Chapter 1

INTRODUCTION

1.1 BASIS OF FATIGUE DESIGN IN STEEL STRUCTURES

1.1.1 General

Fatigue is, with corrosion and wear, one of the main causes of damage in metallic members. Fatigue may occur when a member is subjected to repeated cyclic loadings (due to action of fluctuating stress, according to the terminology used in the EN 1993-1-9) (TGC 10, 2006). The fatigue phenomenon shows itself in the form of cracks developing at particular locations in the structure. These cracks can appear in diverse types of structures such as: planes, boats, bridges, frames (of automobiles, locomotives or rail cars), cranes, overhead cranes, machines parts, turbines, reactors vessels, canal lock doors, offshore platforms, transmission towers, pylons, masts and chimneys. Generally speaking, structures subjected to repeated cyclic loadings can undergo progressive damage which shows itself by the propagation of cracks. This damage is called *fatigue* and is represented by a loss of resistance with time.

Fatigue cracking rarely occurs in the base material remotely from any constructional detail, from machining detail, from welds or from connections. Even if the static resistance of the connection is superior to that of the assembled members, the connection or joint remains the critical place from the point of view of fatigue.

Figure 1.1 shows schematically the example of a steel and concrete composite road bridge subjected to traffic loading. Every crossing vehicle results in cyclic actions and thus stresses in the structure. The stresses induced are affected by the presence of attachments, such as those connecting the cross girders to the main girders. At the ends of attachments, particularly at the toes of the welds which connect them with the rest of the structure, stress concentrations occur due to the geometrical changes from the presence of attachments. The very same spots also show discontinuities resulting from the welding process.

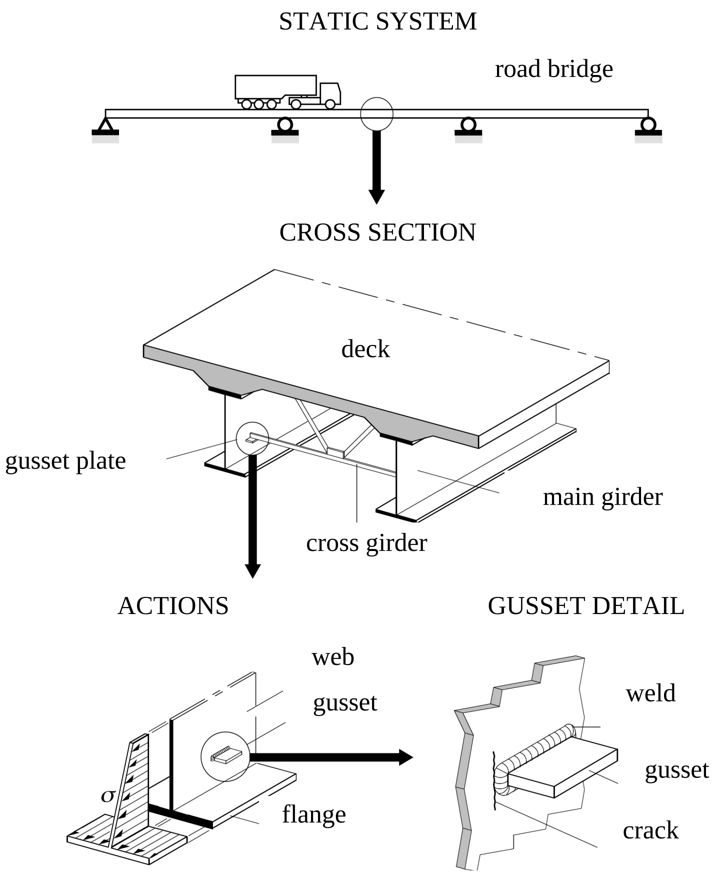


Figure 1.1 – Possible location of a fatigue crack in a road bridge (TGC 10, 2006)

Numerous studies were made in the field of fatigue, starting with Wöhler (1860) on rail car axles some 150 years ago. These demonstrated that the combined effect of discontinuities and stress concentrations could be the origin of the formation and the propagation of a fatigue crack, even if the applied stresses remain significantly below the material yield stress (by applied stresses, it is meant the stresses calculated with an elastic structural analysis considering the possible stress concentrations or residual stresses). A crack develops generally from discontinuities having a depth of the order of some tenth of millimetre. The propagation of such a crack can lead to failure by yielding of the net section or by brittle fracture, mainly depending upon material characteristics, geometry of the member, temperature and loading strain rate of the section. Thus, a structure subjected to repeated cyclic loadings has to be done by careful design and fabrication of the structural members as well as of the structural details, so as to avoid a fatigue failure. The methods of quality assurance have to guarantee that the number and the dimensions of the existing discontinuities stay within the tolerance limits. The purpose of this sub-chapter is to present an outline of the fatigue phenomenon, in order to provide the basic knowledge for the fatigue design of bolted and welded steel structures. To reach this objective, the sub-chapter is structured in the following way:

- Section 1.1.2: The main factors influencing fatigue life are described.
- Section 1.1.3: Fatigue testing and the expression of fatigue strength are explained.
- Section 1.1.4: Variable amplitude and cycle counting.
- Section 1.1.5: Concept of cumulative damage due to random stresses variations.

The principles of fatigue design of steel structures are given in Eurocode 3, part 1-9. For aluminium structures, the principles are to be found in Eurocode 9, part 1-3, fatigue design of aluminium structures. The principles are the same, or very similar, for the different materials. All these standards are based on the recommendations of the European Convention for Constructional Steelwork (ECCS/CECM/EKS) for steel (ECCS, 1985) and for aluminium (ECCS, 1992).

1.1.2 Main parameters influencing fatigue life

The fatigue life of a member or of a structural detail subjected to repeated cyclic loadings is defined as the number of stress cycles it can stand before failure.

Depending upon the member or structural detail geometry, its fabrication or the material used, four main parameters can influence the fatigue strength (or resistance, both used in EN 1993-1-9):

– the stress difference, or as most often called *stress range*,
– the structural detail geometry,
– the material characteristics,
– the environment.

Stress range

Figure 1.2 shows the evolution of stress as a function of the time t for a constant amplitude loading, varying between $\sigma_{\min}$ and $\sigma_{\max}$. The fatigue tests (see following section) have shown that the *stress range* $\Delta\sigma$ (or stress difference by opposition to stress amplitude which is half this value) is the main parameter influencing the fatigue life of welded details. The stress range is defined by equation (1.1) below:

$$\Delta\sigma = \sigma_{\max} - \sigma_{\min} \tag{1.1}$$

where

$\sigma_{\max}$ Maximum stress value (with sign)
$\sigma_{\min}$ Minimum stress value (with sign)

Other parameters such as the minimum stress $\sigma_{\min}$, maximum stress $\sigma_{\max}$, their mean stress $\sigma_m = (\sigma_{\min} + \sigma_{\max})/2$, or their ratio $R = \sigma_{\min}/\sigma_{\max}$ and the cycle frequency can usually be neglected in design, particularly in the case of welded structures.

One could think, a priori, that fatigue life can be increased when part of the stress cycle is in compression. This is however not the case for welded members, because of the residual stresses (σ_{res} in tension introduced by welding). The behaviour of a crack is in fact influenced by the summation of the applied and the residual stresses (see Figure 1.2). A longer fatigue life can however be obtained in particular cases, by introducing compressive residual stresses through the application of weld improvement methods, or post-weld treatments, after welding (see section 4.1.5).

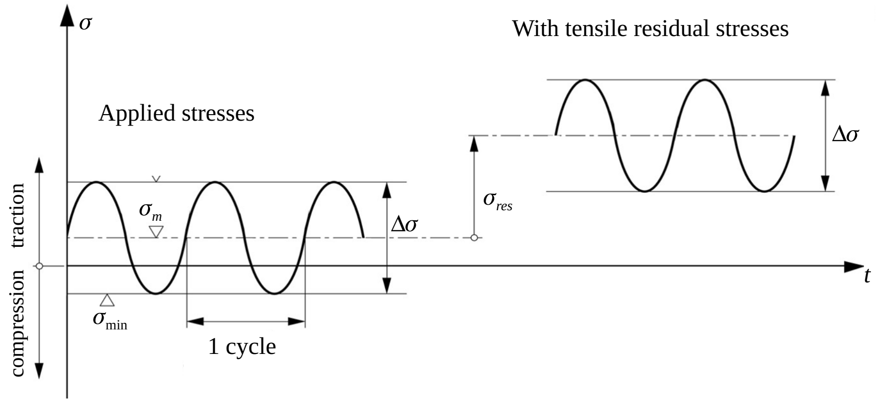


Figure 1.2 – Definition of stresses and influence of tensile residual stresses (TGC 10, 2006)

Structural detail geometry

The geometry of the structural detail is decisive in the location of the fatigue crack as well as for its propagation rate; thus it influences the detail fatigue life expectancy directly. The elements represented in Figure 1.1 allow to illustrate the three categories of geometrical influences:

- effect of the structure's geometry, for example the type of cross section;
- effect of stress concentration, due to the attachment for example;
- effect of discontinuities in the welds.

The effects of the structure's geometry and of the stress concentrations can be favourably influenced by a good design of the structural details. A good design is effectively of highest importance, as sharp geometrical changes (due for example to the attachment) affect the stress flow. This can be compared to the water speed in a river, which is influenced by the width of the river bed or by obstacles in it. In an analogous manner, stresses at the weld toe of an attachment are higher than the applied stresses. This explains why stress concentrations are created by attachments such as gussets, bolt holes, welds or also simply by a section change. The influence of discontinuities in the welds can be avoided by using adequate methods of fabrication and control, in order to guarantee that these discontinuities do not exceed the limiting values of the corresponding quality class chosen using EN 1090-2 (see section 1.3.4 for detailed information). Besides, it must be clarified that discontinuities in the welds can be due to the welding process (cracks, bonding imperfections, lack of fusion or penetration, undercuts, porosities, etc.) as well as to notches due to the rolling process, or to grinding, or also to corrosion pits. According to their shape and their dimension, these discontinuities can drastically reduce the fatigue life expectancy of a welded member. The fatigue life can be further reduced if the poor detail is located in a stress concentration zone.

Material characteristics

During fatigue tests on plain metallic specimens (i.e. non-welded specimens) made out of steel or aluminium alloys, it has been observed that the chemical composition, the mechanical characteristics as well as the microstructure of the metal often have a significant influence on the fatigue life. Thus, a higher tensile strength of a metal can allow for a longer fatigue life under the same stress range, due essentially to an increase in the crack initiation phase and not to an increase in the crack propagation phase. This beneficial influence is not present, unfortunately, in welded members and structures, as their fatigue lives is mainly driven by the crack propagation phase. In fatigue design, the influence of the tensile strength of the material has usually been neglected; there are only a few exceptions to this rule (machined joints and post-weld treated joints in particular). As a rule of thumb, the fatigue resistance of constructional details in aluminium can be taken as $^{1}/_{3}$ of those in steel, which is the ratio between the elasticity modulus of the materials.

Environment influence

A corrosive (air, water, acids, etc.) or humid environment can drastically reduce the fatigue life of metallic members because it increases the crack propagation rate, especially in the case of aluminium members. On one hand, specific corrosion protection (special painting systems, cathodic protection, etc.) is necessary in certain conditions, such as those found in offshore platforms or near chemical plants. On the other hand, in the case of weathering steels used in civil engineering, the superficial corrosion occurring in welded structures stay practically without influence on the fatigue life expectancy; the small corrosion pits responsible for a possible fatigue crack initiation are indeed less critical than the discontinuities normally introduced by welding.

The influence of temperature on fatigue crack propagation can be neglected, at least in the normal temperature range, but must be accounted for in applications such as gas turbines or airplane engines where high temperatures are seen. A low temperature can, however, reduce the critical crack size significantly, i.e. size of the crack at failure, and cause a premature brittle fracture of the member, but it does not affect significantly the material fatigue properties (Schijve, 2001).

Finally, in the case of nuclear power stations, where stainless steels are used, it is known that neutron irradiation induce steel embrittlement (English, 2007), thus making them more prone to brittle failure (chapter 6) and also reducing their fatigue strength properties.

1.1.3 Expression of fatigue strength

In order to know the fatigue strength of a given connection, it is necessary to carry out an experimental investigation during which test specimens are subjected to repeated cyclic loading, the simplest being a sinusoidal stress range (see Figure 1.2). The test specimen must be big enough in order to properly represent the structural detail and its surroundings as well as the corresponding residual stress field. The design of the experimental program must also include a sufficient amount of test specimens in order to properly measure the results scatter. Even under identical test conditions the number of cycles to failure will not be the same for apparently identical test specimens. This is because there are always small differences in the parameters which can influence the fatigue life (tolerances, misalignments, discontinuities, etc.). The test results on welded specimens are usually drawn on a graph with the number of cycles N to failure on the abscissa (or to a predefined size of the fatigue crack) and with the stress range $\Delta\sigma$ on the ordinate (Figure 1.3).

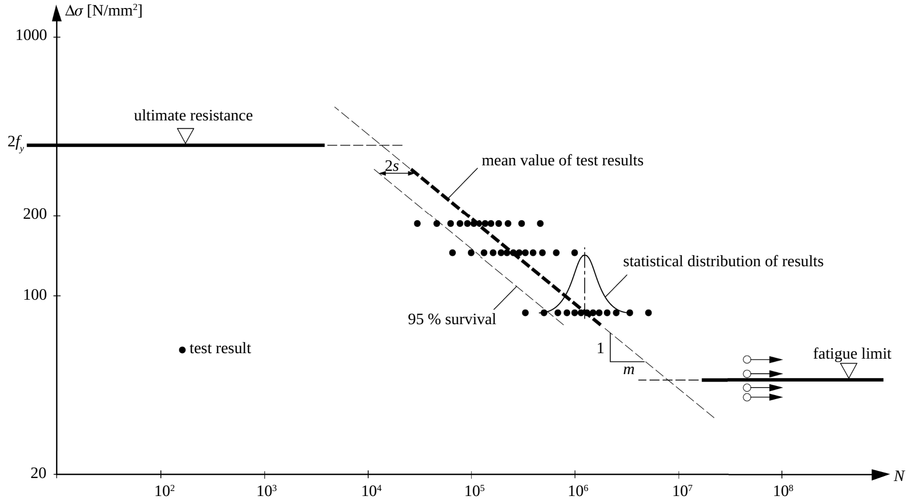


Figure 1.3 – Fatigue test results of structural steel members, plotted in double logarithm scale, carried out under constant amplitude loading (TGC 10, 2006)

The fact is that the scatter of the test results is less at high ranges and larger at low stress ranges, see for example Schijve (2001). By using a logarithmic scale for both axes, the mean value of the test results for a given structural detail can be expressed, in the range between 10^4 cycles and $5 \cdot 10^6$ to 10^7 cycles, by a straight line with the following expression:

$$N = C \cdot \Delta\sigma^{-m} \tag{1.2}$$

where

- N number of cycles of stress range $\Delta\sigma$,
- C constant representing the influence of the structural detail,
- $\Delta\sigma$ constant amplitude stress range,
- m slope coefficient of the mean test results line.

The expression represents a straight line when using logarithmic scales:

$$\log N = \log C - m \cdot \log(\Delta\sigma) \tag{1.3}$$

The expressions (1.2) and (1.3) can also be analytically deduced using fracture mechanics considerations (TGC 10, 2006).

The upper limit of the line (corresponding to high $\Delta\sigma$ values) corresponds to twice the ultimate static strength of the material (reverse cyclic loading). The region with number of cycles ranging between 10 and 10^4 is called low-cycle fatigue (or oligo-cyclic fatigue, with large cyclic plastic deformations). The corresponding low-cycle fatigue strength is only relevant in the case of loadings such as those occurring during earthquakes, or possibly silos, where usually members experience only small numbers of stress cycles of high magnitude.

The lower limit of the line (corresponding to low $\Delta\sigma$ values) represents the constant amplitude fatigue limit (CAFL, or also endurance limit). This limit indicates that cyclic loading with ranges under this limit can be applied a very large number of times (> 10^8) without resulting in a fatigue failure. It explains the wider band scatter observed near the fatigue limit, which results from specimens that do not fail after a large number of load cycles (so-called run-out, see Figure 1.3). This value is very important for all members subjected to large numbers of stress cycles of small amplitude, such as those occurring in machinery parts or from vibration effects. One shall mention that investigations for mechanical engineering applications have shown that at very high number of cycles, over 10^8 cycles, a further decline of the fatigue resistance of steels exists (Bathias and Paris 2005). Also, for aluminium, no real fatigue limit can be seen, but rather a line with a very shallow slope (with a large value of the slope coefficient *m*). It is also important to insist on the fact that a fatigue limit can only be established with tests under constant amplitude loadings. In order to derive a fatigue strength curve for design, i.e. a characteristic curve, the scatter of the test results must be taken into account. To this goal, a given survival probability limit must be set. In EN 1993-1-9, the characteristic curve is chosen to represent a one-sided 95% tolerance bound of survival probability, with a 75% confidence (e.g. a confidence interval on the mean equal to 75%). The exact position of the strength curve also depends upon the number of the available test results. This influence may be accounted for using the recommendations published by the International Institute of Welding (IIS/IIW) (IIW, 2009).

For a sufficiently large number of data points (in the order of 60 test results), this survival probability can be approximated by a straight line parallel to the mean line of the test results, but located on its left, at a two standard deviation 2*s* distance (see Figure 1.3).

1.1.4 Variable amplitude and cycle counting

Remember that the curves used to determine the fatigue strength, or S-N curves (e.g. Figure 1.3), were determined with tests under constant amplitude loadings (constant $\Delta\sigma$ stress ranges) only. However, real loading data on a structural member (for example, as a result of a truck crossing a bridge, see Figure 1.1), consist of several different stress ranges $\Delta\sigma_i$ (variable amplitude loading history). Therefore, it raises the question of how to count stress cycles and how to consider the influence of the different stress magnitudes on fatigue life. To illustrate the subject, Figure 1.4 gives an illustration of a generic variable stress history.

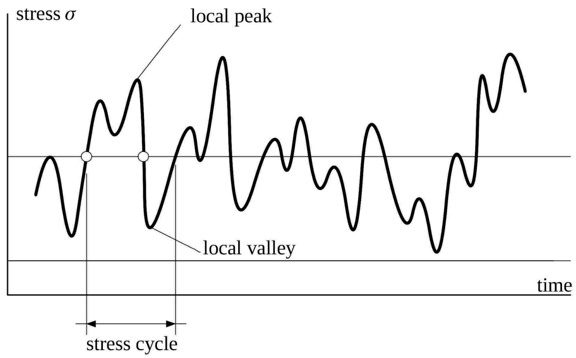


Figure 1.4 – Illustration of generic variable amplitude stress-time history (ECCS, 2000)

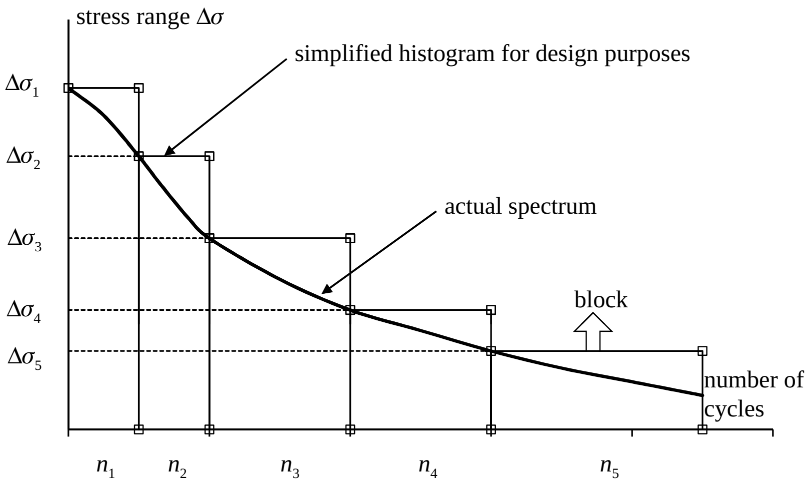


Figure 1.5 – Example of stress spectrum and corresponding histogram (ECCS, 2000)

There are various methods allowing for an analysis of the stress history: peak count methods, level crossing count methods, the rainflow count method and the reservoir count method. Among these methods, the last two shall preferably be used. These two methods, which give identical results if correctly applied, allow for a good definition of the stress ranges; this is of the highest importance since, as seen in section 1.1.2, it is the main parameter influencing fatigue life (i.e. with respect to other parameters such as the maximum or mean stress values). As a result, any stress history can be translated into a stress range spectrum. The algorithm for the rainflow counting method can be found in any reference book on fatigue, as for example Schijve (2001) and IIW (2009). An example of spectrum is shown in Figure 1.5. The spectrum can be further reduced to a histogram, any convenient number of stress intervals can be chosen, but each block of stress cycles should be assumed, conservatively, to experience the maximum stress range in that block histogram.

The rainflow counting method has found some support in considering cyclic plasticity. Also, some indirect information about sequences is retained because of the counting condition in the method, in opposition to level crossing or range counting methods (i.e. if a small load variation occurs between larger peak values, both the larger range as well as the smaller range will be considered in the Rainflow counting method) (Schijve, 2001). it is also this method that is generally suggested to give the better statistical reduction of a load time history defined by successive numbers of peaks and valleys (troughs) if compared to the level crossing and the range counting methods. Two main reasons for preferring rainflow counting (Schijve, 2001):

1) an improved handling of small intermediate ranges
2) an improved coupling of larger maxima and lower minima compared to range counts

The rainflow counting method is thus the best method, irrespective of the type of spectra (steep or flat, narrow-band or broad-band). The other counting methods give more importance to the number and the values of the extrema (peak count), or to the number of crossings of a given stress value (level crossing count). Those are not well suited for welded metallic structures in civil engineering, because of a lower correspondence with the dominating fatigue strength parameters.

As an example (Schumacher and Blanc, 1999), an extract of the stress history measured in the main girder of a road bridge is given in Figure 1.6, and the corresponding stress range histogram after rainflow analysis (corresponding to a total of 2 weeks of traffic measurements) is given in Figure 1.7. The passage of each truck on the bridge can be identified, with in addition a lot of small cycles due to the passage of light vehicles. All cycles below 1 N/mm^2 have been suppressed from the analysis; there is still after the rainflow analysis a significant number of small cycles that can be considered not relevant for fatigue damage analysis (i.e. they are below the cut-off limit, see terminology, of any detail category).

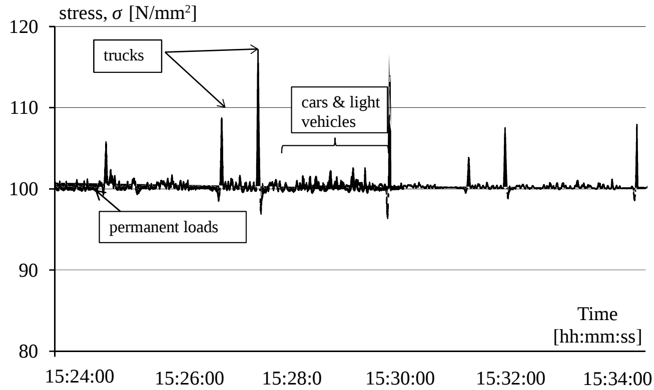


Figure 1.6 – Example of measured stress history on a road bridge (Schumacher and Blanc, 1999)

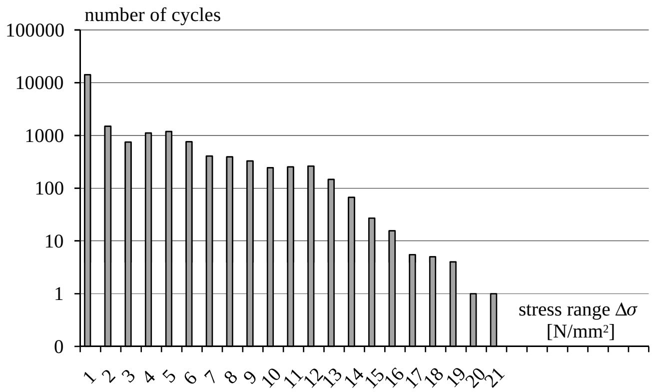


Figure 1.7 – Example of stress range histogram from two weeks measurements on a road bridge (Schumacher and Blanc, 1999)

With help of assumptions on damage accumulation, as explained in the next section, the influence of the various stress ranges on fatigue life can be interpreted with respect to the constant amplitude strength curves

(S-N curves), allowing for the calculation of the fatigue life under real, variable amplitude loading.

1.1.5 Damage accumulation

The assumption of a linear damage accumulation results in the simplest rule, the Palmgren-Miner's rule (Palmgren, 1923)(Miner, 1945), more generally known as the *Miner's rule*. This linear damage accumulation scheme assumes that, when looking at a loading with different stress ranges, each stress range $\Delta\sigma_i$, occurring n_i times, results in a partial damage which can be represented by the ratio n_i/N_i (the histogram being distributed among n_{tot} stress range classes). Here, N_i represents the number of cycles to failure (fatigue life of the structural detail under study) under the stress range $\Delta\sigma_i$. In the case the stress range distribution function is known, the summation of the partial damages due to each stress range level can be replaced by an integral function. The failure is defined with respect to the summation of the partial damages and occurs when the theoretical value D_{tot} = 1.0 is reached, see equation (1.4). This is represented in a graphical way in Figure 1.8.

$$D_{tot} = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + ... = \sum_{i=1}^{n_{tot}} \frac{n_i}{N_i} = \int \frac{dn}{N} \leq 1.0 \tag{1.4}$$

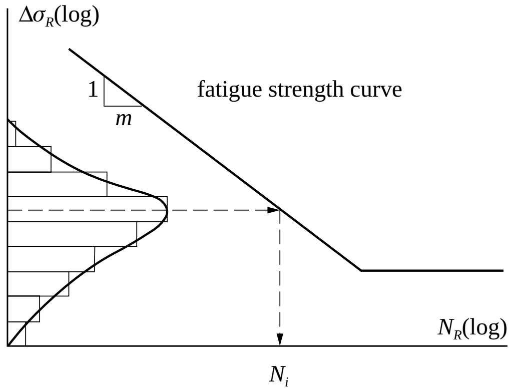


Figure 1.8 – Damage accumulation scheme

It should be noted that in this simple damage accumulation rule, the order of occurrence of the stress ranges in the history is completely ignored, it is thus a simplification. The use in design of equation (1.4) together with suitable safety factors showed itself reliable enough to be considered as the only rule for the fatigue design of welded members of bridges and cranes supporting runways. One shall however be very careful with its applicability to other structure types, especially those subjected to occasional overloads (loads significantly higher than the service loads) such as can be the case in mechanical engineering applications, offshore platforms or in airplanes (IIW, 2009), see section 5.4.4 for more information. All the same, mean stress effect need not be considered when dealing with welded members; they can however be of importance when designing or verifying members of bolted or riveted structures subjected to repeated cyclic loadings, as they can result in significantly longer fatigue lives (compared to the case of every cycle being fully effective in terms of damage as it is the case in welded members).

Stress ranges below the fatigue limit may or may not be accounted for. The first and conservative approach is to ignore the fatigue limit and to extend the straight line with the slope coefficient *m*.

The second approach takes into account the fact that the stress ranges $\Delta\sigma_i$, lower than the fatigue limit, correspond theoretically to an infinite fatigue life. However, one must be careful because this observation was made under constant amplitude fatigue tests. Applying this rule to variable amplitude loadings only holds true in the case where *all* the stress ranges in the histogram are below the fatigue limit. In this particular case, and only in this one, fatigue life tending to infinity (> 10^8 cycles) can be obtained. This is important for given members in machinery or vehicles which must sustain very large numbers of cycles. Let's now look at an histogram with some stress ranges, $\Delta\sigma_i$, above the constant amplitude fatigue limit $\Delta\sigma_D$ as well as others below (Figure 1.9).

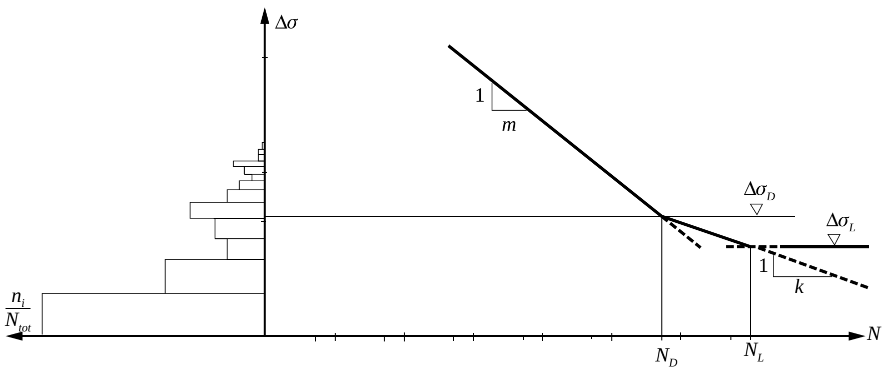


Figure 1.9 – Influence of stress ranges below the constant amplitude fatigue limit, $\Delta\sigma_D$, and the cut-off limit, $\Delta\sigma_L$ (TGC 10, 2006)

In the case the stress ranges are higher than the fatigue limit, the damage accumulation can be computed using equation (1.4). If the stress ranges are lower than the fatigue limit, they do not contribute to the propagation of the crack until the crack reaches a certain size. This is the reason why the part of the histogram below the fatigue limit cannot be completely ignored; it contributes to the accumulation of damage when the crack becomes large. To avoid having to calculate the crack growth rate using fracture mechanics for stress ranges $\Delta\sigma_i$ lower than the fatigue limit a resistance curve is used with a slope k different from Wöhler's slope m ($k = 2m - 1$ according to Haibach (1970) or $k = m + 2$, both giving for $m = 3$ the same value, $k = 5$).

In addition, in order to take into account the fact that the smallest values of stress ranges $\Delta\sigma_i$ do not contribute to crack propagation, a cut-off limit is introduced, $\Delta\sigma_L$. In many applications, including bridges, all the stress ranges lower than the cut-off limit can be neglected for the damage accumulation calculation. The cut-off limit is often fixed at 10^8 cycles, giving $\Delta\sigma_L \approx 0.55 \cdot \Delta\sigma_D$ in the case N_D is equal to $5{\cdot}10^6$ cycles (slope $k = 5$).

It is important to repeat that the part of the fatigue resistance curve (Figure 1.9) below the fatigue limit is the result of a simplification and does not directly represent a physical behaviour. This simplification was adopted in order to facilitate the calculation of the damage accumulation, using the same hypothesis as for stress ranges above the fatigue limit.

For aluminium, the fatigue strength curves follow the same principles as explained above, including the values of the number of cycles, N_D and N_L at which slope changes occur. The only exception is that different values for the Wöhler's slope m were found. These values also differ for structural detail groups. The value of the slope up to the constant amplitude fatigue limit (CAFL) can take the following values: $m = 3.4$, 4.0, 4.3 and 7.0. As for steel, for stress ranges below the CAFL, a strength curve with a slope $k = m + 2$ is used.

1.2 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 1.10, 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.

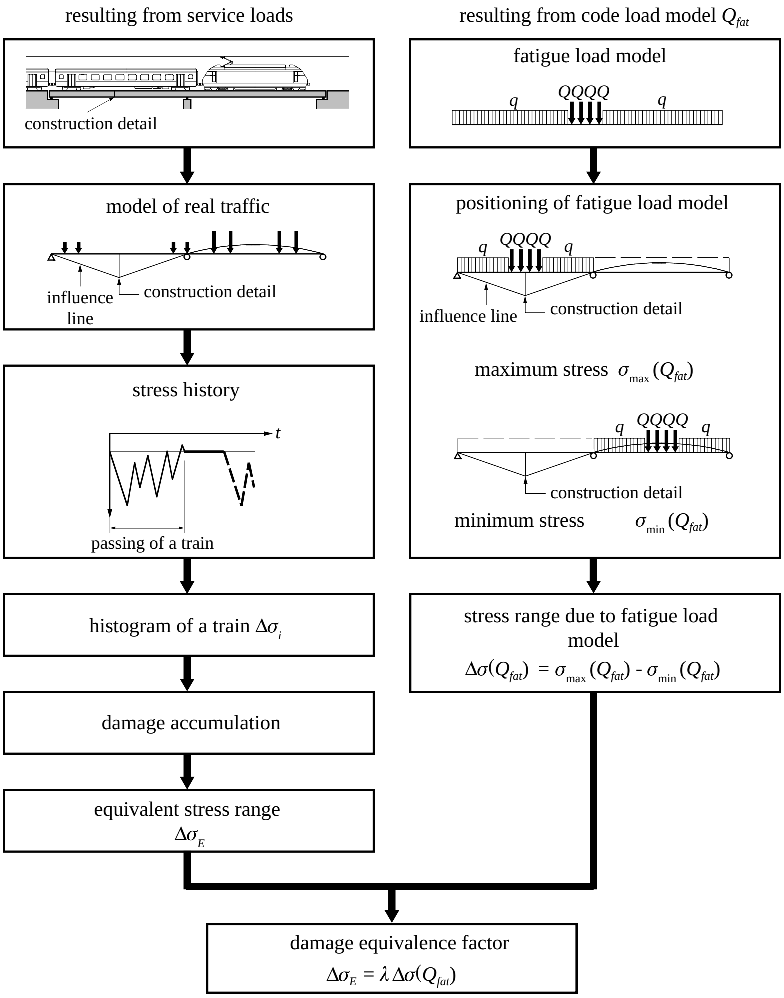


Figure 1.10 – Damage equivalent factor (Hirt, 2006)

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

3) Modelling of real traffic and displacement over the structure,
4) Deduction of the corresponding stress history (at the detail to be checked),
5) Calculation of the resulting stress range histogram $\Delta\sigma_i$,
6) Computation of the resulting equivalent stress range $\Delta\sigma_e$ (or $\Delta\sigma_{E,2}$ for the value brought back at 2 million cycles), making use of an accumulation rule, usually a linear one such as *Miner's rule*.

Finally, one can perform the verification by comparing $\Delta\sigma_{E,2}$ with the detail category or, if there is more than one detail to check, the detail categories. Note that one can also perform the verification directly by performing a damage accumulation calculation and check that the total damage remains inferior to one (in this case, the detail category must be known beforehand to perform the calculation). Detailed information on damage equivalent factors can be found in sub-chapter 3.2.

This procedure is relatively complex 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 $\sigma_{\max}$ and minimum stress $\sigma_{\min}$, by placing this load model each time in the most unfavourable position according to the influence line of the static system of the structure. But the resulting stress range $\Delta\sigma(\gamma_{Ff}\, Q_k)$, 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 range $\Delta\sigma_{E,2}$, the value $\Delta\sigma(\gamma_{Ff}\, Q_k)$ must be corrected with what is called a damage equivalent factor, λ, computed as

$$\lambda = \gamma_{Ff}\Delta\sigma_{E,2} \Big/ \Delta\sigma\left(\gamma_{Ff} Q_k\right) \tag{1.5}$$

The calculations of the correction factor values are made once for all for the usual cases, and are a 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 most important ones.

The main assumptions are the use of the "rainflow" counting method and a linear damage accumulation rule. Therefore, one may ignore 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 the curves can have more than one slope. This is the case for the set of curves in

ECCS, EN 1993-1-9, or for aluminium EN 1999-1-3. The simplified load model should not be too far from reality (average truck or train), otherwise there will be some abrupt changes in the damage equivalent factor values when the influence line length value approaches the axle spacing. The fatigue load models for different types of structures can be found in the various parts of Eurocode 1, see sub-chapter 3.1 for further details. The damage equivalent factor has been further split into several partial damage equivalent factors, see sub-chapter 3.2.

1.3 CODES OF PRACTICE

1.3.1 Introduction

In structural engineering, a great deal of research during the 1960s and 70s focussed on the effects of repetitive loading on steel structures such as bridges or towers. This work, as well as the lessons learned from the poor performance of some structures, led to a better understanding of fatigue behaviour. Still, it was a problem long overlooked in civil engineering codes, but considered in other industries (e.g. mechanical engineering, aeronautical engineering), each industry having its theory and calculation method. The work done in the 60s and 70s in turn led to the first fatigue design recommendations for steel structures and to substantial changes in provisions of steel structures design specifications. The first codes in Europe that considered fatigue were the german code (DIN 15018, 1974) and the British code (BS 5400-10, 1980). It was followed by the first European ECCS recommendations in the 80s (ECCS, 1985), which contained the first unified rules with a standardized set of S-N curves, which is still in use today.

1.3.2 Eurocodes 3 and 4

In Europe, the construction market and its services is regulated through product standards, testing codes and design codes, the whole forming an international standard family. The European standard family prepared by the European Standardization body, i.e. “Comité Européen de Normalisation” (CEN), 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) and European Technical

Approval Guidelines (ETAG), all prepared by the European Organisation for Technical Approvals (EOTA). For steel structures, the relevant parts of the European international standard family are shown on Figure 1.11.

Apart from the general rules, Eurocode 3 contains "Application rules" like part 2 "Steel bridges" or part 6 "Crane supporting structures" on special ranges of application.

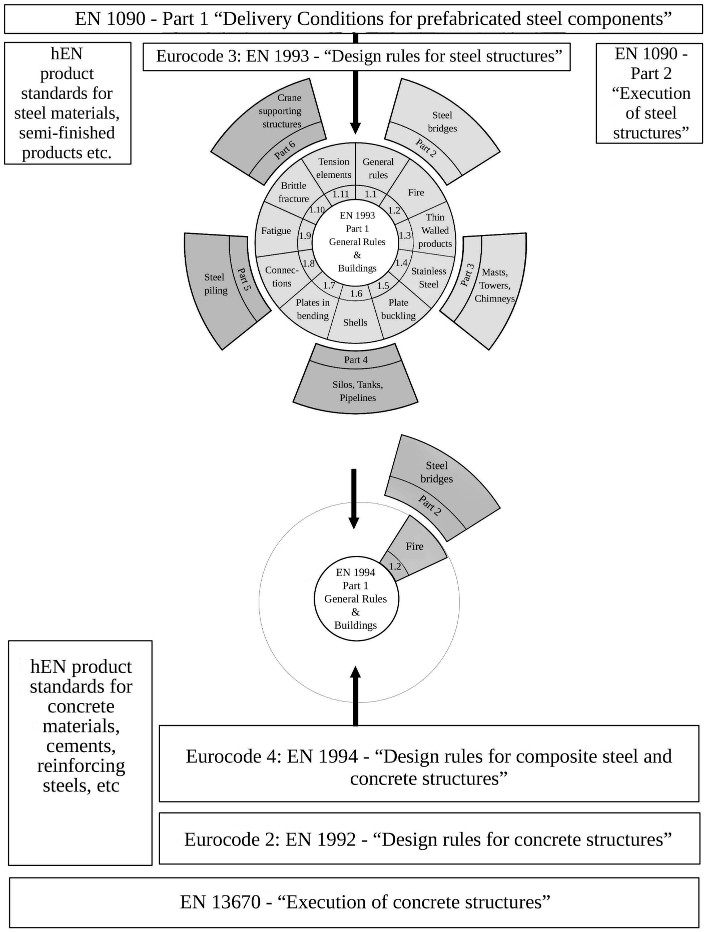


Figure 1.11 – Standard system for steel structures (Schmackpfeffer *et al*, 2005) and composite steel and concrete structures

Altogether, the Eurocode 3 "Design rules for steel structures" consists of 20 parts and Eurocode 4 "Composite construction" of 3 relevant parts. The core forms the so-called basic standard EN 1993-1 "for bases and above ground construction ", which consists again of 11 parts 1-1 to 1-11, to which another part, 1-12 for high strength steel grades (S500 to S700) was added.

In the design standards containing the general rules, two parts are related to fatigue. These are part 1-10: material toughness and through-thickness properties (material quality selection) (EN 1993-1-10:2005), and part 1-9: fatigue (EN 1993-1-9:2005).

Furthermore, the following parts of Eurocode 3 (for definitions of abbreviations see Table 1.1) contain sections on fatigue design of structures which may have to be designed against fatigue:

- Part 1: Steel structures, general rules and rules for buildings EN 1993-1-1)
- Part 2: Steel bridges (EN 1993-2)
- Part 3: Towers and masts and chimneys (EN 1993-3)
- Part 4: Silos and tanks (EN 1993-4)
- Part 6: Crane supporting structures (EN 1993-6).

The same organization holds true for Eurocode 4, for steel and concrete composite structures. Historically, during the revision of ENV-versions into prEN (and thereafter into final versions of EN), the CEN TC250 committee agreed to carry out a reorganization of the rules, including the rules related to fatigue, into the generic and associated Eurocodes. In terms of fatigue in steel and steel and concrete composite structures, it has been agreed that all rules for fatigue were to be compiled and summarized in a new Part 1-9 (in the old versions ENV, they were still in the individual standards) (see Table 1.1). In this new generic part EN 1993-1-9 "fatigue", the rules applicable for the fatigue design of all structures with steel members are regrouped. This part regroups essentially the various so-called "chapter 9" of the former versions of the ENV 1993-1 to ENV 1993-7 parts, see also Sedlacek *et al* (2000). This reorganisation avoids repetition and, in particular, reduces the risk of contradictions between different Eurocode parts. However, not all elements of the fatigue verification are integrated in the new part 1-9. The action effects which are independent from the fatigue resistance are regulated in the EN 1991 parts. Moreover, for some structures, e.g. bridges, towers, masts, chimneys, etc., specific fatigue features remain in the appropriate application parts of EN 1993.

The recommendations in EN 1991-2 on fatigue load models and in EN 1993-1-9 allow for a simplified fatigue verification using fatigue strength (Wöhler, 1860) curves. In addition, a detailed computation with application of the damage accumulation is also possible, allowing for the evaluation of the residual life, for example. In practice, the simplified verification is more user-friendly and more efficient for a daily use.

The limit state of fatigue is characterized by crack propagation followed by a final failure of the structural member. To verify this limit state, a verification of the failure, to avoid brittle fracture of the structural members, is required. The brittle failure is influenced by the material toughness, temperature and thickness. The topic of brittle fracture of steel and the proper choice of material to avoid it, is covered by Eurocode 3, Part 1-10 (EN 1993-1-10) and is presented thoroughly in chapter 6.

1.3.3 Eurocode 9

For aluminium structures, the design codes are only a few in comparison to the steel ones. Eurocode 9 addresses the design of new structures made out of wrought aluminium alloys and gives limited guidance for cast alloys. Eurocode 9 is separated into 5 parts:

– EN 1999-1-1: general structural rules
– EN 1999-1-2: structural fire design
– EN 1999-1-3 : structures susceptible to fatigue
– EN 1999-1-4 : cold-formed structural sheeting
– and EN 1999-1-5: shell structures.

The only part of interest in this book is EN 1999-1-3.

Table 1.1 – Overview and changes in the transition from ENV to EN versions of the various Eurocode 3 and Eurocode 4 parts

ENV-Version	EN-Version	Content
ENV1993-1-1: 1992	EN 1993-1-1: 2005*	General rules and rules for buildings
ENV 1993-1-2: 1995	EN 1993-1-2: 2005*	General rules - Structural fire design
ENV 1993-1-3: 1996	EN 1993-1-3: 2006*	General rules - Supplementary rules for cold-formed members and sheeting

Table 1.1 – Overview and changes in the transition from ENV to EN versions of the various Eurocode 3 and Eurocode 4 parts (continuation)

ENV-Version	EN-Version	Content
ENV 1993-1-4: 1996	EN 1993-1-4: 2006	General rules - Supplementary rules for stainless steels
ENV 1993-1-5: 1997	EN 1993-1-5: 2006*	General rules - Plated structural elements
ENV 1993-1-1: 1992	EN 1993-1-6: 2007*	Strength and stability of shell structures
ENV 1993-1-1: 1992	EN 1993-1-7: 2007*	Strength and stability of planar plated structures subject to out of plane loading
ENV 1993-1-1: 1992	EN 1993-1-8: 2005*	Design of joints
ENV1993-1-1:1992, Chap.9 ENV 1993-2: 1997, Chap.9 ENV 1993-3-1: 1997 ENV 1993-3-2: 1997 ENV 1993-6: 1999	EN 1993-1-9: 2005*	Fatigue
ENV 1993-1-1: 1992, Appendix C ENV 1993-2: 1997, Appendix C	EN 1993-1-10: 2005*	Material toughness and through-thickness properties
ENV1993-2: 1997, Appendix C	EN 1993-1-11: 2006*	Design of structures with tension components
-	EN 1993-1-12: 2007*	General - High strength steels
ENV 1993-2: 1997	EN 1993-2: 2006*	Steel bridges
ENV 1993-3-1: 1997 (prEN 1993-7-1:2003)	EN 1993-3-1: 2006*	Towers, masts and chimneys – Towers and masts
ENV 1993-3-2:1997 (prEN 1993-7-1: 2003)	EN 1993-3-2: 2006	Towers, masts and chimneys – Chimneys

Table 1.1 – Overview and changes in the transition from ENV to EN versions of the various Eurocode 3 and Eurocode 4 parts (continuation)

ENV-Version	EN-Version	Content
ENV 1993-4-1: 1999	EN 1993-4-1: 2007*	Silos
ENV 1993-4-2: 1999	EN 1993-4-2: 2007*	Tanks
-	EN 1993-4-3: 2007*	Pipelines
ENV 1993-5: 1998	EN 1993-5: 2007*	Piling
ENV 1993-6: 1999	EN 1993-6: 2007*	Crane supporting structures
ENV 1994-1-1: 1992	EN 1994-1-1: 2004	General rules and rules for buildings
ENV 1994-1-2: 1994	EN 1994-1-2: 2005*	Structural fire design
ENV 1994-2: 1997	EN 1994-2: 2005*	General rules and rules for bridges

*corrigenda has been issued for this part.

1.3.4 Execution (EN 1090-2)

The Euronorm EN 1090 fixes the requirements for the execution of steel and aluminium structures, in particular, structures designed according to any of the EN 1993 generic parts and associated Eurocodes, members in steel and concrete composite structures designed according to any of the EN 1994 parts and aluminium structures designed according to EN 1999 parts. EN 1090 is divided in three parts, namely:

- EN 1090-1: Execution of steel structures and aluminium structures – Part 1: general delivery conditions.
- EN 1090-2: Execution of steel structures and aluminium structures – Part 2: Technical requirements for the national execution of steel structures.
- EN 1090-3: Execution of steel structures and aluminium structures – Part 3: Technical requirements for aluminium structures.

The implementation and use of EN 1090 rules is closely linked with the implementation of the structural Eurocodes. The withdrawal of national standards codes in CEN member countries and their replacement by the Eurocodes is now completed in most countries.

EN 1090 specifies requirements independently from the type, shape and loading of the structure (e.g. buildings, bridges, plated or latticed elements). It includes structures subjected to fatigue or seismic actions. It specifies the requirements related to four different execution classes, namely EXC1, EXC2, EXC3 and EXC4 (from the less to the more demanding). It is important to note that this classification can apply to the whole structure, to part(s) of it or to specific joints only. Thus, the execution of a building or of any structure would not be, apart from a few exceptions, specified "of execution class 4" as a whole. The particularly severe requirements of this class apply only to certain members, even to only a few essential joints (Gourmelon, 2007).

In order not to leave the design engineer and its client without any clue to answer this question and to avoid the classic, but uneconomical reflex of choosing the most demanding class, guidance for the choice of execution class was elaborated. The principles of the choice are based on three criteria:

– *Consequence classes.* EN 1990: 2002 gives in its Annex B guidelines for the choice of consequence class for the purpose of reliability differentiation. The classification criterion is the importance of the structure or the member under consideration, in terms of its failure consequences. Consequence classes for structural members are divided in three levels, see Table 1.2. The three reliability classes RC1, RC2, and RC3 with their corresponding reliability indexes as given in EN 1990, Annex A1, may be associated with the three consequence classes (Simões da Silva *et al*, 2010).
– *Service categories*, arising from the actions to which the structure and its parts are likely to be exposed to during erection and use (dynamic loads, fatigue, seismic risk, …) and the stress levels in the structural members in relation to their resistance. There are two different possible service categories, see Table 1.3,
– *Production categories*, arising from the complexity of the execution of the structure and its structural members (e.g. complex connections, high strength steels, heavy plates or particular techniques). There are two different possible production categories, see Table 1.4.

Table 1.2 – Definition of consequence classes (adapted from EN 1990, Table B1)

Cons. Class	Description	Examples of buildings and civil engineering works
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings, silos less than 100 t capacity, greenhouses)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. office buildings)
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. concert halls, discretely supported silos more than 1000 t capacity)

The execution classes are then chosen according to the consequence classes, service and production categories determined for the considered members. They can be chosen on the basis of the indications of Table 1.5. Note that in the absence of specification in the contract, execution class 2 applies by default (Gourmelon, 2007).

Table 1.3 – Suggested criteria for service categories (from EN 1090-2, Table B.1)

Cat.	Criteria
SC1	Structures/components designed for quasi static actions only (e.g. buildings) Structures and components with their connections designed for seismic actions in regions with low seismic activity, of low class of ductility (EN 1998-1) Structures/components designed for fatigue actions from cranes (class S_0)*
SC2	Structures/components designed for fatigue actions according to EN 1993 (e.g. road and railway bridges, cranes (classes S_1 to S_9)*) Structures susceptible to vibrations induced by wind, crowd or rotating machinery Structures/components with their connections designed for seismic actions in regions with medium or high seismic activity, of medium or high classes of ductility (EN 1998-1)
* For classification of fatigue actions from cranes, see EN 1991-3 and EN 13001-1	

Table 1.4 – Suggested criteria for production categories (from EN 1090-2, Table B.2)

Cat.	Criteria
PC1	Non welded components manufactured from any steel grade products Welded components manufactured from steel grade products below S355
PC2	Welded components manufactured from steel grade products from S355 and above Components essential for structural integrity that are assembled by welding on construction site Components with hot forming manufacturing or receiving thermic treatment during manufacturing Components of Circular Hollow Sections (CHS) lattice girders requiring end profile cuts

Table 1.5 – Recommendation for the determination of the execution classes (from EN 1090-2, Table B.3)

Consequence classes		CC1		CC2		CC3	
Service categories		SC1	SC2	SC1	SC2	SC1	SC2
Production categories	PC1	EXC1	EXC2	EXC2	EXC3	EXC3*	EXC3*
	PC2	EXC2	EXC2	EXC2	EXC3	EXC3*	EXC4
* EXC4 should be applied to special structures or structures with extreme consequences of a structural failure as required by national provisions							

With emphasis on fatigue behaviour, the execution of welding is of particular importance. Any fault in workmanship may potentially reduce the fatigue strength of a detail. Good workmanship, on the contrary, will result in an increase in the fatigue strength, often above the characteristic S-N curves given in the codes. Indeed, these curves correspond to lower bound test results obtained from average fabrication quality details. Even though good workmanship cannot be quantified in the Eurocodes and used in fatigue verifications, S-N curves referring for most details to failure from undetectable flaws, it can be considered as a welcomed supplementary safety margin.

The good workmanship criteria, however, on which the weld quality specifications in codes and standards are based, are sometimes not directly related to the effect and importance of the feature specified on fatigue strength (or any other strength criteria). Faults in workmanship proven to be detrimental to fatigue include the following (from most to less detrimental, however depending upon original fatigue strength of detail and fault level):

- unauthorised attachments,
- weld lack of fusion/penetration, particularly in transverse butt welds,
- poor fit-up, assembly tolerances, eccentricity and misalignment,
- notches, sharp edges,
- distortion,
- corrosion pitting,
- weld spatter,
- accidental arc strikes.

Speaking again about normative execution requirement, the concern of the design engineer is to choose the class of imperfections tolerated with regards to the reference code EN ISO 5817 (ISO 5817, 2006). In the Eurocode framework and EN 1090-2, the engineer will have, as for the other execution questions, to define the required execution class only. In EN 1090-2, the following requirements are fixed:

- Execution class 1 (EXC1): Quality level D
- Execution class 2 (EXC2): Quality level C
- Execution class 3 (EXC3): Quality level B
- Execution class 4 (EXC4): Quality level B with additional requirements to account for fatigue effects.

For structures or parts of structures which have to be designed against fatigue, quality level D is excluded, quality level C may be used for specific details and quality B is the usual choice. It should be noted that a fatigue detail category 90 seems to be compatible with most imperfections of quality level B according to ISO 5817, with however the exception of the following, where more stringent requirements should be set (Hobbacher *et al*, 2010):

- continuous undercut
- single pore, pore net, clustered porosity
- slag inclusions, metallic inclusions
- linear misalignment of circumferential welds
- angular misalignment (which is not in ISO 5817 at this time)
- multiple imperfections in longitudinal direction of weld.

A project for revising ISO 5817, in particular with respect to fatigue criteria, is under discussion. The allowable imperfections requirements go along with requirements on the company quality system, welding coordination, etc. A summary of the main welding requirements is given in Table 1.6. It can be seen that there is no requirements for EXC1, which once again shows it is not adequate for structures under fatigue loadings.

Table 1.6 – Main weld requirements, extracts from EN 1090-2

	EXC1	EXC2	EXC3	EXC4
Qualif. of welding procedures; welding coordination	Not required.	Required, see EN 1090-2, § 7.4	Required, see EN 1090-2, § 7.4	Required, see EN 1090-2, § 7.4
Temporary attachments	Not req.	Not req.	Use to be specified Cutting and chipping not permitted	
Tack welds	Not req.	Qualified welding procedure		
Butt welds	Not req.	Run on/off pieces if specified	Run on/off pieces For single side welds, permanent backing continuous	
Execution of welding	Not req.	Not req.	Removal of spatter	
Acceptance criteria	EN ISO 5817 Quality level D if specified	EN ISO 5817 Quality level C generally	EN ISO 5817 Quality level B	EN ISO 5817 Quality level B+

The additional requirements for quality B+ are given in EN 1090-2 Table 17. In summary, this table gives additional or more severe limits for imperfections such as undercut (not permitted in B+), internal pores, linear misalignment, etc. It also gives supplementary requirements for bridge decks.

Outside of the welding requirements, it is very important to meet every special requirement in order not to impair fatigue strength. Thus, it should be emphasised that all connections provided for temporary structural members or for fabrication purposes shall also meet the requirements of EN 1090. Also, regarding member identification, a suitable system shall be put into place in order to be able to follow each piece; note that the suitable marking method is function of the material. Furthermore, the marking methods shall be applied in a way not producing damage and only on areas where it does not affect the fatigue life.

Finally, note that in addition to the rules found in EN 1090, some additional information and requirements regarding execution can also be found directly in the detail category tables of EN 1993-1-9, as well as in the other EN 1993 and EN 1994 relevant parts for the different types of structures. For example, in Table 8.5 of EN 1993-1-9, detail 1 to 3, cruciform and tee joints, the following requirement is given: *the misalignment of the load-carrying plates should not exceed 15 % of the thickness of the intermediate plate*.

1.3.5 Other execution standards

With emphasis on fatigue behaviour, the standards related to welding, bolting and erection are the most important ones. The complete list of standards in these areas, about 200 "normative References", is given in EN 1090. The largest group are the standards for products, for which there are around one hundred. Then, there are about thirty standards related to welding, about fifteen dealing with destructive or non-destructive testing applicable to the welds, as well as about twenty standards in relation with corrosion protection (Gourmelon, 2007). To mention only a few, of relevance to the subject presented in this book:

- EN ISO 3834: 2005 (in 6 parts), Quality requirements for fusion welding of metallic materials. This standard defines requirements in the field of welding so that contracting parties or regulators do not have to do it themselves. A reference to a particular part of EN ISO 3834 should be sufficient to demonstrate the capabilities of the manufacturer to control welding activities for the type of work being done. The different parts of the standards deal with the following items: contract and design review, subcontracting, welding personnel, inspection, testing and examination personnel, equipment, storage of parent materials, calibration, and identification/traceability.
- EN ISO 5817: 2003 (corrected version 2005), Welding – fusion-welded joints in steel, nickel, titanium and their alloys – quality levels for imperfections. This standard defines the dimensions of typical imperfections, which might be expected in normal fabrication. It may be used within a quality system for the production of factory-welded joints. It provides three sets of dimensional values (quality levels B, C, and D) from which a selection can be made for a particular application. This standard is directly applicable to visual testing of

welds and does not include details of recommended methods of detection or sizing by non destructive means. It does not cover metallurgical aspects, such as grain size or hardness.

- EN ISO 9013: 2002, Thermal cutting – Classification of thermal cuts – Geometrical product specification and quality tolerances. This standard applies to materials and thickness ranges suitable for oxyfuel flame cutting, plasma cutting and laser cutting.
- EN 12062: 1997, Non-destructive testing of welds – General rules for metallic materials. The purpose of this standard is quality control. It gives guidance for the choice and evaluation of the results of non-destructive testing methods based on quality requirements, material, weld thickness, welding process and extent of testing. This standard also specifies general rules and standards to be applied to the different types of testing (visual inspection, dye-penetrant flaw detection, eddy-current tests, magnetic-particle flaw detection, radiographic testing, and ultrasonic testing), for either the methodology or the acceptance level for metallic materials.
- ISO 15607: 2003: Specification and qualification of welding procedures for metallic materials - General rules. This standard gives the general rules and requirements concerning the qualification of welding procedures, which are further developed in EN 15611, EN 15612, EN 15613, and EN 15614.

1.4 DESCRIPTION OF THE STRUCTURES USED IN THE WORKED EXAMPLES

1.4.1 Introduction

In order to fulfil the objectives of the design manuals of this ECCS collection, three different structures were chosen to be used for the detailed design examples presented in this book. Before being used for fatigue calculations, they are introduced and briefly described in the following paragraphs. The first structure is a steel and concrete composite bridge which is also used in the ECCS design manual about EN 1993 part 1-5 (plate buckling) (Beg *et al*, 2010). The second structure considered is a chimney and the third one is a crane supporting structure.

1.4.2 Steel and concrete composite road bridge (worked example 1)

1.4.2.1 Longitudinal elevation and transverse cross section

This worked example is adapted from the COMBRI research project (COMBRI, 2007) (COMBRI+, 2008). The bridge is a symmetrical composite box-girder structure with five spans, 90 m + 3 × 120 m + 90 m (i.e. a total length between abutments equal to 540 m, see Figure 1.12). It is assumed to be located in Yvelines, near Paris, France. For simplification reasons, the horizontal alignment is assumed straight as well as the road, the top face of the deck is horizontal and the structural steel depth is constant and equal to 4000 mm.

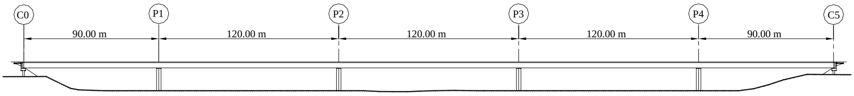


Figure 1.12 – Side view of road bridge with span distribution

A four-lane traffic road crosses the bridge. Each lane is 3.50 m wide and the two outside ones are bordered by a 2.06 m wide safety lane. Normalised safety barriers are located outside the traffic lanes and in the centre of the road (see Figure 1.13). The cross section of the concrete slab and non-structural equipments is symmetrical with respect to the axis of the bridge. The 21.50 m wide slab has been modelled with a constant thickness equal to 0.325 m. The slab span between the main girders is equal to 12.00 m and the slab cantilever is 4.75 m on both sides.

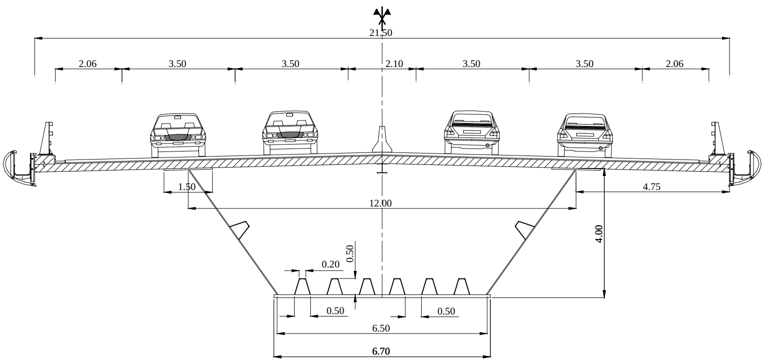


Figure 1.13 – Cross section of road bridge with lane positions

The concrete slab is connected to an open box-section. The centre-to-centre distance between webs in the upper part is equal to 12.00 m, and to

6.50 m in the lower part. The upper flanges are 1500 mm wide whereas the lower flange is 6700 mm wide.

1.4.2.2 Materials and structural steel distribution

A steel grade S355 has been used. Its mechanical properties are given in EN 10025-3 and slightly modified by EN 1993-2 (see Table 1.7). Normal concrete of class C35/45 is used for the reinforced concrete slab; the reinforcing steel bars are class B high bond bars with a yield strength of 500 MPa and a modulus of elasticity equal to 210000 MPa (as structural steel).

Table 1.7 – f_y and f_u according to the plate thickness for steel grade S355

t (mm)	f_y (MPa)	f_u (MPa)	Quality
$t < 16$	355	470	K2
$16 \leq t < 30$	345	470	N
$30 \leq t < 40$	345	470	NL
$40 \leq t < 63$	335	470	NL
$63 \leq t < 80$	325	470	NL
$80 \leq t < 100$	315	470	NL
$100 \leq t$	295	450	NL

The structural steel distribution results from a design according to Eurocodes 1, 3 and 4, see Figure 1.15.

The thickness variations of the upper and lower flanges are found towards the inside of the girder. Due to the concrete slab width, an additional plate welded to each upper flange (1400 mm wide and welded below the main one) needs to be added in the regions of intermediate supports.

Cross frames stiffen the box-section on abutments and on intermediate supports, as well as every 4.0 m in the spans, see Figure 1.14. The bottom flange longitudinal trapezoidal stiffeners are continuous with a plate thickness equal to 15 mm, see Figure 1.16. The web longitudinal stiffeners are discontinuous; these have the same thickness throughout and are located at mid-depth to provide sufficient cross section shear resistance. An additional longitudinal steel rolled I-girder (located right in the middle of the bridge cross section) spans between the transverse frames and is directly connected to the concrete slab. It helps for the slab concreting phases and resists with the composite cross section (as an additional section for the upper steel flanges).

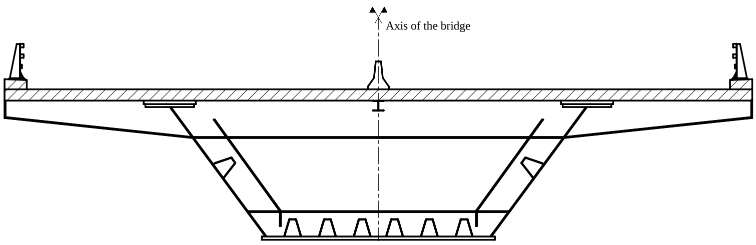
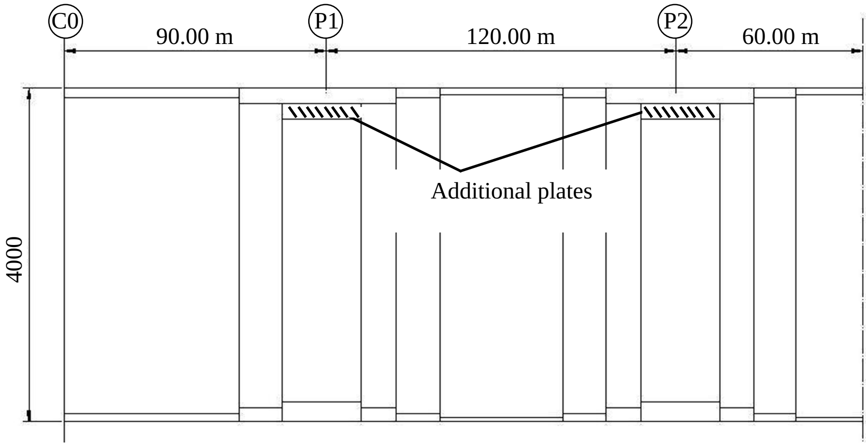


Figure 1.14 – Cross frame on supports



Upper flange	60000 × 70	15000 × 100	15000 × 100 + 12000 × 90	12000 × 100	15000 × 70	42000 × 50	15000 × 70	12000 × 100	12000 × 100 + 12000 × 90	12000 × 70	15000 × 70	21000 × 50
Web	20 × 60000	24 × 15000	27 × 27000	24 × 12000	18 × 15000	18 × 42000	18 × 15000	24 × 12000	27 × 24000	18 × 12000	18 × 15000	18 × 21000
Lower flange	60000 × 35	15000 × 50	27000 × 70	12000 × 50	15000 × 35	42000 × 25	15000 × 35	12000 × 50	24000 × 70	12000 × 50	15000 × 35	21000 × 25

Figure 1.15 – Structural steel distribution (half length of the road bridge)

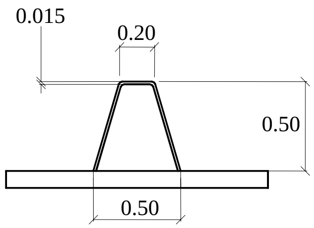


Figure 1.16 – Detailed view of longitudinal stiffener

1.4.2.3 The construction stages

The assumptions pertaining to the construction stages should be taken into account when calculating the internal moments and forces distribution in the bridge deck (EN1994-2, 5.4.2.4) as they have a high influence on the steel/concrete modular ratios. For the bridge example, the following construction stages have been adopted:

- Launching of the structural steel structure
- On-site pouring of the concrete slab segments by casting them in a selected order (first the in-span segments, and second the segments around internal supports):

  The total length of the bridge (540 m) has been broken down into 45 identical 12-m-long concreting segments. The start of pouring the first slab segment is the time of origin (t = 0). The time taken to pour each slab segment is assessed at 3 working days. The first day is devoted to the concreting, the second day to its hardening and the third to move the mobile formwork. The slab is thus completed within 135 days.
- The installation of non-structural equipments is assumed to be completed within 35 days, so that the deck is fully constructed at the date t = 135 + 35 = 170 days.

1.4.3 Chimney (worked example 2)

1.4.3.1 Introduction

The following worked example is based on a real chimney verification made by Kammel (2003) and the ECCS Technical Committee 6. The original example was published in Stahlbaukalendar (2006) and has since been adapted by the authors since the original verification was carried out to determine the cause of observed cracks. Indeed, the existing chimney had fatigue problems due to vortex shedding induced vibrations. The example presented includes a tuned mass damper in order to solve this problem.

Vortex shedding often occurs in cantilevered steel chimneys that are subjected to dynamic wind loads. At a critical wind speed, alternating vortices detach from the cylindrical shell over a specific correlation length causing a vibration of the structure transverse to the wind direction, see section 3.1.5 for further explanations. The cyclic loading is transferred to all structural members and connections. In this type of structure, the following structural details are usually relevant for fatigue verification:

- Bolted flange connection between two sections,
- Welded stiffeners at the bottom,
- Anchor bolts at the bottom
- Inspection manholes and/or inlet tubes details.

The chimney dealt with in this example has a height of 55 m and an outside constant diameter of 1.63 m, as shown in Figure 1.17. It is a double-walled chimney with an outer tube and inner thermal insulating layer. The chimney shaft is composed of 5 separate parts, bolted together using socket joints, see Figure 1.20. At the bottom, a reinforced ring is used and the chimney is held down using 28 anchor bolts, see Figure 1.18, Figure 1.19 and Figure 1.21. Furthermore, an inspection manhole is present in the bottom zone, see Figure 1.19. All dimensions and other information are given in the next paragraphs.

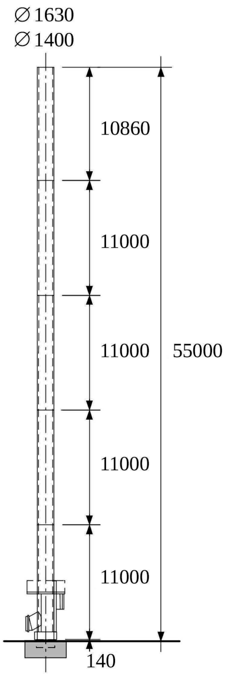


Figure 1.17 – Side view of the example chimney

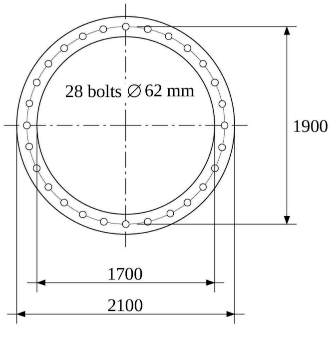


Figure 1.18 – Anchor bolts at +0.350 m (plan view)

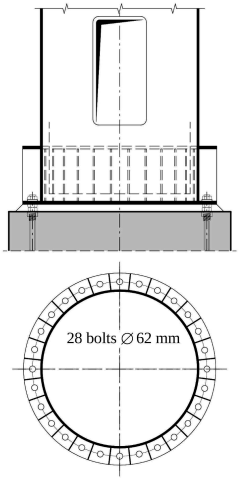


Figure 1.19 – Drawing of bottom part of chimney with manhole position, section and top view, ground plate with anchor bolts at +0.350 m

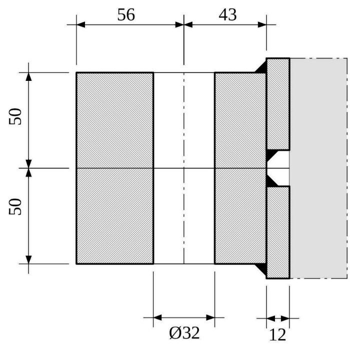


Figure 1.20 – Relevant bolted flange connection between two sections at +11.490 m (remark: this flange design corresponds to standard practice and does not represent optimum design; an improved design is possible)

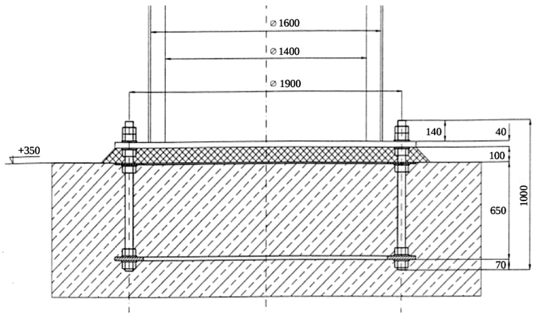


Figure 1.21 – Ground plate with anchor bolts at +0.350 m (section view)

1.4.3.2 General characteristics of the chimney

Height	: h = 55.0 m
Outer diameter	: b = 1630 mm
Slenderness ratio	: $\lambda = h/b = 33.7$
Shell thickness from bottom up to +11.490m	: s = 12 mm
Shell thickness at top	: s' = 6 mm
Steel yield stress	: f_y = 190 N/mm²

(which corresponds to steel S235 operating at a max. temperature T = 100 °C)

In the choice of material quality, since tensile stresses occur perpendicular to the ring surface at the joint between ring and shell (socket joint, see Figure 1.20), attention has to be paid to avoid lamellar tearing and the designer must follow the rules given in EN 1993-1-10.

The equivalent total mass per unit length is taken as m_e = 340 kg/m (EN 1991-1-4, Section F.4: for cantilevered structures with a varying mass distribution m_e may be approximated by the average value of m over the upper third of the structure).

For damping characteristics, the logarithm decrement is taken as (EN 1991-1-4, Section F.5)

$$\delta = \delta_s + \delta_a + \delta_d = 0.03$$

This value includes damping from a tuned mass damper. For the structural part only, for a welded chimney without external thermal insulation, the value given in EN 1991-1-4 is lower, $\delta_s = 0.012$. A tuned mass damper is a type of dynamic vibration absorber which must be specifically analysed, tested and tuned on the structure and periodically inspected (EN 1993-3-2, annex B). Examples of chimneys with vibration problems and their resolution are periodically published, for example (Kawecki *et al*, 2007).

1.4.3.3 Dimensions of socket joint located at +11.490 m (see Figure 1.20)

Bolt diameter (M30; 10.9)	: D_{M30} = 30 mm
Bolt cross section	: $A_{s,30}$ = 561 mm²

Bolt resistance	: f_{ub} = 1000 N/mm²
Bolt preload	: $F_{p,Cd}$ = 350 kN
Total number of bolts	: n = 42
Distance of bolts and shell	: a = 43 mm
Distance between bolts	: $e = \frac{\pi}{n} \cdot (b + 2 \cdot a) = 128.4\,\text{mm}$
Washer dimensions	: t_{was} = 5 mm
	d_a = 56 mm
	d_i = 31 mm
Socket flange cross section	: t_f = 50 mm
	w = 99 mm
	$b' = w - a$ = 56 mm

Section of the chimney (without socket):

$$A_K = \frac{\pi}{4} \cdot \left(b^2 - (b - 2 \cdot s)^2\right) = 61000\,\text{mm}^2$$

Elastic section modulus $: W_y = \frac{\pi}{32} \cdot \frac{b^4 - (b - s \cdot 2)^4}{b} = 24493 \cdot 10^3\,\text{mm}^3$

1.4.3.4 Dimensions of ground plate joint with welded stiffeners located at the bottom, at +0.350 m:

Anchor bolt diameter (M60; 8.8)	: D_{M60} = 60 mm
Anchor bolt cross section	: $A_{s,60}$ = 2362 mm²
Anchor bolt resistance	: f_{ub} = 800 N/mm²
Total number of anchor bolts	: n = 28 (same as number of stiffeners)
Distance of bolts and shell	: a = 135 mm
Radius of anchor bolt circle	: r_s = 1900 / 2 = 950 mm
Distance between bolts	: $e = \frac{\pi}{n} \cdot (b + 2 \cdot a) = 213.2\,\text{mm}$
Longitudinal stiffener	: 646 × 223 × 10 mm
Upper ring stiffener	: 250 × 25 mm

Double-sided fillet welding between stiffener and ground plate: a_w = 6 mm

If flux-cored welding is used, the effective weld throat is larger than a_w , and this value can be used in the verifications. We will assume here that $a_{w,eff} = a_w + 1\,\text{mm}$.

	L_w = 220 mm
Ground plate dimensions	: d_{bp} = 2100 mm
	t_{bp} = 40 mm

1.4.3.5 Dimensions of manhole located betweeen +1.000 m and +2.200 m:

Height	: h_{mh} = 1200 mm
Width	: w_{mh} = 600 mm
Corner radius	: r_{mh} = 300 mm
Opening sides reinforcement plates	: b_{rp}= 90 mm
	t_{rp} = 10 mm
	h_{rp} = 1400 mm

Section of the chimney (with manhole):

$$A_{K,mh} = \frac{\pi}{4} \cdot \left(b^2 - \left(b - 2 \cdot s \right)^2 \right) - w_{mh} \cdot s + 2 \cdot t_{rp} \cdot b_{rp} = 55600\,\text{mm}^2$$

Assumed elastic section modulus at manhole level:

$$W_{y,mh} = 20000 \cdot 10^3\,\text{mm}^3$$

1.4.4 Crane supporting structures (worked example 3)

1.4.4.1 Introduction

Figure 1.22 presents the general geometry of the single crane supporting structure example. This example is adapted from one presented in TGC 11 (2006). The crane supporting structure is composed of a continuous runway beam – HEA 280 in S 355 steel - supported by surge girders, with spans between supports l = 6 m. It is assumed that the end-spans are shorter, thus the relevant span is an inner span. The crane span is s =14.30 m and the nominal hoist load is Q_{nom} = 100 kN.

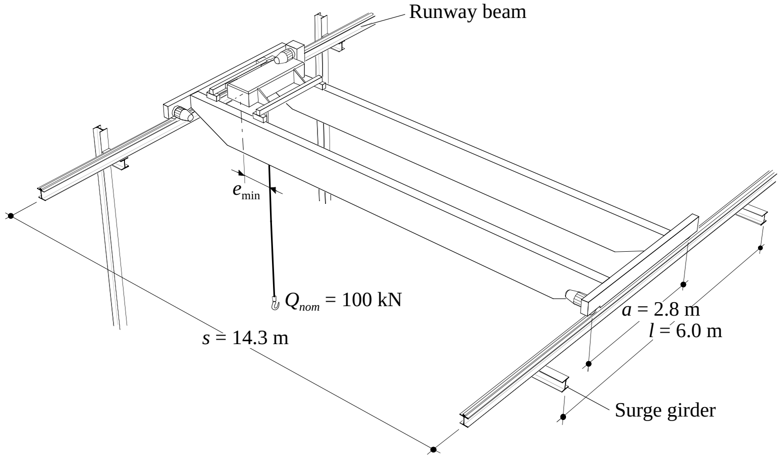


Figure 1.22 – Crane supporting structure

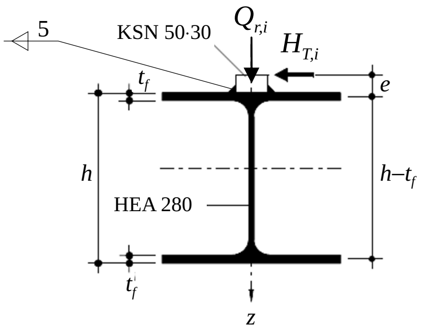


Figure 1.23 – Cross section of the runway beam

The single crane supporting structure is classified according to section 2.12, of EN 1991-3 (the classification table can also be found in EN 13001-1) as a function of the total number of lifting cycles and the load spectrum as follows:

- Class of load spectrum: Q4
- Class of total number of cycles: U4

1.4.4.2 Actions to be considered

The following characteristic values of actions are considered as provided by the crane supplier:

Nominal hoist load	: Q_{nom} = 100 kN
Maximum load per wheel of loaded crane	: $Q_{r,\max}$ = 73.4 kN

Minimum load per wheel of unloaded crane	: $Q_{r,\min} = 18.75$ kN
Horizontal transverse load per wheel	: $H_{T,i} = 9.4$ kN
Self weight runway beam	: $g_k = 88.2$ kg/m·10 m/s^2 =
(HEA 280 + KSN 50×30, see Figure 1.23)	= 0.882 kN/m
Neutral axis position of runway beam measured from bottom fiber (weared rail)	: $z_g = 149$ mm
Inertia of runway beam with weared rail (for static considerations, according to TGC11 (2006))	: $I_y = 155.8 \cdot 10^6$ mm^4
Inertia of runway beam's effective upper flange and weared rail (for fatigue considerations, 12,5% of the rail height)	: $I_y = 189.8 \cdot 10^3$ mm^4