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# The Assessment of Riveted Railway Bridges in accordance with Swiss Codes SIA 269

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## Abstract

The assessment of an existing railway bridge in wrought iron or in early mild steel requires special knowledge on the modelling of the actual actions and the choice of the adequate material properties. In 2011 the Swiss Society of Engineers and Architects (SIA) introduced the series of Codes SIA 269 as a tool for the maintenance of structures. This paper presents some parts related to the assessment of existing railway bridges. Some basic ideas came from the EU-Project "Sustainable Bridges", 2007. The choice of appropriate Wöhler curves for fatigue in the function of the construction details is important. Bridge inspections are fundamental as fatigue cracks do not start where you can calculate.

**Keywords:** Swiss Codes SIA 269, maintenance of structures, railway bridges, assessment of existing railway bridges, riveted details, wrought iron, mild steel, fatigue, bridge inspection.

## 1 Assessment of existing steel railway bridges (SIA 269)

### 1.1 General, load factors for updated actions

The structural analysis and the verifications for ultimate limit states, serviceability limit states and fatigue, to a small extent based on EU-Project "Sustainable Bridges", 2007, are carried out to the principles set out in SIA 260, analogously to those in EN 1990.

The load factors $\gamma_{G,act}$ for a **permanent action** may be updated (German: aktualisiert, French actualisé), if the updating of the permanent actions is carried out in accordance with the conditions given in SIA 269/1, as detailed below (extract of Table 1- SIA 269):

| Permanent actions | Load factor | ULS | Fatigue |
|---|---|---|---|
| - acting unfavourably | $\gamma_{G,sup,act}$ | 1,20 | 1,00 |
| - acting favourably | $\gamma_{G,inf,act}$ | 0,90 | 1,00 |

The load factors according to the SIA 260 (or EN 1990) apply for **variable actions** and actions arising from the ground, e.g.:

$\gamma_Q$ = 1,45 for rail traffic actions, where unfavourable (0 when favourable).

**Principle: The same value is used for existent or new constructions!**

## 1.2 Actions of normal gauge traffic, chapter 11 of SIA 269/1

Remark: The following translation of the above mentioned chapter was initiated one year ago by the author and may slightly differ from the official translated draft made by SIA, which will be printed definitively in English very soon.

*11.1 General*

*11.1.1 The following stipulations apply to loads and forces resulting from normal use by rail traffic of the track classes C3, C4, D3, D4, E4 and E5. The allocation of a structure to a given track class has to be effected by the railway company concerned, in accordance with the relevant authority.*

*11.1.2 In special cases, load tests can be useful for determining the load bearing behaviour of a bridge. These tests shall be planed and carried out in accordance with the relevant authority.*

*11.2 Updating*

*11.2.1 Vertical loads from railway traffic*

*11.2.1.1 The updated actions shall be determined using the load models (model vehicles) for the track classes C3, C4, D3, D4, E4 and E5. The model vehicles of the relevant track class shall be taken with an unlimited number of wagons and with their geometric properties given in Figure 1, the loads and forces placed in the most unfavourable positions. Traffic actions which produce a relieving effect may be neglected.*

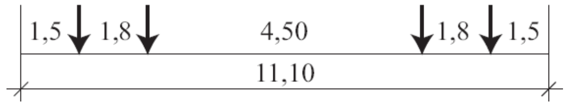
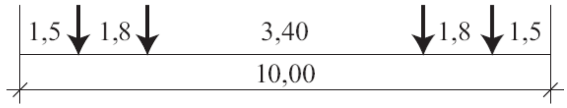
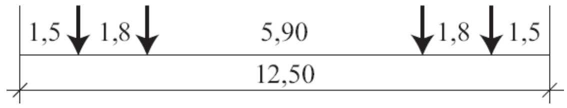
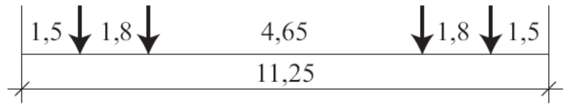
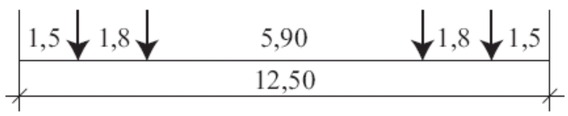
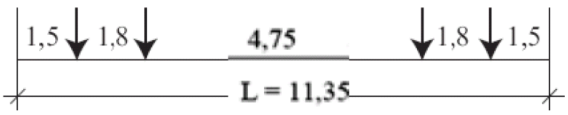
| Track class | Nominal axle load $Q_{act}$ [kN] | Geometrical properties of the model vehicles, dimensions in [m]<br>$Q_{act}$ $Q_{act}$ $Q_{act}$ $Q_{act}$ |
|---|---|---|
| *C3* | *200* |  |
| *C4* | *200* |  |
| *D3* | *225* |  |
| *D4* | *225* |  |
| *E4* | *250* |  |
| *E5* | *250* |  |

*Figure 1: Updated load models*

11.2.1.2 *To take into account an overloading of the axles due to not correct charging of wagons, the nominal axle load of the model vehicles shall be enhanced by 10%. For parts of the structure whose bearing direction is parallel to the track and whose span length or distance between supports is more than 20 m, this overloading factor may be neglected.*

11.2.1.3 *The dynamic effects due to irregularities of the track and of the rolling stock shall be taken into account with the speed dependant dynamic factors given in Annex A* (this annex is only mentioned, but not presented in this paper).

11.2.1.4 *For passenger trains with $v_{max}$ > 200 km/h the necessity of a dynamic analysis should be examined in accordance with EN 1991-2.*

11.2.2 *Horizontal actions from railway traffic*

11.2.2.1 *The updated characteristic braking force shall be determined with reference to Table 1*

| *Track class* | $QB_{\mathrm{k,act}}$ *[kN]* |
|---|---|
| *C3 / D3* | *18 l ≤ 5400* |
| *C4 / D4 / E4* | *20 l ≤ 6000* |
| *E5* | *22 l ≤ 6600* |

*Table 1: Updated characteristic values for braking forces for standard gauge (l = length in [m] on which the rail traffic loads act)*

*11.2.2.2* *The acceleration forces shall be taken in accordance with SIA 261*(or EN 1991-2).

*11.2.2.3* *The centrifugal force is updated with the axle loads of the model vehicles given in Figure 1. For lines with speeds* $v_{max} \geq 120$ *km/h the updated centrifugal force may be reduced by the factor* $\eta$ *given in SIA 261* (or EN 1991-2).

*11.2.2.4* *The actualised value of the nosing force* $QS_{k,act}$ *is 80 kN.*

*11.2.3* *Fatigue* (see SIA 269/3, Annex B or chapter 1.4 below)

*11.2.3.1* *For the assessment of the fatigue relevant stresses, the effective participation of border constructions and other equipments increasing the stiffness, as well as the favourable effect of the superstructure with long welded rails throughout, may be taken into account.*

*11.2.3.2* *The verification with respect to the fatigue cut-off limit shall be done with the updated effects given by the actions in paragraph 11.2.1 and the dynamic factors given in Annex A.*

*11.2.3.3* *The verification with respect to the fatigue strength shall be done with the Code SIA 261 using load model 1* (EN 1991-2: Load Model 71) *and the dynamic factor* $\Phi$.

*For the verification to the fatigue strength the following statements shall be taken for the damage equivalent factors:*

- *The damage equivalent factor* $\lambda_1$ *for the structural member type is the same as for new constructions*
- *The damage equivalent factor* $\lambda_2$*, that takes into account the foreseen annual traffic volume in future is taken as:*

$$\lambda_2 = \left(\frac{G}{25}\right)^{1/5} \qquad \text{(3, SIA269/1)}$$

*where*

*G is the future foreseen traffic volume in [*$10^6$ *t] per year and per track till the end of the remaining design life.*

- *The damage equivalent factor* $\lambda_3$*, that takes account of the total intended design life of the structural member is taken as:*

$$\lambda_3 = \left(\frac{T_{tot}}{100}\right)^{1/5} \qquad \text{(4, SIA269/1)}$$

*where*
*$T_{tot}$ is the total design life [in years], i.e. the sum of the past life time respecting paragraph 11.2.3.4 and the remaining life time*

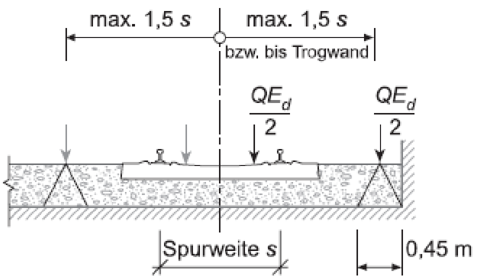
– *The damage equivalent factor $\lambda_4$ that takes into account the loading from more than one track is the same as for new constructions*

*11.2.3.4 Fatigue stresses that occurred before the year 1940 may be neglected.*

*11.2.4 Derailment*

*11.2.4.4 The derailment of rail vehicles shall be considered as an accidental design situation. The updated loads of derailed railway vehicles have to be taken as represented in Figure 2 and 3. By the way the other conditions given in SIA 261* (EN1991-2) *are admitted.*

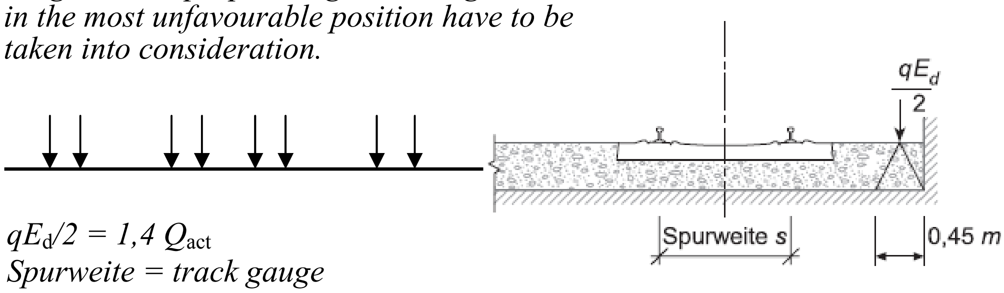
*Parallel to the track an unlimited number of model vehicles with the geometric properties given in Figure 1 are placed in the most unfavourable position. Actions which produce a relieving effect may be neglected.*




*$QE_d/2 = 0{,}7\ Q_{act}$*
*Spurweite = track gauge*
*bzw. bis Trogwand = resp. till against wall*

*Figure 2: Derailment load model 1*

*Parallel to the track two model vehicles with the geometric properties given in Figure 1 in the most unfavourable position have to be taken into consideration.*




*$qE_d/2 = 1{,}4\ Q_{act}$*
*Spurweite = track gauge*

*Figure 3: Derailment load model 2*

Not represented in this paper:

Annex A of SIA 269/1: Values of dynamic factors for the load models of the track classes as well as for service trains:

Table 3: Dynamic factors $1+\varphi$ as defined in EN 1991-2, Annex C, for ULS verifications

Table 4: Dynamic factors $1+\psi$ as defined in EN 1991-2, Annex D, for the verification of fatigue

The structural analysis and the verifications for ultimate limit states, serviceability limit states and fatigue for a railway bridge are carried out in accordance with the principles set out in SIA 260, analogously with those in EN 1990.

## 1.3 Material properties according to SIA 269/3 (and SIA 263)

For wrought iron and mild steel the following characteristic values may be adopted (for more details see paragraph 3.2.2.1 and Table 2 of SIA 269/3):

Wrought iron (1850 – 1900):
- ultimate tensile strength $f_{uk}$ = 320 N/mm² (direction of rolling)
- yield stress $f_{yk}$ = 220 N/mm²
- Young's modulus $E_k$ = 200 000 N/mm²
- Shear modulus $G_k$ = 77 000 N/mm²
- density $\rho$ = 78 kN/m³

Early mild steel (1890 – 1900):
- ultimate tensile strength $f_{uk}$ = 320 N/mm² (direction of rolling)
- yield stress $f_{yk}$ = 220 N/mm²
- Young's modulus $E_k$ = 200 000 N/mm²
- Shear modulus $G_k$ = 77 000 N/mm²
- density $\rho$ = 78 kN/m³

Mild steel (1900 – 1940):
- ultimate tensile strength $f_{uk}$ = 335 N/mm² (direction of rolling)
- yield stress $f_{yk}$ = 235 N/mm²
- Young's modulus $E_k$ = 210 000 N/mm²
- Shear modulus $G_k$ = 81 000 N/mm²
- density $\rho$ = 78 kN/m³

Partial factors for material properties, in general, in accordance with paragraph 4.1.3 of SIA 263:

$\gamma_M$ = 1,05 for ULS and stability, partial factor for material property
$\gamma_M$ = 1,25 for ULS and stability, partial factor for fasteners and net area of cross-sections

The **updated** factors for material properties in accordance with paragraph 5.1.1 of SIA 269/3 are:
$\gamma_{M,act} = \gamma_M \times k\gamma_M$

with:

$k_{\gamma M} = 1{,}10$ for wrought iron
$k_{\gamma M} = 1{,}10$ for early mild steel, 1890 - 1900
$k_{\gamma M} = 1{,}05$ for mild steel, 1900 – 1940

## 1.4 Fatigue strength curves for riveted constructions, a short history

Many of riveted railway bridges, constructed in the second half of the 19th and in the early 20th century are still in service today. The assessment of the remaining fatigue life of these bridges is gaining increased significance due to their advanced age of them. For economical reasons, railways and their relevant authorities are interested in keeping these structures in service as long as possible.
The author, before being retired as head of bridges at Swiss Federal Railways SBB, was involved with this problem over a number of years. As chairman of the expert committee ORE D 154 (Office of Research and Experiments, later called ERRI, European Rail Research Institute, headquarter in Utrecht) he lead a study which resulted in a publication in German in 1986, with the title "DT 176, Statistische Auswertung von Ermüdungsversuchen an Nietverbindungen in Flussstahl". This study contains a presentation of the results of 326 riveted samples tested under fatigue. The results had been extracted from 13 publications. A best fitting Wöhler- or $\Delta\sigma$-N curve covering the lower bound of the data was determined by the likelihood method. The influence of the stress ratio $R = \sigma_{min} / \sigma_{max}$ was studied, but no clear influence was found so its use was finally abandoned.
Later, at the end of the last century, in the context of a bilateral study between specialists of the German and Swiss railways (DB and SBB) and with the help of specialists of the Swiss Federal Institute of Technology in Lausanne as well as of those of the Technical University of Aachen, Germany, some further results of fatigue strength tests were found and analysed. These gave results outside the lower bound $\Delta\sigma$-N curve of ORE D 154. The intermediate conclusion was then to recommend the more conservative fatigue detail category EC 71 according to EN 1991-3, the same curve as that used for fatigue assessment for new constructions, see Figure 4 below.
The most important step for going forward in knowledge of fatigue strength for riveted constructions was the study of Prof. Greiner and his assistant Andreas Taras, Austria , which dealt also with the stress ratio R, see Figure 5. The author of the present article has some doubts about the curves with positive values of R. The concept of taking the influence of R was abandoned for Swiss Codes.
But if Figure 6 is considered, it is obvious, that the work of Greiner and Taras has influenced the work for SIA 269/3 very much.

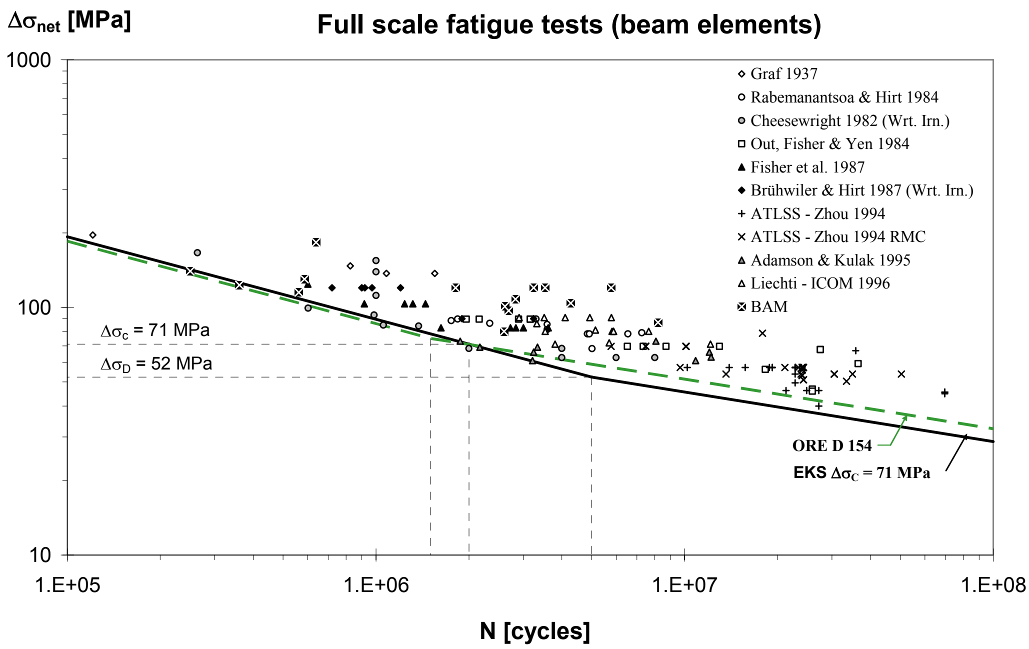



Figure 4: Δσ-N curves ORE D 154 and fatigue detail category EC 71 (EN 1991-3)

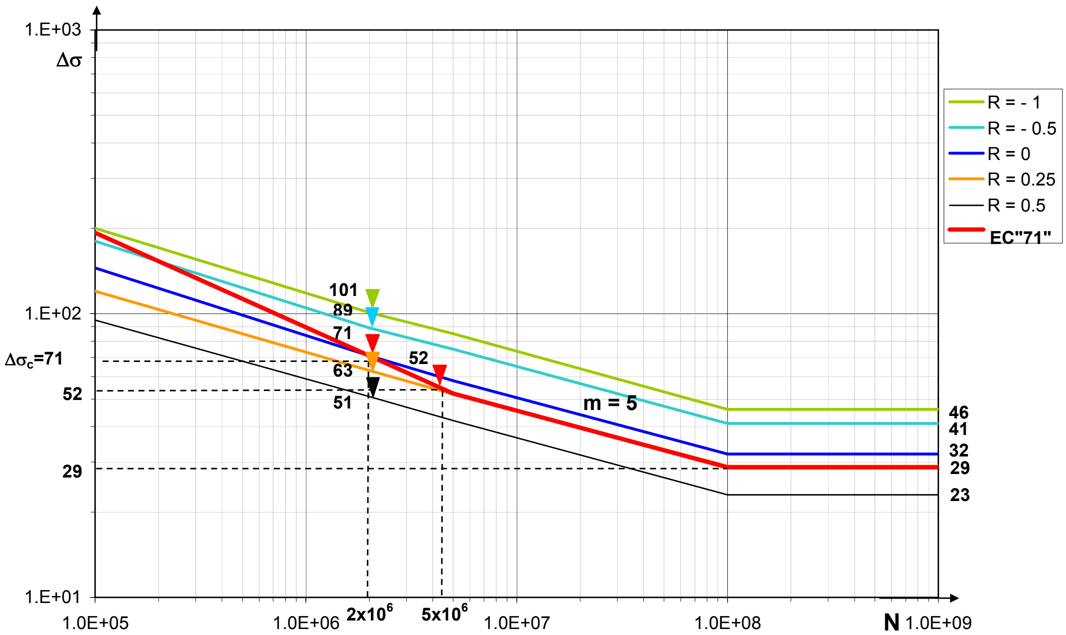



Figure 5: Δσ-N curves due to Prof. R. Greiner/A. Taras in function of R = $\sigma_{min}$ /$\sigma_{max}$

In **SIA 269/3** the notch categories of different riveted construction details (see Table 3 below) can be assigned to the notch category **Δσ = 71 N/mm2** (don't confound with the curve EC 71 given in EN 1991-3 for new steel constructions!) with a **gradient of the fatigue strength curve of m = 5** (see Figure 6).

The residual stresses can be neglected in a riveted structure and the stress difference must be determined on the net cross section. The updated factor for fatigue strength $\gamma_{Mf,act}$ is to be determined according to the accessibility for inspection specified as follows:

| | Minor consequence | Significant consequence |
|---|---|---|
| Inspected detail without cracks | $\gamma_{Mf,act} = 1{,}0$ | $\gamma_{Mf,act} = 1{,}0$ |
| Construction detail without possibility of inspection | $\gamma_{Mf,act} = 1{,}15$ | see 269/3, paragraphs 5.5.2.4 and 7.5.1.3 |

Table 2 (extract of Table 11 of SIA 269/3): Updated resistance factor $\gamma_{Mf,act}$ for fatigue strength for general examination

If the stress spectrum of the construction is known, the verification may be carried out with the Palmgren-Miner damage accumulation theory and a damage limit value equal to 1,0.

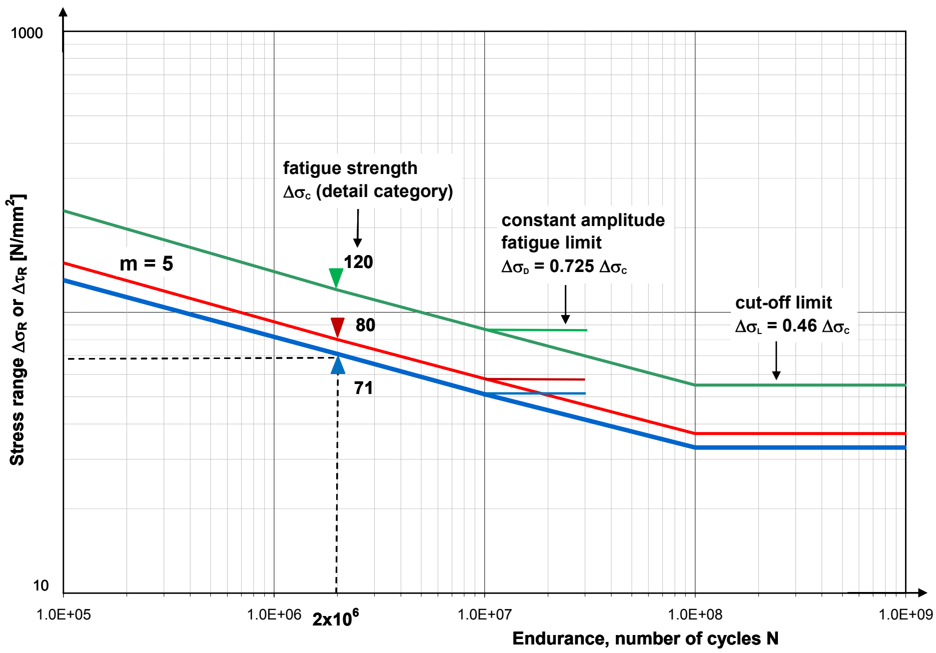



Figure 6: (Figure 5 in Annex B of SIA 269/3): Fatigue strengths according to the notch categories for riveted constructions

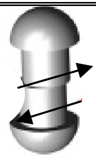
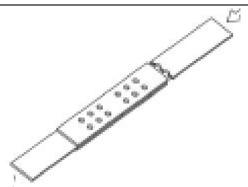
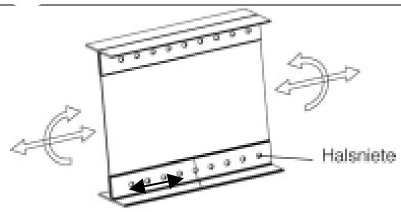
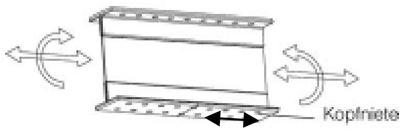
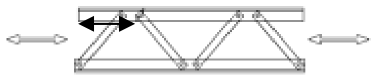
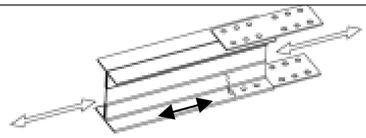
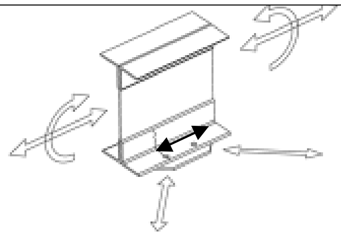
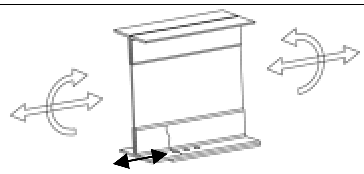
| Notch category [N/mm$^2$] | Construction detail | Description |
|---|---|---|
| $\Delta\tau_C$ = **120** |  1 | Rivets subjected to shearing forces in single-shear or multiple shear connections |
| $\Delta\sigma_C$ = **80** |  2 | Butt joint between plates spliced on both sides, two-shear connection |
| $\Delta\sigma_C$ = **80** |  3 | Continuous connection between flange angle and web plate in built-up flexural members. $\Delta\sigma_{E2}$ at neck rivet (Halsniete) in web. |
| $\Delta\sigma_C$ = **80** |  4 | Continuous connection between flange angle and web plate in built-up flexural members. $\Delta\sigma_{E2}$ at head rivets (Kopfniete) in the flange. |
| $\Delta\sigma_C$ = **80** |  5 | Built-up laced members stressing tension and compression |
| $\Delta\sigma_C$ = **71** |  6 | Butt joint between plates spliced on one side, one–shear connection |
| $\Delta\sigma_C$ = **71** |  7 | Area of the connection of a splice to the tension flange of a flexural member |
| $\Delta\sigma_C$ = **71** |  8 | Area of the end anchoring of a reinforcing plate |

Table 3 (Tab.13 in Annex B of SIA 269/3): Riveted construction details

# 2 The important role of inspections and a proposal for an extension of the usual assessment of existing bridges

## 2.1 General

Not only technical and economical issues, but also aspects of cultural heritage have to be considered when examining the remaining life time of riveted railway bridges constructed in the second half of the 19th and in the early 20th century, see for example Figures 7 and 8.

Figure 7: Eglisauer Bridge (Switzerland), Constructed 1895-97, line Eglisau – Hüntwangen, span of steel truss 90 m

Figure 8: Salzacherbridge (A) constr. 1907, line Salzburg - Wörgl, span 40 m

In old truss bridges with rails on wooden sleepers which lie on a rail bearer and a cross girder grid (open decks), fatigue damage will be less likely to occur in the elements of the main truss girder than in the connections of the “axle-sensitive” elements such as rail bearer*s* and cross girder*s*. The reason is, that, for the same traffic, the short spanned elements have to endure a greater number of stress range cycles than the elements of greater spans. For the short elements each axle load or bogie of a train causes a stress cycle but for the main girder, it is the whole train that causes the stress cycle. Unfortunately, the cracks occur in the region of the connections of the “axle- sensitive” elements where you cannot calculate or measure the stresses. Two typical examples of cracks, one in a rail bearer, one in a cross girder, are shown in Figures 9 and 10.

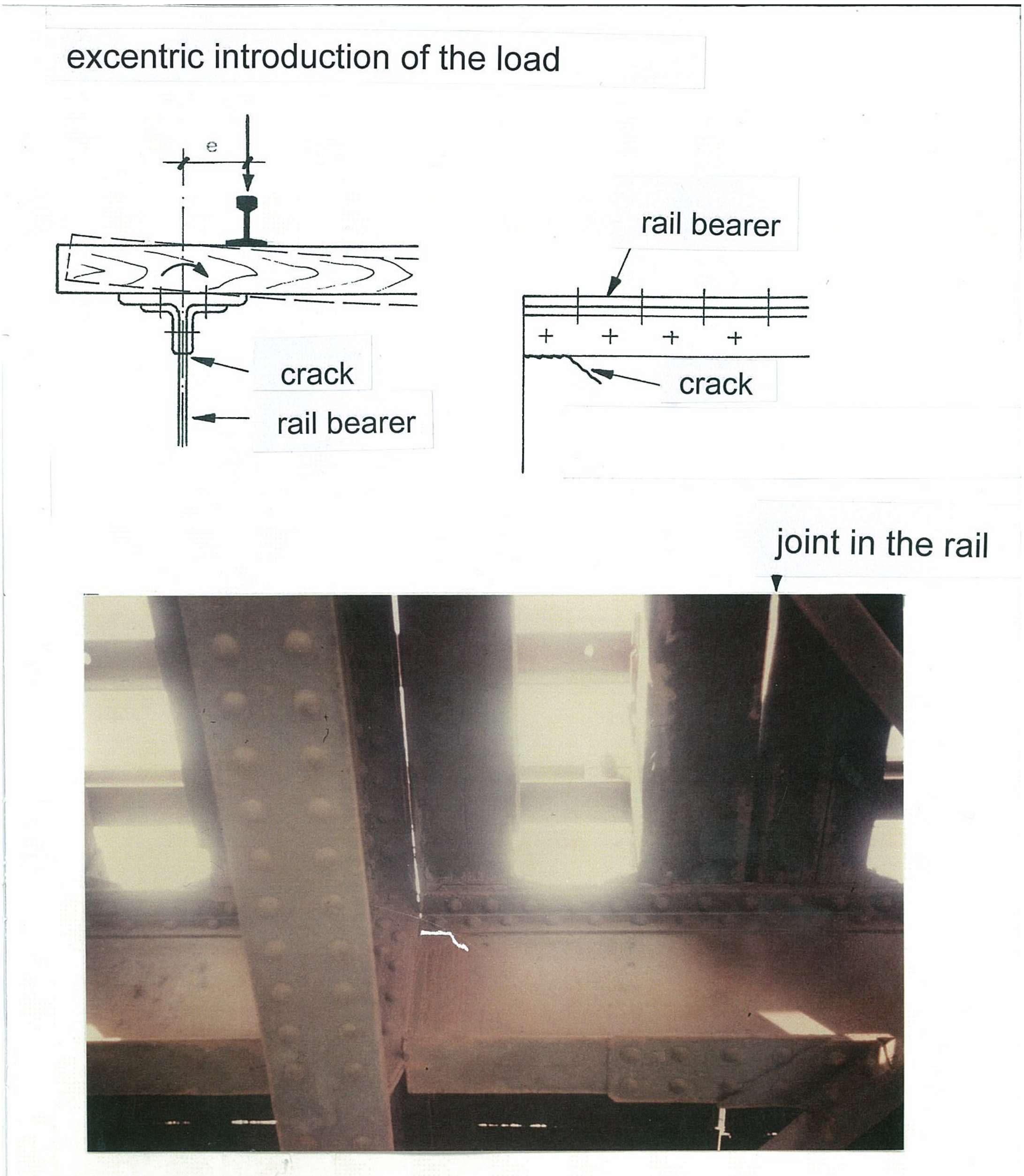



Figure 9: Typical crack in a connection of a rail bearer

The eccentricity between the axle of the rail and the longitudinal axle of the rail bearer provokes the bending of the wooden sleeper under each axle. That gives repeated bending between the upper flange and the web of the rail bearer which leads first to a horizontal crack, which with the time travels to the lower flange.
The final collapse of a rail bearer can lead to a twist in the track and to a derailment. In the example, the source of damage was a joint of the rail just above the connection which gave enormous dynamic increments from the axle loads.

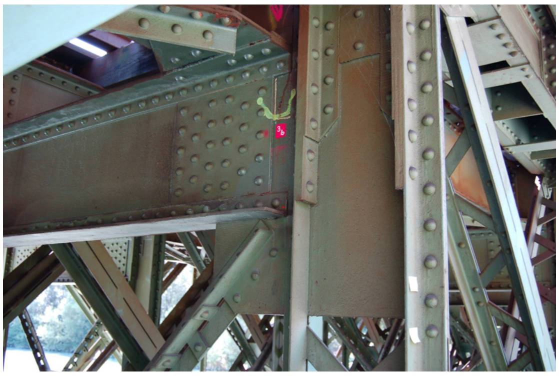
 

Figure 10: Typical crack in a connection of a cross girder to a main girder

## 2.2 The assessment of the Rhine Bridge near Eglisau (Switzerland)

The series of Codes SIA 269 are a very useful instrument for the assessment of existing bridges. This can be proved with the monitoring (strain gauges on various elements of the bridge) and calculations carried out for the railway bridge in early mild steel over the Rhine near Eglisau (see Figure 7).

The calculations were carried out using a three dimensional beam model and the comparison of these results with the measured results was very good.

Limit state design:
The ultimate limit state for track class D4 was verified, giving just sufficient results concerning the safety factors.

Dynamic increments:
Special dynamic studies were not necessary because the long term monitoring of the bridge, which measured the effects of all the real trains for more than one year, gave stress range results that included the dynamic increments.

Fatigue:
The most important aspect in determining the remaining service life of a bridge is the estimation of its remaining fatigue life. The linear damage theory of Palmgren-Miner is used in accordance with SIA 269/3. According to this theory, accumulated damage S resulting from a multi-stage spectrum ($\Delta\sigma_i$, $n_i$) can be expressed by the linear sum

$$S = \Sigma\, n_i / N_i \qquad (1)$$

in which $N_i$ is the number of load cycles on the Wöhler curve (see Figure 6) at the level $\Delta\sigma_i$. Theoretically fatigue failure occurs when

$$S = 1 \tag{2}$$

For the Rhine Bridge Eglisau the stress history is measured for each real train crossing the bridge for a period of more than one year. The histograms of the stress range $\Delta\sigma_i$ are obtained by using the Rainflow counting method and are saved as multi-stage spectrum. Stress ranges less than the cut-off limit shown in Figure 6 are disregarded.

Knowing the traffic data from SBB (Swiss Federal Railways), not only the number of trains per day like shown in Figure 11, but also the actual tonnage carried over in the year 2011 (during the long term monitoring) and in the past, and taking into account an increase in traffic for the future, the remaining estimated service life can be determined. At the time of writing this paper, the results have not yet been published, but I am confident that there will be no negative remaining service life in the points measured. The reason is obvious when you consider the places and types of cracks shown in Figures 9 and 10.

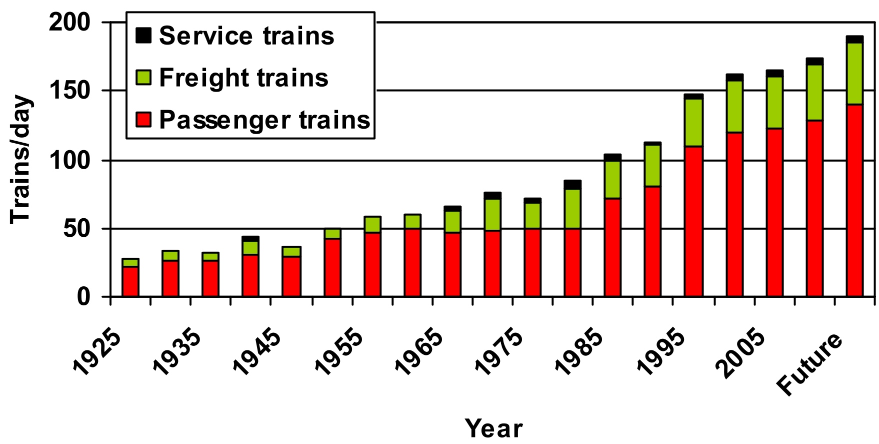



Figure 11: Trains per day on the Rhine Bridge near Eglisau, in the past, actual and admitted for the future

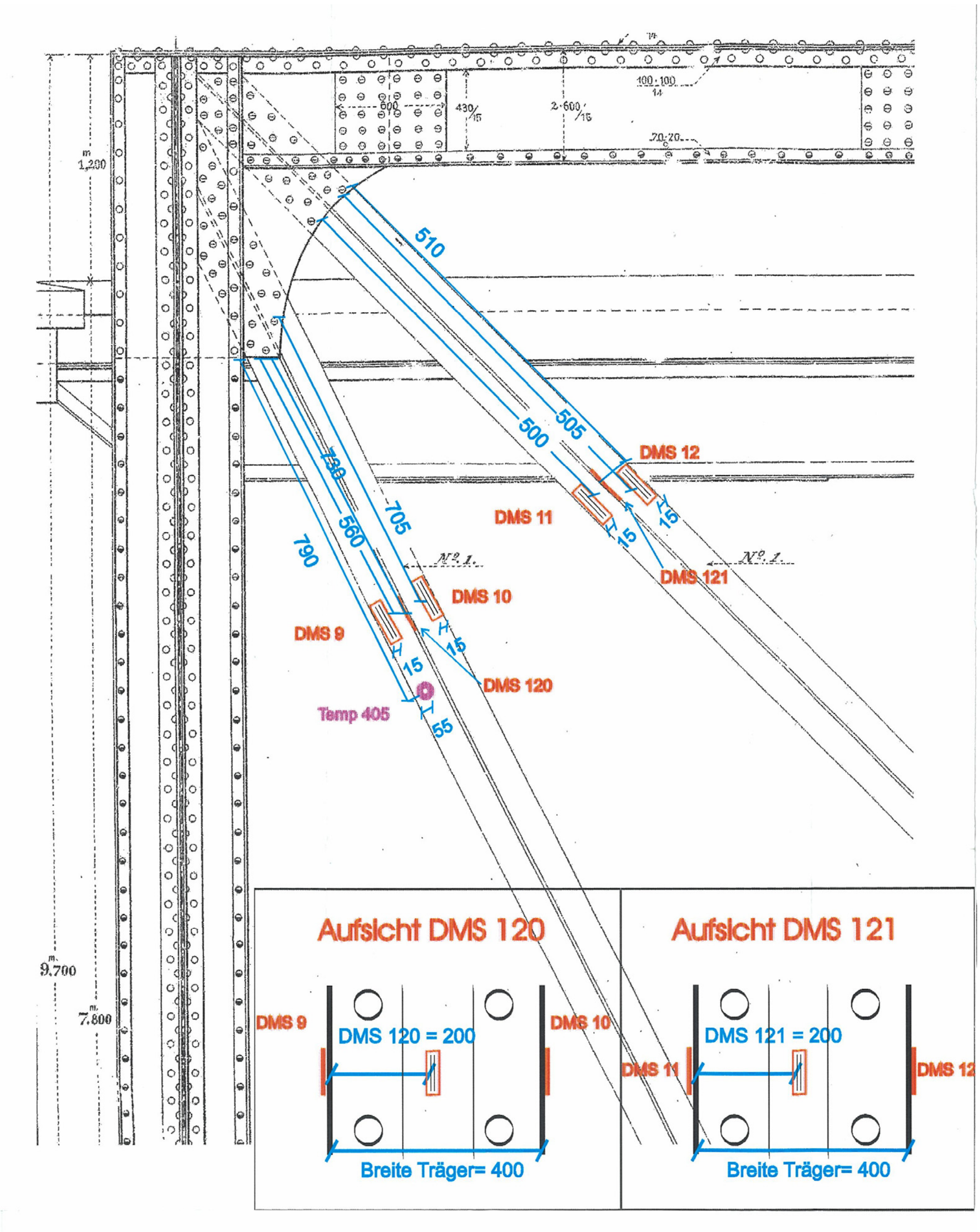
## 2.3 A proposal for an extension of the usual assessments

In general, as previously stated and shown, the measuring with strain gauges is not possible at the places where crack could or will occur.

In Eglisau, first the stresses using strain gauges at different points on different structural elements were measured, under the loading of one or more locomotives placed in different but clearly identified positions on the bridge. A good agreement between measured and calculated stresses is of course assumed.

In addition long term continuous measurements were carried out at several important positions, measuring the strains (stresses) under all the real trains during several months. These results allowed us to determine, with the help of the Rainflow Counting Method, the multi-stage spectrums of the different structural elements.

For the elements of main girders, the transfer of the measured results of stresses (at the places where strain gauges are fixed) to those in the critical verification sections is easy, see for example the situation in the two diagonals of a main girder in Figure 12. These calculations can be carried out with the simple determination of a transfer factor. In the critical sections the net cross section has to be taken into account.




Figures 12: Measuring points (in red) at two end diagonals of a main girder (Rhine Bridge Eglisau)

But for determining the remaining fatigue life of an "axle-sensitive" structural element such as the connection of a cross girder, it is not easy to determine a transfer factor in a similar way to a main girder element, see Figures 10 and 13.

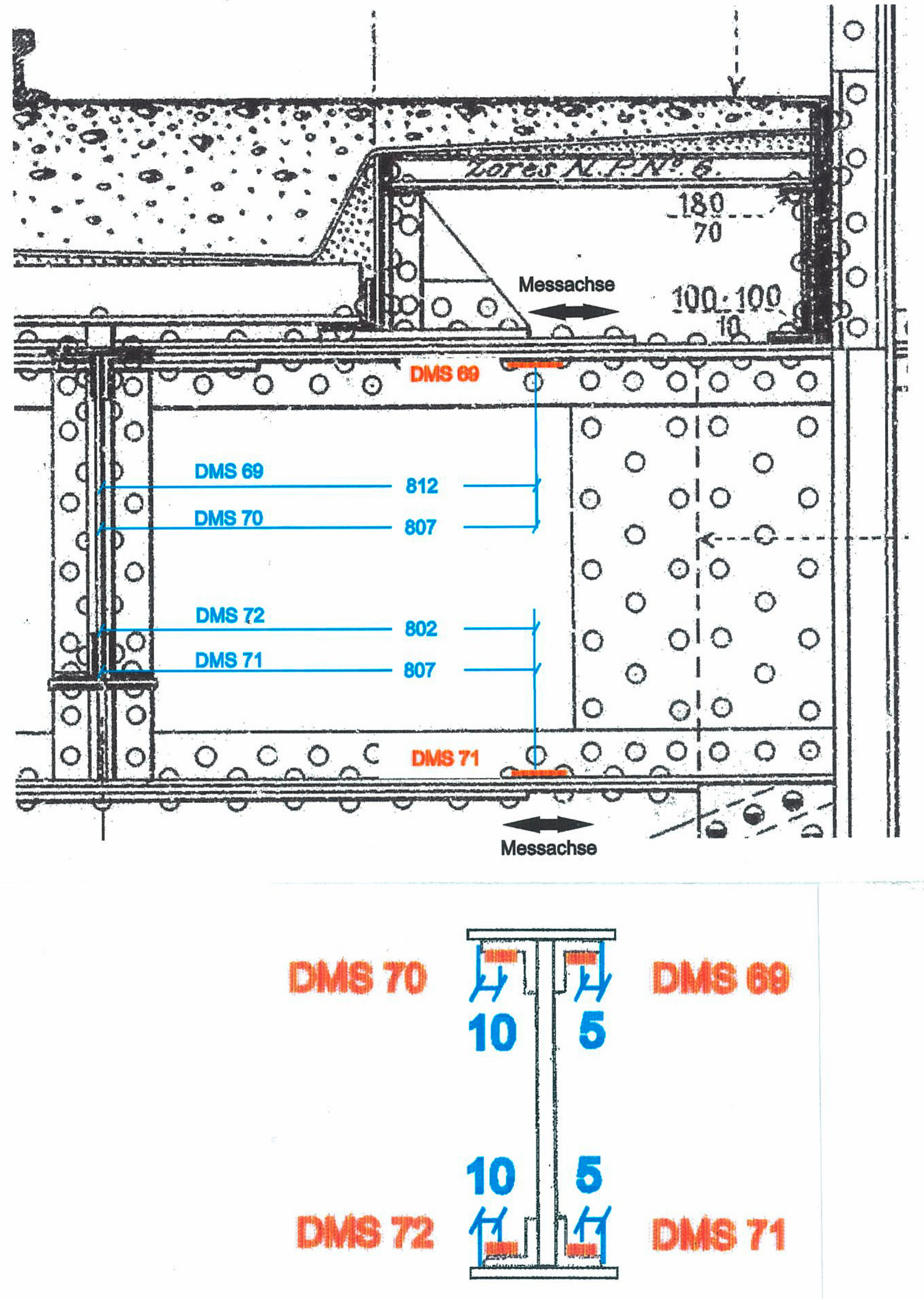



Figure 13: Measuring points (in red) at a cross girder end (Rhine Bridge Eglisau)

For such a case, not only for the Eglisau Bridge, but also for some of the other structures analysed inside the project FADLESS "Fatigue damage control and assessment for railway bridges", the following approach is recommended:

**The measured data should be combined with the data from a FEM (Finite Element Method) study.** This of course requires an extension of the usual studies. The outcome of this supplementary study and the knowledge of the local stresses and their level, then combined with the characteristics of fatigue, provide the basis for determining the remaining fatigue life, either with a **crack initiation** or a **crack growth analysis based on linear fracture mechanical models.**

The following steps are proposed, e.g. for a cross girder connection:

- To obtain the stress distribution in the connected plates and the forces on the rivets a detailed finite element model must be established, and this model must be integrated into the whole coarse bridge FE model.
- Locomotives must be placed on the bridge in the most adverse position in order to determine the forces/stresses in the cross girder connection (measured/calculated).
- The resulting stresses at the critical locations from the finite element model are defined as reference stresses. They are compared with the measured - calculated stresses at the points equipped with strain gauges. So a transfer factor can be determined.
- The reference stresses from the finite element model are corrected by this transfer factor.
- The measured stress spectra (at the points equipped with strain gauges) must now be multiplied by the transfer factor mentioned above. This results in normalized stress spectra for the connected plates, in each critical location.
- These normalized stress spectra are the basis for estimating the remaining fatigue life at each critical location of the connected plates, either by a crack initiation or a crack growth analysis.

If hidden cracks are suspected in the multi layered connection, non-destructive testing can support the local inspection. **A Radiographic test is an appropriate inspection method for the detection of hidden cracks.** The method requires accessibility from both sides, the film is attached to the structure from the opposite side of the detail as the radiation source. Best resolution of the crack is achieved in a radiation direction perpendicular to the main stresses. The resolution is further dependent on the activity of the source, the exposure time and the angle of the radiation in relation to the crack.

I hope that some members of the project FADLESS will follow my recommendation for an extension of their project related to rail bearer and cross girder connections using Finite Element methods.