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SIA 269/3:2011 Construction

Schweizer Norm
Norme Suisse
Norma Svizzera

505 269/3

Erhaltung von Tragwerken – Stahlbau

Maintenance des structures porteuses – Structures en acier

Conservazione delle strutture portanti – Costruzioni in acciaio

Existing structures – Steel structures

269/3

Reference number
SN 505269/3:2011 en

Valid from: 2011-01-01

Published by
Swiss Society of Engineers
and Architects
P.O. Box, CH-8027 Zurich

Number of pages: 32

Price category: 34

Any corrections to and comments on this publication can be found at www.sia.ch/korrigenda.

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2016-09 1st Edition

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FOREWORD

Code SIA 269/3 describes the procedures to be followed in the preservation of structures made of cast iron, iron and steel. It addresses specialists in the preservation of structures as well as owners of construction works and specialists involved in construction management and the execution of construction works.

Code SIA 269/3 forms part of the SIA structural codes relating to the preservation of structures and is supplemented by the following codes:

- Code SIA 269 Existing structures – Bases for examination and interventions
- Code SIA 269/1 Existing structures – Actions
- Code SIA 269/2 Existing structures – Concrete structures
- Code SIA 269/4 Existing structures – Composite steel and concrete structures
- Code SIA 269/5 Existing structures – Timber structures
- Code SIA 269/6 Existing structures – Masonry structures
- Code SIA 269/7 Existing structures – Geotechnics.

Technical Specification SIA 2018 continues to apply to the examination of existing buildings with respect to earthquakes. However, it is planned to supplement the codes for existing structures with a Code SIA 269/8 *Existing structures – Seismic aspects*.

Code SIA 269/3 addresses the aspects of preservation of steel structures that are not covered by Codes SIA 263 and 263/1.

Codes SIA 269 Project Management Team and Working Group SIA 269/3

0 SCOPE

0.1 Limitations

0.1.1 Code SIA 269/3 applies to the preservation of structures made of cast iron, iron and steel as part of existing construction works.

0.1.2 The present code applies in conjunction with Codes SIA 269 and SIA 269/1 as well as with Code SIA 263. The principles of Code SIA 269/3 shall be applied analogously for applications outside of the scope of these codes.

0.1.3 In the case of conversions or extensions to steel structures, new structural members are to be treated in accordance with Code SIA 263, whereas existing structural members and connections are to be treated in accordance with Codes SIA 269 and SIA 269/3. It is possible to deviate from this rule with respect to the definition of the variable actions if different limitations are appropriate on the basis of specific considerations.

0.1.4 Code SIA 269/3 may not be used for the design and dimensioning of new steel structures.

0.2 References

0.2.1 Reference is made to the standards named below. These are also applicable, in full or in part, within the terms of the reference:

– EN 10025:2002, Parts 1–6 *Hot-rolled products of structural steel – general technical delivery conditions*
– EN 10045-1:1990 *Charpy impact test on metallic materials – Part 1: Test method (V- and U-notches)*
– EN 1561:1997 *Founding – grey cast iron*
– SN EN 12944, Parts 1–8 *Coating materials – Corrosion protection of steel structures by protective paint systems*
– SN ISO 4628:2004, Parts 1–10 *Paints and varnishes – Evaluation of degradation of coatings – Designation of quantity and size of defects, and of intensity of uniform changes in appearance*
– SN EN 1993-1-9:2005, Eurocode 3 *Design of steel structures – Part 1–9: Fatigue*
– SN ISO 898-1 *Mechanical properties of fasteners made of carbon steel and alloy steel – Part 1: Bolts, screws and studs with specified property classes – Coarse thread and fine pitch thread*
– ISO 8504, Parts 1–3 *Preparation of steel substrates before application of paints and related products – Surface preparation methods .*

0.2.2 Superseded codes and guidelines to which reference is made may not be incorporated in the service criteria agreement and in the basis of design as code-related requirements, but only by way of reference, and only in order to document former basic principles.

0.3 Deviations

0.3.1 Deviations from the present code are permissible provided they are sufficiently well-founded, theoretically or experimentally, or justified by new developments and new knowledge.

0.3.2 Deviations from the code shall be clearly documented in the construction documents together with the reasons for such deviation.

1 TERMINOLOGY

1.1 Technical terms

In addition to the technical terms used in Codes SIA 260, SIA 263, SIA 263/1 and SIA 269, the technical terms defined below are used in the present code.

Corrosion fatigue *Korrosionsermüdung* *fatigue-corrosion* *fatica da corrosione*	Phenomenon of cracking in a material as a result of combination of fatigue stress (repeated loading) and corrosive environment (liquid or gaseous, under the influence of the oxygen content, temperature, acidity) with cracks developing more rapidly in comparison to cracking in a non-corrosive environment. High-strength and austenitic steels are more susceptible.
Loosing connection *lockere Verbindung* *dislocation d'assemblage* *unione allentata, scorrimento del giunto*	Riveted or bolted connection in which the steel plates are deforming excessively, even start to move relative to one another. Connection which loosens as a result of corrosion or as a result of other deterioration.
Material of a weld *Schweissnahtwerkstoff* *matériau de soudure* *materiale di saldatura*	Material in a weld, consisting of the welding filler and the metallurgical mixture in the transition to the base material (heat affected zone HAZ).
Old structural steel *alter Baustahl* *acier de construction ancien* *acciaio da costruzione obsoleto*	Collective term for wrought iron, mild rimmed iron and mild steel produced before 1956, see definition in Appendix A, Table 12.
Riveting *Heftnietung* *rivure* *rivettatura*	Riveting at regular intervals of two or more elements (generally steel plates and/or angles) so that they form a structural member.

1.2 Symbols

In addition to the symbols defined in Code SIA 263, the symbols defined below are used in the present code.

1.2.1 Latin uppercase letters

A_s	stressed cross-sectional area of the bolt, in the threaded area
A_v	effective shear area
A_w	web surface area $(h-t_f)t_w$ or web surface areas of box girders
D	measured diameter of the rivet head or the washer of a bolt
$F_{b,Rd,act}$	examination value for the bearing resistance per bolt or rivet
$F_{t,Rd,act}$	examination value for the tensile strength per bolt or rivet
$F_{v,Rd,act}$	examination value for the shear resistance per shear plane of a bolt or rivet
I	moment of inertia
K_V	CVN notch impact energy
R	measured radius of the rivet head
W_{pl}	plastic section modulus

1.2.2 Latin lower case letters

b	height, width of the local plate buckling field
d	diameter of the raw rivet or the bolt shaft
d_0	diameter of the rivet or bolt hole
e_0	equivalent initial imperfection $e_0 = e_{0,geom} + e_{0,\sigma}$
$e_{0,geom}$	relative deflection of the element ends, measured on the structure
$e_{0,\sigma}$	equivalent eccentricity to take into account the action effect of the residual stresses on the buckling stability of the member
e_1	distance from edge in the direction of force of a bolted or riveted connection
e_2	distance from edge perpendicular to the direction of force of a bolted or riveted connection
f_{yk}	characteristic value for the yield strength of the base material
f_{ykB}	characteristic value for the yield strength of the bolt material
f_{ukB}	characteristic value for the tensile strength of the bolt or rivet material
f_{ukE}	characteristic value for the tensile strength of the welding filler
k	measured height of the rivet head
$k_{\gamma M}$	correction coefficient for old structural steels and cast iron
p_1	bolt pitch; in the direction of force of a bolted or riveted connection
p_2	bolt pitch; perpendicular to the direction of force of a bolted or riveted connection
t	thickness
t_f	flange thickness
t_w	web plate thickness

1.2.3 Greek letters

α_K	imperfection factor for buckling
$\gamma_{M,act}$	updated resistance factor
ε_{act}	updated reduction factor for the yield strength
ε_{ukB}	characteristic value for ultimate strain of the rivet material
$\bar{\lambda}_K$	relative buckling slenderness
τ_{yk}	characteristic value for shear yield stress $\tau_{yk} = f_{yk}/\sqrt{3}$
χ_K	reduction factor for buckling
$\Delta\sigma_C$	fatigue strength at $2 \cdot 10^6$ load cycles under normal stresses, notch category
$\Delta\tau_C$	fatigue strength at $2 \cdot 10^6$ load cycles under shear stresses, notch category

1.2.4 Abbreviations

HAZ	heat affected zone
WPAR	Welding Procedure Approval Record
WPS	Welding Procedure Specification

2 PRINCIPLES

2.1 General

2.1.1 For structures built after 1956, it is assumed that modern structural steels were used that possess comparable properties to the structural steels commonly used today, as defined in Code SIA 263.

2.1.2 The defects and deteriorations identified on the basis of a condition survey, in particular, as a result of corrosion and fatigue, shall be taken in account in defining the hazard scenarios and in the selection of the failure mechanisms.

2.2 Examination

2.2.1 The geometry of an existing steel structure shall comply with the tolerances defined in Code SIA 263/1. If these tolerances are not complied with, the following examination values shall be taken in account for the purpose of verifications:

– for cross-sectional dimensions, the nominal values or, in the case of a detailed examination, the measured values are to be used. In the event of material loss as a result of corrosion or wear, this loss is to be taken into consideration in accordance with Section 5.1.2
– the effective geometry shall be used for a 2nd order analysis. In the case of frames, the effective value for sway imperfection is to be used. As for the bow imperfection, which includes the permanent deformations, Section 5.4.3.2 may be used when taking the effective bow imperfection into consideration.

2.2.2 The influence of specific defects on the durability and resistance of the structural members as well as the influence of the resulting deterioration shall always be examined.

2.2.3 In the case of cracks detection, the cause and mechanism shall always be established. Possible mechanisms may be attributable to fatigue, corrosion fatigue, hydrogen embrittlement, stress corrosion cracking or a high local stress (notch effect).

2.2.4 Urgent safety measures shall always be implemented in the case of cracks due to corrosion fatigue or stress corrosion cracking.

2.2.5 If originally non-load-bearing structural members made of cast iron assume a load-bearing function, the reliability of this function shall be examined, since failure mechanisms can occur without any advance warning in the case of cast iron.

2.2.6 In the case of loosing connections, interventions in accordance with Sections 7.2.2 or 7.3.2 are recommended.When evaluating durability, particular attention shall be paid to locations with increased (local) corrosion loads (damp spots, dirty or mossy locations or locations with standing water).

2.2.8 The gross cross-section shall be used when examining the serviceability and vibration behaviour of riveted structural members.

3 MATERIALS

3.1 General

3.1.1 Designations of modern structural steels are shown in Table 1.

Table 1: Designations of modern structural steels

SIA code	SIA 161	SIA 161	SIA 161	SIA 161	SIA 263
Year of coming into force	1956	1974	1979	1990	2003
Structural steel, f_{yk} = 235 N/mm^2	St 37	Ac 24/37	Fe 360	Fe E 235	S 235
Structural steel, f_{yk} = 355 N/mm^2	St 52	Ac 36/52	Fe 510	Fe E 355	S 355
Note: Appendix A contains a description of old structural steels and cast irons.					

3.1.2 The characteristic values of the mechanical properties of modern structural steels are listed in Code SIA 263. A plausibility check in accordance with Section 2.1.1 shall be carried out.

3.1.3 The characteristic values of the mechanical properties of structural steel or fasteners as well as the chemical composition serve primarily to classify the steel and thus justify the application of the characteristic values shown in Tables 2 and 5. If the measured values for the mechanical properties can be shown to be representative, they may be used for updating the characteristic values of the steel.

3.1.4 The test methods shall be carried out in accordance with the EN 10025 series of European Standards.

3.2 Old structural steels and cast iron

3.2.1 General

3.2.1.1 The mechanical properties as well as the chemical composition of old structural steels and cast iron can vary significantly depending on the manufacturer, year and country of manufacture.

3.2.1.2 If the old structural steel or the cast iron can be clearly identified with the help of the construction documents or on the basis of the shape or the surface properties of the structural member, the characteristic values shown in Table 2 may be used as the characteristic values of structural steel or cast iron.

3.2.2 Characteristic values

3.2.2.1 The characteristic values of old structural steels and cast iron are shown in Table 2.

Table 2: Characteristic values of old structural steels and cast iron

Material	Period of use[1)]	f_{yk} [2)] [N/mm^2]	f_{uk} [2)] [N/mm^2]	G_k [N/mm^2]	E_k [2)] [N/mm^2]	ε_{uk} [2)] [%]	v	ρ_a [kg/m^3]	α_T [10^{-6}/°C]
Cast iron [3)]	before 1900	+70/–200 [4)]	+120/–600	29 000	78 000	< 0,8	0,26 [5)]	7250	10
Wrought iron	1850–1900	220	320	77 000	200 000	15	0,3	7800	10 [6)]
Mild rimmed iron	1890–1900	220	320	77 000	200 000	25	0,3	7800	10 [6)]
	1900–1940	235	335	81 000	210 000	25	0,3	7800	
Mild steel	1925–1955	235	360	81 000	210 000	25	0,3	7850	10 [6)]

1) Main phase of manufacture
2) Parallel to the direction of rolling (see Section 3.2.2.2)
3) Cast iron with lamellar graphite in accordance with EN 1561:1997
4) Conventional value at 0,1% ultimate strain, since cast iron has no yield range
5) Average value for different types of cast iron
6) Code SIA 263 is valid for all steels and can be used for more precise values.

3.2.2.2 The characteristic values of old structural steels depend greatly on the direction of loading. The characteristic values shall be reduced in the case of loading in the plane of the plate, transverse to the direction of rolling. For wrought iron, the resistance transverse to the direction of rolling is taken as 80% of the resistance in the direction of rolling; for examinations, no resistance is considered in the thickness direction.

3.2.3 Steel toughness of old structural steels

3.2.3.1 The steel toughness of old structural steel or cast iron shall be determined by means of Charpy V-notch bar impact tests in accordance with EN 10045-1.

3.2.3.2 With the exception of cast iron and Thomas steel, old structural steels have a minimum Charpy V-notch impact value of K_V = 27J at a temperature of T_{27J} = 20 °C. This Charpy V-notch impact value corresponds to the steel toughness symbol JR according to EN 10 025.

3.3 Connections

3.3.1 Rivets

3.3.1.1 The determination of the characteristic material properties of rivets by means of mechanical tests is only recommended if the properties cannot be clearly identified on the basis of the construction documents.

3.3.1.2 Characteristic material properties for rivets are shown in Table 3.

Table 3: Characteristic material properties for rivets

Material	Period uf use[1]	f_{ukB} [N/mm^2]	ε_{ukB} [%]
Wrought iron	1850–1900	320	18
Mild rimmed iron	1890–1940	320	28
Mild steel	from 1925	350	30
[1] Main phase of manufacture			

3.3.2 **Old bolts**

3.3.2.1 The determination of the characteristic material properties of old bolts by means of mechanical tests is only recommended if the bolts cannot be clearly identified on the basis of the construction documents or classified with reference to Table 4.

3.3.2.2 The designations and characteristic material properties of old bolts are shown in Table 4.

Table 4: Designation and characteristic material properties of old bolts

	Old designation	Period of use [1]	f_{ykB} [N/mm^2]	f_{ukB} [N/mm^2]	Equivalent strength class [2]
”normal” bolts	4D, St 38	from 1920	240	400	4.6
	5D	from 1920	300	500	5.6
high-strength bolts	8G	from 1950	640	800	8.8
	10K	from 1950	900	1000	10.9
[1] Main phase of manufacture [2] In accordance with SN ISO 898-1					

3.3.2.3 Without precise specifications and clear classification, bolts of an existing structure may be classified under the equivalent strength class 4.6.

3.3.2.4 In the case of pretensioned connections, the corresponding fittings have to be present. A representative random sample of bolts shall be removed to check whether an identification (e.g. 10.9 or 10K) is visible on the fitting components. Section 3.3.2.1 shall be applied when no identification is present.

3.3.3 **Old welded connections**

3.3.3.1 The determination of the characteristic material properties of old weld materials by means of mechanical tests is only recommended if the welds cannot be clearly identified on the basis of a plausibility check with reference to the construction documents and Table 5.

3.3.3.2 The designation as well as the characteristic material properties of old weld materials are shown in Table 5.

Table 5: Designation and characteristic material properties of old weld materials

Period of use[1)]	f_{ukE} [2)] [N/mm^2]
1900–1924[3)]	300
1925–1955[4)]	360
from 1956[4)]	at least equal to the base material

1) Main phase of manufacture
2) Strength values for old weld materials to be applied where documents are unreliable and no additional tests are carried out
3) Designation of old weld materials: “Swedish charcoal iron” or “charcoal iron”
4) Key year, publication of the new SIA codes in 1956

4 CONNECTIONS

4.1 General

4.1.1 The following specifications relate to the verification of connections in existing structures as well as between existing and new structures.

4.1.2 If the choice of material and the detailing of connections take place in accordance with Code SIA 263, the examination of the connections shall be carried out in accordance with Code SIA 263.

4.1.3 An elastic resistance model shall be used for connections made of cast iron, since no plastic redistribution is possible.

4.1.4 The verification of connections shall be carried out with the updated resistance factor $\gamma_{M2,act} = \gamma_{M2} \cdot k_{\gamma M}$ in accordance with Section 5.1.1.

4.2 Riveted connections

4.2.1 General

4.2.1.1 After the rivet has been driven, the rivet hole is completely filled. For this reason it is not the diameter d of the raw rivet, but the diameter d_0 of the driven rivet, which is used in the specifications, on the plans and for the examinations.

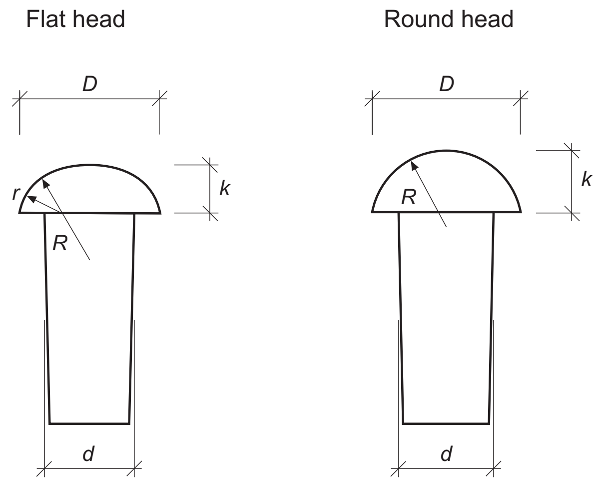
4.2.1.2 If the diameter d_0 of the rivet hole cannot be clearly determined, it may be determined with the following equation (1):

$$d_0 = d + 1 \text{ mm} \tag{1}$$

The diameter d_0 of the rivet hole corresponds to the diameter of the driven rivet.

4.2.1.3 If the diameter d of the raw rivet cannot be clearly determined with reference to the construction documents, it may be estimated on the basis of the measured diameter D, the measured height k and the measured radius R of the rivet head (Figure 1).

Figure 1: Characteristic dimensions of a raw rivet with flat or round head



4.2.1.4 In Table 6, the diameter d of the raw rivet is shown in relation to the measured diameter D, the measured height k and the measured radius R of the rivet head. Since the rivet stamp existed in different forms, the values stated in Table 6 are only indicative values. Essentially, rivets come with two different types of heads: rivets with flat heads and rivets with round heads (more recent).

Table 6: Estimation of the diameter d of the raw rivet

Rivet with flat head	Rivet with round head
$0{,}59D \leq d \leq 0{,}67D$	$0{,}60D \leq d \leq 0{,}63D$
$1{,}5k \leq d \leq 2{,}0k$	$1{,}56k \leq d \leq 1{,}82k$
$0{,}83R \leq d \leq 1{,}0R$	$1{,}11R \leq d \leq 1{,}22R$

4.2.1.5 After the rivet has been driven, the rivet hole is completely filled. A riveted connection may therefore be assumed to be an exact fit.

4.2.2 Ultimate resistance

4.2.2.1 In order to verify the structural safety of riveted connections, the examination values for the ultimate resistance for shearing, bearing stress and tensile stress in the rivet shaft shall be determined as follows.

Examination value for the shearing resistance per shear plane:

$$F_{V,Rd,act} = 0{,}6\ \frac{f_{ukB}\ \dfrac{\pi d_0^2}{4}}{\gamma_{M2,act}} \tag{2}$$

The examination value for the bearing stress resistance applies to the distances in accordance with Code SIA 263. If the distances e_2 and p_2 perpendicular to the direction of force fulfil the conditions $e_2 \geq 1{,}0\,d_0$ and $p_2 \geq 2{,}0\,d_0$:

$$F_{b,Rd,act} = 0{,}8\ \frac{e_1}{d_0}\ \frac{f_{uk}}{\gamma_{M2,act}}\ d_0 \cdot t \qquad \text{if } e_1 \leq 2\,d_0 \tag{3}$$

$$F_{b,Rd,act} = 1{,}6\ \frac{f_{uk}}{\gamma_{M2,act}}\ d_0 \cdot t \qquad \text{if } e_1 > 2\,d_0 \tag{4}$$

$$F_{b,Rd,act} = 0{,}8\ \frac{p_1 - \dfrac{d_0}{2}}{d_0}\ \frac{f_{uk}}{\gamma_{M2,act}}\ d_0 \cdot t \qquad \text{if } e_1 > 2\,d_0 \tag{5}$$

Examination value for tensile strength:

$$F_{t,Rd,act} = 0{,}24\ \frac{f_{ukB}\ \dfrac{\pi d_0^2}{4}}{\gamma_{M2,act}} \tag{6}$$

where: d_0 is the diameter of the rivet hole (= diameter of the driven rivet)

4.2.2.2 The examination value for the reduced resistance as a result of weakening of the rivet hole shall be determined in accordance with Code SIA 263 using the strength values in Table 2 and the updated resistance factor $\gamma_{M2,act}$.

4.2.2.3 The slip resistance resulting from the pretensioning of the rivet may not be taken into account in the examination.

4.2.2.4 If rivets resist a tensile stress, equation (6) shall be used for the examination value of the tensile strength. The residual stress state in the rivet shall not be taken into account.

4.2.2.5 If rivets are subjected to shear and tension, the interaction shall be verified in accordance with Code SIA 263.

4.3 Bolted connections

4.3.1 If it cannot be reliably determined whether the shear plane lies in the threaded area or in the shaft of a bolt, the examination value for the shear resistance per shear plane shall be limited in accordance with the value in equation (7).

$$F_{V,Rd,act} = 0{,}5 \ \frac{f_{ukB} \cdot A_s}{\gamma_{M2,act}} \qquad (7)$$

A_s cross-sectional area of the bolt in the threaded area

4.3.2 If the diameter of the bolt hole cannot be reliably determined, the examination value for the bearing stress resistance of connections that fulfil the structural detailing according to Code SIA 263 shall be determined in accordance with equation (8).

$$F_{b,Rd,act} = 0{,}85 \ \frac{e_1}{d} \ \frac{f_{uk}}{\gamma_{M2,act}} \ d \cdot t \qquad (8)$$

4.3.3 The examination value for the reduced resistance as a result of weakening from bolt holes shall be determined in accordance with Code SIA 263, using the yield strength values shown in Table 2 and the updated resistance factor $\gamma_{M2,act}$.

4.4 Welded connections

4.4.1 The examination value for the ultimate resistance of fillet welds shall be determined in accordance with Code SIA 263 using the material properties shown in Tables 2 and 5, as well as the updated resistance factor $\gamma_{M2,act}$.

4.4.2 A fillet weld may be strengthened by adding weld layers in order to achieve the necessary dimension. Where an old weld has been strengthened, the examination value for the ultimate resistance of the strengthened fillet weld shall be determined using the material properties of the old weld.

4.4.3 When examining the structural safety, an existing weld may only be regarded as a fully penetrated weld if a corresponding non-destructive test has been carried out during the condition survey, or if reliable execution plans are available. Otherwise, the weld shall be assumed to be a fillet weld with the dimensions measured on the structure.

4.4.4 In the case of fully penetrated welds carried out after 1956, verification in the base material in accordance with Code SIA 263 is sufficient. In the case of fully penetrated welds carried out before 1956, the examination value for the ultimate resistance may be determined using the material properties shown in Table 5 and the updated resistance factor $\gamma_{M2,act}$.

5 STRUCTURAL ANALYSIS AND VERIFICATIONS

5.1 General

5.1.1 According to equation (9), the updated resistance factor $\gamma_{M,act}$ is:

$$\gamma_{M,act} = \gamma_M \cdot k_{\gamma M} \quad (9)$$

The resistance factor γ_M is defined in Code SIA 263. Equation (9) is valid for γ_{M1} and γ_{M2}.

The correction coefficient $k_{\gamma M}$ takes into account typical deviations in the case of cast iron and old structural steels. The value of the correction coefficient is shown in relation to the material and its year of manufacture in Table 7.

Table 7: Correction coefficient $k_{\gamma M}$ in relation to the material and its year of manufacture

Material	Period of use [1]	Correction coefficient $k_{\gamma M}$
Cast iron	vor 1900	1,15
Wrought iron	1850–1900	1,10
Mild rimmed iron	1890–1900	1,10
	1900–1940	1,05
Mild steel	1925–1955	1,05
Steel for bolts	1920–1955	1,05
Material of old welds	1900–1924	1,50 [2]
	1925-1955	1,35 [2]
Modern structural steel (base material and welds)	from 1956 [3]	1,00

[1] Main phase of use and manufacture of the material
[2] A reduction in these values is only permitted if this is justified by non-destructive or destructive tests
[3] Key year, publication of the new SIA codes in 1956

5.1.2 In general, the following values may be assumed for the examination value of a geometrical property $a_{d,act}$:
- in the case of a material loss ≤ 5% of the nominal value of the geometrical property: $a_{d,act} = a_d$
- in the case of a material loss > 5% of the nominal value of the geometrical property, the variable has to be replaced by the effective value.

5.2 Basics and modelling

5.2.1 The verification methods in accordance with Code SIA 263 (PP, EP, EE, EER) may be used for modern structural steels. In these cases, Code SIA 263 shall be used as the basis for the structural analysis, modelling and verifications.

5.2.2 The permissible verification methods for the old structural steels are shown in Table 8.

Table 8: Permissible verifiaction methods in relation to the materials

Material	Period of use [1]	Method [2]
Cast iron	before 1900	EE, EER
Wrought iron [3]	1850–1900	EP [4], EE, EER
Mild rimmed iron	1890–1900	EP, EE, EER
	1900–1940	EP, EE, EER
Mild steel	from 1925	PP, EP, EE, EER

[1] Main phase of manufacture
[2] See code SIA 263
[3] Parallel to direction of rolling, see Section 3.2.2.2
[4] See Section 5.3.4.2

5.2.3 If values other than those in Table 2 are used, the applicable verifiaction methods shall be determined in accordance with the criteria of Code SIA 263.

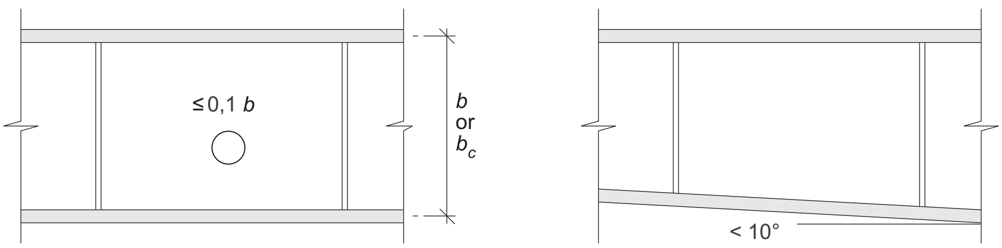
5.2.4 The permissible verifiaction methods in Table 8 shall be selected in accordance with the specifications of Code SIA 263 taking into consideration the cross-sectional classes. In defining the cross-sectional class, the reduction factor to take into account the influence of the yield strength in accordance with equation (10) shall be updated.

$$\varepsilon_{act} = \sqrt{\frac{E_k}{210\,000} \cdot \frac{235}{f_{yk}} \cdot \frac{\gamma_{M1,act}}{1{,}05}} \qquad (10)$$

f_{yk} characteristic value for yield strength
$\gamma_{M1,act}$ updated resistance factor (in accordance with Section 5.1.1)

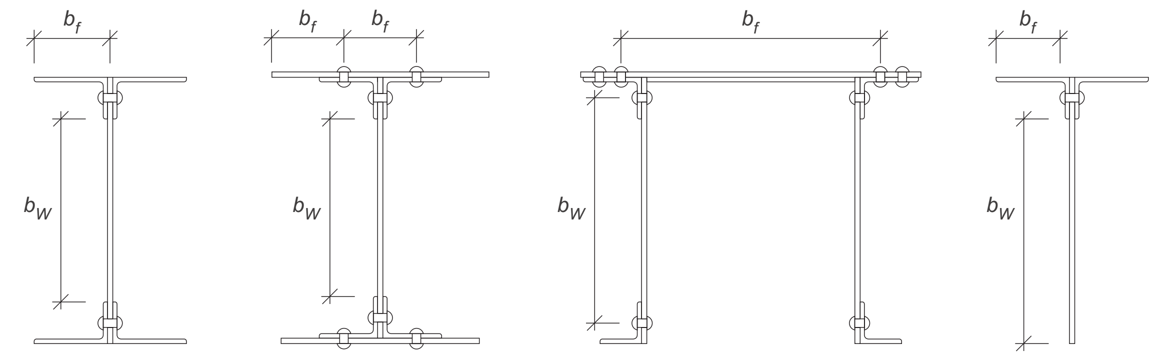
5.2.5 The resistance model based on effective widths in accordance with Code SIA 263 and the model based on the theory of local buckling in accordance with Section 5.4.4 may both only be applied if the two following conditions are met (see Figure 2):
– the diameter of the unstiffened structural opening is not greater than 0,1 *b*, where *b* is the width of the compression or sheared element
– the panels are arranged rectangularly or trapezoidally with an angle between the two non-parallel sides of no more than 10°.

Figure 2: Geometrical conditions for the resistance models (based on effective widths) and for the theory of local buckling



5.2.6 The width b_f or b_w of the buckling field that is to be considered for riveted structural members is defined in Figure 3. For panels in riveted structural members in which at least one of the edges parallel to the direction of loading is supported, the support by riveted steel plates or angles may be taken into account if they are riveted on both sides of the panel.

Figure 3: Definition of the width of the buckling field for different riveted structural members



5.2.7 The influences of member eccentricities at the nodes shall be evaluated in each case. If necessary, the eccentricities shall be taken into consideration in the structural model using the updated values.

5.2.8 Lattice girders made of old and modern structural steel may be modelled using pin-jointed members. In the case of cast iron, the stresses determined in the pin-jointed system should be increased by a factor of 1.2. This accounts globally for the action effects due to the partial fixity of the members at their ends, that are in general secondary bending stresses.

5.2.9 For the examination with regard to fatigue, a detailed structural model with the geometry of the construction details that are to be examined is only recommended if the construction detail cannot be assigned to one of the notch categories listed in Appendix B or in Code SIA 263. The results determined on the basis of the detailed structural model may only be used if the fatigue verification is carried out using the geometric (hot-spot) stress method in accordance with Code SN EN 1993-1-9.

5.3 Cross-sectional resistances

5.3.1 Normal force

5.3.1.1 If buckling and lateral-torsional buckling are not prevented, the examination value for the normal force resistance $N_{Rd,act}$ shall be calculated in accordance with Sections 5.4.1 and 5.4.2.

5.3.1.2 In the case of riveted structural members subjected to tension, the net cross-section is to be used for the calculation of the examination value for the normal force resistance.

5.3.2 Bending

In the case of riveted beams subjected to bending loads, the examination value for the bending resistance $M_{Rd,act}$ in the tensile zone shall be calculated with the net cross-section (cross-section minus rivet holes) and the values f_{yk} and $\gamma_{M1,act}$.

5.3.3 Shear force

5.3.3.1 In the case of old structural steel, if the web slenderness meets the constraint in equation (11), the examination value for the shear force resistance $V_{Rd,act}$ can be determined in accordance with Section 5.3.3.2:

$$\frac{b}{t_w} \leq \sqrt{\frac{4\,E_k}{f_{yk}}} \tag{11}$$

with b according to Section 5.2.6.

5.3.3.2 For all steels, the examination value for the shear force resistance $V_{Rd,act}$ of a structural member that is not in danger of local buckling under shear can be determined as follows (see Figure 4):

– in general using equation (12):

$$V_{Rd,act} = \frac{\tau_{yk} \cdot I \cdot \Sigma t_i}{\frac{W_{pl}}{2} \cdot \gamma_{M1,act}} \tag{12}$$

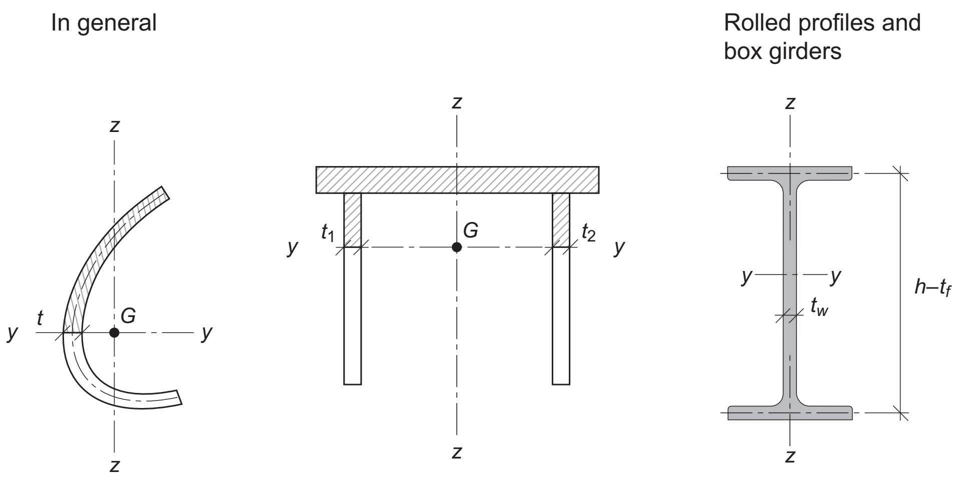
Σt_i sum of widths at the height of the neutral axis

– for double-symmetrical rolled I-profiles and box girders using equation (13):

$$V_{Rd,act} = \frac{\tau_{yk} \cdot A_w}{\gamma_{M1,act}} \tag{13}$$

A_w web surface area $(h–t_f) \cdot t_w$ or web surface areas of box girders

Figure 4: Dimensions for calculation of the shear resistance



5.3.3.3 If the web slenderness in accordance with Section 5.3.3.1 cannot be met and the structural member is in danger of local buckling in shear, the specifications under Section 5.4.4 shall be taken into consideration.

5.3.4 Bending and shear force or torsion

5.3.4.1 If the examination value for the shear resistance is determined in accordance with equation (13), the interaction of bending and shear force may be disregarded. However, if the area A_v is used instead of A_w, the interaction shall be taken into account in accordance with Code SIA 263.

5.3.4.2 In the case of wrought iron, a yield criterion, for example, the von Mises criterion, shall be applied for the interaction between normal and shear stresses.

5.3.5 Bending and normal force

The examination of bending with simultaneously acting normal force shall be carried out on the basis of Code SIA 263 as well as the references contained therein. In addition, in the case of riveted cross-sections, Section 5.3.2 of this code and the resulting eccentricity are to be taken into account.

5.4 Stability

5.4.1 Buckling

5.4.1.1 In general, the gross cross-section of the structural member is used for the examination value for buckling resistance. However, if the rivet and bolt holes are located within the critical cross-section (maximum internal section forces) and reduce the cross-section by more than 15%, the cross-sectional properties shall be calculated taking into consideration Section 5.3.2. The relative slenderness $\bar{\lambda}_K$ shall, however, always be determined with the gross cross-section.

5.4.1.2 The relative buckling slenderness $\bar{\lambda}_K$ is to be calculated using the values E_k and f_{yk} in accordance with Section 3.2.2.

The buckling curves according to Code SIA 263 for old structural members and profiles are shown in Table 9.

Table 9: Assignment of structural members and profiles with the buckling curves

Structural members and profiles	Buckling curve or reduction factor for buckling
Rolled and welded structural members made of old or modern structural steel	According to Code SIA 263
Riveted web members or bolted structural members made of old or modern structural steel	– a: for symmetrical cross-sections (α_K = 0,21) – b: for all other cross-sections (α_K = 0,34)
Structural members made of cast iron	$\chi_K = 1/(1+0{,}0007 \cdot \bar{\lambda}_K^2)$

5.4.1.3 If the straightness of members subjected to buckling does not meet the geometrical tolerances of Code SIA 263/1, as an alternative to the use of buckling curves the verification may be carried out in accordance with Section 5.4.3.2 with $M_{Ed} = 0$ and $M_{Rd,act} = M_{él,Rd,act}$.

5.4.1.4 In the case of riveted lattice members, the buckling lengths shown in Table 10 may be assumed if the prerequisites in accordance with Code SIA 263 are fulfilled.

Table 10: Reduced buckling lengths for riveted lattice members

Structural member	Buckling lengths L_K	
	in-plane	out-of-plane
Chords	0,8 L	in accordance with Code SIA 263
Diagonals: – single – double	 0,75 L 0,4 L	
Simple posts	0,75 L	

5.4.2 Lateral-torsional buckling of beams

5.4.2.1 Buckling curve b (α_D = 0,34) shall be used for riveted beams.

5.4.2.2 The lateral-torsional buckling verification of riveted beams shall be carried out taking into consideration the specifications in Section 5.3.2.

5.4.3 **Bending and compressive force**

5.4.3.1 In the case of members with constant cross-sections subjected to uniaxial or biaxial bending and compressive forces, the verification may be carried out in accordance with Code SIA 263; the same applies to doubly-symmetrical I-profiles of the cross-section classes 1 and 2. The buckling curve shall be selected in accordance with Section 5.4.1.2. The updated geometry is to meet the geometrical tolerances in accordance with Code SIA 263/1.

5.4.3.2 In the case of members with constant cross-sections subjected to bending around a main axis and a compressive force, and for which out-of-plane buckling as well as lateral-torsional buckling are prevented, the examination of stability may be carried out in accordance with equation (14). This equation can also be used if the straightness of the structural member does not meet the geometrical tolerances of Code SIA 263/1.

$$\frac{N_{Ed,act}}{N_{pl,Rd,act}} + \frac{1}{1-\dfrac{N_{Ed,act}}{N_{cr}}} \; \frac{\omega \cdot M_{Ed,act} + N_{Ed,act} \cdot e_0}{M_{Rd,act}} \leq 1{,}0 \qquad (14)$$

ω factor for taking into account the moment distribution, in accordance with Code SIA 263
e_0 equivalent initial imperfection, $e_0 = e_{0,geom} + e_{0,\sigma}$
$e_{0,geom}$ deflection between the member ends, measured on the structure
$e_{0,\sigma}$ equivalent eccentricity to take into account the action effect of the residual stresses on the buckling stability of the member:
– buckling curve a: $e_{0,\sigma} = L_k/500$
– buckling curve b: $e_{0,\sigma} = L_k/430$
– buckling curve c: $e_{0,\sigma} = L_k/330$
– buckling curve d: $e_{0,\sigma} = L_k/250$

5.4.3.3 For structural members made of cast iron and due to the lack of material ductility, the verification in accordance with Section 5.4.3.2 shall be carried out with elastic ultimate resistances, taking into consideration an equivalent eccentricity $e_{0,\sigma} = L_k/200$.

5.4.3.4 When verifying bending with compressive force, the specifications in Section 5.4.1.1 shall be taken into account.

5.4.4 **Buckling of plate-type elements**

5.4.4.1 The resistance to local buckling of built-up, thin-walled plates as well as their support shall be verified.

5.4.4.2 If more favourable conditions cannot be proven, simply-supported boundaries shall be assumed for plate-type elements of riveted girders loaded in compression or shear. The width b_f or b_w of a buckling field is defined in Section 5.2.6.

5.4.4.3 In the case of shear loading of structural members made of wrought iron, the post-buckling behaviour may not taken into account (anisotropy too great). For the other steels, a post-buckling calculation in accordance with Code SIA 263 using the updated resistance factor $\gamma_{M1,act}$ is permissible.

5.5 Fatigue

5.5.1 **General**

5.5.1.1 The resistance of a steel structural member with a fatigue crack shall in principle be calculated using fracture mechanics. However, in the case of thermo-mechanical steels continuum mechanics (calculation using net cross-section) may be used.

5.5.1.2 Residual stresses can be neglected in a riveted structure. Therefore, for the fatigue verification, the stress range resulting from tensile and compressive stresses may be reduced in accordance with the specifications in Code SIA 263.

5.5.1.3 In the case of riveted structural members, the stress range shall be determined on the net cross-section.

5.5.2 Fatigue resistance

5.5.2.1 Generally, the notch categories specified in Code SIA 263 also apply to existing structures provided the requirements with respect to tolerances and controls specified in Code SIA 263 are fulfilled.

5.5.2.2 In general, all riveted construction details can be assigned to the notch category $\Delta\sigma_C$ = 71 N/mm^2 with a fatigue strength curve gradient of m = 5.

5.5.2.3 The notch categories of different riveted construction details are shown in Table 13 in Appendix B. The notch categories correspond to fatigue strength curves with a gradient of m = 5 in accordance with Figure 5 in Appendix B.

5.5.2.4 The updated resistance factor for fatigue resistance $\gamma_{Mf,act}$ shall be determined according to accessibility for inspection and the level of consequence of failure to be expected during the remaining service life. For the general examination, the updated resistance factors shall be taken from Table 11.

Table 11: Updated resistance factor $\gamma_{Mf,act}$ for fatigue resistance for general examination

	Low consequence [1]	High consequence [2]
Inspected construction detail without fatigue damage	1,0	1,0
Construction detail without possibility of inspection	1,15	1,35 and see Section 7.5.1.3

[1] Corresponds to the consequence classes CC1 and CC2 in accordance with EN 1990
[2] Corresponds to the consequence class CC3 in accordance with EN 1990

5.5.3 Fatigue verification

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

5.5.3.2 In the damage accumulation calculation, the previous damage as a result of traffic may be calculated with an updated resistance factor $\gamma_{Mf,act}$ = 1,0.

5.5.4 Estimation of the interval between interim inspections

If cracks are present or suspected, or if critical locations are identified on the basis of the fatigue calculations, the interval between inspections shall be defined in agreement with the client. The interval between interim inspections is determined on the basis of a cost-benefit analysis taking into consideration the structural safety of the structure. Intervals between interim inspections of 6 months to 3 years are recommended.

6 CONDITION SURVEY

6.1 General

6.1.1 The compliance of the executed structure with the plans shall be examined carefully. In particular, it shall be checked whether all bolts, rivets and welds are present in the connections. All structural members subsequently attached by means of bolted connections or welding are to be identified.

6.1.2 The condition survey of connections subjected to fatigue loading as well as deterioration due to fatigue shall be carried out appropriately and with particular care.

6.1.3 When surveying deterioration, a distinction is made between the initiation phase and the development phase of the deterioration (as a result of corrosion, fatigue, …), insofar as this is possible using the test methods for steel structures.

6.1.4 In general, structures made of cast iron and wrought iron are classed as being non-ductile unless it can be proved through material tests that the ultimate strain ε_{uk} reaches a minimum value of 15% and the material can be assigned to at least the quality group JR in accordance with EN 10 025 (minimum Charpy V-notch impact value of K_V = 27J at a temperature of T_{27J} = 20 °C).

6.2 Defects

6.2.1 The following defects, among others, can occur in steel structures:
- accumulation of water, obstructed water run-off or possible accumulation of waste and dirt
- missing structural members or stabilising elements
- missing rivets, bolts or welds in connections
- missing stiffeners or poor structural design of stiffeners
- obstructed access to individual areas of the structure or structural members
- inadequately applied corrosion protection (not complying with SN EN 12944-8, e.g. an unapproved surface protection, improperly executed repairs)
- lack of or insufficient protection against accidental actions (e.g. impact, fire)
- imperfections in manufacture or installation (e.g. eccentricities, deflection in members, misalignment)
- unfavourable flow of forces (e.g. in stabilising elements).

6.2.2 Welds can display both external defects (e.g. welds with too little or too much weld material, offset edges, asymmetrical fillet welds) as well as internal defects (e.g. lack of penetration, lack of fusion, cracks, inclusions, pores) that can be detected as follows:
- the external defects are generally identified through visual checks.
- the internal defects are generally identified using specific test methods. The scope of testing and test methods to be used depend on the level to which the weld is loaded and on the consequences in the event of failure.

6.2.3 During the course of a detailed examination, in order to survey the condition of the welds, the weld quality level in accordance with Code SIA 263 shall first be determined in order to define the acceptance criteria for the defects that are identified.

6.2.4 If vibrations can lead to fatigue damage, a quantitative evaluation of the vibration behaviour is to be carried out.

6.3 Deteriorations

6.3.1 The following types of deterioration, among others, can occur in steel structures:
- material loss due to corrosion or wear
- swellings between steel plates or structural members
- cracking, in particular at locations with geometrical discontinuities (stress concentrations), in connections and in fasteners
- cracking as a result of imposed repeated displacements
- unforeseen deformations in connections, loose nuts, slip in pretensioned connections
- broken fasteners
- permanent deformations in structural members (instability, plastification, etc.), especially following impact, earthquake or fire
- unfavourable changes in the mechanical properties of the steel as a result of fire
- cracking as a result of impact or earthquakes, even without permanent deformations
- deterioration of the coating system (corrosion protection, fire protection).

6.3.2 The size of a crack in a structural member or in a connection is recorded at least by measuring the length of the crack on the surface.

6.3.3 Cracks can be located and measured (length and/or depth of crack) using the MT, UT or PT test methods in accordance with Code SIA 263/1. The test methods MT and UT are preferred. When using the PT test method, the corrosion protection has to be removed first. During removal, it has to be ensured not to close again cracks on the surface.

6.3.4 In the case of weather resistant steels (with increased corrosion resistance), the uniform material loss can be evaluated by observing the material loss and its development. In the case of corrosion or non-uniform development of the material loss, the material losses in the most severely affected areas are recorded by means of measurements of plate thickness on a representative sample.

6.3.5 Deterioration of coating systems is evaluated in accordance with SN ISO 4628.

7 INTERVENTIONS

7.1 General

7.1.1 Monitoring

The monitoring is to primarily be carried out through visual checks on the structure. Defects in accordance with Section 6.2 and deterioration signs in accordance with Section 6.3 shall be checked especially carefully.

7.1.2 Rehabilitation and modification

7.1.2.1 If structural members or connections of the structure are not accessible without special measures and their failure would result in serious consequences, at least one of the following measures is to be implemented:
- more intensive monitoring of the entire structure or of an area of the structure as a supplementary safety measure
- creation of access to the structural member or connection for the purpose of monitoring
- improvement in the redundancy of the structure
- strengthening or replacement of structural members.

7.1.2.2 If new connections are made on old structural steel, bolted connections are to be preferred to welded connections.

7.1.2.3 The suitability of old structural steel for welding is limited due to inclusions, impurities, excessively high carbon equivalent or due to insufficient fracture toughness. For this reason, welding on old structural steel shall be avoided in general, with the exception of mild steel. Welding is prohibited in the case of constructions subjected to fatigue loading.

7.1.2.4 The welding work shall be carried out in accordance with a Welding Procedure Specification (WPS). The welding method (Welding Procedure Approval Record – WPAR) is to be carried out according to the specifications of Code SIA 263/1.

7.2 Riveted connections

7.2.1 Monitoring and maintenance

7.2.1.1 Traces of rust between the rivet head and the plate or between riveted steel plates are indications that the pretensioning of the rivet and thus the resulting frictional force is no longer present.

7.2.1.2 When testing the rivet for pretensioning, care should be taken not to damage the corrosion protection. Striking the rivet heads shall be avoided.

7.2.2 Rehabilitation and modification

7.2.2.1 Defective rivets are generally replaced with high-strength pretensioned bolts. If further rivets are loosened through the pretensioning of the bolts, these also shall be replaced.

7.2.2.2 Hybrid connections between rivets and welds shall generally be avoided due to their different structural behaviour.

7.3 Bolted connections

7.3.1 Monitoring and maintenance

The monitoring of bolted connections shall be carried out by means of visual inspections. If loose bolts are suspected, they have to be checked on the basis of a representative random sample whether the bolts are still tightened.

7.3.2 Rehabilitation and modification

7.3.2.1 Loosened structural steel bolts may be reused, but need to be tightened. In contrast to this, high-strength pretensioned bolts shall be replaced with equivalent new pretensioned bolts.

7.3.2.2 In order to prevent loosening under dynamic actions, high-strength bolts and high-strength fitted bolts need to be pretensioned. The pretensioning also ensures an even contact between the structural members and can thus reduce deterioration as a result of penetrating moisture.

7.4 Welded connections

7.4.1 Monitoring and maintenance

The monitoring of welded connections shall be carried out by means of visual inspections. There are no typical kinds of maintenance work on welds. In the case of deterioration, there shall always be carried out a rehabilitation in accordance with Section 7.4.2.

7.4.2 Rehabilitation and modification

7.4.2.1 In the case of welding, the suitability of the material for welding, as well as Sections 7.1.2.3 and 7.1.2.4 shall be considered.

7.4.2.2 Cracks shall generally never be rehabilitated by welding. Before rehabilitation is carried out, the cause of the cracking shall first be established and the corresponding measures shall then be implemented.

7.4.2.3 In the case of welding on loaded structural members, the sequence of welding shall be defined in such a way that the heating of the steel in the vicinity of the weld is limited to an indicative value of 120 °C. This prevents a reduction in the modulus of elasticity and in the strength of the steel during the welding process. At the same time, the weld may only shrink to a limited extent during cooling. Welding on structural members subjected to dynamic action is prohibited.

7.5 Construction details subjected to fatigue

7.5.1 Monitoring and maintenance

7.5.1.1 Structures subjected to fatigue shall be carefully examined for fatigue cracks, in particular in the case of construction details with abrupt changes in the cross-sectional dimensions and at the location of welds. For connections see Section 7.2.1., 7.3.1 and 7.4.1.

7.5.1.2 If cracks are suspected or identified in a structure, interim inspections in accordance with Section 5.5.4 shall be carried out. Interventions are essential if the cracks run perpendicular to the principal tensile stresses.

7.5.1.3 In the case of construction details subjected to fatigue in accordance with Section 7.1.2.1, first of all accessibility is to provide for the purpose of monitoring. If accessibility is not possible, but monitoring is intensified the updated resistance factor may be reduced to $\gamma_{Mf,act}$ = 1,25.

7.5.2 Rehabilitation and modification

7.5.2.1 In the case of detected fatigue cracks, the following interventions can be carried out:
- drilling holes with a diameter of at least d_0 = 20 mm at the ends of the cracks
- addition of reinforcing splices with pretensioned bolted connections.

7.5.2.2 Damaged rivets in structural members subjected to fatigue are to be bored out and replaced with pretensioned high-strength fitted bolts of strength class 10.9.

7.5.2.3 Reinforcement or modification using old structural steel on a connection subjected to fatigue shall be evaluated in detail taking into consideration Section 7.1.2.3.

7.6 Corrosion protection

7.6.1 Monitoring and maintenance

7.6.1.1 The corrosion protection is to be maintained during inspections in order to defer for as long as possible the time for renewal.

7.6.1.2 During maintenance of the structure, the corrosion protection shall be restored on small defective surface areas by applying suitable protection systems.

7.6.1.3 The surface of the damaged locations is to be prepared manually or using power tools in order to ensure adequate bonding in accordance with the standard ISO 8504-3.

7.6.2 Rehabilitation and renewal

7.6.2.1 Compatibility and bonding between new and existing corrosion protection shall be ensured. For this purpose, the properties of the existing protection layer are to be examined and both the method as well as the products used for the rehabilitation shall be chosen carefully.

7.6.2.2 Due to the overall thickness of the protective coating and a possible embrittlement of the old base and intermediate coatings, the number of successive rehabilitations of the top coat has to be limited.

7.6.2.3 Renewal of the corrosion protection is necessary if:
- the degree of weathering of the corrosion protection and the degree of rusting of the steel are very pronounced. The extent of the deterioration is evaluated in accordance with SN ISO 4628,
- the protective effect of the corrosion protection is insufficient,
- the base coating is no longer bonded.

7.6.2.4 If renewing the corrosion protection, the entire existing coating shall be removed. In general, the surface preparation for the renewal of the corrosion protection is carried out by means of blasting using one of the methods defined in the standard ISO 8504-2.

7.6.2.5 When carrying out rehabilitation or renewal of the corrosion protection, the joints that cannot be blasted shall be cleaned appropriately and sealed with a suitable sealing compound.

APPENDIX A (normative)

DESIGNATION OF CAST IRONS AND STRUCTURAL STEELS

Table 12: Designation of cast irons and structural steels

Cast iron *Gusseisen* *fonte (de fer)* *ghisa*		Alloy with iron as its main component, with a carbon content > 2% and containing between 1% and 4% of other elements. Raw material obtained directly from iron ore in the blast furnace, then refined for partial decarbonisation and removal of other elements in order to make the cast iron workable. Used to manufacture cast parts, since the melting point is lower than that of steel. Until around 1900 cast iron was produced using lamellar graphite, not weldable, brittle, very hard. After 1900 cast iron was produced using nodular graphite.
Old structural steel *alter Baustahl* *acier de construction ancien* *acciaio da costruzione obsoleto*	Wrought iron, puddle iron *Schweisseisen, Puddeleisen* *fer puddlé* *ferro puddellato*	Iron alloy, malleable, difficult to temper, difficult to weld (WPAR necessary), produced in the semi-molten state through decarbonisation of the raw iron by means of vigorous stirring (“puddling”). Produced and used in civil engineering in the second half of the 19th century. Carbon content between 0,1% and 0,5%, anisotropic in through-thickness direction with pure iron layers and inclusions of slag, good mechanical properties only in a longitudinal direction. Sometimes incorrectly referred to as “wrought steel”.
	Mild rimmed iron *Flusseisen* *fer homogène* *ferro omogeneo*	Iron alloy, malleable, not temperable, produced in the molten state in block form, designated according to the Bessemer, Thomas or Martin method of production. Produced and used in civil engineering between 1890 and 1900 (Bessemer), and 1900–1940 (Bessemer, Thomas, Martin). Carbon content between 0,05% and 0,25%.
	Mild steel *Flussstahl* *acier doux* *acciaio dolce*	Iron alloy, similar to mild rimmed iron, but temperable. Carbon content between 0,25% and 1,6%. Used from around 1925.
Modern structural steel [1)] *neuer Baustahl* *acier de construction récent* *acciaio da costruzione attuale*		Iron alloy with a low carbon content of 0,008% to approx. 1,6% (but always < 2%). In terms of this code, a structural steel is classed as being modern if it was produced after 1956, since a complete new series of SIA codes was published in this year. However, such structural steels were already in some cases in production between 1940 and 1950. Modern structural steels are produced either from iron ore or from iron or steel scrap (electric furnace steel). Properties in accordance with Code SIA 263.

[1)] as opposed to old structural steel

APPENDIX B (normative)

FATIGUE NOTCH CATEGORIES FOR RIVETED CONSTRUCTION DETAILS

The fatigue strength curves for riveted construction details are shown in Figure 5. These curves supplement the curves in Code SIA 263.

Figure 5: Fatigue strengths according to the notch categories for riveted construction details

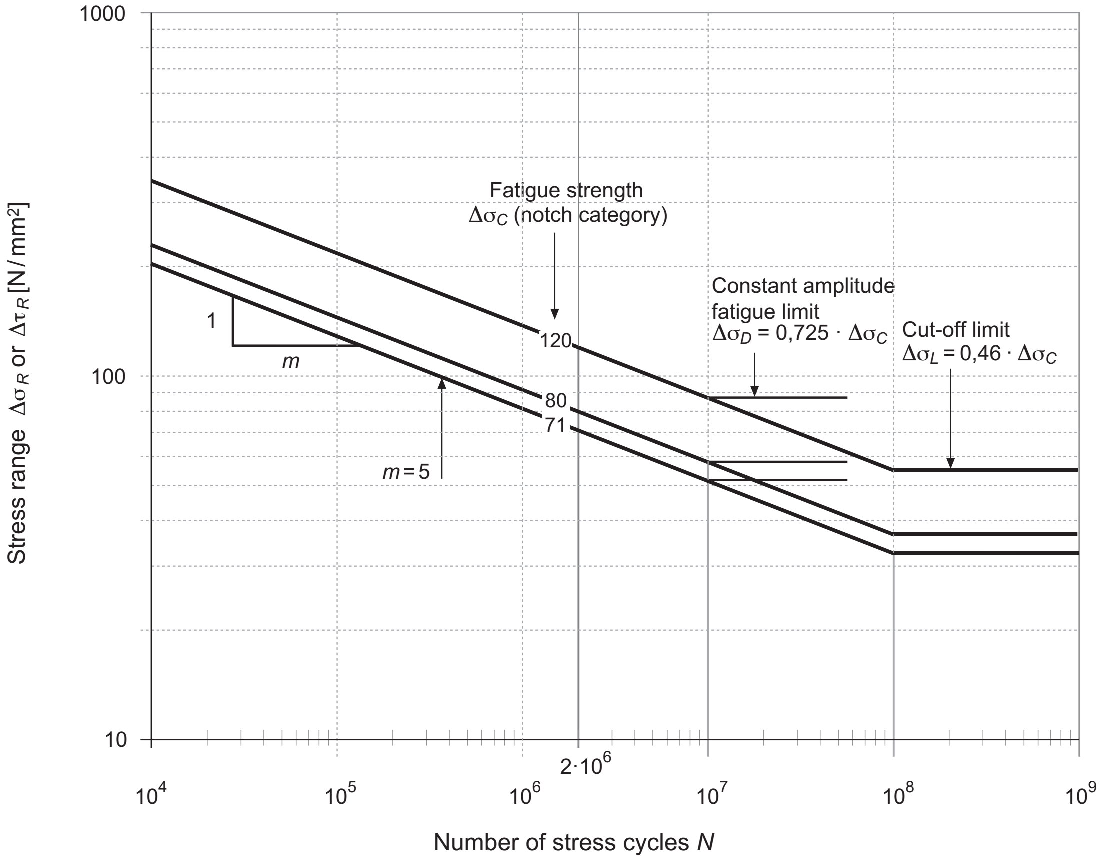
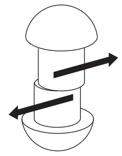
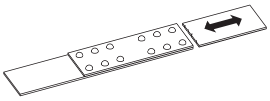
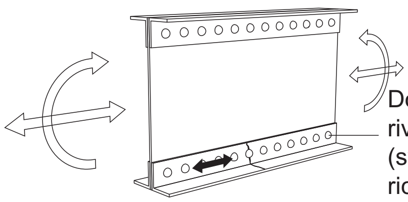
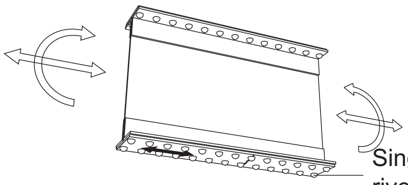
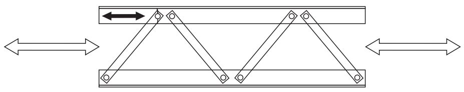
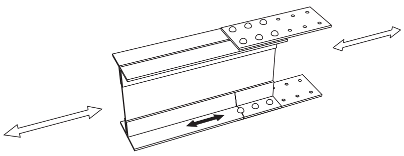
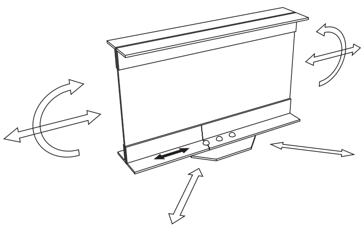
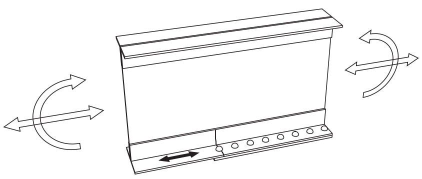


Table 13: Riveted construction details

Notch category [N/mm^2]	Construction detail	Description
$\Delta\tau_c$ = **120**	 **1**	Rivets subjected to shearing forces in single- or multiple-plane shear connections.
$\Delta\sigma_c$ = **80**	 **2**	Butt joint between steel plates spliced on both sides.
$\Delta\sigma_c$ = **80**	 **3**	Continuous riveting between flange angles and web plate in built-up flexural girders. $\Delta\sigma_{E2}$ at rivet level in the web.
$\Delta\sigma_c$ = **80**	 **4**	Continuous riveting between flange angles and web plate in built-up flexural girders. $\Delta\sigma_{E2}$ at rivet position in the flange.
$\Delta\sigma_c$ = **80**	 **5**	Built-up lattice members subjected to tension or compression.
$\Delta\sigma_c$ = **71**	 **6**	Butt joint between steel plates spliced on one side.
$\Delta\sigma_c$ = **71**	 **7**	Connection area of a bracing splice to the tension flange of a flexural girder.
$\Delta\sigma_c$ = **71**	 **8**	Area of the end anchorage of a reinforcing plate.

Abbreviations of organisations represented in the commission SIA 263

EPFL	Swiss Federal Institute of Technology, Lausanne
ETH Zürich	Swiss Federal Institute of Technology, Zurich
SBB	Swiss Federal Railways
SZS	Swiss Institute of Steel Construction

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Approval and validity

The SIA’s central committee for codes and regulations approved the present Code SIA 269/3 on 23rd November 2010.

It is valid as from 1st January 2011.