FATIGUE DESIGN RECOMMENDATIONS FOR STEEL STRUCTURES

BY

JAPANESE SOCIETY OF STEEL CONSTRUCTION

(English Version)

December 1995

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Preface to the English Version

"Fatigue Design Recommendations for Steel Structures" was issued in April 1993 by JSSC (Japanese Society of Steel Construction) ; the Committee on Revision Works on Fatigue Design Recommendations. The original edition, which was published in Japanese, contains "Fatigue Design Recommendations", "Commentary", "Appendix : Estimation of Fatigue Crack Propagation Life Based on Fracture Mechanics Analysis", "Examples of Fatigue Design for Some Steel Structures" and "Reference Materials".

This English version was translated from the Japanese original as Fatigue Design Recommendations, Commentary and Appendix. We hope that the Recommendations will be useful and helpful for the fatigue designs of many kinds of steel structures and for the safety of existing structures against fatigue.

December 1995

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FATIGUE DESIGN RECOMMENDATIONS

1. Scope and limitations

1.1 The Recommendations give methods for assessment of safety in the fatigue limit state of members forming steel structures. The fatigue limit state is the state at which the strength or function of a structure or its structural members or joints is lost by fatigue damage.

1.2 Structures are assumed to be used mainly at ambient temperature and in air. If there is fear of corrosion, then in principle appropriate corrosion protection should be applied to the structure.

1.3 The Recommendations can also be applied to fatigue assessment of existing structures.

1.4 If the algebraic maximum or minimum nominal stress expected during design life exceeds yield stress, the Recommendations cannot be applied.

1.5 Items concerning fatigue design which are not dealt with in the Recommendations should be determined under the judgment and responsibility of the relevant authorities.

1.6 The Recommendations do not provide assessment of safety for limit states other than the fatigue limit state. For other limit states, the design should be carried out according to the relevant codes.

2. Definitions and symbols

Definitions and symbols used in the Recommendations are explained as follows.

2.1 Definitions

Allowable Stress Range : The allowable stress range which is corrected to the basic allowable stress range considering the effects of mean stress and plate thickness.

Basic Allowable Stress Range : The allowable stress range for a given stress cycles, which is determined according to fatigue design curve.

Basic Allowable Stress Range at 2×10^6 Cycles : The basic allowable stress range for two million stress cycles.

Constant Amplitude Cut-off Limit : Stress range which corresponds to fatigue limit under constant amplitude stress. In cases where every applied variable amplitude stress is less than this value, fatigue assessment is not necessary.

Correction Factor for Design Stress : The correction factor to account for the difference between the value of stress which is actually produced in the structural member and the value of stress determined by design calculation.

Dead Load : The weight of the structure itself and the weight of additional utilities, which do not vary and apply continually over a long period of time.

Design Life : The design period for which the structure is expected to be used.

Design Number of Stress Cycles : Total number of stress cycles expected during the design life, or number of cycles with stress ranges larger than the variable amplitude cut-off limit in variable amplitude stresses.

Design Stress Range : Equivalent stress range to be calculated according to fatigue design load, design calculation and the correction factor for design stress.

Equivalent Stress Range : Constant amplitude stress range which causes fatigue damage equivalent to the same repeated number of variable amplitude stresses.

Fatigue Damage : Damage which is caused by fatigue crack initiation and propagation in the structure, structural members or joints by the repeated applications of loads.

Fatigue Design Curve : The curve that represents the relationship between the stress range (normal stress, shear stress) and fatigue life for each strength category.

Fatigue Design Load : The load to be used in fatigue design, which is given as a load unit, a combination of load units or a representative load unit.

Gross Section Stress : Stress determined by gross section area which is the product of the gross width and plate thickness. The gross width is determined by not discounting hole widths such as bolt holes from the gross width of section.

Hot Spot Stress : Local stress at point where fatigue damage has developed, but this is not included in the stress concentration from weld bead form.

Joint Type : Joint classification according to the form and fatigue strength. Each joint is given a strength category.

Load Unit : The load which expresses the characteristics of live load in the unit term, given in terms of magnitude, frequency and loading position.

Maximum Load Unit : The load unit which produces the maximum stress range in a structural member during its design life.

Net Section Stress : Stress calculated by net section area, in which net width is multiplied by plate thickness. The net width is determined by discounting hole widths such as bolt holes from the gross width of section.

Nominal Stress : Stress based on the sectional force calculated by beam theory or frame analysis, etc.

Rain Flow Counting Method : A method for determining a stress range histogram from variable amplitude stresses.

Representative Load Unit : A load unit which represents the plural load units.

Safety Factor : The factor to adjust the level of safety in fatigue assessment, which comprises a redundancy factor, an importance factor and an inspection factor.

Strength Category : Category represented by symbol marks A to H, K1 to K5, and S. Fatigue design curve is given for every strength category.

Stress Range : Algebraic difference of the maximum stress and the minimum stress.

Stress Range Histogram : Frequency distribution of stress range.

Unit Term : The unit term to be used to measure design life. It can be an hour, a day, a week, a month or a year according to the characteristics of the structure.

Variable Amplitude Cut-off Limit : The limit of the stress range below which stress range does not contribute to fatigue damage.

2.2 Symbols

a : crack size

a_i : initial crack size

a_c : critical crack size

C, n : material constants to express the relationship between fatigue crack propagation

rate and stress intensity factor range
C_0, D_0 : constants to express fatigue design curve
C_R : correction factor for allowable stress range taking into account the effect of mean stress
C_t : correction factor for allowable stress range taking into account the effect of plate thickness
D : damage or cumulative damage in linear damage rule
da/dN : fatigue crack propagation rate (m/cycle)
F : correction factor of stress intensity factor
L : design life
l : length of welded gusset or cover plate
m : inverse of slope of fatigue design curve
N : fatigue life (number of stress cycles to reach fatigue limit state)
N_i : fatigue life at stress range $\Delta\sigma_i$
n_b : number of bolts in the direction of stress (one side)
n_i : frequency of $\Delta\sigma_i$ in stress range histogram
n_t : design number of stress cycles
R : stress ratio
r : radius of fillet
s, s_b, s_h : leg length of fillet weld
T : number of unit periods in design life
t : plate thickness of welded joint
U : unit period (year, month, day, etc.)
w : width of welded joint
α : correction factor for design stress
ΔK : stress intensity factor range (MPa$\sqrt{\mathrm{m}}$)
ΔK_e : equivalent stress intensity factor range (MPa$\sqrt{\mathrm{m}}$)
ΔK_{th} : threshold value of stress intensity factor range (MPa$\sqrt{\mathrm{m}}$)
$\Delta\sigma$: normal stress range (MPa)
$\Delta\tau$: shear stress range (MPa)
$\Delta\sigma_b$: stress range due to out-of-plane bending (MPa)
$\Delta\sigma_d$ ($\Delta\tau_d$) : design stress range (MPa)
$\Delta\sigma_{ce}$ ($\Delta\tau_{ce}$) : constant amplitude cut-off limit (MPa)
$\Delta\sigma_e$ ($\Delta\tau_e$) : equivalent stress range (MPa)
$\Delta\sigma_f$ ($\Delta\tau_f$) : basic allowable stress range at 2×10^6 cycles (MPa)
$\Delta\sigma_i$ ($\Delta\tau_i$) : one stress range level in stress range histogram (MPa)
$\Delta\sigma_m$: membrane stress range (MPa)
$\Delta\sigma_R$ ($\Delta\tau_R$) : allowable stress range at design number of stress cycles (MPa)
$\Delta\sigma_{ve}$ ($\Delta\tau_{ve}$) : variable amplitude cut-off limit (MPa)
γ_b : redundancy factor (partial safety factor)
γ_i : inspection factor (partial safety factor)
γ_w : importance factor (partial safety factor)

3. Materials, joints and their required qualities

3.1 Materials

Materials used for members or joints of steel structures designed according to the

Recommendations, should be those provided in Section 3.2 and 3.3.

3.2 Steel

Materials intended for the Recommendations are carbon steel and low alloy steel whose ultimate strength is between 330MPa and 1GPa. However, ultimate strength of wires used for cable should not exceed about 1.6GPa. Ultimate strength of high strength bolts should not exceed about 1.2GPa.

3.3 Materials for connections

3.3.1 Materials for welding

Materials for welding must be suitable for the steel welded.

3.3.2 Materials for high strength bolts

Materials used for friction-type high strength bolted connections must have strength to carry the axial force during installation as indicated in the design.

3.4 Connection methods and required qualities

3.4.1 Welded connections

The recommendations are applicable to joints and structural members having sound welds.

The welding method should be equivalent to or better than arc welding. The welding method and order of welding should be selected so that weld cracking and other weld defects are prevented, and deformation and constraint stresses due to welding are reduced.

3.4.2 High strength bolted connections

The fabrication of friction-type high strength bolted connections should be done so that the axial forces in each of the bolts is within the prescribed range. The surface condition and degree of distortion of steels to be bolted and the order of fastening should be such that sufficient uniform axial forces and friction resistance between plates can be secured.

Drilling of base plates and splice plates used for bearing-type high strength bolted connections should be in the prescribed positions, so that connections have enough bearing resistance. Distortion of steel plates and fastening of bolts should be controlled to prevent opening between base plates and splice plates.

4. Fatigue strength and joint types

4.1 Main factors controlling fatigue strength

Fatigue life of welded members or welded joints varies depending on the kind of joint and the magnitude of the nominal stress range.

4.2 Fatigue design curves ($\Delta\sigma$-N or $\Delta\tau$-N relationships for fatigue design)

(1) The fatigue design curves consist of three types of curves which are given for joints subject to normal stress (Fig. 4.1), cables and high strength bolts subject to normal stress (Fig. 4.2), and joints subject to shear stress (Fig. 4.3). The curves are different depending on the strength category, that is, eight curves (category A to H), five curves (category K1 to K5) and one curve (category S) are given for each kind of joint.

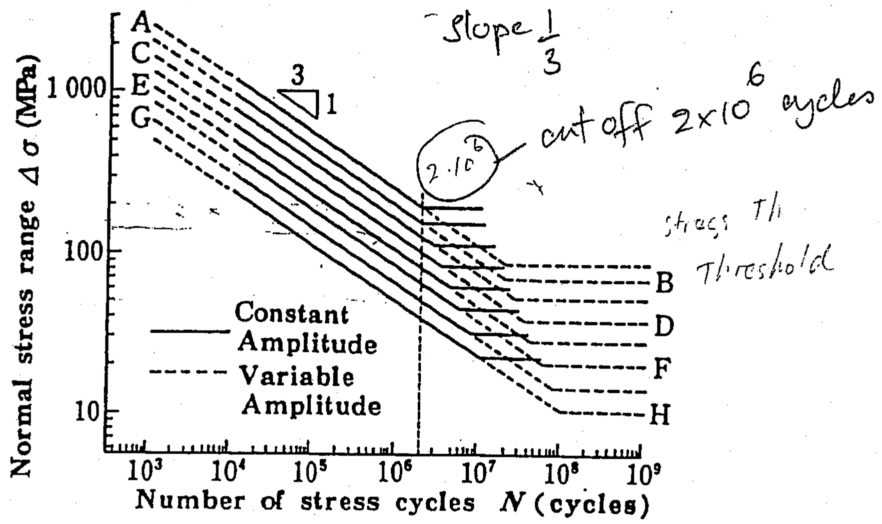


Fig.4.1 Fatigue design curves for joints subjected to normal stress

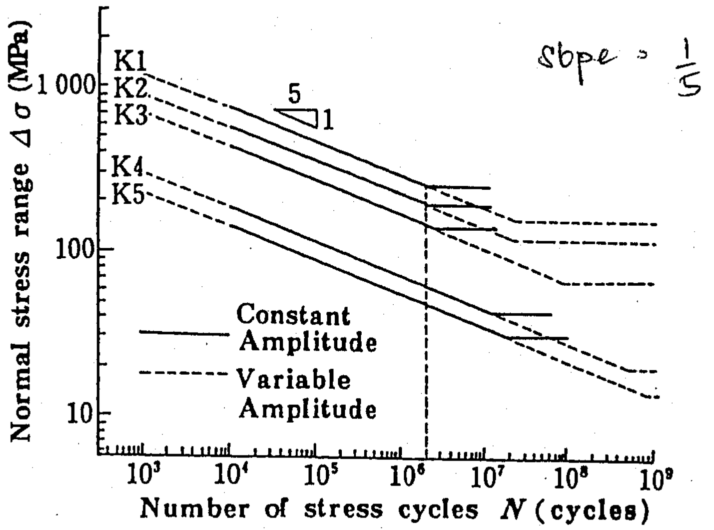


Fig.4.2 Fatigue design curves for cables and high strength bolts subjected to normal stress

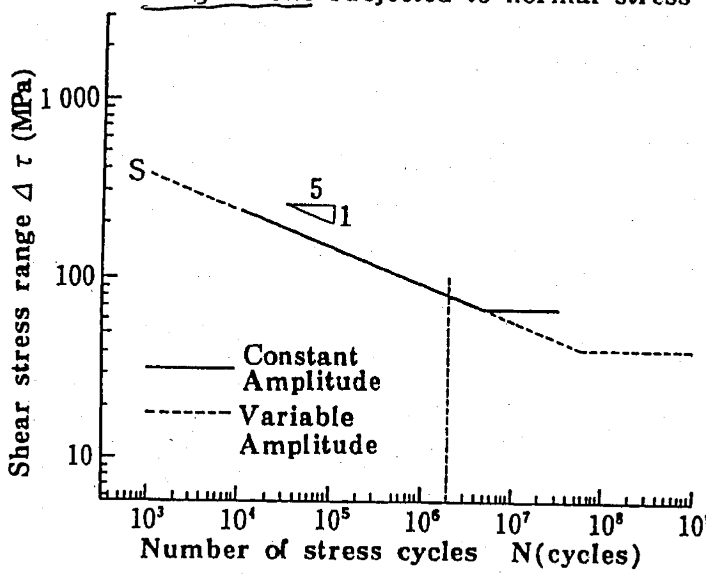


Fig.4.3 Fatigue design curves for joints subjected to shear stress

In Table 4.1 (joints subject to normal stress), Table 4.2 (cables and high strength bolts subject to normal stress) and Table 4.3 (joints subject to shear stress), the basic allowable stress range at 2×10^6 cycles ($\Delta\sigma_f$; $\Delta\tau_f$), constant amplitude cut-off limit ($\Delta\sigma_{ce}$, $\Delta\tau_{ce}$), and variable amplitude cut-off limit ($\Delta\sigma_{ve}$, $\Delta\tau_{ve}$) for each strength category are given. If the maximum stress range of the variable amplitude stress is lower than the constant amplitude cut-off limit, fatigue assessment is not necessary. When one of the stress range in the variable amplitude stresses is higher than the constant amplitude cut-off limit, the variable amplitude cut-off limit gives the limiting value for the fatigue assessment, below which no damage occurs.

(2) The eight fatigue design $\Delta\sigma$-N curves in Fig. 4.1 and five fatigue design $\Delta\sigma$-N curves in Fig. 4.2 are parallel in log-log relationships. The distances between fatigue design curves in Fig. 4.1 are all almost the same at about 25 percent.

(3) The fatigue design curves are given by Eq. (4.1) and Eq. (4.2)

$$\Delta\sigma^{'''}\cdot N = C_0 \quad (\Delta\sigma > \Delta\sigma_{ce}, \Delta\sigma_{ve}) \qquad (4.1)$$
$$N = \infty \quad (\Delta\sigma \leqq \Delta\sigma_{ce}, \Delta\sigma_{ve})$$
$$\Delta\tau^{'''}\cdot N = D_0 \quad (\Delta\tau > \Delta\tau_{ce}, \Delta\tau_{ve}) \qquad (4.2)$$
$$N = \infty \quad (\Delta\tau \leqq \Delta\tau_{ce}, \Delta\tau_{ve})$$
$$C_0 = 2\times10^6\cdot\Delta\sigma_f^{'''},\ D_0 = 2\times10^6\cdot\Delta\tau_f^{'''}$$

where $\Delta\sigma_f$; $\Delta\tau_f$ are the basic allowable stress ranges at 2×10^6 cycles. m is the exponent which shows the slope of the fatigue design curves, and is given as follows.

m = 3 for joints subjected to normal stress, Table 4.4 (a) to (f),
m = 5 for cables and high strength bolts subjected to normal stress, Table 4.4 (g),
m = 5 for joints subjected to shear stress, Table 4.4 (f).

Table4.1 Basic allowable stress range for joints subjected to normal stress(m=3)

Strength categories		Cut-off limit (MPa)	
Category	Basic allowable stress range at 2×10^6 cycles $\Delta\sigma_f$(MPa)	Constant amplitude $\Delta\sigma_{ce}$(N) *	Variable amplitude $\Delta\sigma_{ve}$(N) *
A	190	190 (2.0×10^6)	88 (2.0×10^7)
B	155	155 (2.0×10^6)	72 (2.0×10^7)
C	125	115 (2.6×10^6)	53 (2.6×10^7)
D	100 112	84 (3.4×10^6)	39 (3.4×10^7)
E	80	62 (4.4×10^6)	29 (4.4×10^7)
F	65	46 (5.6×10^6)	21 (5.6×10^7)
G	50	32 (7.7×10^6)	15 (7.7×10^7)
H	40	23 (1.0×10^7)	11 (1.0×10^8)

* The value of N parenthesized is the stress cycles according to the stress range indicated. This value is rough approximate and only given for reference.

UkDEn ≈ D class for JSSC.

Table4.2 Basic allowable stress range for cables and high strength bolts subjected to normal stress($m=5$)

Strength categories		Cut-off limit (MPa)	
Category	Basic allowable stress range at 2×10^6 cycles $\Delta\sigma_f$(MPa)	Constant amplitude $\Delta\sigma_{ce}$(N) *	Variable amplitude $\Delta\sigma_{ve}$(N) *
K1	270	270 (2.0×10^6)	170 (2.0×10^7)
K2	200	200 (2.0×10^6)	126 (2.0×10^7)
K3	150	148 (2.1×10^6)	68 (1.0×10^8)
K4	65	46 (1.1×10^7)	21 (5.7×10^8)
K5	50	32 (1.9×10^7)	15 (8.2×10^8)

* ditto

Table4.3 Basic allowable stress range for joints subjected to shear stress($m=5$)

Strength categories		Cut-off limit (MPa)	
Category	Basic allowable stress range at 2×10^6 cycles $\Delta\sigma_f$(MPa)	Constant amplitude $\Delta\sigma_{ce}$(N) *	Variable amplitude $\Delta\sigma_{ve}$(N) *
S	80	67 (5.0×10^6)	42 (5.0×10^7)

* ditto

If necessary, the fatigue design curve should be corrected to account for mean stress and plate thickness effect described in Section 4.4 and 4.5.

(4) For the relationship between $\Delta\sigma$ and N under the variable amplitude cut-off limit, the relationship over the limit may be used.

4.3 Strength categories of joints

The fatigue strength categories of joints are given by checking that the lowest value of fatigue strength from tests on each joint detail or the 97.7% survival probability of fatigue strength which is almost correspondent to the lowest value are higher than the fatigue design curves defined in Section 4.2.

Strength categories of joints are shown in Table 4.4. This classification takes into account the stress concentration of the weld detail, weld qualities (qualities are defined in Chapter 3 and Table 4.4), direction of stress, metallurgical structure, residual stress and machining of weld reinforcement or finishing of weld toe.

For welded joints which are not included in Table 4.4, a fatigue strength curve can be defined from available experimental data or new fatigue test data. Fatigue crack propagation analysis can also provide the fatigue strength. It should be noted that there is a difference between the real joint and the experimental specimen in scale and residual stress when the fatigue design curve is defined by fatigue test results.

Table4.4 Joint types and strength categories

(a) Non-welded joints

Joint types		Strength categories ($\Delta\sigma_f$)	Remarks
1. Plates	(1) with surfaces and edges by machining (JIS roughness of 50s or less)	A (190)	
	(2) with mill scale surfaces and edges by flame cutting (JIS roughness of 100s or less)	B (155)	
	(3) with mill scale surfaces and edges by flame cutting (large drag lines must be removed)	C (125)	
2. Shaped steel	(1) with mill scale surfaces and edges	B (155)	
	(2) with mill scale surfaces and edges by flame cutting (JIS roughness of 100s or less)	B (155)	
	(3) with mill scale surfaces and edges by flame cutting (large drag lines must be removed)	C (125)	
3. Seamless tubes ไม่มี ตะเข็บ		B (155)	
4. Base plates with circular holes (net section stress)		C (125)	
5. Base plates with cut out gussets	(1) $1/5 \leq r/d$ (JIS roughness of 50s or less)	B (155)	
	(2) $1/10 \leq r/d < 1/5$ (JIS roughness of 50s or less)	C (125)	
	(3) $1/5 \leq r/d$ (JIS roughness of 100s or less)	C (125)	
	(4) $1/10 \leq r/d < 1/5$ (JIS roughness of 100s or less)	D (100)	
6. Base plates of friction type high strength bolted connections (gross section stress)	(1) $1 \leq n_b \leq 4$	B (155)	
	(2) $5 \leq n_b \leq 15$	C (125)	
	(3) $16 \leq n_b$	D (100)	
7. Base plates of bearing type high strength bolted connections (net section stress)		B (155)	n_b : number of bolts in the direction of stress (one side)
8. Base plates with holes and high strength bolts which do not transfer the loads along the direction of stress to be considered (net section stress)		B (155)	※ (4,6,7,8)If holes are manufactured by punching, the strength category should be reduced by one category.

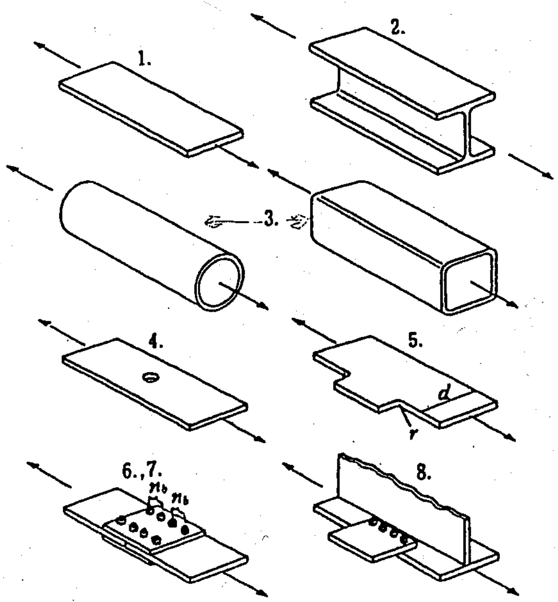
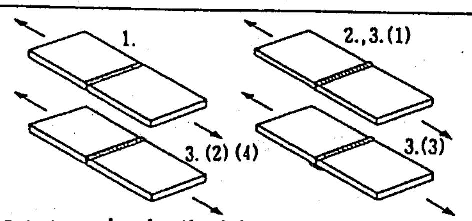


Table4.4 Joint types and strength categories (continued)

(b) Transverse butt welded joints

Joint types		Strength categories ($\Delta\sigma_f$)	Remarks
1. With ground flush surfaces Butt Weld Stress Concentration		B (155)	
2. With finished weld toe		C (125)	
3. As-welded joint	(1)both side welds	D (100)	
	(2)one side welds with smooth back-side weld geometry	D (100)	※ As a general rule, the joints should posses fully penetrated and sound weldments. ※ Transition in thickness and width : slope should be lower than 1:5.
	(3)one side welds with backing bars	F (65)	※ Undercuts deeper than 0.5mm must be removed.
	(4)one side welds unable to inspect those back surfaces	F (65)	※ (1.,2.) Finishing should be done so that undercuts do not remain. Direction of grinding should be the same as the direction of stress.

(c) Longitudinal welded joints

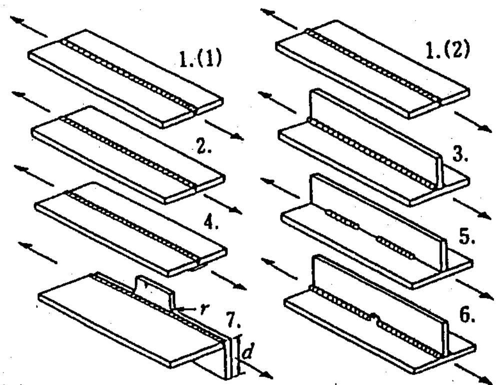
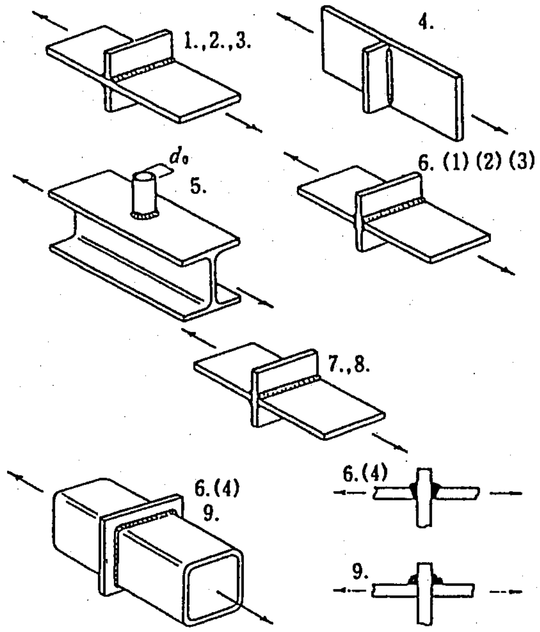
Joint types		Strength categories ($\Delta\sigma_f$)	Remarks
1. Complete penetration groove welded joints from both sides	(1)with ground flush surfaces	B (155)	
	(2)as-welded	C (125)	
2. Partial penetration groove welded joints		D (100)	
3. Fillet welded joints		D (100)	
4. Welded joints with backing bars		E (80)	
5. Intermittents fillet welded joints		E (80)	
6. Welded joints with copes		G (50)	
7. Welded joints adjacent to fillets of cut out gussets	(1)$1/5 \leq r/d$	D (100)	(1.(2),2.,3.) Any significant roughness on the weld surface caused by stop/start operation must be removed.
	(2)$1/10 \leq r/d < 1/5$	E (80)	※ (2.,3.) If it can be ascertained by non-destructive testing that the size of inner defects (roundish forms such as blowhole) is smaller than 1.5mm in width and smaller than 4.0mm in height, strength category C is applicable.

Table4.4 Joint types and strength categories (continued)

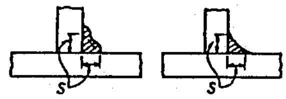
(d) Cruciform joints

Type		Joint types		Strength categories ($\Delta\sigma_f$)
Non load-carrying type		1. Fillet welded joints with smooth weld toes welded by confirmed method		D (100)
Non load-carrying type		2. Fillet welded joints with finished weld toes		D (100)
Non load-carrying type		3. As-welded fillet welded joints		E (80)
Non load-carrying type		4. Fillet welded joints including start and stop position *point*		E (80)
Non load-carrying type		5. Fillet welded joints of hollow section	(1) $d_o \leq 100$ mm	F (65)
Non load-carrying type			(2) $d_o > 100$ mm	G (50)
Load-carrying type		6. Complete penetration weld	(1) with smooth weld toes welded by confirmed method	D (100)
Load-carrying type			(2) with finished weld toes	D (100)
Load-carrying type			(3) as-welded	E (80)
Load-carrying type			(4) hollow section (one side welds)	F (65)
Load-carrying type	Fillet welds or partial penetration welds	7. Toe failure	(1) with smooth weld toes welded by confirmed method	E (80)
Load-carrying type	Fillet welds or partial penetration welds		(2) with finished weld toes	E (80)
Load-carrying type	Fillet welds or partial penetration welds		(3) as-welded	F (65)
Load-carrying type	Fillet welds or partial penetration welds		(4) including start and stop positions	F (65)
Load-carrying type	Fillet welds or partial penetration welds	8. Root failure (throat section)		H (40)
Load-carrying type	Fillet welds or partial penetration welds	9. Hollow section (one side welds)	(1) toe failure	H (40)
Load-carrying type	Fillet welds or partial penetration welds		(2) root failure (throat section)	H (40)

Remarks



※ (2.,6.(2),7.(2)) Finishing by grinding, TIG, peening and etc should be done so that undercuts do not remain. If finishing is done by grinding, grinding should be made in the direction of stress.
※ (1.,6.(1),7.(1)) Undercuts must be removed.
※ (3.,4.,5.,6.(3)(4),7.(3)(4),9.(1)) Undercuts deeper than 0.5mm must be removed.
※ (8.,9.(2)) The weld throat area is given by the product of weld throat and weld length.



The weld throat is calculated as $s/\sqrt{2}$.
Weld throat for partially penetrated welds is calculated as (s + depth of groove)/$\sqrt{2}$.
※ (8.,9.(2)) Strength category for the joint with s shorter than (0.4×plate thickness) is not specified.

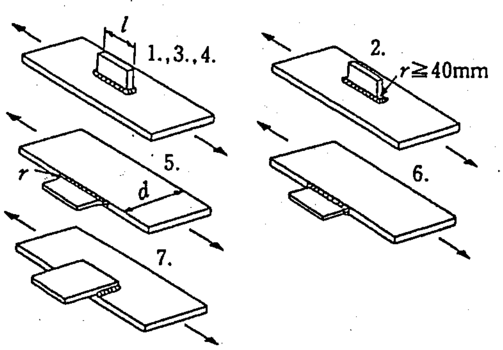
Non load-carrying type
Load-carrying type
Fillet welds or partial penetration welds

Table4.4 Joint types and strength categories (continued)

(e) Gusset joints (including welded attachments)

Joint types			Strength categories ($\Delta\sigma_f$)
Out of plane gussets	1. Joints with fillet welded or groove welded gusset ($l \leq$ 100mm)	(1)with finished weld toes	E (80)
		(2)as-welded	F (65)
	2. Joints with groove welded gusset with fillet (fillet to be finished)		E (80)
	3. Joints with fillet welded gusset ($l >$ 100mm)		G (50)
	4. Joints with groove welded gusset ($l >$ 100mm)	(1)with finished weld toes	F (65)
		(2)as-welded	G (50)
In plane gussets	5. Joints with groove welded gusset with fillet (fillet to be finished)	(1)$1/3 \leq r/d$	D (100)
		(2)$1/5 \leq r/d < 1/3$	E (80)
		(3)$1/10 \leq r/d < 1/5$	F (65)
	6. Joints with groove welded gusset	(1)with finished weld toes	G (50)
		(2)as-welded	H (40)
7. Base plate with lap-welded gusset			H (40)

Remarks



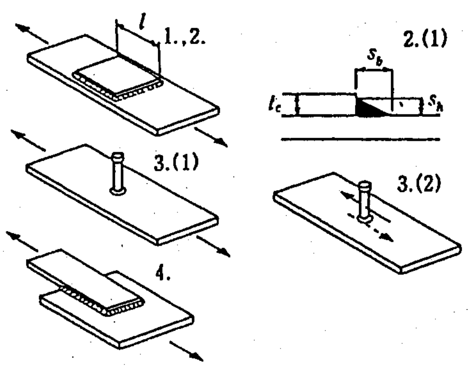
※ (1.(1),2.,4.(1),5.,6.(1)) Finishing should be done so that undercut do not remain. If finishing is done by grinding, grinding should be made in the derection of stress.

※ (1.(2),3.,4.(2),6.(2),7.) Undercuts deeper than 0.5mm must be removed.

(f) Other welded joints

Joint types		Strength categories ($\Delta\sigma_f$)
1. Joints with fillet welded cover plates ($l \leq$ 300 mm)	(1)with finished weld toes	E (80)
	(2)as-welded	F (65)
2. Joints with fillet welded cover plates ($l >$ 300 mm)	(1)with profiled end welds	D (100)
	(2)as-welded	G (50)
3. Welded studs	(1)at base plates	E (80)
	(2)at stud sections	S (80)
4. Lapped joints	(1)at base plates	H (40)
	(2)at splice plates	H (40)
	(3)at throat sections of end fillet welds	H (40)
	(4)at throat sections of side fillet welds	S (80)

Remarks



※ (1.(1),2.(1)) Finishing should be done so that undercuts do not remain. If finishing is done by grinding, grinding should be made in the direction of stress.

※ (1.(2),2.(2)) Undercuts deeper than 0.5mm must be removed.

※ (2.(1))Weld leg length S_b, S_h must be $S_b \geq 0.8t$, and $S_h \geq 2S_b$.

4.4 Effect of mean stress

For cables given in Table 4.4 (g), the allowable stress range should be obtained by multiplying the basic allowable stress by C_R as given in Eq. (4.3).

$$C_R = (1\text{-}R)/(1\text{-}0.9R) \tag{4.3}$$

where the stress ratio, R, is the ratio of the minimum to the maximum values of stresses under consideration. The value of R is based on the fatigue design load and dead load. This correction for the stress ratio should be also done to constant and variable amplitude cut-off limits. For high strength bolts given in Table 4.4 (g), the modification due to stress ratio should not be applied.

As a general rule, the effect of stress ratio on the joints given in Table 4.4 (a) to (f) should be ignored, except for the compressive mean stress region. That is, the allowable stress ranges may be corrected by multiplying the basic available stress ranges by the value of C_R given in Eq. (4.4) in cases where R is lower than -1.

$$C_R = \{1.3(1\text{-}R)/(1.6\text{-}R)\} \qquad (\text{for } R \leqq -1) \tag{4.4}$$

In cases where both values of the maximum and minimum stresses are in the compressive region, the value of C_R should be taken as C_R=1.3 as shown in Fig. 4.4. In this case, the residual stress should not be taken into account in the estimation of maximum and minimum stresses. The constant and the variable amplitude cut-off limit may be modified by considering the effect of mean stresses.

4.5 Effect of plate thickness

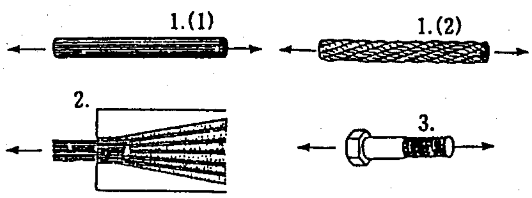
The fatigue strength decreases with increase of plate thickness in some kinds of joint. The allowable stress range should be obtained by multiplying the basic allowable stress range by the correction factor C_t when the plate thickness of the joint exceeds 25mm.

$$C_t = \sqrt[4]{25/t} \tag{4.5}$$

where t is the plate thickness in mm. The joints which should be corrected by Eq. (4.5) are as follows.

Table4.4 Joint types and strength categories (continued)
(g) Cables and high strength bolts

Joint types		Strength categories ($\Delta\sigma_f$)	Remarks
1. Cables	(1)parallel wire stands	K1 (270)	
	(2)wire ropes	K2 (200)	
2. Cable anchorages	(1)fatigue-proof anchorages for parallel wire strands	K1 (270)	
	(2)Zinc-poured anchorages for parallel wire strands	K2 (200)	
	(3)Zinc-poured anchorages for ropes	K3 (150)	
3. High strength bolts	(1)rolled	K4 (65)	※ (2.(1)) Fatigue-proof anchorages means those whose fatigue strength is nearly equal to the parallel wire strands.
	(2)cut	K5 (50)	



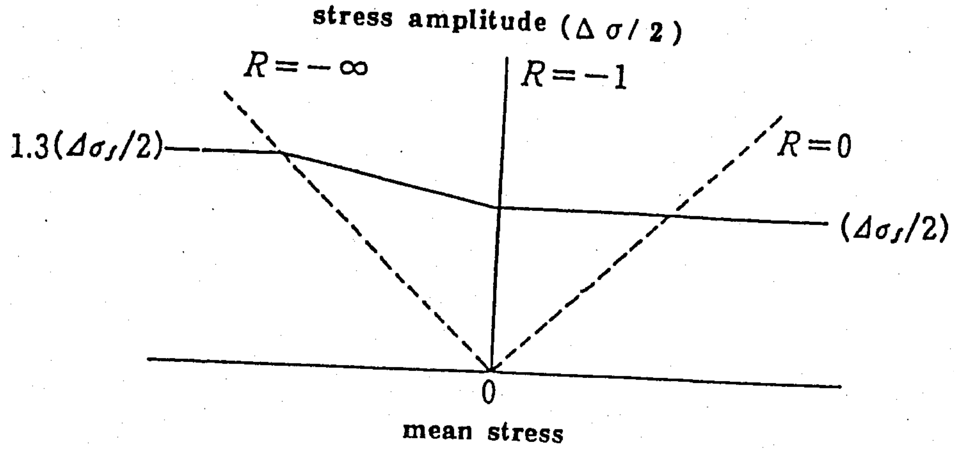


Fig.4.4 Correction of allowable stress range due to mean stress

Cruciform joints (Table 4.4 (d))................. 1., 3., 4., 5., 6.(1), 6.(3), 6.(4), 7.(1), 7.(3), 7.(4), 8., 9.

Other welded joints (Table 4.4 (f))............... 1.(2), 2.(2)

For cruciform joints which fail at the weld toe, the correction should not be applied to joints where the thickness of attachments is less than 12mm.

The constant amplitude and variable amplitude cut-off limit should be modified by multiplying the basic value by C_t in order to obtain the cut-off limits for joints of thicker plate.

5. Load and stress range histogram

5.1 Design loads

The fatigue design loads should be defined as a load unit, a combination of plural load units, or a representative load unit, representing all the actual fatigue loads which act on the structure during the design life.

5.1.1 Load unit

A load unit should be defined with consideration of the expected service conditions of the structure. A load unit consists of the magnitude of loads, frequency of loads and the loading location and its movement, and it should represents the characteristics of the fatigue loading in a unit period. In cases where a combination of plural load units is adequate for the design loads, the each load unit should be defined individually.

When it is considered to be adequate that the magnitude of the load unit is non-determinate (variable), the magnitude of the design load should be defined as variable.

5.1.2 Representative load unit

To simplify the design calculation, the fatigue design load can be defined as one load unit which represents multiple load units or a determinate load unit which represents a variable load unit. Such a load unit is referred to as a 'representative load unit'.

5.1.3 Maximum load unit

A load unit which will induce the maximum stress range in the structural members during the design life is referred to as the 'maximum load unit'. In cases where a load unit defined in 5.1.1 a representative unit load defined in 5.1.2 does not correspond to the load that induces t maximum stress range, the maximum load unit may be used as the design load in the simplifi fatigue assessment described in Section 6.3.

5.1.4 Dynamic effects

When dynamic effects of the load on the magnitude of the sectional force are considered not be negligible, these should be taken into account in the fatigue assessment.

An impact factor which represents the effect of dynamic loads should be determined based c the data measured on similar structures. In the absence of more detailed information, the impa factor used in the other limit state assessments should be used.

5.1.5 Design life

The structure to be designed is assumed to have a pre-defined design life, L.

5.1.6 Unit term

In cases where a load unit is used as the fatigue design load, the design life, L, is defined as times an appropriately defined unit term, U, ($L=T\cdot U$). During a unit term, more than one load uni should be included. For a unit term, any unit time such as, hour, day, week, month or year, can b used corresponding to the service characteristics of the structure.

5.2 Stress

5.2.1 Stress calculation

The nominal stress at the objective section is used as the stress for fatigue assessment. Nominal stress of the member at a point is calculated at the section perpendicular to the arrows as shown in Table 4.4. If the section is defined for stress calculation, the stress calculated is based on that section.

5.2.2 Stresses due to out of plane bending

When stress is caused by out of plane bending at a point due to eccentricity between two joined steel plates or the loading mode, the normal stress range $\Delta\sigma$ is obtained by adding the stress range due to out of plane bending $\Delta\sigma_b$, as shown in Eq. (5.1).

$$\Delta\sigma = \Delta\sigma_m + (4/5)\Delta\sigma_b \qquad \Delta\sigma_m\text{: membrane stress range} \qquad (5.1)$$

Eq. (5.1) is applied to joints with plate thicknesses less than 25mm. For joints with plate thicknesses greater than 25mm, the multiplying factor 4/5 for out of plane bending should be changed to 1.

5.2.3 Correction of design stress

When the actual stress is obviously different from the nominal stresses calculated in the design calculations, the nominal stress range is multiplied by a correction factor α (termed correction factor for design stress) in order to correct the nominal stress range.

5.2.4 Hot spot stress

If the calculation and the definition of nominal stress is difficult, the hot spot stress concept can be used, as described in Chapter 7.

5.3 Stress range histogram

The stress variations, which occur at the point of interest during the design life, are calculated by using the fatigue design load. A stress range histogram can be determined by applying the rain flow counting method or equivalent counting method to the corresponding stress variations.

When the relationship between the stress range histogram and the stress range which is calculated from the representative load unit or maximum load unit is clear, the stress range histogram may be used in the following fatigue assessment.

5.4 Design number of stress cycles

The total number of stress cycles in the design life n_t (design number of stress cycles) is calculated by Eq. (5.2).

$$n_t = \Sigma n_i \cdot T \qquad \Sigma n_i\text{: total number of stress cycles in a unit term} \tag{5.2}$$

5.5 Equivalent stress range

The equivalent stress range $\Delta\sigma_e$ or $\Delta\tau_e$ is obtained from Eq. (5.3), which gives equivalent fatigue damage for the same number of stress cycles as the variable amplitude stresses during the design life.

$$\Delta\sigma_e = \sqrt[m]{\Sigma\Delta\sigma_i^m \cdot n_i / \Sigma n_i}$$
$$\Delta\tau_e = \sqrt[m]{\Sigma\Delta\sigma_i^m \cdot n_i / \Sigma n_i} \tag{5.3}$$

normal stress $m = 3, 5$, shear stress $m = 5$,

where $\Delta\sigma_i$ and $\Delta\tau_i$ are each stress ranges comprising the stress range histogram obtained in Section 5.3, and n_i is the frequency of $\Delta\sigma_i$ or $\Delta\tau_i$. Eq. (5.3) is introduced from the contents described in Section 6.4. If the stress range is less than $\Delta\sigma_{ve}$ or $\Delta\tau_{ve}$, then n_i=0. However, when a representative load unit is used as the fatigue design load, $n_i \neq 0$ even if the corresponding stress range is less than $\Delta\sigma_{ve}$ or $\Delta\tau_{ve}$.

If the stress variations calculated from the fatigue design load are of constant amplitude, the equivalent stress range is equal to that of the constant amplitude stress.

6. Fatigue assessment

6.1 Structural details to be assessed

The fatigue assessment must be done at the detail for which the stress range is the maximum if there is a continuous series of joint details of the same strength category.

6.2 Safety factors

The safety factors consist of three partial safety factors as given below.

(1) The redundancy factor γ_b considers the fatigue damage effect to the whole structure strength or its function when damage occurs in the objective joint or member.
(2) The importance factor γ_w shows the degree of importance of a structure, i.e. it is a factor which considers the social effect when a structure reaches the fatigue limit state.
(3) The inspection factor γ_i considers the damage-detection probability in a structure by periodic inspections before it reaches the critical state.

Each of the partial safety factor values is to be determined by the organizations concerned. The partial safety factor references are as follows.

γ_b : γ_b=1.10 is applied when the occurrence of fatigue damage in an objective structural member or a joint causes the whole structure to collapse (termed a non-redundan member), values of γ_b=1.00～1.10 are applied when damage influences to some extent th strength or function of a structure. When there is little problem relating to the strength anc function of the structure, the factor should be 0.80.

γ_w : The factor is varied from 0.80 to 1.10, depending upon the significance of a structure.

γ_i : In cases where periodic inspections for maintenance and management as described in Chapter 8 is carried out, the factor is varied from 0.90 to 1.00. When the inspection can not be made, the factor should be 1.10.

The upper limit for the product of γ_b, γ_w and γ_i should be 1.25, and the lower limit should be 0.80.

6.3 Simplified procedure of fatigue assessment

When the predicted maximum stress range and the constant amplitude cut-off limit of an object satisfy the equation below, a fatigue assessment described in Sections 6.6 and 6.7 is not necessary.

$$(\gamma_b \cdot \gamma_w \cdot \gamma_i)\Delta\sigma_{max} \leqq \Delta\sigma_{ce} \cdot C_R \cdot C_t \qquad (6.1)$$
$$(\gamma_b \cdot \gamma_w \cdot \gamma_i)\Delta\tau_{max} \leqq \Delta\tau_{ce}$$

$\Delta\sigma_{max}$, $\Delta\tau_{max}$: the predicted maximum stress range during the design life.

However, when only the representative load unit is used as the fatigue design load, even if the maximum stress range is under the constant amplitude cut-off limit, the following fatigue assessment should be done.

6.4 Fatigue strength evaluation under variable amplitude stresses

Fatigue strength under variable amplitude stresses can be evaluated based on the linear damage rule. The fatigue design curves to be used are shown in Chapter 4.

In cases where the fatigue design load is provided in a representative load unit, one must not use the variable amplitude cut-off limit, that is, the variable amplitude cut-off limit of all strength categories is 0.

6.5 Fatigue assessment

Safety assessment of the fatigue limit state should be done on the basis of the methods described in Section 6.6 or Section 6.7.

6.6 Fatigue assessment based on equivalent stress range

Fatigue assessment is carried out by using design stress ranges, allowable stress ranges and safety factors.

6.6.1 Design stress range

The design stress range ($\Delta\sigma_d$, $\Delta\tau_d$) can be obtained from Eq. (6.2)

$$\Delta\sigma_d = \Delta\sigma_e \qquad (6.2)$$
$$\Delta\tau_d = \Delta\tau_e$$

$\Delta\sigma_e$, $\Delta\tau_e$: equivalent stress range

The equivalent stress range should be calculated by also considering the design stress correction factor, α.

6.6.2 Allowable stress range

The allowable stress range can be obtained from Eq. (6.3)

$$\Delta\sigma_R = \sqrt[m]{C_0/n_t} \cdot C_R \cdot C_t$$
$$\Delta\tau_R = \sqrt[m]{D_0/n_t} \qquad (6.3)$$

C_0, D_0 : Constants for the fatigue design curve [Ref. Eqs. (4.1) and (4.2)]

n_t : Design number of stress cycles. The cycles do not include cycles of stress range under the variable amplitude cut-off limit. But when a representative load is used as the fatigue design load, it should include all the stress range cycles.

C_R : Correction factor for basic allowable stress range taking into account the effect of mean stress.

C_t : Correction factor for allowable stress range taking into account the effect of the plate thickness.

6.6.3 Fatigue assessment

In the fatigue assessment, it should be confirmed that Eq. (6.4) is satisfied.

$$(\gamma_b \cdot \gamma_w \cdot \gamma_i)\Delta\sigma_d \leqq \Delta\sigma_R$$
$$(\gamma_b \cdot \gamma_w \cdot \gamma_i)\Delta\tau_d \leqq \Delta\tau_R \qquad (6.4)$$

6.7 Fatigue assessment based on cumulative damage

The fatigue assessment is carried out using the cumulative damage and safety factors.

6.7.1 Cumulative damage

Cumulative damage D is defined by Eq. (6.5)

$$D = \Sigma(n_i/N_i) \qquad (6.5)$$

n_i : Number of stress cycles of $\Delta\sigma_i$, $\Delta\tau_i$ comprising the stress range histogram, which can be obtained by the contents in Section 5.3.

N_i : Fatigue life according to $\Delta\sigma_i$ or $\Delta\tau_i$, which can be obtained from the fatigue design curve modified by mean stress and plate thickness effect.

6.7.2 Fatigue assessment

The fatigue assessment should be carried out by using Eq. (6.6).

$$D \leqq 1/(\gamma_b \cdot \gamma_w \cdot \gamma_i)^m \qquad (6.6)$$

m: inverse of slope of fatigue design curve

6.8 Combined stresses

When the objective section is subjected to normal stress and shear stress or to multi-axial stresses, the fatigue assessment should be carried out separately for both normal stress and shear stress.

When normal stress and shear stress act at the same time, and when from the engineering view point they have a large effect on the principal stress, the maximum principal stress should be determined by appropriate numerical calculations. A fatigue assessment as given in Sections 6.3, 6.6 or 6.7 should then be carried out by replacing the stress range with the principal stress range. The allowable stress range should be checked against the allowable stress range for normal stress.

7. Fatigue assessment based on hot spot stress

7.1 Definition of hot spot stress

The hot spot stress is a local stress where a fatigue crack initiates. It is the stress which resu at a point due to the global geometry of the structure without considering local stre concentration due to weld geometry and weld toe profile.

7.2 Calculation of hot spot stress

The hot spot stress can be obtained by empirical formulae, by the finite element method or b measuring strain in model specimens. When empirical formulae are used, attention should be pai to ensure that the objective details are within the relevant limitations. When the finite elemer method is used, attention should be paid to the types and sizes of elements.

7.3 Fatigue design curve

For as-welded joints the fatigue design curve of Category E is used. For welded joints whicl have finished weld toes that of Category D is used. However, if the joint is a load-carrying typ one with fillet welds or partial penetration welds, the fatigue design curve of Category F is usec for as-welded joints and that of Category E is used for welded joints with finished weld toes.

If the plate thickness of as-welded joints is more than 25mm, Eq. (4.5) should be used to obtain the allowable stress range.

7.4 Fatigue assessment

Fatigue assessment should be carried out as described in Chapter 6 by using hot spot stress ranges for the stress range.

8. Maintenance and management

8.1 Inspection

8.1.1 Inspection plan

When assessment for fatigue is considered to be necessary in the design of a structure, it is desirable that the manager or authority of the structure prepare guidelines for inspection.

8.1.2 Inspection interval

A regular inspection should be made in accordance with the inspection program, for the sites, at which fatigue assessment and past experience suggest that fatigue damage is apt to occur.

The interval of inspection to find damage should be set so that the assumed safety levels can be ensured. In such cases, a fatigue crack propagation analysis using fracture mechanics is effective.

If the load or its frequency is very different from that assumed at the time of design, it will be necessary to conduct another fatigue assessment, and the results should be reflected in the interval of inspection. The interval of inspection for the members should preferably be set with consideration to the effects of their fracture on the function of the structure.

8.1.3 Method of inspection

The inspection should be performed at the positions requiring inspection using an appropriate method. Inspection methods are given below. An appropriate method suited to the purpose is selected and a non-destructive inspection is then performed.

(a) Methods aimed mainly at finding fatigue cracks : visual inspection, magnetic particle inspection, eddy current flaw detection, etc.

(b) Methods aimed mainly at checking the features of the fatigue cracks, such as their size and shape : magnetic particle inspection, ultrasonic inspection, etc.

8.2 Evaluation of fatigue damage

If damage is found, its extent and effects should be evaluated in order to ensure the safety levels assumed in the fatigue design and to make appropriate treatment. At this time, it is effective to calculate the remaining life by using a fatigue crack propagation analysis based on fracture mechanics.

Evaluation should be conducted on the following items.

(a) Influence of a fracture at a site which has a crack.
(b) Propagation behavior of cracks (direction and propagation rate).
(c) Critical crack length.
(d) Other appropriate items.

8.3 Repair of fatigue damage

Safety against fatigue can be improved by increasing the fatigue strength of and reducing the stress in objective joints. To judge whether the repair is necessary or not, it is important to consider the expected behavior of the crack if it is left. To repair the damage by an appropriate method, it is necessary to study fully the cause of the fatigue crack.

A fatigue assessment of the structure after repair should be conducted. Care should also be taken that any repair does not have a bad effect on the fatigue strength of another part of the structure.

COMMENTARY

1. Scope and limitations

1.1 In the former Recommendations (published in 1974)[1], fatigue assessment was made unde the condition that fatigue cracks do not occur during the design life. In thes Recommendations, however, the fatigue limit state is defined as the state at which th strength and/or function of a structure is lost by fatigue cracking and crack propagation. Thi conception is taken because the definition of fatigue crack initiation is vague, and the fatigu life of welded components which are especially liable to suffer fatigue damage is mainl occupied by the fatigue crack propagation period. A further reason is that the fatigue strengt of joints, which forms the basis of a fatigue assessment, is normally evaluated in terms of a fatigue life up to failure.

1.2 These Recommendations cannot be applied to fatigue assessment in environments of severe corrosion and/or high temperatures, because the mechanism of fatigue damage progression under these conditions is essentially different from that in the normal atmosphere at ambient temperature.

1.3 The fatigue assessment and remaining life calculation for existing structures can be made by evaluating the fatigue damage of the structure or components up to the present time on the basis of these Recommendations. In this type of assessment, careful attention should be paid to the actual loading to structures in the past and that expected in the future. Based on this. the fatigue loads should be determined.

1.4 The allowable stress range in the Recommendations should be determined on the basis of fatigue test results obtained for the high-cycle region. The Recommendations cannot therefore apply in conditions where maximum and/or minimum nominal stress exceed the yield stress.

1.6 A limit state design procedure generally covers the ultimate limit state, service limit state and fatigue limit state. In these Recommendations, design procedures for the fatigue limit state are presented.

3. Materials, joints and their required qualities

3.2 Steel

Properties of steel plates, cables and high strength bolts are to be equivalent to or better than those given in JIS (Japan Industrial Standards).

3.3 Materials for connections

Properties of materials for welding and high strength bolted connections are to be equivalent to or better than those given in JIS.

3.4 Connection methods and required qualities

Whenever non-destructive tests are required they should be made in accordance with the methods described in JIS or by methods superior to these standards, because in the strength category of joints in Table 4.4 it is assumed that the joint is sound.

Because the tips of planar defects such as a weld crack or lack of fusion are sharp, the fatigue strength of joints having these defects is considerably lower than joints having no defects. Therefore, if joints have planar defects, the basic allowable stress range in Table 4.4 must not be used. Defects of class 3 (a crack or a similar defect) specified in JIS Z 3104, which describes methods of radiographic test and classification of radiographs for steel welds, are regarded as planar defects. When planar defects occur, it is desirable to remove these defects. In such cases, the fatigue assessment may be carried out by using fatigue test results of specimens representing similar welds and defects, or by using the fatigue strength obtained from a fatigue crack propagation analysis as described in the Appendix.

Compared to planar defects, the effects on fatigue strength of volumetric defects such as blowholes, slag inclusions or undercuts are insignificant because of their round shape. When these type of defects are repaired carelessly, it is even possible that new defects will occur which reduce the fatigue strength even further.

Full penetration butt welded joints are usually inspected to detect inner defects, such as blowholes or slag inclusions, according to JIS Z 3104. Fatigue strength of butt welded joints with ground flush surfaces are known to have an interrelation with inner defect ratios (which are ratios of the area of defects to the view point area). In JIS Z 3104, defects are classified as Grade 1, Grade 2, Grade 3 and Grade 4 according to the number and size of defects. The relationship between these Grades and the defect ratios are shown as follows[2)].

Grade 1 : $\leqq$ 1% Grade 2 : $\leqq$ 3% Grade 3 : $\leqq$ 6% Grade 4 : $>$ 6%

The relationship between fatigue strength (net section stress range) and defect ratios of blowholes and slag inclusions is shown in Fig.C. 3.1 and 3.2[2)]. The reduction in fatigue strength due to defect ratios of blowholes and slag inclusions are almost the same, so that it is thought to be not necessary to distinguish blowholes from slag inclusions. Even if it is taken into account that the net section stress range was used in these figures and the strength category is based on the gross section stress range, it is concluded that joints with Grade 2 defects in JIS Z 3104 should satisfy strength category B and those with Grade 3 defects in JIS Z 3104 should satisfy strength category C safely.

Here, X-ray radiographic tests are described as an example of a non-destructive test, but if the same results can be obtained by another test, the grade of defects may be judged by the defect ratios obtained from this test. When the relationship between non-destructive tests and fatigue strength is distinct, it is desirable that the required quality should be determined in accordance with this relation.

It is known from recent research that the fatigue strength of longitudinal partially penetrated welded joints or longitudinal fillet welded joints is influenced by defect size at the root (refer to Fig.C. 3.3[3)]). This is because these type of joints have roots which are not fused in welding, which is different from transverse butt welded joints. Therefore, considering the effects of defects in this type of joint on fatigue strength, when it can be ensured that the size of defects is smaller than a given size, the strength category of this type of joints is graded as C ; a rise from D. The limiting size of defects, which is the same as the standard followed by the Honshu-shikoku Bridge Authority[4)], is 1.5mm in width and 4.0mm in height.

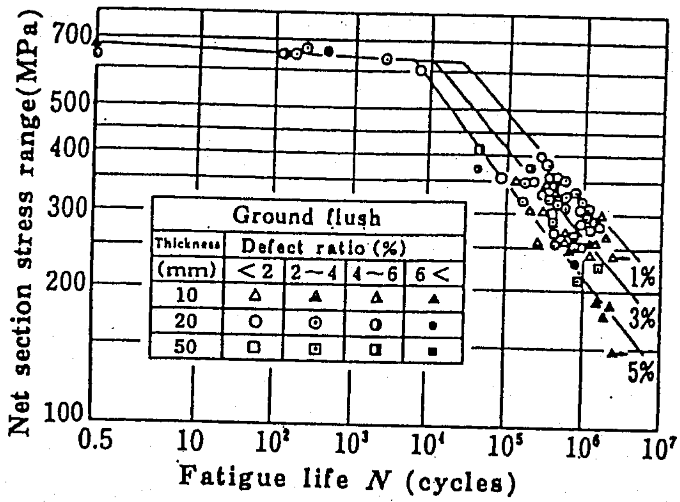


Fig.C.3.1 The relationship between defect ratios and fatigue strength (blowholes)[2)]

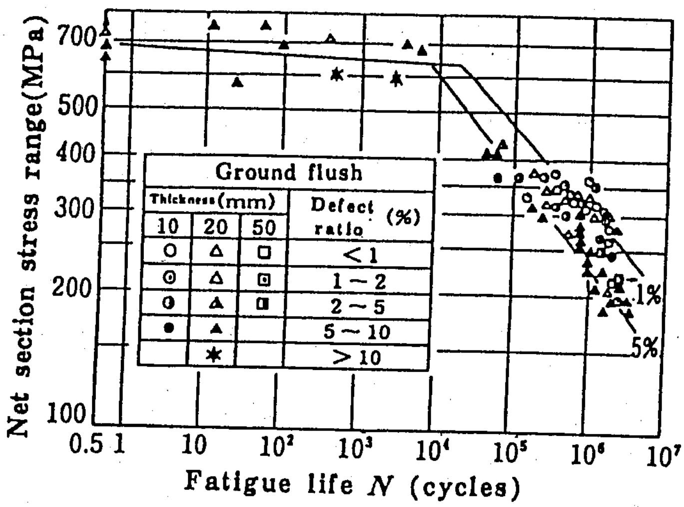


Fig.C.3.2 The relationship between defect ratios and fatigue strength (slag inclusions)[2)]

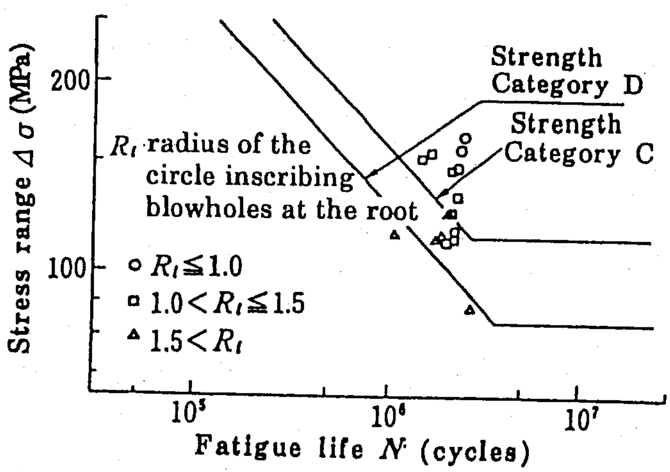


Fig.C.3.3 Effects of blowholes on fatigue strength of longitudinal partially penetrated welded joints [3)]

Table.C.3.1 The relationship between depth of undercut and fatigue strength (transverse butt welded joints) [5)]

Depth of undercut (mm)		0.0	0.3	0.6	0.9
Fatigue strength at 2×10^6 cycles (MPa)	μ	181	152	136	83
	μ -2σ	108	129	111	63

μ : mean , σ : standard deviation

Table.C.3.2 The relationship between depth of undercut and fatigue strength (non-load carrying fillet welded joints) [5)]

Depth of undercut (mm)		0.01 ~0.11	0.13 ~0.25	0.27 ~0.32	0.43 ~0.58
Fatigue strength at 2×10^6 cycles (MPa)	μ	149	129	118	133
	μ -2σ	139	113	96	94

μ : mean , σ : standard deviation

The effect of depth of undercut on fatigue strength is significant, so that the allowable size of undercut is given by its depth. The relationship between the depth of undercut and fatigue strength of a transverse butt joint and non-load-carrying fillet welded joint are shown in Table C.3.2 and C.3.3[5)], respectively. From these tables, the basic fatigue allowable stress range at 2×10^6 cycles shown in Table 4.4 is satisfied in joints with an undercut depth less than 0.5mm. Therefore, the allowable depth of undercut is set at 0.5mm.

4. Fatigue strength and joint types

4.1 Main factors controlling fatigue strength

The fatigue life of a welded joint depends on the stress at the fatigue crack initiation point and the stress distribution in the section where the fatigue crack propagates. In these Recommendations, the main controlling factor for these stresses and stress distributions is considered to be the intensity of nominal stress and the type of joint.

The intensity of stress and its distribution also depends on the weld geometry and the shape of the weld toe in addition to the type of joint. Therefore, the classification of joints differs with the smoothness of the weld toe for non-load-carrying cruciform joints. The neighboring higher strength category is given for joints which received weld toe finishing treatment. The methods of weld toe finishing are, for example, grinding, TIG dressing and peening. Any other methods which are confirmed to increase the fatigue strength similar to the method described above are permitted for use in finishing weld toes.

4.2 Fatigue design curves (design $\Delta\sigma$-N and $\Delta\tau$-N relationship)

(1) Constant amplitude cut-off limits are given referring to the fatigue limits obtained fr the results of constant amplitude fatigue tests. The design $\Delta\sigma$-N curves are given reflecting results of tests which show that the number of stress cycles to the fatigue limit is larger in low fatigue strength joints.

Once fatigue cracks initiate under a variable amplitude stress, lower stress ranges below t constant amplitude cut-off limit become effective in causing fatigue damage. Therefore, t variable amplitude cut-off limit is given with consideration of this phenomenon. However, te results relating to the variable amplitude cut-off limit are very scarce, so that fatigue cra propagation analyses for typical types of joints using the method given in the Appendix are utilize to determine the variable amplitude cut-off limit[6].

The following relationships are applicable between the constant amplitude and variab amplitude cut-off limit, $\Delta\sigma_{ce}$, $\Delta\sigma_{ve}$ and the basic allowable stress range at 2×10^6 cycles, $\Delta\sigma_f$:

$$\Delta\sigma_{ce} = 1.357 \times 10^{-1} \cdot \Delta\sigma_f^{1.396}$$
$$\Delta\sigma_{ve} = 6.295 \times 10^{-2} \cdot \Delta\sigma_f^{1.396} \qquad \text{(c. 4.1)}$$

where $\Delta\sigma_{ce}$ and $\Delta\sigma_{ve}$ are the basic allowable stress ranges corresponding to 2×10^6 and 10^7 cycle respectively, for joints which have a basic allowable stress range of more than 155MPa at 2×10^6 cycles.

(2) The fatigue design $\Delta\sigma$-N curves are given for strength categories from A to H. In the former Recommendations, the fatigue design $\Delta\sigma$-N curves were given for typical kinds of joints In these Recommendations, eight fatigue design $\Delta\sigma$-N curves have been prepared, which are at equal spacing, and determined by considering a range of fatigue strength for all the objective joints (from 190MPa to 40MPa). The strength category for such joints is determined by comparing the fatigue strength of the joint to the relevant fatigue design $\Delta\sigma$-N curve.

In these Recommendations, the fatigue design $\Delta\sigma$-N curves for cables and high tensile strength bolts subject to normal stress are also indicated. These are based on the fatigue strength for each type of joint (lower limit of fatigue strength or the 97.7 percent probability of survival).

(3) In the former Recommendations, the slope of $\Delta\sigma$-N curves was selected to be -0.18. In these Recommendations, the slope has been taken to be -1/3 for welded joints.

The fatigue design $\Delta\sigma$-N curves were given mainly referring to the fatigue test results of usual-sized specimens. The slopes of $\Delta\sigma$-N curves obtained from fatigue tests on these specimens are usually less than -1/3. However, test results for full-sized or large-sized specimens show a slope of about -1/3.

Fatigue life is divided into the fatigue crack initiation life and the fatigue crack propagation life. High tensile residual stresses usually exist in welded locations, whose magnitude is about yield stress of the steel used, and fatigue damage is frequently initiated in welded locations. Therefore, the slope of the fatigue design $\Delta\sigma$-N curves was determined to be -1/3 by considering the results for the fatigue crack initiation life in areas of high tensile residual stress[7]. The results of the fatigue crack propagation analysis given in the Appendix also suggest the new value of slope.

In Fig.C. 4.1, fatigue test results for typical types of joints and the corresponding fatigue design curves are shown. It is easy to confirm that the slope of $\Delta\sigma$-N curves for the lower limit of the fatigue test results is about -1/3.

For cables and high strength bolts (Table 4.4(g)), the slope of the $\Delta\sigma$-N curves was determined to be -1/5 by referring to fatigue test results on very high stress ratio conditions for very high tensile strength steels (the strength level was very high compared with other joints).

(4) Assessment of fatigue strength under variable amplitude stress by using a straight line

extending below the variable amplitude cut-off limit gives a conservative evaluation.

4.3 Strength categories of joints

The joints were classified according to their fatigue strength determined from Japanese fatigue test results[8),9)]. Fatigue test data obtained in the U.S. was also considered[10)]. The data was re-examined according to the weld quality specified in Chapter 3 and the size of the specimens. This was because results of large scale fatigue tests carried out after the former Recommendations were published revealed a reduction in fatigue strength compared with the usual-sized test specimens (thickness of 10 to 15mm and width of about 100mm) on which the former Recommendations were based. It appears that small scale specimens do not reflect welding residual stresses and/or bi-axial or tri-axial stresses which normally exist in actual steel structures.

Fatigue strengths at 2×10^6 cycles for various joints (50% and 97.7% probability of survival) obtained by statistical analysis are listed in Table C.4.1. Test data for typical joints and the corresponding fatigue design curves are also shown in Fig.C. 4.1.

Table C.4.1(a) Fatigue strengths at 2×10^{6} cycles (MPa) (non-welded joints)

joint number	number of data	probablity of survival 50%	97.7%	basic allowable stress range
1.(1)	204	360	219	190
1.(2)	453	323	224	155
2.(1)	46	243	197	155
4.	42	184	126	125
5.(1),(3)	11	188	158	155,125
5.(2),(4)	12	165	144	125,100

Table C.4.1(b) Fatigue strengths at 2×10^{6} cycles (MPa) (transverse butt welded joints)

joint number	number of data	probablity of survival 50%	97.7%	basic allowable stress range
1.	363	306	189	155
3.(1)	886	166	122	100
3.(3)	65	139	88	65

Table C.4.1(c) Fatigue strengths at 2×10^{6} cycles (MPa) (longitudinal welded joints)

joint number	number of data	probablity of survival 50%	97.7%	basic allowable stress range
1.(1)	59	244	149	155
1.(2)	150	235	164	125
2.	107	234	144	125,100
3.(beam)	97	164	108	125,100
3.(plate)	24	223	163	125,100

Table C.4.1(d) Fatigue strengths at 2×10^{6} cycles (MPa) (cruciform joints)

joint number	number of data	probablity of survival 50%	97.7%	basic allowable stress range
2.	219	175	116	100
3.	341	126	82	80
8.	57	84	46	40

Table C.4.1(e) Fatigue strengths at 2×10^{6} cycles (MPa) (gusset joints)

joint number	number of data	probablity of survival 50%	97.7%	basic allowable stress range
1.(2)	110	96	75	65
3.	11	90	77	50
3.(beam)	55	74	60	50
5.(3)	13	125	82	65
6.(1)	25	93	66	50
6.(1)(beam)	10	122	110	50
6.(2)	31	68	45	40
6.(2)(beam)	61	61	45	40

Table C.4.1(f) Fatigue strengths at 2×10^{6} cycles (MPa) (other welded joints)

joint number	number of data	probablity of survival 50%	97.7%	basic allowable stress range
1.(2)	141	106	87	65
2.(1)	8	175	146	100
2.(2)	211	69	47	50
3.(1)	59	118	86	80

The basic allowable stress ranges at 2×10^6 cycles specified in these Recommendations were compared with those of the former Recommendations and other fatigue design codes[11)-14)], as shown in Table C.4.2. Note that different slopes of the design $\Delta\sigma$-N curves appear depending on the type of code, and that the former Recommendations only dealt with mild steel. Note also that the Design Specifications for Japan National Railway Structures (1983)[11)] changes the slope of the design $\Delta\sigma$-N curves depending on the tensile strength of the steel and even the basic allowable strength at 2×10^6 cycles for some joints. The former Recommendations specified the fatigue strength at one million cycles, which is listed in Table C.4.2. The design $\Delta\sigma$-N curves in the former Recommendations were based on the fatigue crack initiation life, which corresponds to a half of the fatigue life of the specimens.[1)]

In these Recommendations, the effect of the static tensile strength of steel on the fatigue strength of joints was not considered. For some joints with higher fatigue strength, the fatigue

Table.C.4.2(a) Comparison of basic allowable stress range at 2×10^6 cycles in these Recommendations and other codes

(non-welded joints)

joint number	these Recommendations	former Recommendations	Japan National Railways	AASHTO	BS 5400	ECCS
	m = 3	m = 5.6	m = 5	m = 3	m = 3, 3.5, 4	m = 3
1.(1)	190 (less than 50s)	176	150 (less than 50s)	165	150	160
(2)	155 (less than 100s)				150	160
(3)	125				125	140,125
2.(1)	155	176	150 (less than 50s)	165	150	160
(2)	155 (less than 100s)				150	160
(3)	125				125	140,125
3.	155	–	–	–	–	160
4.	125	118	124	–	150*	
5.(1)	155	118 (r/d≧1/5)	150	–	150*, 125*	–
(2)	125		150,124	–		–
(3)	125		–	–	150 (machine finishing)	–
(4)	100		–	–	125 (automatic flame cutting)	–
6.(1)	155	176 (net section)	150	123	125	140
(2)	125		150-4(n-4) n:number of bolts in the stress direction			
(3)	100		103			
7.	155 (net section)	176 (net section)	124 (net section)	123 (net section)	125 (net section)	140 (net section)
8.	155 (net section)	176 (net section)	124 (net section)	–	125 (net section)	140 (net section)

*including stress concentration

Table.C.4.2(b) Comparison of basic allowable stress range at 2×10^6 cycles in these Recommendations and other codes

(transverse butt-welded joints)

joint number	these Recommendations	former Recommendations	Japan National Railways		AASHTO	BS 5400	ECCS
			mild steel	high strength steel			
	m = 3	m = 5.6	m = 4	m = 3	m = 3	m = 3, 3.5	m = 3
1.	155	176	150	150	123	125	112
2.	125	147	–	–	–	–	–
3.(1)	100	118, 98	–	–	89	91 (downhand welding)	90, 80
(2)	100	–	–	–	89	–	(36)
(3)	65	–	–	–	–	68	71
(4)	65	–	–	–	–	–	36

Table.C.4.2(c) Comparison of basic allowable stress range at 2×10^6 cycles in these Recommendations and other codes (longitudinal welded joints)

joint number	these Recommendations	former Recommendations	Japan National Railways mild steel	Japan National Railways high strength steel	AASHTO	BS 5400	ECCS
	m = 3	m = 5.6	m = 4	m = 3	m = 3	m = 3	m = 3
1.(1)	155	176	150	124	123	150	–
(2)	125	147, 118				125	125(automa weldin 112(automat weldin 100(manual weldin,
2.	125, 100	147, 118	150	124	123	–	100
3.	125, 100	147, 118	150	124	123	125	125(automati welding 112(automati welding 100(manual welding
4.	80	–	–	–	–	125	112(automati welding
5.	80	–	–	–	–	80	80
6.	50	–	–	–	–	68	71
7.(1)	100	–	125 (r/d≧1/3)	125	–	–	–
(2)	80		125 (1/5≦r/d<1/3)	103	–	–	–

strength actually increases as the static strength of the steel becomes higher[15),16)]. However, for some joints with lower fatigue strength, the fatigue strength does not depend on the static tensile strength of steel, and sometimes decrease as the tensile strength becomes higher[16)]. This can also be implied from the fact that fatigue crack propagation rates are not affected by the static tensile strength of the steel[17),18)].

For joints which are not found in Table 4.4, it is recommended that allowable stress ranges should be established from fatigue test data by using specimens adequately designed for the joints of interest. In this case caution should be taken so that the specimens are appropriate for the joints of interest in terms of the size and/or the residual stresses. In order to include the effect of high tensile residual stresses in the fatigue tests, a testing method has been proposed where the applied maximum stress is kept close to the yield stress of the steel[19)].

For some structures, transverse butt welded joints with partial penetration and load-carrying fillet welded joints with small leg length (that is $s/t \leqq 0.4$, where s : leg length of fillet weld, t : plate thickness) are used. The strength categories listed in Table 4.4 should not be used for these special joints. Fatigue strength for such joints should be determined based on fatigue test results and the fatigue crack propagation analysis described in the Appendix.

(a) Non-welded joints (Table 4.4 (a))

When punched holes were adopted in high strength bolted joints, the strength category is reduced by one grade from that of joints with drilled holes. The surface of the punched hole is normally rougher than that of a drilled hole, which leads to the lower fatigue strength[20].

For friction type high strength bolted joints, fatigue strength is altered according to the number of bolts in the direction of stress. This is due to the fact that stress concentration increases at the end of bolted joints with an increase in the number of bolts in the direction of stress[21].

For bearing type high strength bolted joints, the number of bolts in the direction of stress, n_b , is limited to less than four, because no fatigue test data is available for joints with more than four. It is rare to use more than four bolts for this type of joint because of the tolerance of the bolt holes, and limitation of the number of bolts in the direction of stress is unlikely to lead to problems in practice.

(c) Longitudinal welded joints (Table 4.4 (c))

The strength category of complete penetration welded joints is B or C for joints with ground flush or as-welded weldments, respectively. When a smooth surface of weld bead is achieved by automatic welding, such as submerged arc welding, strength category B may be adopted even if ground flush treatments are not used. However, for partial penetration welded joints or fillet welded joints, strength category B should not be used regardless of the surface condition of the bead even if ground flush, because fatigue cracks may also initiate from the weld root.

Fatigue test results from partial penetration longitudinal groove welded joints and longitudinal fillet welded joints are shown in Figs 4.1 (c-2) and (c-3), respectively. Some data falls below strength category C. This is because relatively large defects (blowholes) exist in the roots of these joints. Therefore, these joints are classified as strength category D. When it can be assured that a defect of more than a specified size (width 1.5mm, height 4mm) does not exist in the weld,

Table.C.4.2(d) Comparison of basic allowable stress range at 2×10^6 cycles in these Recommendations and other codes

(cruciform joints)

joint number	these Recommendations	former Recommendations	Japan National Railways mild steel	Japan National Railways high strength steel	AASHTO	BS 5400	EECCS
	m = 3	m = 5.6	m = 4	m = 3	m = 3	m = 3	m = 3
1.	100	–	103	103	–	–	–
2.	100	118	124	124	–	–	–
3.	80	98	103	78	89	80($\tau>0.5\sigma$) 68($\tau\leqq0.5\ \sigma$)	80 ($t\leqq$ 12mm) 71 ($t>$ 12mm)
4.	80		103	78	89	80($\tau>0.5\sigma$) 68($\tau\leqq0.5\ \sigma$)	80 ($t\leqq$ 12mm) 71 ($t>$ 12mm)
5.(1) (2)	65 50	– –		– –	– –	– –	71 –
6.(1) (2) (3) (4)	100 100 80 65	– 118 98 –	103 103 103 –	103 103 78	– – – –	– – 68 –	71 50, 45
7.(1) (2) (3) (4)	80 80 65 65	– – 78 78	103 103 103 103	103 103 78 78	– – – –	– – 60 60	71
8.	40	49	80 (shear)	64	–	43	36
9.(1) (2)	40 40						– 40, 36

Table.C.4.2(e) Comparison of basic allowable stress range at 2×10^6 cycles in these Recommendations and other codes

(gusset joints)

joint number	these Recommendations	former Recommendations	Japan National Railways		AASHTO	BS 5400	ECCS
			mild steel	high strength steel			
	m = 3	m = 5.6	m = 4	m = 3	m = 3	m = 3	m = 3
1.(1) (2)	80 65	– 78	103 78 (smooth weld profile)	103 78	– 69 (L ≤ 100mm)	80(τ >0.5σ) 68(τ ≤0.5σ) (L ≤ 150mm)	71
2.	80	–	–	–	123(r>600mm) 89(r>150mm) 69(r> 50mm) 55(r ≤50mm)	–	–
3.	50	78	78 (smooth weld profile)	78	55	60	50
4.(1) (2)	65 50	– 78	– –		– 55	– 60	– 50
5.(1) (2) (3)	100 80 65	98 (r>20mm)	124 124 103	124 103 –	123(r>600mm) 89(r>150mm) 69(r> 50mm) 55(r ≤50mm)	–	90(1/3<r/d) 71(1/6<r/d<1 45(r/d<1/6)
6.(1) (2)	50 40	98 (r>20mm) 59	– –	– –	55 55	– 50	– –
7.	40						

Table.C.4.2(f) Comparison of basic allowable stress range at 2×10^6 cycles in these Recommendations and other codes

(other welded joints)

joint number	these Recommendations	former Recommendations	Japan National Railways		AASHTO	BS 5400	ECCS
			mild steel	high strength steel			
	m = 3	m = 5.6	m = 4	m = 3	m = 3	m = 3	m = 3
1.(1) (2)	80 65	–	103 78	103 –	– 55(t ≤ 20mm) 40(t> 20mm)	– 60	– 55(t ≤ 20mm) 36(t> 20mm)
2.(1) (2)	100 50	118 –	103 78	103 –	– 55(t ≤ 20mm) 40(t> 20mm)	– 60	– 55(t ≤ 20mm) 36(t> 20mm)
3.(1) (2)	80 80 (shear)	–	78 (smooth weld profile) – (smooth weld profile)	78 –	89 –	80(τ >0.5σ) 68(τ ≤0.5σ) 100 (shear)	80 80 (shear)
4.(1) (2) (3) (4)	40 40 40 80 (shear)	49 49 49 –	– – 80 (shear) 80 (shear)	– – 64 80	40 40 – 62 (shear)	50 50 43 –	63 45 – 80 (shear)

Table.C.4.2(g) Comparison of basic allowable stress range at 2×10^6 cycles in these Recommendations and other codes (cables and high strength bolts)

joint number	these Recommendations	former Recommendations	Japan National Railways	AASHTO	BS 5400	ECCS
	m = 5	–	–	–	–	m = 3
1.(1)	270	–	–	–	–	–
(2)	200	–	–	–	–	–
2.(1)	270	–	–	–	–	
(2)	200	–	–	–	–	
(3)	150	–	–	–	–	
3.(1)	65	–	–	–	–	36
(2)	50	–	–	–	–	36

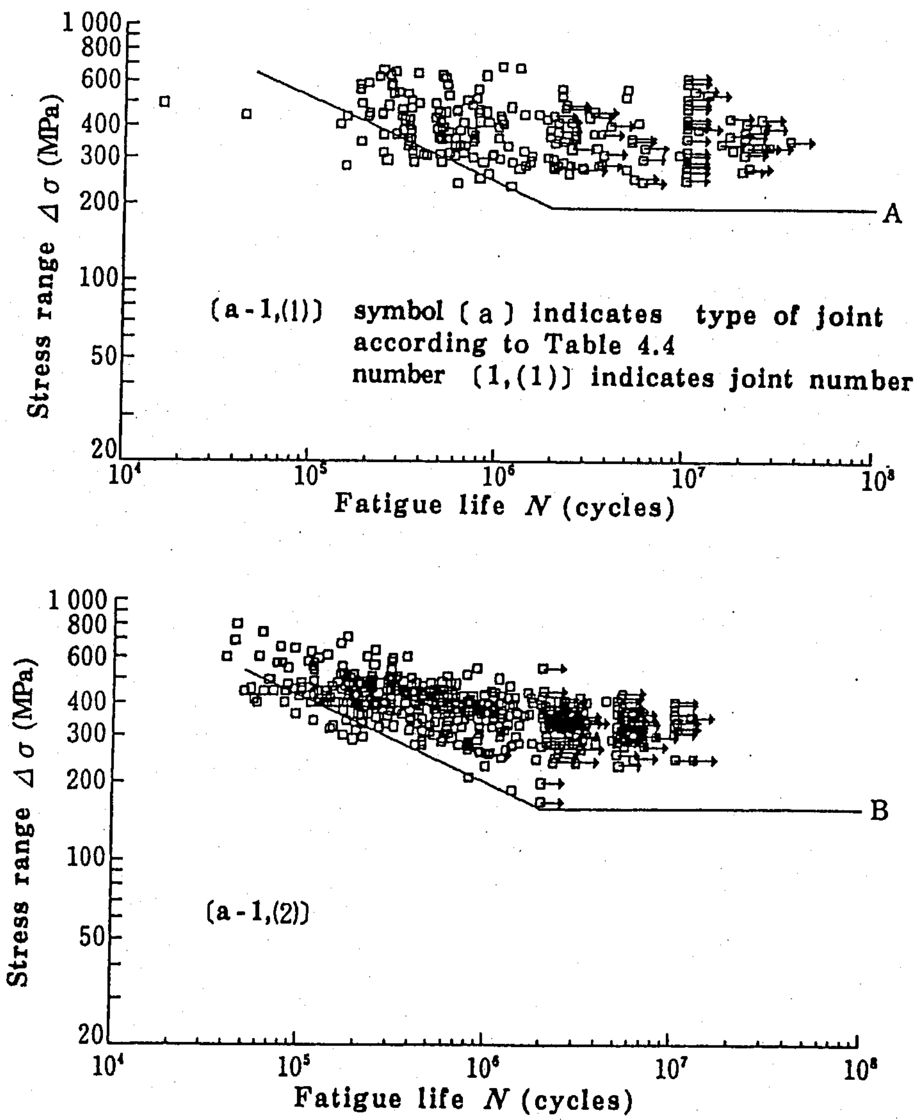
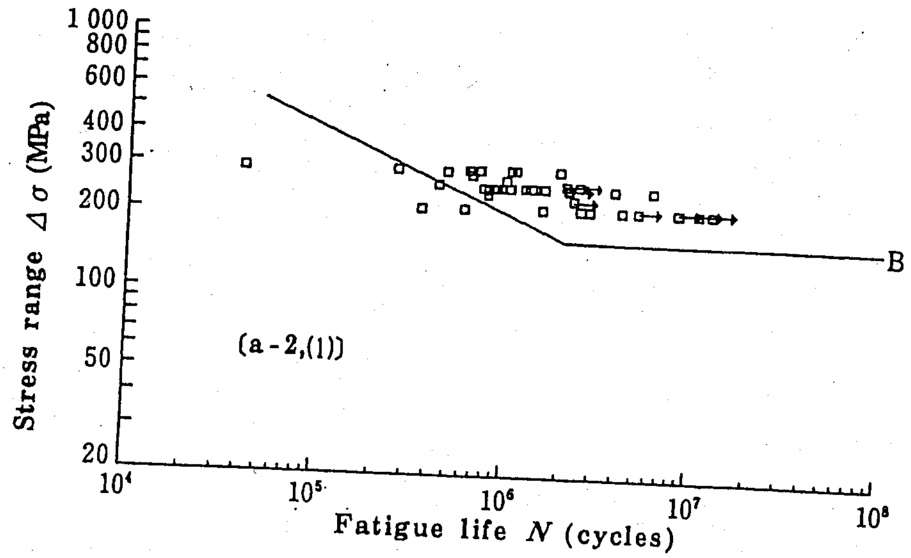
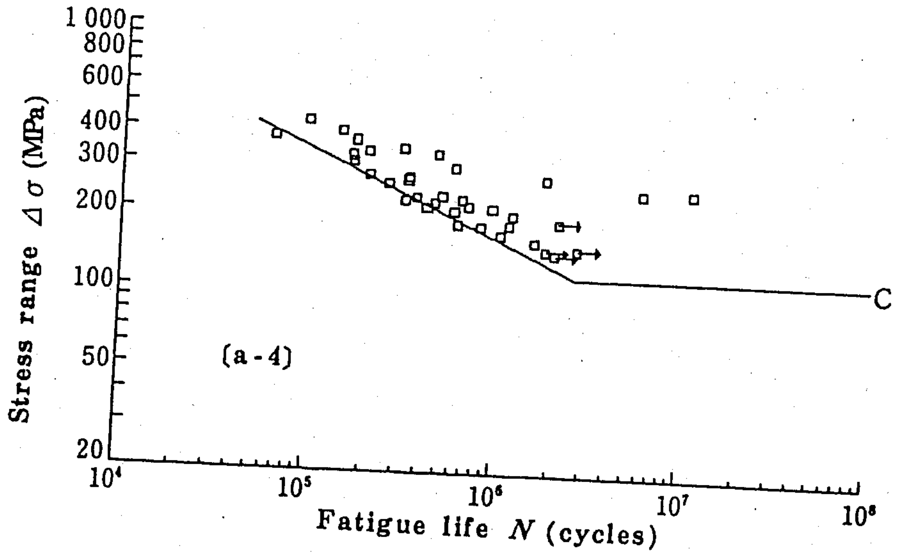


Fig.C.4.1 Fatigue test results and fatigue design curves





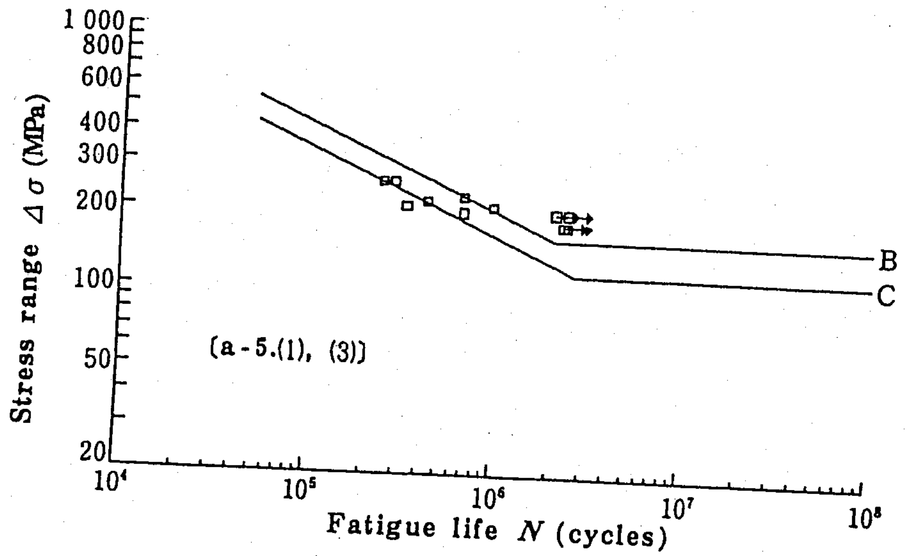


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)

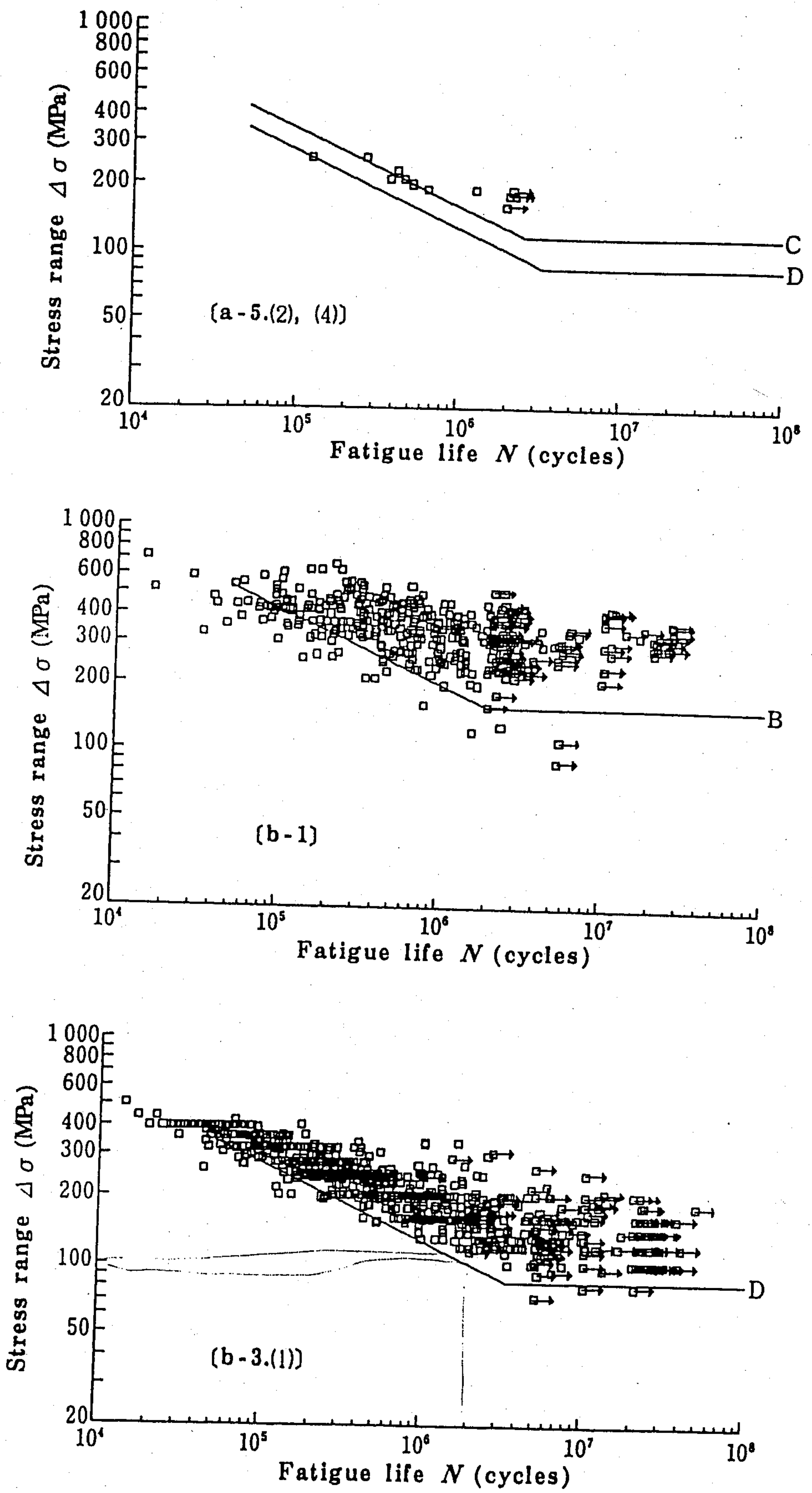
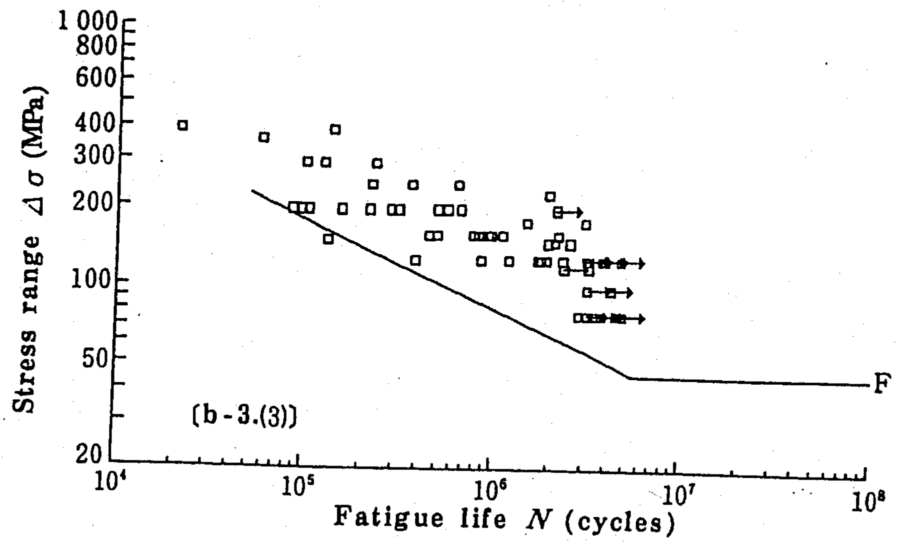
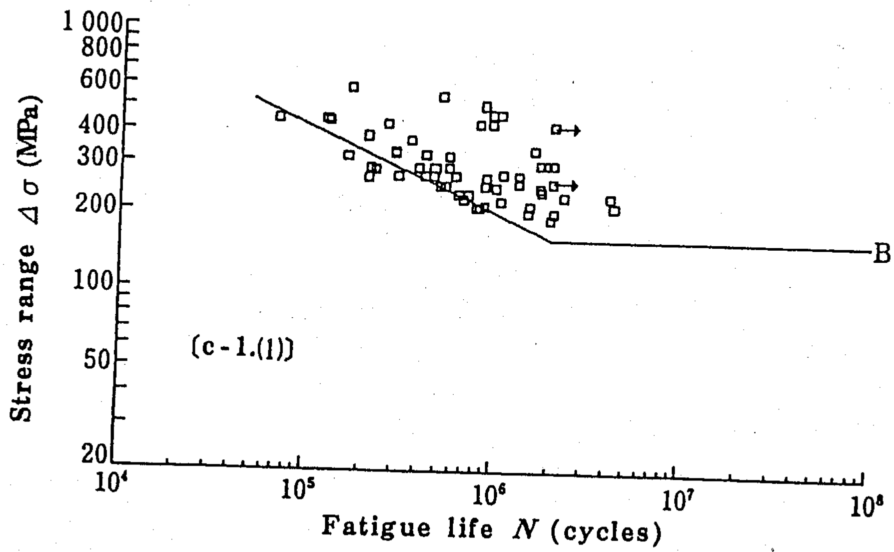


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)





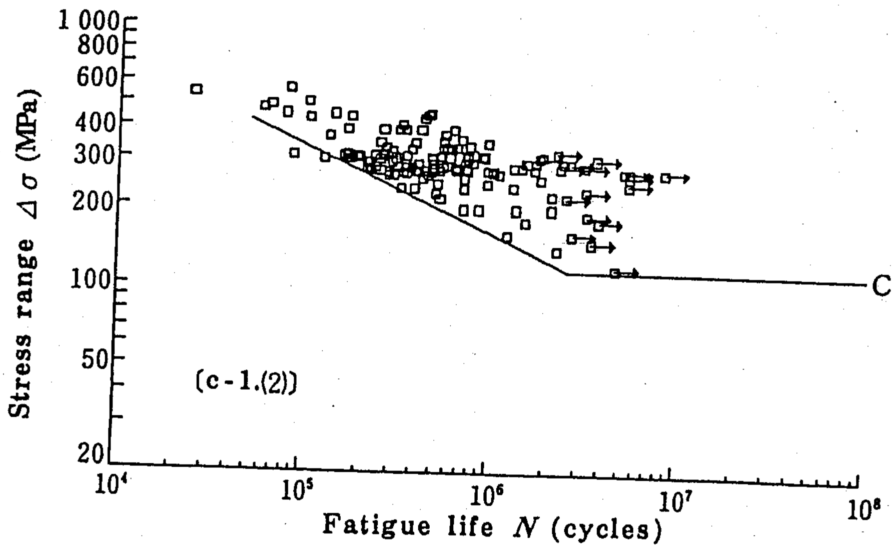
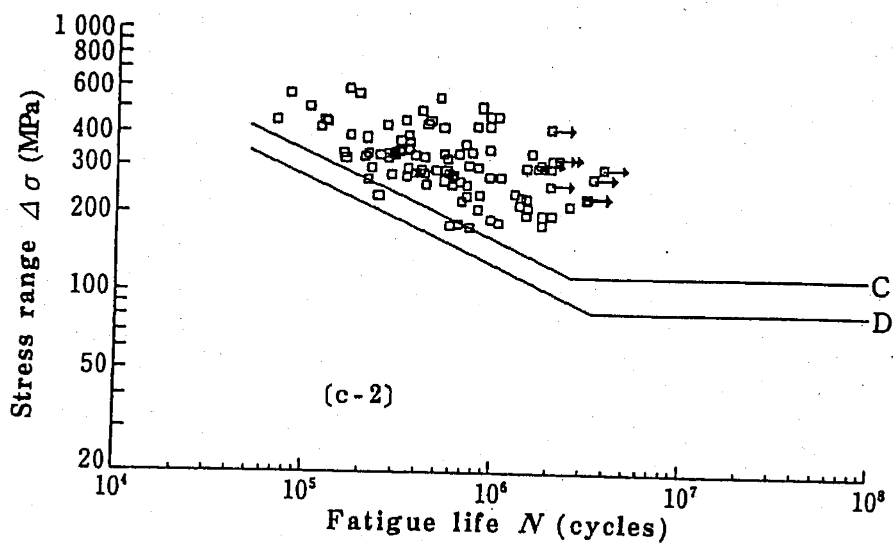
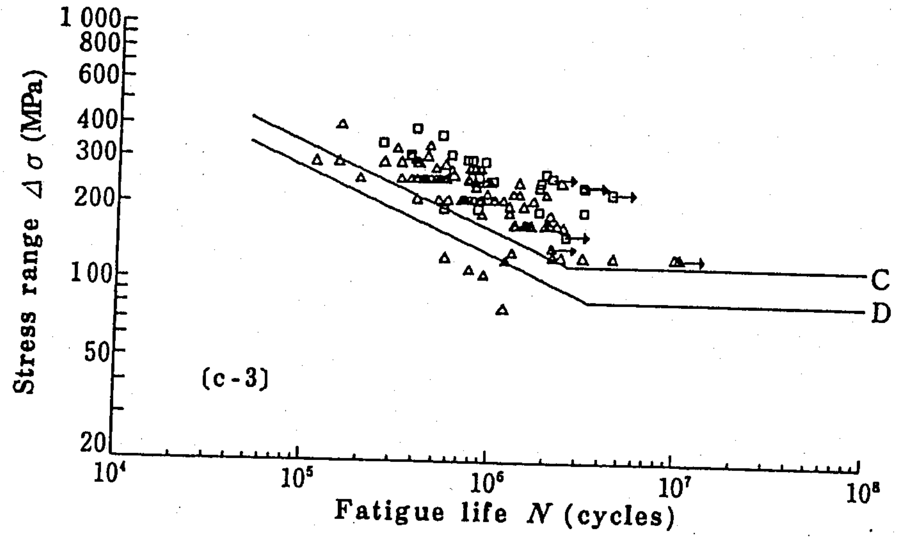


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)





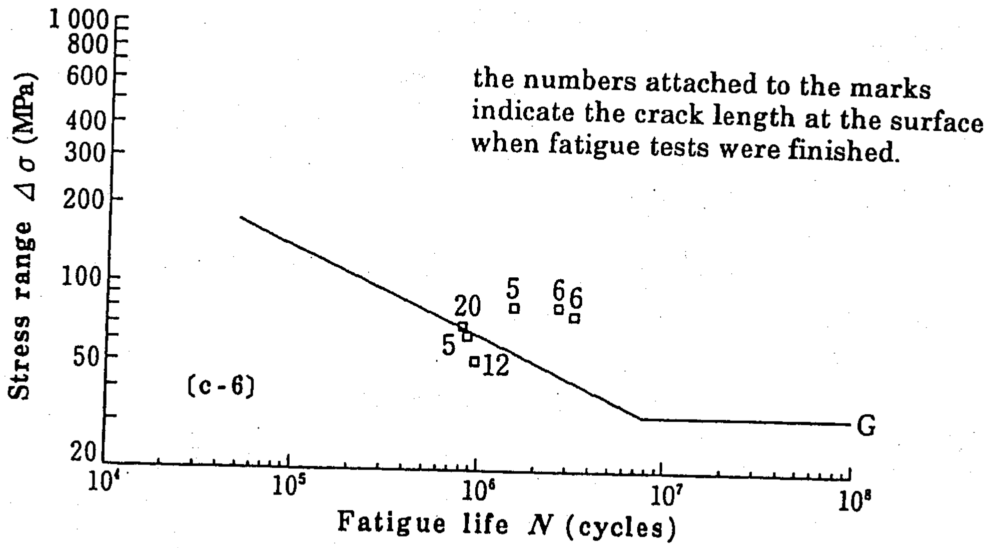
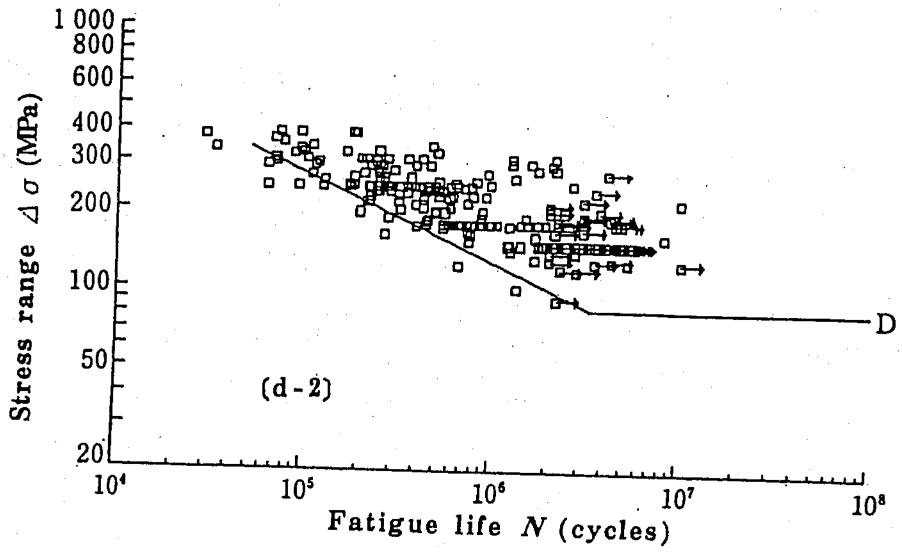
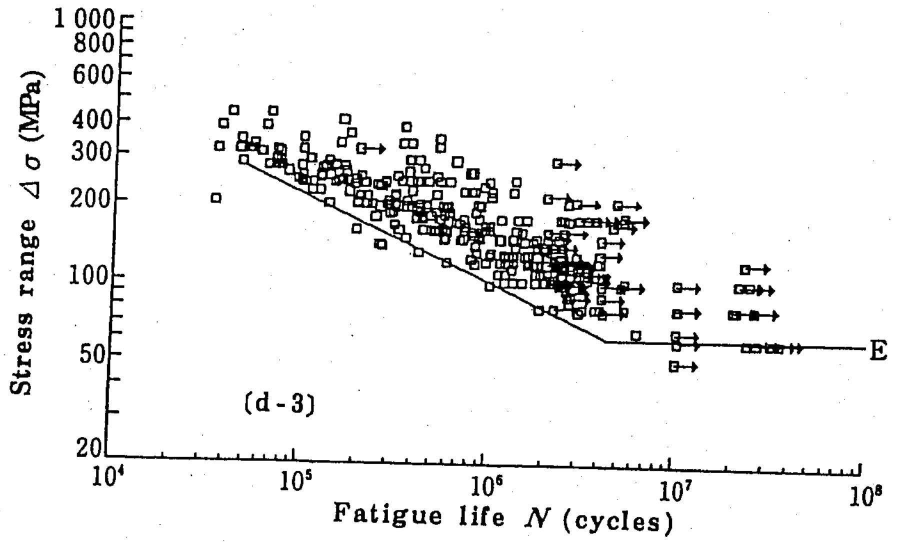


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)





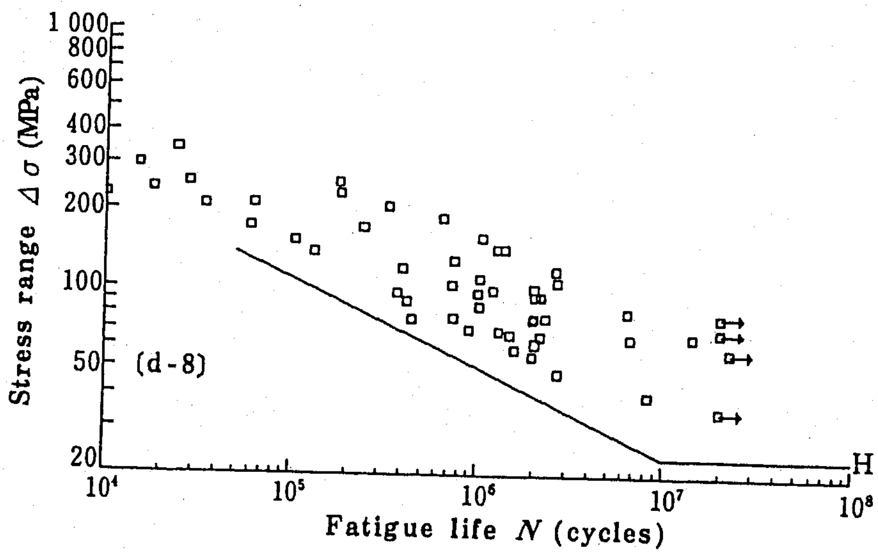
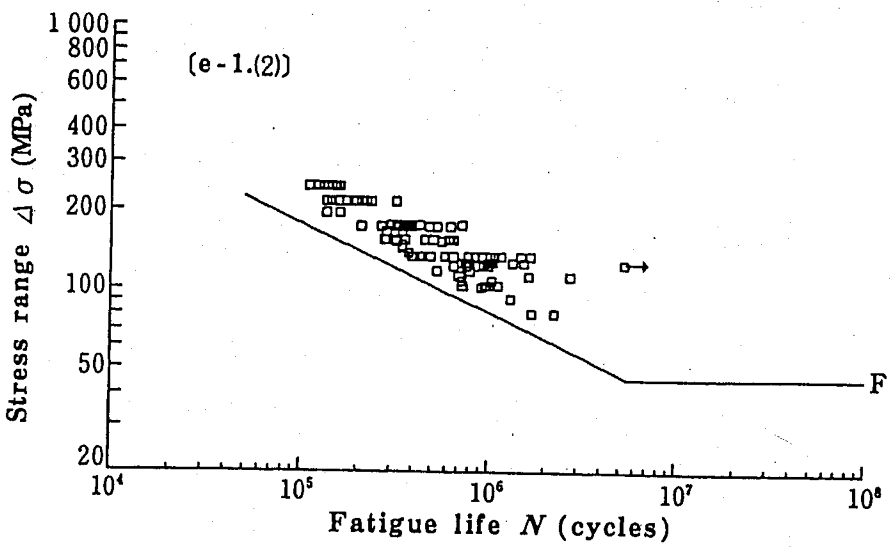
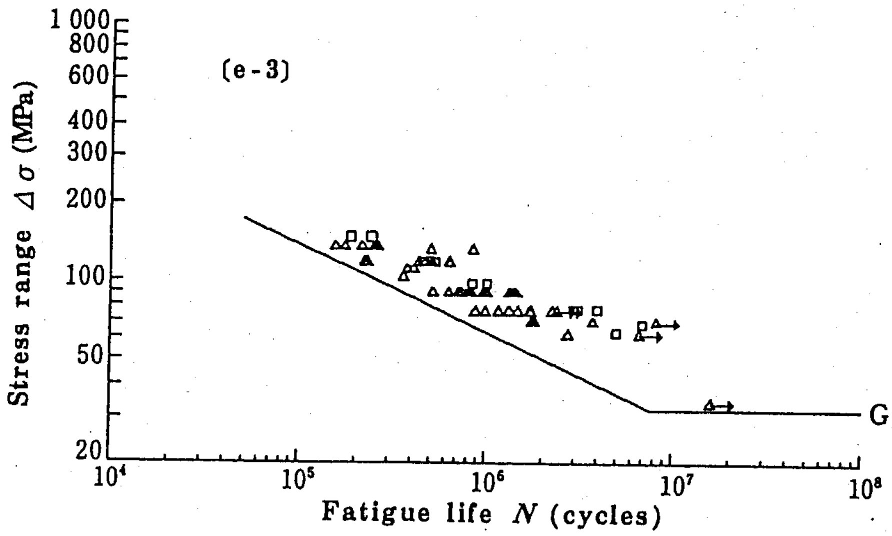


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)





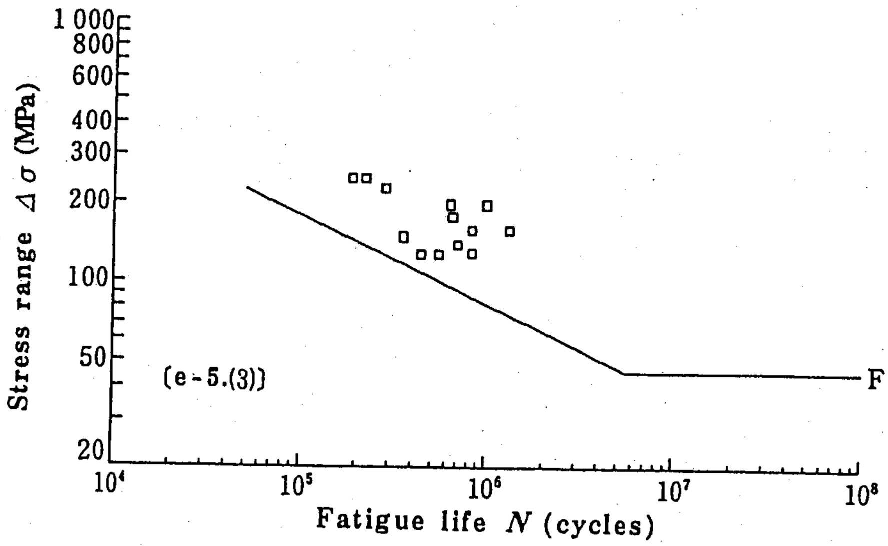
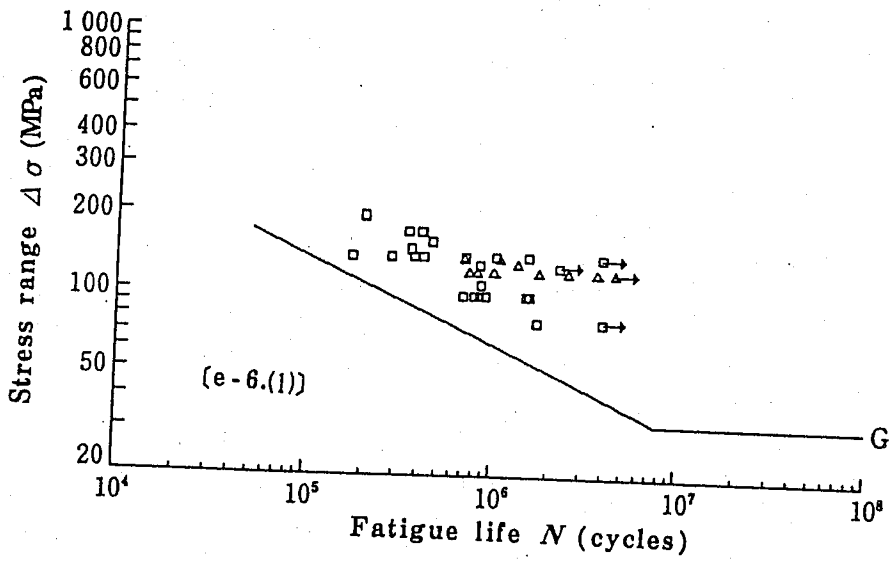
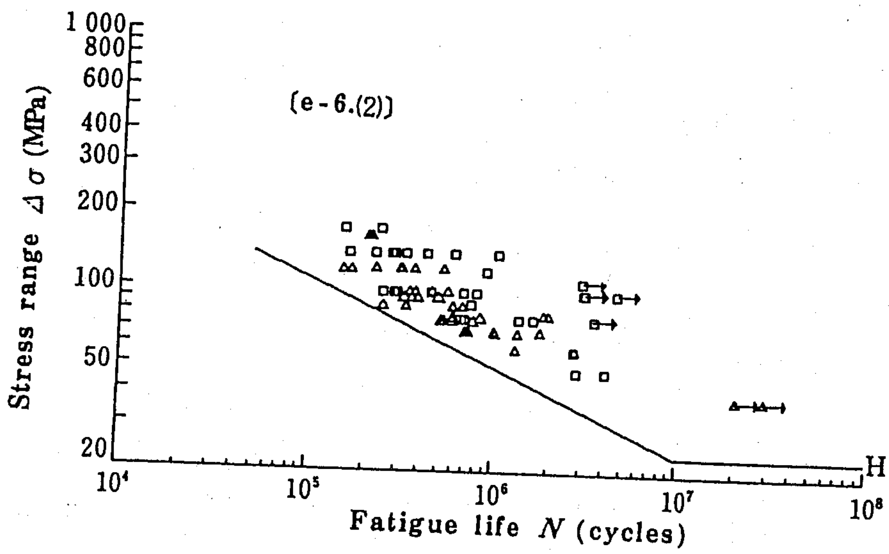


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)





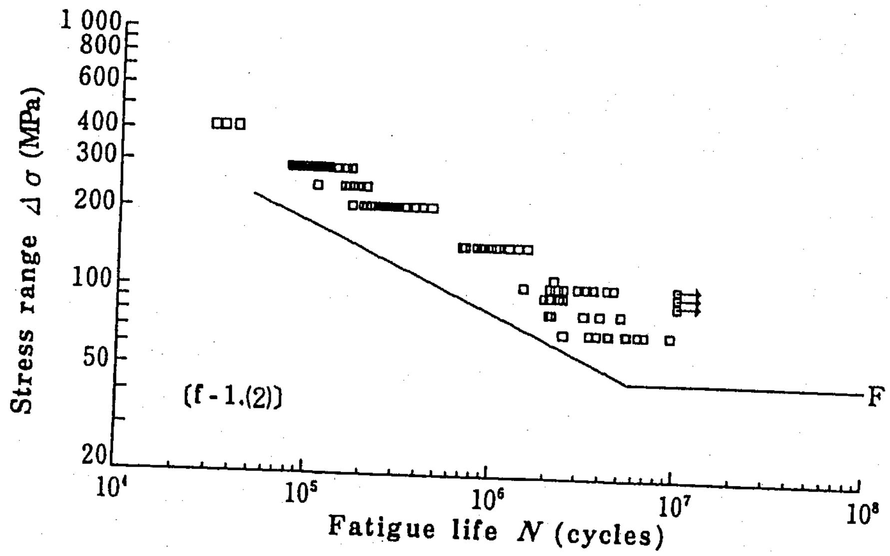
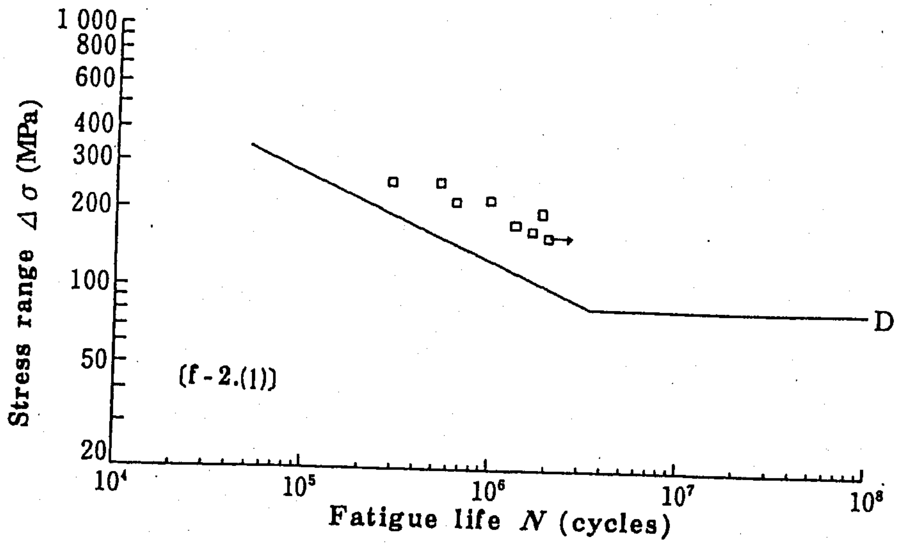
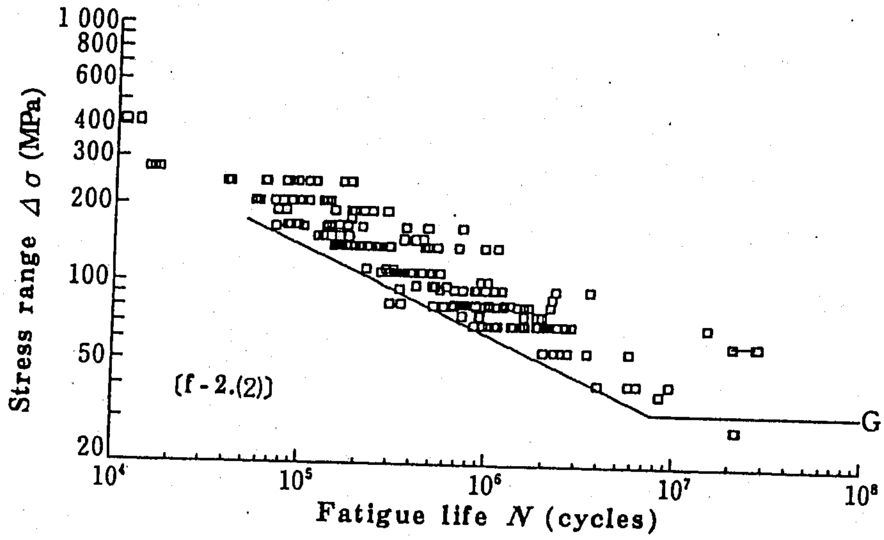


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)





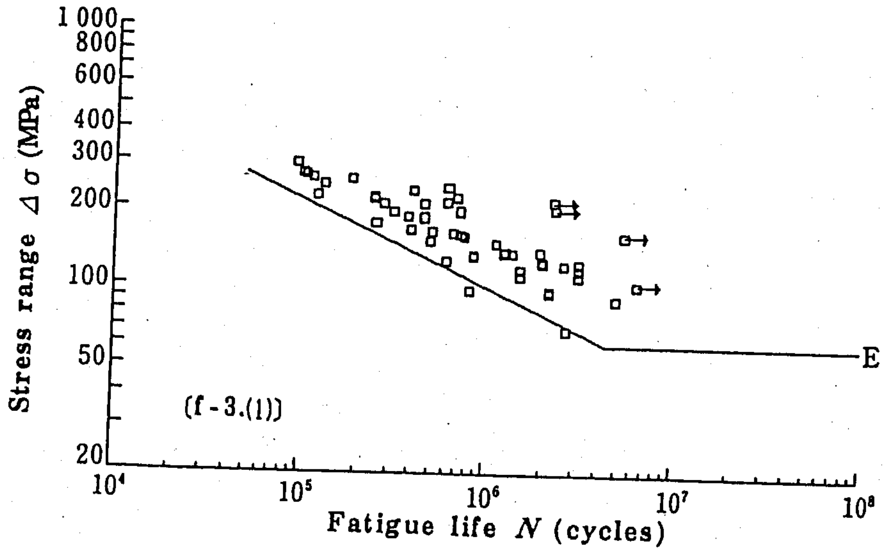


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)

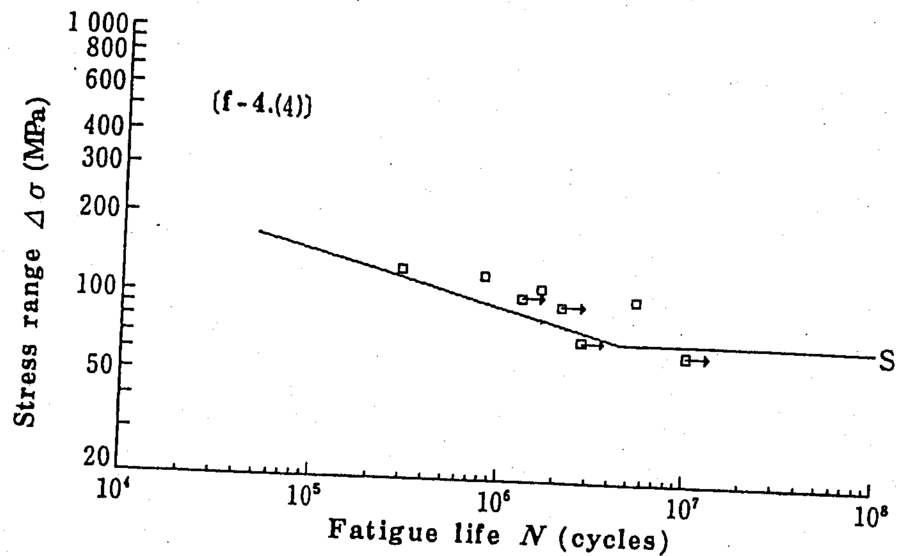


Fig.C.4.1 Fatigue test results and fatigue design curves (continued)

strength category C may be used. It should be noted that high strength steel was mainly used to obtain these test data. The notch sensitivity of so-called mild steel with a tensile strength less than 500MPa is generally lower than that of high strength steel, and welding residual stresses in mild steel are relatively low, so the effect of blowholes in mild steel may be less compared with that in high strength steel. This is also indicated in some fatigue test data obtained from girder type specimens[22]. Further study is need on the strength category and the allowable defect size of these joints made in mild steel.

In Fig.C. 4.1 (c-6), the numbers attached to the fatigue test data[23] for a longitudinal weld with a cope indicate the crack length at the surface when fatigue tests were finished. Although some test data do not satisfy the allowable stress range, the allowable stress range was set by considering the length of the fatigue cracks. Test data for these joints is still insufficient, and further investigations are needed.

(d) Cruciform joints (Table 4.4(d))

Fatigue assessment for load carrying fillet welded cruciform joints should be carried out both for sections at the main plate (toe failure) and at the throat (root failure). The strength category for toe failure of load-carrying cruciform joints is one grade lower than that for non-load-carrying cruciform joints. This is because the mechanism of stress flow causes different stress concentration at the weld toes, and the fatigue strength of load-carrying cruciform joints is lower than that of non-load-carrying fillet welded joints. The difference in the fatigue strength and the stress concentration has been confirmed through fatigue tests[24] and stress analysis.

For fatigue assessment of root failure, normal stress is used. Fatigue cracks initiating at the tips of the unwelded zone (the roots of the welds) normally propagate in the same direction as the unwelded zone. Therefore, it is considered appropriate to calculate the stress by dividing load by the sectional area at the leg length. However, in design calculations the stress is usually calculated at the throat, and the relationship between the leg length and the throat is, (leg length)=$\sqrt{2}\times$(throat). Therefore, in these Recommendations the throat area is used for fatigue assessment.

(e) Gusset joints (Table 4.4 (e))

Joints with gussets welded perpendicular to the plane of the main plate (joints with out-of-plane gussets) have different strength categories depending upon the gusset length. This is due to the fact that stress concentration at the weld toe at the end of the gusset, where fatigue cracks initiate, is affected by the length of the gussets[25].

(f) Other welded joints (Table 4.4 (f))

For joints with different lengths of cover plates, different strength categories are applied considering the effect of the length of the cover plate for the same reason as joints with out-of-plane gussets.

The strength category for joints with profiled end welds, where the leg length on the side of the main plate is comparatively long and the weld is finished to achieve a smooth transition, is increased by three categories. The stress concentration at the weld toes on the side of the main plate, where fatigue cracks initiate, can be reduced by enlarging toe radius and by making the weld to scalene triangle shape.

If the weld toe is ground, the strength category for joints with cover plates longer than 300mm should be the same to that for as welded joints. This is due to the fact that when the cover plate is long and when the fillet weld is relatively small, fatigue cracks initiate at the weld root, which reduces the beneficial effect of grinding weld toes.

(g) Cables and high strength bolts (Table 4.4 (g))

The fatigue strength of cables has been defined as the strength when 5% of cable wires are broken.

Fatigue-proof cable anchorages are such that the fatigue strength at anchorages is improved, so that they have fatigue strengths comparable to the cable itself. These are anchorages using wedge effects due to steel balls and the adhesive influence of epoxy resin[26], and anchorages using epoxy resin near the starting point of splaying the cable wires[27], instead of the conventional zinc-copper alloy.

For the conventional zinc-copper alloy type of anchoring method, an improved anchorage method[28] in which the bending technique and position of the cable wires is modified, has been proposed to improve fatigue strength. In cases where these new anchoring methods with improved fatigue strength are used, the upper grade of strength category, K1, can be used, provided that the fatigue strength is assessed by fatigue tests of these anchorages with a comparable number of cable wires to the actual structure.

4.4 Effect of mean stress

In large welded structures, tensile residual stresses resulting from welding will normally be as high as the yield stress of the steel, so that the real stress will fluctuate downwards from tensile yield stress, regardless of the applied stress condition. Therefore, the stress ratio of the applied stress has no influence on the fatigue crack initiation life or the fatigue crack propagation rate for small cracks. For that reason, the stress ratio also has little influence on the fatigue life of welded joints. In view of these facts, the allowable stress range was kept constant in spite of the variation of the stress ratio when tensile stress is major such as in the range $R \geqq -1$. However, after the fatigue crack propagates to a certain length, the tensile residual stress is relieved, and the effect of stress ratio on the fatigue crack propagation rate does appear. The critical crack length which induces brittle fracture or other critical fracture also varies with stress ratio. These influences of the stress ratio become significant in the range $R \leqq -1$. From these results, the allowable stress range for the stress ratio condition $R \leqq -1$ was set so as to increase with decrease of the stress ratio. The thirty percent increase in the allowable stress range at $R=-\infty$ is based on fatigue test results[29] performed on welded joints containing very high tensile residual stress (refer to Fig.C. 4.2).

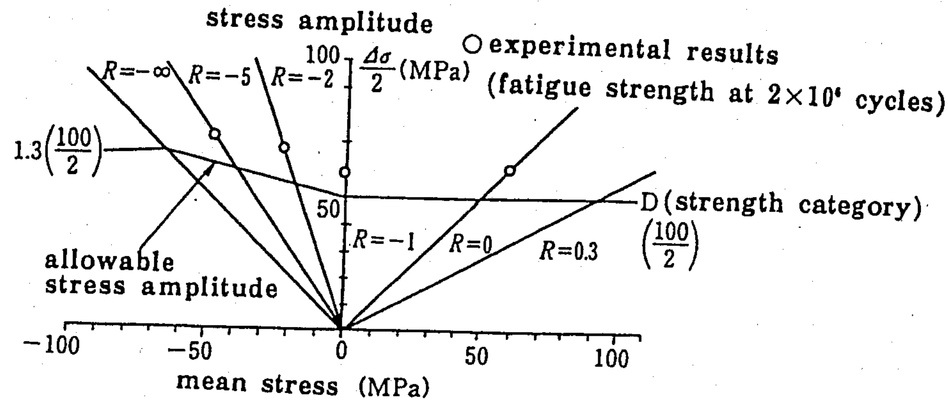


Fig.C.4.2 Influence of mean stress on fatigue strength [29]

The stress ratio effect on non-welded joints has been dealt with as for the case of welded joints in order to simplify the fatigue assessment, although the stress ratio has a large effect on the fatigue strength of non-welded joints. In the case of cables given in Table 4.4 (g), a method for correction of the allowable stress range due to stress ratio is given because cables are used in a very high stress ratio condition.

Fatigue strength can be improved by reducing the tensile residual stress or inducing compressive residual stress by means of post-weld heat treatment, peening or spot heating and cooling. The allowable stress range can be increased by applying such an improvement method. However, it is necessary to confirm the intensity of residual stress at the fatigue-critical area and to evaluate the improved fatigue strength in the light of previous experimental results or from references[30)-32)] or by new experiments.

4.5 Effect of plate thickness

In these Recommendations the fatigue design $\Delta\sigma$-N curves were determined based on results for joints in which plate thickness is less than 25mm. The allowable stress range for thicker plate has been given by correcting the effect of plate thickness on the fatigue strength of joints with plate thicker than 25mm.

It is usually considered that the decrease of fatigue strength with increase of plate thickness occurs due to stress concentration on the weld toe or stress gradients around the weld toe. Therefore, the decrease of fatigue strength with plate thickness depends on the type of joint. The plate thickness effect has been observed on as-welded non load carrying cruciform welded joints and cover plate joints. There are few experimental results on the plate thickness effect for other types of joint. In these Recommendations, Equation (4.5) is proposed to correct the allowable stress range for joints in which a plate thickness effect has been confirmed.

It was considered that the plate thickness effect would not be observed on transverse butt welded joints and longitudinal welded joints because the stress concentration at the fatigue crack initiation point is low in these joints. It is also considered that the variation of stress concentration with plate thickness is small in in-plane gusset joints. The variation of stress concentration for cruciform welded joints and cover plate joints with ground weld toes is also considered to be small. It was decided that the correction of allowable stress range is not necessary for these joints which have a small variation of stress concentration with plate thickness.

For non-load-carrying cruciform welded joints, the stress concentration at the weld toe (which is the fatigue crack initiation point) varies with not only the main plate thickness but also the attached plate thickness. In the case of an attached plate thickness of 12mm, it was confirmed that the stress concentration at the weld toe varies slightly with the main plate thickness in the region where main plate thickness is larger than 25mm[33]. From this result, it was decided that the correction of main plate thickness for fatigue strength is not necessary on joints in which the attached plate thickness is less than 12mm. However, the correction for plate thickness should be made on fillet welded joints in which leg length is larger than 8mm.

In the case of root failure of load carrying cruciform welded joints, correction for plate thickness on the allowable stress range is recommended. This is due to the increase of the length of unwelded zone as a result of the increase in main plate thickness.

Numerals under the decimal point may be omitted when an increase in the design plate thickness becomes necessary by the correction of allowable stress range using Equation (4.5). This is a result of allowing some safety margin when the correction factor in Equation (4.5) was determined and a basic plate thickness of 25mm in Equation (4.5) was selected. In an engineering sense, it is meaningless to increase the plate thickness in the range below 1mm, when the selection of plate thickness can only be made to the order of millimeters.

5. Loads and stress range histogram

5.1 Fatigue design load

5.1.1 Load unit

It is rarely the case that the fatigue design load can be defined with one load unit ; usually it is defined with a combination of plural load units.

In the case of overhead crane girders, for example, a load unit consists of the weight of a crane and lifting materials, movement of the crane and frequency of operations. Fig.C. 5.1 shows an example of a load unit for the runway girder of a ladle crane. In this case, two load units are taken into consideration. The movement of two load units, A and B in the figure, both comprise the following ; start from the rest position ①, lift of an empty ladle at ①, lowering it at an inlet ② and wait while the ladle is filled with melted steel, move to ③ and pour it into ingot molds in series, pass through ④, ⑤, and complete pour at ⑥, turn back to ① and detach the ladle and wait. For the case of load unit A, the ingot molds stand on one side of the center of the crane runway girder. For load unit B, the ingot molds stand on the opposite side. The magnitude of the load unit varies from the 'weight of the empty ladle' at ① to the 'weight of the empty ladle plus the rated weight of melted steel', the 'weight of the empty ladle plus the rated weight of melted steel minus the poured ingot steel' and the 'weight of the empty ladle' successively at the respective locations. The frequency of load unit is defined based on the operation plan of the ladle crane. For example, the planned frequency of operation can be defined as 121 for load unit A and 98 for B in one week.

In the case of highway bridges, load units can be defined by weight of vehicles, number of axles, spacing of axles, passing locations, and frequency of vehicles. The fatigue design load is defined by load units for several representative types of vehicles and combining these load units. For example, by grouping vehicles into trucks with two axles, trucks with three axles and trailers with four axles, these groups of vehicles may be defined as load units. If the distribution of weight of vehicles can be given, the design load can be defined as a better approximation to the service loading condition.

5.1.2 Representative load unit

When defining a representative load unit, it is recommended that reference is made to 'fatigue strength evaluation under variable-amplitude stress' described in Section 6.4. In this case, care should be taken that the fatigue assessment using a representative load unit is not less conservative than a fatigue analysis using multiple load units.

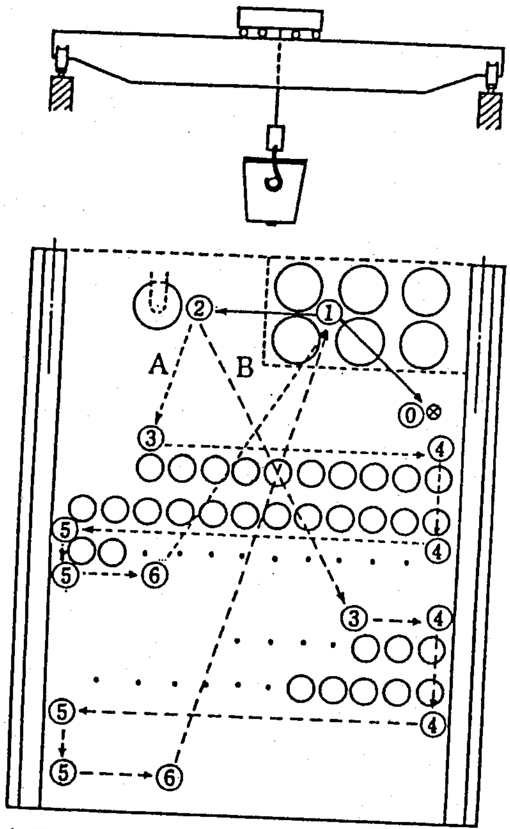


Fig.C.5.1 Example of load units for ladle crane

5.1.3 Maximum load unit

The load unit may not always take full account of the load that will induce the maximum stress range during the design life. In such cases, the maximum load unit should be defined taking into account the service conditions and the design life of the structure. The maximum load unit is defined only to calculate the maximum stress range during the design life, so that its frequency need not be a consideration.

5.1.4 Dynamic effects

Impact coefficients, which are used to check the ultimate or service limit states of the structure, are usually defined to estimate the maximum dynamic effects during the design life. Consequently when such an impact coefficient is used in a fatigue analysis, the result is always conservative.

5.1.5 Design life

These Recommendations contain engineering design principles and practices to check the safety of structures or structural members against fatigue loading during the design life. The designer should therefore define the design life beforehand for each individual case.

5.1.6 Unit term

The unit term is defined as the time unit used to measure design life. The unit term should be defined so that the structure may be subjected to single or plural load units (fatigue design load) defined in the unit term, repeatedly during the design life. For example, in the case of railway bridges, a unit term could be defined as one day, because a railway schedule is designed for the one day unit. Based on the information concerning the kind of trains and the frequency of passing over the bridge during this unit term, the load unit and the fatigue design load can be defined.

5.2 Stress

5.2.1 Calculation of stress

The nominal stress used in these Recommendations is based on elemental analyses such as beam theory, elemental elastic theory and so on which are used in usual design calculations. The nominal stress includes no account of stress concentration due to weld beads or other irregularities, but includes those induced by structural mechanisms such as the stress calculated by using effective width in consideration of the influence of shear lag on stress. If there is a hole at the joint, the stress is calculated using the minimum section (net section) excluding the hole.

For stress calculation, only the fatigue design load is considered. However, when the allowable stress range is corrected in consideration of the mean stress effect (see Section 4.4 of these Recommendations) , stress due to the dead load should also be calculated.

5.2.2 Stresses due to out-of-plane bending

The fatigue strength of a joint is influenced not only by the local stress at a fatigue crack origin but also by the stress distribution on the section where the fatigue crack propagates. Therefore, even though the type and form of the joint and the nominal stress at the fatigue crack origin are the same, the fatigue strength is likely to be different depending on whether an axial force or a bending force perpendicular to the plate surface is acting. Fig.C. 5.2 shows the relationship between the nominal stress range and fatigue life when axial force acts on a cruciform joint of the non load carrying type and when bending force acts on a T-shaped joint in the out-of-plane direction. The fatigue strength is higher by 50% on the average and about 40% of the value of ' average-2xstandard deviation' when bending force in the out-of-plane direction is present. Therefore, it is extremely conservative to make a fatigue assessment by comparing the bending stress range in the out-of-plane direction with the allowable stress range determined based on fatigue test results under axial force. This fact is also verified by the fatigue crack propagation analysis. For the above reasons, bending stress in the out-of-plane direction is corrected using Equation 5.1.

However, Fig.C. 5.2 shows the fatigue test results of joints with thicknesses up to 25mm. It is not clear at present to what degree fatigue strength of thick joints varies under bending stress in the out-of-plane direction and under in-plane membrane stresses. Considering that the fatigue strength is different under out-of-plane bending stress and under membrane stress because of the

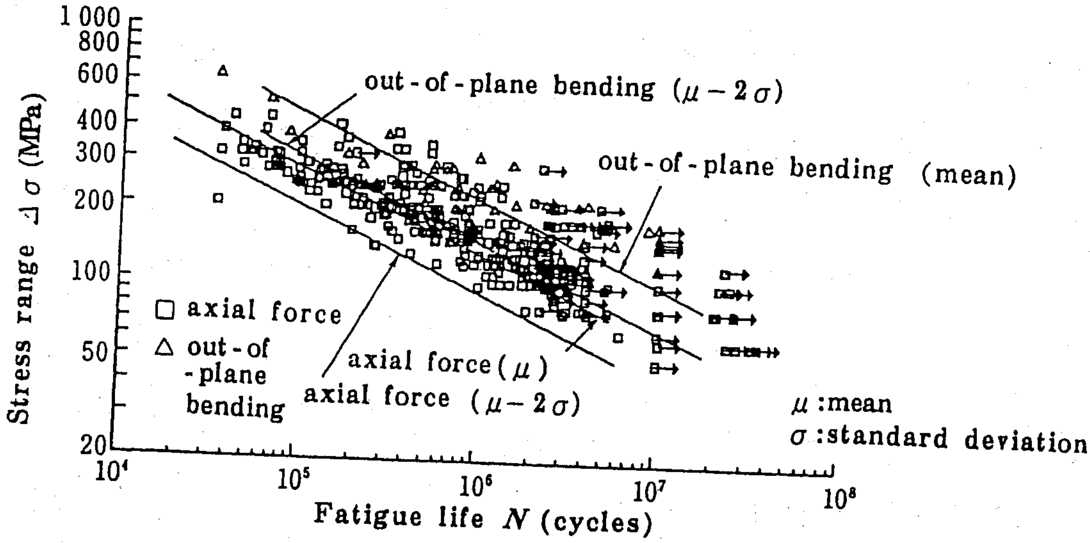


Fig.C.5.2 Fatigue strengths of cruciform joints subjected to out-of-plane bending different stress distribution as described above, it is anticipated that the slope in the thicknes direction of the out-of-plane bending stress becomes more gentle as the thickness increases, whil the difference between the fatigue strength under out-of-plane bending stress and that unde membrane stress decreases. Therefore, Equation 5.1 was applied only to joints of thicknesse below 25mm.

5.2.3 Correction of design stresses

Generally, in the design stress calculation of structures, the influence of secondary members and attachments is not taken into consideration and therefore the stress generated actually is often smaller than that calculated. In such cases, fatigue assessment is extremely conservative[34]. The value of the stress correction factor, α, is determined based on the results of measurements of a similar structure or test results on modeled specimens.

This concept has already been adopted in American specifications for highway bridges[12], in which the value of α is set at 0.8 for transverse members and 0.7 for longitudinal members in multiple beams bridges[35].

Fig.C. 5.3 shows the ratio of the stress measured in a highway bridge and the calculated design stress[36], and Fig.C. 5.4 shows the ratio of the stress measured in a railway bridge and calculated design stress[37]. In the case of the highway bridge, the upper limit of the actual stress ratio is about 0.7 for a non-composite girder bridge, which can be considered for use as a stress correction factor. In the Japan Railway design standard 1991 edition[37], the stress correction factor, α (called structural analysis factor, γ_a) is set at 0.85 based on the results of Fig.C. 5.6.

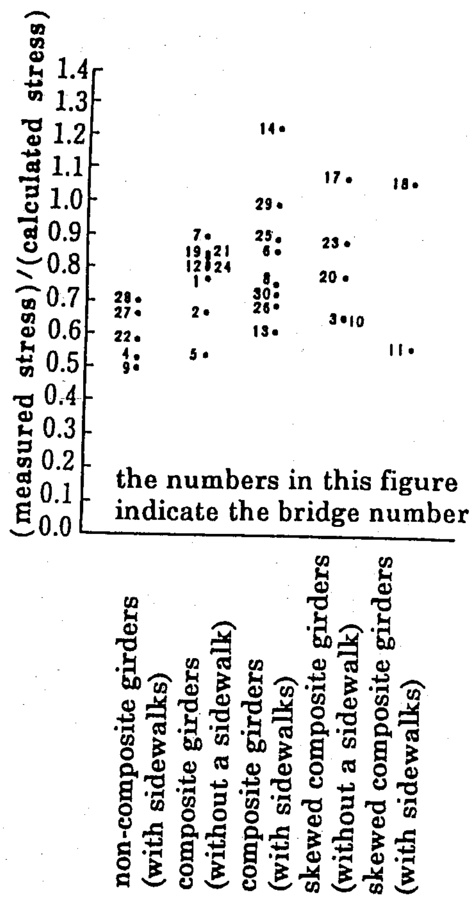


Fig.C.5.3 Measured stress in actual bridges (highway bridges) [36]

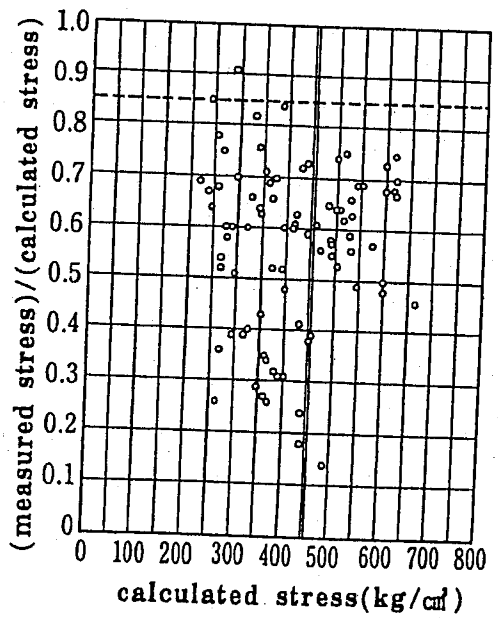


Fig.C.5.4 Measured stress in actual bridges (railway bridges) [37]

5.3 Stress range histogram

Stress variations of various scale are caused by the fatigue design load at the position where the fatigue assessment is to be carried out. A frequency histogram of the stress range is obtained for such variable amplitude stress by using the rain flow counting method or other stress range counting methods from which equal results can be obtained.

The stress range is calculated as follows by the rain flow counting method[38]. In Fig.C. 5.5, the vertical axis shows the time of the variable amplitude stress wave form and the horizontal axis shows the stress. A water source is placed at the first peak stress position (point A) and water is poured. The stress range is then calculated from the range of the flow line of the water. That is, the range ($\sigma_B-\sigma_C$), ($\sigma_F-\sigma_E$), ($\sigma_D-\sigma_G$), ($\sigma_J-\sigma_K$), ($\sigma_L-\sigma_I$), ($\sigma_H-\sigma_A$) is calculated as the stress trange.

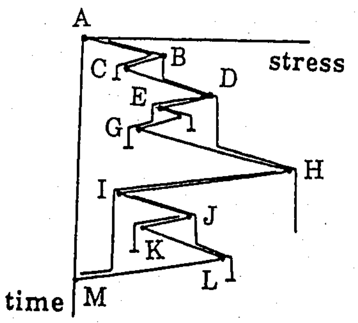
To formulate the method, when four subsequent peak values, σ_1, σ_2, σ_3 and σ_4 satisfy the relation $\sigma_1 \geqq \sigma_2 \geqq \sigma_3 \geqq \sigma_4$ or $\sigma_1 \leqq \sigma_2 \leqq \sigma_3 \leqq \sigma_4$, the range $|\sigma_2-\sigma_3|$ is calculated and σ_2 and σ_3 are deleted from the variable amplitude stress wave form. By continuing the calculation in this way, the gradually increasing or decreasing variable amplitude stress as shown in Fig.C. 5.6 may sometimes remain. In such cases, stress range may be calculated as the difference between the highest and lowest peak values, the second highest and lowest peak values, the third highest and lowest peak values, the fourth highest and lowest peak values, and so on.

When the relationship between the stress range calculated by the representative load unit or maximum load unit and the stress range histogram is clear from detailed analysis or experience, a more rational fatigue assessment is possible by using that stress range histogram. In the Norwegian standard for mobile offshore structures (Det Norske Veritas)[39], the stress range histogram is given by the Weibull distribution based on the stress range calculated from a 20-year return period of waves. In the German standard for crane structures (DIN-15018)[40], the stress range histogram is given for each operating condition.

5.5 Equivalent stress range

As shown in Section 6.4 of these Recommendations, the fatigue strength under variable amplitude stress is evaluated on the basis of the linear damage law. The equivalent stress range is also obtained based on the linear damage law. A conception of the law may be explained as follows.

It is assumed for the stress range histogram that a certain stress range level is $\Delta\sigma_i$ and its frequency is n_i, and the fatigue life when only $\Delta\sigma_i$ acts repeatedly is N_i. If $\Delta\sigma_i$ is below the variable amplitude cut-off limit, then $N_i=\infty$. It is also assumed that fatigue failure occurs when the fatigue damage after $\Delta\sigma_i$ is repeated n_i times is (n_i/N_i) and the total damage D (cumulative damage) reaches unity as shown in following equation.



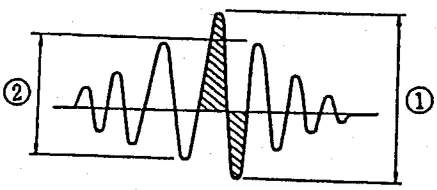


Fig.C.5.6 Incresing and decreasing stress wave

Fig.C.5.5 Stress range counting by rain-flow method

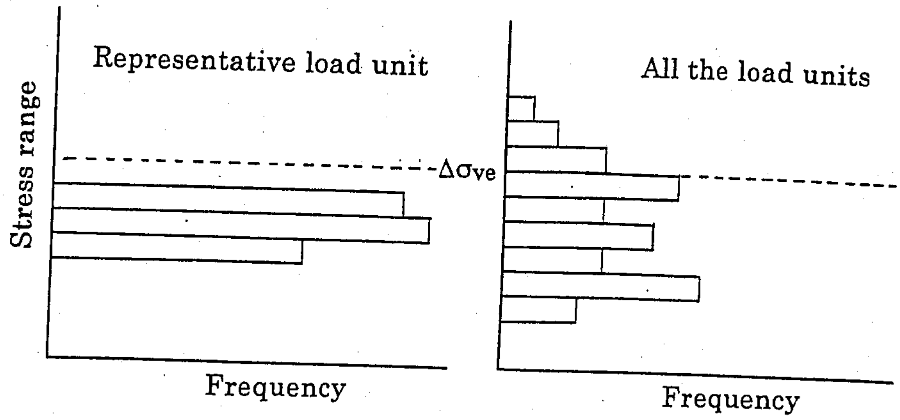


Fig.C. 5.7 Stress range histograms obtained by representative load unit and all the load units

$$D = \Sigma(n_i/N_i) = 1 \qquad \text{(c.5.1)}$$

$\Delta\sigma_i$ and n_i are obtained by using the method described in Section 5.3. When the stress range distribution is given beforehand and the stress range needs to be dispersed in order to apply the linear damage law, care should be taken regarding the dispersion interval. It is desirable that the interval in the stress range is below 1/20 of the maximum stress range.

The relationship between the stress range and fatigue life ($\Delta\sigma$-N curve) is expressed as follows:

$$\begin{aligned} \Delta\sigma^m \cdot N &= C_0 \qquad (\Delta\sigma > \Delta\sigma_{ve}) \\ N &= \infty \qquad (\Delta\sigma \leqq \Delta\sigma_{ve}) \end{aligned} \qquad \text{(c.5.2)}$$

where C_0: constant depending on the strength category ($=2\times10^6 \cdot \Delta\sigma_f{}^m$)

By substituting this for Equation (c.5.1), the cumulative damage D is as follows :

$$D = \Sigma(\Delta\sigma_i{}^m \cdot n_i)/C_0 \qquad \text{(c.5.3)}$$

The damage when a certain size of stress range $\Delta\sigma_e$ acts Σn_i times is given by the following equation :

$$D = \Delta\sigma_e{}^m \cdot \Sigma n_i/C_0 \qquad \text{(c.5.4)}$$

When the damage given by Equations (c.5.3) and (c.5.4) is the same, $\Delta\sigma_e$ is termed the equivalent stress range.

$$\Delta\sigma_e = \sqrt[m]{\Sigma\Delta\sigma_i^m \cdot n_i / \Sigma n_i} \qquad \text{(c.5.5)}$$

The representative load unit represents all the load units. In other words, the stress range histogram obtained from the representative load unit represents the stress range histogram obtained from all the load units. For this reason, the stress range components calculated from the representative load unit may sometimes be smaller than some of those calculated from all the load units. Therefore, if the influence of the stress range component obtained using the representative load unit is ignored when it is below $\Delta\sigma_{ve}$, the stress range over $\Delta\sigma_{ve}$ calculated from all the load units does not contribute to fatigue damage, as shown in Fig.C 5.7. Therefore, it was assumed that when the representative load unit is used as the fatigue design load, its repetition number ni should not be 0 even though the stress range is below $\Delta\sigma_{ve}$.

6. Fatigue assessment

6.1 Structural details to be assessed

Most fatigue damage in welded structures occurs at welded joints. Accordingly the fatigue assessment should normally be conducted at welded joints.

Where the same type of joints continue as for built-up beams subject to bending moment, the section in the maximum stress range should be assessed.

6.2 Safety factor

As the safety of joints against fatigue failure depends on the degree of redundancy (fatigue damage effects on the strength and the function of the whole structure), the importance of the structure (the social effects of failure of the structure), the inspection potential (possibility of discovery of fatigue damage before failure), the redundancy factor γ_b, the importance factor γ_w, and the inspection factor γ_i are employed. These partial safety factors are used to compare the design stress range with the allowable stress range.

6.3 Simplified procedure for fatigue assessment

The constant amplitude cut-off limit for each joint was established based on the fatigue limit. If the predicted maximum stress range is below this, then fatigue damage should not occur. Hence, the fatigue assessment described in Section 6.6 and 6.7 need not be conducted.

6.4 Fatigue strength evaluation under variable-amplitude stress

You can refer to Section 5.5 'Equivalent stress range'.

6.8 Combined stresses

It is not difficult to calculate a nominal normal stress or a nominal shear stress for steel structures using beam theory or elemental elastic theory. Furthermore, most fatigue tests on welded joints are conducted under simple normal stress or shear stress, and the data obtained from these fatigue tests are reliable and convenient for fatigue assessment. Therefore, these Recommendations recommend that fatigue assessment is conducted using normal stress and shear stress separately.

However, there are some cases where fatigue assessment using combined stresses is appropriate, but it is difficult to determine a range for the ratio of shear stress to normal stress in cases where fatigue design using combined stresses is suitable. It is recommended that designers and organizations concerned judge whether evaluation using combined stresses is suitable or not.

According to the British standard for bridges, BS 5400[13)], it is acceptable to neglect shear stress and use only the normal stress for the fatigue assessment in cases where ratio of the shear stress to the normal stress is equal to or less than 0.15.

7. Fatigue assessment based on hot spot stress

When the structural details are complex, it may not be possible to define and calculate the nominal stress. In such cases, the hot spot stress is used for fatigue assessment in place of nominal stress.

7.1, 7.2 Definition and calculation of hot spot stress

The hot spot stress is defined as the stress at the point under consideration (weld toe) taking into account only the overall geometry of the structure and excluding local stress concentration effects due to the weld geometry. In general, the hot spot stress is obtained from an extrapolation of the stress distribution to the weld toe as shown in Fig.C. 7.1. Table C.7.1 shows examples of parametric formulae used to calculate the hot spot stress in typical steel tubular joints[41].

7.3 Fatigue design curve

When the hot spot stress is used for the fatigue assessment, the fatigue design curves for non load carrying or load-carrying fillet welded joints should be used. This requirement arises from the fact that the hot spot stress takes into account only the effect of structural geometry on the stress and does not take account of the influence of the weld geometry.

Fig.C. 7.2 shows the fatigue design curves used in these Recommendations and other fatigue design codes.

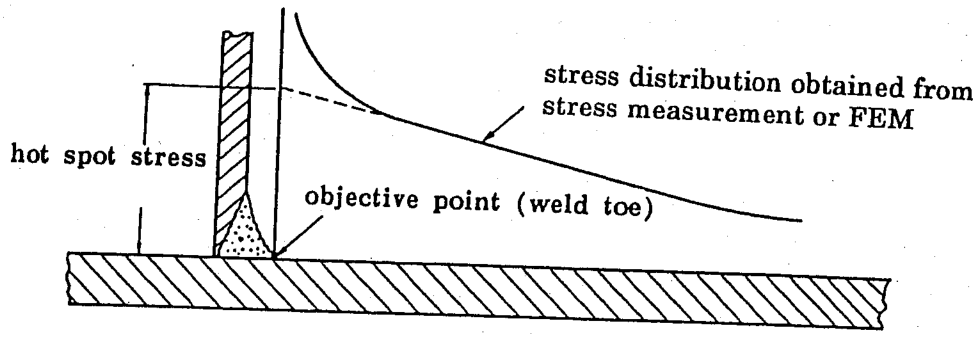


Fig.C.7.1 Definition of hot spot stress

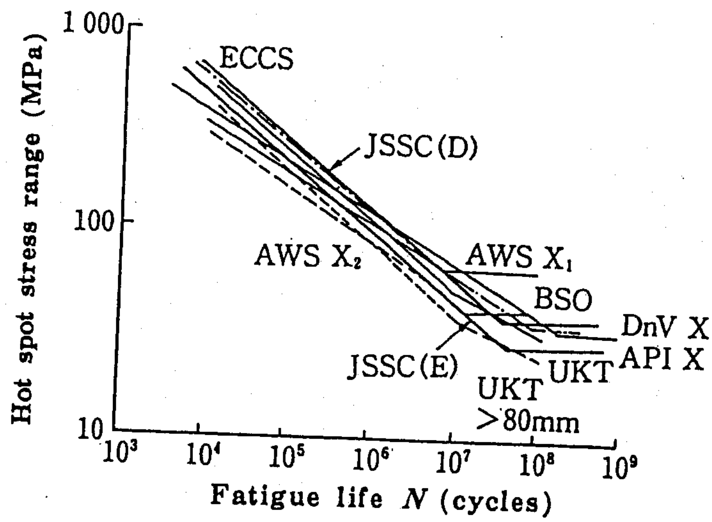
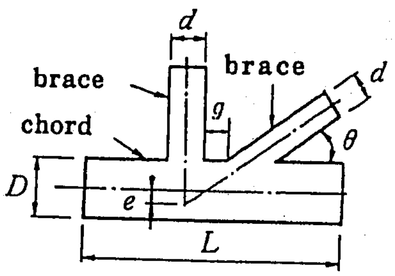


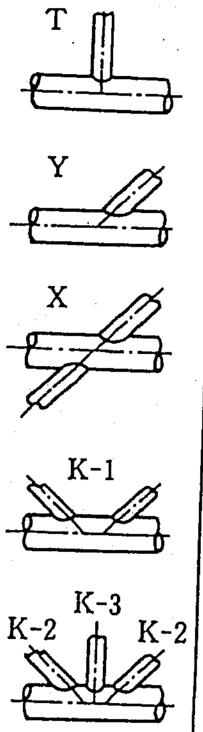
Fig.C.7.2 Fatigue design curves (hot spot stress)

Table.C.7.1 Examples of expression for hot spot stress [41)]



$\beta = d/D \quad p = g/D$
$\gamma = R/T \quad \alpha = L/D$
$\tau = t/T$

T : wall thickness of chord
t : wall thickness of brace
R : outer radius of chord wall
r : outer radius of brace wall

			axial force
	SCF_{chord}	T&Y	$[1.5-3.88(\beta-0.47)^2]\cdot\gamma^{0.87}\cdot\tau^{1.37}\cdot\alpha^{0.06}\cdot\sin^{1.694}\theta$
		X	$1.35\cdot\gamma\cdot\tau\cdot\beta(2.42-2.28\cdot\beta^{2.2})\cdot\sin^{\beta^2(15-14.4\beta)}\theta$
		K-1	$1.51\cdot\gamma^{0.67}\cdot\beta^{-0.59}\cdot\tau^{1.1}\cdot p^{0.067}\cdot\sin^{1.52}\theta$, $30^\circ<\theta<90^\circ$
		K-2 K-3	$1.83\cdot\gamma^{0.54}\cdot\beta^{0.12}\cdot\tau^{1.07}\cdot\sin\theta$, $0^\circ<\theta<90^\circ$
	SCF_{brace}	T&Y	$[1.09-1.93(\beta-0.5)^2]\cdot\gamma^{0.76}\cdot\tau^{0.57}\cdot\alpha^{0.12}\cdot\sin^{1.94}\theta$
		X	$0.794+0.63\cdot SCF_{chord}$
		K-1	$0.92\cdot\gamma^{0.16}\cdot\beta^{-0.44}\cdot\tau^{0.56}\cdot p^{0.058}\cdot e^{1.45\sin\theta}$, $30^\circ<\theta<90^\circ$
		K-2	$6.06\cdot\gamma^{0.1}\cdot\beta^{-0.36}\cdot\tau^{0.68}\cdot(p_1+p_2)^{0.13}\cdot\sin^{0.5}\theta$, $0^\circ<\theta<45^\circ$ $1.38\cdot\gamma^{0.1}\cdot\beta^{-0.36}\cdot\tau^{0.68}\cdot(p_1+p_2)^{0.13}\cdot\sin^{2.9}\theta$, $45^\circ<\theta<90^\circ$
		K-3	$4.89\cdot\gamma^{0.12}\cdot\beta^{-0.4}\cdot\tau^{0.68}\cdot(p_1+p_2)^{0.16}\cdot\sin^{2.27}\theta$

hot spot stress for chord $= SCF_{chord} \times \dfrac{P_{chord}}{A_{\cancel{chord}\ \text{brace}}}$

hot spot stress for brace $= SCF_{brace} \times \dfrac{P_{brace}}{A_{brace}}$

with exception of X, Y and T-joints, SCF for brace may be reduced in accordance with the following equation

$1.0 + Q_R(SCF_{brace} - 1)$ should not be less than 2.5

$Q_R = \exp\left(-\dfrac{0.5T + t}{\sqrt{r \cdot t}}\right)$ should not be less than 0.7

8. Maintenance and management

8.1 Inspection

To protect a structure in use from fatigue, it is effective to inspect for the presence and extent of damage by an appropriate method and at an appropriate interval. In consideration of this fact, the partial safety factor (inspection factor γ_i) varies depending on the level of inspection (see Section 6.4).

The inspection factor γ_i should be set in consideration of the following. When the inspection interval is set at one-fifth of the design life, γ_i=1.00. If a crack whose size can be detected by the inspection method is used as an initial crack, the crack propagation life is calculated by a fatigue crack propagation analysis as shown in the Appendix and the inspection interval is found to be shorter than the calculated life, γ_i=0.90. In that case, a conservative curve is used as an equation for the fatigue crack propagation rate. If the inspection interval is below twice the calculated crack propagation life, γ_i=0.90 ~ 1.00 depending on the ratio of the inspection interval to the crack propagation life. If these conditions are not satisfied, γ_i=1.10.

If points susceptible to fatigue damage are known in advance from fatigue assessments at the time of design and/or experience, priority inspection of these points is effective.

8.2 Evaluation of fatigue damage

When fatigue damage is found as a result of inspection, a study of its cause should be made. Possible causes of fatigue damage include heavier actual load than that presumed at the time of design, higher frequency of load, high stress which was induced by distortion modes unexpected in the design, inappropriate structural details, and impermissible flaws in welds. Investigation of causes is important in order to select a repair method.

When a fatigue crack is found, it is effective to predict the remaining life using the method shown in the Appendix in consideration of the loads acting on the structure. This helps with evaluation of the urgency of the repair.

8.3 Repair of fatigue damage

When the evaluation of fatigue damage shows a necessity for repair, the damage should be repaired using an appropriate method. Once the cause has become clear, an applicable method should be used for repair. An inappropriate repair may result in further damage.

Experimental studies and actual applications to structures have been reported with regard to repair and reinforcement methods for fatigue damage. Some methods are shown below.

① Opening a circular hole at the end of a fatigue crack (stop hole).
② Opening a circular hole at the end of a fatigue crack and bolting it.
③ Welding repair (without reinforcement members).
④ Welding repair involving reinforcement members.
⑤ Mechanical repair using splice members and high strength bolts.
⑥ Post-tensioning repair using PC steel wires or rods.
⑦ Repair by structural composition with concrete.
⑧ Others.

In case of repair, 'Recommendations for repair and strengthening of existing steel structures by welding'[+2)] and 'Recommendations for repair and strengthening of existing steel structures by high strength bolts'[+3)] edited by the Japanese Society of Steel Construction may be helpful.

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APPENDIX

ESTIMATION OF FATIGUE CRACK PROPAGATION LIFE BASED ON FRACTURE MECHANICS ANALYSIS

A.1 Scope

A.1.1 This appendix is applicable to ;

1) estimation of fatigue crack propagation life of a structural component with crack-like defects,
2) estimation of fatigue crack propagation life of a welded joint which is not included in Table 4.4,
3) selection of a duration (a period of time) before inspection and maintenance of a structure,
4) estimation of remaining life of an in-service structure in which fatigue cracks have initiated.

A.1.2 The results obtained from this procedure cannot replace the strength category and the joint quality requirement described in this Recommendations. These results should be considered to be a supplement to the application of the Recommendations.

A.1.3 This appendix describes the fatigue crack propagation analysis for cracks which propagate only in Mode I loading under stress normal to the crack plane.

A.2 General method of analysis

The fatigue crack propagation life N can be obtained by integrating the equation for fatigue crack propagation rate from an initial crack size a_i to a critical crack size (final crack size) a_c as follows :

$$da/dN = f(\Delta K)$$

$$N = \int_{ai}^{ac} da/f(\Delta K) \qquad \text{(A.1)}$$

where, da/dN : fatigue crack propagation rate, ΔK : stress intensity factor range

A.3 Initial crack size, a_i

In cases where it is difficult to decide whether an indication detected by non-destructive inspection represents a crack-like defect or not, this indication should be regarded as a crack-lik defect. The procedure in this Section should then be applied to the defect.

A surface defect is substituted by a semi-elliptical crack. In this case, the axis lengths of th semi-elliptical crack coincide with the edge lengths of the smallest rectangle which encloses th projected shape of the defect (see Fig. A.1). An inner defect is also substituted by an elliptica crack as shown in Fig. A.1. If there are two or more defects in the same plane, and the distanc between the defects is as small as the conditions indicated in Fig. A.1, these defects should b regarded to be a single crack formed by the coalescence of the defects, as shown in Fig. A.1 However, the assumption as to the transformation of multiple defects into a single crack is too conservative in consideration of the actual crack initiation and propagation behavior from the defects. It is necessary for estimating the fatigue crack propagation life more accurately to study the transformation further.

When the defect plane is not perpendicular to the stress axis, the crack size is evaluated by projecting the defect on to a plane perpendicular to the stress axis.

A.4 Critical crack size, a_c

The final crack size for propagation analysis is defined as the critical size for some other types of fractures or conditions of structural non-integrity, for example :

1) unstable fracture such as brittle fracture,
2) ductile fracture,
3) general yielding of a component,
4) penetration of a crack through the plate thickness.

A.5 Equation of fatigue crack propagation rate

The fatigue crack propagation rate is expressed by the following equation :

$$\begin{aligned} da/dN &= C\,(\Delta K^n - \Delta K_{th}^n) && \text{when } \Delta K \geqq \Delta K_{th}, \\ da/dN &= 0 && \text{when } \Delta K \leqq \Delta K_{th} \end{aligned} \tag{A.2}$$

where, C, n : constants, ΔK_{th} : threshold value of stress intensity factor range

The following equations are also applicable :

$$\begin{aligned} da/dN &= C\,(\Delta K)^n, && \text{when } \Delta K > \Delta K_{th} \\ da/dN &= 0, && \text{when } \Delta K \leqq \Delta K_{th} \end{aligned} \tag{A.3}$$

Furthermore, a more simplified equation can be used as follows :

$$da/dN = C\,(\Delta K)^n \tag{A.4}$$

Eqs. (A.3) and (A.4) give a more conservative estimation of fatigue crack propagation life than Eq. (A.2).

For the constants C, n and the threshold value ΔK_{th} in the above equations, the following values are recommended :

	C	n	ΔK_{th} (MPa$\sqrt{\text{m}}$)
Conservative curve	2.7×10^{-11}	2.75	2.0
Mean curve	1.5×10^{-11}	2.75	2.9

where units are ; da/dN in m/cycle, ΔK in MPa$\sqrt{\text{m}}$

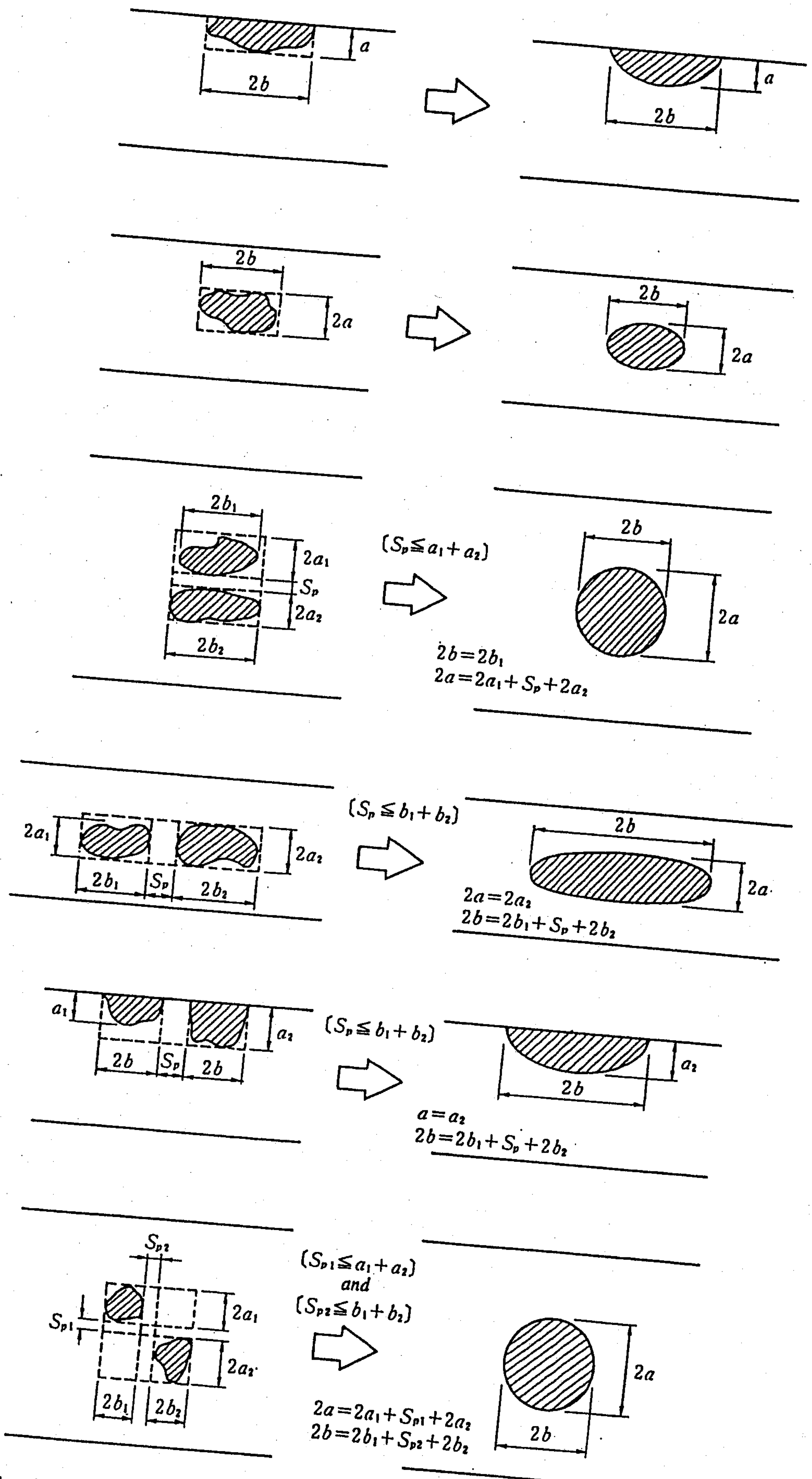


Fig.A.1 Substitution of actual defects by a simplified crack

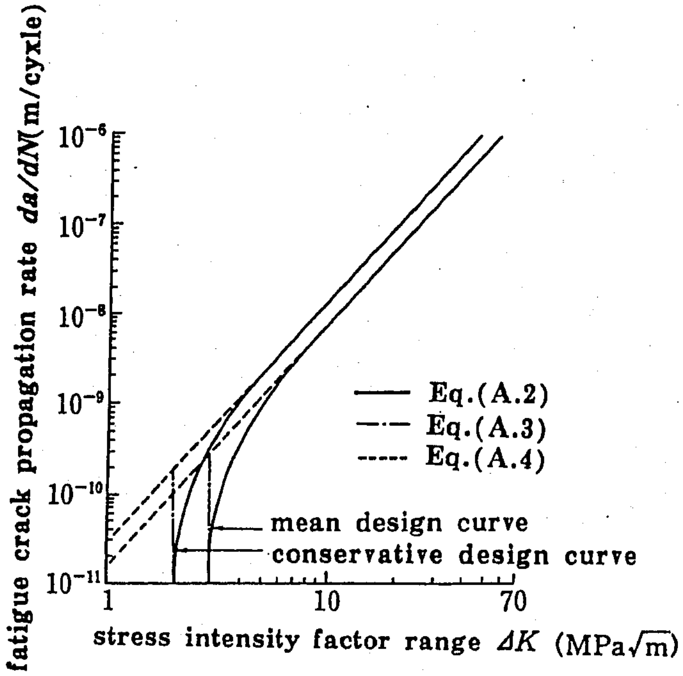


Fig.A.2 ***da/dN*-Δ*K* relationships**

The applicable upper limit of Eqs. (A.2), (A.3) and (A.4) is $\Delta K \leqq 100\text{MPa}\sqrt{\text{m}}$.

Usually, the values of C, n and ΔK_{th} for the conservative curve are used in the analysis. But in cases where evaluation of a mean fatigue strength or a mean fatigue life is requested, the values for the mean curve should be applied. Fig. A.2 shows da/dN-ΔK relationships defined by Eqs. (A.2), (A.3) and (A.4) and the values of C, n and ΔK_{th} given above.

These values of C, n and ΔK_{th} have been determined through analysis of data which were obtained by fatigue crack propagation tests simulating high tensile residual stress conditions. Therefore, for application to a non-welded component, it is possible to modify C and ΔK_{th} in the crack propagation rate equations according to the stress ratio (including stress induced by dead load and constraint stress)[1].

A.6 Stress intensity factor range

The stress intensity factor range ΔK is expressed as follows[2] :

$$\Delta K = F \Delta\sigma \sqrt{\pi a} \qquad \text{(A.5)}$$

F : general correction factor $(= F_g \cdot F_e \cdot F_s \cdot F_t \cdot F_h)$
F_g : correction factor for stress gradient
F_e : correction factor for crack shape
F_s : correction factor for surface crack
F_t : correction factor for finite thickness and width of plate
F_h : correction factor for eccentricity of crack against central axis of plate
$\Delta\sigma$: nominal stress range
a : crack size

A.6.1 Size and shape of crack

Cracks can be categorized into two types, surface cracks and embedded cracks. The size of a crack is represented by the depth a and width b of the crack as shown in Fig. A.3. The shapes of cracks can be further categorized into idealized shapes of elliptical cracks, semi-elliptical cracks, quarter-elliptical cracks, through-thickness cracks and edge cracks.

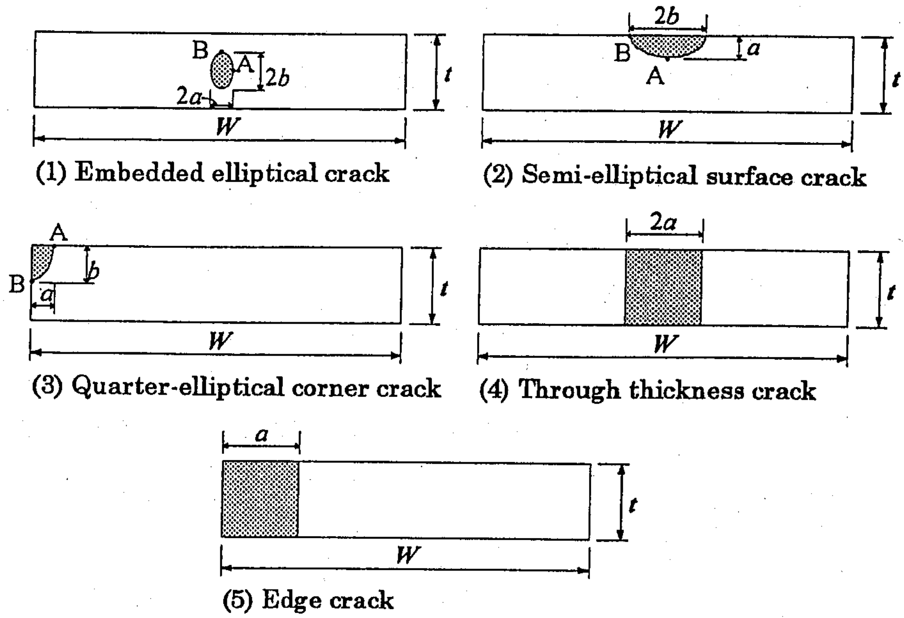


Fig.A.3 Idealized crack shapes

A.6.2 Correction factor for stress intensity factor range

Generally, the correction factor F can be obtained through mathematical analyses or numerical analyses such as the finite element method. Three-dimensional analyses are usually necessary to obtain the factors for elliptical cracks, semi-elliptical cracks, and quarter-elliptical cracks. In cases where it is clear that the three-dimensional effects rarely influence the factors, a two-dimensional analysis can be applied.

The correction factors can also be obtained based on calculations using various simplified equations proposed in the literature[3]. Some of the proposed equations are given as follows :

< Correction factor for stress gradient, F_g >

For cracks in a non-load carrying cruciform joint (see Fig. A.4)[4] :

when $l/t < 2$,

$$F_g = 0.51(l/t)^{0.27} \cdot (a/t)^{-0.31} \qquad a/t \leqq 0.05(l/t)^{0.55}$$
$$F_g = 0.83(a/t)^{-0.15(l/t)^{0.46}} \qquad a/t > 0.05(l/t)^{0.55} \qquad \text{(A.6)}$$

when $l/t > 2$,

$$F_g = 0.615(a/t)^{-0.31} \qquad a/t \leqq 0.073$$
$$F_g = 0.83(a/t)^{-0.2} \qquad a/t > 0.073$$

where, $F_g \geqq 1$.

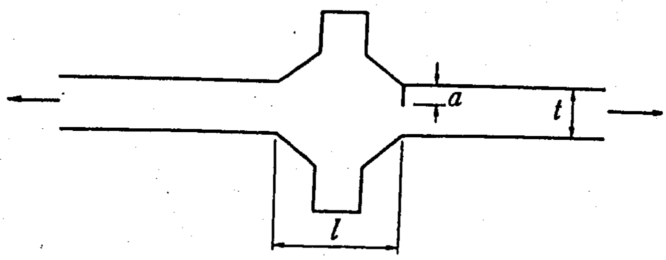


Fig.A.4 Non-load-carrying cruciform joint with a crack

< Correction factor for shape of crack, F_e >

For a crack tip at the end of the shorter axis of an elliptical crack, a semi-elliptical crack or a quarter-elliptical crack :

Qa $\simeq F_e = 1/E(k)$ (A.7)

$$E(k) = \int_0^{\pi/2} \sqrt{1-k^2\sin^2\xi}\, d\xi$$

$$k = 1-a^2/b^2$$

where $E(k)$ can be approximated by the following equation with sufficient accuracy[5].

$$E(k) = \sqrt{1+1.464(a/b)^{1.65}}$$

For a crack tip at the end of the longer axis of these cracks :

Qc $\simeq F_{e'} = F_e\sqrt{a/b}$ (A.8)

< Correction factor for surface crack, F_s >

For a crack tip at the end of the shorter axis of a semi-elliptical surface crack or a quarter-elliptical surface crack :

$$F_s = 1.12-0.12a/b \quad \text{(A.9)}$$

For an edge crack :

$$F_s = 1.12 \quad \text{(A.10)}$$

< Correction factor for finite thickness and width of plate, F_t >

$$F_t = (1-0.025\lambda^2+0.06\lambda^4)\sqrt{\sec(\pi\lambda/2)} \quad \text{(A.11)}$$

$\lambda = 2a/w$ (through-thickness crack)
$\lambda = a/w$ (edge crack)
$\lambda = 2a/t$ (elliptical crack)
$\lambda = a/t$ (semi-elliptical or quarter-elliptical crack)

< Correction factor for eccentricity of crack, F_n >

The correction factor for the eccentricity of a crack in relation to the central axis of a plate is expressed as follows (see Fig. A.5) :

$$F_n = \sqrt{\sin(2\lambda\varepsilon)/2\lambda\varepsilon} \quad \text{(A.12)}$$

$$\varepsilon = 2e/w,\ \lambda = a/d_1$$

This factor is for the crack tip closest to the plate edge. For the opposite crack tip (the crack tip more remote from the plate edge), $F_n=1$ can be applied.

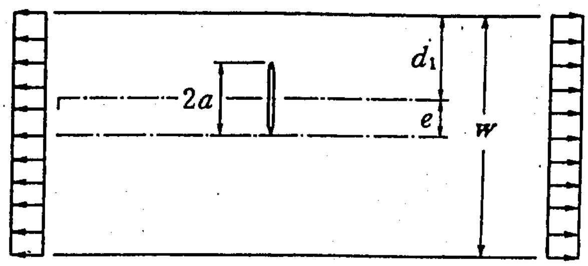


Fig.A.5 Eccentric through-thickness crack in a plate

A.7 Calculation of crack propagation

A.7.1 This appendix allows the application of the linear cumulative damage rule to crack propagation analysis under variable amplitude stress. In cases where Eq. (A.3) or (A.4) is used, it is convenient to apply the equivalent stress intensity factor range defined by the following equation :

$$\Delta K_e = F \cdot \Delta\sigma_e\sqrt{\pi a} \qquad \text{(A.13)}$$

$\Delta\sigma_e$: equivalent stress range

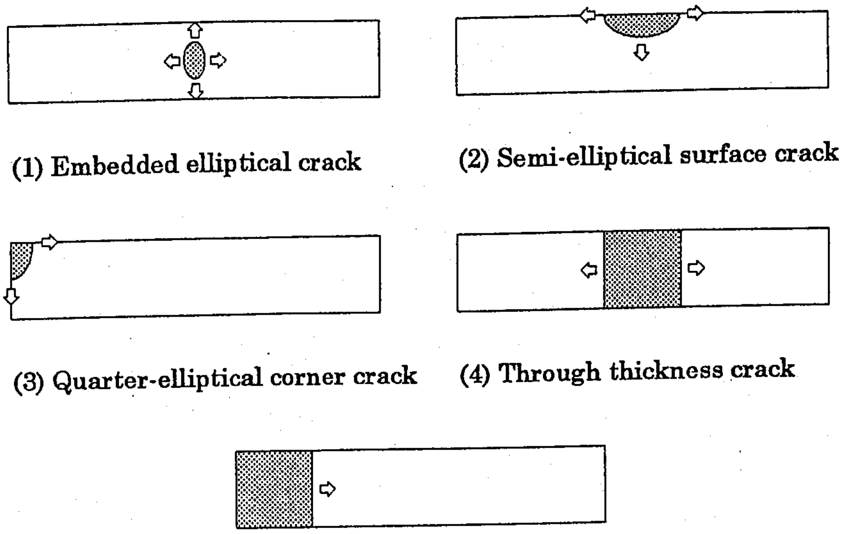
Particularly in cases where Eq. (A.3) is applied, it must be considered in the calculation that the stress range which corresponds to the threshold of the stress intensity factor range ΔK_{th} varies as the fatigue crack propagates. In other words, the stress range cut-off limit for obtaining the equivalent stress range changes as the crack size becomes large.

A.7.2 The crack propagation should be evaluated in all directions on the crack plane. Four directions should be considered for an embedded elliptical crack, three directions for a semi-elliptical surface crack, two directions for a through-thickness crack or a quarter-elliptical corner crack, and only one direction for an edge crack, as shown in Fig. A.6. But in cases where the change of the crack profile with crack propagation can be determined in advance, calculation in only one direction may be enough.

A.7.3 The numerical integration of Eq. (A.1) is made by step-wise calculation as follows :

(1) The stress intensity factor range for a given crack size is obtained from the equation for stress intensity factor range in Section A.6.
(2) The amount of crack extension for a given number of stress cycles is calculated from the crack propagation rate in Section A.5.
(3) The crack size is enlarged by the amount of crack extension obtained above.

It is recommended that the number of stress cycles for one step of this stepwise calculation is appropriately selected so that the total number of steps is greater than 100 before reaching the critical crack size or the fatigue life.



(1) Embedded elliptical crack

(2) Semi-elliptical surface crack

(3) Quarter-elliptical corner crack

(4) Through thickness crack

(5) Edge crack

Fig.A.6 Directions of crack propagation to be considered in the analysis

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