THE USE OF VERY HIGH STRENGTH STEELS IN METALLIC CONSTRUCTION

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1. EUROCODES – BUILDING CODES FOR EUROPE

(1) The globalisation of the construction market comprising construction products, engineering- and construction services requires International Standard Families in order to avoid inconsistencies due to the use of various national codes [1].

(2) So far there are two sources of International Standard Families: one in the USA, the other in Europe, each consisting of a design code in connection with product standards and testing codes, figure 1.

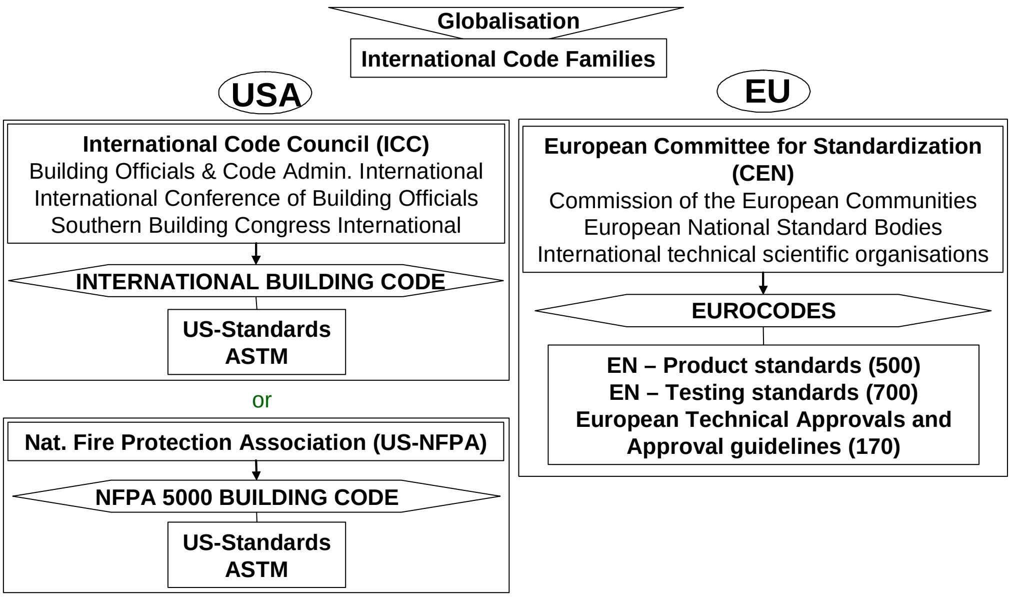

Figure 1: International Standards Families [1]

(3) The European Standard Family is being prepared by CEN and so far includes 10 Eurocodes with 58 parts, ~500 EN-standards for products and ~700 EN-standards for testing. It also contains ~170 European Technical Approvals and European Technical Approval Guidelines worked out by EOTA.

(4) Figure 2 shows as an example the standard system for steel construction:

EN 1990-Part 1 gives the delivery conditions for prefabricated steel components taking reference to

1. Product Standards for semi-finished materials, products for connections etc.,
2. the Eurocodes, in particular Eurocode – Basis of Structural Design –, Eurocode 1 for actions and Eurocode 3 for the design of steel structures,
3. the execution standard EN 1090 – Part 2.

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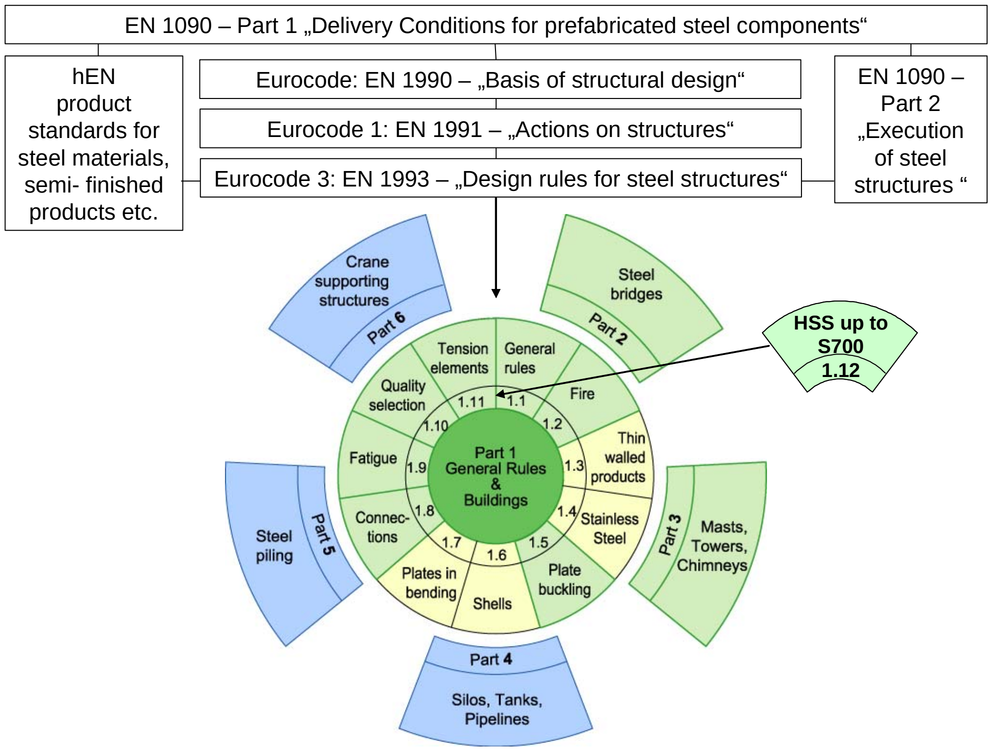

Figure 2: Standard system for steel structures [1]

(5) The crucial condition for the architecture of the design rules in Eurocode 3 and all the other Eurocodes is, that the manufacturer of prefabricated components may determine the properties of these components to be declared for CE-marking either by tests or by calculations and that for the calculative determination of properties the Eurocodes are the only design codes referred to, see figure 3.

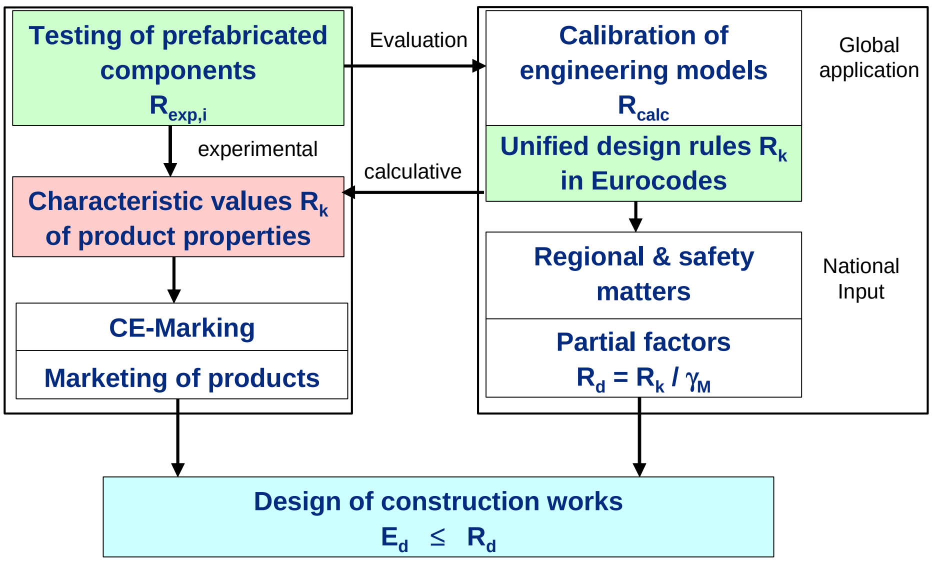

Figure 3: Means to determine characteristic properties R_K [1]

(6) By this condition a link between experimental results from tests with prefabricated components and the design rules in the Eurocodes is established that is specified by the reliability requirements of EN 1990 – Eurocode: Basis of structural design – in the following way:

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1. The product property to be declared, that may be determined directly from testing, shall represent a certain fractile of the statistical distribution of the experimental results. It is denoted as characteristic value R_K (in general the 5%-fractile equivalent to the mean – 1,645 standard deviation), and this value declared with CE-marking will be acknowledged throughout Europe without any impact from national safety levels. The method to determined R_K from tests is therefore a unified European rule in EN 1990 – Annex D, see figure 4.
2. Eurocodes shall be used as an alternative to experimental testing and therefore provide calculative methods to determine numerical values of R_K. These calculative values R_K are in competition with those from direct experimental tests. Therefore the characteristic values of resistances in the Eurocodes must be calibrated to test results such, that the manufactures prefers them to any experimental determination, see figure 4.
3. Eurocodes have a double role; besides their role as tool for determining R_K their main role is to be used as a design code for the design of structures. This design however needs design values R_d, that represent a far lower fractile than the characteristic value R_K,; they shall however be determined using the declared characteristic values R_K.
   Hence the design values needed for the design of structures shall be
   $$R_d = \frac{R_K}{\gamma_M}$$
   where γ_M is a global partial factor related to the resistance R_K.
4. The choice of the global partial factor γ_M is in the responsibility of Member States (Nationally Determined Parameter); however the Eurocodes provide recommendations for the numerical values of these NPD's that result from the same test evaluations that are used to obtain R_K. These recommendations aim at a fractile of the design value R_d equivalent to the mean value – 3,04 standard deviation.

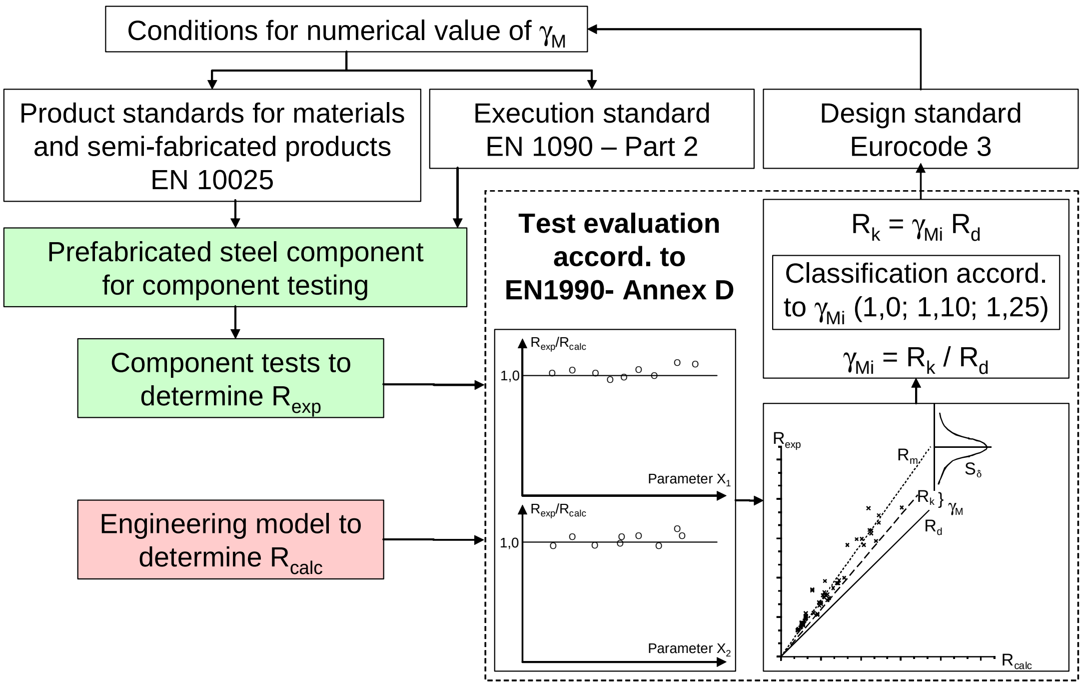

Figure 4: Determination of R_K by tests [1]

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(7) The Eurocodes represent a strong tool of market promotion for any type of material and way of construction they cover [2]. Hence the question is, to what extend they cover also the use of very high strength steels in metallic construction.

2. STRENGTH- AND TOUGHNESS PROPERTIES OF HIGH STRENGTH STEELS [3]

(1) The development of new high strength steels has been driven by the following reasons:

1. Economy: By increasing the strength of steel, the structural section can be reduced. This may reduce the weight of the structure, see figure 5 and subsequently the volume of weld metal (~ t^2) and hence fabrication and erection costs.
2. Architecture: The size of structural elements can be reduced enabling special aesthetic and elegant structures, which embed in the environment in an outstanding manner.
3. Environment: Construction with less steel means also a reduced consumption of our worlds rare resources.
4. Safety: Modern high strength steel grades do not only show high strength values. Special grades combine this strength with excellent toughness properties so that a high safety both in fabrication and application of the structures is applied. A good example for this are modern offshore steel grades performing at lowest service temperature.

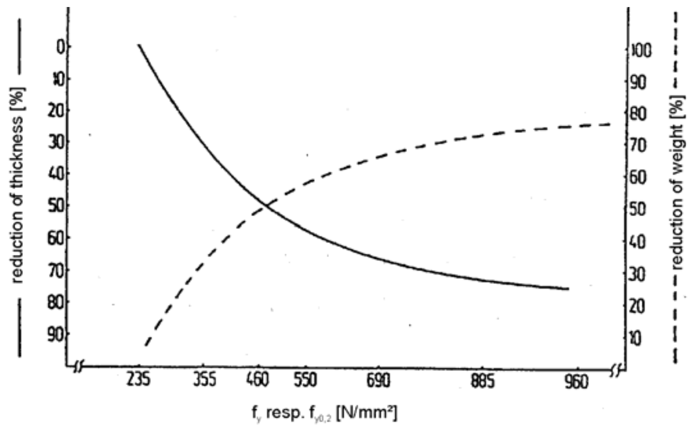

Figure 5: Reduction of wall thickness and weight with increasing strength of steel

(2) Figure 6 demonstrates the historical development of production processes for rolled steel products [3].

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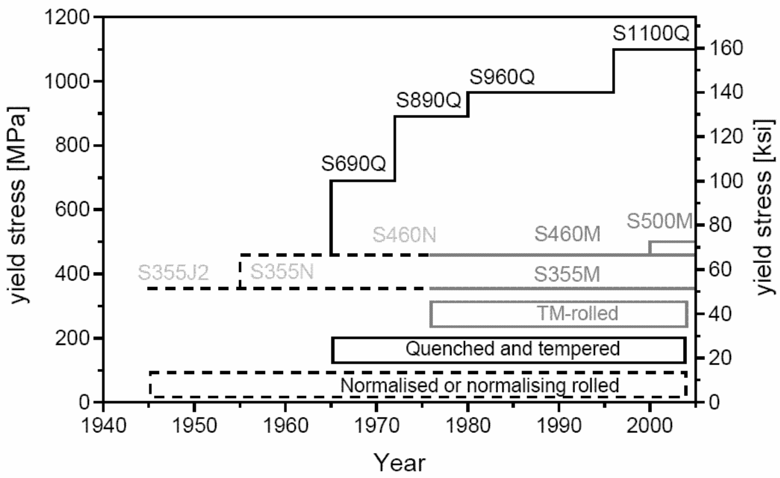

Figure 6: Historical development of production processes for rolled steel products [3]

Until 1950 steels S355J2 were regarded as high tensile steels.

(3) In the 1960's the application of the quenching and tempering process for structural steel grades began. Beside the special 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 carbon nitrides.

(4) Today this process enables steel grades with a yield strength up to 1100 Mpa, although only grades up to 960 Mpa yield stress are standardized (EN 10025-6). The mobile crane industry uses these "ultra-high" strength steels because of the extraordinary role of light weight for performance. For European classical steel construction, e.g. for buildings and bridges, the strength is mostly limited to steel grades up to S690.

(5) In the 1970's the thermo mechanical rolling process was developed and first applied for pipeline plates, but then fast found the way into the fields of ship building and construction of offshore platforms both for plates and rolled sections. TM rolling is 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. Plates with guaranteed minimum yield strength up to 500 Mpa are available in thickness up to 80 mm used in shipbuilding and offshore construction. For construction steel work even plates of 120 mm have been produced in particular for bridges.

(6) Figure 7 gives a survey on the Charpy-V-temperature transition curves for S355J2, S460ML and S690QL [3].

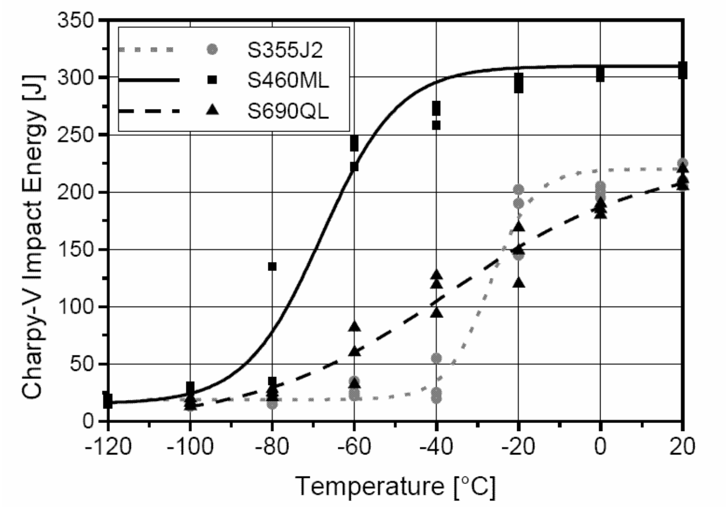

Figure 7: Charpy-V-temperature transition curves for S460ML and S690QL with S355J2 for comparison [3]

3. PARTICULAR PROBLEMS INVESTIGATED TO INCLUDE HIGH STRENGTH STEELS IN THE EUROCODES

3.1 Toughness requirements [4]

(1) Toughness properties of ferritic steels vary with temperature. Figure 8 gives the function of the toughness-temperature dependency for which the following regions are distinguished:

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

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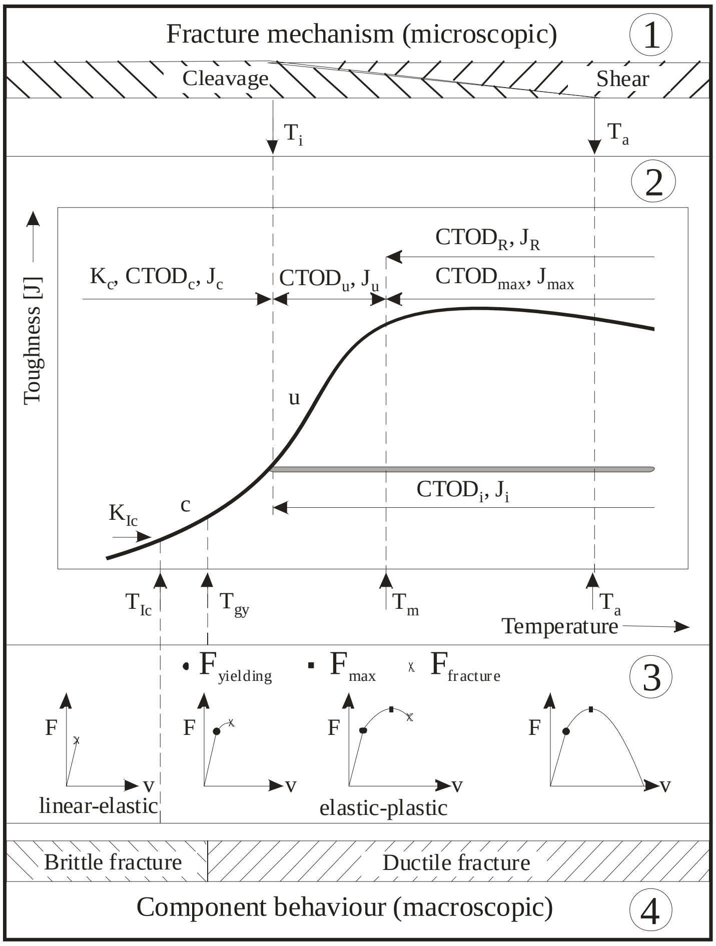

Figure 8: Toughness-temperature-curve and related load-deformation curves for tension elements using various parameters for toughness properties [4]

(2) 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 used for the design rules is that upper shelf toughness behaviour and ductile stress-strain behaviour govern the performance of test pieces, see figure 9.

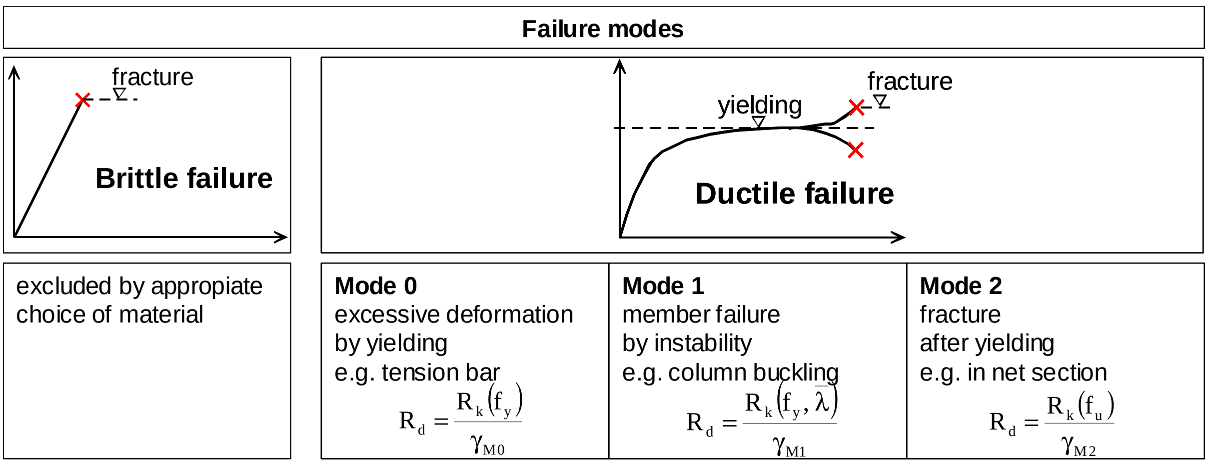

Figure 9: Ductile and brittle failure modes in structural design [1]

(3) Therefore it is necessary to avoid brittle fracture by an appropriate choice of material to comply with toughness requirements.

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

(5) EN 1993 – Eurocode 3 – Part 1-10 – Material toughness and through-thickness properties – gives a standardized procedure for this safety check, which is performed by comparing the design value of fracture mechanical action effect $K^*_{appl,d}$ with the design value of the fracture mechanical resistance $K^*_{mat,d}$

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

for the following design situation, see figure 10:

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 depth of surface crack) and is also subjected to residual stresses from fabrication.
   The crack size a_d is assumed to result from an undetected cracksize a_0 from fabrication and a subsequent crack growth Δa from fatigue under service conditions.
2. The temperature $T_{min,d}$ of the structural component attains its minimum (characteristic) 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 assumptions made above is classified as accidental.

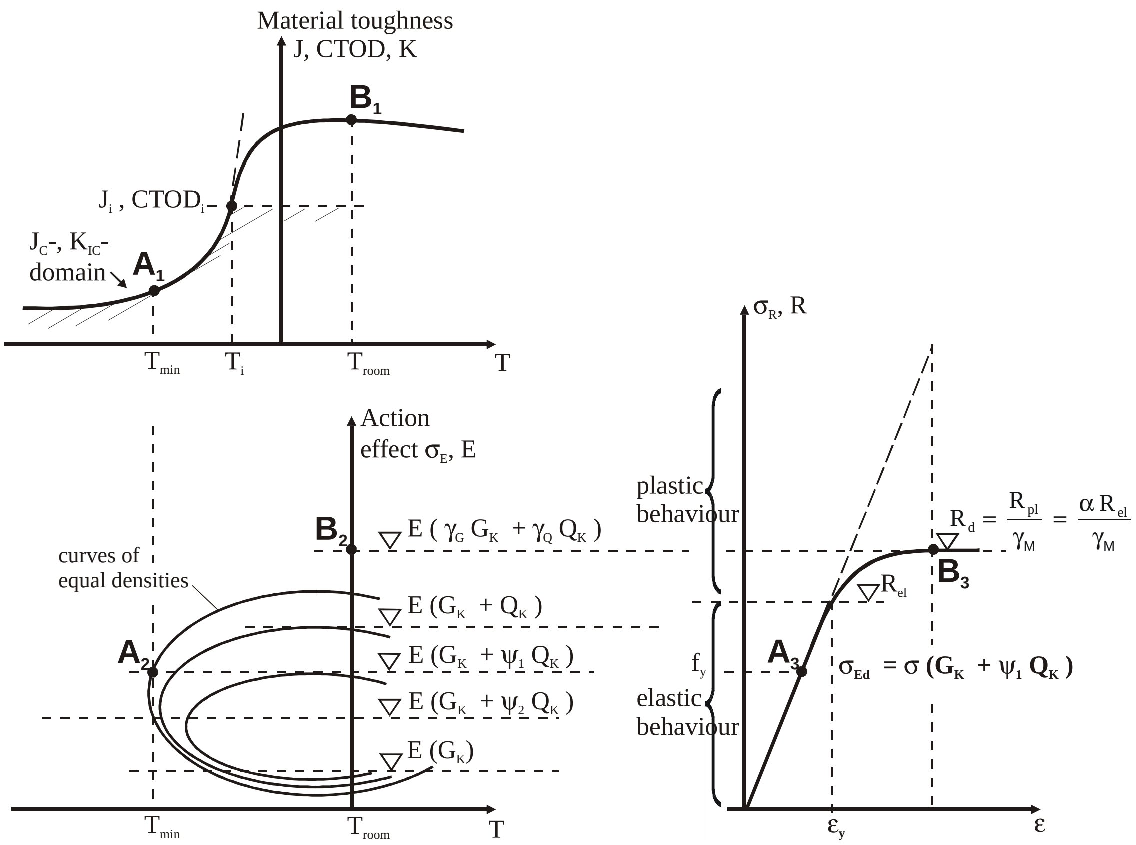

Figure 10: Design situation for choice of material in EN 1993-1-10

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(6) Figure 11 shows details of the determination of $K^*_{appl,d}$, using linear fracture mechanics for selected details taken form the tables for fatigue classes in EN 1993 - Eurocode 3 - Part 1-9 – Fatigue – and the CEGB-Failure assessment diagram.

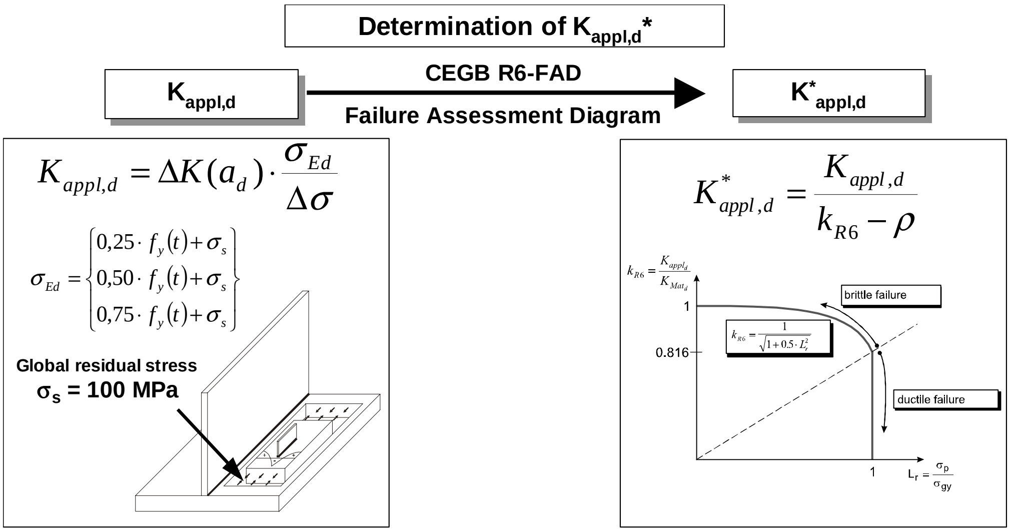

Figure 11: Determination of toughness requirements $K^*_{appl,d}$ [4]

(7) Figure 12 illustrates the determination of $K^*_{mat,d}$ from the material properties T_{27J} as standardized in product standards, the minimum temperature $T_{min,d}$ and the safety term ΔT_R by which the reliability of the verification is governed. A modified Sanz-correlation and the Wallin-Master-curve are important models used in this procedure.

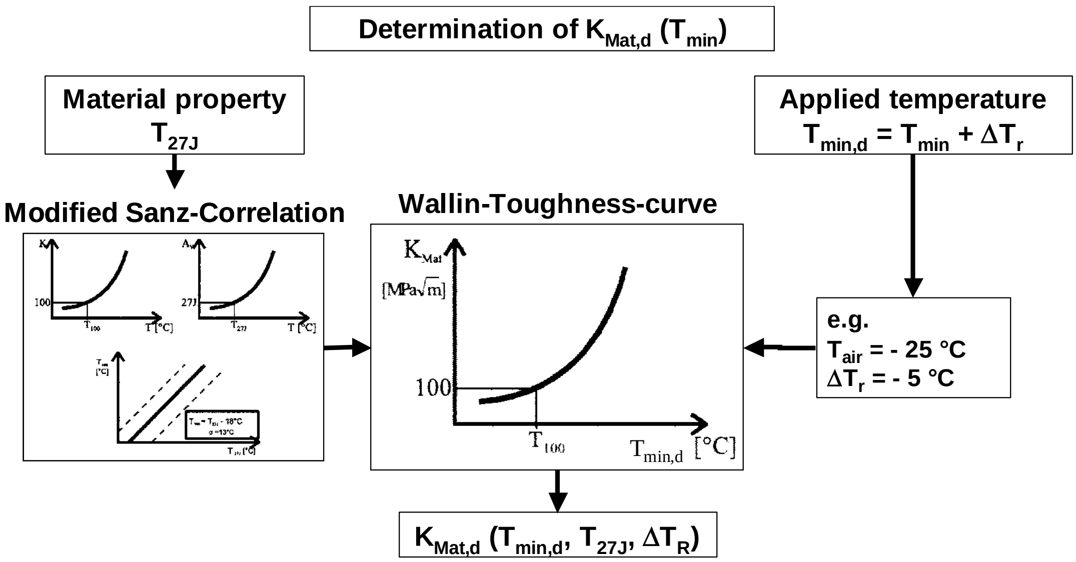

Figure 12: Determination of toughness resistance $K^*_{appl,d}$ [4]

(8) Figure 13 gives maximum plate thicknesses calculated according to EN 1993-1-10 depending on the steel grade, the Charpy-V-energy, the applied temperature T_{Ed} and the class of applied stress σ_{Ed}.

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

Figure 13: Table for the choice of material based on the standard requirement curve [8]

3.2 Yield to strength ratio requirement [4]

(1) The technical stress-strain curves of various steel grades show that the yield to strength ratios and the ultimate strains depend on the steel grade, figure 14. This fact results from the limitation of the tensile strength by the stability limit

$$\delta A \cdot \sigma_w - \delta\sigma \cdot A = 0$$

when using the true stress-strain curves, see figure 15.

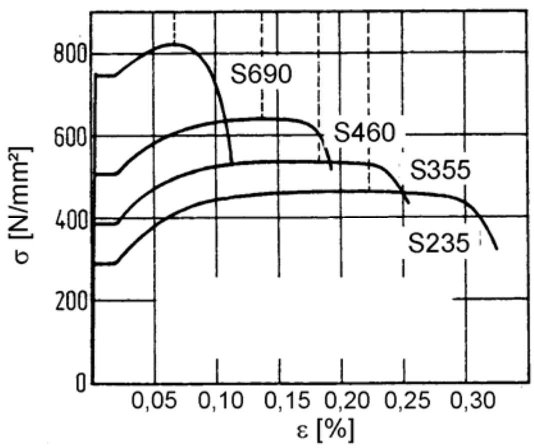

Figure 14: Load-deflection curves for different steel grades

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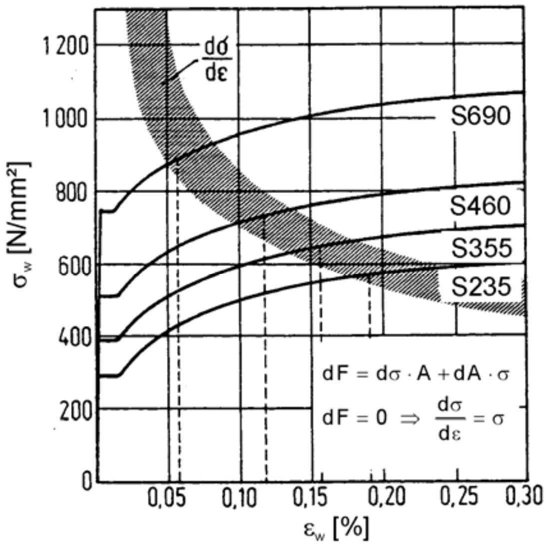

Figure 15: True stress-strain curves and stability criterion

(2) Figure 16 shows the yield to tensile ratio of low and high strength ferritic steels depending on the yield strength, which demonstrates that any limits of the yield-to strength ratios (still given in the Eurocodes) should be abandoned, as they penalize the use of high strength steels.

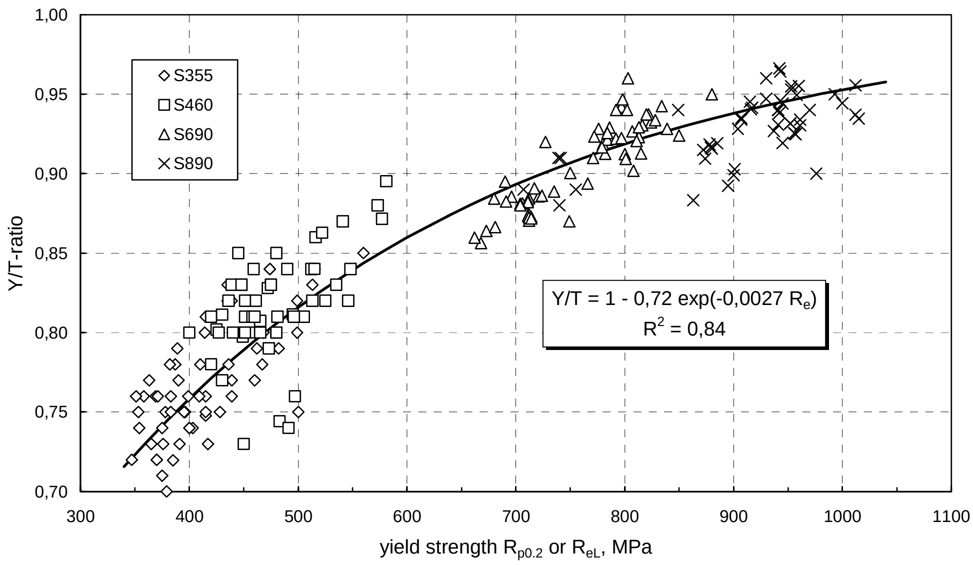

Figure 16: Yield to tensile strength ratio of low and high strength ferritic steels depending on the yield strength [3]

(3) Figure 17 shows as an example the net section stresses of large scale DECT-(Double Edge Crack Tension)-test specimens made of S890: whereas failure at -50°C is brittle and controlled by the appropriate choice of material, fracture in the upper shelf only occurs after general yield. This behaviour is clearly controlled by toughness.

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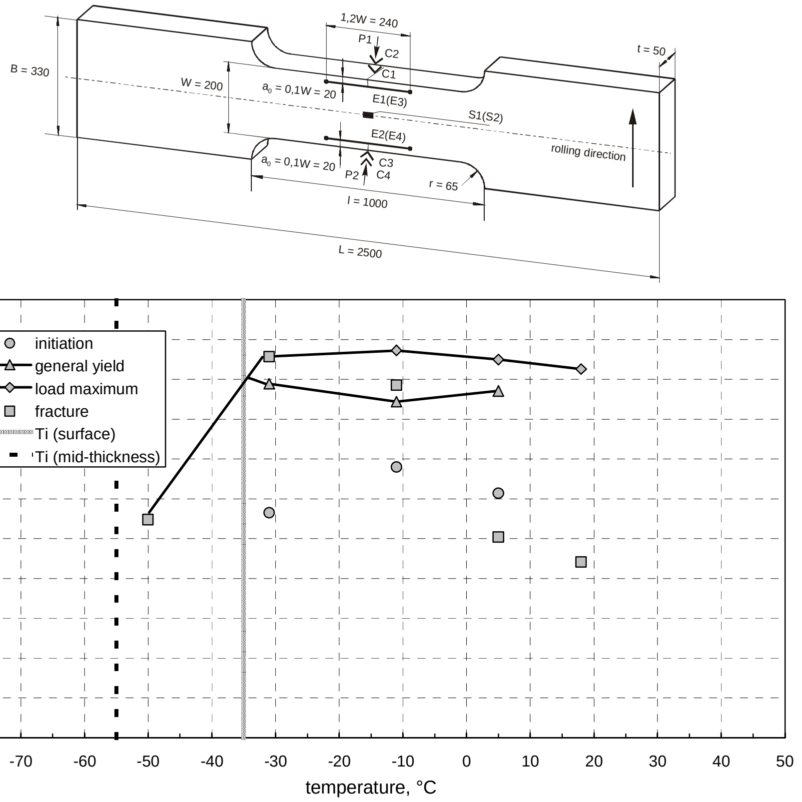

Figure 17: Net stress-temperature curve of a large scale DECT-test specimen [3]

(4) Figure 18 shows that toughness values are fully independent from the yield strength ratio; hence there is no reason to limit f_y/f_u because of ductility reasons.

(5) In conclusion the net section resistance of members in tension made of high strength steels, see EN 1993-1-12, has been modified to

$$N_{t,Rd} = \frac{0{,}9\,A_{net}\,f_u}{\gamma_{M2}} \geq \frac{A_{net}\,f_y}{\gamma_{M0}}$$

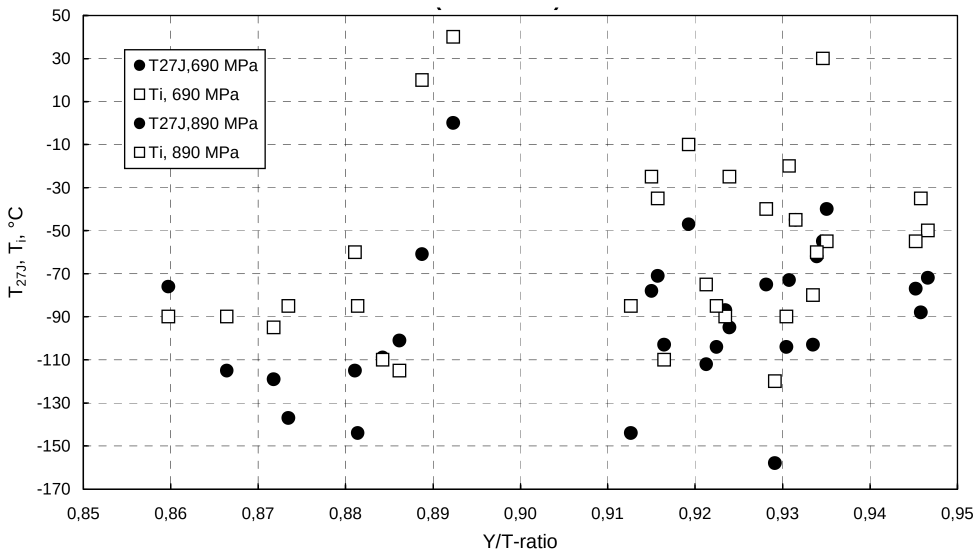

Figure 18: Toughness properties depending on the yield to tensile strength ratio for S690 and S890 [4]

3.3 Welding and weld resistance

(1) General recommendations for welding of TM- and QT-steels are given in EN 1011-2 Welding – Part 2 – Welding of ferritic steels. TM-steels have due to their low contents of alloying elements and low carbon equivalents a wider window for heat input and preheating than QT-steels. Even for thicker plates of S460M preheating can be omitted and welding costs thus be reduced. Figure 19 gives an example for a recommendation for preheating.

Steel grade	Maximum combined plate thickness, mm										
	30	40	50	60	70	80	90	100	110	120	130
S460M, ML	Room temperature, RT							75°C			
S690Q, QL, QL1	RT		75°C				100°C		150°C		
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.											

Figure 19: Example for preheating recommendations [3]

(2) For high strength steels the position of welds normally is not provided at the location of maximum stress. Therefore EN 1993-1-12 allows the use of undermatching electrodes and the design of welds according to the rules for other structural steels by substituting the tensile strength f_u of the parent metal by the ultimate strength of the filler metal f_{eu}, see figure 20.

Strength class	35	42	55	62	69
Ultimate strength f_{eu} N/mm^2	440	500	640	700	770

Figure 20: Ultimate strengths f_{eu} of electrodes [9]

(3) To limit the ductility requirements for structures made of high strength steels EN 1993-1-12 gives some limitations for plastic design, e.g. limitation of fillet welds to a length of 50a unless realistic shear distributions are calculated and exclusion of semi-rigid joints.

3.4 Stability

(1) There are two limit states that may adversely affect the economic exploitation of the full potential of resistance of high strength steels [6]:

1. The effects of local buckling for thin walled components,
2. Fatigue.

(2) EN 1993 - Eurocode 3 - Part 1-5 – Plate buckling – offers various methods to verify plate buckling [7]

1. Method with plate buckling curves
   a) with several slendernesses for stress components of the full stress field and subsequent interaction,
   b) with a single slenderness for the full stress field using FEM,

2. Methods based on test simulation techniques using FEM.

(3) For the mobile crane construction industry, where ultra-high strength steels are used, the method based on test simulation techniques is usual, as high computer costs are compensated by the effects of serial production. Figure 21 shows a test specimen for a rectangular hollow section in a 4-point-bending test and figure 22 gives a view on the local buckling developing in the compression zone under loading.

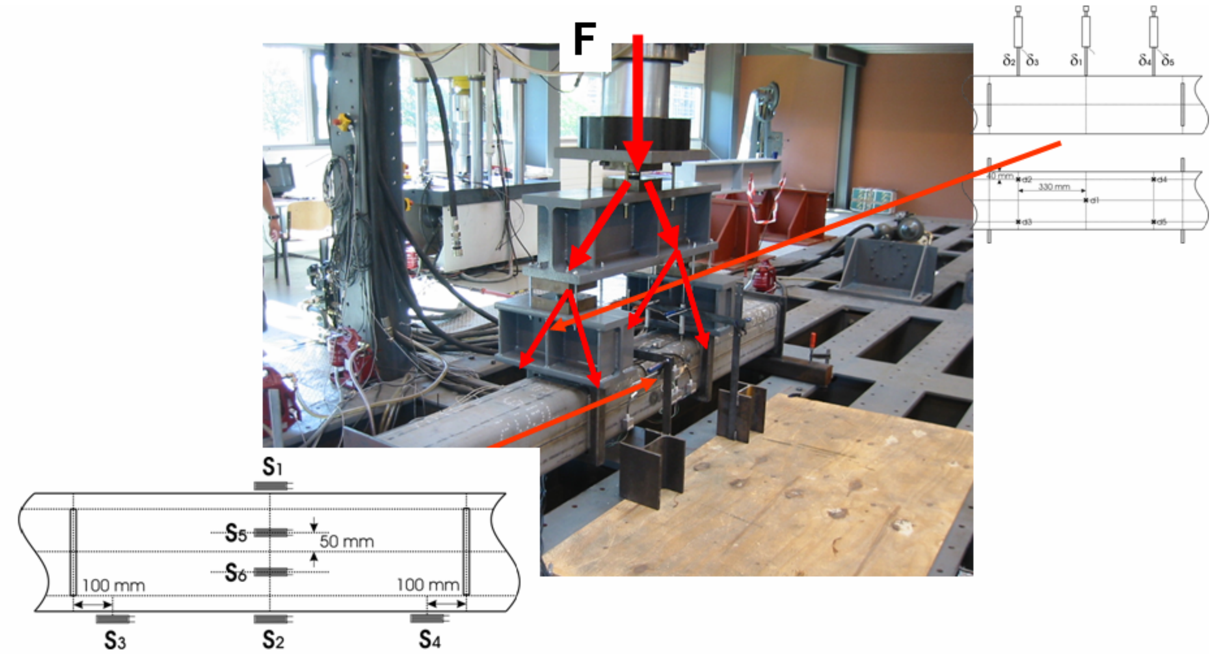

Figure 21: 4-point-bending tests of beams with hollow sections [6]

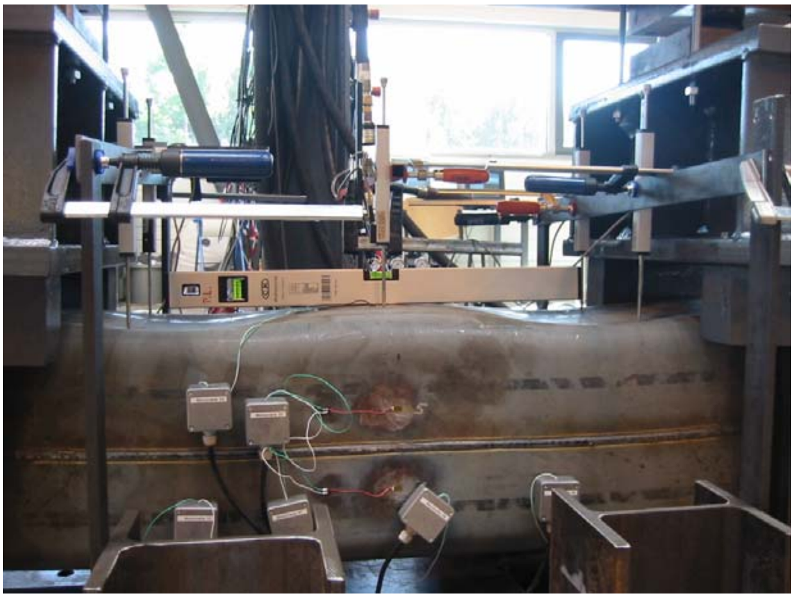

Figure 22: Formation of plate buckles in the flange in compression [6]

(4) Figure 23 shows the FE-Models with a meshing according to figure 24 that produce force-deformation curves as given in figure 25.

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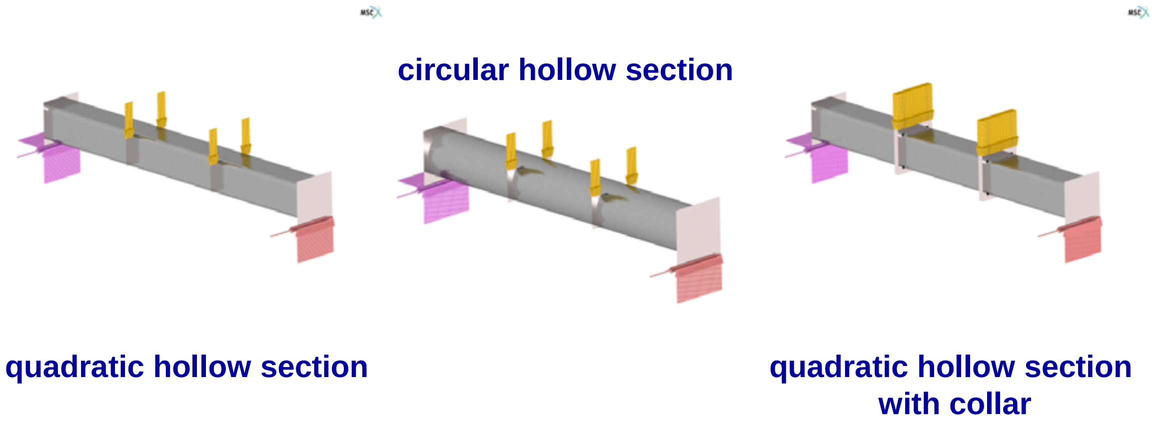

Figure 23: FE-models for beams with various profiles [6]

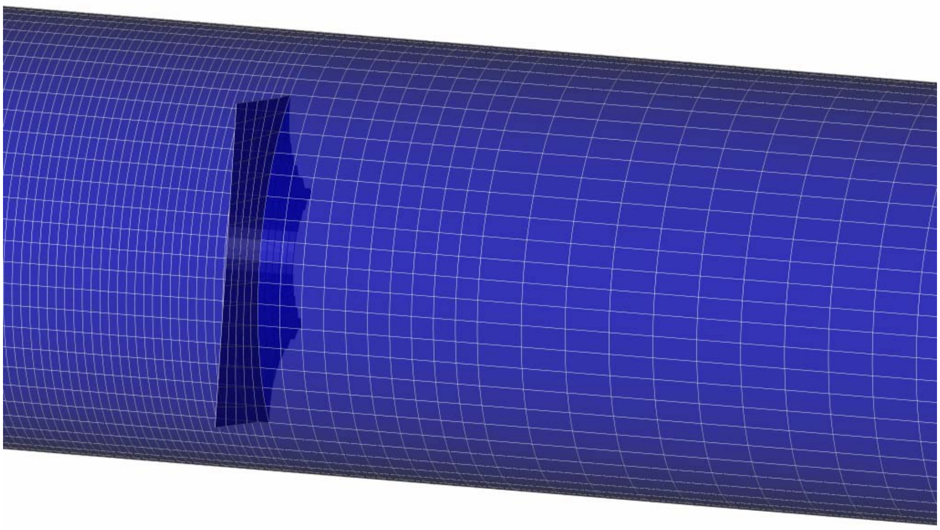

Figure 24: Meshing of FE-Model [6]

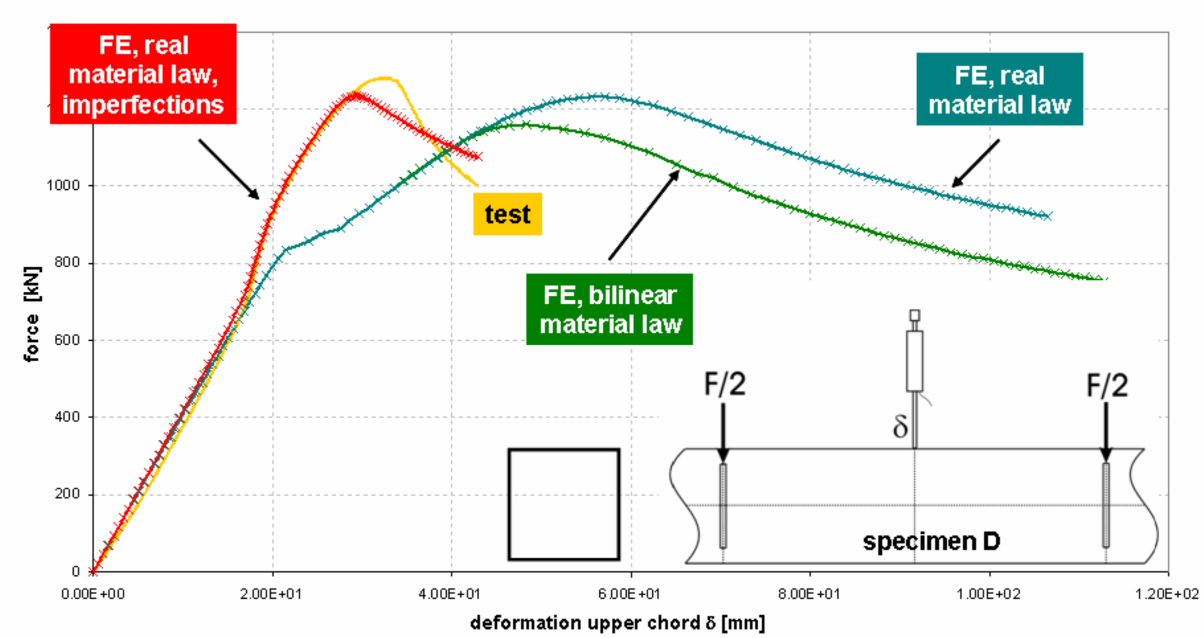

Figure 25: Force-deformation curves from test and from calculations [6]

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(5) Figure 26 gives a comparison of the test results for various cross-sections with the results from numerical test simulations with and without taking imperfections into account and with the results of hand calculations using Eurocode 3 – Part 1-5.

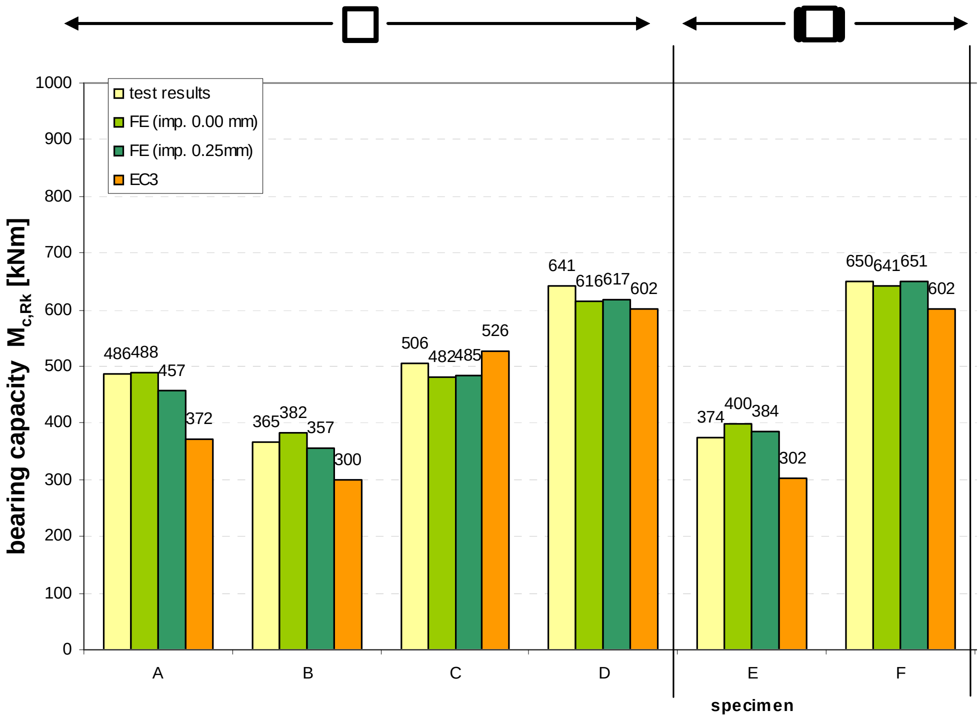

Figure 26: Maximum loads from tests and from calculations [6]

(6) For rectangular hollow sections figure 27 demonstrates the differences between various approaches of hand calculations according to Eurocode 3 – Part 1-5 and test results. The approach with a single slenderness gives the best results.

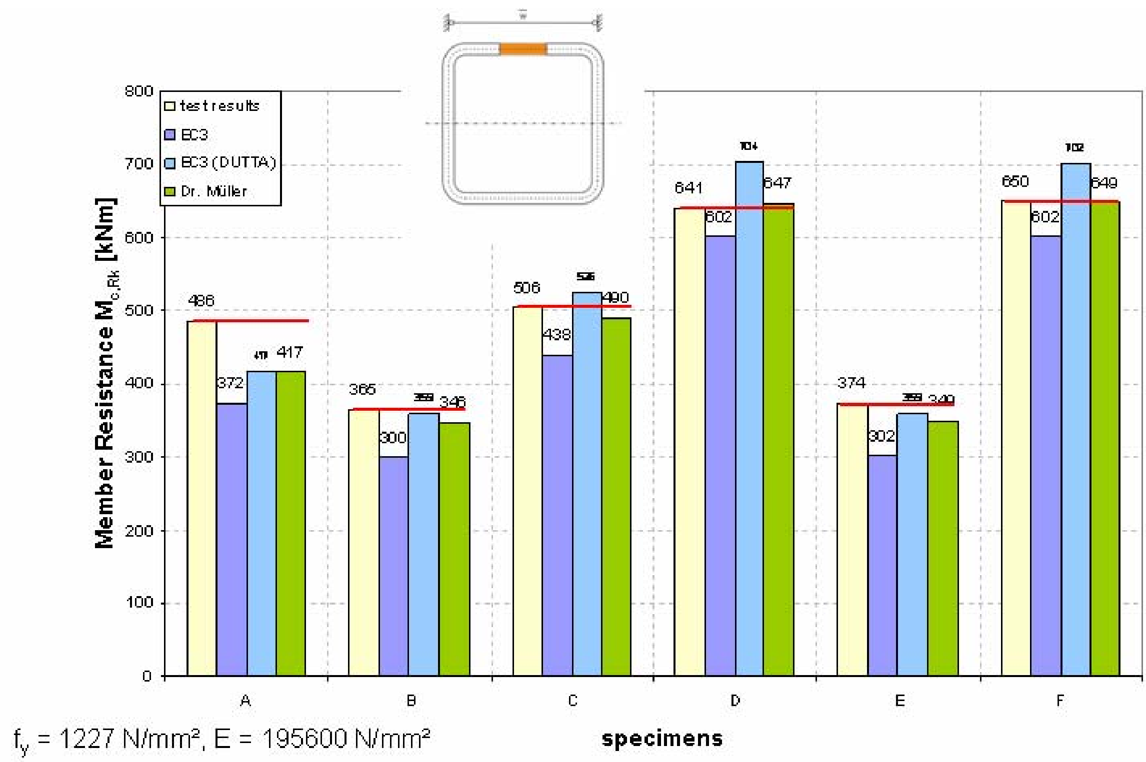

Figure 27: Maximum loads form tests and various methods of hand calculation [6]

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(7) The conclusion from effects of local buckling of flat plates is that strength reductions from these effects should be avoided by an appropriate shape of the cross-section taking advantages from either shell effects or effects of multiple folding. Stiffeners are in general not used to avoid strength reductions due to fatigue at welded areas, see figure 28.

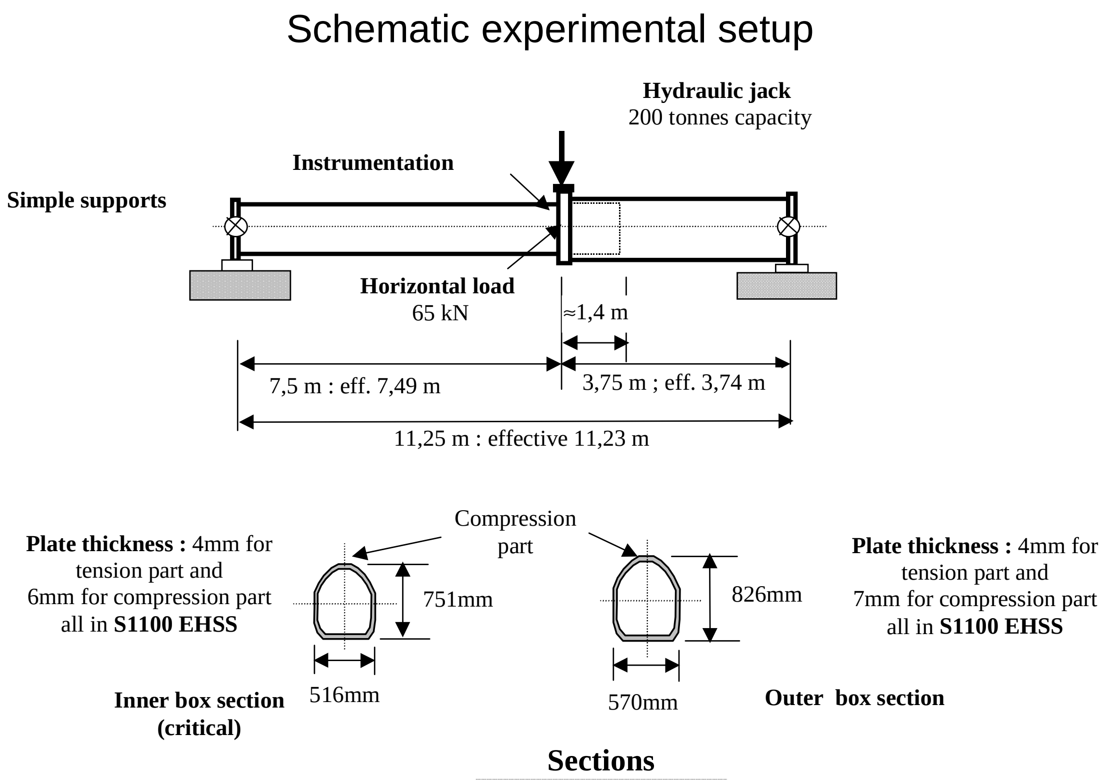

Figure 28: Test with a realistic mobile crane structure [6]

3.5 Fatigue [5]

(1) Fatigue rules are given in EN 1993 – Eurocode 3 – Part 1-9. The fatigue resistance of welded steel components is in general approximately independent on the steel grade [5].

(2) Hence in welded structures a balance has to be kept between static and fatigue design the more the ratio between variable and permanent loads and thus the magnitude of the stress variations is increased.

(3) Solutions can be [5]:

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

(4) In order to get a better fatigue strength with high strength steels the detailing is such that the stress flow is improved and structural discontinuities in highly stressed regions are avoided. Welds and details are put in zones near the neutral axis or where the mean

stress is compressive. If welded, welds are full penetration butt welds, and any misalignment is avoided that could cause secondary bending moments.

(5) The requirements for welding high strength steels are set higher than for normal structural steels to get better fatigue strength; e.g. manual arc welding should preferably be avoided and the rules in EN 1011-2 for pre- and post-heating temperatures should be followed.

(6) Post-weld-improvement methods (e.g. grinding, TIG dressing of the weld toe, needle peening or hammer peening) aim at:

- reduction of local stress concentrations
- creation of crack initiation phase
- alteration of the residual stress-field of the superficial layer.

(7) Consequently the improvement methods may either smoothen the weld bead – base plate transition and eliminate surface defects or change tensile residual stresses into compressive stresses at hot spots.

(8) The fatigue strength increase is more significant for low fatigue categories. An upper bound is achieved, when the fatigue strength reaches about category 125 [5].

4. EXAMPLES FOR THE USE OF HIGH STRENGTH STEEL FOR BUILDINGS AND BRIDGES

4.1 Buildings

(1) The use of S460 in buildings, e.g. for columns and beams of composite decks in parking houses is already frequent [4].

(2) An example for the use of steels S690 is the roof truss of the Sony Centre in Berlin, that is used to suspend several storeys of the building to protect an old masonry building "Kaisersaal" integrated in the building from being loaded, figure 29.

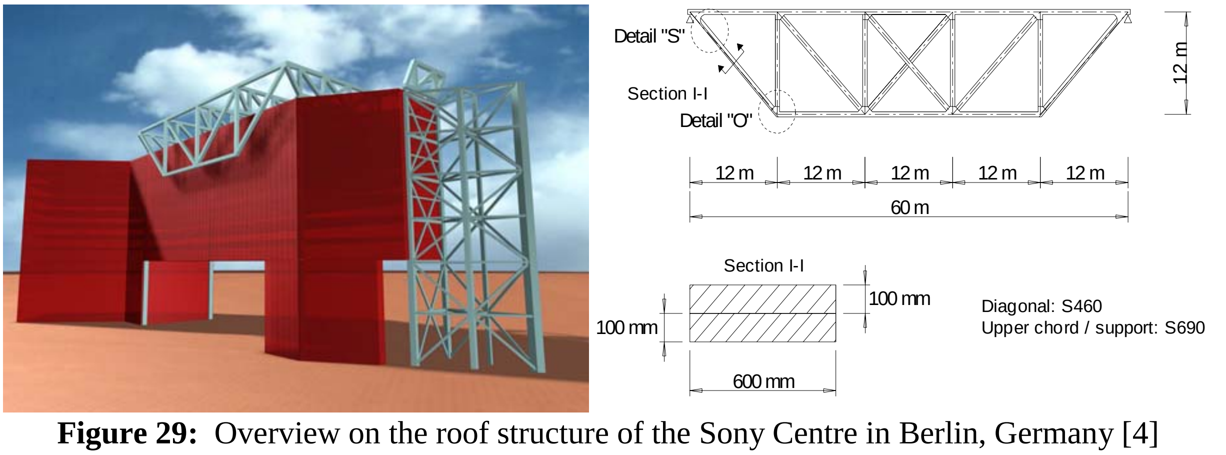

Figure 29: Overview on the roof structure of the Sony Centre in Berlin, Germany [4]

(3) The truss structures composed of components with solid rectangular shape was made of steel S460 and S690 to keep the dimensions of the cross-sections small. The safety

check to avoid brittle fracture followed EN 1993-1-10 assisted by testing with large scale specimens, see figure 30.

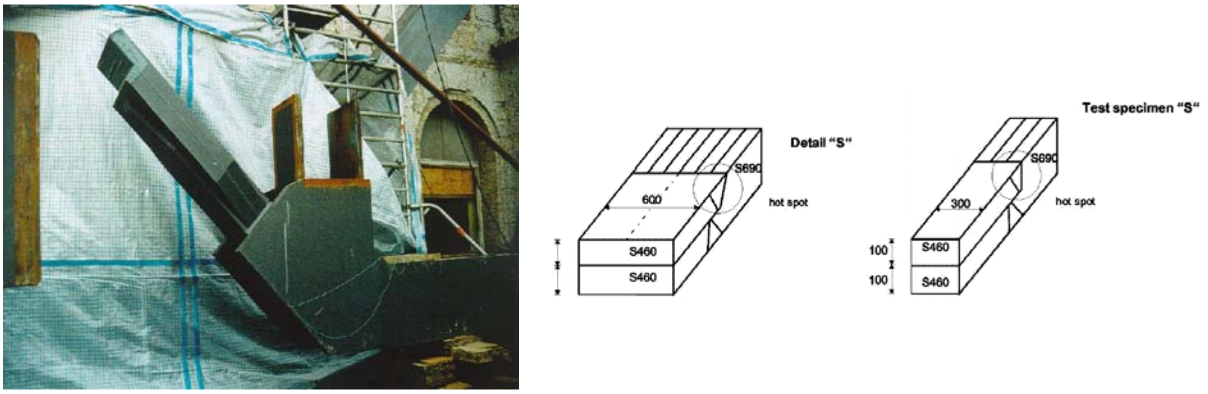

Figure 30: Details of connection of thick plates made of S460 and S690 for testing

4.2 Bridges

(1) Examples for the use of steels S460 in bridges are the Rhine bridge Düsseldorf Ilverich, where the pylon was made of S460 to avoid preheating and keep the welding costs low, figure 31.

Figure 31: Rhine bridge Düsseldorf-Ilverich with tension tie in the pylons made of S460 TM [4]

(2) A spectacular example for the use of S460 M is the Millau-Viaduct in France, figure 32, where a total of 43000 t of steel plates have been applied with thicknesses up to 80 mm for the entire central box and some connecting elements.

Figure 32: Millau-Viaduct and launching of pylon [3]

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4.3 Future aspects

(1) An ideal structural concept for the use of high strength steel is to detail such that:

1. predominantly membrane actions occur in the steel plates and local bending and local buckling is reduced,
2. welding is as far as possible restricted to butt-welds with a high fatigue class where welding quality measures are effective,
3. welded stiffeners and other welded attachments with a low fatigue class are reduced to a minimum or displaced in areas without large stresses.

(2) This could lead to sandwich concepts as the SPS-Steel Polymer Sandwich-Concept, that first has been used for repairing the decks of ferry-boats by SPS-Overlay technique, see figure 33, but that also is the basis for new solutions for the hull of a ship with a minimum of welded ribs and without intersections of ribs.

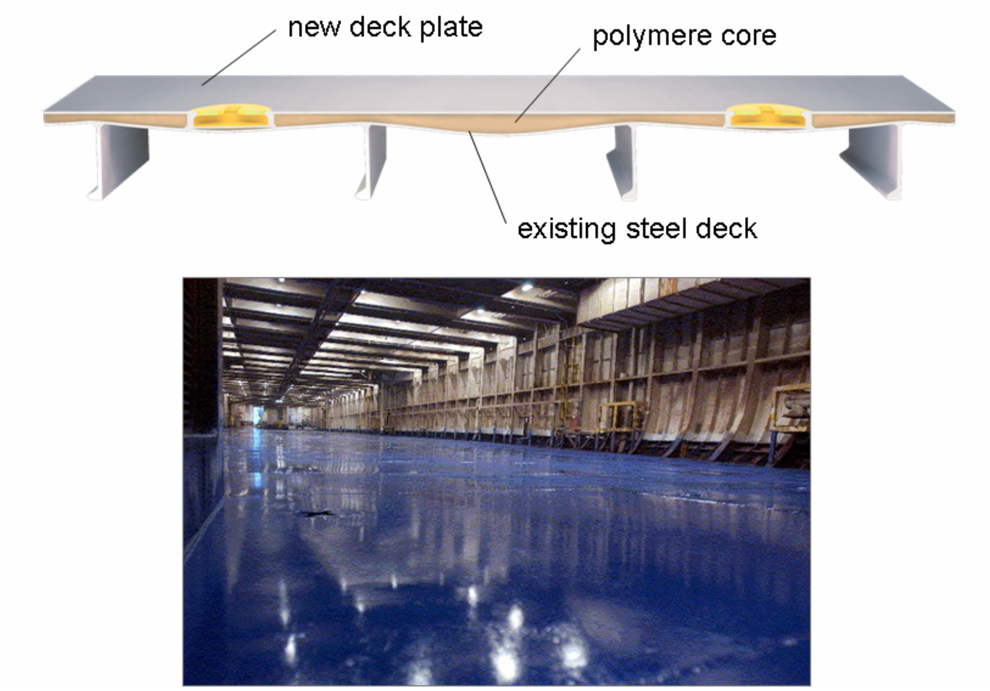

Figure 33: SPS-Overlay-techniques to refurbish the steel deck of roll-on-roll-off-ferry boats [IE-Engineering, Canada]

(3) First examples for bridges are also overlay-solutions to refurbish existing steel decks of steel bridges, figure 34 and to design new plate structures, see figure 35 and bridge cross-sections, figure 36.

Figure 34: SPS-Overlay-technique to refurbish the steel deck of the Schönwasserpark-bridge Krefeld, Germany [Krupp]

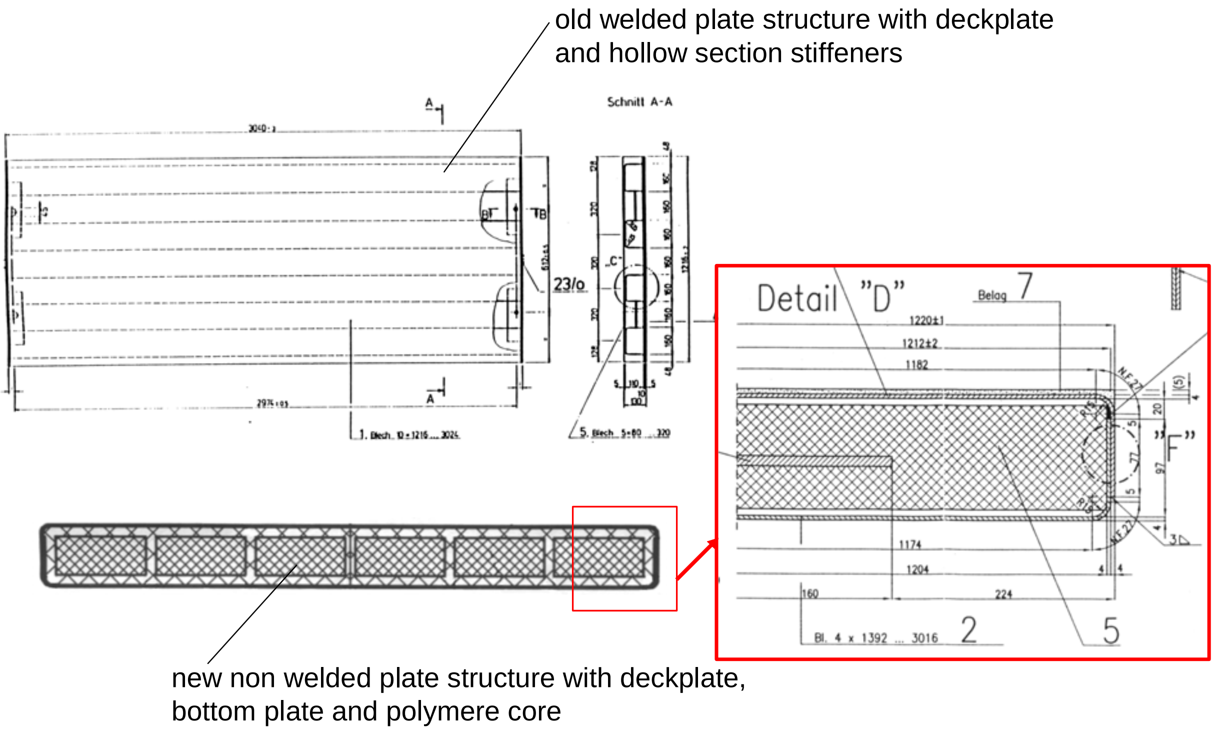

Figure 35: New plate structure using SPS for temporary bridges [Krupp]

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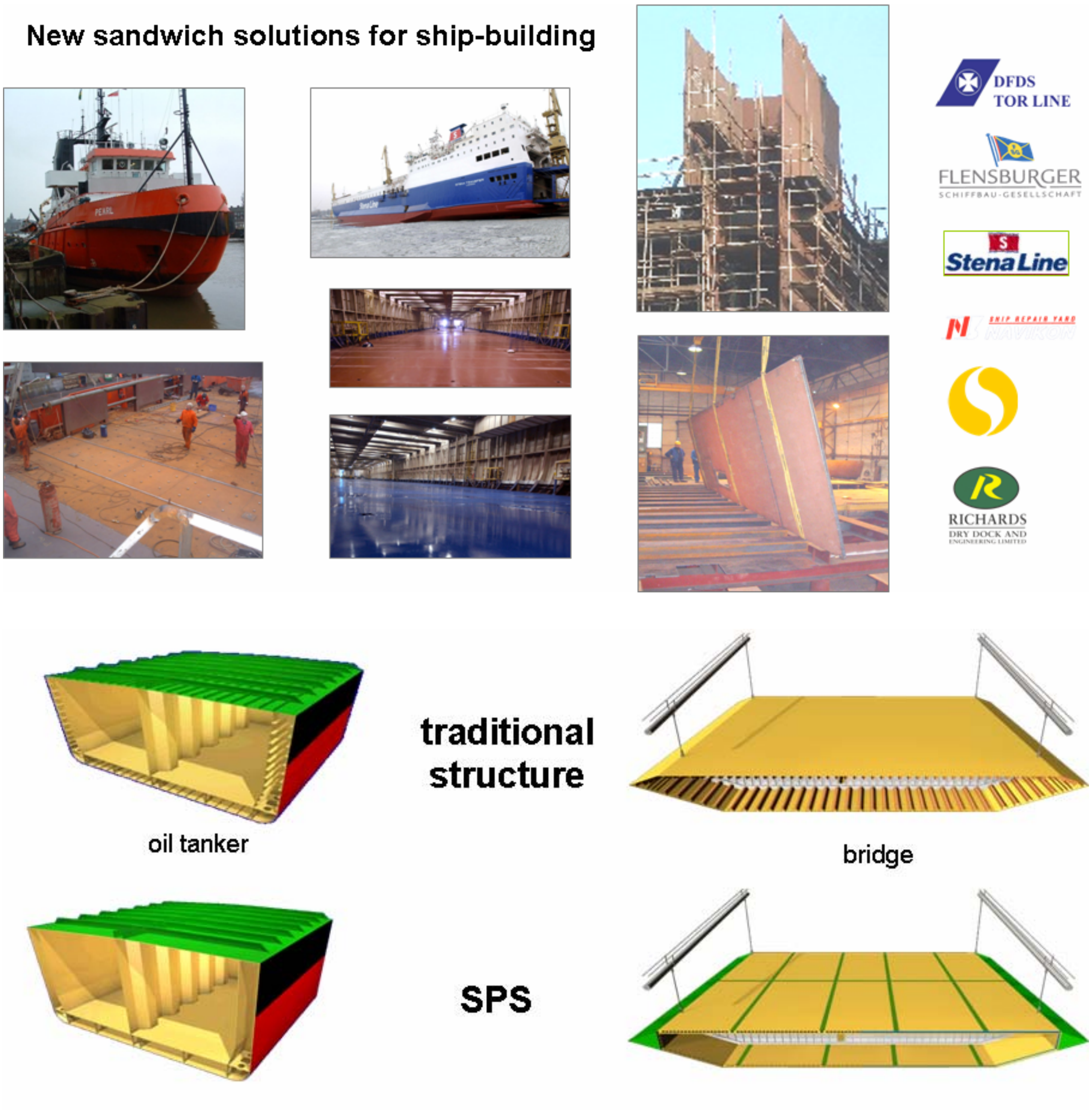

Figure 36: Plated structures without stiffeners for ship hulls and bridges [IE-Engineering, Canada]

5. CONCLUSIONS

(1) Modern high strength steels and ultra high strength steels offer besides their strength excellent toughness properties and good weldability.

(2) Whereas S460 is already included in EN 1993 - Eurocode 3 - Part 1-1 and is frequently used for buildings and bridges, the use of steel grades higher than S460 so far is rare. The new EN 1993 - Eurocode 3 - Part 1-12 – Additional rules for the extension of EN 1993 up to steel grades S700 – gives all design rules necessary to avoid obstacles for the use of such steels in the Civil Engineering field.

(3) In other areas, e.g. for mobile crane construction, ultra high strength steels up to S1100 are usually used because of the extreme light weight requirements.

(4) An extension of Eurocode 3 to include also these grades would be easy.

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