Type of steel	Chemical composition type	Plate thickness (mm)	Welding method	Weld heat input (kj/mm)	Weld joint impact test		
					Notch position	Test temp. (c)	Absorbed energy (J)
Tensile strength 490 MPa class steel	1.5% Ni-0.3% Mo	50	SAW	6	FL	0	95
					HAZ1mm		208
	Extremely low C-0.3% Cu-2.5% Ni	50	SAW	4.9	FL		255
					HAZ1mm		319
	0.3% Cu-2% Ni-0.5% Cr-0.3% Mo	25	SAW	6	FL		181
					HAZ1mm		253
	1% Cu-1% Ni-0.05% Ti	25	Electro-gas	12	FL		96
					HAZ1mm		134
Tensile strength 570 MPa class steel	0.3% Cu-3% Ni	40	SAW	5	FL	-5	156
					HAZ1mm		366
	Extremely low C-0.3% Cu-2.5% Ni	50	SAW	4.9	FL		205
					HAZ1mm		288
	0.3% Cu-2% Ni-0.5% Cr-0.3% Mo	50	SAW	6	FL		246
					HAZ1mm		277

Table 4.4.4: Weld joint performance test results for nickel-containing advanced weathering steels

4.4.2 Weathering Resistance Index

4.4.2.1 Quantification of Weathering Resistance in High Air-Borne Salt Environments by Weathering Resistance Index

The excellent corrosion resistance of nickel-containing advanced weathering steels in high air-borne salt environments depends mainly on nickel. However, commercial nickel-containing advanced weathering steels contain various combinations of alloy elements of copper, molybdenum, titanium, etc. other than nickel in order to improve the weathering resistance as shown in Table 4.4.1. Each element helps reduce the corrosion rate to a certain content level, depending on the type of element. It is difficult to evaluate weathering resistance quantitatively compared to mechanical properties, workability, and prices, but such information helps users to select suitable steel materials. A weathering resistance index formula is therefore proposed, which identifies the quantitative influence of each alloy element on weathering resistance.

The concept of a weathering index itself is not new. For example, a typical index given in the U.S. ASTM Designation: G 101-94 "Standard Guide for Estimating the Atmospheric Corrosion Resistance of Low-Alloy Steels" [4.23] is as follows:

$$I = 26.01(\%Cu)+3.88(\%Ni) + 1.20(\%Cr) + 1.49(\%Si) + 17.28(\%P) - 7.29(\%Cu)(\%Ni) - 9.10(\%Ni)(\%P) - 33.39(\%Cu)^2 \quad (1)$$

where $(\%Cu) \leq 0.51$, $(\%Ni) \leq 1.1$, $(\%Cr) \leq 1.3$, $(\%Si) \leq 0.64$, $(\%P) \leq 0.12$.

The larger the value of I, the higher is the weathering resistance.

The weathering index of Formula (1) can be used in environments where JIS weathering steels are applicable. However, in the case of nickel-containing advanced weathering steels, the nickel content exceeds the upper limit of validity for the formula, and some nickel steels contain molybdenum or titanium which are not even considered in the formula. Furthermore, the exposure test data used to derive Formula (1) were taken from an industrial zone (Kearny, New York, U.S.), so the formula does not properly reflect the influence of air-borne salt content. As a result, this weathering index cannot be used to judge the weathering resistance of nickel-containing advanced weathering steels, so we studied a weathering resistance index.

4.4.2.2 Proposal of Weathering Resistance Index

In order to develop a weathering resistance index, we examined the effect of each chemical element on weathering resistance, based on the results of exposure tests or dry-wet cycle tests of low-alloy steels in coastal areas conducted by the steel manufacturers. We used only the data of low-alloy steels in which the content of alloy elements varied systematically with changes in intensity of corrosive conditions and type of base steel materials. This was done to avoid having to perform multiple regression analysis, which involves collecting test results of steel materials that contain various alloy elements such as existing low-alloy steels, and which may have led to incorrect conclusions on the effect of alloy elements due to unbalanced chemical components. The changes in the effects of alloy elements in accordance with the intensities of corrosive conditions were not considered in the analysis.

The corrosion ratios of respective alloy elements to pure iron were regressed with the linear function of the content rate. The product of the results gave:

$$U = (1.0 - 0.16(\%C))\cdot(1.05 - 0.05(\%Si))\,(1.04 - 0.016(\%Mn))\,(1.0 - 0.5(\%P))\,(1.0 + 1.9(\%S))\,(1.0 - 0.10(\%Cu))\,(1.0 - 0.12(\%Ni))\,(1.0 - 0.3(\%Mo))\,(1.0 - 1.7(\%Ti)) \quad (2)$$

where $(\%C) \leq 1.5\%$, $(\%Si) \leq 5\%$, $(\%Mn) \leq 10\%$, $(\%P) \leq 0.15$, $(\%S) \leq 0.03$, $(\%Cu) \leq 1.1$, $(\%Ni) \leq 5$, $(\%Mo) \leq 0.6$, $(\%Ti) \leq 0.12$, $0.4 \leq U \leq 1.1$.

The smaller the value U in Formula (2), the lower the corrosion rate, or the higher the weathering resistance. In order to change the formula so that higher values represent higher weathering resistance, the weathering index V is introduced:

$$V = 1/U = 1/\{(1.0 - 0.16(\%C))\cdot(1.05 - 0.05(\%Si))\,(1.04 - 0.016(\%Mn))\,(1.0 - 0.5(\%P))\cdot(1.0 + 1.9(\%S))\,(1.0 - 0.10(\%Cu))\,(1.0 - 0.12(\%Ni))\,(1.0 - 0.3(\%Mo))\,(1.0 - 1.7(\%Ti))\} \quad (3)$$

where, $0.9 \leq V \leq 2.5$.

Both Formulas (2) and (3) do not include the effect of chrome. Since there are reports on both the positive and negative effects on corrosion resistance of steels containing chrome in salty environments, the formulae excluded this uncertain factor.

4.4.2.3 Estimation of Reduction in Plate Thickness by means of Weathering Resistance Index

The usefulness of the weathering index can be best judged when applied to weathering steels for paint-free applications. It may be impossible to use a steel material with comparatively good corrosion resistance if the environment is more corrosive than the performance of the material. For a precise judgment of suitability, a long-term estimation of reduction in plate thickness is indispensable, as the following example shows.

The permissible amount of corrosion in paint-free applications of nickel weathering steels is considered to be the same as that specified for JIS weathering steels shown in Figure 4.4.4. The following formula is commonly used to estimate plate thickness reduction in both nickel-containing advanced weathering steels and JIS weathering steels.

$$Y = A \cdot X^B \tag{4}$$

where X is exposure period (years) and Y is plate thickness reduction (mm).

The values A and B depend on the weathering resistance of steel material and the corrosive environment. The portion related to the weathering resistance of steel material can be expressed using the weathering resistance index, whereas the portion related to the corrosive environment is best expressed using the results of JIS weathering steel exposure tests, which is the most reliable means in Japan. In practice, using both values A and B, the ratio of nickel-containing advanced weathering steel to JIS weathering steel should be determined in the form of a value ratio A/A_{SMA} and B value ratio B/B_{SMA}. The values A and B of JIS weathering steel obtained from exposure tests conducted throughout Japan should then be used to establish a reliable means of estimating the plate thickness reduction in nickel-containing advanced weathering steels.

The values A/A_{SMA} and B/B_{SMA} as a function of V are presented below. These formulas (5) and (6) were obtained from a 9-year exposure test conducted at a quay in Kimitsu City.

$$A/A_{SMA} = H_A(V) = -0.144 + 4.95V^{-1} - 13.37V^{-2} + 15.03V^{-3} - 5.45V^{-4} \tag{5}$$

$$B/B_{SMA} = H_B(V) = 0.5545 + 0.45V^{-1} \tag{6}$$

Fig. 4.4.7 shows the weathering resistance index, and how it is used to estimate corrosion in nickel-containing advanced weathering steels.

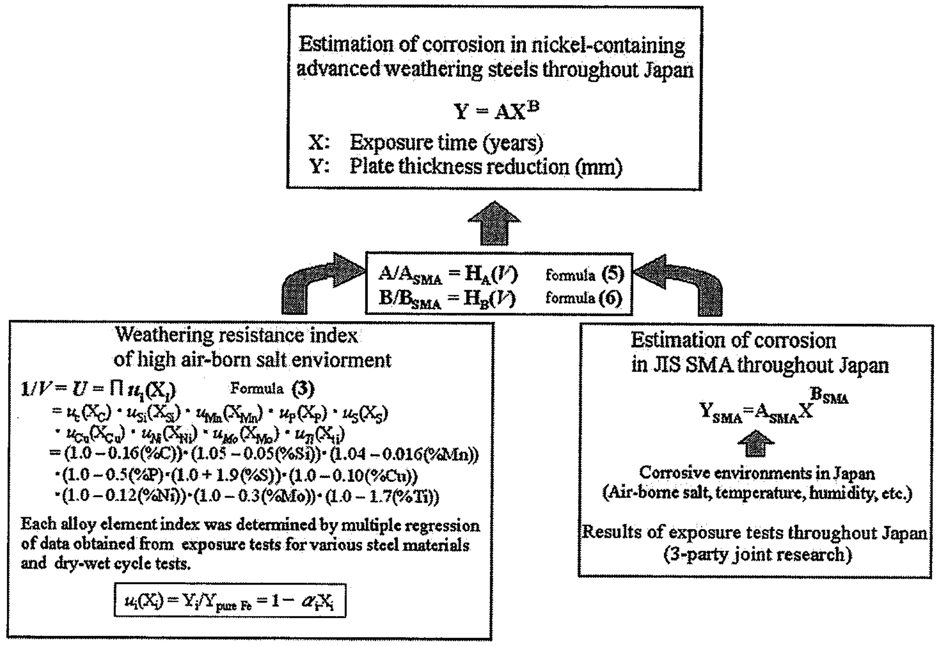

Fig. 4.4.7: Estimation of corrosion in nickel-containing advanced weathering steels using weathering resistant index

A weathering resistance index is supposed to represent material factors, independent of environmental factors. However, the coefficients in Formulas (2) and (3) will depend on the environment conditions and duration of the corrosion tests from which they are derived. The coefficients should be confirmed with respect to the influence of such test conditions. At present in Japan, air-borne salt content is a representative environmental factor and the application range of JIS weathering steels is specified based on this factor. As the use of weathering steels over longer periods has increased, various cases of corrosion that cannot be accounted for by air-borne salt have been reported. An index that covers temperature, humidity, and influence of structural members needs to be established.

4.4.3 Summary

Advanced weathering steels are proposed and their excellent performance has been proved. For the corrosion design of weathering steel considering environmental conditions, a weathering resistance index was proposed.

4.5 Improvement of Fatigue Strength

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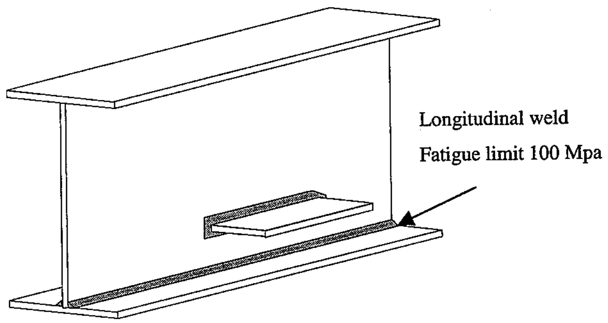
4.5.1 Target for Improvement of Fatigue Strength

When increasing the strength of a bridge without changing the span, the stress from live load on the bridge increases, which reduces the fatigue life compared to a bridge with lower strength. In order to capitalize on the improved strength of steel materials, therefore, improvement of fatigue strength is the key issue. However, it should be noted that higher fatigue strength does not always result in better results. The requirement for the fatigue strength for bridges is to have a fatigue limit of 2 million times at 100 MPa. It is associated with the requirement that the fatigue limit is required to be 2 million times at 100 MPa for the fatigue strength of the longitudinal fillet weld that joins the flange and web in the girder structure shown in Fig. 4.5.1. In summary, any higher fatigue strength than this level cannot be utilized effectively. Fatigue strength needs to be in compliance with the requirements of a fatigue limit of 84 MPa at 3.4 million times and 100 MPa at 2 million times and an S-N curve with an inclination of 1:3 is specified as JSSC-D class [4.24].



Class	Stress at 2×10^6	Fatigue limit (MPa)
A	190	190 (2×10^6)
B	155	155 (2×10^6)
C	125	115 (2.6×10^6)
D	100	84 (3.4×10^6)
E	80	62 (4.4×10^6)
F	65	46 (5.6×10^6)
G	50	32 (7.7×10^6)
H	40	23 (1.0×10^7)

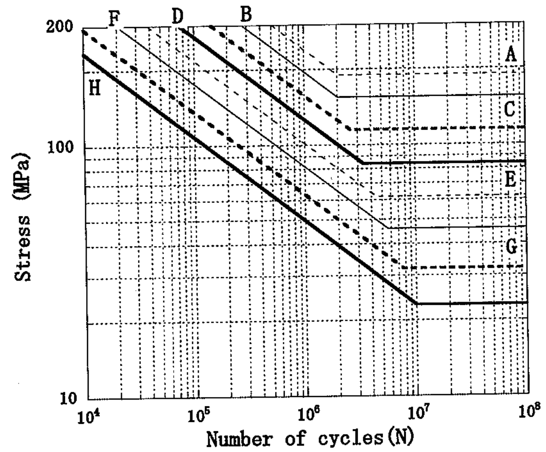


Fig. 4.5.1: Fatigue strength of typical girder structure

4.5.2 Mechanism of Low-Temperature Transformation Welding Material and Basic Characteristics

Low-temperature transformation (LTT) welding material has been developed only recently. The residual stress state at the weld toe is compressed for improved fatigue life of this advanced material, which is attracting the world's attention today. Ohta et al. [4.25]-[4.29] showed that the application of low-temperature transformation welding material to various types of welded joints and welding elements can improve the fatigue life.

Fig. 4.5.2 shows the mechanism of low-temperature transformation welding material. The vertical axis of this graph is the elongation of the weld metal, and the horizontal axis is the temperature of the weld metal. The temperature at which martensitic transformation expansion starts (martensitic transformation startup temperature: Ms temperature, martensitic transformation finish temperature: Mf temperature) is set lower than ordinary steel materials to establish a different relationship with respect to volumetric difference between the base metal and weld metal from ordinary welding materials. As a result, the residual stress state at the weld toe changes.

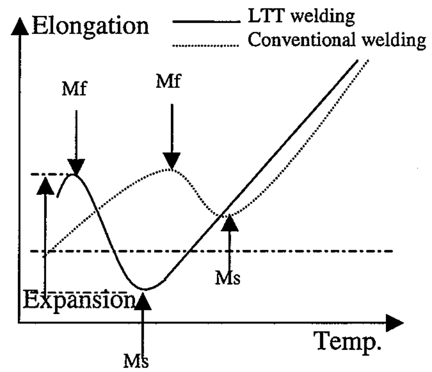


Fig. 4.5.2: Temperature-elongation relationship of low-temperature transformation welding material

Low-temperature transformation welding material can be used for joining members directly. However, there are shortcomings such as (1) the material is expensive as it contains large amounts of nickel and chrome and (2) the mechanical properties such as toughness may be low. An additional welding method is recommended for the application of this material. In the additional welding method, as shown in Fig. 4.5.3, an additional bead of low-temperature transformation welding material is formed so that the material covers the main bead of an ordinary welding material in the boxing weld point where fatigue develops, such as in TIG-dressing. This method results in such advantages as (1) the low-temperature transformation weld does not need to have any especially high strength, (2) the leg becomes long to have the toe distanced away from the gusset where stress concentrates, reducing the stress concentration, and (3) material costs are lowered from the limited use of the low-temperature transformation welding material.

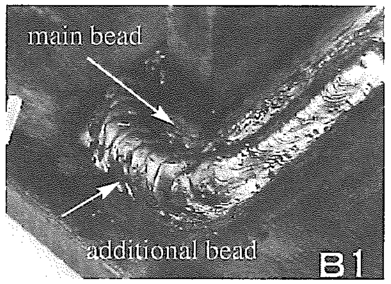


Fig. 4.5.3: Additional LTT welding bead

At present, out-of-plane gusset (boxing joint), cruciform joint (non load-transmission), lap joint, and pipe panel-point joint (see Fig. 4.5.4) are detailed items of weld joints for which improvement of fatigue strength by the use of such low-temperature transformation welding material has been verified [4.25]-[4.29]. Of these joints, the out-of-plane gusset and non load-transmission type cruciform joint are suitable for the use of low-temperature transformation welding material as a girder-structure fatigue test employing the additional welding process mentioned below proved their fatigue performance [4.30].

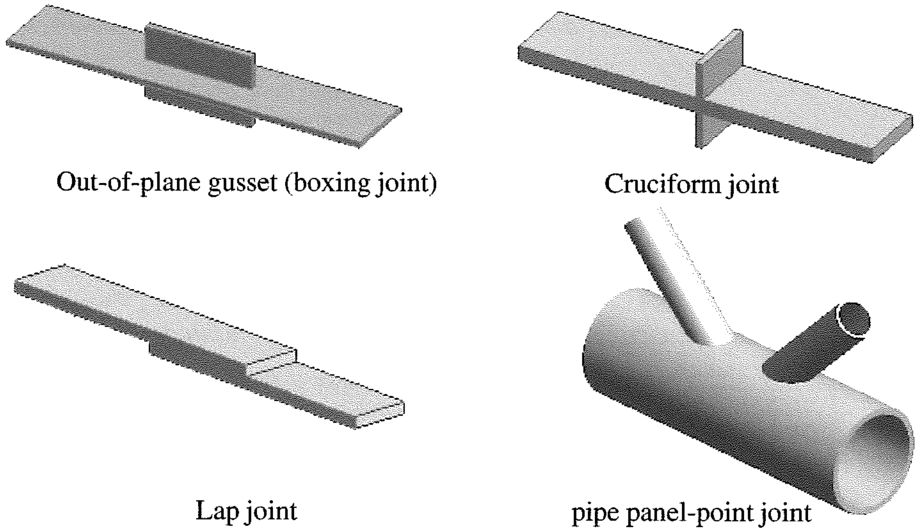


Fig. 4.5.4: Fatigue improved weld joint details by LTT

The following basic characteristics of low-temperature transformation welding materials are known in general.

1 The higher the strength of steel material for which the low-temperature transformation material is used, the higher the fatigue strength becomes [4.31].
2 When a steel plate having a thickness of 12 mm or less is welded on both sides, the second welding is likely to destroy the residual stress formed by the first welding. In other words, it is necessary to reduce welding heat input when low-temperature transformation welding material is used for such a thin plate [4.32],[4.33].

In general, the more additional elements such as nickel increase, the lower the Ms temperature becomes. Lowered Ms temperatures tend to improve the residual stress. However, there is a trade-off; the lower the Ms temperature, the lower the weldability becomes. As a result, weld toe shape and weld metal property substantially deteriorate and the fatigue life may not be improved as desired. Furthermore, increased amounts of additional elements may also lower the toughness of the weld metal remarkably. To overcome such shortcomings, it is necessary to select a proper welding material having a suitable Ms temperature in accordance with the welding procedure and/or welding position by identifying the influence of the strength of steel material and Ms temperature on the residual stress. For example, if a flat position welding is possible with a good toe shape, it is advantageous to introduce much compressive residual stress by using a welding material of a lower Ms temperature. When a horizontal position welding is absolutely necessary for an existing structure, it is recommended to use a welding material of higher Ms temperature for better weldability in spite of low compressive residual stress.

Table 4.5.1 shows the tabulated characteristics of the low-temperature transformation welding materials whose fatigue performance has been confirmed so far. Table 4.5.2 and Table 4.5.3 respectively show the components and mechanical properties of the low-temperature transformation welding materials. From these tables, it is clear that the strength of low-temperature transformation welding materials is high and the toughness of the materials decreases with the Ms temperature level.

Type	Ms (centigrade)	Weldability	Toe profile	Toughness
A	350	Normal	Normal	About 100J
B	250	bad	undercut	Under 30J

Table 4.5.1: Outline of LTT basic properties

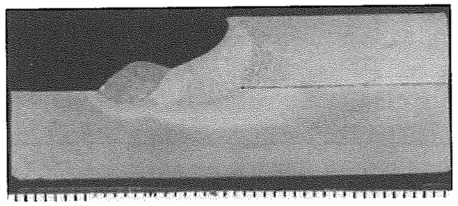
Type	Ms	C	Si	Mn	P	S	Ni	Cr	Mo
A	350	0.055	0.17	0.25	0.007	0.004	10.20		
B	250	0.054	0.55	0.9	0.004	0.002	10.20	10.30	
									(weight %)

Table 4.5.2: Chemical components of LTT materials

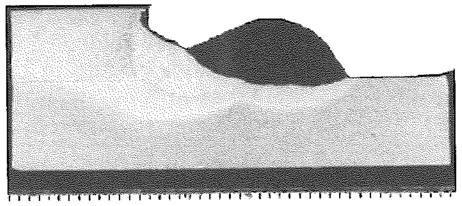
	Tensile test				Charpy test		
	0.2%strength	TS	elongation	φ_z	Absorb energy (J)		
	(N/mm^2)		(%)		vE-120	vE-80	vE-0
A	808	852	18	65	49,49,47,(48)	58,54,56,(56)	94,96,98,(96)
B	592	1063	15	28			20,18,20,(19)

Table 4.5.3: Mechanical properties of LTT materials

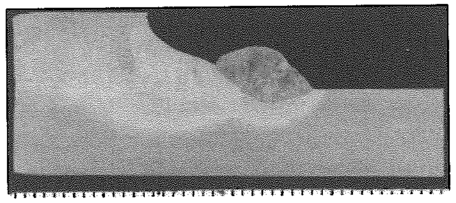
Fig. 4.5.5 shows the cross-sectional macro photos of the toe sections welded by a horizontal position welding with heat input levels changed. As shown in this figure, the welding material B which has a lower Ms temperature tends to result in an undercut when heat input is low. When using the material B for a horizontal position welding, weaving becomes necessary, which means that it is difficult to control heat input to a low level.



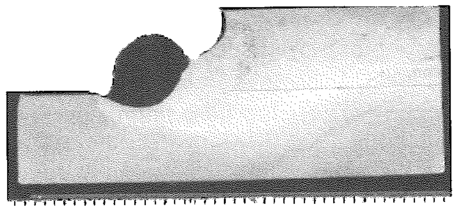
Material A / 3.2mm / 120A / 28sec



Material B / 3.2mm / 100A / 56sec



Material A / 4.0mm / 130A / 37sec



Material B / 4.0mm / 120A / 42sec

Fig. 4.5.5: Macro photos of typical cross sections

4.5.3 Study based on Fatigue Tests

4.5.3.1 Girder Fatigue Test

A girder fatigue test was conducted using a welded plate girder of I-shape cross section having a total length of 5.500 mm and a height of 500 mm. Fig. 4.5.6 shows the shape and dimensions of the girder. The test aimed at obtaining D-class fatigue strength (100MPa at 2×10^6) with the type of steel (steel strength), plate thickness, and type of welding material used as the test parameters.

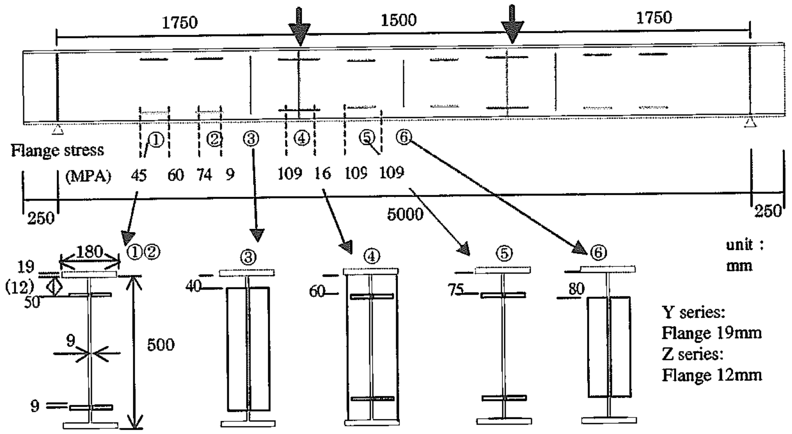


Fig. 4.5.6: Sketch of girder test specimen

The divisions of the use of welding materials on the test specimen were determined as shown in Fig. 4.5.7. The girder was divided into left and right sides from the center of the girder and the same welding materials were used on the opposite sides across the web respectively. Welding was applied by carefully adjusting heat input so that the introduced residual stress would not be destroyed by an increase in the temperature.

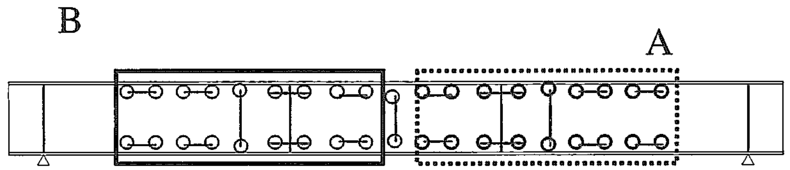


Fig. 4.5.7: Area divisions for each welding material

Table 4.5.4 shows the standard welding conditions for the respective welding materials. Each welding was conducted in a horizontal position. Only the material A satisfied the heat input limit of 13 kJ/mm [4.34]. Fatigue test was conducted for an amplitude between the maximum load at which the flange stress becomes 180 MPa over the 1,500-mm pure bending section and the minimum load of 1 tf.(flange stress is about 5MPa)

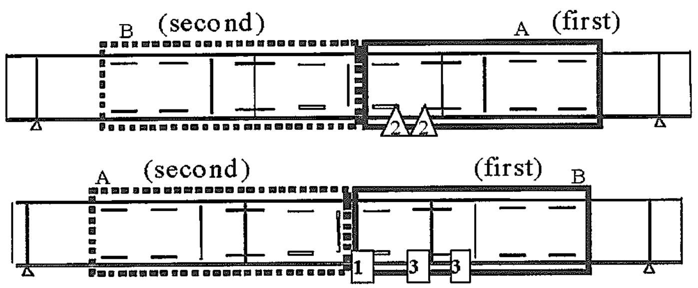
TYPE	Diameter	Welding condition	Arc time(sec)	Heat input(J/cm)
A	4.0	120A-25V-1.8cm/min	30	10588
B2	3.2	100A-23V-9.6cm/min	53	14341

Table 4.5.4: Welding conditions

Fig. 4.5.8 shows the list of the positions, types, and sequence of fatigue cracks developed in each test specimen. The words (First) and (Second) in the figure indicate the welding sequence on each web surface. As welding cannot be conducted from both sides at the same time, heat sequence of welding can give effect to residual stress condition. Fig. 4.5.9 shows photos of typical cracks, while Fig. 4.5.10 shows the S-N relationship.

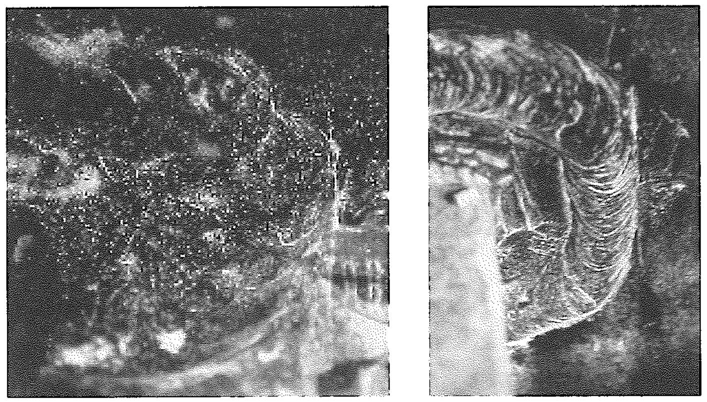
The test specimens of high-strength steel SM570 and SM490 (12 mm) with a large web thickness satisfied the target class D strength (100MPa at 2×10^6), without fatigue crack. The test specimen SM490 with a thickness of 9 mm partially satisfied the class D, with some cracks generated at class E strength (80MPa at 2×10^6) level.

The influence of the welding sequence was checked, with the result that only the first welding caused cracks at the toes or bead joints.



No crack with (SM570, 9 mm) and (SM490, 12 mm) specimens
Symbols: △ indicates a crack in a toe. △ indicates a crack in a bead joint.

Fig. 4.5.8 Crack position and turn



(a) Crack near weld toe (b) Crack between beads

Fig. 4.5.9: photo of each crack type

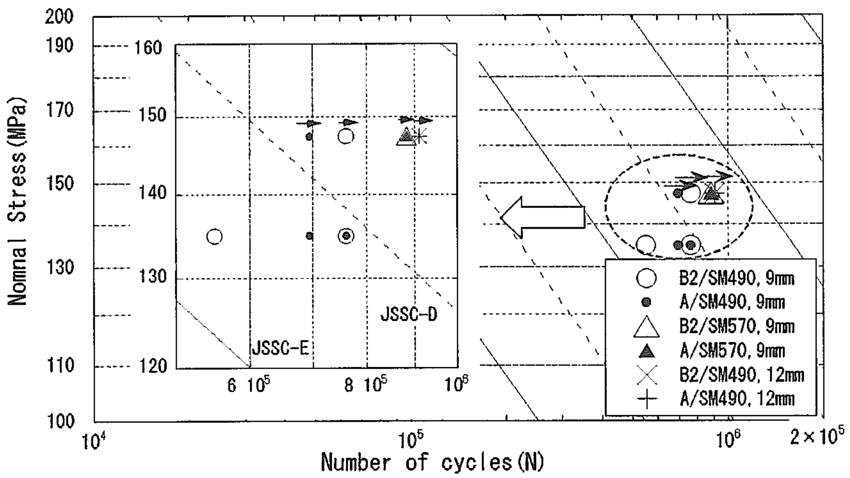


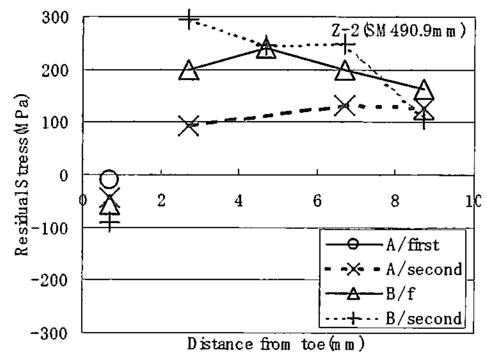
Fig. 4.5.10: Fatigue test result

From the results mentioned above, it was confirmed that higher strength of steel materials and larger thickness of webs contribute to higher fatigue strength. The Ms temperatures of welding materials did not indicate noticeable variations.

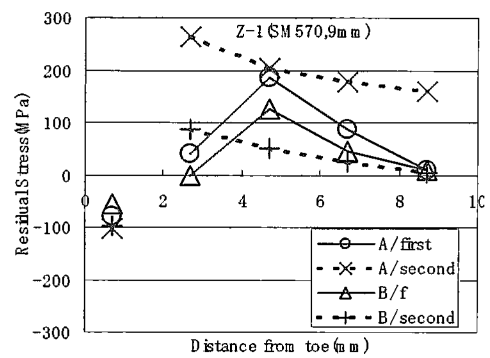
4.5.3.2 Residual Stress Measurement

Residual stress measurement was conducted at the weld toe in the girder test specimen after the fatigue test. Stress was measured by the cut-out method in the toe where no crack was finally caused. Fig. 4.5.11 shows the distribution of measured residual stress. The X axis shows the distance from the toe.

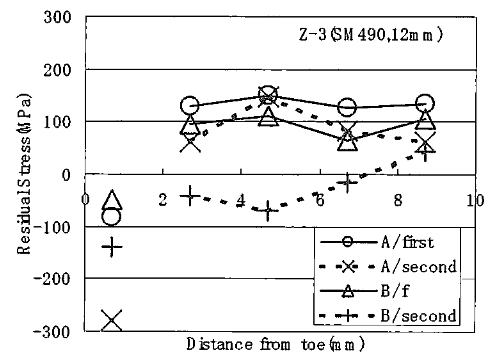
From the figure, it is first confirmed that a compressive residual stress is observed near each toe. Comparing the compressive residual stress between two welded sides, the second welded sides indicate higher levels. The difference in the fatigue strength between the front and rear surfaces seems to be caused by the difference in the residual stress. On the other hand, there does not seem to be any clear trend in the difference between the welding materials A and B.



(a) Specimen (SM570,9mm)



(b) Specimen (SM490,9mm)



(c) Specimen (SM490,12mm)

Fig. 4.5.11: Results of residual stress measurement by cut-out method

4.5.4 Summary

Fatigue life D class (100MPa at 2×10^6) can be realized by applying an additional welding process with low-temperature transformation welding material to the out-of-plane gusset boxing section and longitudinal rib bottom section of steel girder. The thicker the web and the higher the strength of the steel material, the higher the effect of fatigue strength improvement becomes. Weldability and toughness improve with the higher Ms temperature of the welding material. When applying welding in a horizontal position, it is recommended to use a welding material of high Ms temperature and control heat input to 13 kJ/mm.

4.6 Examples and Applications

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4.6.1 BHS Projects in the Future

For the growth of BHS applications and to familiarize the public with the effects of BHS, application of BHS to large-scale projects of high attention-grabbing effect is scheduled.

The Tokyo Bay Seaside Road is routed to connect the reclaimed lands on the off-shore side of the coastal road at Tokyo Bay to reduce traffic congestion in the area. The construction of a bridge 760 m in length is scheduled across the sea for the Seaside Road. BHS will be used for this project.

Fig. 4.6.1 shows the present basic structure plan. A truss bridge design, not a cable-stayed bridge design, is employed for this bridge having a central span of 440 m since the height under the girder is required to be large enough for a maritime transportation seaway under the bridge and at the same time the total height of the bridge is required to be low enough for the normal operation of Haneda Airport. Though the main material for the structure will be BHS500, BHS700 will be applied to the truss members near the bridge piers where a large tensile force acts, for rational utilization of the strength-improved materials. The FEM-based optimum design for this project currently under way is also based on the plan to use BHS materials. As a result of the optimum designing, some configurations included in this basic drawing may be modified. The superstructure construction work is scheduled to begin in 2005 and be completed in 2007. The total amount of steel material BHS500 and BHS700 will be in the order of 20,000 tons.

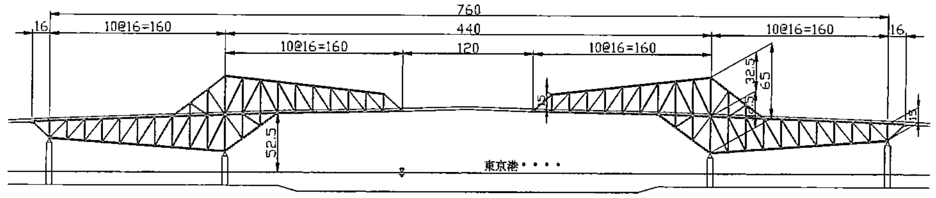


Fig. 4.6.1: Basic structure plan of No.3 Seaway Bridge

Prior to the application of BHS to this large-scale project, there are some plans for the application of BHS to small and medium-scale bridges to confirm the rational process in the manufacturing process.

4.6.2 Current Status of Application of Nickel Containing Advanced Weathering Steels

4.6.2.1 Application in the Past

A total of 19,400 tons of nickel containing advanced weathering steels has been used for bridges as of April 2003, including experimental applications since 1997. Fig. 4.6.2 shows the distribution of the applications in each prefecture in Japan.

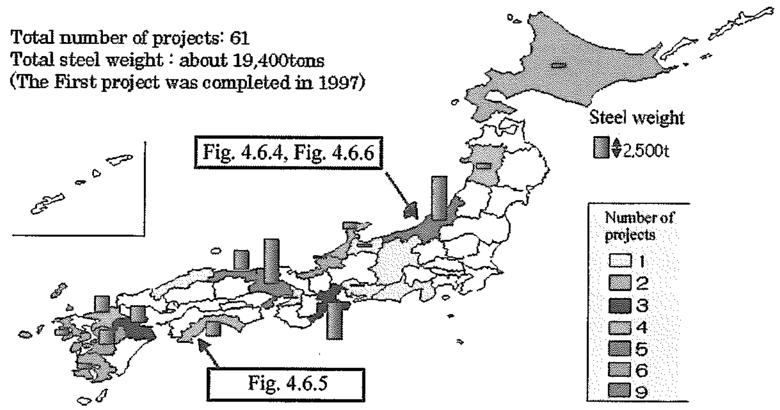
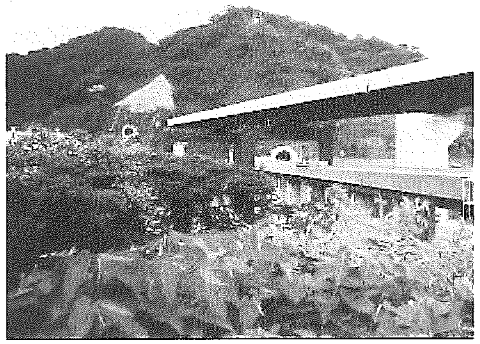


Fig. 4.6.2: Distribution of applications in each prefecture in Japan. (Source: Investigation by Japan Iron and Steel Federation in April 2003)

Most of the application sites are located in areas where air-borne salt from the sea exceeds 0.05 mdd. Use of the conventional weathering steel materials JIS G3114 (hot-rolled weathering steels for welded structure) in these areas is considered to be unsuitable. Nickel-containing advanced weathering steels have also been used in some inland areas where the influence of an antifreeze agent is an issue in connection with the corrosion in bridges.

4.6.2.2 Application Examples

Fig. 4.6.3, Fig. 4.6.4, and Fig. 4.6.5 show bridges using nickel containing advanced weathering steels while Fig. 4.6.2 shows the location of these bridges.



Distance from seashore: 0.7 km from the Japan Sea

Location: Nigata Prefecture

Application purpose: Measures against air-borne salt from the Japan Sea

Note: Surface treatment is applied in combination with the material

Steel material: 3%Ni-Cu type steel
Maximum thickness: t=33mm

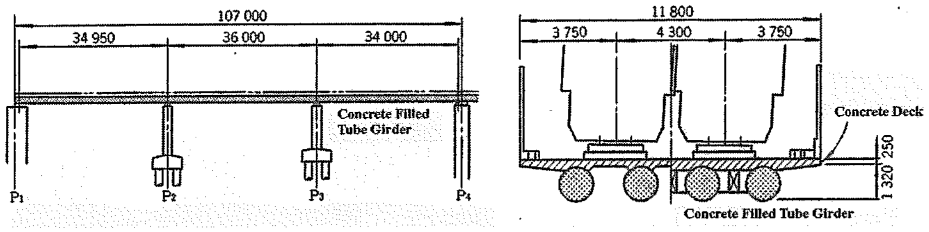
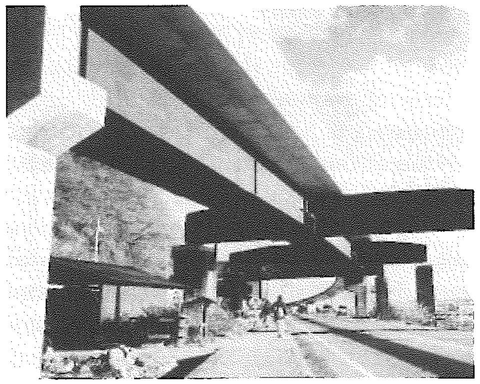


Fig. 4.6.3: Example #1 of application of nickel containing advanced weathering steels (Hokuriku Shinkansen Line [railroad], Hokuriku over-road bridge)



Distance from seashore: 0.05 km from the Pacific Ocean

Location: Kochi Prefecture

Application purpose: Measures against air-borne salt from the Pacific Ocean

Steel material: 3%Ni-Cu type steel
Maximum thickness: t=32mm

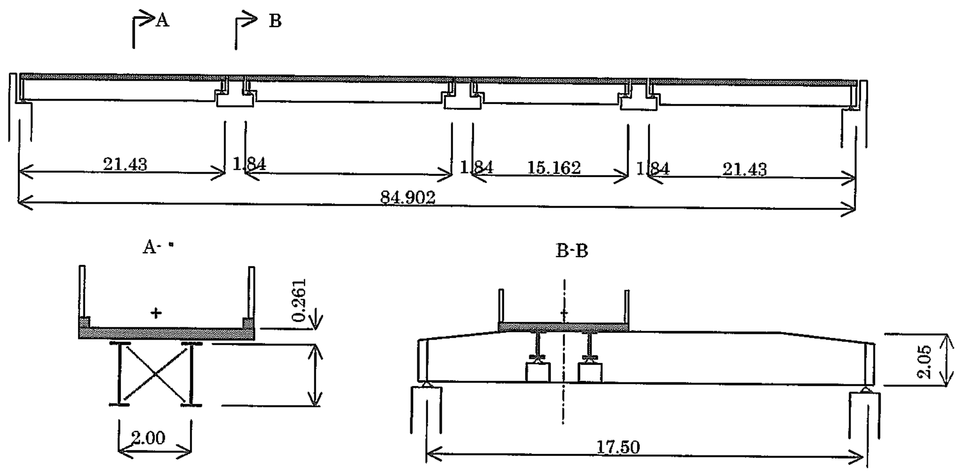


Fig. 4.6.4: Example #2 of application of nickel-containing advanced weathering steels (Kochi Prefecture Asa Line [railroad], Nishichi over-road bridge)

Distance from seashore: 20 km from the Japan Sea

Location: Nigata Prefecture

Application purpose: Measures against air-borne salt from the Japan Sea.

Steel material: 1.5%Ni-Mo type steel
Maximum thickness: t=28mm

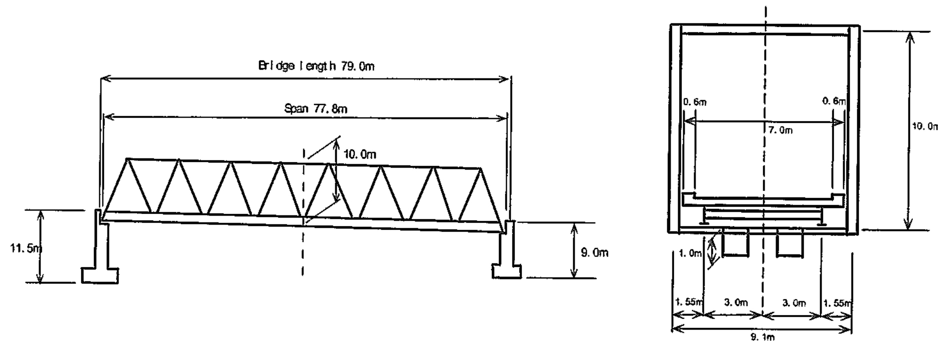


Fig. 4.6.5: Example #3 of application of nickel-containing advanced weathering steels (Road bridge in Nigata Prefecture, Shigemizawa No.1 bridge)

By applying these new steels, the life cycle costs of those bridges are decreased, as the cost of painting and re-painting disappears. For example, the typical initial cost of heavy-duty type painting is about 8,000 yen/m^2, while the initial (additional) cost of using these new steels is about 4,200 yen/m^2. So, for only the initial painting cost, these new steels can decrease the costs by about 50%. And the life cycle cost benefit can be larger as the re-painting cost is not needed. This cost is evaluated at a 3 span continuous girder bridge with a 2 girder structure. The length of its span is 40m+40m+40m and surface area is 11.7m^2/ton.

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5 High-Performance Steels in Europe

5.1 Production Processes, Mechanical and Chemical Properties, Fabrication Properties

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5.1.1 Introduction

Since the first application of steel in steel structures in the 19th century the development of steel construction has been closely linked to the development in material properties and production methods. Significant achievements concerning strength, economy, design versatility, fabrication and erection techniques and service performance would not have been possible without the substantial improvements of steel. Especially with the application of "new" production processes for carbon steels such as the thermo-mechanical rolling and the quenching and tempering process, steel with a high construction strength but guaranteeing also good fabrication properties such as weldability was introduced into the construction market. Today, the application of these grades is driven by the following major reasons:

- Economy: By increasing the strength of steel, the structural section can be reduced depending on the structural problem. This may reduce fabrication and erection costs – an important task in high-wage economies.
- Architecture: The size of structural elements can be reduced, enabling special aesthetic and elegant structures, which embed in the environment in an outstanding manner.
- Environment: Construction with less steel means also a reduced consumption of our world's scarce resources.
- Safety: Modern high strength steel grades exhibit not only high strength values. Special grades combine this strength with excellent toughness so that a high safety both in fabrication and application of the structures is ensured. In particular, modern offshore steel grades performing at some of the lowest service temperatures are a good example.

It should not be neglected that several other branches started with the application of high strength grades earlier. Mobile crane construction uses today steel grades up to a yield stress of 1100 MPa; in the offshore industry thermo-mechanically rolled steels in higher strength classes are likely to be the steel group the most often used for cold water applications. Even the shipbuilding industry has started to design with high strength steel. Nevertheless, this article focuses on the steel grades which are today used in steel construction (bridges, buildings, hydraulic steelwork) in Europe although we know that engineers in this field can profit a lot from the good experience made in other branches.

5.1.2 Production Processes for High-Strength Steel

The development of new steel grades was always driven by the demand of the users wanting materials exhibiting good mechanical characteristics such as yield strength and toughness as well as excellent fabrication properties ensuring an efficient fabrication technology in the workshop and during the erection of a steel structure. Among others there are two major ways of increasing the yield strength of steel:

- Alloying: By alloying elements such as carbon and manganese the strength of steel products can be "easily" increased. But it is known that an addition of alloying elements in most cases also worsens the fabrication properties of steel products, in particular the weldability.
- Heat treatment: Heat treatment has an effect on microstructure and grain size. The main advantage of this process consists in the achievement of a fine-grained structure resulting in higher strength as well as better toughness of the material compared to a coarse-grained structure (Relation of Hall-Petch).

For this reason the heat treatment is of major importance in the development of new steel grades and the historical context is shown in Fig. 5.1.1. Until 1950 the steel which is today known as S355J2 according to the European standard EN 10025 was regarded as high tensile steel. As a plate this grade is usually produced by conventional hot rolling (see Fig. 5.1.2, process A) followed by a normalizing heat treatment – a heating slightly above the A_{c3}-temperature (temperature where the ferritic-perlitic structure has totally changed to austenite) followed by a slow cooling resulting in a fine and homogeneous grain structure (see Fig. 5.1.2, process B). This process can be replaced by a normalizing rolling where – simply expressed – this heat treatment is included in the rolling but leads to a similar result.

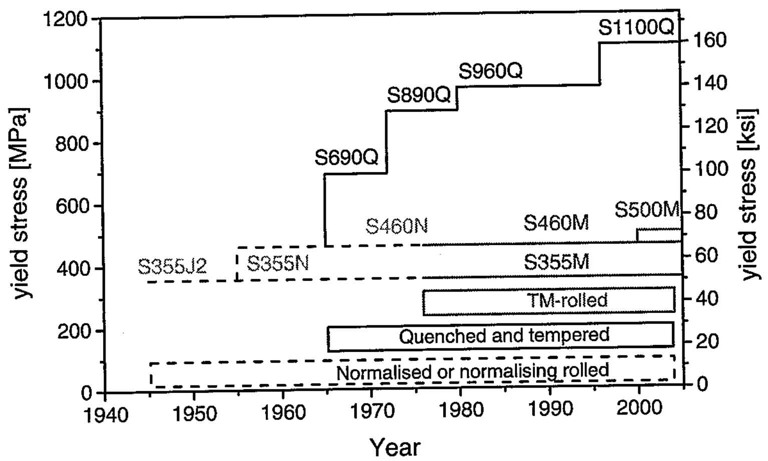

Fig. 5.1.1: Historical development of production processes for rolled steel products

During the 1960s the application of the quenching and tempering process for structural steel grades began (process C). This process consists of a rolling followed by heating above the A_{c3}-temperature and a rapid cooling, normally in water, plus a subsequent tempering below Ar_1 (temperature where austenite begins to form. See Fig. 5.1.2 process C). Roughly speaking during the first step a "strong" martensitic or bainitic grain structure is obtained whose toughness properties are significantly improved during the tempering process. See Fig. 5.1.3. Besides this heat treatment the good balance between strength and toughness is based on the fact that these steels are alloyed by adding micro-alloying elements (niobium, vanadium, titanium) precipitating as finely distributed carbonitrides.

Today this process gives steel grades with yield strength up to 1100 MPa, although only grades up to 960 MPa yield stress are standardised. Furthermore, classical European steel construction, i.e. buildings and bridges, profits only very rarely from this "ultra-high" strength steel and is mostly limited to steel grades up to S690.

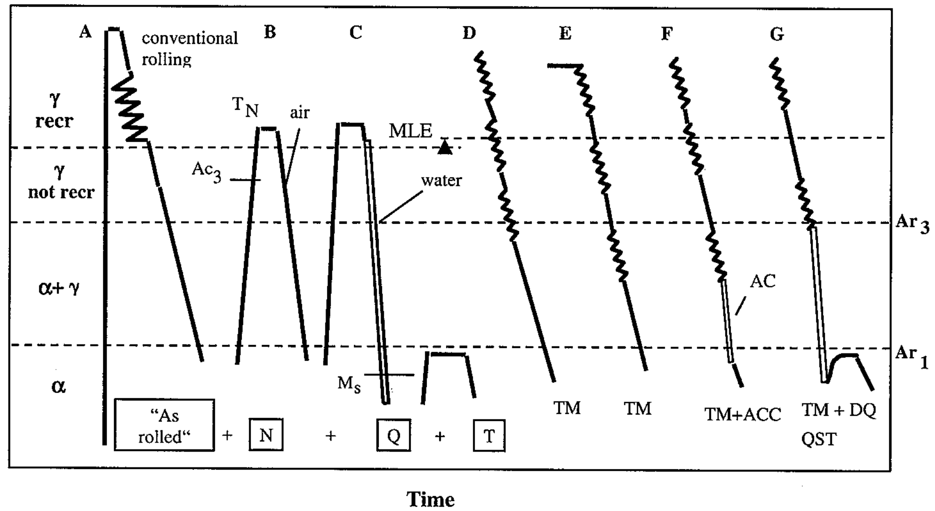

Fig. 5.1.2: Different types of heat treatment and rolling processes. Temperature on the vertical axis. γ recr denotes recrystallized austenite, γ not recr denotes non recrystallized austenite, α + γ the temperature range for austenite + ferrite and α the temperature region for ferrite and pearlite in conventional steels. MLE shows the increase in the temperature for recrystallization due to micro-alloying. T_N is the normalization temperature.

In the 1970s thermo-mechanical rolling process was developed and first applied for line pipe plates, but then quickly found its way into the fields of shipbuilding and the construction of offshore platforms, both for plates and for rolled sections. TM-rolling is defined as a process in which final deformation is carried out in a certain temperature range leading to material properties which cannot be achieved by heat treatment alone. The resulting steel grade has high strength as well as high toughness and at the same time a minimum alloying content resulting in best weldability.

Also here it is usual to add to the steel some micro-alloying elements such as niobium, vanadium or/and titanium in a very small amount in order to achieve an additional strengthening effect by the formation of fine carbonitrides and to increase the recrystallisation temperature. First rolling passes are carried out at traditional rolling temperature. Further rolling passes are accurately defined at a temperature below the recrystallisation temperature (process D) and sometimes even in the temperature

range of coexisting austenite and ferrite/pearlite (process E). The process may be finished by an accelerated cooling especially for thicker plates (process F).
All these varieties of the TM-process produce a very fine-grained microstructure of ferrite and pearlite or – partly also bainite – as shown in Fig. 5.1.3, avoiding high alloying content and therefore providing very good toughness properties and an excellent weldability. Furthermore, high yield strength grades can be produced by these techniques. Plates with guaranteed minimum yield strength value up to 500 MPa are available in thicknesses up to 80 mm and are already used in shipbuilding and offshore construction. For constructional steelwork even plates of 120 mm have been produced and applied successfully in particular in bridges.
Process G shows TM-rolling followed by direct quenching and self tempering. Here an outer layer of the material is quenched. The interior, being warmer, subsequently gives a tempering of the quenched material.

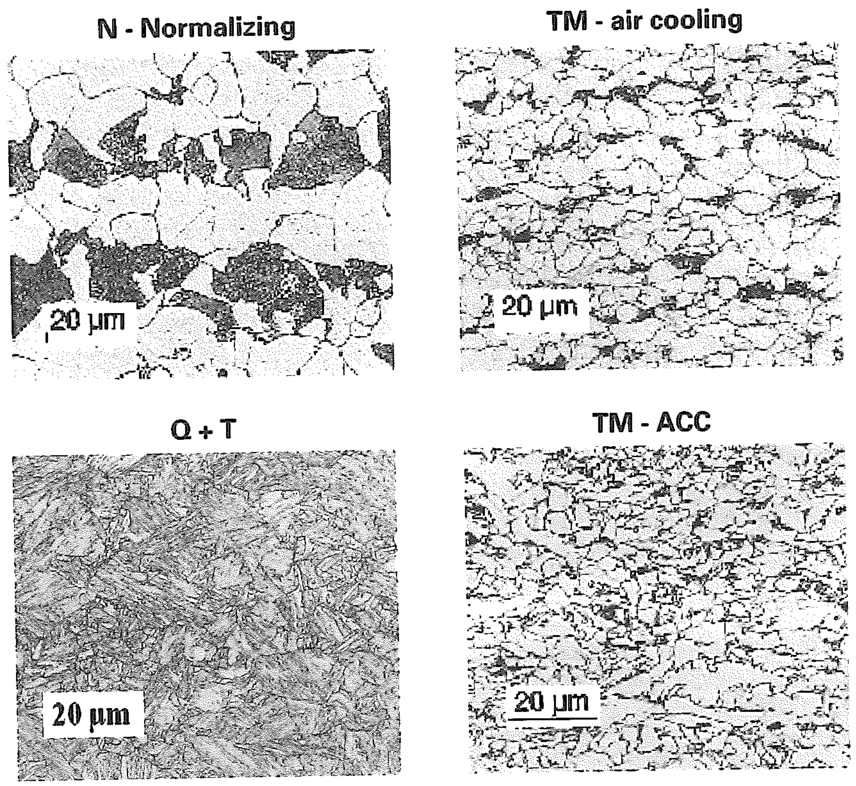

Fig. 5.1.3: Grain microstructure of QT and TM-steel compared to normalized steel

In the remainder of this article quenched and tempered steels will be referred to as QT and thermo-mechanically rolled steels as TM since Q and M (formal delivery conditions) are also quality designations for impact toughness.

5.1.3 Products and Properties

5.1.3.1 Standards

In November 2004 the new standard for hot-rolled steel products for usage in steel construction EN 10025 was published. This standard has six parts and defines the most common structural steel grades which were formerly treated in the independent standards EN 10113 and EN 10137. With reference to the higher strength grades, which are under discussion in this document, for each state of delivery explained in chapter 4.1.2, one part of EN 10025 (2004) is now reserved. However, as far as chemical and mechanical properties of the defined steel grades are concerned, these parts do not show big changes in comparison to the former standards EN 10113 and 10137. Regarding High-Performance Steels EN 10025 Part 4 (replacing EN 10113-3) describes TM steels with minimum yield stress of 420 and 460 MPa at the lowest product thickness. For each yield stress grade, two qualities exist with different guaranteed toughness levels measured by the Charpy-V test with the specimen in the longitudinal direction: an M-quality with 40 J at -20°C and an ML-quality with 27 J at -50°C.
QT steel grades are now standardised in EN 10025 Part 6 (replacing EN 10137-2) with yield stress grades from 460 MPa up to 960 MPa, whereas constructional steelwork in Europe today is limited to steel grades up to S690. Higher strength grades are still the domain of the construction equipment industry. Here, for each yield stress grade except S960 three different qualities exist Charpy-V tested in the longitudinal direction: a Q-quality with 30 J at -20°C, a QL-quality with 30 J at -40°C and a QL1-quality with 30 J at -60°C.
Table 5.1.1a, 1b and 2 summarises the mechanical properties of these steel grades according to the standard. Nevertheless, it must be recognised that products from actual production mostly exceed these minimum values by far.

	Nominal thickness, mm					
	≤16	>16 ≤40	>40 ≤63	>63 ≤80	>80 ≤100	>100 ≤120
Grade	Minimum yield strength R_{eH}, MPa					
S420M	420	400	390	380	370	365
S460M	460	440	430	410	400	385
	Tensile strength R_m, MPa					
S420M	520-680		500-660	480-650	470-630	460-620
S460M	540-720		530-710	510-690	500-680	490-660

Table 5.1.1a: Strength requirements for structural steels EN 10025-4, TM-steels

Grade	Nominal thickness t, mm		
	3 ≤t ≤50	50 < t ≤100	100 < t ≤150
	Minimum yield strength R_{eH}, MPa		
S460Q	460	440	400
S500Q	500	480	440
S550Q	550	530	490
S620Q	620	580	560
S690Q	690	650	630
S890Q	890	830	-
S960Q	960	-	-
	Tensile strength R_m, MPa		
S460Q	550-720		500-670
S500Q	590-770		540-720
S550Q	640-820		590-770
S620Q	700-890		650-830
S690Q	770-940	760-930	710-900
S890Q	940-1100	880-1100	-
S960Q	980-1150	-	-

Table 5.1.1b: Strength requirements for structural steels EN 10025-6, QT-steels

Quality	Test temperature, °C			
	-20	-40	-50	-60
M	40 J			
ML			27 J	
Q	30 J			
QL		30 J		
QL1				30 J
Energy requirements at the lowest test temperature are given for each quality. Transverse impact testing can be ordered as an option. For further details refer to the relevant standard.				

Table 5.1.2: Minimum energy values for impact tests on longitudinal specimens. TM and QT steels

5.1.3.2 Chemical Properties

The maximum alloying contents for high strength steels as given in the standards are often considered to give very conservative upper limits. Actual values for the products are usually much lower. Furthermore, it should be taken into account that not only the steel grade has an influence on the alloying content – in addition the chemical composition may vary with the thickness range. It is obvious that also differences between products of different producers are quite normal.

Table 5.1.3 gives examples of the chemical compositions of S460ML, S460QL and S690QL in comparison to the common European constructional steel S355J2. It can be seen that for grades up to S460 TM-rolled grades show a very "clean" chemical composition resulting in excellent weldability. But also the alloying concepts of the higher strength grade, in particular S690, allow for efficient fabrication processes, as described below.

	S355J2		S460ML		S460QL		S690QL	
	EN 10025 Part 2	typical analysis[a]	EN 10025 Part 4	typical analysis	EN 10025 Part 6	typical analysis	EN 10025 Part 6	typical analysis
C	≤0.22	0.17	≤0.16	0.08	≤0.20	0.15	≤0.20	0.16
Si	≤0.55	0.45	≤0.60	0.45	≤0.80	0.45	≤0.80	0.30
Mn	≤1.60	1.50	≤1.70	1.65	≤1.70	1.50	≤1.70	1.30
P	≤0.025	0.018	≤0.025	0.011	≤0.020	0.012	≤0.020	0.012
S	≤0.025	0.015	≤0.020	0.002	≤0.010	0.005	≤0.010	0.005
Nb	-	-	≤0.05	< 0.04	≤0.06	0.017	≤0.06	<0.04
V	-	-	≤0.12	-	≤0.12	-	≤0.12	-
Ti	-	-	≤0.05	-	≤0.05	-	≤0.05	-
Mo	-	-	≤0.20	-	≤0.70	0.115	≤0.70	0.37
Ni	-	-	≤0.80	0.19	≤2.0	-	≤2.0	0.15
Cu	≤0.55	-	≤0.55	0.17	≤0.50	-	≤0.50	0.08
Cr	-	-	≤0.30	-	≤1.50	-	≤1.50	0.40
B	-	-	-	-	≤0.0050	-	≤0.0050	<0.003
CE	0.47	0.42	0.47	0.39	0.47	0.39	0.65	0.54
Pcm	-	0.26	-	0.19	-	0.19	-	0.29
CET	-	0.32	-	0.26	-	0.26	-	0.35

[a] Wide variation of the composition is possible due to a variety of possible production routes.
Carbon equivalents:
CE = C + Mn/6 + (Cr + Mo + V)/5 + (Ni + Cu)/15
Pcm = C +Si/30 + (Mn + Cu + Cr)/20 + Ni/60 + Mo/15 + V/10 + 5B
CET = C + (Mn + Mo)/10 + (Cr + Cu)/20 + Ni/40

Table 5.1.3: Chemical compositions of high-strength steel, 50 mm thick (weight-%). S355J2 is given for comparison. Excerpt from the standard requirements and examples of actual values.

5.1.3.3 Mechanical Properties

It has to be clearly stated that the values guaranteed by the standards are minimum values. The user can normally expect considerably better values, in particular for toughness. Fig. 5.1.4 shows as an example typical transition curves for the Charpy-V energy against the test temperature for an S460ML and an S690QL steel and compares them to a conventional steel, S355J2. It can be seen that these high-strength steels show significantly higher Charpy-V values at the testing temperature than given in the standard (27 J at -50°C and 30 J at -40°C respectively). Even at room temperature the toughness behaviour is better than for a conventional S355J2. These high toughness values also result in good welding properties as described below.

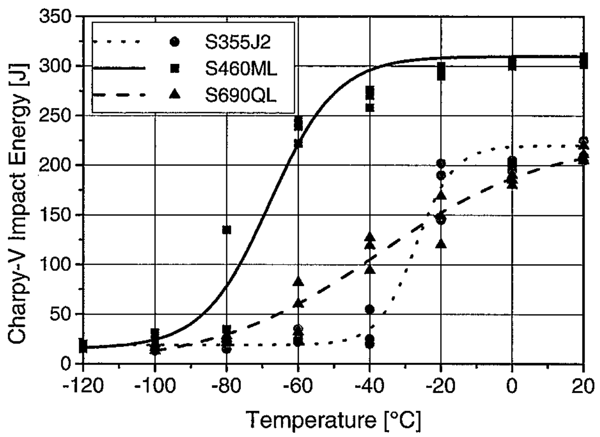

Fig. 5.1.4: Charpy V-temperature transition curves for S460ML and S690QL with S355J2 for comparison

5.1.4 Fabrication Properties

5.1.4.1 Welding

General recommendations for welding of TM and QT steels are given in EN 1011-2, Welding – Recommendations for welding of metallic materials – Part 2 Arc welding of ferritic steels.

Most steel producers give detailed information on welding on request. Such information may also be found on the respective Internet web sites of the steel producers. It is recommended to contact the steel producer for detailed information since the alloying concepts may differ between different producers.

The steels treated here have low contents of alloying elements and low carbon equivalents. They can be easily welded to all ordinary structural steels using any conventional arc welding method. This is especially true for the TM steels since an S460M steel has a lower carbon equivalent than an ordinary structural steel. The TM steels have a wider window for heat input and preheating than QT steels. Even for thicker plates of S460M preheating can be omitted and thus welding costs reduced.

The main points to avoid cold cracking are:

- Preheating the parent material when recommended. This is also most important for tack welding and the root pass.
- The joint surfaces should be perfectly clean and dry.
- Minimise the shrinkage stress by ensuring a good fit and a well planned sequence of weld runs.
- Use a filler material with low hydrogen content.

An example of preheating recommendations from one producer is given in Table 5.1.4.

<table>
<tr><td rowspan="2">Steel grade</td><td colspan="11">Maximum combined plate thickness, mm</td></tr>
<tr><td>30</td><td>40</td><td>50</td><td>60</td><td>70</td><td>80</td><td>90</td><td>100</td><td>110</td><td>120</td><td>130</td></tr>
<tr><td>S460M, ML</td><td colspan="7">Room temperature, RT</td><td colspan="4">75°C</td></tr>
<tr><td>S690Q, QL, QL1</td><td colspan="2">RT</td><td colspan="4">75°C</td><td colspan="2">100°C</td><td colspan="3">150°C</td></tr>
<tr><td colspan="12">Combined plate thickness is the sum of the thicknesses of the plates joined. Maximum hydrogen content of weld metal 5 mg/100 g. Heat input approximately 1.7 kJ/mm.</td></tr>
</table>

Table 5.1.4: Example of preheating recommendations

The heat input determines the properties of the weld. Low heat input increases the maximum hardness and the risk for cold cracking, whereas high heat input decreases the toughness. Examples of recommendations are: For S420M, S460M up to 5.3 kJ/mm; for S690Q up to 3.5 kJ/mm, depending in both cases on the combined plate thickness. For thinner combined thicknesses, below 60 to 80 mm, the heat input must be reduced.

On the choice of filler material: Choose a filler material giving a hydrogen content ≤ 10 ml/100 g. For an S690 plate thicker than 20 mm, ≤ 5 ml/100g is recommended. Regarding S460M steels matching or overmatching material can be chosen, while for S690Q matching or undermatching is appropriate. For S690Q steel it is advisable to use an undermatching material for the root run. For fillet welds it is always advisable to select an undermatching filler material. The major benefits of selecting lower strength filler material for QT steel with yield strength > 500 MPa are: higher toughness of the weld metal, improved ductility of the joint, reduced sensitivity to cracking.

On stress relieving: EN 10025-1 has the following information. Stress relieving at more than 580°C for more than 1 h may lead to a deterioration of the mechanical properties of the steel grades as defined in parts 2 to 5. (Comment: this corresponds to the old standards EN 10025+A1, EN 10113-2, EN10113-3, EN 10155). For normalized or normalized rolled grades with minimum $R_{eH} \geq 460$ MPa the maximum stress relief temperature should be 560°C. For the QT steel grades of EN 10025-6 (corresponding to EN 10137-2) the maximum stress relief temperature should be 30 K below the tempering temperature. For QT steels the purchaser is recommended to contact the steel producer. The maximum temperature is normally in the range 550°C to 580°C.

Our general practical experience is that post weld heat treatment is not necessary. The toughness and hardness of the weldment normally meet the requirements and is not substantially improved by stress relieving. It should only be carried out if a reduction of the residual stresses is needed for some special purpose or if specified in the design codes.

5.1.4.2 Flame Straightening

TM and QT steels can be flame straightened, but this requires more care than for conventional normalized or hot rolled steels. Gas heating and/or induction heating are recommended. The skill of the operator is essential. The temperature should be measured using thermocouples. Temperatures not exceeding 600°C for more than ten minutes do not affect the material properties detrimentally for QT steel of the type S690QL and S960QL [5.1]. For TM-material up to S460M it was shown that a superficial heating up to 950°C does not influence the mechanical properties significantly, whereas for more severe heating conditions up to the middle of the plates a maximum of 700°C should be observed [5.2]. The material manufacturer should be contacted if more detailed information is needed.

5.1.4.3 Cold Forming

General recommendations are found in ECSC IC 2 [5.3].
The material standards (EN 10025-4, -6) have a Note: "Cold forming in general leads to a reduction of the ductility. Furthermore it is necessary to draw attention to the risk of brittle fracture in connection with hot-dip zinc coating."
For TM steels EN 10025-4 has options regarding flangeability for material with a nominal thickness $t \leq 12$ mm and roll forming for material with $t \leq 8$ mm. For the steel grades S420 and S460 the minimum bend radius is 4 times t with the axis of the bend in the transverse direction and 5 times t in the longitudinal direction.
For QT steels EN 10025-6 has an option regarding flangeability for material with a nominal thickness $t \leq 16$ mm. For the S690 grade the minimum bend radius is 3.0 times t with the axis of the bend in the transverse direction and 4.0 times t in the longitudinal direction. Transverse and longitudinal refer to the rolling direction.

However, also in this respect the steels often have far better properties than specified in the standards. An example from one manufacturer is given in Table 5.1.5.

Grade	Thickness mm	Transverse R/t	Longitudinal R/t	Springback degrees
S420M, ML; S460M, ML		1.0	1.5	3-6
S690Q, QL, QL1	$t < 8$	1.5	2.0	6-10
	$8 \leq t < 20$	2.0	3.0	
	$t > 20$	3.0	4.0	
R denotes the punch radius and t the actual thickness				

Table 5.1.5: Example of bending recommendations from one manufacturer. They refer to shot blast and shop primed plate. As delivered plate may be bent somewhat narrower.

Most manufacturers give recommendations for bending and can be contacted for more detailed information such as estimates of the bending force needed.

5.1.4.4 Hot Forming

General recommendations are found in ECSC IC 2 [5.3].
According to EN 10025-4 TM steels shall not be hot formed. If deemed necessary the manufacturer shall be consulted. Often forming at a maximum temperature of 580°C for short times is allowed. QT steels can be hot formed. EN 10025-6 permits hot forming up to the stress relief annealing temperature, normally in the range 550°C to 580°C. This agrees with the recommendations of the steel manufacturers.
In practice hot forming is rarely used due to the good cold formability of these steels.

5.1.4.5 Heat Treatment

TM and QT steels are not intended for heat treatment after delivery. The only exception is stress relief annealing as described above. TM steels obtain their properties during rolling, i.e. controlled deformation at precise temperatures. This cannot be repeated by heat treatment. QT steels are heat treated. However, the steel manufacturer normally has much more efficient quenching equipment than can be found elsewhere. The steel manufacturer adjusts the steel composition to match the properties of the quenching and tempering equipment to obtain the intended properties. Therefore a renewed quenching treatment may not restore the material properties and makes the inspection documents invalid. In rare cases, if heat treatment is deemed necessary the steel manufacturer must be consulted.

5.1.4.6 Zinc Coating

All of the steels in EN 10025 except those of part 5 have options for ordering grades suitable for hot-dip zinc coating. Reference is made to EN ISO 1461 and EN ISO 14713 or to special chemical requirements in the standards, see Table 5.1.6. These limits concern the thickness and appearance of the zinc layer.

Classes	Composition wt - %		
	Si	Si + 2.5 P	P
Class 1	≤ 0.030	≤ 0.090	-
Class 2[a]	≤ 0.35	-	-
Class 3	$0.14 \leq Si \leq 0.25$	-	≤ 0.035
[a]Class 2 applies only for special zinc alloys			

Table 5.1.6: Classes for the suitability for hot-dip zinc coating (ladle analysis)

For class 2 the maximum carbon equivalent CE shall be increased by 0.02, for class 3 by 0.01.
For the steels treated here class 2 is the most relevant one.
Galvanizing is a complex issue and only general advice will be given. For detailed information the steel manufacturer or companies performing galvanizing or their national associations should be consulted. The outcome for the highest strength steels is greatly influenced by details of the galvanizing process and the application. It is important that the galvanizing company is informed about the steel type.
The main problem for high strength steels is hydrogen cracking or zinc penetration into the grain boundaries of the parent material. Local stress concentrations or residual stresses from welding, gas cutting or cold forming will increase the risk. These problems may occur even for steels of the S355J2 type. Stress relieving and an abrasive water jet instead of flame cutting may reduce the problem.

The risk for hydrogen cracking increases with the material hardness or tensile strength. S420M and S460M are thus more suitable than S690Q. For the latter both successful and unsuccessful applications exist and hot-dip zinc coating is in general not recommended for this and other martensitic steels.
In order to reduce the hydrogen-induced cracking phenomena, the cleaning of the surface in an acid bath prior to galvanising should be reduced to a minimum or better omitted completely. Instead shot blasting is preferable to remove oxides. A special fine-blasting may be performed to reduce or eliminate the time in the pickling bath where the hydrogen pick up occurs. Different types of pickling agents and cleaning methods exist which will reduce the problem.
Hot dip zinc coating gives a zinc layer of 70 to 80 µm with a metallic bonding. There is a risk for zinc penetration into the grain boundaries of the parent material due to the thermal stresses created.
Electroplating is a less severe process which may be used for smaller parts. The thickness of the zinc layer is in the range 5 to 25 µm.
For steels with a yield strength above 650 MPa a decrease of the fatigue strength of the material after galvanizing has to be taken into account. This is due to minor cracks in the zinc layer acting as crack initiation sites.

5.1.5 Conclusions

Extra high strength steels with yield strengths above 355 MPa and up to 690 MPa are well suited to 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 the 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 of high-performance

steels will be shown. A discussion on the limiting value of yield-to-tensile 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 characteristics 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 characteristics of test pieces in tension show full ductile behaviour and nonlinear 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.

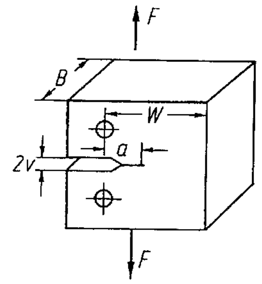

Fig. 5.2.1: CT-specimen for determining J-, CTOD- or K-values

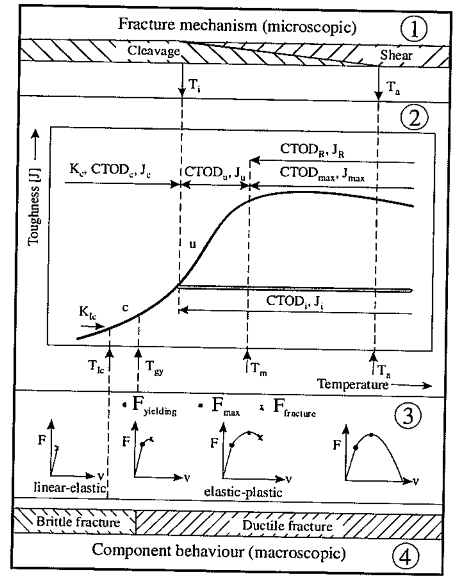

Fig. 5.2.2: Toughness-temperature curve and related load-deformation curves for tension elements using various parameters for toughness properties [5.4]

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.

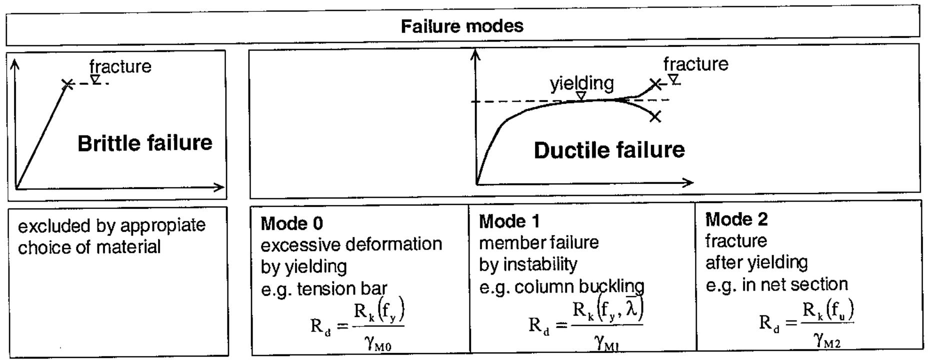

Fig. 5.2.3: Ductile and brittle failure modes for structural design

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 Mechanics Safety Assessment to Avoid Brittle Fracture

5.2.2.1 Introduction

In the following the fracture mechanics 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 the one side, design values of fracture mechanics action effects $K^*_{appl,d}$ and, on the other, design values of fracture mechanics resistance $K^*_{mat,d}$, see Equation (1).

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

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 mechanics values K_V and values obtained from Charpy-V-notch impact tests, as specified in the suppliers' 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.

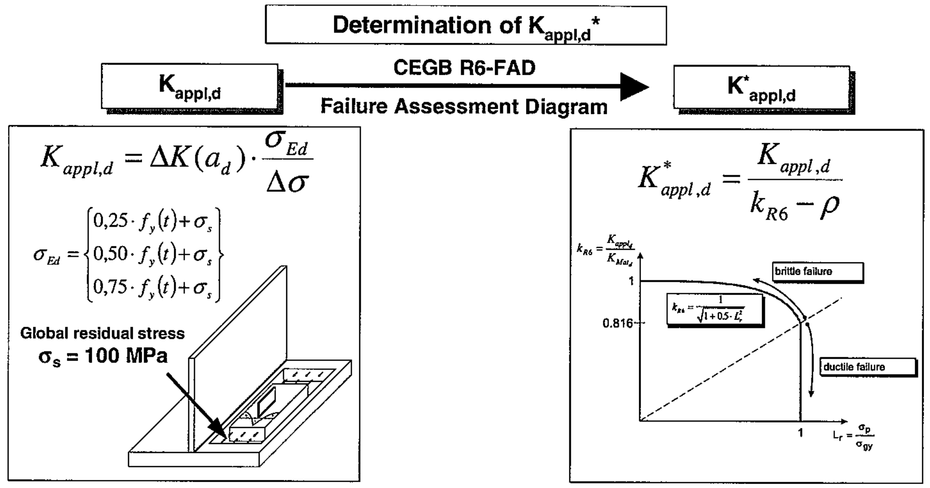

Fig. 5.2.4: Determination of toughness requirement $K_{*appl,d}$ [5.4]

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 = characteristic 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.

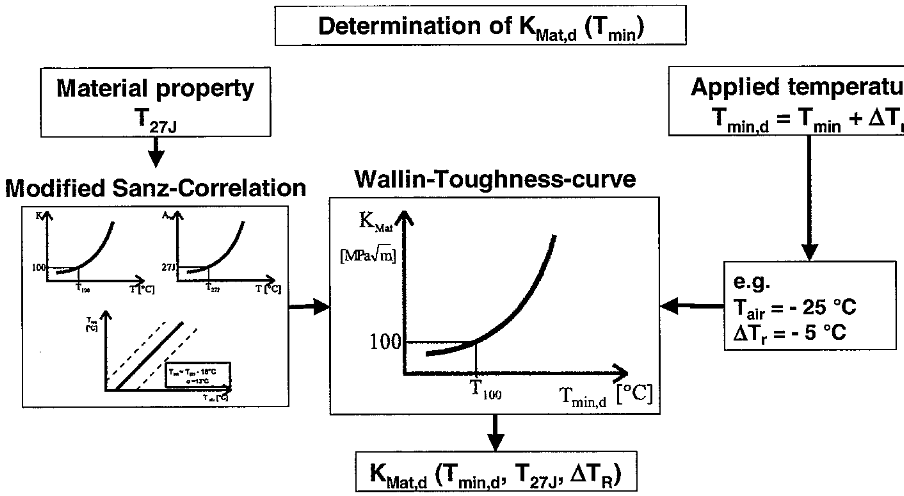

Fig. 5.2.5: Determination of toughness resistance $K_{Mat,d}(T_{min,d})$ [5.4]

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 has 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 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 of crack, and stress σ_{Ed}, resulting in ΔT_σ,
- the additive safety element ΔT_R,
- the influence of strain rate ΔT_ε,
- the influence of cold forming $\Delta T_{\varepsilon pl}$.

Details of the calculation are given in Figure 5.2.6.

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

Assessment scheme

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

Action side

$T_{Ed} = T_{min} + \Delta T_r + \Delta T_\sigma + \Delta T_R \left[+ \Delta T_\varepsilon + \Delta T_{\varepsilon pl} \right]$

- 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

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

- Additive safety element

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

---

may be supplemented by

- Influence of the strain rate

$$\Delta T_\varepsilon = -\frac{1440 - f_y(t)}{550} \cdot \left(\ln \frac{\varepsilon}{\varepsilon_0} \right)^{1{,}5} \ [°C]$$

  with $\varepsilon_0 = 0{,}0001\ s^{-1}$

- Influence from cold forming

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

  with DCF = Degree of Cold Forming [%]

Resistance

$T_{Rd} = T_{100}$

- 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 Material Choice

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.

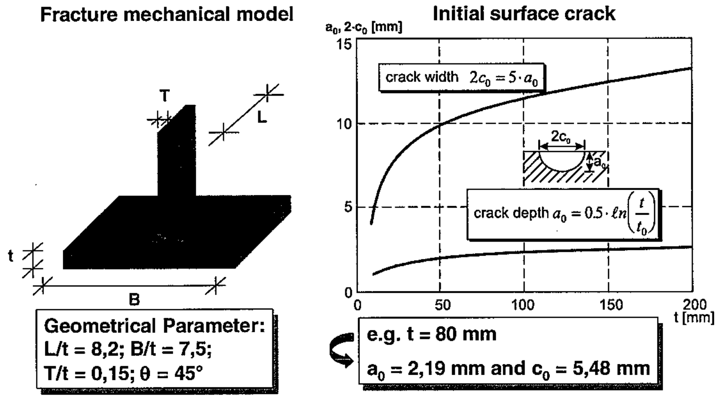

Fig. 5.2.7: Assumptions for details and initial sizes of surface cracks [5.4]

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.

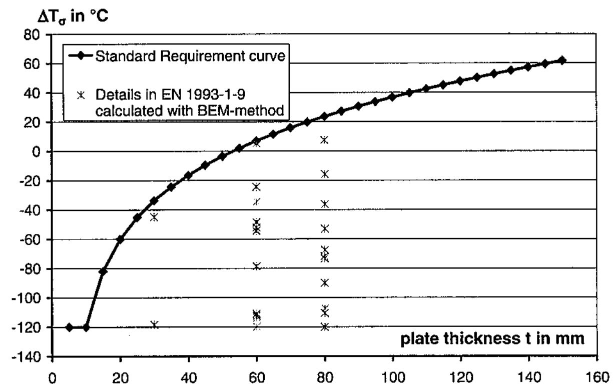

Fig. 5.2.8: Enveloping standard toughness requirement curve for details according to EN 1993-1-10 [5.4]

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

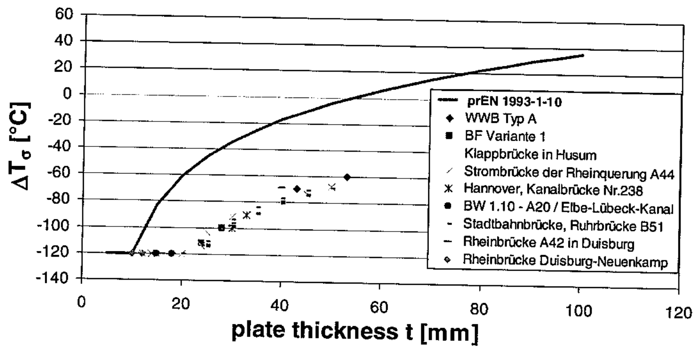

Fig. 5.2.9: Comparison of toughness requirements for steel bridges with the standard requirement curve [5.4]

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 at T °C	charpy energy CVN J min.	applied temperature T_{Ed} in °C, $\sigma_{Ed}=0{,}25 \cdot f_y(t)+\sigma_s$: 10	0	-10	-20	-30	-40	-50	$\sigma_{Ed}=0{,}50 \cdot f_y(t)+\sigma_s$: 10	0	-10	-20	-30	-40	-50	$\sigma_{Ed}=0{,}75 \cdot f_y(t)+\sigma_s$: 10	0	-10	-20	-30	-40	-50
			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-Tensile Strength Ratio Requirement

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

$$\frac{f_u}{f_y} > 1.10 \quad \text{or} \quad \frac{f_y}{f_u} \leq 0.90 \qquad (4)$$

is recommended.

Fig. 5.2.11 gives the yield/tensile 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 penalises the use of high-performance steels in structural applications and can be shown to be of no relevance.

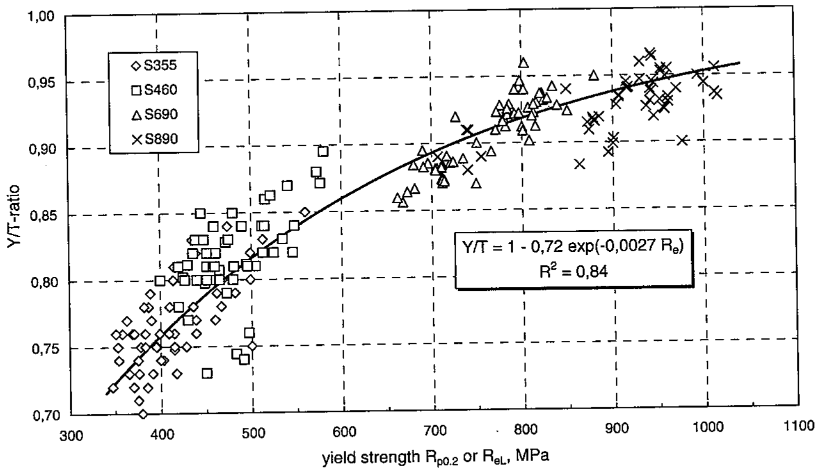

Fig. 5.2.11: Yield-to-tensile ratio of low and high strength ferritic steels depending on the yield strength [5.8]

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.

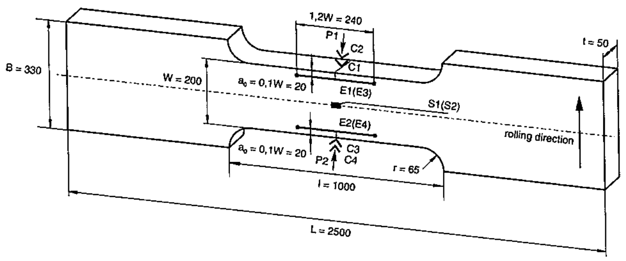

Fig. 5.2.12: Component-like large scale specimen with measuring devices [5.7]

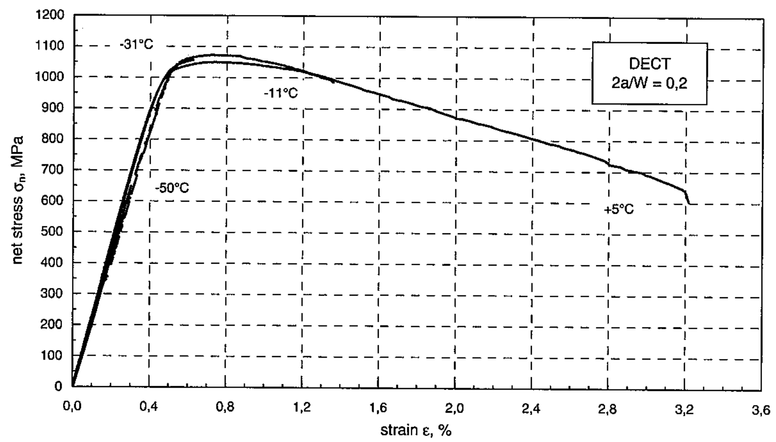

Fig. 5.2.13: Typical net stress temperature-yielding curves of steel [5.7]

Fig. 5.2.14 gives the maximum net section stresses versus 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.

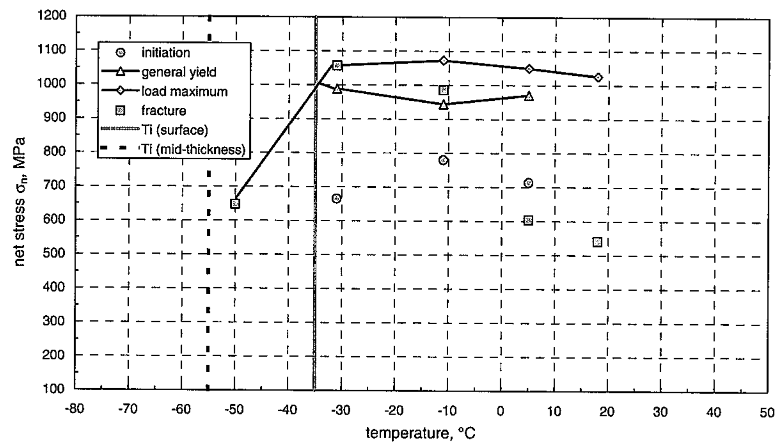

Fig. 5.2.14: Net stress temperature curve of steel [5.7]

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.

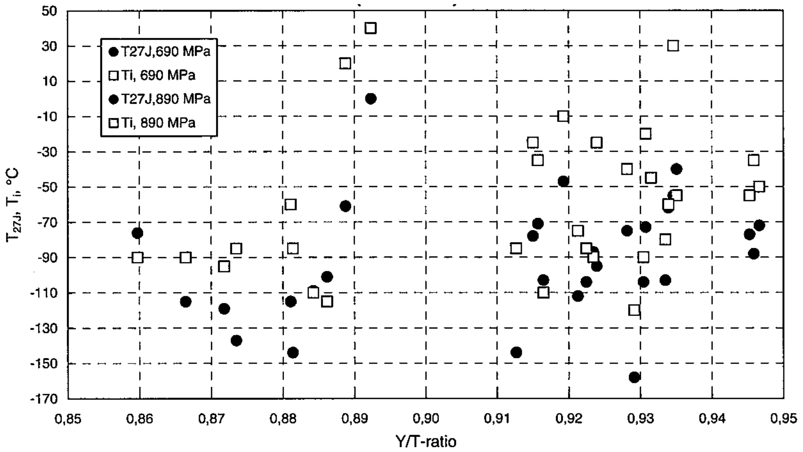

Fig. 5.2.15: Toughness properties depending on the yield to tensile strength ratio for S690 and S890 [5.7]

5.3 Buckling Resistance of Structures of High Strength Steel

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5.3.1 Introduction

The resistance to instability frequently governs the design of steel structures. This is an obstacle to the use of high strength steel as the resistance to instability increases more slowly than the yield strength or in the 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 costs increase 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 costs 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 USA. 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 proportional to the square of the plate thickness and a reduction of the plate thickness gives a substantial reduction of the cost. A reduced weight reduces costs for handling and transportation. On the other hand, increased wear of drills increases the cost for drilling holes. Here only the cost of material will be studied.

Present design rules are mainly based on tests with low strength steel (S235-S355). There are several indications that the rules can be improved in order to take full advantage of high strength steel. Some examples will be given below.

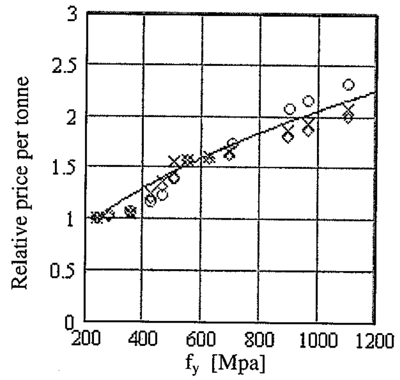

Fig. 5.3.1: Approximate price per tonne of hot rolled steel normalised with price of S235 as function of yield strength

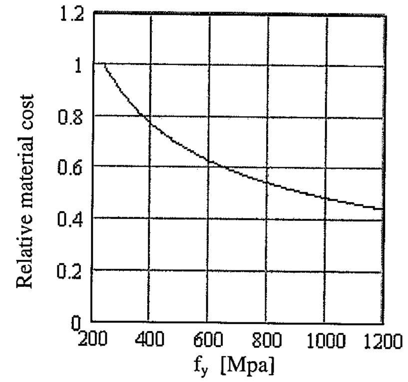

Fig. 5.3.2: Approximate material cost normalised with the cost of S235 assuming that the strength can be fully utilised

5.3.2 Eurocode 3 Format for Resistance to Instability

The design procedures for instability in Eurocode 3 [5.13], [5.14], as well as in many other codes, have a general format that includes the following parameters:

1 A yield resistance F_y or a yield strength f_y

2 A slenderness parameter:

$$\lambda = \sqrt{\frac{F_y}{F_{cr}}} = \sqrt{\frac{f_y}{\sigma_{cr}}} \tag{1}$$

where F_{cr} is the elastic buckling force and σ_{cr} is the same but expressed as a stress

3 A resistance function $\chi(\lambda)$ which reduces the yield resistance for λ larger than a certain limiting value

$$F_R = F_y \cdot \chi(\lambda) \tag{2}$$

Alternatively, a reduction of the gross area A_{gr} to an effective area is used

$$A_{eff} = \rho \, A_{gr} \tag{3}$$

The format using force and strength reduction is used e.g. for flexural buckling, shear buckling and patch loading. The concept of effective area is used for plate buckling due to direct stress. This is based on the assumption that failure will not be reached until the plate yields at the supported longitudinal edges. The reduction of the cross-section is thought of as a reduction of the width, while the thickness is retained.
The resistance function is usually obtained by curve fitting to experimental results. There are several different mathematical expressions that are used for this purpose. Some examples from Eurocode 3 are shown in Fig. 5.3.3 together with two reference curves; the Euler curve $\chi = 1/\lambda^2$ and the von Karman curve $\rho = 1/\lambda$.

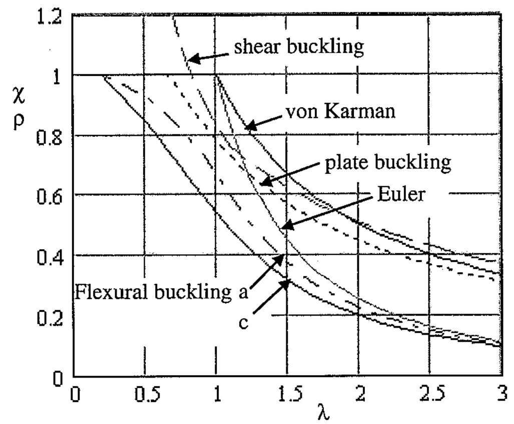

Fig. 5.3.3: Buckling curves according to Eurocode 3 [5.13], [5.14]

5.3.3 Resistance to Flexural Buckling

The resistance to flexural buckling of a column with cross section class up to 3 [5.13] is given by

$$N_R = \chi f_y A$$

$$\chi = \left[\Phi + \sqrt{\Phi^2 - \lambda^2}\right]^{-1} \tag{6}$$

$$\Phi = 0.5(1 + \alpha(\lambda - 0.2) + \lambda^2)$$

The imperfection factor α is for instance 0.21 for curve a and 0.49 for curve c. It reflects the column's sensitivity to imperfections, mainly residual stresses and out of straightness. In order to investigate the effect of a change in the yield strength it is useful to differentiate the resistance

$$\frac{dN_R}{N_R} = \left[\frac{\partial \chi}{\partial f_y}\frac{f_y}{\chi} + 1\right]\frac{df_y}{f_y} \tag{7}$$

The expression in brackets gives a non-dimensional measure of how the resistance changes when the yield point is changed.

$$\delta = \frac{\partial \chi}{\partial f_y}\frac{f_y}{\chi} + 1 \tag{8}$$

The measure δ depends on the geometrical slenderness l/i of the column and an example is shown in Figure 5.3.4 for buckling curve a. The diagram shows δ as function of f_y and $\delta = 0$ means that the resistance does not increase with f_y. $\delta = 1$ means that the resistance is directly proportional to f_y. As expected, the increase in the resistance is quite substantial for low geometric slenderness and small for high slenderness. For a column with l/i = 40 and f_y = 400 MPa Fig. 5.3.4 gives $\delta = 0.90$ which means that a 10% increase in f_y will give a 9% increase in the resistance.

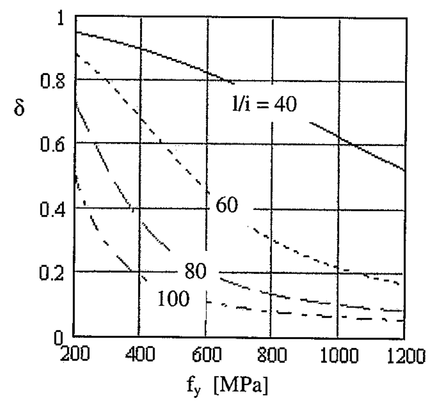

Fig. 5.3.4: Relative change in resistance for flexural buckling (curve a) for a relative change in yield strength δ as function of yield strength f_y

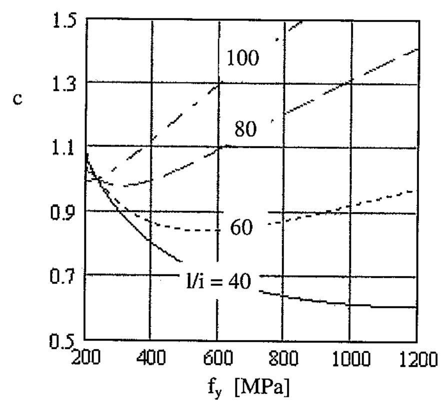

Fig. 5.3.5: Relative material cost for column subject to flexural buckling (curve a) as function of yield strength f_y. Reference cost is for steel S235

The relative material cost of a column that carries a certain load using S235 as reference and with a material cost according to Figure 5.3.1 can be written as

$$c = \frac{\chi(235)\sqrt{235}}{\chi(f_y)\sqrt{f_y}} \tag{9}$$

This relation is shown in Fig. 5.3.5 for buckling curve a. If the cost index c is smaller than one, the material cost is lower than that for steel S235. It can be seen that the curves have a minimum for a certain yield strength. For a stocky column with l/i = 40 this minimum occurs at a yield strength above the practical range. Such slenderness is typical for columns in a braced high rise building and it is clear that high strength steel is economical for such applications. This is also confirmed by practice in the USA. For l/i = 100 the minimum is at about 200 MPa and such a column may occur in a single storey unbraced building. This is obviously not a market for high strength steel.
The above conclusions are based on material cost and they are applicable to the total cost only if components of the same type are compared. Columns with moderate loads are usually made of hot rolled H-profiles, which are available in S235 to S460. For high loads welded box columns are common and for those the comparison is relevant for the whole range S235 to S1100. It has also been assumed that the column has a fixed geometrical slenderness and that its area is reduced when the yield strength is increased. This would be true, for instance, for a box or RHS column if the area reduction is caused by reducing the wall thickness. If, instead, the overall dimensions are

decreased the geometrical slenderness will increase and the effect of increased yield strength will be less favourable.
The study has so far used the design rules of the current Eurocode 3. There are several studies showing that the effects of imperfections such as out of straightness and residual stresses are less severe for high strength steel. The effect of bow imperfection can easily be demonstrated by a calculation of the resistance according to elastic theory. The result of such a calculation is shown in Fig. 5.3.6 for buckling in the strong direction of a H-profile with bow imperfection l/1000 and no residual stresses. It can be seen that the relative resistance is higher for a higher yield strength. It should be noted that the curves do not give a correct resistance as the effect of residual stresses is not included. The residual stress expressed as a fraction of the yield strength is the relevant parameter. For high strength steel the residual stresses are a smaller fraction of the yield strength and therefore the detrimental effect is smaller for high strength steel than for ordinary steel.

A method to take the increase in the relative resistance into account has been proposed [5.16] in which the imperfection factor is reduced with the yield strength according to

$$\alpha = \alpha_0 (235/f_y)^n \tag{10}$$

Here α_0 is the values of α in Eurocode 3. The first proposal was to put n = 0.8 but it was later modified to n = 1.0. The proposal was based on studies with steel grades up to S400. It is likely to be applicable to higher grades but this remains to be proven by further research. The potential of this proposal with n = 1 is shown in Fig. 5.3.7 giving the material cost index. A comparison with Fig. 5.3.5 shows a notable improvement. For example a column with l/i = 60 has a minimum cost index of 0.85 for a yield strength about 500 MPa according to Fig. 5.3.5 and 0.72 at about 700 MPa according to Fig. 5.3.7.

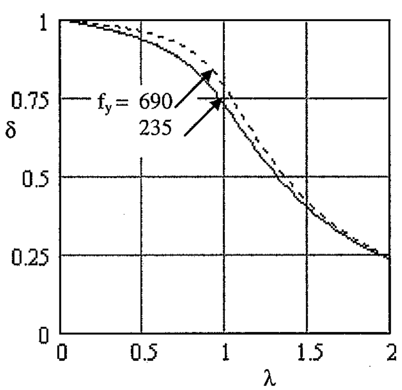

Fig. 5.3.6: Resistance to flexural buckling in the strong direction according to elastic theory for a column with bow imperfection l/1000 and no residual stresses

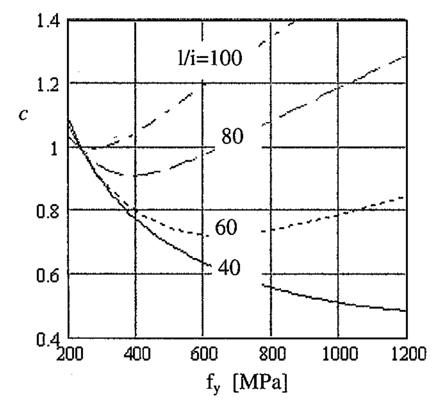

Fig. 5.3.7: Relative cost for column subject to flexural buckling (curve a modified according to eq. (10)) as function of yield strength f_y. Reference cost is for steel S235

5.3.4 Resistance to Local Buckling

The resistance to local buckling is given in Eurocode 3-1-5 [5.14]. For a plate simply supported along all edges with uniform compression the effective area is according to (3) with

$$\rho = 1.0 \qquad \text{if } \lambda < 0.673$$

$$\rho = \frac{\lambda - 0.22}{\lambda^2} \qquad \text{if } \lambda > 0.673 \tag{11}$$

$$\lambda = \frac{b}{56.8t}\sqrt{\frac{f_y}{235}} \tag{12}$$

Following the same procedure as for flexural buckling the change in resistance and the relative material cost for a plate subjected to local buckling can be calculated with result according to Fig. 5.3.8 and Fig. 5.3.9. The change in resistance for a unit change in yield strength is quite moderate unless the plate is stocky. For the plate with b/t = 30 there is a discontinuity at about f_y = 400 MPa, which is caused by the abrupt change in slope at λ = 0.673. The material cost for slender plates is almost independent of the yield strength.

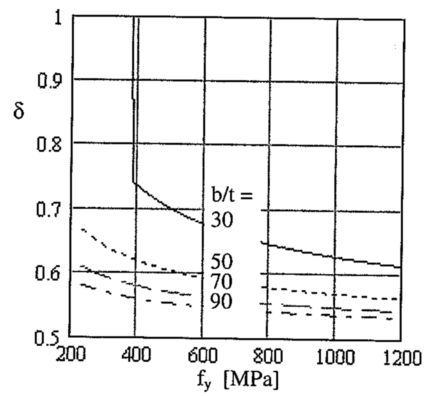

Fig. 5.3.8: Relative change in resistance for a plate simply supported along all edges and subject to uniform compression for a relative change in yield strength δ as function of yield strength f_y

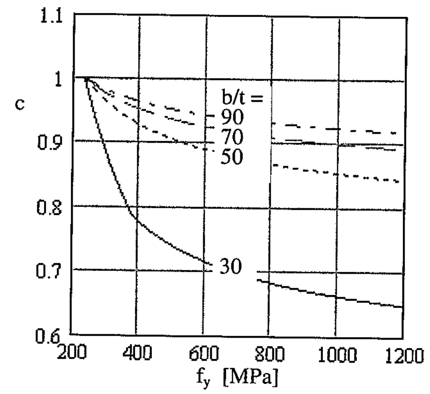

Fig. 5.3.9: Relative cost for a plate simply supported along all edges and subject to uniform compression as function of yield strength f_y. Reference cost is for steel S235

In order to get an incentive for using high strength steel the plate slenderness has to be kept low. One possibility for large plates is to provide longitudinal stiffeners, which is the traditional solution, e.g. for a flange in a box girder. The welding of such stiffeners is quite expensive and the economy is questionable. A better solution is to make the flange composite by adding shear studs and concrete. For smaller profiles it is also possible to use folds as stiffeners, see Fig. 5.3.10. A fold of 15 to 20 degrees is usually sufficient to act as a support for the adjacent panels.

The discontinuity of slope in the reduction function (11) is not a physical reality but only a consequence of how the test results have been approximated. The scatter in test results makes it impossible to see the actual shape, but a good theoretical solution makes this clear. Fig. 5.3.11 shows such a solution for the buckling of a flange [5.17] and it shows that the reduction function is smooth. It also shows that the reduction is less severe for high strength steel. A possible way to take this into account is to use the format for reduction functions proposed in [5.15]. However, the imperfection factor and plateau length is here made dependent on the yield strength.

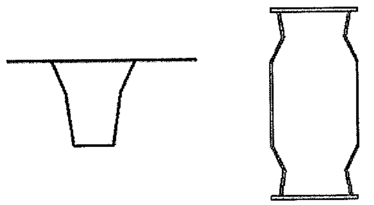

Fig. 5.3.10: Example of profiles with stiffeners of cold formed folds. Left; Stiffener of deck plate in S1100. Right; Launching beam for military bridge in S690

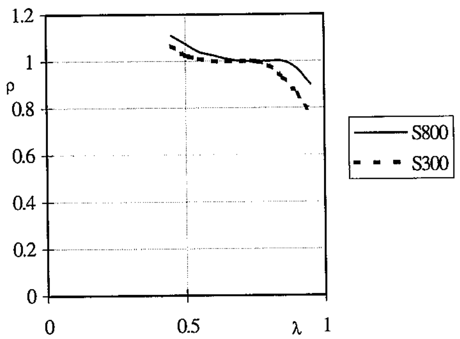

Fig. 5.3.11: Reduction factor of yield resistance with respect to flange buckling as a function of the slenderness parameter $\lambda = \sqrt{(f_y/\sigma_{cr})}$ [5.17]

$$\rho = \left[\Phi + \sqrt{\Phi^2 - \lambda}\right]^{-1}$$
$$\Phi = 0.5(1 + \alpha(\lambda - \lambda_0) + \lambda) \tag{13}$$

$$\alpha = \alpha_0\sqrt{235/f_y} \tag{14}$$

$$\lambda_0 = 0.67\sqrt[4]{f_y/235} \tag{15}$$

As an example buckling curves with $\alpha_0 = 0.13$ is shown in Figure 5.3.12. The modification of the imperfection factor and the plateau length are somewhat speculative and correct values have to be determined by further research. So far they are only based on indications. One such indication is shown in Figure 5.3.10 and another in Figure 5.3.13. The latter shows that results for S1100 are consistently higher than those for low strength steel. Figure 5.3.13 includes tests with and without residual stresses. The tests with S1100 were made on welded specimens and should be compared with the squares.

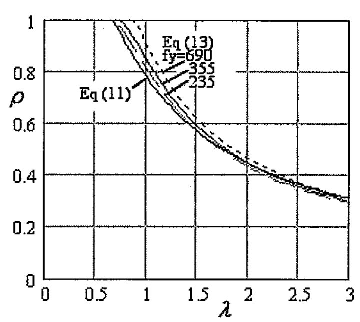

Fig. 5.3.12: Reduction factor for local buckling according to Eurocode 3-1-5 compared with Eq. (13) with $\alpha_0 = 0.13$

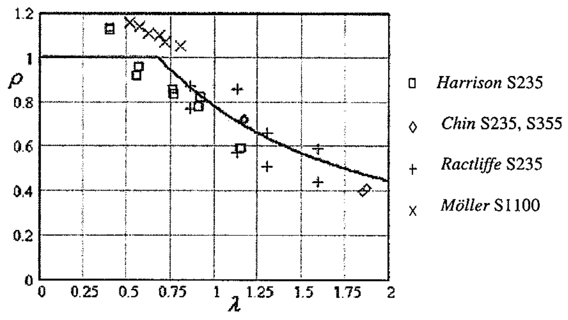

Fig. 5.3.13: Test results for plate buckling due to uniform compression for low strength steel and high strength steel [5.18]

5.3.5 Resistance to Shear Buckling

Eurocode 3-1-5 gives rules for shear resistance taking buckling into account. Only the resistance from the web will be considered here. The web is assumed to be unstiffened except at the support and the end stiffener is assumed to be flexible. Following the same procedure as before the relative material cost comes out as shown in Fig. 5.3.14. There is a cost reduction for stocky webs but such are not used for plate girders.

The conclusion is that there is little to gain from using high strength steel in a girder web and the solution is to use a hybrid girder with high strength steel only in the flanges. An example is shown in Fig. 5.3.15. When the yield strength is reached in the flanges the stresses in the web are limited by its lower yield strength. This gives a small loss in bending resistance but the cheaper material in the web compensates. Eurocode 3-1-5 gives rules for hybrid girders and recommends that the yield strength of the flanges should not be larger than twice that of the web. A summary of the design procedure including simplified formulae can be found in [5.19].

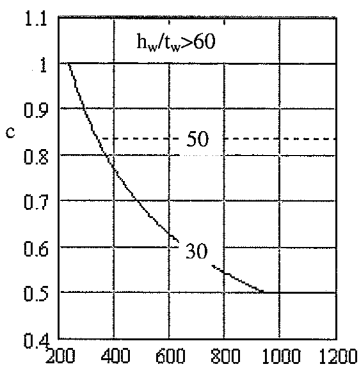

Fig. 5.3.14: Relative cost for web with flexible end stiffener subject to shear

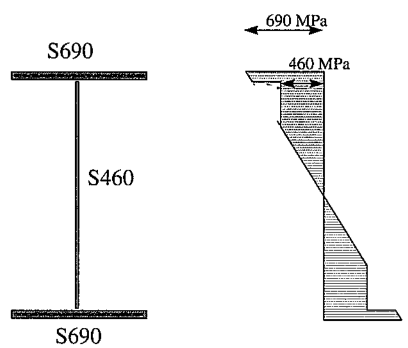

Fig. 5.3.15: Example of hybrid girder with web of lower strength than the flanges

5.3.6 Discussion and Conclusions

This chapter has been devoted to the buckling resistance of structures of high strength steel with special focus on economic aspects. The economic comparisons are simplified in the sense that only the cost of material has been considered. The fabrication costs are more important but they may be affected both up and down by increasing the strength and they have to be checked for each specific application. In any case the cost comparisons give indications when it is likely to be favourable to use high strength steel.

- Columns in braced multi-storey frames are favourable for high strength steel and the more so the heavier load they carry.
- Slender plates in compression should be avoided. Preferably the slenderness should be within the plateau length. One cheap way of reducing the slenderness is cold formed folds and another is to make a curved surface instead of a flat one.
- Girders in bending should preferably be made hybrid, i.e. with flanges in high strength steel and webs of lower strength than the flanges.

Some indications have been given about future improvements in the design rules in order to make better use of the high strength steel and reduce the costs. The following tasks for further research seems promising:

- The imperfection factor governing the resistance to flexural buckling is likely to become smaller as the strength increases. This would make columns of high strength steel more competitive. This is likely to apply also to lateral torsional buckling.
- The plateau length for plate buckling is likely to increase with increasing yield strength and this may be taken into account in a similar way as for flexural buckling.
- Design by FEM makes it possible to account for the better performance of high strength steel but generally accepted rules for how it should be done are still lacking.

5.4 Improving the Fatigue Resistance

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5.4.1 Introduction

High strength steels are intended for structures subjected to high stress levels to create lighter and more slender structures. But, as the design stress level is increased to take advantage of the high strength, so does the ratio between variable and permanent loads and thus the magnitude of the stress variations. In welded structures, in order to keep a balance between static and fatigue design, more emphasis has to be put on the latter since the fatigue resistance does not increase in the same manner as the steel tensile strength. For the details located in the critical sections that cannot fulfil the safety criteria, the solution can be:

- New or modified detailing, displacement of details in less stressed sections,
- Improved welding procedures, better workmanship,
- Post-weld improvement methods.

The objective of this section is to provide practical information on these solutions, with an emphasis on post-weld improvement methods.
To explain why the fatigue resistance does not increase in the same manner as the tensile strength, one can take advantage of the analytical modelling of fatigue failure. Fatigue failure can be separated into the following two different phases:

1 Crack initiation: depending upon the material and stress level, various mechanisms take place to create voids and micro-cracks in the material. These micro-cracks progress through the material grains. Once the size of a micro-crack becomes important and it grows in a preferential direction, it will develop in a preferential way, unloading the other micro-cracks. In a steel structure, characteristic crack lengths of 0.5 to 1 mm are often considered.
2 Crack propagation: the presence of the crack creates a stress concentration at the crack front. The opening and closure of the crack corresponding to each stress cycle gives rise to large plastic slips which form striations on the fracture surface. The crack continues to propagate until a critical size corresponding to the instability of the cracked element is reached.

Crack initiation fatigue life is highly dependant upon material characteristics, as shown by the parameters in the Manson-Coffin relationship [5.20]. All other things being equal, crack initiation fatigue life in steels increases with tensile strength. On the other hand, crack propagation fatigue life in steels, particularly C-Mn steels, is independent of mechanical properties. Thus, the total fatigue life and the relative importance of the initiation versus the propagation fatigue life depends on the presence of defects which, according to their size and shape, reduce or suppress the initiation period.
In machined steel parts, it is common to say that the higher the steel grade, the better its fatigue strength since the initiation period is the largest part of the total fatigue life. For welded structures, on the contrary, the effects of notches, defects, and corrosion reduce the fatigue strength, by reducing the number of cycles needed to initiate a fatigue crack. That is, the largest part of the fatigue life comes then from the crack propagation period. As a consequence, the fatigue strength of welded structures made out of high steel grades is not higher than that of mild steel (when both have the same defects). This is reflected in most structural codes, for example Eurocode 3 [5.21]. There may even be an inverse material dependency, i.e. welded elements using high strength steels may have a lower fatigue strength in the high cycle region, as mentioned in section 5.1 and [5.22]. But it can be argued that the welding of high strength steels requires more skilled fabricators. As modern welding procedures and good workmanship results in better weld geometry and fewer defects, it comes not as a surprise that some studies, for example [5.23], [5.24], show a better fatigue resistance for welded high strength steel elements.
Finally, fatigue life is also influenced by the fracture toughness of the material (see also section 5.2). In this respect, thermo-mechanical steels (TM steels, often called "low alloy steels", according to EN 10025 Part 4), are very interesting. TM steels have excellent toughness properties, with values higher than 50 J at -20°C (or even lower

temperatures). This results in longer allowable crack sizes in components. This means a longer crack growth period (extended lifetime), less frequent inspection intervals and cracks that are easier to detect. Furthermore, because of lower carbon content, preheating before welding can be reduced or even be dispensed with in the case of TM steels. All these aspects have a significant economic impact and make TM steels very competitive [5.26].

5.4.2 Detailing

Stress concentrations depend on the shape of the component and on the manufacturing process. They occur at corners, loading positions, abrupt section changes, etc. Indications of good fatigue resistant design are given for example in an ECCS publication [5.27]. In order to get better fatigue strength with high strength steels, it is essential to follow the precautions given below:

- Avoid structural discontinuities in highly stressed regions.
- Put welds and details in zones near the neutral axis or where the mean stress is compressive.
- Design details where bending moment is minimized, for instance by avoiding misalignment or offset, which causes secondary bending stresses (example: converging axes of truss diagonals and chords).
- Avoid the combination of several stress concentrations in the same region, like welds in zones affected by holes, tapering, attachments, etc., as this increases further the stress concentration factor.
- Do not use fillet or partial penetration welds in load carrying welded joints but full penetration welds.
- For welds made from one side only (root not accessible), use a backing strip to ensure better root weld.
- Avoid using stiffeners, except at supports, if the self-weight increase of the panel without stiffeners is only 10 to 15 % more than the weight of the original stiffened panel (web and flanges); this design will, in the end, be more economical and fatigue-resistant.
- Change detail from a welded to a bolted shear connection.
- Ensure that support stiffeners are at the axes of the supports.

5.4.3 Welding Procedures and Workmanship

As stated in the introduction, the requirements (steel fabricator, supervisor, welder) for welding high strength steels are set higher, which should help to have structures with better fatigue strength. This is however not sufficient and it also requires state of the art production processes (see section 5.1). Manual arc welding for example should if possible not be used and, for at least longitudinal welds, automatic welding machines should be used. With fine-grained steels, since they are very sensitive to welding parameters, it is very important to follow the rules given in EN1011-2 [5.28] for pre- and post-heating temperatures, etc. to avoid the forming of large grains and martensite. In addition, a couple of precautions about the fabrication procedure such as the systematic use of backing strips were already mentioned in the previous paragraph.

In the case of welds not meeting the quality requirements, a fitness-for-purpose assessment shall be carried out as a first step, especially for high strength steel welds since repairing may worsen the problem. If it shows that the weld needs to be repaired, the repair procedure must be well thought out and approved by a welding engineer. For all steels, most repair welding is made using manual arc welding. The defects created, in particular at the start and stop positions, exceed those from other welding processes and therefore make it difficult to recover the original quality from automatic welding. In the case of high strength steels, this problem is emphasized because the consequences of downgrading the fatigue strength (fatigue class) of a welded detail are larger. In some cases, solutions such as post-weld improvement methods (grinding for example) can be proposed. The latter is the subject of the next paragraph. Some information about good workmanship can be found for example in ECCS recommendations [5.27].
To improve the fatigue strength, new developments such as low temperature transformation welding material may also be considered (see section 4.5).

5.4.4 Post-Weld Improvement Methods

The methods consisting in treating the **weld toe regions** of a joint after welding have been studied by various industries, in particular the offshore industry [5.29]. Large databases of test results can be found in the literature, for example [5.30]. These post-weld improvement methods beneficially influence, within certain limits, the fatigue strength of the improved joint. For the reasons given in the introduction, they are especially efficient in the case of welded structures made out of high strength steels. Numerous improvement methods exist and we will only deal with four of them, which are among the most popular ones, i.e.:

- Grinding;
- Tungsten Inert Gas (TIG) dressing of the weld toe;
- Needle peening;
- Hammer peening.

The main parameters responsible for the beneficial influence of the improvement methods are the following

1 Reduction of local stress concentrations;
2 Creation of a crack initiation phase;
3 Alteration of the residual stress field of the superficial layer.

Consequently, the improvement methods can be divided into two distinct groups:

1 Methods which smooth the weld bead-base plate transition (parameter 1) and eliminate surface defects (parameter 2), i.e. grinding and TIG dressing.
2 Methods which change tensile residual stresses into compressive stresses around locations of potential fatigue cracks (parameter 3), i.e. needle and hammer peening. Note that in some cases these methods can also smooth the weld-plate transition but this is not their primary intended goal.

The beneficial influence of an improvement method on the fatigue strength can be represented on a standard S-N diagram (for constant amplitude loading) by both a translation and a rotation of the resistance curve, see Fig. 5.4.1. As can be seen, the fatigue class of the improved detail is raised by both translation and rotation. Parameter 1 is responsible only for a translation and parameters 2, 3 for both a translation and a rotation. One must be careful because there are several restrictions to the benefits of the improvement methods.

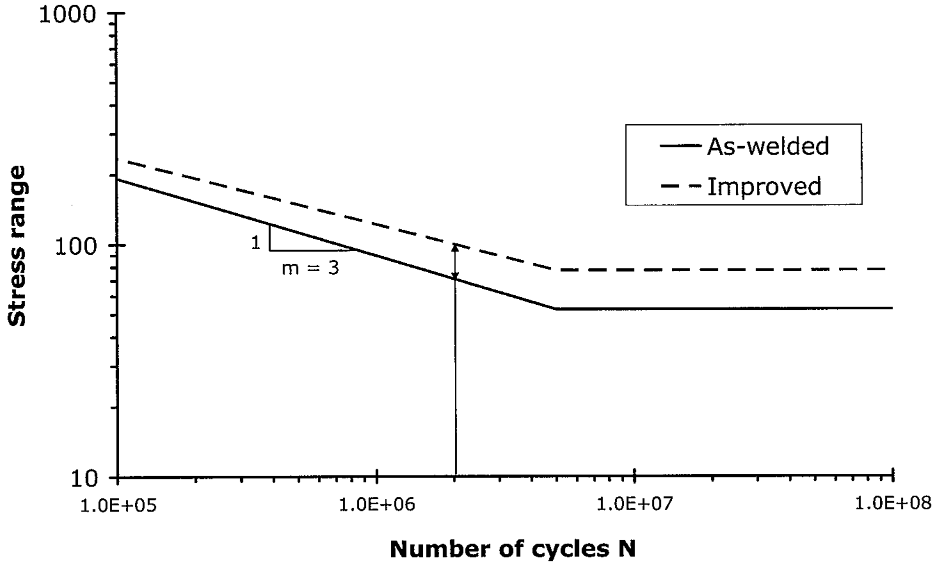

Fig. 5.4.1: Influence of the use of an improvement method on the S-N curve of a welded joint

All improvement methods have in common the potential problem that a crack may initiate at another location than the treated area, for example a crack may develop from the root in a cover-plate weld, or from blow-holes in load-carrying fillet welds. In such cases, the benefit obtained may be negligible [5.29]. When considering the use of improvement methods, the possibility of crack initiation at other locations must always be considered. All the same, successful implementation of these methods depends on the proper training of operators as well as inspectors.

Currently no international code contains rules to cover post-weld improvement methods. The only exception is grinding, as it is included in the requirements for a few detail categories (e.g. butt welded joints). Some recommendations do exist however, such as the ones written by Maddox and Haagensen within Commission XIII of the International Institute for Welding [5.31]. These cover steels of yield strength up to 900 MPa, including austenitic steels and will serve as a basis in the following paragraphs. The reader is referred to these recommendations for equipment, procedures, inspection and quality control criteria.

5.4.4.1 Grinding and TIG Dressing

In order to take into account both the improvement in the geometry and the introduction of an initiation phase due to the improvement method, the simplest is to use the existing set of fatigue categories. Unfortunately, when doing the statistical analysis of

the test results, one may find that the slope coefficients differ from the usual fatigue slope coefficient 3, which is not surprising (see Fig. 5.4.1). Currently, it is difficult to justify a different slope coefficient for each improved joint fatigue curve. Therefore, the fatigue curves for improved joints are simple translations from the as-welded code curves, i.e. without change in the slope coefficient.

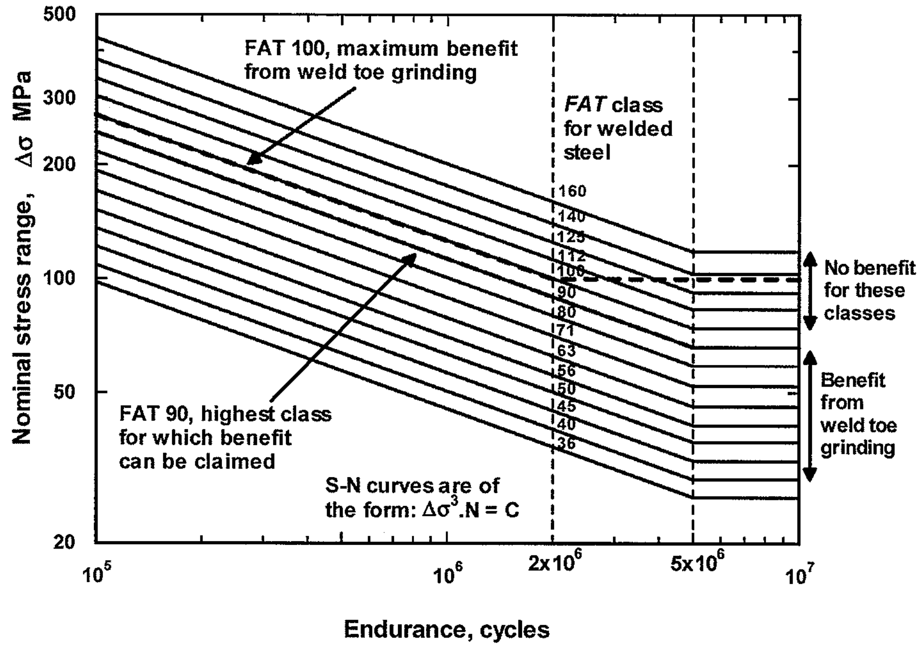

Fig. 5.4.2: Design S-N curves for weld toe burr ground welds in steel structures, (extract from [5.31])

For **burr grinding,** the proposal from the International Institute of Welding (IIW/IIS) [5.31] can be summarized as follows (see Fig. 5.4.2):
- Benefit only if category of as-welded joint is equal to or lower than 90.
- If yield strength is lower than 350 MPa, increase in fatigue strength by a factor 1.3 up to a maximum of category 100.
- If yield strength is equal to or higher than 350 MPa, increase in fatigue strength by a factor 1.5 up to a maximum of category 100.

The first limitation is due to the fact that the higher categories include non-welded details, details whose lives are not governed by weld toe failure or the welds that have already been improved, e.g. by grinding the weld flush with the surface.

In the case of **TIG dressing,** the proposition is very similar, and can be summarized as follows:
- Benefit only if category of as-welded joint is equal to or lower than 90.
- If yield strength is lower than 350 MPa, increase in fatigue strength by a factor 1.3 up to a maximum of category 112.
- If yield strength is equal to or higher than 350 MPa, increase in fatigue strength by a factor 1.5 up to a maximum of category 112.

For both improvement methods, the level of improvement is not significantly influenced by the R ratio ($R = \sigma_{min}/\sigma_{max}$).

In the case of TIG dressing, it is interesting to compare this above proposal with the results reported by Dahle [5.32] concerning two inter-Nordic projects conducted on welded plates in high strength steels with longitudinal attachments (see Fig. 5.4.3). In these projects, four steels of nominal yield strength 350, 590, 700 and 900 MPa were welded and TIG-dressed. The first two are micro-alloyed structural steels and the last two quenched and tempered low-alloyed steels. Tests on as-welded and TIG-dressed specimens were carried out under both constant and variable amplitude. Dahle found that the influence of the yield strength on the fatigue category (CAT), for design curves, can be given by the following relationship:

$$CAT_{increase} = (0.001056\, f_y + 0.65982)\, CAT \qquad (1)$$

where f_y and CAT are in MPa.

This relationship as well as the IIW have been plotted in Fig. 5.4.3. Dahle's increase seems lower, except for very high strength steels, but the reference value for the as-welded case (CAT 67) is higher than in IIW recommendations (CAT 56). Therefore, when computing the category for a 350 MPa steel detail, the resulting categories are almost identical (CAT_{IIW} 73 vs CAT_{Dahle} 70).

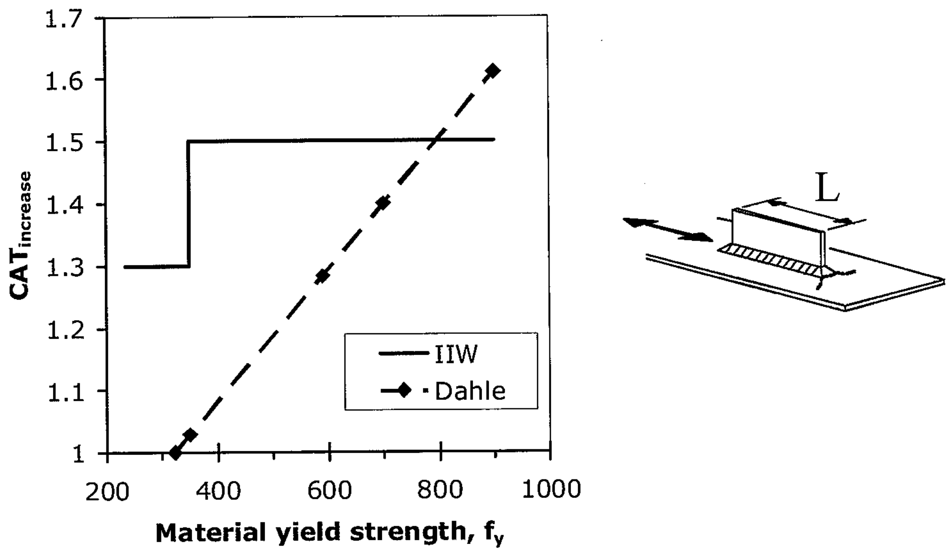

Fig. 5.4.3: Increase in fatigue category ($CAT_{increase}$) of longitudinal attachment due to TIG as a function of material yield strength

This comparison confirms the fact that high strength steels do benefit more from post weld improvement methods, in this case from TIG dressing.

5.4.4.2 Needle and Hammer Peening

Most studies on improved methods have kept the concept of the applied stress range, $\Delta\sigma$. The problem with this approach is that several S-N curves are needed for each detail class because the fatigue strength is R-ratio dependent. Therefore, the definition of the stress range has to be modified. In the IIW recommendations, the following change in the stress range definition is made:

$$\begin{cases} \Delta\sigma' = \sigma_{max}; & R \ge 0 \\ \Delta\sigma' = \sigma_{max} - \sigma_{min} = \Delta\sigma; & R < 0 \end{cases} \qquad (2)$$

Due to the sensitivity of hammer peened welded joints to applied mean stress (or stress ratio R), the higher S-N curves can only be used if the maximum nominal compressive stress in the load spectrum is lower than 0.25 f_y. Moreover, the stress ratio R must stay below 0.5 because under high stress ratios the compressive stresses introduced by the post weld improvement are eliminated by the applied loading.

For **hammer peening,** the proposition from the International Institute of Welding (IIW/IIS) [5.31] can be summarized as follows (see Fig. 5.4.4):

- Benefit only if category of as-welded joint is equal to or lower than 90.
- If yield strength is lower than 350 MPa, increase in fatigue strength by a factor 1.3 up to a maximum of category 125.
- If yield strength is equal to or higher than 350 MPa, increase in fatigue strength by a factor 1.6 up to a maximum of category 125.
- For structural elements with thicknesses above t = 20 mm, the benefit is reduced to a factor 1.5 on strength and to a maximum of category 100.

The last condition was set because fatigue tests on large-scale structures indicate a lower benefit from hammer peening than for small-scale specimens.

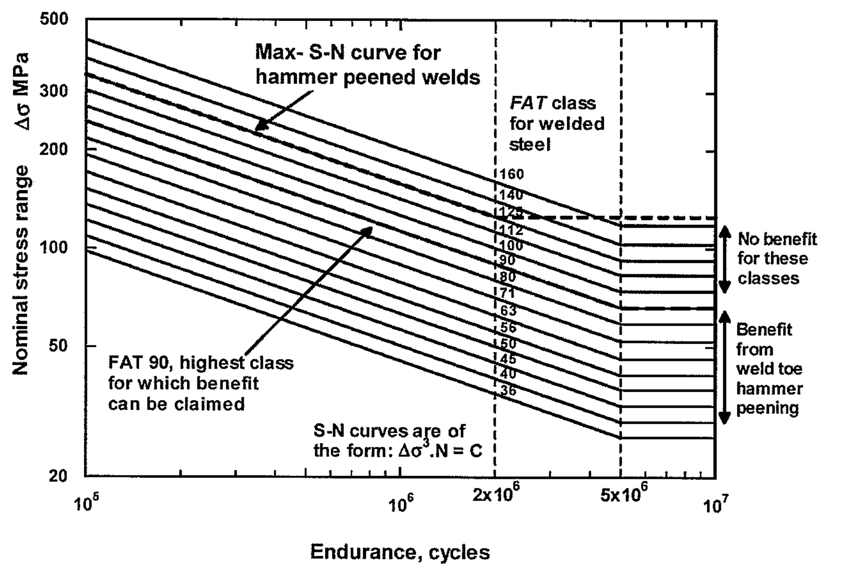

Fig. 5.4.4: Design S-N curves for weld toe hammer peened welds in steel structures (extract from [5.31]).

For **needle peening,** the proposition from the International Institute of Welding (IIW/IIS) [5.31] is identical to the one for hammer peening and can be summarized as follow:

- Benefit only if category of as-welded joint is equal to or lower than 90.
- If yield strength is lower than 350 MPa, increase in fatigue strength by a factor 1.3 up to a maximum of category 125.
- If yield strength is equal to or higher than 350 MPa, increase in fatigue strength by a factor 1.6 up to a maximum of category 125.
- For structural elements with thicknesses above t = 20 mm, the benefit is reduced to a factor 1.5 on strength and to a maximum of category 100.

5.4.5 Conclusions

Conclusions are that improvement in fatigue strength can be achieved:

- By improving the stress flow in welded details.
- By modern welding processes and good workmanship which contribute to reduce the size of the defects to a level where the remaining defects are small notches from which the crack has to initiate.
- By increasing the crack initiation period through the reduction of surface defect adversity and size. For high strength steels, this can be best achieved using post-weld improvement methods.
- Improvement is limited by cracks starting from other locations, such as the root in a cover plate weld or internal defects (blowholes in longitudinal fillet welds).
- The fatigue strength increase is more significant for low fatigue categories. An upper bound is achieved when the fatigue strength of improved details reaches the fatigue strength of built-up sections, i.e. the fatigue strength of longitudinal fillet welds, which is category 125.

5.5 Examples and Applications

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5.5.1 Fast Bridge 48 Military Bridge, Sweden [Höglund]

5.5.1.1 General description

Fast Bridge 48 is a 48 m single-span bridge system for loads up to Military Class 70 (MLC70, approximately 64 metric tonnes) according to North Atlantic Treaty Organization (NATO) standards. The bridge is made of extra high strength steels (HPS steels S960 and S1100) and can be deployed in less than 90 minutes and retrieved in the same time from either side of a river or dry gap.

The bridge is the result of about eight years of research and development in cooperation mainly between the Swedish Defence Material Administration (FMV) and Karlskronavarvet AB. The design is patented.

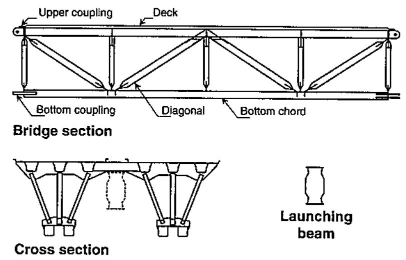

Fig. 5.5.1: Cross section of the Fast Bridge 48

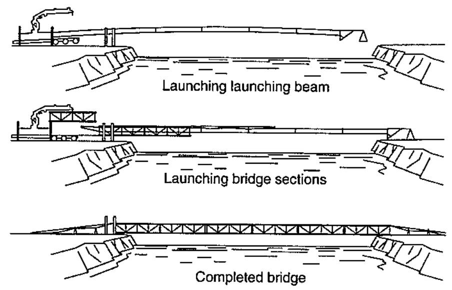

Fig. 5.5.2: Launching procedure of the Fast Bridge 48

5.5.1.2 Conceptual Design

The span length can range from 32 m to 48 m, made up by four to six 8 m sections, width 4 m and depth about 1.6 m, see Figure 5.1.1. The deck is made of a plate of S1100, thickness 5 mm, which is stiffened by cold-formed channel sections with web folds. Also the bottom chord is made of cold-formed sections whereas the diagonals in the truss are made of S460 rectangular hollow sections. The coupling plates are made of 50 mm S960 plates. The Fast Bridge 48 required steel with 1100 MPa in yield point and impact toughness 40 J at -40°C. Such steel is not included in existing regulations which is why it was necessary to verify whether existing criteria for the design and manufacturing methods were valid. Extensive tests on components concerning local and distortional buckling were made as well as fatigue tests and static tests on welded components and examination of welding procedures. For instance, it turned out that the Swedish Code for cold-formed structures was applicable, in spite of exceeding the strength and thickness limits in the code to a great extent.

5.5.1.3 Benefit of High Strength Steel

As there are no limitations of the deflections of military bridges the strength of HPS steels can be fully utilized. This results in a light-weight structure which can compete with aluminum alloys and polymers. The great benefit commonly gained with modern steel with improved performance is:

- weight savings
- increased life
- reduced fabrication costs

A further development of the bridge makes it possible to span up to 200 m with intermediate supports dropped from the bridge during launching.

5.5.1.4 Reference Data
Owner: Swedish Armed Forces
Purchaser and project manager: Swedish Defense Material Administration
Steel designer: Kockums AB, Karlskronavarvet and Royal Institute of Technology, Stockholm

5.5.2 Hybrid Girder Bridge, Mittådalen, Sweden [Collin]

5.5.2.1 General Description
The bridge is located in Mittådalen in the middle of Sweden. It replaces an outdated existing bridge at the same site.

5.5.2.2 Conceptual Design
The bridge is simply supported with a span of 25.6 m, and a free width of 7 m. The girder height is 1245 mm, and the steel weight 103 kg/m^2 deck area. The cost of the steel contract was 43.4 kEUR, including bearings. Since the site is located far away from the nearest concrete plant, the deck of the bridge was prefabricated, with joints cast in situ. At the abutments, back-walls are connected to the end-plates of the bridge with headed shear connectors. The bridge rests on four steel roller bearings, with a roller diameter of 180 mm. The use of S690 in the bottom flange meant that the following design criteria were met at the same time:
- The Ultimate Limit State
- The Serviceability Limit State (no yielding)
- The Serviceability Limit State (δ<L/400)

Hybrid girders are now included in the Swedish Bridge Code. The limitation is mainly that the effective area of the web is based on the yield strength of the stronger flange plates, which must not exceed 1.5 times the strength of the web plate.

Fig. 5.5.3: *Mittådalen hybrid bridge girder*

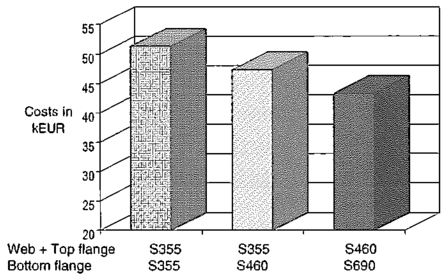

Fig. 5.5.4: Comparison of costs

5.5.2.3 Use and Benefit of High Strength Steel
By using S690 in the bottom flange, the following advantages can be obtained:
- smaller butt weld volume
- cheaper steel plates
- easier handling and transport
- smaller area to paint

One limitation for the concept with HPS can be that the time for delivery of material might be critical. Furthermore, on heavily trafficked roads, the fatigue strength may very well govern the dimensions of the flanges.

5.5.2.4 Reference Data
Owner: National Road Administration
Steel designer: Scandiaconsult, Luleå
Steel contractor: DEM-VERK MEK AB

5.5.3 Road Bridge over the River Rhine at Düsseldorf-Ilverich, Germany [Müller]

5.5.3.1 General Description
The Rhine bridge Düsseldorf-Ilverich is a cable-stayed road bridge of the highway A44 near Düsseldorf (Figure 5.5.5). The bridge was built in the period 1998 – 2002. For the steel structure 7180 t of steel grade S355 and 520 t of grade S460 were used.

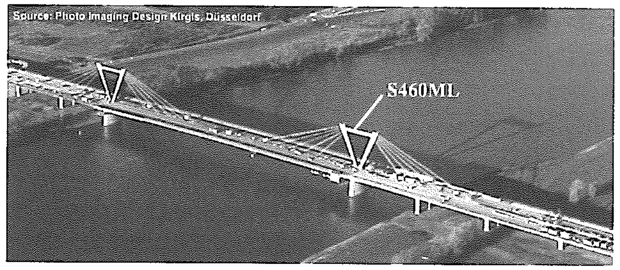

Fig. 5.5.5: Rhine bridge Düsseldorf-Ilverich

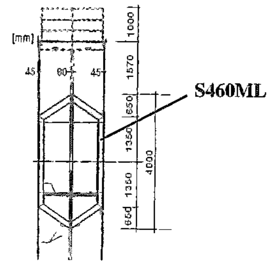

Fig. 5.5.6: Cross section of tension ties made of S460 TM

5.5.3.2 Conceptual Design
The main span of the bridge consists of two V-shaped steel pylons supporting the main girder by steel cables. The V-shaped pylons were necessary in order to keep the height of the towers small because of the proximity to the Düsseldorf-Airport. The approach bridges were made in prestressed concrete and the piers in reinforced concrete.

5.5.3.3 Use and Benefit of High Strength Steels
In order to transmit the horizontal cable forces between the two sides of the V-shaped pylons, they are connected in the upper part by tension ties consisting of a welded hollow steel cross section see Figure 5.5.6. For these tension ties a thermo-mechanical rolled steel grade S460ML was used for two reasons:
- minimum plate thickness to keep the weld volume for butt welds small
- welding without preheating

Both arguments helped to improve the economy and the quality of welds.

5.5.3.4 Reference Data
Owner: Federal Republic Germany, represented by Straßenbauverwaltung Nordrhein-Westfalen
Steel contractor and designer: Stahlbau Plauen

5.5.4 Composite Bridge near Ingolstadt, Germany [Müller]

5.5.4.1 General Description
This composite highway bridge near Ingolstadt is a multi-span bridge having span lengths of 24.0, 5×30.0 m and 20.0 m, carrying a 15.0 m wide concrete slab, see Fig. 5.5.7. The bridge is designed as an integral structure where the steel girders are directly connected to the columns by flexible steel plates, meaning that no bearings were needed.

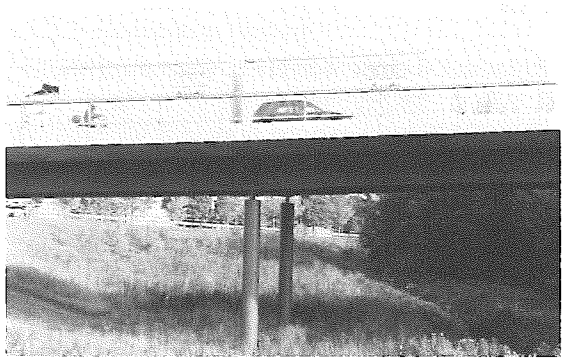

Fig. 5.5.7: Composite bridge near Ingolstadt, Germany

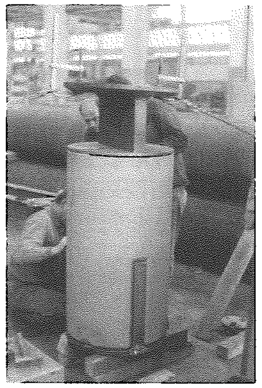

Fig. 5.5.8: Detail of the bridge bearing

5.5.4.2 Use and Benefit of High Strength Steels
For the semi-rigid connections between the composite piers and steel girders lamellas of steel grade S690QL was used, see Fig. 5.5.8. In order to ensure a semi-rigid connection the flexible steel plates must be designed for the following requirements for stiffness and strength:
- The plate thickness must be small enough to reduce restraints from translatory and rotatory movements of the structure at the columns.
- The plates must be thick enough to resist the normal forces and the restraining moments from movements safely.

These contradictory requirements lead to an optimization problem which was satisfactorily solved by using S690QL.

5.5.4.3 Reference Data
Owner: IFG Industrie-Förder-Gesellschaft, Ingolstadt, Germany
Steel designer: Hilzinger Bettcher-Zeitz Habisreutinger, München, Germany
Steel construction: Max Bögl, Bauunternehmung GmbH&CoKG, Neumark, Germany

5.5.5 Roof Truss of the Sony Centre in Berlin, Germany [Müller]

5.5.5.1 General Description
Various storeys of a hotel building at the Sony Centre in Berlin are suspended from a roof truss to protect an old masonry building from overloading with the "Kaisersaal" integrated in the hotel building, see Fig. 5.5.9.

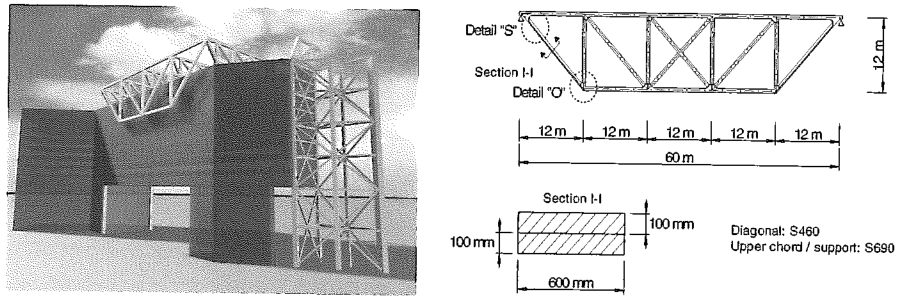

Fig. 5.5.9: Overview on the roof truss of the Sony Centre in Berlin, Germany

5.5.5.2 Use and Benefit of High Strength Steels
The truss structure composed of components with a solid rectangular shape was made of steel grade S460 and S690. High strength steel was used to keep the dimensions of the cross sections small that were provided with an envelope for fire protection.

5.5.5.3 Verification to avoid Brittle Fracture
The verification to avoid brittle fracture at low temperatures was performed by calculation according to the European design code EN 1993-1-10 [5.5] assisted by testing. The structural detailing and the dimensions of the "large scale test specimens" are given in the following figure.

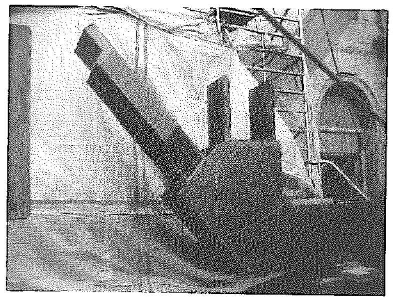

Fig. 5.5.10: Truss joint of the roof structure

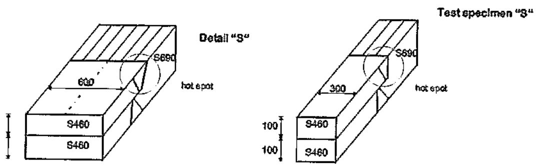

Fig. 5.5.11: Test specimens for the verification to avoid brittle fracture

5.5.5.4 Reference Data
Owner: Sony
Steel contractor: Waagner Biró Binder AG, Wien, Austria

5.5.6 Millau Viaduct, France [Schröter]

5.5.6.1 General Description

This 320 million Euro viaduct is the last link in the French highway A75 between Clermont-Ferrand and Béziers closing a gap across the valley of the river Tarn next to the city of Millau. The search for an aesthetic solution led to the adoption of a multi-span cable stayed bridge with a light steel deck crossing the river at a height of 270 m. With a total construction height of 343 m the bridge takes the world record of the highest bridge in the world.

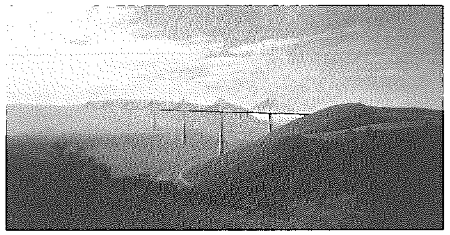

Fig. 5.5.12: Visualisation of the Millau Viaduct

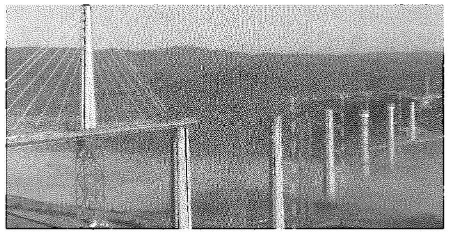

Fig. 5.5.13: Launching with pylon

5.5.6.2 Conceptional Design

The 2460 m long deck is composed of 6 main spans of 342 m each and two side spans of 204 m each. The deck is composed of a steel girder with a total height of up to 4.20 m and a total width of 32.00 m in order to optimize the resistance against high wind loads in the valley. The cross section consists of a central box, which is also linked to the steel pylons, lateral connecting panels as well as lateral side boxes. Boxes and panels are stiffened by trapezoidal stiffeners. The 7 steel pylons are erected in an inverted Y-shape and hold two times 11 cables each.

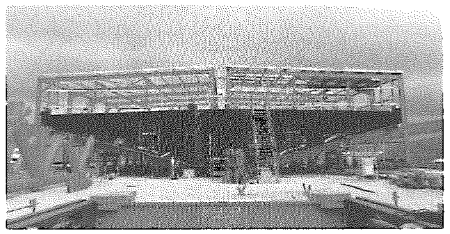

Fig. 5.5.14: The cross section of the bridge

5.5.6.3 Use and Benefit of High Strength Steel.

In total 43,000 t of steel plates have been applied for the deck and the pylons. High-strength steel grade S460ML (nominal yield stress of 460 MPa) has been used for the entire central box and some connecting elements with a thickness up to 80 mm in order to:

– resist high loads without increasing the amount of steel used,
– reduce cantilever bending moments during launching of the bridge,
– apply a more efficient welding process,
– reduce transport weights from the workshop to the site,

Furthermore, the pylons have been constructed in S460ML steel grade in a thickness of up to 120 mm.

5.5.6.4 Launching

The deck was launched from platforms on either side of the river Tarn. The deck was equipped with a launching nose and with one pylon at each end in order to increase stiffness during launching. With the use of auxiliary piles in the middle of each span the launching cycle was 171 m. After connecting the two deck spans the five remaining pylons were welded together, brought to their final position and erected. Then the cable-stays were assembled and tightened and the auxiliary piles removed.

5.5.6.5 Reference Data

Owner: Eiffage Group, France
Steel designer: Greisch, Belgium
Steel contractor: Eiffel Construction Métallique, France

5.5.7 Verrand Viaduct, Italy [Miazzon]

5.5.7.1 General Description

The Verrand viaduct is an orthotropic deck bridge, part of the Mont Blanc-Aosta highway, located in the third building lot, Mont Blanc Tunnel-Morgex. In particular, the viaduct is located at Prè Saint Didier (Aosta), near Courmayeur, at the side of the existing S.S.26 (Mont Blanc Tunnel-Aosta) and it is necessary to overpass the valley with the country road and the Dora Baltea river. It was finished in August 2002 after 2 years of work and with a total steel quantity of 6100 t. More information is given in [5.36]

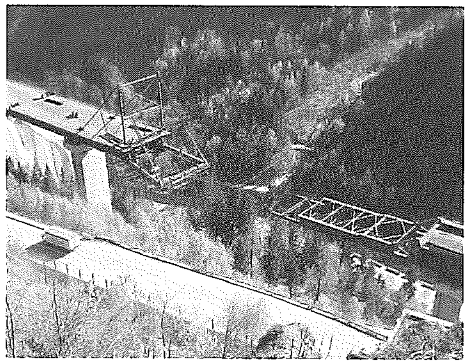

Fig. 5.5.15: Connection of the bridge parts, built in two different yards (Aosta and Mont Blanc sides)

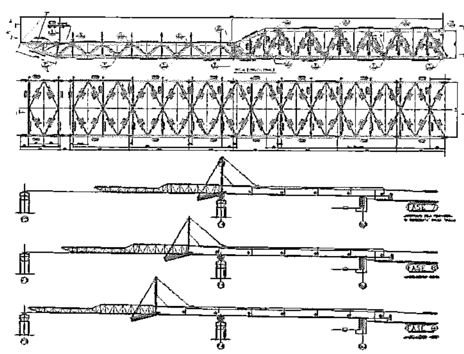

Fig. 5.5.16: Lattice launch girder using S690. Lateral prospect and plan – Some launching phases

5.5.7.2 Conceptual Design

The possible structural alternatives were all characterized by the choice to realize a unique motorway viaduct for all the roadways, with width of around 20 m. It was decided to use an orthotropic deck bridge, with two principal beams and interior bracing, of five spans, respectively, 97.5+135+135+135+97.5 m, with four intermediate piers.

5.5.7.3 Use and Benefit of High Strength Steel

The lattice launch girder with a length of 85 m was realized using high strength steel tubular sections of grade S690. Thereby the weight of the launch girder could be significantly reduced which finally allowed a design process of the final steel-deck bridge with no changes in cross sectional dimensions because of the launching process.

5.5.7.4 Reference Data

Owner: R.A.V. (Valley of Aosta highway) S.p.A., Rome, Italy
Steel designer: SPEA S.p.A., Milan, Italy
Steel contractor: OMBA I.&E. S.p.A., Torri di Quartesolo, Vicenza, Italy

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6 Summary and Conclusions

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6.1 General

New steel production processes have lead to a remarkable improvement of steel products in the last couple of years and allows producing steels according to the desired mechanical and chemical properties. Developments in the USA, Japan and Europe showed a remarkable increase in the use of these steel grades for structural purposes, especially in the field of bridge construction. Within these countries, a variety of steel grades were developed according to the specified requirements and priorities. As this document is an assembly of papers from various countries with a focus on different aspects on HPS, the following sections give a short summary concerning research, developments and applications, and include an overview and comparison of the properties of the commonly used international HPS steel grades.

6.2 HPS Developments, Research and Applications

6.2.1 HPS in the USA

Within the USA a large research and development program was initiated in the mid 1990's in order to develop new high-performance steel grades for cost effective steel bridges, having high strength, improved weldability, higher toughness and better weathering and fabrication characteristics. Through a strong collaborative partnership and joint effort including various agencies such as FHWA, AASHTO, AISI, Navy and ASTM, fabricators, suppliers and universities the new steel grades HPS 50W, HPS 70W and HPS 100W were successfully developed matching the above mentioned criteria. The cost effective application of these new steels grades has already been demonstrated by the successful performance of about 200 in-service HPS bridges in over 40 states, as partly documented here. Significant progress has also been made in the AASTHO design codes to allow for more efficient use of high strength steels in bridges that can, for example, be attained by using the hybrid girder concept.

6.2.2 HPS in Canada

When high-performance steels are proposed for bridges, the utilization of the higher yield strength may often not be possible because the fatigue limit state is likely to control the design. In Canada research on HPS is thus underway focusing primarily on fatigue behavior. Initial research results from the University of Alberta could, for example, show that HPS base material does not generally exhibit better fatigue performance than conventional steels although there seems to be a potential benefit for the fatigue endurance limit. Further research is currently under progress in order to study experimentally the fatigue performance of welded HPS details. Additionally, an HPS demonstration bridge, fabricated from the U.S.A. steel grade HPS 485W, is planned in order to gain experience on the use of HPS in Canada.

6.2.3 HPS in Japan

Similar to the USA the Japanese developments in the area of HPS are closely linked to bridge design, and their steel grades are termed Bridge High-Performance Steels (BHS500 or BHS700). With the goal to minimize all bridge construction costs related to material, fabrication, transportation, erection and maintenance, the main development and research has been done to enhance the weldability and toughness and to improve the fatigue and weathering performance. In order to improve the fatigue performance new welding technologies such as the Low Transition Temperature (LTT) welding were developed. Because of the environmental conditions along the Japanese coastal regions a lot of research work has been done on the development of new nickel-containing weathering steels with an excellent corrosion resistance. By introducing an advanced weathering index taking into account environmental conditions, progress has also been made with the corrosion design of these steels. This is documented in the related actual examples and applications within this document.

6.2.4 HPS in Europe

In Europe currently much effort is being spent to set up advanced design rules for HPS and to include them within the framework of the joint European technical standard for civil engineering structures, the so-called Eurocodes. This generally aims to use HPS in all kinds of structural applications, not only in bridge design. New design procedures could for example be derived concerning material toughness requirements or by the introduction of new flexural buckling curves that benefit from the high material strength even if stability seems to be the governing limit state. Thus the structural application of HPS in Europe is not only linked to bridges but also to building structures where high strength steels have, for example, been used in joints where high forces have to be transmitted to concentrated areas of the structure.

6.3 High-Performance Steel Properties

6.3.1 Mechanical Properties

Table 6.3.1 shows a comparison of the required mechanical properties of typical HPS used in the USA and Canada, Japan and Europe.

	Steel grade	Production process	Thickness [mm]	Yield strength [MPa]	Tensile strength [MPa]	Min. elongation at fracture [%]	Required toughness	
							Temp. [°C]	Impact energy [J]
US	HPS70W [1]	Q&T [4] TMCP [5]	≤ 100 ≤ 50	485	586-760	19	-23	48 (L) [6]
US	HPS100W [1]	Q&T [4]	6-64	690	760-895	18	-34	48 (L) [6]
Japan	BHS500 [2]	TMCP [5]	6-100	500	> 570	19	-5	100 (T) [7]
Japan	BHS700 [2]	TMCP [5]	6-100	700	> 780	16	-40	100 (L) [6]
Europe	S460M [3]	TMCP [5]	≤16 17-40 41- 63 64-80 81-100 101-120	460 440 430 410 400 385	540-720 540-720 530-710 510-690 500-680 490-660	17	-20	40 (L) [6]
Europe	S690Q [3]	Q&T [4]	3-50 51-100 101-150	690 650 630	770-940 760-930 710-900	14	-40	30 (L) [6]

[1] ASTM A709-01a and AASHTO M270-02 / [2] JIS Z2201 and JIS G3106 / [3] EN 10025
[4] Quenched and Tempered (Q&T) / [5] Thermo-Mechanical Control Process (TMCP)
[6] measured in longitudinal direction (L) / [7] measured in transverse direction (T)

Table 6.3.1: Comparison of required mechanical properties

Tensile strength properties were one of the major considerations within the development of High-Performance Steels. The compared steels have almost the same characteristics concerning the required yield strength and tensile strength. The only difference lies in the specification of the strength properties in relation to the plate thickness. Whilst according to the European steel production standards the strength still depends on the plate thickness, the US and Japanese standards have a uniform strength over the entire range of plate thickness.

For bridge structures that are exposed in many cases to extreme low service temperatures and where service and loading conditions make fatigue cracking a possible and significant issue, fracture toughness is an important part of the mechanical properties. Fracture-tough HPS therefore usually have minimum Charpy-V-notch toughness requirements. The requirements according to Table 6.3.1 differ in the above-named countries, depending on the relevant testing temperature and the minimum impact temperature. High toughness is of beneficial use and needed to resist cracks in structures induced by fatigue loading, e.g. in bridges or in earthquake regions, where high strain rates occur.

Despite the fact that high strength steels usually have a high yield-to-tensile strength ratio, Table 6.3.1 clearly indicates that concerning ductility all steels achieve a minimum axial elongation at fracture greater than 14% that has to be considered as excellent.

6.3.2 Chemical Properties

Table 6.3.2 summarizes the chemical composition in terms of the maximum alloying content requirements given in the relevant standards. Concerning the given values it has to be mentioned, that in many cases these values can be considered as very conservative upper limits. Actual values of the products are usually far lower and also vary with the product size and thickness.

	Steel grade	Production process	C	Si	Mn	P	S	Cu	Cr	Ni	Mo	V
US	HPS70W	Q&T TMCP	≤0.11	0.30-0.50	1.10-1.35	≤0.020	≤0.006	0.25-0.40	0.45-0.70	0.25-0.40	0.02-0.08	0.04-0.08
US	HPS100W	Q&T	≤0.11	0.15-0.35	0.95-1.50	≤0.015	≤0.006	0.90-1.20	0.40-0.65	0.65-0.90	0.40-0.65	0.04-0.08
Japan	BHS500	TMCP	≤0.11	≤0.50	≤2.00	≤0.020	≤0.006	0.30-0.50	0.45-0.75	0.05-0.30	-	-
Japan	BHS700	TMCP	≤0.14	≤0.50	≤2.00	≤0.015	≤0.006	≤0.30	0.45-0.80	0.30-2.00	≤0.60	≤0.05
Europe	S460M	TMCP	≤0.16	≤0.6	≤1.70	≤0.025	≤0.020	≤0.55	≤0.30	≤0.80	≤0.20	≤0.12
Europe	S690Q	Q&T	≤0.20	≤0.8	≤1.70	≤0.020	≤0.010	≤0.50	≤1.50	≤2.0	≤0.70	≤0.12

Table 6.3.2: Comparison of chemical composition and maximum alloying content

The primary strength-controlling chemical element in all steel is carbon (C). In comparison to conventional mild steel ($0.18<C<0.25$) the carbon content of HPS has been reduced to values between 0.11 and 0.16, with other elements making up the strength loss that is associated with the lower C content, e.g. manganese (Mn).
The improved toughness and weldability of this steel is mainly gained through the required low sulfur level of about 0.006% as maximum.

6.3.3 Weldability and Fabrication

It is essential that steels have a chemical composition that promotes the fusion of the base metal and the filler metal without the formation of cracks or other imperfections. A big issue in the development of HPS steels was to improve the weldability and to reduce fabrication costs associated with high preheat and interpass temperature, high heat input, post-welding treatment and other stringent measures to eliminate hydrogen induced cracking in the weldment. The carbon equivalent is the most suitable criteria to characterize the weldability. In general, the lower the value the better the weldability. The CE-value of the International Institute of Welding (IIW) and the Pcm-value are the most commonly applied equations. Table 6.3.3 summarizes both carbon equivalents based on the maximum values given in Table 6.3.2 and alternatively if available based on specific values according to the relevant standards.

	Steel grade	Production process	CE [1] according to values in Table 6.3.1	CE [1] according to relevant standard	Pcm [2] according to values in Table 6.3.1	Pcm [2] according to relevant standard
US	HPS70W	Q&T/TMCP	0.43-0.56	-	0.22-0.27	-
US	HPS100W	Q&T	0.54-0.79	-	0.27-0.36	-
Japan	BHS500	TMCP	0.56-0.65	-	0.27-0.30	0.20
Japan	BHS700	TMCP	0.73-0.92	-	0.37-0.42	0.30-0.32
Europe	S460M	TMCP	0.66	0.47	0.35	-
Europe	S690Q	Q&T	1.11	0.65	0.53	-

[1] $CE = C + Mn/6 + (Cr + Mo + V)/5 + (Ni + Cu)/15$
[2] $Pcm = C + Si/30 + (Mn + Cu + Cr)/20 + Ni/60 + Mo/15 + V/10 + 5B$

Table 6.3.3: Comparison of carbon equivalents

Since HPS material, especially TMCP steels, exhibits low carbon equivalents in combination with high toughness it provides the best performance for welding. They can be easily welded to all ordinary steels allowing high heat input and in many cases preheating can be omitted or has to be done at low temperature. Roughly speaking one can conclude that for all the above mentioned TMCP steel grades, preheating can be omitted up to a plate thickness of 50 mm. The favourable welding properties in combination with a reduced welding volume because of smaller plate thicknesses compared to ordinary steels, significantly reduces fabrication and welding costs.
Concerning an appropriate filler material it is recommended to directly contact steel producers because the alloying concept may differ between different producers.

6.3.4 Fatigue

When high-performance steels are supposed to be used for bridges, in particular high fatigue resistance is required. In welded structures however the fatigue strength does not depend on the strength of the steel material. Increasing the fatigue strength of welded structures is thus very important in order to exploit the high strength properties in bridges. To overcome this problem section 4.5 and 5.4 provided answers on how to improve the fatigue strength of welded structures, for example by using post welding treatment methods or favourable welding methods or proper detailing in order to reduce stress concentrations.

6.3.5 Weathering Characteristics

Corrosion resistance is often referred to as the weathering characteristic of HPS and other steels. In contrast to the original weathering steels, developed in the 1960s, steel manufacturers especially in Japan and the USA have recently developed a new advanced weathering steel material by adding various alloy elements such as high contents of nickel (Ni) and copper (Cu). These new steels have a much higher corrosion resistance than the original ones and thus allow structures to be built without painting or other corrosion resistance techniques. Additionally, for the corrosion design, an advanced weathering index allows a better prediction of the weathering resistance depending on the amount of alloy element and the environmental conditions.

6.4 Applications of HPS

The major application of HPS steels so far is in the area of bridge structures. This is due to the fact that building structure design is largely controlled by stiffness, governed by the modulus of elasticity, and increasing the material strength for buildings would be neither sufficient nor effective. The included examples are therefore focused on bridge design.
In comparison to common mild steel grades, the major benefit of using HPS in bridge structures based on the documented applications can be summarized as follows:
- reduced weight and cost,
- the use of shallow girders with an enhanced architectural appearance,
- the possibility of increasing span length to reduce the number of piers or to reduce the number of girders,
- the reduced fabrication costs during welding, because of thinner plates and therefore a smaller welding volume in combination with lower preheat requirements,
- the easier handling and transportation due to reduced weight,
- the higher fracture toughness minimizes the risk of sudden failure due to brittle fracture and increases the cracking tolerance, thus adding redundancy and a high reliability to the structure,
- the good weathering characteristic of HPS assures long-term performance of unpainted bridges,
- the reduction of cantilever bending moments during launching of bridges due to reduced weight, and
- the lower life-cycle costs and increased life time.

An optimized use of HPS is often in combination with other grades of steel, by using hybrid combinations with mild steel. That means it is especially advantageous to use HPS at certain locations of structures where stresses are high or where there are specific requirements concerning safety, redundancy, etc. Especially the so-called hybrid girder concept documented in sections 2.3 and 5.5.2 gave very economical solutions for bridge structures.

6.5 Concluding Remarks

The development of advanced steel production methods, especially the thermo-mechanically controlled rolling process (TMCP) in combination with enhanced alloying concepts lead to a new generation of steel material with highly improved properties such as high strength, high toughness, excellent weldability and good corrosion resistance.
Taking into account all the benefits of these properties in combination with a good design and construction philosophy will lead to significant savings within the overall construction process. Additional cost savings, often not considered in a cost analysis, like reduced transportation costs, slender structures, lower foundation costs and a reduced use of natural resources, give HPS a great potential to be used as a well qualified and sustainable material for the future.

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