r)_{0.90}}{(S_r)_{0.95}}$ **	$\frac{(S_r)_{0.99}}{(S_r)_{0.95}}$ **
1(1)	1.08	0.83
1(2)	1.09	0.81
2(1)	1.10	0.80
2(2)	1.11	0.78
3	1.10	0.81
4	1.16	0.71
5	1.12	0.76
6	1.16	0.71
7	1.15	0.72
8	1.11	0.78
9(1)	1.12	0.78
9(2)	1.11	0.79
10	1.18	0.68
11	1.13	0.75
12	1.19	0.67
13	1.11	0.78
14	1.15	0.73
15	1.14	0.74
16	1.17	0.69
17	1.15	0.72
18	1.20	0.66
19(1)	1.13	0.75
19(2)	1.14	0.74
20(1)	1.16	0.70
20(2)	1.14	0.73
21	1.12	0.77
22	1.15	0.72
23	1.13	0.75
24	1.13	0.75
25	1.20	0.65
26	1.13	0.76
27(1)	1.13	0.75
27(2)	1.12	0.77
	Avg = 1.14	Avg = 0.74

* For description of details, see Figure 1 and Table 10 of Chapter 3, Volume I.

** 0.90, 0.95, and 0.99 subscripts refer to reliability level of allowable stress range S_r.

$$\frac{(S_r)_{.99}}{(S_r)_{.95}} = 0.75 \qquad \text{[Eq F14]}$$

where the 0.90, 0.95, and 0.99 subscripts refer to the reliability levels of the allowable stress range.

The values of S_{r1}/S_{r2} given by Eq F13 and F14 are used as the reliability factors (R) in Table F1.

The relationship between two allowable stress ranges, S_{r1} and S_{r2}, corresponding to two different load types, for a given reliability level, detail, and life can be shown to be:

$$\left(\frac{S_{r1}}{S_{r2}}\right)_{C_L} = \frac{\xi_1}{\xi_2} \qquad \text{[Eq F15]}$$

where ξ_1 and ξ_2, given by Eq F5, refer to the load types (I, II, III, or IV in Figure F2). As evident from Eq F5, ξ depends only on r, q (load type distribution parameters), and m (detail parameter).

Values of S_{r1}/S_{r2} based on Eq F15 are shown in Table F4 for different load types, using load type III as a base. Based on Table F4, representative ratios between allowable stress ranges for different load types were chosen as:

$$\frac{(S_r)_{I}}{(S_r)_{III}} = 1.90 \qquad \text{[Eq F16]}$$

$$\frac{(S_r)_{II}}{(S_r)_{III}} = 1.35 \qquad \text{[Eq F17]}$$

$$\frac{(S_r)_{IV}}{(S_r)_{III}} = 0.75 \qquad \text{[Eq F18]}$$

where I, II, III, and IV subscripts refer to load type. The values of the ratios of Eq F16, F17, and F18 are used as the C_L load type factors in Table F1.

Table F4

Values of S_{r1}/S_{r2} Used to Determine Load Type Factor, C_L

Detail Number*	$\frac{(S_r)_I}{(S_r)_{III}}$**	$\frac{(S_r)_{II}}{(S_r)_{III}}$**	$\frac{(S_r)_{IV}}{(S_r)_{III}}$**
1(1)	1.58	1.28	0.81
1(2)	1.65	1.30	0.80
2(1)	1.73	1.32	0.79
2(2)	1.76	1.32	0.78
3	1.80	1.33	0.78
4	2.10	1.38	0.74
5	2.04	1.37	0.75
6	2.10	1.38	0.74
7	1.99	1.37	0.75
8	1.67	1.30	0.80
9(1)	1.68	1.31	0.79
9(2)	1.68	1.31	0.79
10	2.01	1.37	0.75
11	1.95	1.36	0.76
12	2.08	1.38	0.75
13	1.88	1.35	0.77
14	2.00	1.37	0.75
15	2.00	1.37	0.75
16	1.97	1.36	0.76
17	2.00	1.37	0.75
18	2.14	1.39	0.74
19(1)	1.76	1.32	0.78
19(2)	1.76	1.32	0.78
20(1)	2.05	1.38	0.75
20(2)	1.88	1.35	0.77
21	1.72	1.32	0.79
22	2.10	1.38	0.74
23	2.03	1.37	0.75
24	2.03	1.37	0.75
25	2.14	1.39	0.74
26	1.96	1.36	0.76
27(1)	1.87	1.35	0.77
27(2)	1.88	1.35	0.77
	Avg. = 1.91	Avg. = 1.35	Avg. = 0.76

* For description of details, see Figure 1 and Table 10 of Chapter 3, Volume I.

** I, II, III, and IV subscripts refer to load types.

Fatigue Design Procedure

The fatigue design data discussed above make possible fatigue design of T/O bridges for random loadings on a reliability basis. The steps necessary for such a design, or design check, are

1. The detail in question shall be categorized in terms of the details in Figure 1 and Table 10 of Chapter 3, Volume I (details 1 through 27).

2. The load frequency distribution of Figure F2 that best represents the expected loading over the life of the detail shall be selected (Type I, II, III, or IV). If load type information is not available, Load Type III is recommended.

3. The number of cycles of loading expected at the detail in question during the useful life of the structure shall be established (50,000 or less, 100,000 or less, 500,000 or less, or 2,000,000 or less). See Table 11, Chapter 3, Volume I.

4. The desired level of reliability (90, 95, or 99 percent) shall be chosen. Based on extensive analyses and comparisons with reliability levels corresponding to permanent bridge fatigue criteria, a 95 percent reliability level is recommended for T/O bridge fatigue allowable stress ranges.

5. The magnitude of the maximum allowable design stress range for the detail in question shall then be determined from Table F1 (using the appropriate classifiers from steps 1 through 4 above).

APPENDIX G:

MEAN AND VARIABILITY OF RESISTANCE OF FASTENERS AND CONNECTIONS FOR THE STATIC LOAD CASE

Resistance Functions - Laboratory Data - Fasteners and Connections

To establish the design relationships for steel fasteners and connections based on statistical concepts, data are necessary from which the mean strengths and values of the coefficients of variation (COV) for these strengths can be determined. Numerous references were assembled to obtain the necessary information for welds, rivets, and bolts, and for various types of riveted, bolted, and welded connections under different loading conditions. These data were then studied and analyzed in detail to derive the necessary statistical information.

Welds

Butt Welds. The welding electrodes for structural welds are generally selected to match or slightly over-match the strength of the base metals with which they are used. Consequently, full-penetration groove or butt welds deposited with electrodes selected on this basis will have yield strengths that over-match, and ultimate strengths that at least match and generally over-match the corresponding strengths of the base metals being joined. Under these conditions, such welds, whether subjected to tension or compression, would be adequately designed if the same allowable stresses are used for the base metals and welds.

When base metals of different strengths are joined, the weld metal will generally be matched to the weaker base metal; the permissible weld stress, regardless of the weld classification used, should not exceed that specified for the weaker base metal.

Partial-penetration groove welds subjected to compression normal to the axis of the welds should also be designed for the same allowable stresses as the base metal (assuming that they are fabricated with the flat lands of the joint in contact). However, when subjected to tension normal to the axis of the welds, the stress should be based on the effective throat of the weld.* Welds such as groove or fillet welds that join the elements of built-up members and are stressed in a direction parallel to the axis of the welds (e.g., flange-to-web welds of girders) participate directly with the elements and should be designed without special regard to the tensile or compressive stress in these elements parallel to the axis of the welds.

Fillet Welds. The "minimum" shear resistance of fillet welds stressed in a direction parallel to the axis of such welds is a function of the tensile strength of the weld metal and can be taken as

$$R_n = CF_{EXX} \qquad \text{[Eq G1]}$$

where F_{EXX} = the minimum specified ultimate tensile strength of the weld metal (e.g., F_{EXX} = 60 ksi for E60XX electrodes)

C = a coefficient that relates the shear and tensile strengths of the weld metal. (A value of $C = 0.7$ is recommended on the basis of existing data.)

However, to obtain the mean shear resistance of the weld metal, the strength obtained from Eq G1 must be adjusted for the difference between the actual strength of the weld metal and its minimum specified tensile strength and for the difference expected between the actual and nominal area of the throat of a weld.

The results of an extensive study of fillet welds[24] (Table G1) indicated that the mean shear strength of 1/4 to 1/2 in. longitudinal fillet welds based on the measured area of the weld throat was

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* The effective throat of a partial-penetration groove weld should be taken as the depth of the groove or grooves.

[24] *AWS - AISC Fillet Weld Study-Longitudinal and Transverse Shear Tests* (Testing Engineers Inc., May 1968).

Table G1

Mean Properties of Weld Metals

Table G1(a)

Mean Shear Strength of Weld Metals

Electrode	Computed Mean Shear Strength (ksi): Based on Nominal Weld Size*	Computed Mean Shear Strength (ksi): Based on Actual Weld Area**	Coefficient of Variation of Values in Parentheses**
E60XX	58.7	56.1 (55.1)	0.14
E70XX	65.6	63.0 (65.9)	0.09
E80XX	72.1	69.1	---
E90XX	77.6	74.5 (76.0)	0.09
E100XX	82.6	79.1	---
E110XX	86.6	83.0 (86.2)	0.12

* Values computed from Eq G3 with $C = 0.70$, $\ell = 1.10$, $C_r = 0.95$, and $M_w = (1.65\text{-}0.0052F_{EXX})$

** Values in parentheses are the mean strengths obtained in the laboratory tests performed by Testing Engineers, Inc.

Table G1(b)

Mean Dimensions of Fillet Welds

Nominal Leg Size (in.)	Mean Measured Size (in.)	Coefficient of Variation
1/4	0.30	0.10
3/8	0.43	0.12
1/2	0.525	0.125

approximately 10 to 30 percent greater than the strength given by Eq G1 and can best be characterized by the following:

$$\text{mean shear strength} = CF_{EXX}(1.65 - 0.0052\ F_{EXX}) \quad \text{[Eq G2]}$$

with a COV of 10 percent. In addition, the actual leg size was observed to be approximately 10 percent greater than the nominal leg size with a coefficient of variation of 12 percent. This should also be taken into account in establishing the strength of fillet welds.

The fillet weld study also found[25] that the shear strength of a transverse fillet weld (load applied in a direction normal to the axis of the weld) is significantly greater than that indicated by Eq G2, apparently because of the combined state of stress that exists on such a weld. Based on the area of the measured throat of such welds, the strength was found to be approximately 50 percent greater than the nominal strength of the weld metal. Although this is considerably greater than the strength of the longitudinal welds, the use of a single level of strength (based on longitudinal welds) will greatly simplify design and compensate for the bending that may sometimes be introduced in transverse welds due to the eccentricity of the loading. It is therefore recommended that no distinction be made between transverse and longitudinal welds in design.

Once placed in the field, welds can be expected to lose some of their section as a result of corrosion. However, except in unusual circumstances, T/O structures will have relatively short lives and the loss due to corrosion can be expected to be very small. Nevertheless, providing an allowance of 5 percent reduction in weld area to compensate for this factor is recommended. On this basis, the mean ultimate shear strength of the throat of fillet welds based on the nominal weld size can be given by

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[25] *AWS - AISC Fillet Weld Study-Longitudinal and Transverse Shear Tests* (Testing Engineers Inc., May 1968).

$$\overline{R} = CF_{EXX}M_w \ell C_r \qquad \text{[Eq G3]}$$

where ℓ = factor to adjust for the actual leg size of a weld

M_w = factor to adjust for the ratio of the actual to the minimum specified weld metal strength

C = ratio of shear to tensile strength

C_r = factor to compensate for the effect of corrosion.

Recommended values for the coefficients in Eq G3 are $C = 0.70$; $\ell = 1.10$ (based on data of Table G1b); $C_r = 0.95$; and $M_w = 1.65 - 0.0052F_{EXX}$.

Rivets

Use of rivets is generally not recommended, but the values presented in this section should be used if rivets are required.

Rivets and bolts are used to transmit forces either in tension or shear (on the fasteners), or a *combination of tension and shear.* The strength of connections made with such fasteners is thus a function of the nature of the applied forces and the type and strength of fasteners and fastener elements (rivets, bolts, and nuts), or the strength of the base metal.

The minimum specified material requirements for fasteners are covered by a variety of ASTM specifications; Table G2 lists the principal applicable fastener specifications.

The A 502 specification provides the requirements for rivets and covers both low- and high-strength rivet materials. The various bolt specifications--A 307, A 325, A 449, and A 490--cover bolts (and the associated nuts and washers) of three strength levels.

Tension. On the basis of several studies of the basic tensile and

Table G2

ASTM Fastener Specifications for Structural Applications

Specification Designation	Description
A 502	Steel structural rivets. (Grade 1 corresponds to former A 141 rivet steel. Grade 2 corresponds to former A 195 high-strength rivet steel.)
A 307	Carbon steel externally and internally threaded standard fasteners (Grade A).
A 325	High-strength bolts for structural steel joints including suitable nuts and plain hardened washers.
A 449	Quenched and tempered steel bolts and studs (similar to A 325 - to be used for anchor bolts and special applications requiring high strength).
A 490	Quenched and tempered alloy steel bolts for structural steel joints.

shear strengths of various types of rivets,[26] it is noted that

(1) Driving a rivet increases the tensile strength of the rivet material approximately 5 percent.

(2) Since rivet holes are drilled or punched 1/16 in. larger than the nominal size of the rivets for which they are made, the area of a driven rivet will be greater than its nominal area. This increase in

[26] W. H. Munse and H. L. Cox, *The Static Strength of Rivets Subjected to Combined Tension and Shear*, Engineering Experiment Station Bulletin No. 437 (University of Illinois, 1956); C. R. Young and W. B. Dunbar, *Premissible Stresses on Rivets in Tension*, Bulletin No. 8 (University of Toronto, 1928); and M. W. Wilson and W. A. Oliver, *Tension Tests of Rivets*, Engineering Experiment Station Bulletin No. 210 (University of Illinois, 1930).

area ranges from approximately 11 to 21 percent for the usual structural size rivets. However, because of the likelihood of a slight mismatch of the holes in the members of assembled structural connections (more than one fastener in length), the full increase in rivet area produced during driving will not be realized. This fact must be considered in developing the allowable stress provisions for rivets in large connections.

Thus, since the selection of rivets in design is based on the nominal size of the fasteners, the expected minimum tensile strength will be 16 to 26 percent greater than the minimums required by ASTM specification A 502.

Item 5 of Table G3 summarizes the mean tensile strengths obtained from two grades of rivet materials. As these values indicate, manufacturers provide a further increase in strength to insure that all rivets meet the minimum requirements of the material specifications; in this case the mean tensile strengths of the undriven rivets were 20 and 7 percent greater than the minimum required tensile strength.

The mean tensile strengths of the two grades of driven rivets based on the nominal area of the fasteners are given in item 6 of Table G3 and can be obtained from the following:

$$\sigma_{ut} = M_R \cdot D_R \sigma_u \qquad \text{[Eq G4]}$$

where M_R = the material minimum strength factor

D_R = the driving effect on the rivets

σ_u = the minimum specified strength of the rivets.

Recommended values for M_R are 1.21 and 1.07 for Grade 1 and 2 rivets respectively. A value of 1.24 is recommended for D_R for both Grades 1 and 2.

Shear. The basic shear strength of structural rivets has also

Table G3

Ultimate Tensile and Shear Strengths of Rivets
Based on Nominal Area

	Type of Rivet: A 502 Grade 1	Type of Rivet: A 502 Grade 2
1. Specified minimum hardness	55 R_B	76 R_B
2. Specified maximum hardness	72 R_B	85 R_B
3. Approximate minimum tensile strength	47 ksi	67 ksi
4. Approximate maximum tensile strength	63 ksi	82 ksi
5. Mean tensile strength of rivet material (lab tests)*	57 ksi	72 ksi
6. Mean tensile strength of driven rivets (based on nominal area) (lab tests)* See Eq G4.	71 ksi	89 ksi
7. Coefficient of variation - (tension)	0.11	0.08
8. Mean shear strength of driven rivets (based on nominal area) (lab tests)* See Eq G5.	50 ksi	63 ksi
9. Coefficient of variation - (shear)	0.07	0.03

* Data obtained from rivets meeting earlier specifications. However, the information is considered to be representative for the A 502 materials.

been studied extensively.[27] Such studies have indicated that the shear strength will be approximately 70 percent of the tensile strength of the driven rivet (Table G3). Thus

$$\text{shear strength rivets } \sigma_{us} = 0.70\sigma_{ut} = C\sigma_{ut} \qquad \text{[Eq G5]}$$

where σ_{ut} = the ultimate tensile strength of the fastener.

Combined Stresses. For rivets subjected to tension and shear, an elliptical relationship has been provided to best fit the test data[28] (see Table G4):

$$\frac{\sigma_t^2}{1.0^2} + \frac{\sigma_s^2}{0.70^2} = (\sigma_{ut})^2 \qquad \text{[Eq G6]}$$

where σ_t = tensile component of stress (based on nominal area)

σ_s = *shear component of stress (based on nominal area).*

However, to simplify design, this relationship can be replaced by a series of three straight lines of the form,

$$\sigma_t = G - 1.6\sigma_s \leq \sigma_{ut} \qquad \text{[Eq G7]}$$

and

$$\max \sigma_s \leq \sigma_{us}$$

where G = an empirically selected constant. The approximate straight line relationships for the mean strength of A 502 rivets under combined tension and shear can be given by (Figure G1)

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[27] W. H. Munse and H. L. Cox, *The Static Strength of Rivets Subjected to Combined Tension and Shear*, Engineering Experiment Station Bulletin No. 437 (University of Illinois, 1956).

[28] Munse and Cox.

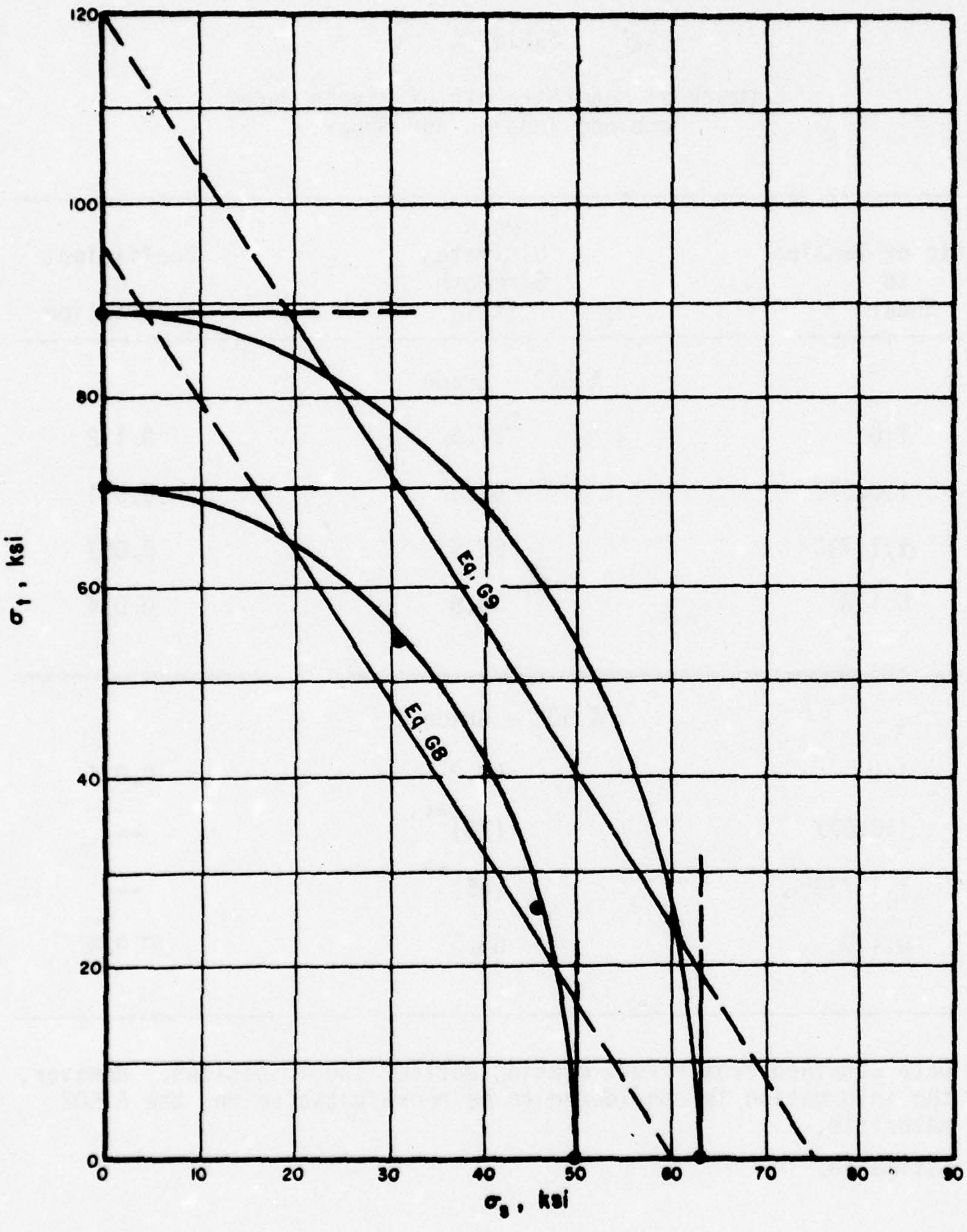

Figure G1. Relationship between rivet strengths and the elliptical and straight line models for resistance.

Table G4

Summary of Mean Strength of Rivets Under Combined Tension and Shear

Ratio of Tension to Shear	Mean Ultimate Strength* (ksi)	Coefficient of Variation
	A 502 - Grade 1	
1:0	71.5	0.112
1:0.577	62.3	0.071
1:1.733	52.5	0.067
0:1.0	49.8	0.074
	A 502 - Grade 2	
1:0	89.2	0.076
1:0.577	(78)**	---
1:1.733	(65)**	---
0:1.0	63.5	0.030

* Data obtained from rivets meeting earlier specifications. However, the information is considered to be representative for the A 502 materials.

** Estimated.

A 502 Grade 1 Rivets

$$\sigma_t = 96 - 1.6\sigma_s \leq 71 \text{ ksi} \qquad \text{[Eq G8]}$$

where $\sigma_s \leq 50$ ksi.

A 502 Grade 2 Rivets

$$\sigma_t = 120 - 1.6\sigma_s \leq 89 \text{ ksi} \qquad \text{[Eq G9]}$$

where $\sigma_s \leq 63$ ksi.

Bolts

Three types of bolts are generally used in bolted structures--those meeting the ASTM A 307, A 325, or A 490 specifications. The minimum specified tensile strengths for these fasteners are given in Table G5 along with the stress area* for bolt sizes ranging from 1/4 to 1 1/2 in.

As in the case of rivets, bolts may be used to transmit forces either in tension, shear, or combinations of tension and shear. Studies of the basic tensile, shear, and combined tension-and-shear strengths of the various types of structural bolts have shown that the ultimate strength and mode of failure of the fasteners are functions of a variety of factors, all of which should be considered in deriving the allowable design stresses.[29]

Tension. Under tensile loading, bolts may fail (1) in tension

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* The stress area is calculated from the equation $A_s = 0.7854[d-(0.9743/n_t)]^2$, where d is the nominal bolt size and n_t the threads per inch.

[29] E. Chesson Jr., N. L. Faustino, and W. H. Munse, "High-Strength Bolts Subjected to Tension and Shear," *Journal of the Structural Division*, ASCE, Vol 91, No. ST5 (October 1965), pp 155-180; and J. J. Wallert and J. W. Fisher, "Shear Strength of High-Strength Bolts," *Journal of the Structural Division*, ASCE, Vol 91, No. ST3 (June 1965).

Table G5

ASTM Tensile Requirement for Bolts*

Bolt Size (in.), Threads per Inch, and Series	Stress Area** (sq in.)	Minimum Tensile Strength (lb)		
		A 307	A 325	A 490
1/4 - 20	0.0318	1,900	--	--
5/16 - 18	0.0524	3,100	--	--
3/8 - 16	0.0775	4,650	--	--
7/16 - 14	0.1063	6,350	--	--
1/2 - 13 UNC	0.1419	8,500	17,050	21,300
9/16 - 12	0.182	11,000	--	--
5/8 - 11 UNC	0.226	13,550	27,100	33,900
3/4 - 10 UNC	0.334	20,050	40,100	50,100
7/8 - 9 UNC	0.462	27,700	55,450	69,300
1 - 8 UNC	0.606	36,350	72,700	90,900
1 1/8 - 7 UNC	0.763	45,800	80,100	114,450
1 1/8 - 8 UN	0.790	--	82,950	118,500
1 1/4 - 7 UNC	0.969	58,150	101,700	145,350
1 1/4 - 8 UN	1.000	--	105,000	150,000
1 3/8 - 6 UNC	1.155	69,300	121,300	173,250
1 3/8 - 8 UN	1.233	--	129,500	185,000
1 1/2 - 6 UNC	1.405	84,300	147,500	210,750
1 1/2 - 8 UN	1.492	--	156,700	223,800

* From *Annual Book of ASTM Standards, Part 4 - Structural Steel* (American Society for Testing and Materials, 1974).

** Stress area = $0.7854[d - 0.9743/n_t]^2$.

through the threads, (2) by stripping of the bolt threads, or (3) by stripping of threads in the nut. In addition, the tensile strength will be a function of the materials in the bolt and nut and of the geometry of these elements. The theoretical tensile strength of the components of a bolt assembly can be represented by three relationships; the actual tensile strength of the assembly will then be given by the minimum of the three relationships.

$$\text{Minimum bolt breaking tensile load} = \sigma_B \times A_s \qquad \text{[Eq G10]}$$

$$\text{Minimum bolt stripping load} = 0.73\sigma_B \times C_1 C_b \qquad \text{[Eq G11]}$$

$$\text{Minimum nut stripping load} = 0.73\sigma_N \times C_1 C_n \qquad \text{[Eq G12]}$$

where A_B = the nominal area of the bolt

A_s = the stress area of the threaded portion of the bolt

A_s = $0.7854[d-(0.9743/n_t)]^2$

d = nominal bolt size

n_t = number of threads per inch

σ_B = minimum ultimate tensile strength of bolt material (see Table G6)

σ_N = minimum ultimate tensile strength of nut material

C_1 = nut parameter = $\frac{H}{2n_t}\sqrt{1 + \pi^2 E^2 n_t^2}$ [Eq G13]

C_b = material factor for bolt stripping (bolt stronger than nut) = $1 - 0.625\left(\frac{\sigma_B - \sigma_N}{\sigma_B + \sigma_N}\right)$ [Eq G14]

C_n = material factor for nut stripping = [Eq G15]

$$1 + 0.625\left(\frac{\sigma_B - \sigma_N}{\sigma_B + \sigma_N}\right)$$

H = height of nut.

The parameters controlling the strength of such fasteners become readily evident from these relationships. Using the minimum specified bolt strengths given in Tables G5 and G6, the minimum tensile requirements of Table G7 can be verified. However, it should be noted that

Table G6

Minimum Tensile Strength for Bolts

Diameter (in.)	Minimum Specified Tensile Strength, psi		
	A 307 - Grade A	A 325	A 490
1/4 to 7/16	60,000	--	--
1/2 to 1	60,000	120,000	150,000
1 1/8 to 1 1/2	60,000	105,000	150,000
1 1/2 to 4	60,000	--	--

the materials are generally produced to slightly exceed the minimum tensile requirements and that the number of threads in the grip of a bolt (distance between the head and face of nut) will also affect its strength. Both of these factors should be considered in establishing the mean strength of bolts.

Numerous tensile tests conducted on A 325 and A 490 bolts provide a mean tensile strength for A 325 bolts of 131 ksi (with coefficient of variation of 12 percent) and 160 ksi (with coefficient of variation of 3 percent) for A 490 bolts, based on the stress area of the bolts (see Table G7). Thus, the bolts have a mean tensile strength approximately 8 percent greater than the minimum specified strength, and M_B, the material minimum strength factor, is 1.08. This mean strength includes bolts from both the low and high sides of the specification strength requirements, various diameters, and various lengths and numbers of threads in the grip. Based on these values the mean tensile strength of the fasteners can then be given by

$$\sigma_{ut} = M_B \cdot \sigma_B \cdot A_s \qquad \text{[Eq G16]}$$

Table G7

Ultimate Tensile and Shear Strengths of Bolts
(1/2 to 1 in. diameter)

	Type of Bolt		
	A 307 - Grade A	A 325	A 490
1. Minimum specified tensile strength, lb	60,000 A_s	120,000 A_s	150,000 A_s (170,000 A_s = maximum)
2. Mean tensile strength, psi on stress area (tests)	66,000*	131,000	160,000
3. Coefficient of variation	0.12*	0.12	0.03
4. Minimum bolt shear strength (shear through shank), lb, based on specified min. tension	39,500 A_B	75,000 A_B	93,750 A_B
5. Minimum bolt shear strength (shear through threads), lb	39,500 A_s	75,000 A_s	93,750 A_s
6. Mean shear strength, psi on bolt shank (tests)	41,000*	82,000	106,000
7. Coefficient of variation	0.07*	0.073	0.065
8. Minimum specified unit strength of nut material, psi	54,000	71,000	120,000

* Estimated value.

Shear. The shear strength of a bolt through the shank of the fastener is given by

$$\text{bolt shear strength (shank)} = 0.625 M_B \sigma_B A_B = C M_B \sigma_B A_B \qquad \text{[Eq G17]}$$

The strength through the threads is given by

$$\text{bolt shear strength (threads)} = 0.625 M_B \sigma_B A_S = C M_B \sigma_B A_S \qquad \text{[Eq G18]}$$

These values are based on the results of tests conducted on both A 325 and A 490 bolts. The average ratio of shear strength to tensile strength (shear through the shank to the tension on the stress area) is reported to be 0.625 with a standard deviation of 0.033.[30] The resulting minimum bolt shear strengths are given in Table G7.

As in the case of the tensile strength, the mean shear strength of the fasteners can be expected to exceed the minimum computed strength by approximately 8 percent, and is taken into account by the factor M_B in Eq G16, G17, and G18.

Combined Stresses. Bolts subjected to combined tension and shear behave similarly to rivets. However, tensile failures in bolts are always through the threads of the bolts, while the shear failures may be either through the shanks or through the threads. This could result in a larger variation in strength when the shear component is significant. Nevertheless, extensive experimental data are available and provide the strengths summarized in Table G8.

Straight line relationships, such as those presented in Eq G8 and G9, have been established for bolts also, *based on the nominal area of the fasteners*. These mean strength relationships are as follows:

[30] J. W. Fisher and J.H.A. Struick, *Guide to Design Criteria for Bolted and Riveted Joints* (John Wiley and Sons, 1974), p 50.

Table G8

Mean Strength of A 325 Bolts Under Combined Loadings

Ratio of Tension to Shear	Ultimate Strength: Based on Nominal Area (ksi)	Ultimate Strength: Based on Stress Area (ksi)	Coefficient of Variation
1.0:0	(100.32)*	131.14	0.117
1.0:0.20	92.42	(120.8)	0.041
1.0:0.42	94.06	(122.9)	0.133
1.0:0.67	87.50	(114.4)	0.122
1.0:1.0	84.32	(110.2)	0.142
1.0:2.38	73.78	(96.4)	0.190
0:1.0	82.30 (shank)	(107.6)	0.073
	77.51 (threads)	(101.32)	0.132

* Values shown in parentheses are based on approximate value for the ratio between stress area and nominal area (0.765).

For A 307 $$\sigma_t = (90 - 1.9\sigma_s) \leq 50 \text{ ksi} \qquad \text{[Eq G19]}$$

$$\sigma_s \leq 41 \text{ ksi}$$

However, when threads are in the shear plane,

$$\sigma_t = (68 - 1.9\sigma_s) \leq 50 \text{ ksi} \qquad \text{[Eq G20]}$$

$$\sigma_s \leq 31 \text{ ksi}$$

For A 325 $$\sigma_t = (180 - 1.9\sigma_s) \leq 100 \text{ ksi} \qquad \text{[Eq G21]}$$

$$\sigma_s \leq 82 \text{ ksi}$$

However, when threads are in the shear plane,

$$\sigma_t = (139 - 1.9\sigma_s) \leq 100 \text{ ksi} \qquad \text{[Eq G22]}$$

$$\sigma_s \leq 63 \text{ ksi}$$

For A 490 $$\sigma_t = (233 - 1.9\sigma_s) \leq 122 \text{ ksi} \qquad \text{[Eq G23]}$$

$$\sigma_s \leq 106 \text{ ksi}$$

However, when threads are in the shear plane,

$$\sigma_t = (179 - 1.9\sigma_s) \leq 122 \text{ ksi} \qquad \text{[Eq G24]}$$

$$\sigma_s \leq 81 \text{ ksi}$$

Connections

Numerous combinations of fasteners and members can be assembled to produce structural connections that will resist shear, tension, flexural and torsional forces, either individually or in combination with one another. Developing resistance functions for every conceivable type of connection would be prohibitive. However, the resistance of all such connections can be established on the basis of the resistance of the individual components. This section discusses the resistance of welded, riveted, and bolted connections in terms of moment-resistant type connections, shear-type connections, and tension-type connections.

Moment-Resistant Connections. Various types of connections are used to provide moment capacity. The AISC specification[31] classifies three basic types of construction in terms of connection types:

1. Type 1 - A rigid frame (or continuous frame) is assumed to have sufficient rigidity to hold the original angles between intersection members virtually unchanged.

2. Type 2 - Simple framing (free-ended) is assumed to rotate freely under gravity loads. The ends of such beams and girders are connected for shear only.

3. Type 3 - Semirigid framing (partially restrained) is assumed to possess, in the connections of beams and girders, known and dependable moment capacity intermediate in degree between the rigidity of Type 1 and the flexibility of Type 2.

Rigid frame connections (Type 1) must be capable of developing the plastic-moment capacity of the members that frame into the connections and of resisting the shears and axial forces that correspond to the plastic moments noted. The connection details necessary to meet the above requirements are provided by the connection requirements of the AISC specifications. If the AISC requirements are used, it is only necessary to insure that the fasteners and connected materials can develop the strengths required to achieve full plastic action in the connected

[31] *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* (American Institute of Steel Construction, February 12, 1969).

members. This can be done by fulfilling the AISC requirements for restrained members and providing sufficient fastener strength to develop the strength of the materials being joined.

Welded Connections. Welded moment-resistant connections are assumed to have sufficient weld capacity to develop the full plastic strength of the connected members. Consequently, the welds--whether groove or fillet--must be proportioned to achieve this requirement, and the various elements must be designed to resist the forces indicated in Figure G2.

In addition, the connected members must have sufficient stiffness to essentially retain the original angle between the intersection members. Such requirements are provided in the current AISC specification and may require use of stiffeners or other details to insure the desired behavior.

Flexible welded beam-to-column connections are seldom used. However, if desired, such connections must be designed to provide members of the required flexibility and welds proportioned on the basis of their known resistance and the design loads.

Riveted or Bolted Connections. As in the case of welded connections, riveted or bolted moment-resistant connections must be designed with sufficient fastener strength to resist the forces shown in Figure G2. To resist these forces, the fasteners function in shear or in tension and their resistance is governed by the strengths reported in the earlier sections of this appendix.

The connected members, as in the case of welded connections, must be selected to provide the desired strength and rigidity for the Type 1 connections, or the desired strength and flexibility for the Type 2 and Type 3 connections.

Riveted and Bolted Connections - Tension Connections. The tensile strength of riveted or bolted connections is a function of such factors as the fastener strength, the strength of the connected materials, the type of connections (whether single-plane or double-

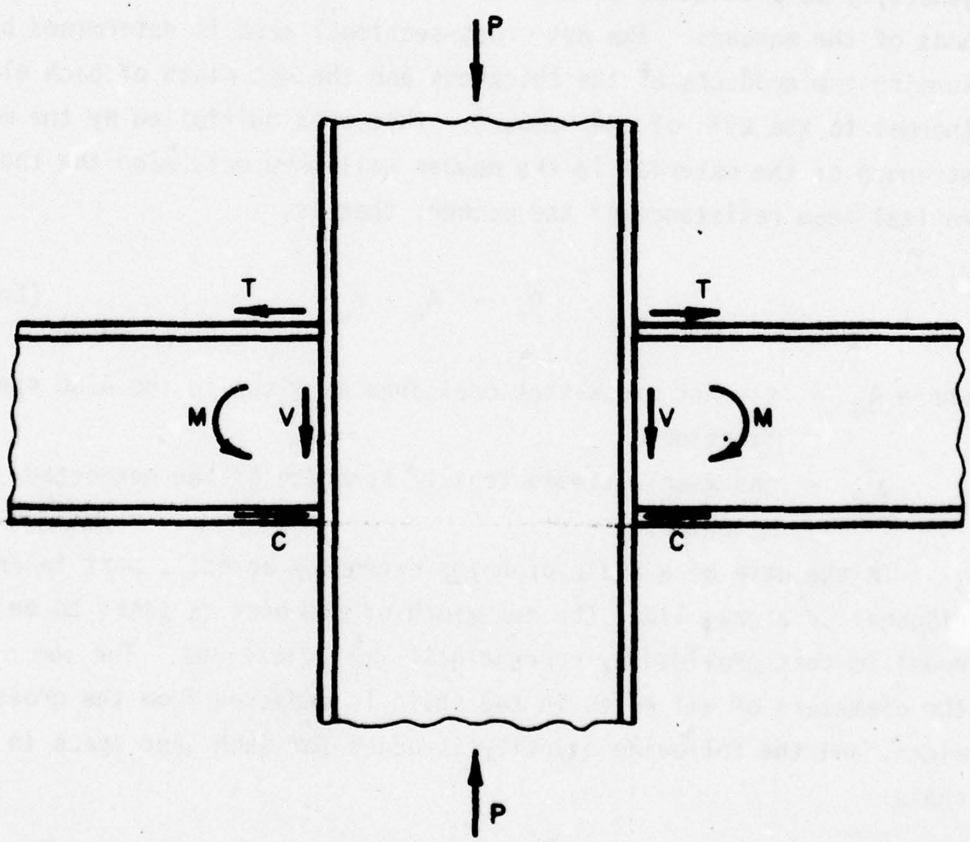

Figure G2. Load and forces on beam-to-column connections.

plane tensile connections), and the length of the connection , etc. Consequently, the design resistance of a connection must consider all of these various factors.

To develop the appropriate relationships for design resistance, the connections have been separated into (1) tensile bearing-type connections (net section tension, shear, and bearing), (2) friction-type connections, and (3) fasteners in tension (prying).

Tensile Strength - Net Section. For tension members of riveted or bolted construction, the ultimate strength of the member will generally be a function of the net section of the connections at the ends of the members. The net cross-sectional area is determined by summing the products of the thickness and the net width of each element (normal to the axis of the member). This area multiplied by the mean strength of the material in the member will then determine the theoretical mean resistance of the member; that is,

$$\bar{R}_n = A_n \cdot \bar{\sigma}_u \qquad \text{[Eq G25]}$$

where A_n = the net cross-sectional area as given in the AISC specification

$\bar{\sigma}_u$ = the mean ultimate tensile strength of the connected material.

In the case of a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part is taken to be equal to that provided by current AISC specifications. The sum of the diameters of all holes in the chain is deducted from the gross width, and the following quantity is added for each gage space in the chain:

$$\text{connection stagger} = s^2/4g \qquad \text{[Eq G26]}$$

where s = longitudinal spacing of any two consecutive holes (pitch)

g = transverse spacing of the same two holes.

This is the basic procedure long used for the design of tension members.

Extensive tests and analyses of the behavior of large structural connections[32] have shown that the efficiency* of a connection should also be expressed as a function of the gage and fastener diameter in the connection (a geometrical parameter) and the shear lag in the connection. To include these factors directly would greatly complicate design. However, by introducing the appropriate factors to Eq G25 and G26, these same relationships can be used to compute the resistance of the material in flat-plate-type connections as well as double-plane truss-type connections (see Figure G3). The mean resistance of flat-plate-type connections can be expressed in terms of the following relationship,

$$\bar{R} = 0.97\ A_n\bar{\sigma}_u = 0.97\ \bar{R}_n \qquad \text{[Eq G27]}$$

with a COV of the resistance equal to 10 percent, and the mean resistance of the members of double-plane truss-type connections in terms of the following relationship:

$$\bar{R} = 0.90\ A_n\bar{\sigma}_u \qquad \text{[Eq G28]}$$

with a COV of the resistance of 11 percent.

For design, expressing $\bar{R}$ in terms of the nominal yield stress F_y rather than $\bar{\sigma}_u$ is convenient. Table G9 shows the mean and nominal ultimate tensile strengths $\bar{\sigma}_u$ and σ_u respectively, based on typical results for A 36 and A 572, Grade 50 steels.** The $\bar{\sigma}_u$ has been adjusted to

[32] W. H. Munse and E. Chesson Jr., "Riveted and Bolted Joints: Net Section Design," *Journal of the Structural Division*, ASCE, Vol 89, No. ST1 (February 1963), pp 107-126.

* The efficiency (Figure G3), a measure of the ability of a connection to develop the strength of the connected material, is (net area x ultimate plate strength)/(gross area x ultimate plate strength).

** Private communication with J. England, U. S. Steel, 9 August 1973.

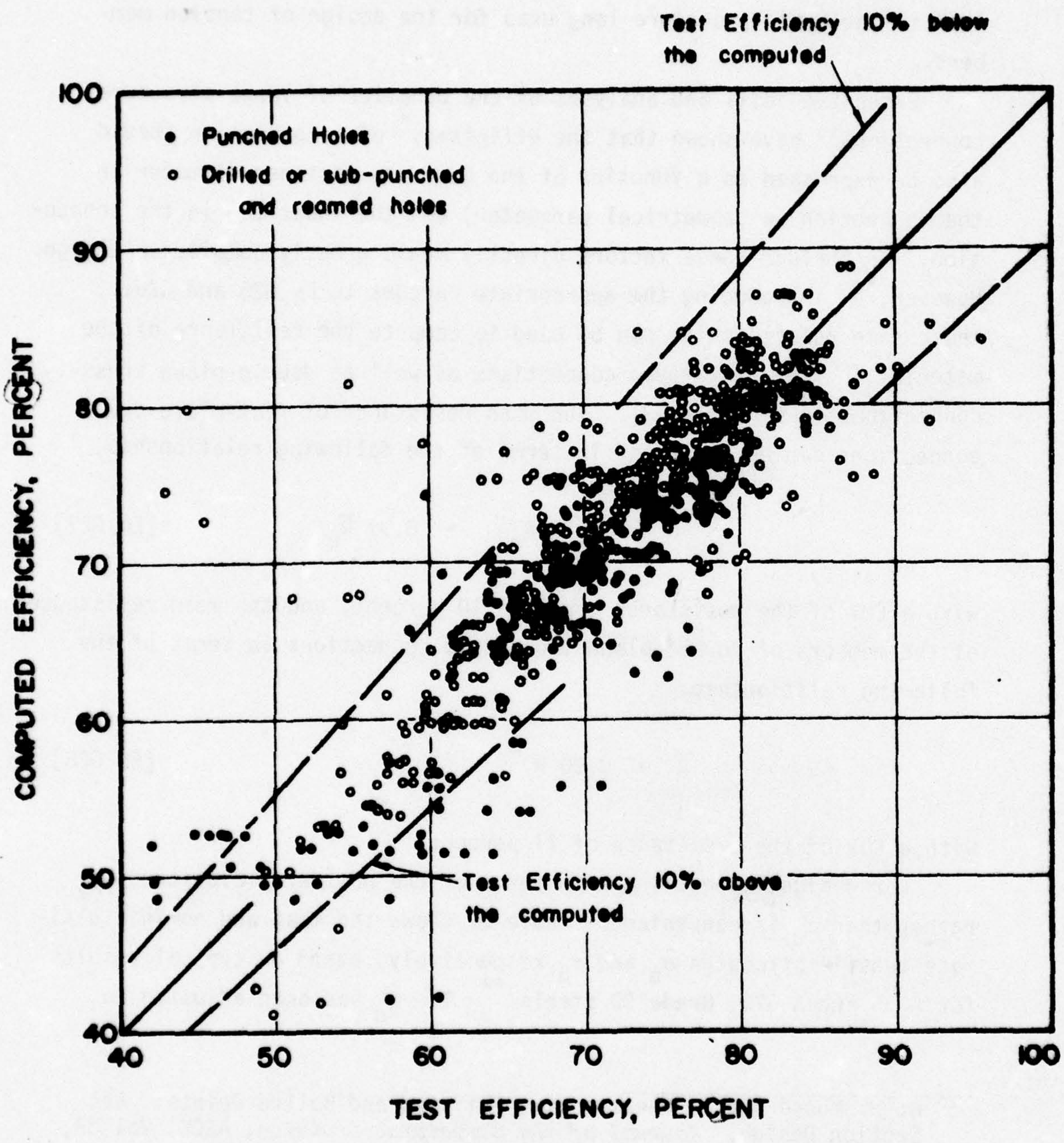

Figure G3. Correlation of theoretical and test efficiencies. Efficiency = (net area x ultimate plate strength)/(gross area x ultimate plate strength); i.e., a measure of loss of strength due to holes.

Table G9

Static Ultimate Tensile Strength Data

Steel Type	Nominal (Minimum) Yield Stress F_y (ksi)	Nominal (Minimum) Ultimate Tensile Strength σ_u (ksi)	Mean Static Ultimate Tensile Strength $\overline{\sigma}_u$ (ksi)	COV σ_u	$\overline{\sigma}_u/F_y$
A 36	36	58*	65**	0.09†	1.81
A 572	50	65*	80.4**	0.10†	1.61

* Based on ASTM specifications.

** Typical mill $\overline{\sigma}_u$ value reduced 2 ksi (e.g. $\overline{\sigma}_u$ for A 36: 67-2 = 65 ksi) to approximate mean static value. The mill distributions were obtained through private correspondence with J. England, U. S. Steel, 9 August 1973.

† Based on σ_u distributions, obtained through private correspondence with J. England, U. S. Steel, 9 August 1973.

approximate static strain rate values because the ultimate strength data before adjustment are based on higher strain rates than would be expected for static loading.

Equations G27 and G28 can be written as

$$\bar{R} = 0.97\ A_n(\bar{\sigma}_u/F_y)F_y \qquad \text{[Eq G29]}$$

$$\bar{R} = 0.90\ A_n(\bar{\sigma}_u/F_y)F_y \qquad \text{[Eq G30]}$$

Table G10 shows estimated values of $\bar{\sigma}_u/F_y$ for steels having F_y values ranging from 36 to 50 ksi. Since a large proportion of the steel corresponds to F_y = 36 and 50, a value of $\bar{\sigma}_u/F_y$ of 1.67 was considered as a representative mean value. Using $\bar{\sigma}_u/F_y$ = 1.67 in Eq G29 and G30 results in:

(flat-plate type connections) $$\bar{R} = 1.62\ A_nF_y \qquad \text{[Eq G31]}$$

(double-plane truss-type tension connections) $$\bar{R} = 1.50\ A_nF_y \qquad \text{[Eq G32]}$$

The mean resistances for connected material as given by Eq G31 and G32 are representative of steels having 36 ksi $\leq F_y \leq$ 50 ksi.

Shear Strength of Fasteners. The strength of fasteners in both flat-plate and truss-type connections must also be considered in design. When a connection has more than two fasteners in a line (in the direction of stressing), the shear strength of the fasteners (particularly for some connections with high-strength bolts) is found to decrease with an increase in the length of the connection and with the eccentricity introduced in double-plane truss-type members.

An analysis of existing data indicated that factors can be introduced to take length and eccentricity into account. To account for

Table G10

Information Used to Determine $\bar{\sigma}_u/F_y$ Ratio

Nominal Yield Strength* F_y (ksi)	Nominal (Minimum) Ultimate Tensile Strength* σ_u (ksi)	σ_u/F_y	$\bar{\sigma}_u$**$/\sigma_u$	$\bar{\sigma}_u/F_y$
36	58	1.61	1.12†	1.81
40	60	1.50	1.15††	1.73
42	60	1.43	1.15††	1.64
42	63	1.50	1.15††	1.73
45	60	1.33	1.15††	1.53
46	67	1.46	1.15††	1.68
50	65	1.30	1.24†	1.61
50	70	1.40	1.15††	1.61

* Values correspond to steels treated in ASTM specifications.
** $\bar{\sigma}_u$ is adjusted to correspond to a static strain rate.
† Based on available σ_u statistical distributions obtained through private correspondence with J. England, U.S. Steel, 9 August 1973.
†† Estimated value.

length, connections are separated into two groups: those less than or equal to 50 in. in length and those more than 50 in. long.

The mean shear resistance of fasteners in connections 50 in. or less in length can be based on the mean strength obtained from connection tests reported in the literature. Table G11 summarizes the mean resistances (either from tests or estimated) for rivets and bolts.

For connections longer than 50 in. which are generally found in relatively large structures, special analyses should be made to define the mean shear resistance of the fasteners.

A detailed examination of available data on the eccentricity introduced in double-plane truss-type connections indicates that the fasteners in double-plane connections have shear strengths 10 to 30 percent below that of a single rivet or bolt. A mean factor of 16 percent is recommended for all double-plane connections where actual data are not available (see Table G11).

Bearing Pressure. In various laboratory investigations, evaluations[33] have been made to determine the ultimate bearing strength that can be developed in riveted and bolted joints. Winter suggests that the ultimate bearing strength can be taken as equal to 4.9 times the yield strength of the steel in a member. Other tests reported in the literature,[34] however, indicate that for the lower strength structural

[33] W. H. Munse, *The Effect of Bearing Pressure on the Static Strength of Riveted Connections*, Engineering Experiment Station Bulletin No. 454 (University of Illinois, July 1959); John B. Kennedy and George R. Sinclair "Ultimate Capacity of Single Bolted Angle Connections," *Journal of the Structural Division*, ASCE, Vol 95, No. ST8 (August, 1969), pp 1645-1660; and George Winter, "Tests of Bolted Connections on Light Gage Steel," *Journal of the Structural Division*, ASCE, Vol 82, No. ST2 (March 1969).

[34] W. H. Munse, *The Effect of Bearing Pressure on the Static Strength of Riveted Connections*, Engineering Experiment Station Bulletin No. 454 (University of Illinois, July 1959); W. M. Wilson and W. H. Munse, *Tests of Riveted Joints with High Rivet Bearing*, Progress Report (University of Illinois, Dept. of Civil Eng., August 1, 1948); and John B. Kennedy and George R. Sinclair, "Ultimate Capacity of Single Bolted Angle Connections," *Journal of the Structural Division*, ASCE, Vol 95, No. ST8 (August 1969), pp 1645-1660.

Table G11

Mean Shear Resistance for Fasteners in Connections of Various Lengths and Types

Connection Type		Shear Resistance, for Various Fasteners (ksi)				
		A 502-Gr.1	A 502-Gr.2	A 307*	A 325*	A 490*
Single Fasteners	Resistance	49.80	63.00	41.00**	82.30	106.00
	COV	0.07	0.03	0.07**	0.07	0.07
Flat plate L < 50 in.	Resistance	49.70	58.00**	38.00**	69.50	90.00**
	COV	0.13	0.14**	0.14**	0.11	0.11**
Double-plane truss	Resistance	41.30	53.00**	34.00**	56.20	76.00**
	COV	0.06	0.06**	0.06**	0.05	0.06**

* Mean resistance in ksi based on nominal area when threads are excluded from the shear planes. Values should be multiplied by 0.765 when threads are not excluded from the shear planes.

** Recommended values -- no data available.

carbon steels, the lower values of bearing strength of a plate can be related to its ultimate tensile strength by the following:

$$\sigma_u^b \simeq 2.25\ \sigma_u \qquad \text{[Eq G33]}$$

where σ_u^b = ultimate bearing strength of a plate

σ_u = ultimate tensile strength.

Using the ratio of $\bar{\sigma}_u/F_y$ of 1.67 (as developed in Eq G33) results in

$$\sigma_u^b \simeq 3.75\ F_y \qquad \text{[Eq G34]}$$

This can be compared with the value of 4.9 F_y reported by Winter.

Since Eq G33 and G34 are based on lower values of ultimate bearing strength, they underestimate the mean ultimate bearing strength.

The current AISC specification provides for a maximum allowable tensile design stress of 0.60 F_y which, using a $\bar{\sigma}_u/F_y$ value of 1.67 corresponds to 0.36 $\bar{\sigma}_u$. Using the same factor of safety for the bearing stress results in an allowable bearing pressure of 0.36 x 2.25 σ_u = 0.81 σ_u. In terms of the yield strength, this corresponds to 1.35 F_y. For A 36 steel, this results in 48.6 ksi, which is almost identical to the value of 48.5 ksi currently specified in TM 5-744.[35]

It is estimated that the mean bearing strength is 4.35 F_y and the total uncertainty for bearing is about 12 percent (COV of resistance = 0.12).

Friction Connections - Bolted. In the AISC specification, bolted connections are classified as either friction or bearing type.

Friction-type connections are those which are subjected to stress reversals, severe stress fluctuations, or where slippage would be undesirable. For T/O structures, it is assumed that friction-type

[35] *Structural Steelwork*, TM 5-744 (Department of the Army, October 1969).

connections need be considered only under stress reversals, and then only under certain cases of reversal. The other factors noted such as slippage, etc., would rarely be important.

It is further assumed that bolts in T/O structures will be installed by the turn-of-nut procedure, since it is the procedure that can best be controlled and properly applied to obtain the necessary bolt tensions. However, when appropriate procedures are developed, torque wrenches, calibrated wrenches, or load-indicating washers can be used to insure the minimum required bolt tensions indicated in Table G12.

Bolts which are installed by the turn-of-nut method (snug plus one-half turn) will generally have an initial tension that substantially exceeds the required minimum tension of the current specification.[36] The initial tension in ASTM A 325 bolts is reported to be approximately 120 percent of the required minimum with a coefficient of variation of 9.1 percent.[37] In A 490 bolts the initial tension can be expected to be approximately 126 percent of the required minimum tension.[38] This provides mean clamping forces in the bolt equal to the values shown in Table G12.

Prying - Tension Connections. In some instances, the fasteners of bolted connections are subjected to direct tensile loadings. For example, tee-stubs are frequently used in this manner. However, because the deformation of the connected parts in such connections can produce an increase in the tensile load on the fasteners as a result of a prying action,[39] current specifications require that the design load be the

[36] *Structural Joints Using ASTM A 325 or A 490 Bolts* (Research Council on Riveted and Bolted Structural Joints, May 8, 1974).

[37] J. W. Fisher and J.H.A. Struick, *Guide to Design Criteria for Bolted and Riveted Joints* (John Wiley and Sons, 1974).

[38] J. W. Fisher and J.H.A. Struick.

[39] W. H. Munse, "Bolted Connections - Research," *Transactions of the ASCE*, Vol 121 (1956), pp 1255-1266.

Table G12

Mean Clamping Forces in Bolts Installed by the Turn-of-Nut Procedure (snug plus one-half turn)

Bolt Size (in.)	Minimum Specified Tension (kips)*		Mean Bolt Tension (kips)†	
	A 325	A 490	A 325	A 490
1/2	12	15	14.4	18.9
5/8	19	24	22.8	30.2
3/4	28	35	33.6	44.1
7/8	39	49	46.8	61.7
1	51	64	61.2	80.6
1 1/8	56	80	67.2	100.8
1 1/4	71	102	85.2	128.5

† Coefficient of variation - approximately 9 percent.

* From *Structural Joints Using ASTM A 325 or A 490 Bolts* (Research Council on Riveted and Bolted Structural Joints, May 8, 1974).

sum of the external load and any tension resulting from prying action produced by deformation of the connected parts. The AISC *Manual of Steel Construction*[40] and Nair et al.[41] provide design guidance for prying tension connections.

Summary

Tables G13 and G14 summarize the resistances for fasteners and connections.

The mean resistance functions developed in the preceding sections and their equation numbers can be summarized as follows:

Welds (Matching or slightly over-matching base metals.)

1. *Butt welds-full penetration.* Same as base metal.
2. *Butt welds-partial penetration.* Same as base metal.
3. *Fillet welds-shear resistance*

$$\bar{R} = CF_{EXX} \cdot M_W \cdot \ell \cdot C_r \qquad \text{[Eq G3]}$$

$$= (1.20 - 0.0038F_{EXX})F_{EXX}$$

Rivets

1. *Tension*

$$\sigma_{ut} = M_R \cdot D_R \cdot \sigma_u \qquad \text{[Eq G4]}$$

2. *Shear*

$$\sigma_{us} = C \cdot \sigma_{ut} = 0.70\sigma_{ut} \qquad \text{[EQ G5]}$$

[40] *Manual of Steel Construction*, 7th Ed. (American Institute of Steel Construction, 1970).

[41] R. S. Nair, P. C. Birkemoe, and W. H. Munse, "High Strength Bolts Subject to Tension and Prying," *Journal of the Structural Division*, ASCE, Vol 100, No. ST2 (February 1974), pp 351-372.

Table G13

Summary of Mean Resistances and Coefficients of Variation

Resistance Type*	Mean Ultimate Strength (ksi)** f_m	Coefficient of Variation V_R		Identification Number (See Table B2)	
Bolts					
A 307					
Tension	50.0	0.12		15	
S,S,PL	38.0	0.12		17	
S,T,PL	29.1	0.12		11	
S,S,TR	34.0	0.12		14	
S,T,TR	26.0	0.12		4	
A 325					
Tension	100.3	0.12		6	
S,S,PL	69.5	0.12		18	
S,T,PL	53.2	0.12		19	
S,S,TR	56.2	0.12		7	
S,T,TR	43.0	0.12		9	
A 490					
Tension	122.0	0.12		1	
S,S,PL	90.0	0.12		8	
S,T,PL	68.8	0.12		13	
S,S,TR	76.0	0.12		2	
S,T,TR	58.1	0.12		3	
Shop Rivets					
A 502 Grade 1					
Tension	71.0	0.11		20	
S,PL	49.7	0.11		22	
S,TR	41.3	0.11		12	
A 502 Grade 2					
Tension	89.0	0.11		16	
S,PL	58.0	0.11		10	
S,TR	53.0	0.11		5	
Bearing	$4.35\ F_y^{\dagger}$	0.12		21	
Connected material††					
PL	$1.62F_y^{\dagger}$	0.14		24	
TR	$1.50F_y^{\dagger}$	0.14		23	
Welds (shear)		Shop	Field	Shop	Field
E60	58.7	0.18	0.27	30	36
E70	65.6	0.18	0.27	29	35
E80	72.1	0.18	0.27	28	34
E90	77.6	0.18	0.27	27	33
E100	82.6	0.18	0.27	26	32
E110	86.6	0.18	0.27	25	31

* Key
S = shear
S, T = shear, threads in shear plane
TR = double-plane, truss-type connection
S, S = shear, threads not in shear plane
PL = flat-plate type connections

** Based on nominal area.

† F_y = minimum specified yield stress.

†† Tension members.

Table G14

Summary of Mean Resistance and Coefficient of Variation for Bolts and Rivets Under Combined Tension and Shear

Resistance Type	Mean Ultimate Strength* (ksi)	Coefficient of Variation V_R
Bolts		
A 307		
Shank**	$\sigma_t^{\dagger} = 90 - 1.9\ \sigma_s^{\dagger\dagger} \leq 50$	0.12
	$\sigma_s \leq 41$	
Threads+	$\sigma_t = 68 - 1.9\ \sigma_s \leq 50$	0.12
	$\sigma_s \leq 31$	
A 325		
Shank**	$\sigma_t = 180 - 1.9\ \sigma_s \leq 100$	0.12
	$\sigma_s \leq 82$	
Threads+	$\sigma_t = 139 - 1.9\ \sigma_s \leq 100$	0.12
	$\sigma_s \leq 63$	
A 490		
Shank**	$\sigma_t = 233 - 1.9\ \sigma_s \leq 122$	0.12
	$\sigma_s \leq 106$	
Threads+	$\sigma_t = 179 - 1.9\ \sigma_s \leq 122$	0.12
	$\sigma_s \leq 81$	
Shop Rivets		
A 502 Grade 1	$\sigma_t = 96 - 1.6\ \sigma_s \leq 71$	0.11
	$\sigma_s \leq 50$	
A 502 Grade 2	$\sigma_t = 120 - 1.6\ \sigma_s \leq 89$	0.11
	$\sigma_s \leq 63$	

* Based on nominal area.
** Threads excluded from shear plane.
† σ_t = mean ultimate tensile stress in the presence of the shear stress, σ_s.
†† σ_s = shear stress on fastener.
\+ Threads not excluded from shear plane.

3. Combined Stresses

$$\sigma_t = G - 1.6\sigma_s \le \sigma_{ut}$$ [Eq G7]

$$\sigma_s \le \sigma_{us}$$

Bolts

1. Tension

$$\sigma_{ut} = M_B \cdot \sigma_B \cdot A_s = 1.08\sigma_B A_s$$ [Eq G16]

2. Shear (shank)

$$\sigma_{uss} = C \cdot M_B \cdot \sigma_B \cdot A_B = 0.675\sigma_B A_B$$ [Eq G17]

3. Shear (threads)

$$\sigma_{ust} = C \cdot M_B \cdot \sigma_B \cdot A_B = 0.675\sigma_B A_s$$ [Eq G18]

4. Combined Stresses (shank)

$$\sigma_t = G_s - 1.9\sigma_s \le \sigma_{ut}$$ [Eq G19, G21, G23]

$$\sigma_s \le \sigma_{uss}$$

5. Combined Stresses (threads)

$$\sigma_t = G_t - 1.9\sigma_s \le \sigma_{ut}$$ [Eq G20, G22, G24]

$$\sigma_s \le \sigma_{ust}$$

Connections

1. Tension-Flat Plates

$$\overline{R} = 0.97A_n\overline{\sigma}_u \qquad \text{[Eq G27]}$$

2. Tension Double-Plane Truss-Type Connection

$$\overline{R} = 0.90A_n\overline{\sigma}_u \qquad \text{[Eq G28]}$$

3. Bearing Pressure

$$\sigma_u^b = 4.35F_y$$

Analysis of Resistance Uncertainty

Shop Welds

The COVs in weld strength for various electrodes varied from 9 to 14 percent (Table G1(a)), with an average value of approximately 12 percent. The quality of fabrication, however, may introduce additional uncertainty in the material strength of a given weld. It is realistic to assume that the actual mean strength (for shop welds) resulting from the varying quality of fabrication could vary by as much as 10 percent from the laboratory measured mean strength, yielding an additional uncertainity of about 6 percent; thus, the total uncertainty of the weld material is

$$V_1 = \sqrt{.12^2 + .06^2} = 0.13 \qquad \text{[Eq G34]}$$

Table G1(b) indicates that the dimensions of a weld vary with an average coefficient of variation which is also around 12 percent. However, in this case there is probably no further uncertainty, and the uncertainty in the weld size can therefore be taken as *12 percent*.

Once placed in the field, welds can be expected to lose some of their section as a result of corrosion. However, except in unusual circumstances T/O structures will have relatively short lives, and the loss due to corrosion can be expected to be small. Accordingly, an allowance of 5 percent uncertainty should be sufficient to cover this factor.

Thus, the total uncertainty in the estimated strength of a weld is

$$V_R = \sqrt{0.13^2 + 0.12^2 + 0.05^2} = 0.18 \qquad \text{[Eq G35]}$$

Field Welds

The uncertainties in field welds can be expected to be considerably higher than those of shop welds. It is probably reasonable to assume that the uncertainty of the strength of field welds is uniformly 1.5 times that of shop welding. Accordingly, V_m equals 0.20. The variability of the weld size of field welds may similarly be assumed to be 1.5 times that of shop welds; thus, V_L equals 0.18, and the total uncertainty in the estimated strength of a field weld, including the effect of corrosion, is

$$V_R = \sqrt{0.20^2 + 0.18^2 + 0.05^2} = 0.27 \qquad \text{[Eq G36]}$$

Rivets

The strength of a rivet is a function of the actual area of the shank of the driven rivet and the strength of the rivet material. The variability of the tensile strength of rivets ranges from 8 to 11 percent, whereas the variability of the shear strength ranges from 3 to 7 percent (Table G3). On this basis, a coefficient of variation of 10 percent can be used; however, the results shown in Table G3 are for laboratory specimens. For rivets in actual structures, assuming an additional uncertainty of 5 percent for the (mean) rivet strength is probably not unreasonable. Thus, the total uncertainty in the strength (tension or shear) of rivets is

$$V_R = \sqrt{0.10^2 + .05^2} = 0.11 \qquad \text{[Eq G37]}$$

Since rivets and bolts are enclosed when placed in a structure, they are not susceptible to corrosion, as are welds, and need not be adjusted for this factor.

Under a combination of tension and shear, the uncertainty, as shown in Table G4, is approximately the same as that noted above. Therefore, the same coefficient should be used for all types of loadings on rivets.

Bolts

The variability of the tensile and shear strength of laboratory-tested bolts (A 307, A 325, and A 490 bolts in Table G7) ranges between 3 and 12 percent. Since it is believed that bolt specimens prepared in the laboratory are comparable in quality and workmanship to those used in actual structures, the uncertainty in the strength of a bolt (in tension, shear, or any combination thereof) can be taken conservatively to be 12 percent.

Tensile Strength - Net Section (Connected Material)

As previously discussed, the COVs of the strength of the connected material for plate and truss-type connections are 0.10 and 0.11, respectively. The total uncertainty, however, must include the variability in the ultimate tensile strength σ_u, which is about 0.09 (Table G9). In addition, the uncertainty in the $\bar{\sigma}_u/F_y$ ratio of Table G9, with a range of 1.53 to 1.81, is estimated at 0.05. Therefore, an estimate of the total uncertainty V_R is

$$V_R = \sqrt{.10^2 + .09^2 + .05^2} = 0.14 \qquad \text{[Eq G38]}$$

Bearing

The total uncertainty in bearing ultimate strength is estimated at about 0.12.

Summary

Table G13 summarizes the values of the uncertainty for the fasteners and connections.

APPENDIX H:

DEVELOPMENT OF THE LATERAL LOAD DISTRIBUTION FACTORS AND FORMULAS FOR STEEL STRINGER MILITARY HIGHWAY BRIDGES

The distribution of wheel (or track) loads for design for flexure is one of the critical factors in the design of the floor system of military highway bridges. During the past 25 years, a number of criteria have been proposed for the determination of this distribution. Currently, the Department of the Army[42] uses the procedure which was outlined by Roberts in 1959.[43] These criteria were reviewed in light of test results,[44] recent research in the field of civilian highway bridges,[45] and current bridge types used by the military.[46] This appendix briefly summarizes the findings in each of these areas and makes recommendations on the adequacy of current military lateral load distribution formulas for designing beams for flexure, including possible changes which will improve their validity.

Study Program

The types of bridges currently being designed and constructed by the military were reviewed; the main reference for this review was TM 5-302, supplemented by designs recommended for use in Vietnam.[47] The TM 5-302 bridges are all timber deck construction, while the more permanent Vietnam bridges have concrete decks. Although the timber

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[42] *Military Fixed Bridges*, TM 5-312 (Department of the Army, December 1968) (with changes 1 and 2).

[43] N. P. Roberts, *Load Distribution Effects in Bridge Deck Systems*, Unpublished Report (August 1959).

[44] *Load Distribution of Stringer Bridges*, Report on Project 8-67-01-400 (U.S. Army Engineer Research and Development Laboratories).

[45] W. W. Sanders and H. A. Elleby, *Distribution of Wheel Loads on Highway Bridges*, Report No. 83 (National Cooperative Highway Research Program, 1970).

[46] *Army Facilities Components System--Design*, TM 5-302 (Department of the Army, September 1973).

[47] *Standard Highway Bridges - RVN*, Drawing No. Q-3034 (Quinton-Budlong International Division, November 1970).

decks varied in type (nailed-laminated, plank, and multiple-layered), the deck was assumed solid (i.e., glued-laminated) for the purposes of this study. The effect of the actual type of construction is discussed later. Except for a few very short-span timber stringer bridges, all were designed using rolled steel W (old WF) sections for stringers.

Studies conducted for the National Cooperative Highway Research Program (NCHRP) have shown that the distribution in a bridge can best be related to a relative flexural stiffness parameter θ and a relative torsional stiffness parameter α.[48] Since α is usually very low and relatively constant (or an assumption of a low value which results in a conservative result) primary consideration was given to a study of θ:[49]

$$\theta = W/2L(\sqrt[4]{D_x/D_y}) \qquad \text{[Eq H1]}$$

where W = width of bridge floor (out to out)

L = span of bridge

W/L = aspect ratio

D_x = E_xI_x, flexural rigidity per unit width in x direction

D_y = E_yI_y, flexural rigidity per unit width in y direction

I_x, I_y = the moments of inertia in the x and y directions

E_x, E_y = the moduli of elasticity in the x and y directions.

A detailed review of the available bridge types (Table H1) showed that except for a few bridges, the value of θ ranged between 0.25 and 1.25. The exceptions were gererally bridges with aspect ratios greater than 1.0. A study of the table shows that for

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[48] W. W. Sanders and H. A. Elleby, *Distribution of Wheel Loads on Highway Bridges*, Report No. 83 (National Cooperative Highway Research Program, 1970).

[49] W. W. Sanders and H. A. Elleby.

Table H1

Range of θ^* for Typical Military Bridges

Type of Bridge	W_R** (ft)	L(ft)	Class	θ
Vietnam concrete deck/W stringers	24.6	34-64	AASHTO	0.60-0.78
	39.4	34-64	AASHTO	0.86-1.11
	24.6	35-40	AASHTO	0.68-0.72
	39.4	35-40	AASHTO	1.02-1.08
Timber deck/ 36WF stringers†	27.0	60-80	70	0.49-0.58
Timber deck/ I beam stringers††				
27WF	24.0	39-79	20-60	0.40-0.80
36WF	24.0	60-119	20-60	0.35-0.62
51DPG	24.0	99-130	40-60	0.35-0.41
Timber deck/ I beam stringers††				
27WF	13.5	42-79	20-60	0.23-0.47
36WF	13.5	68-119	20-60	0.22-0.36
51DPG	13.5	130	60	0.21
Timber deck/ WF stringers††	13.5	30-60	50	0.29-0.50
Timber deck/ DPG stringers††	24.0	20-30	60	1.12-1.57+
Timber deck/ WF stringers††	24.0	19-99	25(two-way) 50(controlled)	0.43-1.12+
	13.5	19-129	50(posted)	0.20-0.89+
1959 Fort Belvoir test bridges++	12-22	24-39	-----	0.66-0.83
	24.0	15	-----	2.15
Typical class 100-150 bridges (designed by WWS/HAE)	27.0	60-100	100	0.55-0.77
	29.0	60-100	150	0.62-0.81

* $\theta = W/2L\ (\sqrt[4]{D_x/D_y})$ (Diaphragms assumed 25 percent effective)

** W_R = width of roadway ($W = W_R + 2$)

† From TM 5-312.

†† From TM 5-302.

\+ High value for shorter spans.

++ From *Load Distribution of Stringer Bridges*, Report on Project 8-67-01-400 (U.S. Army Engineer Research and Development Laboratories).

single-lane bridges: $\theta = 0.20 - 0.80$ [Eq H2]

double-lane bridges: $\theta = 0.35 - 1.15$ [Eq H3]

The analytical procedures developed for the NCHRP study by Sanders and Elleby were used to study a broad spectrum of bridges within the above ranges of θ. These procedures are based on the use of orthotropic plate theory. The validity of the theory to predict behavior of a broad spectrum of civilian bridges was checked during the NCHRP study. However, before the procedures were used for military bridges, the validity was also checked with results of available field tests of typical military bridges.[50] The results of this comparison (Table H2) indicate that the procedures are also applicable to military bridges if an average value of $\alpha = 0.16$ (torsional constant) is used.

Using the procedures and the results of the NCHRP analysis, bridges typifying the actual bridges studied were analyzed. The major analyses were made for Class 60 vehicles, although a limited study was made for Class 80 to 90 vehicles. Table H3 gives the results.

Each single-lane bridge was analyzed for the vehicle being placed fully eccentric on the roadway (E1) and for the vehicle being centered on the roadway (C1) (hereafter referred to as the "Caution Crossing Position Case") as shown in Figure H1. All bridges were analyzed with the minimum width roadway permitted in TM 5-312. Wider bridges, although resulting in a slightly higher θ, would have a similar behavior because of the requirement of additional stringers.

Each double-lane bridge was analyzed for three conditions (Figure H1): one lane eccentric (E1), two lanes centered (C2), and one lane centered (C1). Two lanes eccentric is not possible for minimum

[50] *Load Distribution of Stringer Bridges*, Report on Project 8-67-01-400 (U.S. Army Engineer Research and Development Laboratories).

Table H2

Comparison of Effective Number of Stringers, N_E, from Field Tests* and Theories

Bridge	Field	NCHRP Theory** (α = 0.16)	Tripartite*	Engr. School*	Current Military†	AASHTO
No. 3* Center-to-center stringer spacing S = 2.21 ft θ = .67	3.26	2.78	4.50	4.76	3.26	4.07 (D = 4.5)
No. 4* One-Way	2.49	2.41	3.00	2.28	1.99	1.98 (D = 5.0)
Two-Way S_s = 5.04 ft θ = .83	2.35	1.98	1.87	1.67	1.88	1.69 (D = 4.25)
No. 6* S_s = 2.00 ft θ = .71	4.07	3.67	4.88	5.52	3.50	4.50 (D = 4.5)
No. 7* S_s = 1.75 ft θ = .66	4.07	4.28	5.43	6.02	3.86	5.14 (D = 4.5)

* From *Load Distribution of Stringer Bridges*, Report on Project 8-67-01-400 (U.S. Army Engineer Research and Development Laboratories).

** As modified from W. W. Sanders and H. A. Elleby, *Distribution of Wheel Loads on Highway Bridges*, Report No. 83 (National Cooperative Highway Research Program, 1970).

† *Military Fixed Bridges*, TM 5-312 (Department of the Army, December 1968) (with changes 1 and 2).

Note: All bridges have timber decks with steel I-beam stringers.

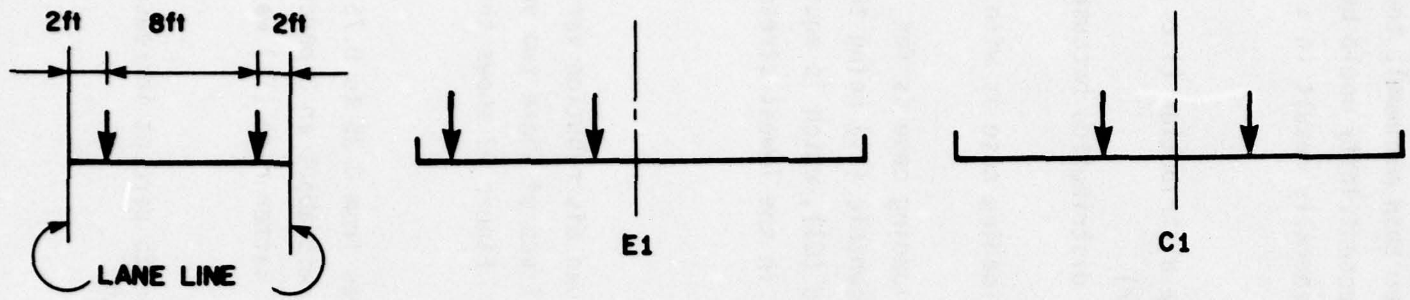
Table H3

Summary of Theoretical Distributions--N_E/N_s and Equivalent AASHTO "D" Values

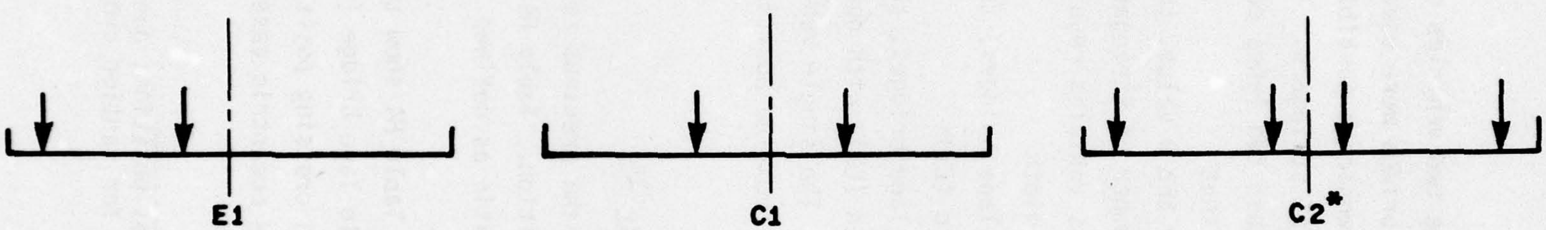
	Single-Lane				Double-Lane					
Bridge Class	60		80, 90		60			80, 90		
Load Case / θ	E1	C1	E1	C1	E1	C2	C1	E1	C2	C1
					N_E/N_s					
0.25	.895	.959			.622	.490	.927			
0.50	.772	.842	.779	.870	.528	.466	.803	.508	.463	.820
0.75	.672	.726			.484	.436	.675			
1.00	.587	.622			.445	.403	.584			
1.25	.509	.530			.414	.375	.525			
					Equivalent AASHTO "D"					
0.25	6.94	7.43			8.09	6.37	12.05			
0.50	5.98	6.53	6.62	7.40	6.87	6.06	10.44	7.36	6.71	11.89
0.75	5.21	5.62			6.29	5.66	8.77			
1.00	4.55	4.82			5.78	5.24	7.59			
1.25	3.95	4.11			5.39	4.88	6.82			

N_E = number of effective stringers (to carry one vehicle)
N_S = total number of stringers
$W = W_R + 2$

	W_R (ft)	
	Class 60	Class 80, 90
Single-lane	13.5	15.0
Double-lane	24.0	27.0



a. Single lane (W_R = 13.5 ft)



b. Double lane (W_R = 24 ft)

*C2 same as E2 at W_R = 24 ft
(minimum: TM 5-312)

Figure H1. Position of vehicles on roadway (Class 60 vehicle).

width bridges, as the two vehicles essentially cover the entire bridge. If a wider bridge were used (i.e., wider than minimum), then two lanes eccentric would be possible, but the eccentricity would be balanced by additional stringers. This would generally result in a less critical case than two lanes centered.

Table H3 shows that

1. The smaller the θ value, the better the distribution (i.e., N_E, the effective number of stringers, is larger)

2. Larger class vehicles result in better distribution because of the larger track width

3. For single-lane bridges, the critical loading case is with the vehicle eccentric (E1)

4. For double-lane bridges, the critical loading case is for two vehicles centered (C2), with one vehicle eccentric (E1) being the next most critical. The single vehicle centered (C1), which is equivalent to the caution crossing position,* results in the lowest stresses (highest N_E).

Discussion of Results

It is noted in the previous section that load distribution varies with θ and load position. Table H4 shows the effect of these two variables on the load ratio as defined in Table H4. Figure H2 shows these effects graphically.

The results in Table H4 show that

1. For a single-lane bridge (where θ ranges from 0.25 to 0.75, the C1 case (caution crossing position) will permit about an 8 percent higher load than the eccentric case (E1). This latter case (E1) was

* Related to lateral position; does not include 25 percent increase in vehicle class for caution crossing loading.

the one which controlled the development of the current military criteria for single-lane bridges:[51]

$$N_1 = \frac{5}{S_s} + 1 \qquad \text{[Eq H4]}$$

where N_1 = effective number of stringers for one-way classification

S_s = center-to-center stringer spacing in feet.

Table H4

Effect of Number of Vehicles and Vehicle Position (Class 60)

	Load Ratio*			
	Single-Lane Bridge	Double-Lane Bridge		
θ	C1/E1[a]	C1/E1[a]	E1/C2[a]	C1/C2[a]
0.25	1.07	1.49	1.27	1.89
0.50	1.09	1.52	1.13	1.72
0.75	1.08	1.39	1.11	1.55
1.00	1.06	1.31	1.10	1.45
1.25	1.04	1.27	1.10	1.40

* Ratio of load that can be carried with vehicle in position indicated by numerator of ratio (a) to that with vehicle in position indicated by denominator of ratio (a) with same maximum stress.

2. For a double-lane bridge, the two lanes centered case (C2) is critical with the single lane centered (C1) case, which simulates the caution crossing position, permitting a 40 to 89 percent higher load (C1/C2) (Table H4). If a single vehicle is placed in the most

[51] *Military Fixed Bridges*, TM 5-312 (Department of the Army, December 1968), with Changes 1 and 2; and N. P. Roberts, *Load Distribution Effects in Bridge Deck Systems*, Unpublished Report (August 1959).

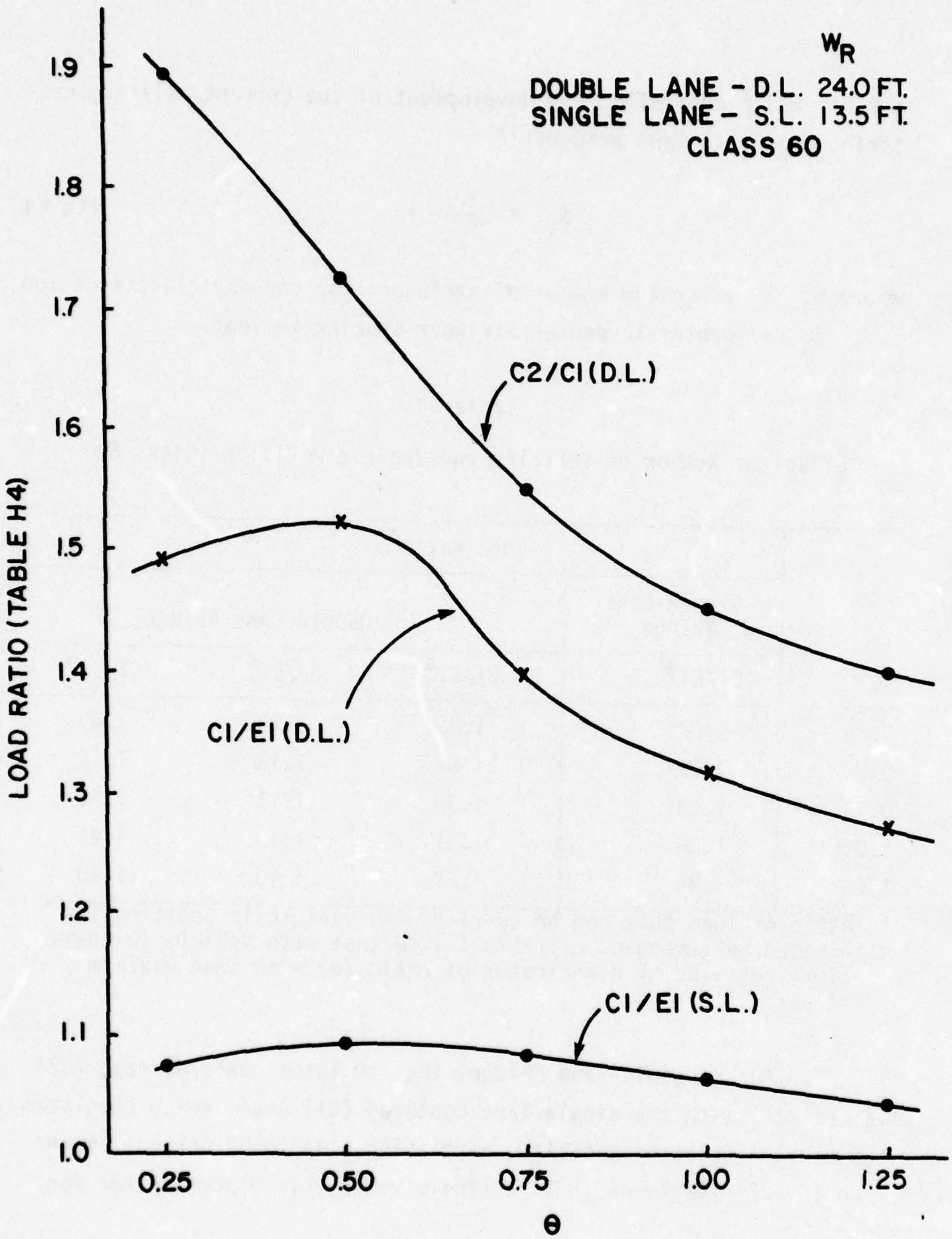


Figure H2. Range of load ratios.

critical position (E1), a 27 to 49 percent higher load can still be permitted without changing the maximum stress.

3. For double-lane bridges, the design case (N_1) which normally controls[52] in current military design is not the most critical.

Tables H5 and H6 show the relationships of the current military load distribution criteria to those specified for civilian bridges by AASHTO and to theoretical distribution factors developed earlier.

Table H5 shows the effective number of stringers N_E for TM 5-312 and AASHTO specifications, along with those for the various cases studied herein. The comparison is shown for a typical narrow stringer spacing S_s and for typical wide stringer spacings. It can be seen that for single-lane bridges the current military criteria are more conservative than AASHTO and are substantially more conservative than predicted from the theory (which has been verified by comparison with test results).

For double-lane bridges, the military criteria are more conservative than AASHTO at narrow S_s, but less conservative (compared to theory) at wide spacing. In any case, the current military criteria are the same or more conservative than any theory case considered. The conservatism ranges from slight for the extreme case (case C2, wide S_s - concrete deck, $\theta = 1.25$) to very conservative for the caution crossing position case (case C1, narrow S_s - timber deck, $\theta = .25$).

Table H5b also shows that N_2 is smaller than N_1 (for two-lane bridges) only for very large stringer spacings. Since the smaller of N_1 or N_2 is used for two-lane bridge design, N_2 is actually effective only when the stringer spacing exceeds about 7 ft; even then N_2 is only slightly less than N_1.

Since the purpose of this study is to evaluate the current

52 *Military Fixed Bridges*, TM 5-312 (Department of the Army, December 1968), with Changes 1 and 2.

Table H5

Relationship of Specification Capacity and Theory Capacity (Class 60) at Minimum Width--in Terms of Effective Number of Stringers

Table H5a

Single-Lane Bridge ($\theta = 0.25 - 0.75$; $\alpha = 0.16$)

Basis for N	Wide S_s	Narrow S_s
Stringer Spacing	$S_s = 4.50$ ft	$S_s = 2.70$ ft
Number of Stringers	$N_s = 4$	$N_s = 6$
Military N_1	2.11	2.85
AASHTO*	2.22	3.70
Theory (E1)		
Full range - θ	2.69-3.58	4.03-5.37
Most common - θ**	3.09	4.63
Theory (C1)		
Full range - θ	2.90-3.84	4.36-5.75
Most common - θ	3.37	5.05

* AASHTO values given for interior stringers; D = 5.0. Timber deck (strip 6 in. or more thick) assumed.

** Most common $\theta = 0.50$.

Table H5 (Continued)

Table H5b

Double-Lane Bridge (θ = 0.25 - 1.25; α = 0.16)

Basis for N	Wide S_s	Wide S_s	Narrow S_s
Stringer spacing	(concrete deck) S_s = 8.00 ft	(timber deck) S_s = 6.00 ft	(timber deck) S_s = 2.40 ft
Number of stringers	$N_s = 4^*$	$N_s = 5^{**}$	N_s = 11
Military N_1	1.63	1.83	3.08
Military N_2	1.50	$1.88^\dagger$	$4.13^\dagger$
AASHTO (one-lane) ††	1.75	2.00	4.17
AASHTO (two-lane)+	1.38	1.42	3.54
Theory (E1)			
Full range - θ	1.66-2.49	2.07-3.11	4.55-6.84
Most common - θ^{++}	2.11	2.64	5.81
Theory (C2)			
Full range - θ	1.50-1.96	1.88-2.45	4.13-5.39
Most common - θ	1.86	2.33	5.13
Theory (C1)			
Full range - θ	2.10-3.71	2.63-4.64	5.78-10.20
Most common - θ	3.21	4.02	8.83

* Minimum for concrete deck.
** Minimum for timber deck.
† Does not apply since $N_2 > N_1$.
†† D = 5.0 for timber deck; 7.0 for concrete deck.
\+ D = 4.25 for timber deck; 5.5 for concrete deck.
++ Most common θ = 0.50.

Table H6

Relationship of Stress (Maximum) from Specifications to that from Theory (Class 60)

Base Stress = 1.00 for Military Single Lane (N_1)

Maximum Stress				
Single-lane:		θ = 0.25-0.75		
Military:		1.00		
AASHTO:		0.77-0.95		
(E1) Theory:		0.53-0.78		
		(0.62-0.68)*		
(C1) Theory:		0.50-0.73		
		(0.56-0.63)		
Double-lane:		θ = 0.25-1.25		
Military:	One-lane:	1.00	Two-lane:	0.75-1.09**
AASHTO:	One-lane:	0.74-0.93	Two-lane:	0.87-1.29
(E1) Theory:		0.45-0.88		
		(0.53-0.69)†		
(C2) Theory:		0.57-0.97		
		(0.60-0.79)		
(C1) Theory:		0.30-0.70		
		(0.35-0.46)		

* Most common range expected (θ = 0.5, S_s = 2.70 ft to 4.50 ft).
** Does not apply if ratio less than 1 ($N_2 > N_1$).
† Most common range expected (θ = 0.5, S_s = 2.40 ft to 8.00 ft).

military lateral load distribution as compared to real behavior and other criteria, a more realistic comparison can be seen in Tables H6 and H7. Table H6 indicates the relative stress for various cases compared to a maximum relative stress of 1.0 for the current military (N_1) critical case. The reciprocal of this relationship is the increased load that can be carried at the same maximum allowable stress for all cases. Table H7 shows the relationship between N (the number of effective stringers) as predicted by theory and that predicted by current military criteria. It should be remembered that the theory values assume the deck to be solid (i.e., concrete or glued-laminated). Many timber decks (nailed-laminated, plank, etc.) will result in poorer distribution. This condition is discussed later.

The theoretically computed stress ratio (as shown in Table H6) in the critical girder for a single-lane bridge is at least 25 percent less than predicted. Furthermore, only a slight decrease in stress (slightly higher load) results from the caution crossing position case (C1) compared to the normal design case (E1).

For double-lane bridges, although single lane N (i.e., N_1) generally controls for design of beams for flexure, the critical case is actually two lanes loaded. It can be seen that the stress computed using the controlling military criteria is 5 to 40 percent higher (most likely at least 20 percent) than the critical stress computed by theory (for any loading). For the caution crossing position case (C1), the stress computed by theory is 30 to 70 percent below the military stress. Even if a caution class of 125 percent of base stress is used, the computed stress will still be less than that predicted by the military criteria.

Table H7 shows that for single-lane bridges, the caution loading case (defined in Table H7) is critical. Assuming that a 125 percent of normal one way class would be permitted in that case, the ratio of $N_{theory}/N_{military}$ ranges from 1.10 to 1.61 (i.e., the true class

Table H7

Relationship of Number of Effective Stringers from Theory to Current Military Criteria

N_L*	S_s	Range of N_θ/N_M**		
		General Case†	Central Loading††	Caution Loading+
1	Wide	1.27-1.70 (1.46)++	1.37-1.82 (1.60)	1.10-1.46 (1.28)
1	Narrow	1.41-1.88 (1.62)	1.53-2.02 (1.77)	1.22-1.61 (1.42)
≥2	Wide (concrete deck)	1.00-1.31 (1.24)	1.29-2.28 (1.97)	1.03-1.82 (1.58)
≥2	Wide (timber deck)	1.03-1.34 (1.27)	1.44-2.54 (2.20)	1.15-2.03 (1.76)
≥2	Narrow (timber deck)	1.34-1.75 (1.67)	1.88-3.31 (2.87)	1.50-2.65 (2.29)

* N_L = number of lanes.

** N_θ = number of effective stringers from theory; N_M = number of effective stringers from current military criteria; values of N obtained from Table H5.

† For single-lane bridges, case E1 controls; for double-lane bridges, case C2 controls (Figure H1).

†† One lane centered loading (C1) as used for caution crossing position case.

\+ 125 percent of N_M for caution crossing position case (C1) used to consider 125 percent of one-way class for caution loading.

++ The numbers in parentheses are the ratios for most common value of θ, which is about 0.50.

without actually exceeding the design stress is 110 to 161 percent of design class), with it normally expected to range from 1.28 to 1.42. This compares with a range in that ratio from 1.27 to 1.88 for the general loading case (normally from 1.46 to 1.62).

The table also shows that for double-lane bridges the general case is critical. In this case the ratio ranges from 1.00 to 1.75, with it most likely to fall between 1.24 and 1.67. For the caution loading case (defined in Table H7), the ratio of N_{theory} to $N_{military}$ ranges from 1.03 to 2.65 with the typical value being between 1.58 and 2.29.

Unusual Cases

When the width of bridges becomes relatively large with respect to the span (i.e., aspect ratio, W/L, greater than 1), the values of θ approach the upper part of the range of the θ values considered (0.25 to 1.25). In fact, it is possible to have θ values higher than 1.25. A review of Tables H5 and H7 indicates that at these higher values of θ the current military criteria for load distribution are, for the critical loading cases, reasonably accurate in predicting behavior. It is possible, however, for extreme cases (a short-span double-lane bridge), that the current military criteria may actually be unconservative when compared to theory.

Bridges with large aspect ratios are generally short double-lane bridges. It is not likely that large aspect ratios will occur often in single-lane steel stringer bridges.

Nailed-Laminated Decks

In all studies outlined previously in this appendix, the deck was assumed to be solid (i.e., concrete or glued-laminated). However, it is expected that occasions will arise where decks which are not "solid" will be used in the T/O. These decks will generally be of the nailed-laminated type. Research has shown that the nailed-laminated deck is initially not as efficient in distributing load as the "solid" glued-laminated deck. Furthermore, with time, the nails tend to work loose, further causing a reduction in distribution capabilities due to loss of torsional rigidity.

A review of field tests of military bridges with timber decks (Table H2) indicated that an α value (torsional constant) of 0.16 is realistic for a timber deck bridge. This value of α was determined from studies of bridges with narrow stringer spacings. It is expected that wider spacings may be used, possibly resulting in lower α values. If this rigidity is reduced to zero (no torsional rigidity), an indication of the effect of loss of laminating characteristics can be obtained. Using the results of the NCHRP study, it is estimated that a reduction in the effective number of stringers of 10 to 15 percent can result from the loss of the torsional rigidity. Thus, the average difference between "true class" and that computed by current military criteria (Table H7) will be reduced to about 110 to 140 percent with a typical value of about 120 percent.

Other Studies

This study did not deal with bridges with prestressed concrete stringers. However, a review of current AASHTO criteria and the results of the NCHRP study indicated that the behavior of prestressed concrete stringers (i.e., distribution of loads) should be comparable to the solid deck bridges with steel stringers studied in this investigation.

Although this study was directed toward class 60 through 90 bridges, the results should be applicable to both higher and lower class bridges. Although the lighter class vehicles have narrow wheel spacings (track), the θ values for bridges of these classes (20 through 40) are lower than expected for class 60 through 90. The lower values of θ have the highest conservatism.

For the heavier class bridges (up to Class 150), the θ values are still within the range studied and the wider track would assist in providing better load distribution. Thus, it is felt this study shows trends which should be applicable for all classes of vehicles.

The distribution of load indicated in this report was developed for static loadings, but should also apply equally well for vehicle impact as specified in the current military criteria.

General Summary

Although these results are based on the ranges of variables studied, it is felt that, within general limits, they will apply to most steel stringer military bridges. The following results apply to lateral load distribution criteria and specifications only; other criteria, such as allowable stress, are not considered. Unless otherwise stated, use of "criteria" or "specification" refers to the military requirements in TM 5-312.

1. For most military bridges the current criteria for the design of beams for flexure (moment) generally underestimate the load-carrying capacity of the bridge.

2. For typical single-lane solid deck* bridges, the true moment for the general loading case (Table H7) is at least 20 percent less than predicted by current criteria. The moment for the caution crossing position case** averages about 10 percent higher than that for the general loading case. For the caution crossing position case with a 25 percent increase above the normal crossing class, the true capacity is at least 10 percent higher than predicted by current criteria.

* Glued-laminated timber panel or concrete deck are considered solid.

** Unless stated otherwise, the caution crossing position case relates to lateral position and does not include a 25 percent increase in vehicle class for the caution crossing loading.

3. For typical double-lane solid deck bridges, the true moment capacity for the general case (Table H7) is generally from 25 to 65 percent higher than predicted by specification; however, for wide spacings this percentage can become very low. For typical bridges, the increase for the caution crossing position case with a 25 percent increase in class from the general case is at least 30 percent.

4. The current criteria seem to adequately predict the behavior for the bridges with high aspect ratios (i.e., $W^*/L > 1$). However, in cases where W/L is significantly greater than 1, the current criteria can actually become unconservative.

5. The AASHTO load distribution criteria do not appear to provide any significantly better indication of behavior than the current military load distribution criteria. However, both criteria appear to be conservative for most bridges.

6. For typical single-lane solid deck bridges, the caution crossing position case with 125 percent of normal one-way crossing is critical. For double-lane solid deck bridges, the general loading case (using current criteria for N_1 or N_2) is critical. For the critical case for single-lane solid deck bridges the ratio of N by theory to N by military specification ranges from 1.10 to 1.61 for the expected spectrum of bridges, with an expected value of 1.28 to 1.42. For typical double-lane solid deck bridges, the ratio for the critical condition ranges from 1.00 to 1.75 with the expected range to be from 1.24 to 1.67. In both the single- and double-lane cases a ratio of 1.35 is about the average value.

7. For both single- and double-lane bridges with nailed-laminated, planked, or multiple-layered decks, the ratio of N by theory to N by military specification ranges from about 1.10 to 1.40 with an

* W = width of bridge floor; L = span length.

Table H8a

Recommended Equations and the Current TM 5-312 Equations for Determining the Effective Number of Stringers

Current TM 5-312 Equations	Recommended Equations
Single-lane: $N_1 = \frac{5}{S_s} + 1$ (Eq 6-7a)	Single-lane: $N_1 = c[\frac{5}{S_s} + 1]$
Two-lane: $N_2 = \frac{3}{8} N_s$ or $N_2 = N_1$ (Eq 6-7b) whichever is smaller.	Two-lane: $N_2 = c[\frac{3}{8} N_s]$ or $N_2 = N_1$ whichever is smaller

Where N_1 = effective number of stringers for single-lane bridges

S_s = center-to-center stringer spacing in feet

N_s = number of stringers

N_2 = effective number of stringers for two-lane bridges

c = reduction factor given in Table H8b

Table H8b

Values of Reduction Factor, c,* Used in Recommended Formulas in Table H8a for Determining the Effective Number of Stringers

Bridge Deck Type	Ratio of Bridge Floor Width (out-to-out) to Bridge Span Length (W/L)	
	W/L ≤ 1.0	W/L > 1.0
Glued-laminated timber or concrete	1.0	0.75
Nailed-laminated timber, plank, or multiple-layered	0.90	0.70

* The factor c accounts for the reduction in lateral load distribution when using nailed-laminated timber, plank, or multiple-layered decks and/or bridges which are very wide compared to their span length.

average value of about 1.20 (compared to an average value of 1.35 for solid decks).

8. The conservatism of the current lateral load distribution criteria can be compensated by increasing the allowable stresses for flexure (moment) as developed in Appendices C and D. The effects of large W/L ratios and the use of decks which are not solid, such as nailed-laminated, plank, or multiple-layered decks, can be accounted for by reducing the effective number of stringers N, by a reduction factor c (i.e., cN), as given in Table H8b.

Recommendations

Based on the material presented herein, the following recommendations are made:

1. The current equations given for determining the effective number of stringers in Paragraph 6-5 of TM 5-312 should be replaced by the recommended equations given in Tables H8a and H8b. The

recommendations for the effective number of stringers should be combined with the allowable stress recommendations given in Appendix D.

2. A significant increase in class of vehicle permitted on the bridge may result in extremely high deck stresses. The designer should be cautioned to check the deck design.

APPENDIX I:

DEVELOPMENT OF SHEAR FORMULAS FOR MILITARY STEEL STRINGER HIGHWAY BRIDGES

Determination of the distribution of loads to the supporting elements in the floor system of military highway bridges is one of the major factors in designing these elements. The study described in Appendix H outlined a procedure for determining load distribution factors for moment in the stringers of steel beam bridges. Although moment is generally the critical factor in the design of these stringers, the shear is critical in some instances. In addition, the shears must be known to undertake the design of the stringer connections.

The current Army criterion for shear for steel stringer bridges (TM 5-312) simply states that one-half of the shear from a single vehicle shall be carried by each stringer. The study reviewed herein indicates the lack of suitability of this criterion and gives recommendations for new shear criteria.

General Discussion

The distribution of shear to the steel stringers depends significantly on the longitudinal, as well as the transverse, placement of loads on the bridge deck surface. The longitudinal placement of vehicles on the bridge which will maximize shear results in the vehicle being placed as close as possible to the reaction.

The transverse distribution of these loads is then affected by the flexibility of the entire floor as well as the transverse placement of the loads. As the loads are moved longitudinally away from the reactions, the floor tends to deform more, resulting in distributions which conform to those which are used for moment.

Since the beams do not deflect at the reactions, the loads are distributed laterally by the slab or deck behaving as if it were a series of simple beams supported by the stringers. This is based on the assumption that transverse continuity of the deck is minimal at the stringers. This approach to shear distribution is the same as that used in the current AASHTO specifications.

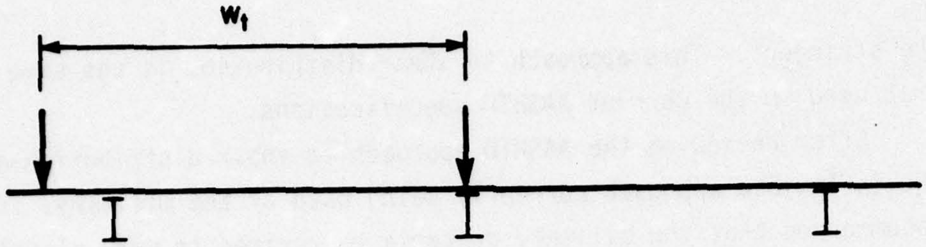
After reviewing the AASHTO approach to shear distribution and the inflexible approach currently being used by the military, it is recommended that the military criteria be revised to more closely follow those currently being used in civilian design. It should be remembered, however, that the vehicle types are different and direct use of the AASHTO criteria is not possible.

Shear Distribution

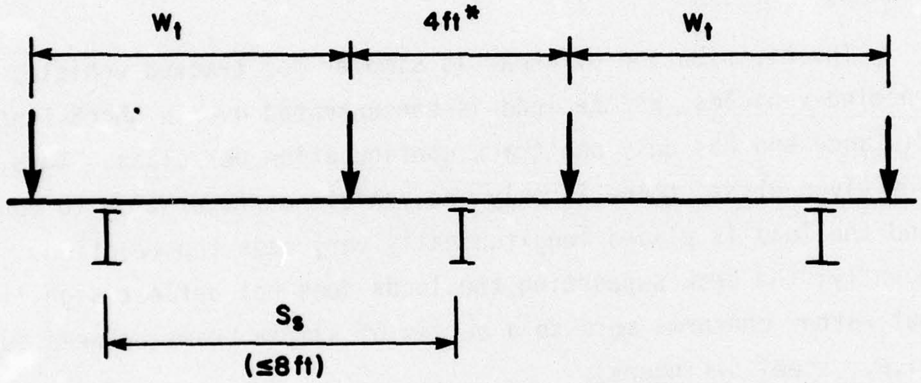
Tracked Vehicles

The distribution of shear is simpler for tracked vehicles than for wheeled vehicles, as the load is concentrated over a short longitudinal distance and has only one track configuration per class. Thus, for any given class, there is only one vehicle configuration to consider, and the load is placed longitudinally very near the reaction. Consequently, the deck supporting the loads does not deflect significantly, but rather conforms more to a series of simple beams between supports (i.e., steel stringers).

Figure I1, which shows the possible placement (transverse) of tracks, indicates that the critical situation for a single vehicle (Figure I1a) generally would be for one track (or one-half vehicle) to be directly over the stringer. Unless the stringer spacing exceeds the track spacing, the track load would be the only load to the stringer. The exception would occur only in cases with very wide stringer spacing ($S_s \geq 6$ ft) together with light class vehicles (class < 16). Even for the lightest class (4) and widest spacing (8 ft), the error in assuming only one track is 35 percent; this percentage decreases rapidly as either the class is increased or stringer spacing reduced. Thus, for a single-lane bridge (or a single vehicle on a double-lane bridge), the single track concept is suggested.



a. Single vehicle.



b. Multiple vehicles.

(W_t = track or wheel spacing - i.e., c. to c. distance between tracks.)

* Minimum distance between tracks assumed to be 4 ft.

Figure I1. Transverse load positions for tracked vehicles for reaction shear.

For multiple vehicle loadings, the most likely critical configuration is shown in Figure I1b. The track of each vehicle farthest from the "design" stringer affects the design only if the stringer spacing is greater than the track width (i.e., the farthest track is between adjacent stringer and design stringer). The design criteria (transverse distribution) are again based on simple beam loading neglecting this outside track. Even for the widest possible S_s (8 ft) and minimum class (4), the error is only about 25 percent. The error again decreases rapidly and the criterion shown (Figure I1b) is correct for all classes above 20, even at the widest stringer spacing.

For both single and multiple vehicles, the shear to be distributed for tracked vehicles is assumed to be equal to the reaction shear given in Appendix D of TM 5-312. Because tracked vehicle loads are considered to be short in length compared to the span length, when they are near the reactions (such as for shear), no transverse distribution due to deck distortion is assumed and all reaction shear is distributed by the simple beam concept.

Wheeled Vehicles

The shear distribution for wheeled vehicles is different than for tracked vehicles, since many of the wheels are longitudinally placed significantly farther out on the span. Thus, the deck has the opportunity to deform and it can be assumed that the transverse distribution of load for shear conforms substantially to that for moment (in this case, the wheels are assumed to be near midspan). This is a conservative assumption which will be adequate for design.

Thus, the suggested design criteria are based on a simple beam distribution for the axle over the reaction and on the distribution criteria for moment for axles out on the span proper. The appropriate loads to be distributed can be determined using Appendix D of TM 5-312. The shear given in that appendix is for the entire vehicle and the reaction shear (due to axle over the reaction) is simply the weight of the heaviest axle* with the shear to be "distributed" equaling the remainder.

In the case of wheeled vehicles, however, the transverse configuration of the wheels can take three forms (Figure I2) as given in Appendix D of TM 5-312, with a possible fourth form of a solid roller wheel. In the case of multiple vehicles, this results in the same type of critical transverse loading as tracked vehicles for the reaction axle (Figure I2), since type A will always provide a more critical simple beam loading than the other three types.

For single vehicles, however, types C and D can result in higher simple beam loads, because the distributed wheels can concentrate more loads over a centered stringer. The results of a study of the effect of the various types of wheel configurations are given in Table I1. The equation derived from this study is shown in the table.

As noted earlier, the axles out on the span are distributed on the same basis as moment.

Recommendations

Based on the previous discussion, it is recommended that the military criteria for distribution of loads for shear be revised to conform to the concepts used in the current AASHTO design specifications (Article 1.3.1A). Considering the different configurations possible for military vehicles, the following specific design criteria are recommended for incorporation into Paragraph 6-6 of TM 5-312 and should be combined with the allowable shear stress recommended in Appendix D:

* For Classes 40 and 50, the critical shear does not occur with the heaviest axle at the reaction. However, using the heaviest axle as the axle at the reaction will result in conservative estimates of the critical shear.

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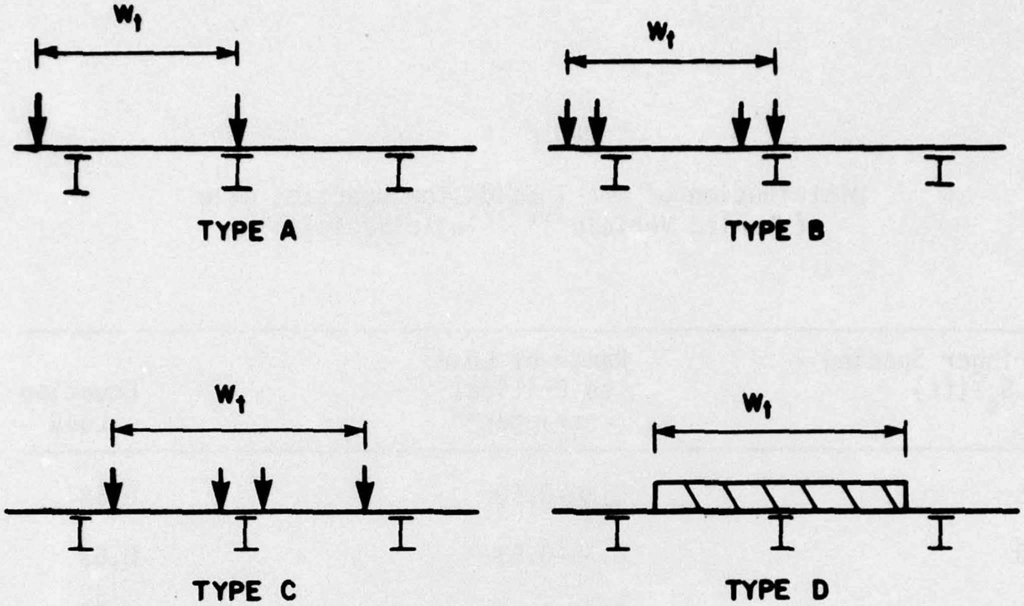
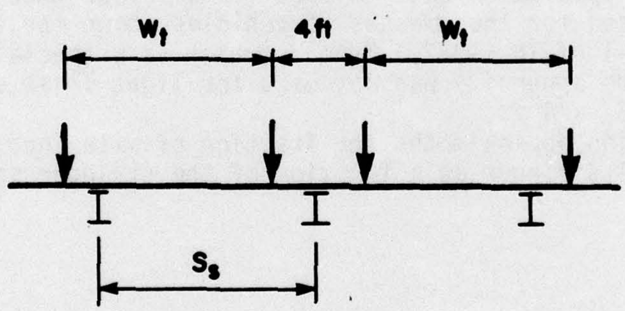


(Types A through C refer to Columns 5 through 7 of Appendix D-1 of TM 5-312)

a. Single vehicle.



b. Multiple vehicles.

Figure I2. Transverse load positions for wheeled vehicles for reaction shear.

Table I1

Distribution of Wheel Loads for Reaction Axle
(Wheeled Vehicles)* (Single Vehicles)

Stringer Spacing S_s (ft)	Range of Load to Critical Stringer**	Equation Load
2.5	0.30-0.50	0.58
4.0	0.35-0.53	0.63
6.0	0.40-0.69	0.69
8.0	0.51-0.77	0.75

$$\text{Equation:}^{\dagger} \quad 0.5 + \frac{S_s}{32}$$

* Figure I2a shows wheel configurations.

** Assuming simple beam reaction load for all four wheel types. Each type was used for the classes of vehicles shown for that type in Appendix D-1 of TM 5-312. Type D (which is a special type not shown in the appendix) was not used for light class vehicles (< 16) or $S_s < 4$ ft.

† This equation approximates the fraction of axle load distributed to critical stringer as a function of the stringer spacing S_s.

6-6. Shear Check (Shear Design)

a) Steel Stringer Bridges

(2) *Live Load Shear:* The maximum shear loading for one stringer (v_{LL}) occurs when the vehicle is near the abutment or support. The value of the total live load shear in kips (V_{LL}) is obtained from the shear curves (Appendices D-4 and D-5). The value of v_{LL}, the total live load shear per stringer, including a 15 percent increase for impact, is:

$$v_{LL} = 1.15\, v'_{LL} \qquad \text{[Eq I1]}$$

where v_{LL} = total live load shear per stringer in kips, including impact

v'_{LL} = live load shear force per stringer in kips from Table I2, excluding impact

Based on the above, the total design shear is:

$$v = \frac{V_{DL}}{N_s} + v_{LL} \qquad \text{[Eq I1a]}$$

[Revised Eq 6-14b, TM 5-312]

where v = total design shear per stringer in kips

V_{DL} = total dead load shear in kips

N_s = total number of stringers in bridge.

Table I2

Value of Live Load Shear Force per Stringer, v'_{LL}, Excluding Impact

	v'_{LL} for Single Lane* (kips)	v'_{LL} for Double Lane** (kips)
Wheeled vehicle	$1.25\left[\left(0.5 + \frac{S_s}{32}\right)V_A + \left(\frac{V_{LL_W} - V_A}{N_1}\right)\right]$	$\left(\frac{S_s - 2}{S_s}\right)V_A + \left(\frac{V_{LL_W} - V_A}{N_2}\right)$
Tracked vehicle	$1.25\left(\frac{V_{LL_T}}{2}\right)$	$\left(\frac{S_s - 2}{S_s}\right)V_{LL_T}$

where V_{LL_W} = wheeled vehicle shear in kips, as given in Appendices D-4 and D-7† of TM 5-312

V_{LL_T} = tracked vehicle shear in kips, as given in Appendices D-5 and D-7† of TM 5-312

V_A = the heaviest axle load in kips, as given in column 3† of Appendix D-1 of TM 5-312

S_s = center-to-center stringer spacing in ft

N_1 = effective number of stringers for single-lane bridge defined in Table H8a

N_2 = effective number of stringers for two-lane bridge defined in Table H8a

* The coefficient of 1.25 is used to adjust shear from the normal crossing case to the caution crossing case.

** For the double lane bridge case, v'_{LL} shall be computed for both single and double lanes and the larger value of v'_{LL} shall be used.

† The entries in Appendix D-7 and Column 3 of Appendix D-1 are given in tons, which must be converted to kips.

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SI CONVERSION FACTORS

1 ft	= 0.3048 m
1 ft-lb	= 1.3558 Nm
1 in.	= 2.54 cm
1 kip	= 4.448 kN
1 kip-ft	= 1.3558 kNm
1 ksi	= 0.69 kN/cm^2
1 sq in.	= 6.4516 cm^2
(°F -32) 5/9	= °C

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