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European standard for fatigue design of steel structures and perspectives

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Abstract

The drafting of a European standard for the fatigue design of steel structures has started during the eighties with, as a basis, the ECCS (*European Convention for Constructional Steelwork, Brussels*) fatigue recommendation. In EN1993-1-9, one can recognize ECCS original work; however, a lot of new knowledge has been included. The code continues to use the nominal stress approach together with a set of 14 S-N curves equally separated. However, rules for verification with the structural stress approach are now also given. Several concepts included in this code are European specificities, and are presented in relation with AISC Spec fatigue provisions. This concerns the fatigue damage equivalent factor concept, which allows to use a simple check format and still account for real loading effects, the treatment of the size effects on fatigue strength, the link between fatigue and fracture criteria, and the partial resistance factor choice. Ideas for code revision and further developments are given.

Keywords: standard; fatigue; damage equivalence; resistance curves; geometric stress; perspectives

1 Introduction

The European standard family system

There are so far two sources of International Standard Families in the world, one in the USA and one in Europe. Both these International Standard Families regulate the construction market and its services through product standards, testing codes and design codes. The European standard family is prepared by the European Committee for Standardization (*CEN*) and includes so far 10 Eurocodes with design rules, for a total of 58 parts, and many hundreds of EN-standards for products and testing. It also contains so far around 170 European Technical Approvals (*ETA's*) and European Technical Approval Guidelines (*ETAG's*), all prepared by the European Organization for Technical Approvals (*EOTA*). For steel structures, the relevant parts of the European international standard family are shown on figure 1.

In figure 1, the colors indicate the state of advancement of the various design standard parts, the parts indicated in light grey, mainly general rules parts (Part 1), having already the status of national standards within the CEN members. Figure 2 gives a survey on the various parts of Eurocode 3 involved in the design of steel bridges. Within the design standards containing the general rules, two parts are related to fatigue. This is part 1-10: material toughness and through-thickness properties (material quality selection) [prEN1993-1-10:2005], and part 1-9: fatigue [prEN1993-1-9:2005].

Material quality selection

When a steel structure is built, it shall satisfy the requirements for the execution of steel structures (from the codes EN 1090, execution and ISO 5817, welding) but will however contain imperfections that are within the required tolerances. Since it is possible that fatigue cracks develop from these imperfections, or that undetected defects may be present, brittle failure shall be excluded by a proper choice of the material. Part 1-10 of Eurocode 3 provides a method for selecting such a material, for the different possible applications and service conditions (characterized by a minimum reference temperature and stress level). The method in the code is summarized in figure 3. It is based on fracture mechanics calculations and a failure assessment diagram such as the R6 method [PD 7910:1999]. For a given detail, the accidental existence of a defect (modeled as an initial crack with size a_0), that should normally have been detected and repaired during welding inspection, is assumed. Under service conditions, the initial crack may grow due to fatigue loading to a size a_d until it is detected during an inspection. As a conservative assumption, fatigue crack initiation is not considered in the computation because of the severity of the defects in welded structures. Failure will occur after crack growth either by yielding of the remaining section or by brittle failure if the material in front of the crack isn't tough enough. The last case has to be avoided by proper choice of material. With assumptions on the fatigue damage occurring between two inspections, one can compute using fracture mechanics principles the crack size a_d and perform a safety check by comparing stress intensity factors values ($K_{appl,d} \leq K_{mat,d}$). The computations were carried out for various details and loading conditions [RWTH, 2001].

Since the member thickness influences both fatigue strength and brittle fracture, the material selection is presented in the form of tables, as presented in table 1, with maximum allowable product thicknesses for different steel grades, minimum reference temperatures and stress levels. The stress level considered in this selection is the one corresponding to an accidental load combination, owing to the fact that it occurs with the assumption of having simultaneously the lowest temperature, the presence of a crack, and the lowest admissible material properties. Once the material is properly selected, it can be assumed that fatigue cracking can occur without resulting in a brittle fracture and thus fatigue assessment can be undertaken using either a damage tolerant approach or a safe life one, see paragraph on partial resistance factor in section 2.

2 Fatigue assessment

Verification format

In the mid-eighties, the ECCS (European Convention for Constructional Steelwork, Brussels) published recommendations for the fatigue design of steel structures [ECCS, 1985]. The first European standard for the fatigue design of steel structures was based on these recommendations and so is the new code for fatigue design, Eurocode 3 Part 1-9. The principle of the fatigue verification, which has remained the same, is based on the classification method, or nominal stress method. In this method, a comparison between stress ranges from the loading to the fatigue strength at two million cycles is made. For direct stress range, it can be written as follows:

$$\gamma_{Ff}\Delta\sigma_{E2} \leq \Delta\sigma_C / \gamma_{Mf} \tag{1}$$

with

$$\gamma_{Ff} \cdot \Delta\sigma_{E2} = \lambda \cdot \sigma(\gamma_{Ff} Q_k) \tag{2}$$

where

γ_{Ff}	partial factor on actions, recommended value is 1.00 (see EN 1991)
γ_{Mf}	partial factor for fatigue strength
$\Delta\sigma_{E2}$	equivalent constant amplitude stress range related to 2 million cycles
$\Delta\sigma_C$	reference value of the fatigue strength at 2 million cycles
λ	global damage equivalent factor, see section 3
$\Delta\sigma(\gamma_{Ff}\, Q_k)$	stress range caused by the fatigue loads specified in EN 1991, loads which, depending on the type of structure, already include a dynamic amplification factor.

In the case of shear stress range, the verification is similar. The verification using the geometric (hot spot) stress approach has also been integrated in the code. The verification format stays the same and the hot spot stress range is computed using FEM analysis or parametric formulas. The corresponding S-N curves are explained in section 4. This method is particularly relevant for the design of tubular structures, in conjunction with the CIDECT publication for fatigue design [CIDECT, 2000]. Moreover, Part 1-9 contains a table (table 8.7) with detail categories for circular and rectangular tubular joints to be used with the classification method, but its application range is limited to small tubes (diameter ≤ 300 mm and thickness ≤ 8 mm).

The damage equivalent factor, called above global, is a combination of several different λ_i factors which allows for taking into account the real traffic, the static system, the service life value and the influence of more than one fatigue load on the structure (train crossings, two cranes on the same supporting beams, etc.). This is explained further in the next section. This verification corresponds to a simplified procedure; it is however always possible to perform a verification using the Miner damage accumulation rule, limiting value being one, when knowing the real loading spectra on the structure (annex A of part 1-9).

Damage equivalent factor concept

The fatigue check of a new structure subjected to a load history is complex and requires the knowledge of the loads the structure will be subjected to during its entire life. Assumption about this loading can be made, still leaving the engineer with the work of doing damage accumulation calculations. The concept of the fatigue damage equivalent factor was proposed to eliminate this tedious work and put the burden of it on the code developers.

The computation of the usual cases is made once for all. The concept of the damage equivalent factor is described in figure 4, where $\gamma_{Ff}\, Q_k$ is replaced by Q_{fat} for simplicity. On the left side of the figure, a fatigue check using real traffic is described. On the right side, a simplified model is used. The damage equivalent factor λ links both calculations in order to have damage equivalence.

The description of the procedure on the left side, procedure that was used by the code developers, is :

1. Modeling of real traffic and displacement over the structure,
2. Deduction of the corresponding stress history (at the detail to be checked),
3. Calculation of the resulting stress range histogram $\Delta\sigma_i$,
4. Calculation of the damage accumulation (using a accumulation rule, usually a linear one : Palmgren-Miner),
5. Either check using total damage must remain inferior to one (in this case, the detail category must be known to make the damage accumulation) or deduction of the resulting equivalent stress range $\Delta\sigma_e$ (or $\Delta\sigma_{E,2}$ for the value brought back at 2 million cycles) and check by comparing it with the detail category curve.

This procedure is relatively complex, notably in comparison with usual static calculations where simplified load models are used. It is however possible to simplify the fatigue check, using a load model specific for the fatigue check, in order to obtain a maximum stress σ_{max} and minimum stress σ_{min}, by placing this load model each time in the most unfavorable position according to the influence line of the static system of the structure. But the resulting stress difference $\Delta\sigma(\gamma_{Ff}\, Q_k)$, due to the load model, does not represent the fatigue effect on the bridge due to real traffic loading! In order to have a value corresponding to the equivalent stress difference $\Delta\sigma_{E,2}$, one must correct the value $\Delta\sigma(\gamma_{Ff}\, Q_k)$ with what is called a damage equivalent factor, λ, computed as :

$$\lambda = \gamma_{Ff}\Delta\sigma_{E2} \Big/ \Delta\sigma\left(\gamma_{Ff} Q_k\right) \tag{3}$$

The calculation of the correction factor values are made once for all for the usual cases, and are in function of several parameters such as the real traffic loads (in terms of vehicle geometry, load intensities and quantity) and influence line length, to mention the more important ones.

The main assumptions are the use of the rainflow counting method and of a linear damage accumulation rule. One can therefore not account for phenomena such as crack retardation, influence of loading sequence, etc. The S-N curves must belong to a set of curves with slope changes at the same number of cycles, but he curves can have more than one slope. This is the case for the set of curves in ECCS or in prEN1993-1-9. The simplified load model has to be not too far from reality (average truck or train). If not, there are some abrupt changes in the damage equivalence factor values when the influence line length value approaches the axle spacing. In the Eurocodes, the fatigue load models for different types of structures can be found in the various parts of Eurocode 1. The damage equivalent factor has been further split into four partial damage equivalence factors in order to allow for more parameters to be accounted for:

$$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \quad but \quad \lambda \le \lambda_{\max} \tag{4}$$

where:

λ_1 factor accounting for the span length (in relation with the length of the influence line), see figure 5 for values for road and rail bridges.

λ_2 factor accounting for the traffic volume

λ_3 factor accounting for the design life of the structure, the reference life for which the factor is one being, for road bridges, equal to 100 years.

λ_4 factor accounting for the influence of more than one load on the structural element

λ_{max} maximum damage equivalent factor value, taking into account the fatigue limit.

The damage equivalent factor λ_1 values depend on the closeness of the fatigue load model to real loadings, thus a direct comparison between the different curves plotted in figure 5 cannot be made. However, it can be seen that the correction is much more significant in the case of road bridges. This means that the load model does not represent closely the real traffic loads and volume. It also explains why there is a need for two different curves depending on the position of the detail on the bridge (midspan or support). The opposite is true for the railway bridges and the damage equivalent factors are closer to unity.

The limiting maximum damage equivalent factor value, λ_{max}, is dictated by fact that the multiplication of the individual partial factor may result in a value far exceeding the one obtained from a design using the fatigue limit. Again, there is a significant difference between road and railway bridges. In the case of railway bridges, the load model represents an upper bound value in terms of the maximum stress range it generates. Thus, the limiting value is bound by the CAFL value, $\Delta\sigma_D$. It can be expressed as:

$$\lambda_{\max} = \frac{\Delta\sigma_C}{\Delta\sigma_D} = \left(\frac{5}{2}\right)^{1/3} = 1.36 \qquad (5)$$

This value was rounded up to 1.4 in Eurocode 3, part 2 [prEN1993-2].

In the case of road bridges, it cannot be expressed as a single value since the load model does not represent an upper bound value in terms of the maximum stress range it generates. Therefore, as for the damage equivalent factor λ_1, simulations must be carried out. It results in values for λ_{max} comprised between 1.8 and 2.7, in function of the bridge span as for λ_1 [prEN1993-2].

3 Fatigue strength

Fatigue strength curves

The fatigue strength curves correspond to the original set of 14 curves from ECCS. All curves are parallel and each curve is characterized by the detail category (value of the fatigue strength at 2 million cycles) and a slope change at 5 million cycles. For shorter lives, a slope coefficient m = 3 is used. For longer lives, the slope coefficient m = 5 is used, until 100 million cycles. This last value corresponds to the cut-off limit, which means that all cycles having stress ranges below the stress range value at 100 million can be neglected when performing a damage accumulation because their contribution to the total damage is considered as being negligible. The double slope S-N curve represent better the damaging process due to cycles below the constant amplitude fatigue limit (CAFL) when the spectra follows a distribution close to Rayleigh's, compared to the unique slope curve. The CAFL is fixed at 5 million cycles for all detail categories, which is not the case in the AISC code where this value ranges from 1.8 to 22 million cycles. Similarly to AISC, the different structural details are classified into different tables (non-welded details, butt welds, weld attachments, …) and detail categories. Figure 6 shows a comparison between EN1993-1-9 and AISC classifications for a bolted joint (mechanically fastened joint) as well as for a longitudinal attachment (without transition radius and, in EN, for attachment length over 100 mm). As can be seen, the strength curves do not differ much in finite life part, but differ significantly when looking at the CAFL and variable amplitude extensions. The debate is continuing over the number of cycles corresponding to the fatigue limit under constant amplitude loading. The Eurocode continues to use 5 millions cycles for all detail categories because of the use of the damage equivalence factor, even if it seems to be acknowledged that this limiting value increases with an increase in notch severity, i.e. with decreasing detail categories.

In both codes, the detail category attributed to a particular detail comes from statistical analysis of the available worldwide test results on specimen of sufficient size to represent correctly the built-in welding residual stresses. Table 2 shows some structural details and the results, still partial at this time, of the statistical analysis carried out within the Eurocode framework to check the classification. As can be seen, test results from different sources cause a large scatter in the characteristic fatigue strength. The data can however not be pooled together without careful analysis as they may not be from the same population because of significant differences in the welding process or procedure specifications, in the failure criteria, etc. Also, the test database includes tests dating from the 1960's up to today. Therefore, the classification adopted is both quantitative as well as qualitative and includes some engineering judgment. For attachments, one can see that the detail categories at the limits of the validity range for different details are often identical. For example, a very short longitudinal attachment (L < 50mm) is not different from a large transverse stiffener on a plate ($l \leq 50$ mm) or a cruciform full penetration joint ($l \leq 50$ mm); they are all classified as category 80.

In both the EN as well as AISC, the test results considered were all carried out in a laboratory environment and therefore the code requires an adequate corrosion protection system; all the same, the allowable temperature range is limited to -50 ° and +150°C. In EN1993-1-9, the range of application extends to structures in weathering steels, but the categories for plain members must be lowered by one category to account for possible rust points. It also extends to steels with yield stress up to 690 N/mm^2 as well as to austenitic steels.

Size effects

In part 1-9, the influence of the size of the detail on its fatigue strength is recognized in different ways. Firstly, the test results used to fix the fatigue strengths of the details were carried out on specimens with dimensions that are sufficient to represent correctly the built-in welding residual stresses. Secondly, some details in the tables have been separated according to the variation of one or two geometrical dimensions; for example a longitudinal attachment can corresponds to four different categories according to the attachment length (see table 2). This can be called a non-proportional scaling effect, since only some dimensions are scaled and not the others; it is also accounted for in the AISC. Thirdly, for cases that are close to proportional scaling, one can see that the size effect in fatigue is essentially influenced by the plate thickness in which the fatigue crack grows and therefore has often been called the “thickness effect”. This effect is not accounted for in AISC. For these cases, the reduction formula for size effects proposed originally by Gurney is used in part 1-9:

$$\Delta\sigma_{C,red} = k_s \cdot \Delta\sigma_C \qquad with \quad k_s = \left(25/t\right)^n < 1.0 \tag{6}$$

The value of the exponent n in the formula (6) is function of the detail considered. In part 1-9, it is equal to 0.2 for butt joints and 0.25 for bolts in tension. In IIW recommendations [IIW 2003], the exponent n takes values comprised between 0.1 and 0.4 depending upon the detail considered (the exponent increases proportionally to the stress concentration factor at the crack location).

Partial resistance factor

Regarding the partial resistance factor for fatigue, it has no more a unique value as in the previous codes. Its value varies according to the particularities of the structure or element designed (for example redundancy, regular inspection) as well as the consequences of a failure. Two fatigue assessment methods can be differentiated: damage tolerant and safe life. In the damage tolerant method, an acceptable reliability level is achieved through prescribed inspection and maintenance plans throughout the structure’s life. Redundant static systems and structural elements allowing for load redistribution, such as orthotropic deck details, multi-planar tubular trusses, etc., are concerned with this method. With the safe life method, an acceptable reliability level can be achieved with a structure performing satisfactorily for its design life without the need for regular in-service inspection for fatigue damage. Inspections and maintenance for other reasons, corrosion protection system, bridge bearings, expansion joints, etc., will however be carried out. The acceptable reliability level is achieved by specifying different values for the partial factor for fatigue strength γ_{Mf}, see table 3.

The choice of a partial factor is however not easy as the criteria are fuzzy. It must also be said that the values in table 3 are only recommended ones. Thus, every CEN member state has the right to fix its own values. In order to use the damage tolerant method, the following requirements must all be met :

- selection of details, materials and stress levels so that in the event of the formation of cracks, a low rate of crack propagation and a long critical crack length would result,
- provision of multiple load path, that means alternative load paths must exist when a fatigue crack develops,
- provision of crack-arresting details,
- provisions of details easy to inspect during regular inspections.

When using the safe-life method, the details and stress levels must be chosen in order to guarantee a reliability index value equal to those for ultimate limit state verifications at the end of the design service life.

Logically, one sees that the possibility of detecting cracks or other signs of damage during inspections and thus to be able repair them on time –this is particularly true when there is redundancy in the structure– decreases the uncertainty in structural reliability and is acknowledged for through a reduction of the partial factor for fatigue strength γ_{Mf}.

The other parameter considered in table 3 is the consequence of failure. Even though redundancy influences the failure consequences, it is not what is meant by this parameter in the code philosophy. The consequences of failure are related to the perception of the engineer and of the owner regarding: partial or total failure, importance of the structure in the network, probability of human live losses when failure occurs (from people in, on or near the structure), etc.

4 Special considerations

Geometric stress resistance curves

The geometric stress approach has been proven to be the best solution to properly account for the complexity in stress distribution in welded details. The design value of the geometric stress range can be computed by multiplying the nominal stress range by a stress concentration factor. This is given in the form of the following equation in the part 1-9:

$$\gamma_{Ff} \cdot \Delta\sigma_{E2} = \lambda \cdot k_f \cdot \Delta\sigma\left(\gamma_{Ff} Q_k\right) \tag{7}$$

with:

k_f Stress concentration factor taking into account stress concentration effects due to the overall geometry of a particular constructional detail, the local stress concentration effects e.g. from the weld profile shape already being included in the geometric stress resistance curves.

Thus, the same verification format as given by expression (1) can be used, but with the set of geometric stress resistance curves (detail categories) given in Annex B. The determination of the stress concentration factor can be determined either by FEM calculations using a validated standard procedure as explained in [IIW 2003], or by using parametric formulas such as the ones existing for tubular joints [CIDECT 2000].

For the application of the geometric stress method, the detail categories different detail categories are given in function of the location of the crack and the geometry of the weld. This result in different categories for crack initiating at

- toes of butt welds,
- toes of fillet welded attachments,
- toes of fillet welds in cruciform joints.

Examples of detail categories are given in table 4. Note that for tubular joints, the CIDECT recommendations [CIDECT 2000] make the geometric stress resistance curves depending upon the tube wall thickness.

Web breathing limitations

Another fatigue problem is the design against cyclic out-of-plane displacements that can occur in slender webs of plate girders under fatigue loads. Eurocode 3, Part 2 contains a verification formula with a limit on the combination of normal and shear stress ranges values. This fatigue verification is rather complicated but an alternative is proposed. It consists in plate slenderness limitations. In order not to have to verify web breathing, the following criteria for slenderness in length direction of non-stiffened plates are set :

$b/t \leq 30 + 4.0 \cdot L$ and b/t ≤ 300 for road bridges (8)

$b/t \leq 55 + 3.3 \cdot L$ and b/t ≤ 250 for railway bridges (9)

with

L bridge span in [m] and L ≥ 20 m

b, t plate width and thickness

The background for these slenderness limitation formulas comes from numerous simulations of damage accumulation made by [Kuhlmann & Günther 2002] on web plates with imperfections from bridge main girders under realistic load models.

5 Perspectives

The trend continues to be in the direction of more slender, lighter and architecturally stunning civil engineering structures, often in conjunction with the use of steels with higher performances. This results in structures subjected to higher stresses under service conditions and thus to increased significance of fatigue verifications compared to other limit states. The current detail categories are based on test data mixing different steel grades, welding processes and procedure specifications; for selected details, this could result in the future in different detail categories. The engineers have also tools to build more complex models to predict structural behavior, particularly under static loads. Both the assumptions behind these models and the resulting stresses ranges may not be suitable for verifications with the current fatigue design rules, in particular when using the nominal stress approach. In addition, an optimal static design with modern tools decreases the hidden reserves in some structural elements and may trigger fatigue problems not experienced before. Moreover, fatigue evaluation of existing civil engineering works requires more detailed models in order to better predict crack locations, remaining fatigue lives and establish inspection plans.

These evolutions should be acknowledged by developments in the existing fatigue verification rules. I think the emphasis of future normative developments within the range of fatigue lie therefore in the following areas:

- The geometrical or hot-spot stress method shall be promoted and developed. A reanalysis of test data for selected details on this basis should be carried out to propose an alternative geometric stress detail classification for these details, including if possible a difference in classification according to the welding process or quality.
- The question of differentiation of the fatigue strength for different steels grades in the case of non-welded details, in particular bolted connections, remains and shall be addressed in a further revision of part 1-9.
- The handling of the safety concept for the verification using the total damage sum and its limiting value D_{lim} = 1.0 shall be reviewed and depend directly on the different required reliability levels required instead of partial safety factors on action effects and detail categories.

- Explicit rules for post-weld treatments with corresponding higher detail categories shall be included as they are to be used more in the future both for existing and new structures made out of higher steel grades.
- The handling of the size effect shall be revised to differentiate more clearly the influence of thickness on different details and include other geometrical parameters where needed, as for example in tubular connections.

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Tables

Table 1. Examples of maximum allowable plate thickness (extract from EN 1993-1-10).

Steel grade	Sub-grade	Charpy energy CVN		Reference temperature T_{Ed} [°C]													
		at T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
				$\sigma_{Ed} = 0{,}50\ f_y(t)$							$\sigma_{Ed} = 0{,}25\ f_y(t)$						
S235	JR	20	27	90	75	65	55	45	40	35	135	115	100	85	75	65	60
	J0	0	27	125	105	90	75	65	55	45	175	155	135	115	100	85	75
	J2	-20	27	170	145	125	105	90	75	65	200	200	175	155	135	115	100
S355	JR	20	27	65	55	45	40	30	25	25	110	95	80	70	60	55	45
	J0	0	27	95	80	65	55	45	40	30	150	130	110	95	80	70	60
	J2	-20	27	135	110	95	80	65	55	45	200	175	150	130	110	95	80
	K2,M,N	-20	40	155	135	110	95	80	65	55	200	200	175	150	130	110	95
	ML,NL	-50	27	200	180	155	135	110	95	80	210	200	200	200	175	150	130
S420	M,N	-20	40	140	120	100	85	70	60	50	200	185	160	140	120	100	85
	ML,NL	-50	27	190	165	140	120	100	85	70	200	200	200	185	160	140	120
S460	Q	-20	30	110	95	75	65	55	45	35	175	155	130	115	95	80	70
	M,N	-20	40	130	110	95	75	65	55	45	200	175	155	130	115	95	80
	QL	-40	30	155	130	110	95	75	65	55	200	200	175	155	130	115	95
	ML,NL	-50	27	180	155	130	110	95	75	65	200	200	200	175	155	130	115
	QL1	-60	30	200	180	155	130	110	95	75	215	200	200	200	175	155	130

Table 2. Longitudinal attachment details and results of statistical evaluation [Sedlacek, 2004].

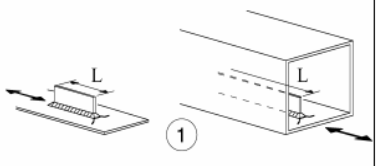
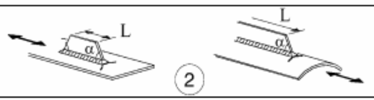
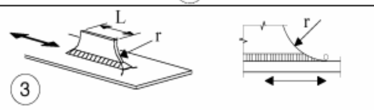
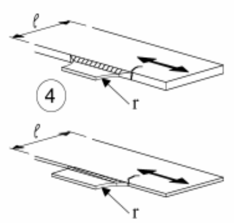
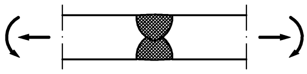
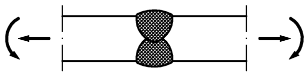
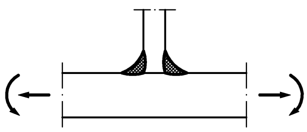
Detail category	Constructional detail		New Evaluation for prEN 1993-1-9 (2002)					
			Detail	# of data	m		$\Delta\sigma_c$	
					variable	constant	m=var.	m=const.
80	L≤ 50mm		1	17	3,26	3	89,10	87,00
71	50<L≤ 80mm							
63	80<L≤ 100mm			109	2,45	3	67,04	77,14
56	L>100mm			62 18 15 17 6 8 12 4 9	3,24 3,32 3,05 3,27 3,81 3,48 3,54 4,59 2,43	3 3 3 3 3 3 3 3 3	58,06 76,08 94,57 79,37 80,59 84,22 69,12 78,39 53,43	55,96 72,31 94,73 76,49 59,53 76,73 60,14 50,12 74,94
71	L>100mm α<45°		2	53 27 39	2,92 2,99 2,73	3 3 3	69,24 58,97 78,95	70,58 59,85 83,41
80	r>150mm	 reinforced	3	6 4 4 10	3,06 3,29 3,31 3,12	3 3 3 3	100,94 97,27 36,16 59,00	105,20 96,41 62,94 63,76
90	$\frac{r}{L} \geq \frac{1}{3}$ or r>150mm		4					
71	$\frac{1}{6} \leq \frac{r}{L} \leq \frac{1}{3}$			13	1,26	3	18,43	72,84
50	$\frac{r}{L} < \frac{1}{6}$							

Table 3. Recommended values for partial factors for fatigue strength γ_{Mf} (table 3.1 from EN 1993-1-9).

Assessment method	**Consequence of failure**	
	Low consequence	High consequence
Damage tolerant	1,00	1,15
Safe life	1,15	1,35

Table 4. Examples of geometric stress classification according to EN1993-1-9.

Category	Detail	Description	Requirements
112		Full penetration butt joint (X- or V-weld)	All welds ground flush to plate, checked by NDT
100		Full penetration butt joint	Plate edges to be ground flush in direction of stress, weld angle ≤ 30°, checked by NDT
100		Non load-carrying fillet welds (transverse attachment)	Weld angle ≤ 60°

Figures

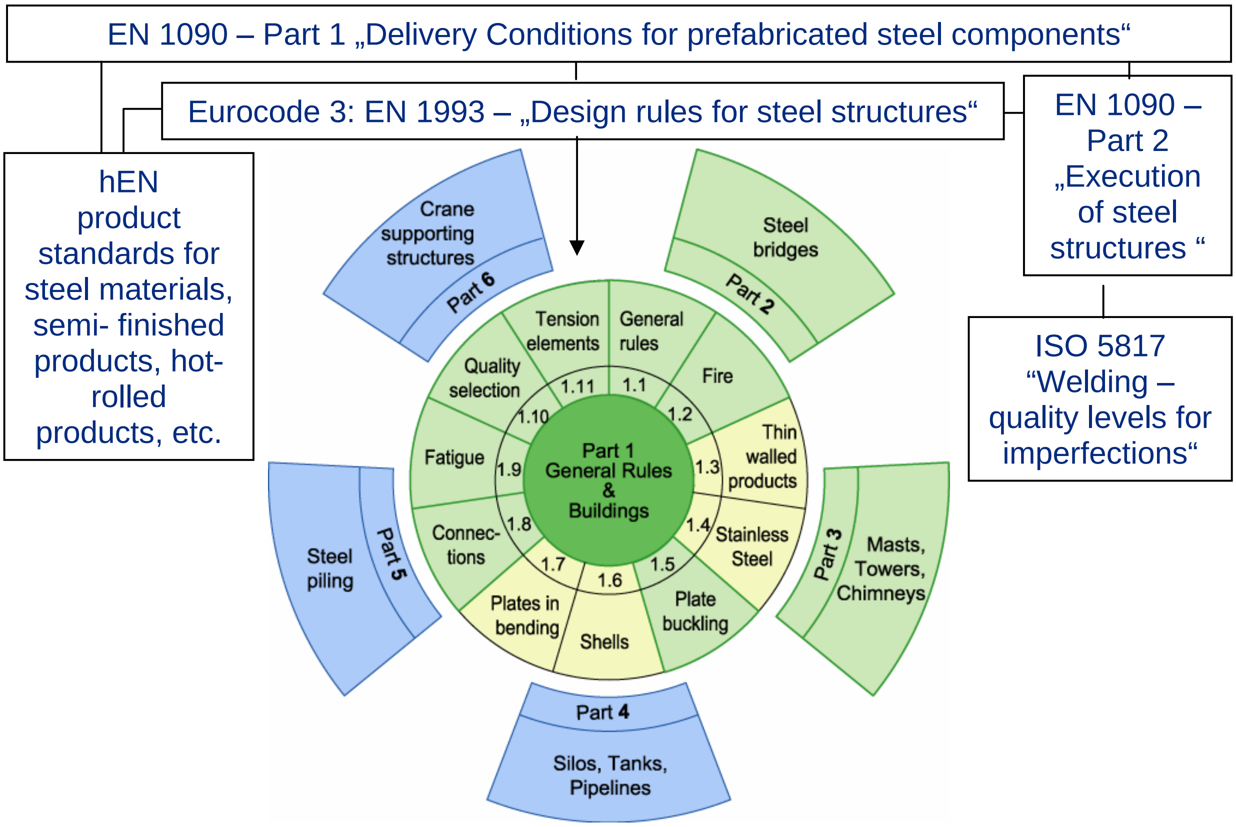


Figure 1. Standard system for steel structures, extract from [Schmackpfeffer, 2005].

EN 1993-Part 1-1 General rules
- **1-5 Plate buckling**
- **1-8 Connections**
- **1-9 Fatigue**
- **1-10 Choice of material**
- **1-11 Rope structures**

EN 1993-Part 2 Steel bridges
- **Annex A Requirements for bearings**
- **Annex B Requirements for expansion joints**
- **Annex C Recommendations for orthotropic plates**

Figure 2. Design rules for steel bridges within Eurocode 3 parts, extract from [Schmackpfeffer, 2005].

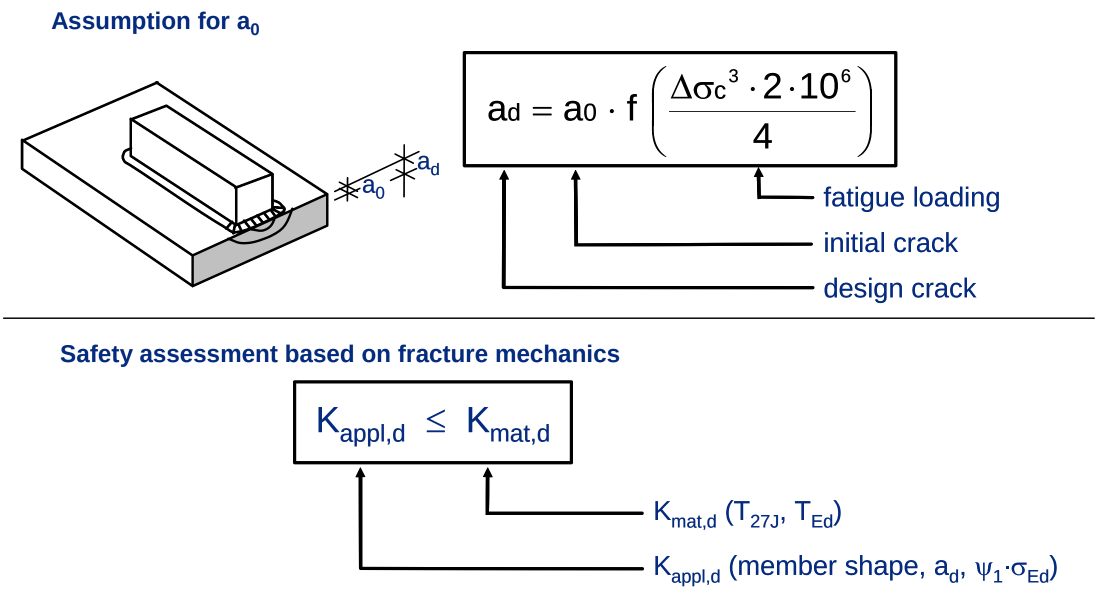


Figure 3. Method used in part 1-10 of Eurocode 3 for avoiding brittle fracture [Schmackpfeffer, 2005].

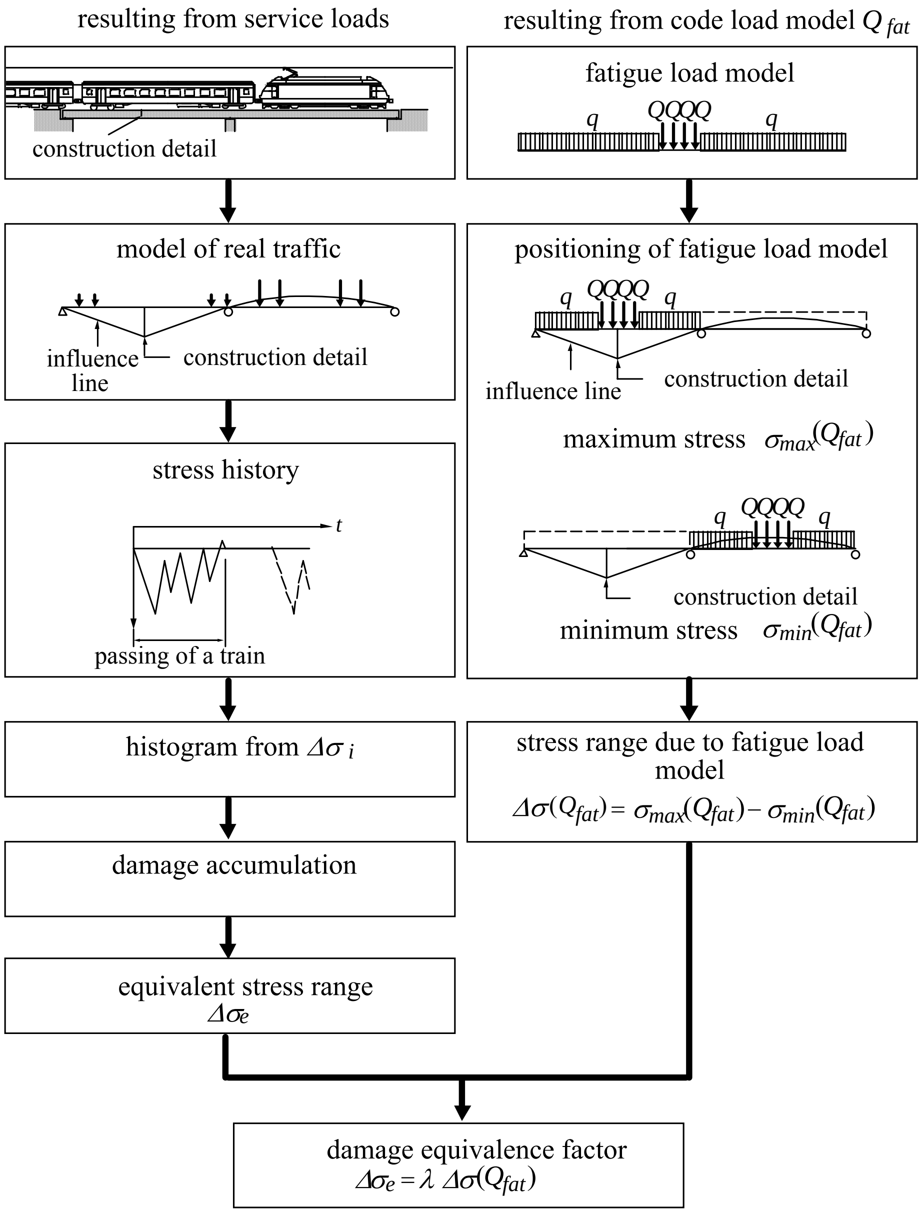


Figure 4. Damage equivalence factor.

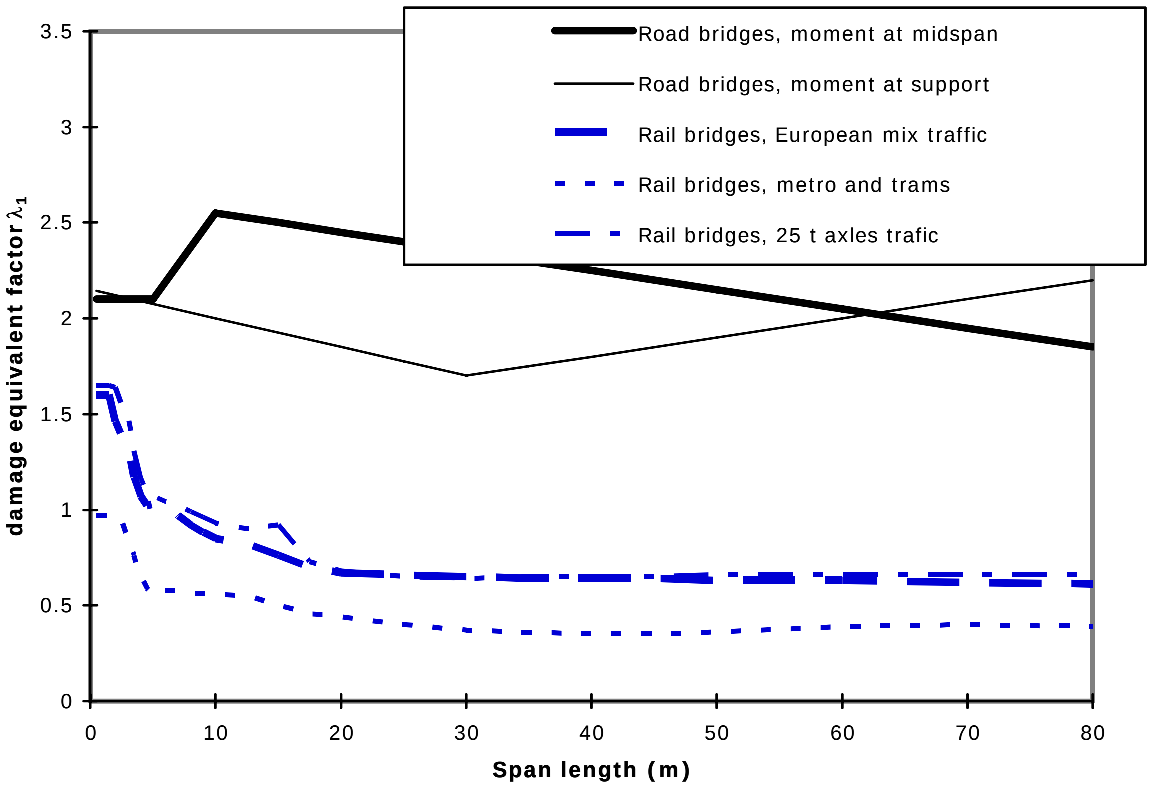


Figure 5. Damage equivalence factor values for road and rail bridges.

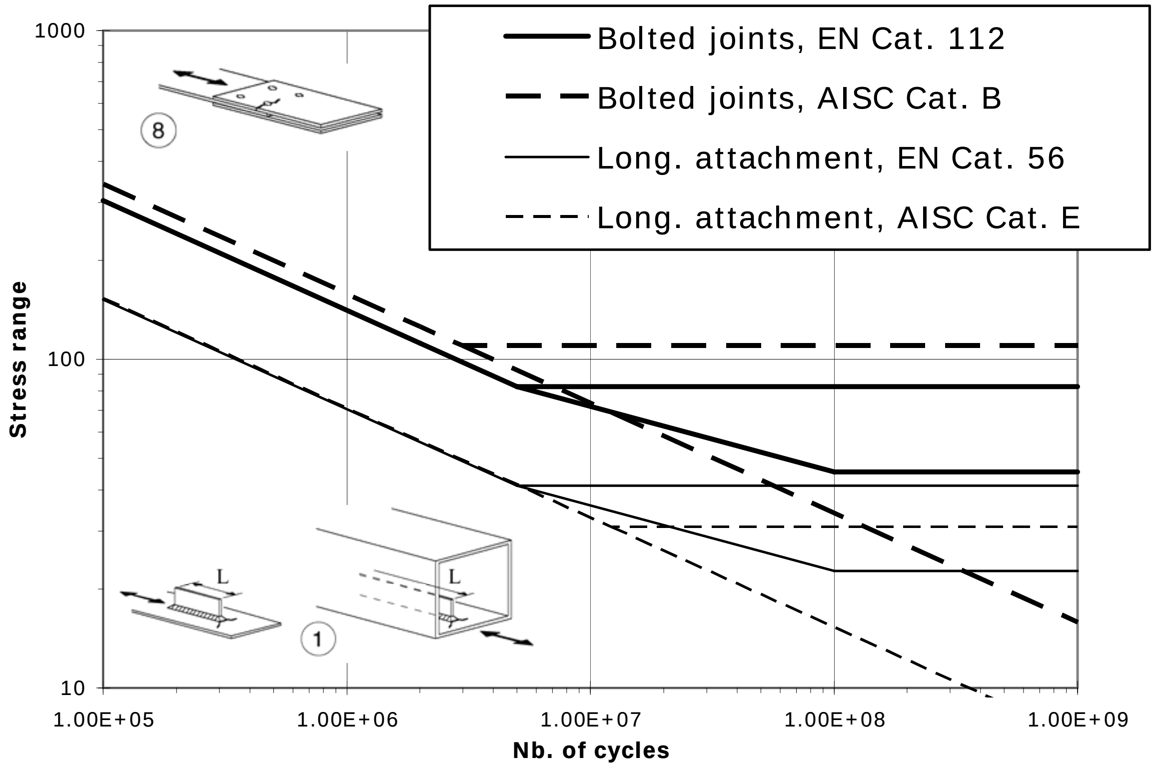


Figure 6. Comparison between EN and AISC fatigue strength curves for bolted joints and longitudinal attachments