Structural Engineering Document

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Use and Application of High Performance Steels (HPS) for Steel Structures

Extract, chapter 5.2

International Association for Bridge and Structural Engineering IABSE
Association Internationale des Ponts et Charpentes AIPC
Internationale Vereinigung für Brückenbau und Hochbau IVBH

5.1.5 Conclusions

Extra high strength steels with yield strength above 355 MPa up to 690 MPa are well suited for steel construction purposes. Their fabrication properties are similar to those of ordinary steels. Thermomechanically rolled steels in particular have a very lean composition giving high toughness and superior welding properties. Quenched and tempered steels allow higher strengths to be exploited while still having good toughness and good weldability.

Using high performance steel allows less material to be used. It reduces the volume of weld metal and thus the time for welding and also the areas to be painted if needed. Less material has to be transported and reduced weight simplifies the erection of structures. All this is of great benefit in high wage economies. The total impact on environment is reduced and the high strength offers new opportunities to designers.

5.2 Toughness requirements in structural applications

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5.2.1 General

In this section, the background of modern toughness requirements as given in the Eurocodes, EN 1993-1-10 will be given. Examples of application to high performance steels will be shown. A discussion on the limiting value of yield to strength ratio as given in most codes will be made.

Quantitative toughness properties of steel in general are determined by standardized J-integral or CTOD tests. Fig. 5.2.1 shows one of the standardised test specimens often used.

The toughness properties vary with temperature. Fig. 5.2.2 gives the function of the toughness-temperature dependency for ferritic steels, for which the following regions are distinguished:

1. lower shelf region, where the load-deformation characteristic of test pieces in tension show brittle behaviour and linear elastic fracture mechanics may be used featuring stress intensity factors K_{IC} as toughness values,
2. upper shelf region, where the load-deformation characteristic of test pieces in tension show full ductile behaviour and non linear elastic plastic fracture mechanics applies,
3. transition region with partial plastic deformations where modified linear elastic fracture mechanics may be used and the temperatures T_{gy} signifies the point where general yield in a net-section (e.g. for a plate with bolt holes) occurs before fracture.

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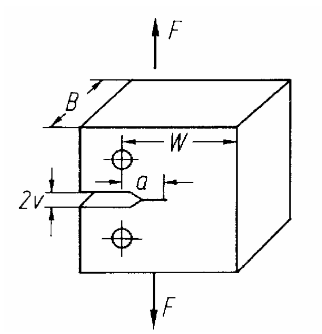

Fig. 5.2.1: CT-specimen for determining J-, CTOD- or K-values

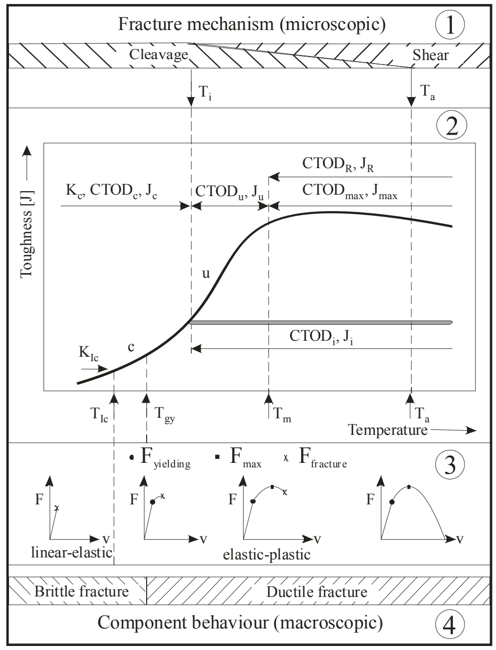

Fig. 5.2.2: Toughness-temperature-curve and related load-deformation curves for tension elements using various parameters for toughness properties [5.4]

The design rules for achieving sufficient mechanical resistance and stability of structural components and structures are based on continuum mechanics and tests that are carried out in laboratories at room temperature. The assumption behind the design rules is that upper shelf toughness behaviour and ductile stress-strain-behaviour govern the performance of test pieces, see Fig. 5.2.3. Therefore it is necessary to avoid brittle fracture by an appropriate choice of material in view of toughness.

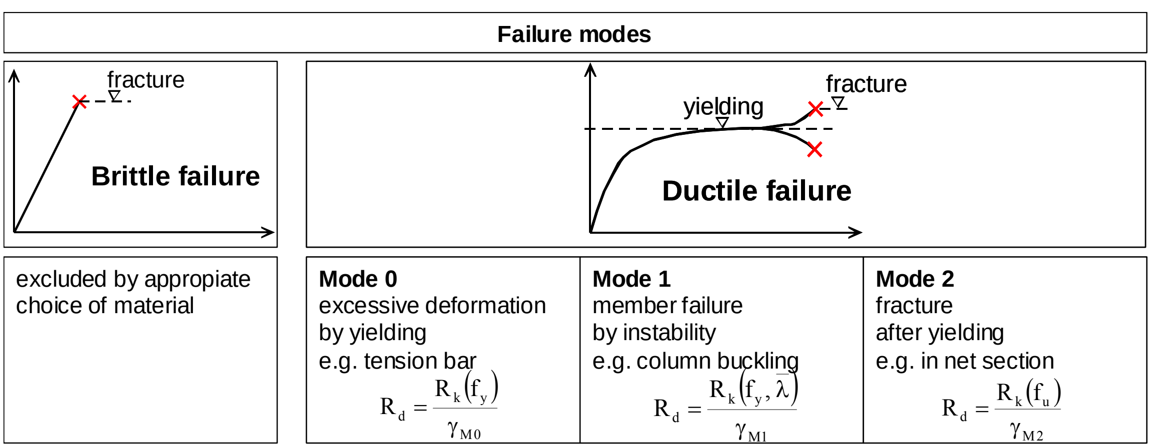

Fig. 5.2.3: Ductile and brittle failure modes for structural design

Such choices are based on toughness related safety checks carried out in the transition region of the toughness-temperature diagram.

5.2.2 Background of Fracture Mechanical Safety Assessment to Avoid Brittle Fracture

5.2.2.1 Introduction

In the following the fracture mechanical safety assessment to avoid brittle fracture is presented as it is standardized in Eurocode 3, Part 1-10 (EN 1993-1-10) "Material toughness and through thickness properties" [5.5]. More information can be found in the background document to EN 1993-1-10 [5.8].

The verification is performed by comparing K-values (stress intensity factors) from, on one side, design values of fracture mechanical action effects $K^*_{appl,d}$ and, on the other side, design values of fracture mechanical resistance $K^*_{mat,d}$, see Equation (1).

$$K^*_{appl,d} \le K^*_{mat,d} \tag{1}$$

The design values are chosen from statistical distributions in such a way, that the reliability required for ultimate limit state assessments is achieved.

The verification is based on the following conservative assumptions:

1 the structural component has a crack-like flaw at the point of maximum stress concentration (hot spot) with the size a_d (e.g. design value of crack depth) and also is subjected to residual stresses from fabrication,

2 the temperature $T_{min,d}$ of the structural component attains its minimum value and hence produces the minimum toughness properties,

3 the structural component is stressed from permanent and variable loads accompanying the leading action $T_{min,d}$,

4 the design situation comprising the combination of the assumptions made above is accidental.

By using K-values for the assessment, see Equation (1), it is possible to take advantage of the Sanz-correlation between fracture mechanical values K_V and values obtained from Charpy V notch impact tests, as specified in the delivery standards for steels, so that the steels may be selected without referring to toughness data determined for a specific project.

5.2.2.2 Toughness Requirements

The toughness requirement $K^*_{appl,d}$ resulting from applied stresses may be determined for a given detail, e.g. for a welded attachment on the bottom chord of a girder, as given in Fig. 5.2.4.

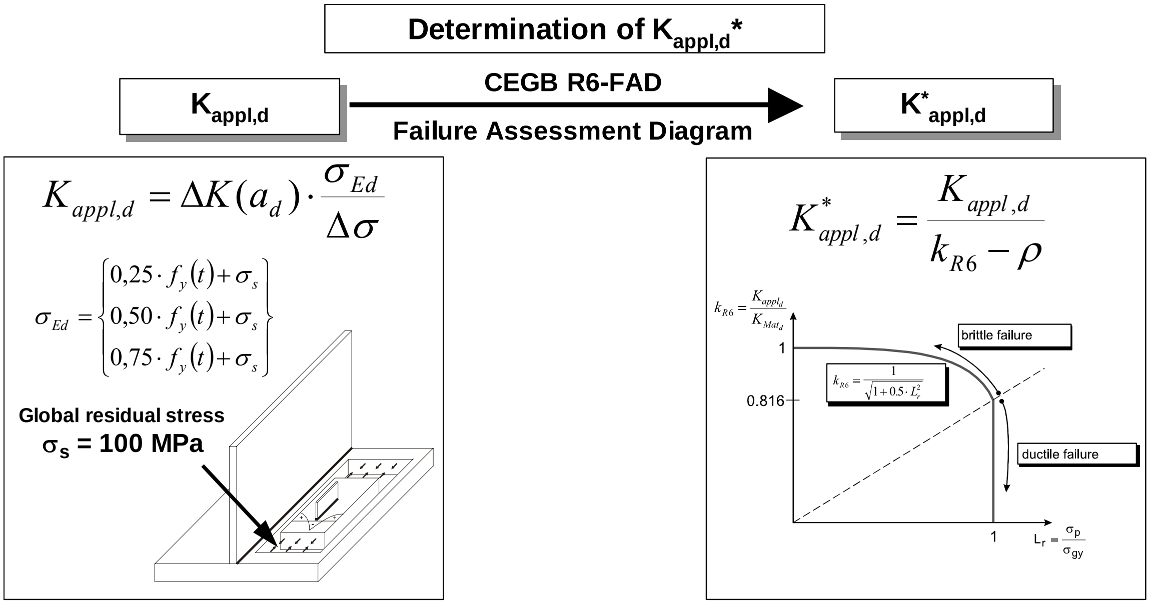

Fig. 5.2.4: Determination of toughness requirement $K^*_{appl,d}$ *[5.4]*

Stresses σ_{Ed} are a portion of the yield strength resulting from

a) the frequent load

$$G + \psi_2 Q_K \tag{2}$$

where G = permanent load; Q_K = charactistic value of variable load; ψ_2 = combination factor for frequent loads, see EN 1990 [5.9].

b) residual stresses σ_S in the tension flange from remote restraints to shrinkage effects from the manufacture of the beam. Local residual stresses at the hot spot from e.g. welding the attachment are included in the verification procedure.

$K^*_{appl,d}$ is determined in two steps:

1 Determination of the linear elastic value $K_{appl,d}$ (e.g. via $\Delta K(a_d)$-values),

2 Modification of $K_{appl,d}$ to obtain $K^*_{appl,d}$ by the CEGB R6-Failure Assessment Diagram (FAD) [5.10], to cope with local plastification of the crack tips.

5.2.2.3 Toughness Resistance

The toughness resistance $K_{mat,d}(T_{min,d})$ is calculated from the specified impact energy K_V expressed in terms of the temperatures T_{KV}, for which a minimum impact energy value K_V is reached (e.g. T_{27J} for K_V = 27 J) and from the minimum temperature of the component $T_{min,d}$ as input values, see Fig. 5.2.5.

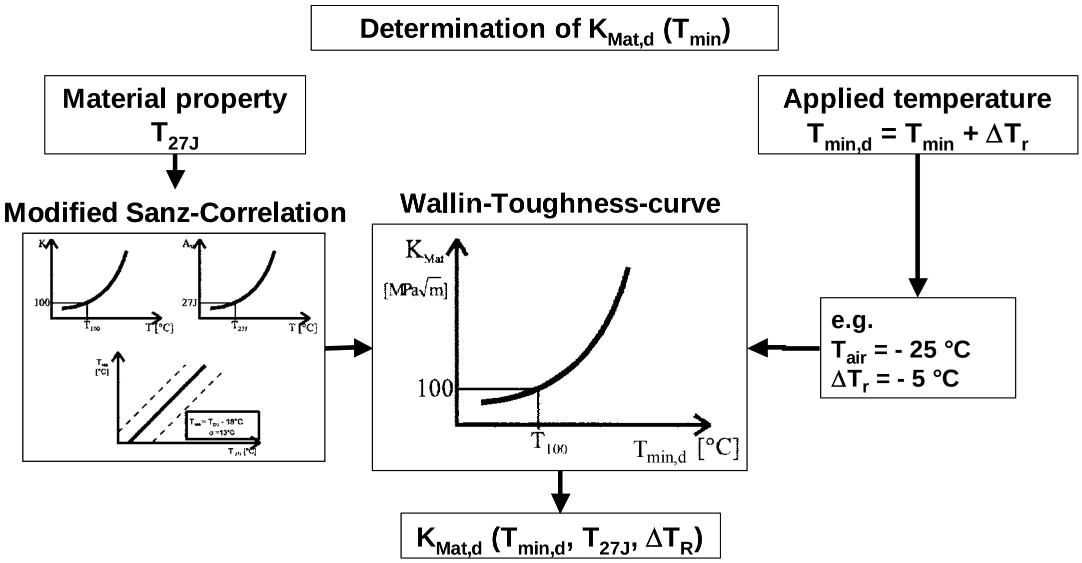

Fig. 5.2.5: Determination of toughness resistance $K_{Mat,d}(T_{min,d})$ [5.4]

Using the Sanz-correlation for linking T_{27J} to the stress intensity factor K_{100} and the Wallin master curve for determining K_{mat} from K_{100} and $T_{min,d}$, see [5.8] for more information, $K_{mat,d}(T_{min,d})$ may be obtained by introducing an additional safety element ΔT_R by which $T_{min,d}$ is shifted, to achieve sufficient reliability for the verification.

5.2.2.4 Method for Safety Assessment

The safety assessment as described above, see Equation 1, is transformed to temperature values and hence receives the form, see Equation (3) and Fig. 5.2.6:

$$T_{Ed} \geq T_{Rd} \tag{3}$$

where T_{Ed} is a reference temperature including all input values by taking them into account by temperature shifts. The input values are:

- the lowest air temperature T_{min} and radiation losses ΔT_r of the component,
- the influence of shape and dimensions of the member, imperfection from crack, and stress σ_{Ed}, resulting in ΔT_σ.
- the additive safety element ΔT_R,
- the influence of strain rate $\Delta T_{\&}$,
- the influence from cold forming $\Delta T_{\varepsilon pl}$.

Details of the calculation are given in Figure 6.

$$K^*_{appl,d} \leq K_{mat,d} \rightarrow \textbf{Transformation} \rightarrow T_{Ed} \geq T_{Rd}$$

Assessment scheme

$$T_{Ed} \geq T_{Rd}$$

Action side	**Resistance**
$T_{Ed} = T_{min} + \Delta T_r + \Delta T_\sigma + \Delta T_R \left[+\Delta T_{\dot{\varepsilon}} + \Delta T_{\varepsilon pl}\right]$	$T_{Rd} = T_{100}$

Action side:

- Lowest air temperature in combination with σ_{Ed}

  $T_{min} = -25°C$

- Radiation loss

  $\Delta T_r = -5°C$

- Influence of stress, crack imperfection and member shape and dimension

$$\Delta T_\sigma = -52 \cdot \ln\left[\frac{\left(\frac{K_{appl}}{k_{R6} - \rho} - 20\right) \cdot \left(\frac{b_{eff}}{25}\right)^{1/4} - 10}{70}\right] \quad [°C]$$

- Additive safety element

  $\Delta T_R = +7°\,C \;(\text{with } \beta = 3{,}8)$

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may be supplemented by

- Influence of the strain rate

$$\Delta T_{\dot{\varepsilon}} = -\frac{1440 - f_y(t)}{550} \cdot \left(\ln\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right)^{1,5} \quad [°C]$$

  with $\dot{\varepsilon}_0 = 0{,}0001\ s^{-1}$

- Influence from cold forming

  $\Delta T_{\varepsilon pl} = -3 \cdot DCF\ [°C]$

  with DCF = Degree of Cold Forming [%]

Resistance:

- Influence of material toughness

  $T_{100} = T_{27J} - 18\ [°C]$

Fig. 5.2.6: Assessment scheme based on temperatures [5.4]

The resistance side contains solely the test value T_{27J} and the temperature shift 18 °C caused by the Sanz correlation.

The additional safety element ΔT_R is obtained from a calibration of the procedure to large scale tests database, which contains tests on various steel grades, various welded attachments including local residual stresses and also cracks a_d produced by artificial initial cracks grown by subsequent fatigue loading, see [5.8].

5.2.2.5 Standardisation of Choice of Material

For a simplified procedure for the choice of material tables are necessary, that give the permissible plate thicknesses of structural members with the most common structural details depending on the steel grades, the toughness properties, the reference temperatures T_{Ed} and stress levels σ_{Ed}.

To this end for various structural details assumptions for initial surface cracks with the depths a_0 as given in Fig. 5.2.7 were made that were supposed to grow to the depths a_d by the application of a reference fatigue loading which depends on the fatigue detail class $\Delta\sigma_c$ according to [5.6] and corresponds to a quarter of the full fatigue damage D = 1.

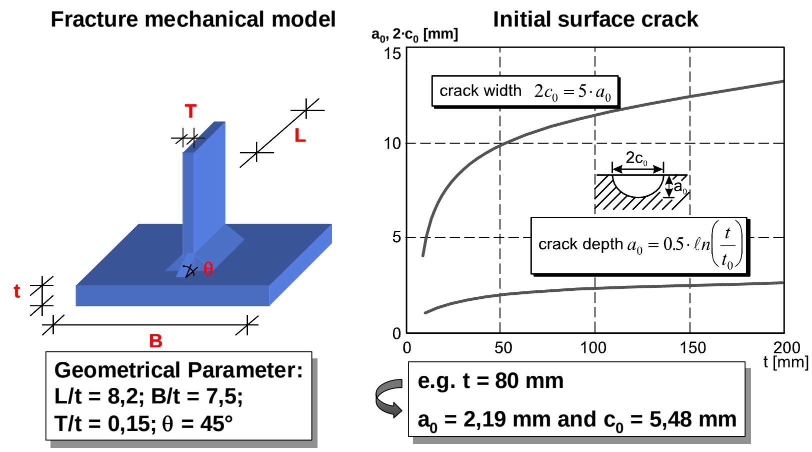

Fig. 5.2.7: Assumptions for details and initial sizes of surface cracks [5.4]

Fig. 5.2.8 shows values of the toughness requirements expressed as ΔT_σ obtained in this way for various detail classes as specified in Eurocode 3, Part 1-9 (EN 1993-1-9) "Fatigue" [5.6] and the enveloping standard requirement curve obtained from these calculations.

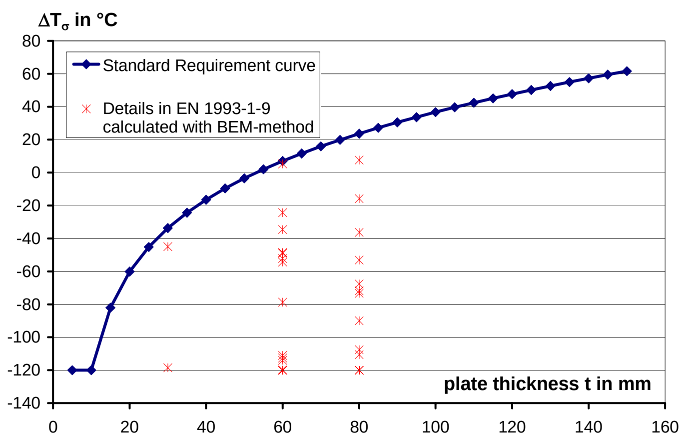

Fig. 5.2.8: Enveloping standard toughness requirement curve for details according to EN 1993-1-10 [5.4]

In Fig. 5.2.9 this standard requirement curve is compared with actual requirements from various steel and composite bridges.

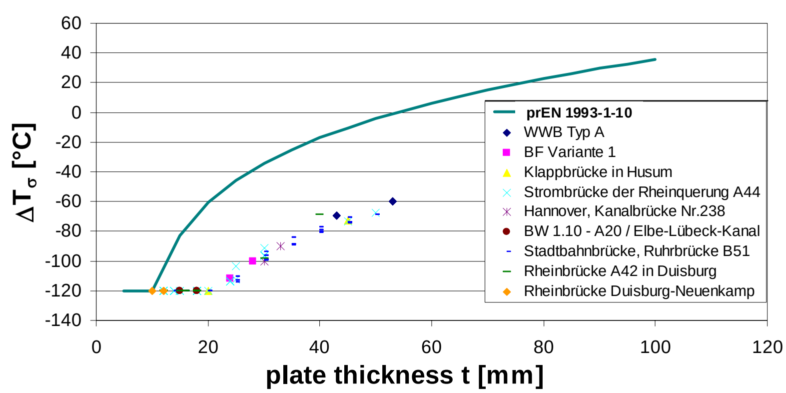

Fig. 5.2.9: Comparison of toughness requirements for steel bridges with the standard requirement curve [5.4]

Fig. 5.2.10 gives the table for the choice of material in EN 1993-1-10 based on this standard requirement curve. This table also includes high strength steels S460 and S690.

steel grade	charpy energy CVN		applied temperature T_{Ed} in °C																				
	at T °C	J	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
			$\sigma_{Ed}=0,25*f_y(t)+\sigma_s$							$\sigma_{Ed}=0,50*f_y(t)+\sigma_s$							$\sigma_{Ed}=0,75*f_y(t)+\sigma_s$						
		min.	max. permissible plate thickness t_z in mm (safety element ΔT_R included)																				
S235	20	27	135	115	100	85	75	65	60	90	75	65	55	45	40	35	60	50	40	35	30	25	20
	0	27	175	155	135	115	100	85	75	125	105	90	75	65	55	45	90	75	60	50	40	35	30
	-20	27	200	200	175	155	135	115	100	170	145	125	105	90	75	65	125	105	90	75	60	50	40
S275	20	27	125	110	95	80	70	60	55	80	70	55	50	40	35	30	55	45	35	30	25	20	15
	0	27	165	145	125	110	95	80	70	115	95	80	70	55	50	40	75	65	55	45	35	30	25
	-20	27	200	190	165	145	125	110	95	155	130	115	95	80	70	55	110	95	75	65	55	45	35
	-20	40	200	200	190	165	145	125	110	180	155	130	115	95	80	70	135	110	95	75	65	55	45
	-50	27	230	200	200	200	190	165	145	200	200	180	155	130	115	95	185	160	135	110	95	75	65
S355	20	27	110	95	80	70	60	55	45	65	55	45	40	30	25	25	40	35	25	20	15	15	10
	0	27	150	130	110	95	80	70	60	95	80	65	55	45	40	30	60	50	40	35	25	20	15
	-20	27	200	175	150	130	110	95	80	135	110	95	80	65	55	45	90	75	60	50	40	35	25
	-20	40	200	200	175	150	130	110	95	155	135	110	95	80	65	55	110	90	75	60	50	40	35
	-50	27	210	200	200	200	175	150	130	200	180	155	135	110	95	80	155	130	110	90	75	60	50
S420	-20	40	200	185	160	140	120	100	85	140	120	100	85	70	60	50	95	80	65	55	45	35	30
	-50	27	200	200	200	185	160	140	120	190	165	140	120	100	85	70	135	115	95	80	65	55	45
S460	-20	30	175	155	130	115	95	80	70	110	95	75	65	55	45	35	70	60	50	40	30	25	20
	-20	40	200	175	155	130	115	95	80	130	110	95	75	65	55	45	90	70	60	50	40	30	25
	-40	30	200	200	175	155	130	115	95	155	130	110	95	75	65	55	105	90	70	60	50	40	30
	-50	27	200	200	200	175	155	130	115	180	155	130	110	95	75	65	125	105	90	70	60	50	40
	-60	30	215	200	200	200	175	155	130	200	180	155	130	110	95	75	150	125	105	90	70	60	50
S690	0	40	120	100	85	75	60	50	45	65	55	45	35	30	20	20	40	30	25	20	15	10	10
	-20	30	140	120	100	85	75	60	50	80	65	55	45	35	30	20	50	40	30	25	20	15	10
	-20	40	165	140	120	100	85	75	60	95	80	65	55	45	35	30	60	50	40	30	25	20	15
	-40	30	190	165	140	120	100	85	75	115	95	80	65	55	45	35	75	60	50	40	30	25	20
	-40	40	200	190	165	140	120	100	85	135	115	95	80	65	55	45	90	75	60	50	40	30	25
	-60	30	200	200	190	165	140	120	100	160	135	115	95	80	65	55	110	90	75	60	50	40	30

Fig. 5.2.10: Table for the choice of material based on the standard toughness requirement curve [5.4],[5.5]

5.2.3 Yield to Strength Ratio Requirement

Most design codes give a limit of the yield/strength ratio for the applicability of the design rules. In Eurocode 3 (EN 1993-1-1) [5.11] the limiting value

$$\frac{f_u}{f_y} > 1.10 \quad \text{or} \quad \frac{f_y}{f_u} \leq 0.90 \tag{4}$$

is recommended.

Fig. 5.2.11 gives the yield/strength ratio versus the yield strength as obtained from tests. It can be seen that the value of the ratio increases with the yield strength. The value 0.9 is reached for a yield strength of about 720 MPa. This limitation is penalising the use of high performance steels in structural applications and can be shown not to be of relevance.

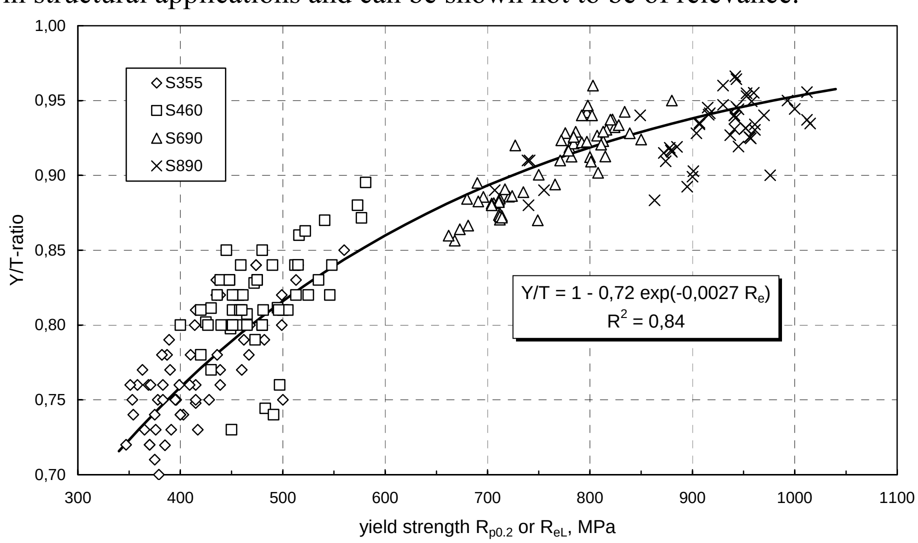

Fig. 5.2.11: Yield to tensile ratio of low and high strength ferritic steels depending on the yield strength [5.8]

As a typical example the net section stress - strain curve of large scale DECT (Double Edge Crack Tension) - test specimens made of S890 as shown in Fig. 5.2.12 is given in Fig. 5.2.13. This figure demonstrates fully ductile behaviour and fracture after general yield in the net section.

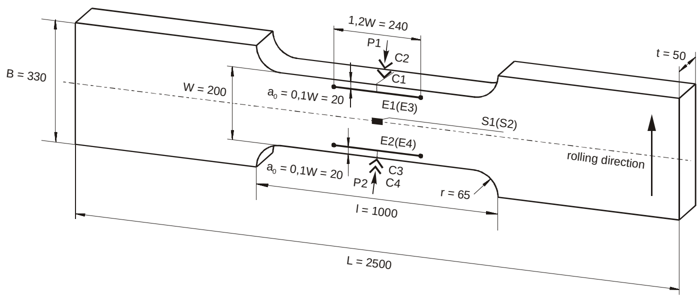

Fig. 5.2.12: Component like large scale specimen with measuring devices [5.7]

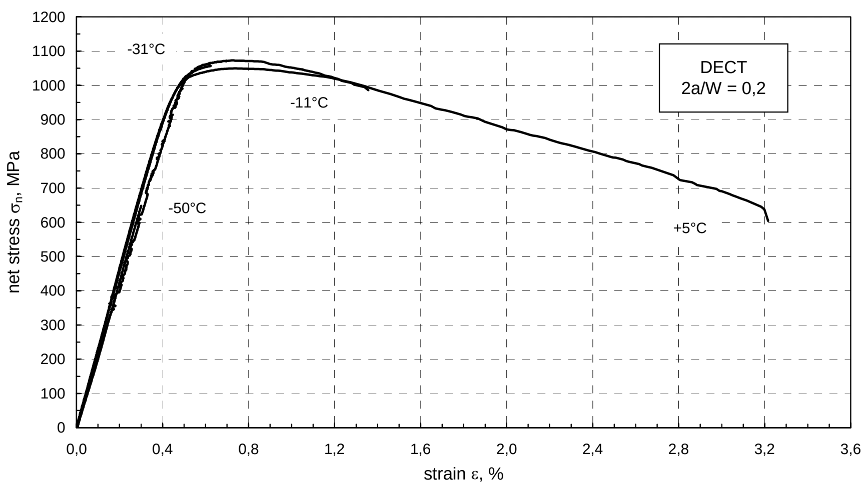

Fig. 5.2.13: Typical net stress temperature-yielding curves of steel [5.7]

Fig. 5.2.14 gives the maximum net section stresses versus the temperature. Whereas the test piece tested at -50 °C shows brittle fracture before yielding, ductile behaviour with stable crack growth is achieved for a temperature of -31 °C. This behaviour is clearly controlled by toughness.

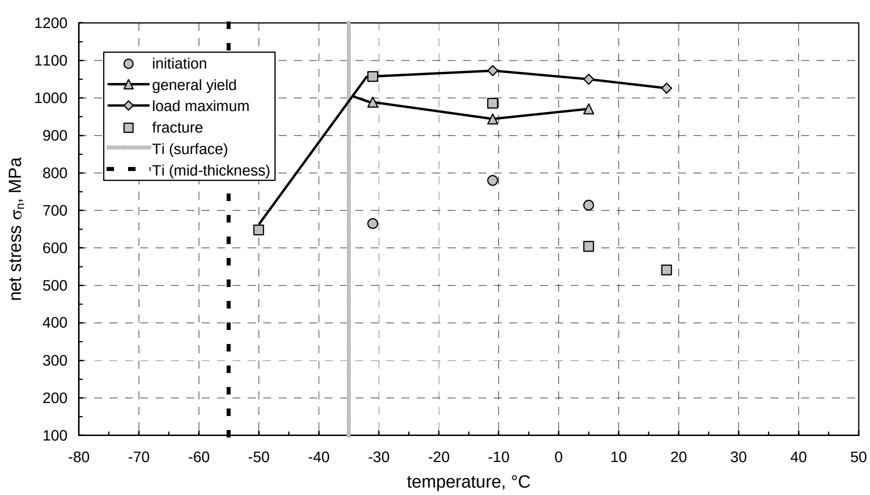

Fig. 5.2.14: Net stress temperature curve of steel [5.7]

Fig. 5.2.15 demonstrates, that toughness values are fully independent from the yield strength ratio. Hence there is no reason to limit f_y/f_u because of ductility reasons.

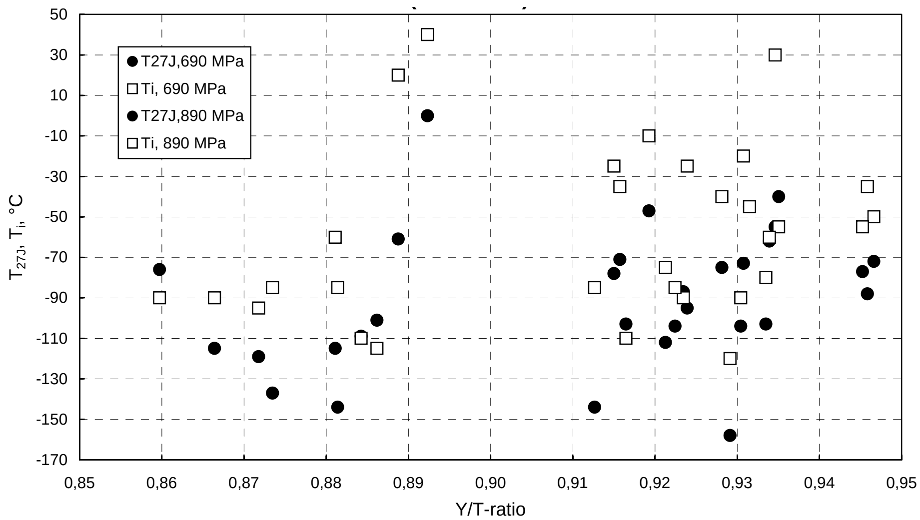

Fig. 5.2.15: Toughness properties depending on the yield to tensile strength ratio for S690 and S890 [5.7]

5.3 Buckling Resistance of Structures of High Strength Steel

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5.3.1 Introduction

The resistance to instability is frequently governing the design of steel structures. This is an obstacle to the use of high strength steel as the resistance to instability increases slower than the yield strength or in worst case not at all. This is so because the resistance to instability depends on the elastic modulus as well as the yield strength and the elastic modulus is the same for low and high strength steel. One relevant question is: When do you save money on using high strength steel when stability governs? A related question is how the structure should be designed to minimise the detrimental effects of instability. Both those question will be discussed in this chapter.
In order to study the economy of using high strength steel an estimate of prices is needed, which is a quite intricate question. The price of structural steel usually increases with the strength, which can be seen from Fig. 5.3.1. It shows relative prices for heavy plates from three leading European producers of high strength steel in which S235 has been chosen as reference. Fig. 5.3.1 also shows a trend curve, which follows the square root of the yield strength. Similar results have been shown for hot rolled strips in [5.12]. There is a substantial scatter in prices from time to time due to the market situation and the marketing strategy of the producer. The production cost increases mainly when the production process changes e. g. from TM to QT. Also the number of grades that has to be produced influences the production cost and it is a matter of strategy where to allocate these costs. An unusual example is that you can buy S355 cheaper than lower grades in the US. Anyway, the trend curve in Fig. 5.3.1 will be used in this study as a reflection of probable prices.
If the strength can be fully utilised the cost of material will be lowered as the strength is increased, see Fig. 5.3.2. The cost of a structure depends however more on costs for fabrication and erection than on the price of the material. The savings in fabrication costs depends very much on the type of structure. The cost of a butt weld is roughly proportionate to the square of the plate thickness