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ALUMINIUM PLATED STRUCTURES AT ELEVATED TEMPERATURES

Jørgen Amdahl
Department of Marine Structures
Faculty of Marine Technology
Norwegian University of Science and Technology
Trondheim, Norway

Nina Kristin Langhelle
Division of Structural Engineering
MARINTEK
Trondheim, Norway

Steinar Lundberg
Hydro Aluminium
R&D Materials Technology
Karmøy, Norway

ABSTRACT

The paper focuses on the behaviour of aluminium plate girders, predominantly loaded in shear. Such girders, which are frequently used in ships and modules (living quarters) in aluminium, may experience a dramatic reduction in strength due to the vulnerability of aluminium material to heating. Accurate assessment of the strength may therefore be essential to ensure adequate safety during accidental fire. Laboratory tests with plate girder specimens have therefore been conducted at room temperature and elevated temperature. Test results are compared with non-linear FE predictions and Eurocode 9 capacity formulations.

INTRODUCTION

Aluminium is typically used as construction material in weight-critical structures. Examples of this in the marine industry are high-speed vessels, superstructures of cruise ships and accommodation units on floating platforms for oil exploitation. A structural component of considerable interest is the plate girder. The web plate of such girders is very slender so that shear buckling may occur for a very low stress level. However, in the post-buckling range tensile membrane stresses may be allowed to develop so that the shear capacity of the web plate may increase considerably beyond level of shear buckling.

If the plate girder does not undergo repeated buckling due to cyclic loading this extra capacity may be taken into account. Numerous experiments have been carried out for plate girders, and the tension field theory has been found to be a very good predictor for the ultimate strength notably for steel girders. For aluminium girders the experiments show a wider scatter, probably due to the lack of yield plateau for aluminium, but reliable design formulations have been developed on the basis of tension field theory. This has been implemented in design codes, such as Eurocode 9. A thorough presentation of the theory and comparison with experimental results can for example be found in Høglund (1997).

While ultimate capacity has been thoroughly investigated at normal temperatures, little work has been undertaken to document the behaviour at elevated temperatures. This is of relevance during accidental fires. Unprotected aluminium subjected to the effect of proximate hydrocarbon fires will reach critical temperature for collapse within a few minutes. Protected structures will experience smaller rates of heating whereby accurate assessment of time to failure may become important. The purpose of the work reported is to study the degradation in the capacity of plate girders at elevated temperatures and to check whether simple formulas can be used to assess this degradation of strength. In addition, with the advent of advance finite element analysis of fire exposed platform structures, test results are also used to verify the performance of two computer codes (ABAQUS, 1997 / USFOS, 2000).

RULE REQUIREMENTS

The **SOLAS** (Safety of Life at Sea) regulations have reqSuirements to fire safety in general and some additional requirements for different materials. The regulations of "Fire safety measures for passenger ships" (Chapter II-2 Part B) in 3.1 Regulation 23. Structure section 2 states for use of aluminium:

For ships or part of ships in aluminium alloy the following requirements shall apply:

The temperature of the structural aluminium shall not rise more than 200 °C above the ambient temperature at any time during a standard fire exposure.

Special attention shall be given to the insulation of aluminium alloy components of columns, stanchions and other structural members required to support lifeboat and liferaft stowage, launching and embarkation areas, and "A" and "B" class divisions to ensure that the members supporting lifeboat and liferaft areas and "A" class divisions, the temperature rise limitation shall apply at the end of one hour, and for members supporting "B" class divisions, the temperature rise limitation shall apply at the end of half an hour.

The same requirements apply also for cargo ships (Chapter II-2 Part C Regulation 42).

The requirement for load-bearing aluminium structures is a maximum temperature rise above the ambient temperature. The maximum allowable temperature is therefore dependent of the ambient temperature. In standard fire tests of decks and bulkheads in aluminium the ambient temperature is often 15 – 25 °C. This means that the maximum allowable temperature will be in the range of 215 – 225 °C.

Eurocode 9 for Design of aluminium structures became in 1997 a European pre-standard. Part 1.2 deals with basic principles and rules, material properties at elevated temperatures, calculation of temperature development in members and calculation of the structural resistance.

The calculation of temperature development in members is the most challenging part. The standard gives some simple rules for both unprotected and protected members. For more exact calculations finite element methods have to be used.

The properties of both aluminium and the fire protection materials change with increasing temperatures. For unprotected aluminium structures, this can easily be taken into consideration by changing the aluminium properties when the temperature of the structure rise.

For *insulated* aluminium structures this is more difficult since the properties of the insulation materials also change, and the temperature gradient over the insulation layer may be rather high. For steel structures a European pre-standard has been established: "Method of test for the determination of the contribution to fire resistance of structural members. Part 4 Steel elements". The standard describes a method for determining the thermal properties of insulation materials to be used in calculation of the steel temperatures. A similar standard for insulated aluminium structures does not exist.

The load-bearing function of an aluminium structure or a structural member may be assumed to be maintained after a time *t* in a given fire if:

$$E_{fi,d} \leq R_{fi,d,t}$$

where $E_{fi,d}$ is the design effect of actions for the fire design situation (the internal forces and moments $M_{fi,Ed}$, $N_{fi,Ed}$, $V_{fi,Ed}$ individually or in combination); $R_{fi,d,t}$ is the design resistance of the aluminium structure or structural member, for the fire design situation, at time t, ($M_{fi,t,Rd}$, $M_{b,fi,t,Rd}$, $N_{fi,t,Rd}$, $V_{fi,t,Rd}$ individually or in combination).

$R_{fi,d,t}$ shall be determined for the temperature distribution in the structural members at time t by modifying the design resistance for normal temperature design, determined from Eurocode 9 Part 1.1, to take account of the mechanical properties of the aluminium alloys at elevated temperature. This is done by using a stress ratio, $k_{0.2,\theta}$. The stress ratio depends on alloy, temper and temperature level of the member. The classification of cross sections according to normal temperature design can be used for fire design without any changes.

Du to the high thermal *diffusivity* of aluminium (~ 5-6 times that of steel) the temperature distribution over the cross section tends to be uniform. In this case the following equations can be used for calculating the design resistance, $R_{fi,d}$:

Tension members: $$N_{fi,q,Rd} = k_{0.2,q} N_{Rd} (g_{M1} / g_{M,fi})$$

Beams: $$M_{fi,q,Rd} = k_{0.2,q} M_{Rd} (g_{M1} / g_{M,fi})$$

$$V_{fi,q,Rd} = k_{0.2,q} V_{Rd} (g_{M1} / g_{M,fi})$$

Columns: $$N_{b,fi,q,Rd} = k_{0.2,q,\max} N_{b,Rd} (g_{M1} / 1.2\, g_{M,fi})$$

where: $k_{0,2,\theta}$ is the 0,2% proof stress ratio for the aluminium alloys strength at temperature θ_{al}, $k_{0,2,\theta,max}$ is the 0,2% proof stress ratio of aluminium alloy at temperature θ_{al} equal to the temperature $\theta_{al,max}$.N_{Rd} is the design resistance of the net section for normal temperature design, M_{Rd} is the moment resistance of the cross-section for normal temperature design, V_{Rd} is the shear resistance of the net cross-section for normal temperature design, $N_{b,Rd}$ is the buckling resistance for normal temperature design. $\gamma_{M,fi}$ is the partial safety factor for the relevant material property, for the fire situation. γ_{M1} is the partial safety factor for the relevant material property.

PLATE GIRDER TESTS

Test specimens

Test specimen dimensions are given in Figure 1. Due to the problem of heating large specimens the girder contains only one stiffener spacing. The thickness of the flanges is such that significant anchoring of tension filed in the flanges is to be expected. Double stiffening is provided at the ends in order to ensure anchoring of the tension field in the web plate.

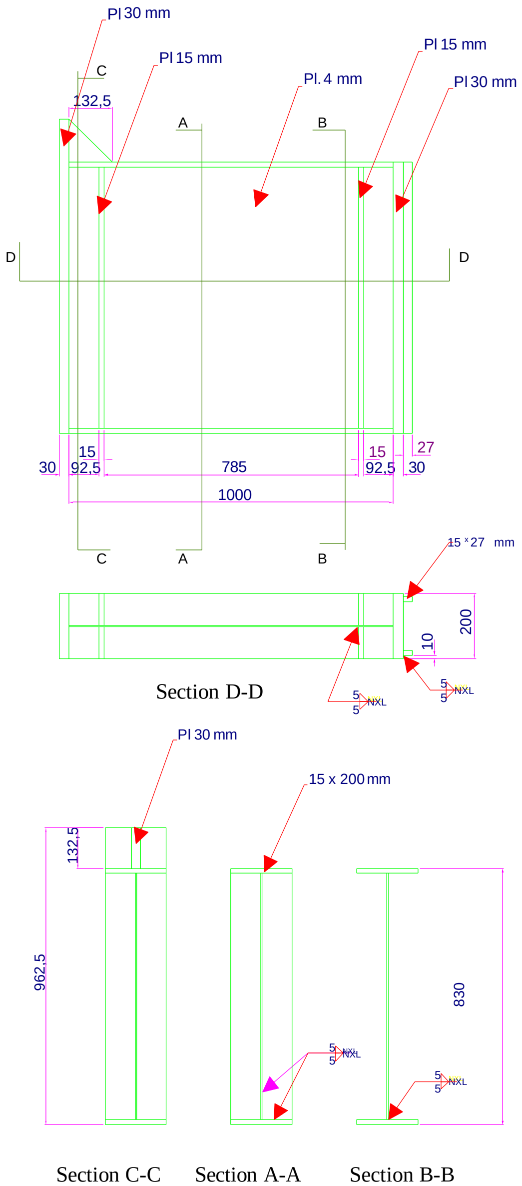

Figure 1 Test specimens

Alloy EN AW-5754 H34 is used in the web, while EN AW-5083 H34 is used in the flange and end plates. The welds are in accordance with BS 8118 Part 2, normal quality level using weld thread 5183. Material properties are measured in tension coupon tests. Stress-strain curves are shown in Figure 2. Key figures are given in Table 2 and Table 3 The high temperature tests are carried out at constant temperature, subsequent to a heating period of 20 minutes. A few additional test were carried out with both a heating period of 60 minutes and a heating period of 60 minutes followed a holding period of 60 minutes prior to the test. The latter resembles best the procedure adopted in the girder test. These results are given in Table 4. The properties of EN AW-5754 are relatively insensitive to the heating procedure, but EN AW-5083 exhibits considerable scatter both a 200 ^{0}C and 225 ^{o}C. It is observed that both materials are more ductile at high temperatures. The ultimate strength is, however, reached at a decreasing strain level followed by a long softening phase prior to fracture. This property tends to cause localisation of strains in a structure subjected to large inelastic deformations.

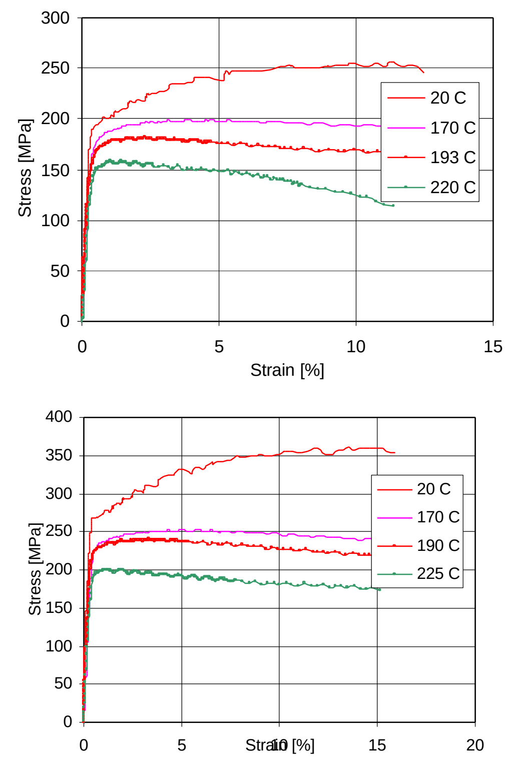

Figure 2 Stress-strain relationship for EN AW-5754-4mm (left) and EN AW-5083-15mm (right) - 20 minutes heating period

Heating equipment

The test specimens are heated by use of electrical heating elements attached to the specimen surface. Langhelle (1998) has used this technique with satisfactory results in aluminium beam - and column buckling test. A constant, uniform temperature level of approximately 200 °C and 225 °C is aimed at. The 4 mm thick web is supplied with heating elements on one side, while the strain gauges and thermocouples are placed on the other side in order to avoid direct contact with heating elements. Flanges with wall thickness 15 mm are also supplied with heating elements on one side. The 30 mm thick end plate has heating elements on both sides.

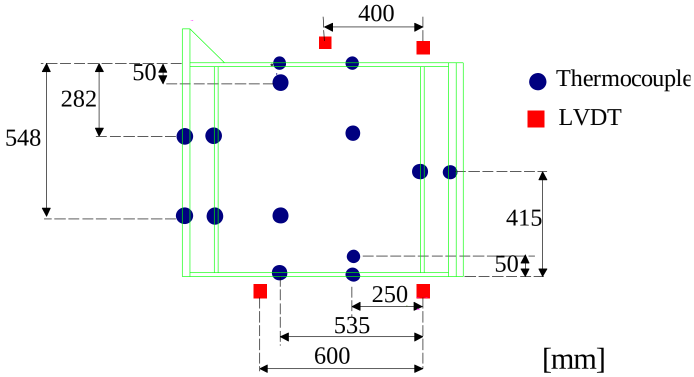

Figure 3 Positions of thermocouples and linear variable transducers (LVDT).

Figure 3 shows the positions of the thermocouples and the linear variable transducer (LVDT). The LVDT under the stiffener is applied to check any possible compression of the stiffener / end plate. Two LVDTs are placed on the lower and upper flange to check the extent of deformation of the flanges during the test.

Thermocouples are provided in order control the hating and to check how well a uniform temperature distribution is obtained. The thermocouples on the web are placed outside the expected location of tension field to avoid local stress concentrations, because a hole with diameter 0.9 mm is needed to fix each thermocouple. The thermocouples on the flanges and stiffeners are all placed 50 mm from the free edge, while they are positioned on the free edge on the end plates.

Test results

The load-displacement curves from the tests are shown in Figure 4. For the specimen tested in room temperature it is possible to follow development of web buckling and crack growth in the heat affected zone (HAZ). A tension field develops along the diagonal of the web.

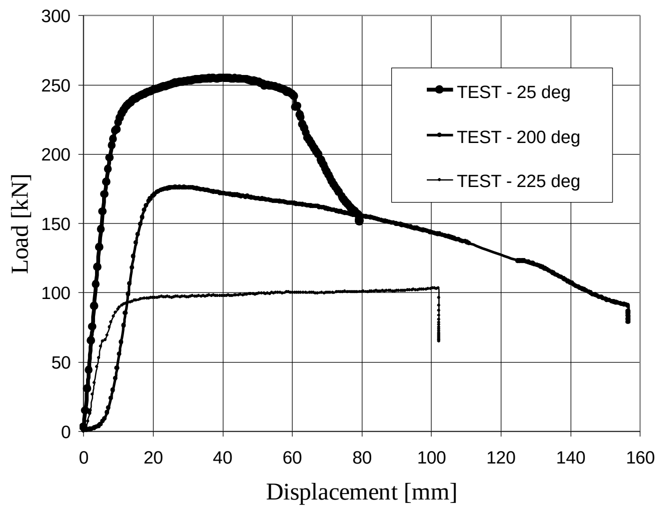

Figure 4 Load displacement characteristics for the specimens.

After web buckling has become visible, the specimen can carry a load increase of 10 % before the ultimate capacity is reached. It evident from the pull-in of the flanges, shown in Figure 5, that a significant tension field has developed and that this has been partly anchored in the flanges. The load level remains approximately constant while the displacement increases up to 60 mm. Then a crack occurs in the HAZ in lower right corner of the specimen, see Figure 6. As the crack grows, the capacity falls dramatically, and the test is terminated at a displacement of 80 mm.

Figure 5 Specimen tested at room temperature

Figure 6 Crack developing during the last part of the test

Figure 7 shows a photo of the specimen tested at 200 ^{0}C. The deformation field resembles that obtained in the normal temperature test. The ultimate capacity is 70 % of the capacity at room temperature and is achieved after a displacement of only 20 mm, compared to the 40 mm at room temperature. The capacity degrades monotonously after the ultimate strength is reached. These effects are attributed to the small ultimate strain (ε_u) and material softening observed in the tension coupon tests. In spite of this the deformation is taken up to 200 mm with no cracks occurring. This confirms the increase in ductility obtained in the high temperature, tension coupon tests and is also in agreement with observations reported by Langhelle (1998).

Figure 7 Specimen tested at 200 °C.

The ultimate capacity of the specimen tested at 225 ^{0}C is only 40% of that at room temperature. This specimen shows also a ductile behaviour, with slightly increasing resistance up to 100 mm when the test is terminated. In this case the hinge formation in the flanges are not so evident, refer Figure 8, which could indicate less anchoring in the flanges.

Figure 8 Specimen tested at 225 ^{0}C

The recorded capacity is considerably lower than anticipated. It is not possible explain this with certainty, but according to thermocouple recording one did not succeed in keeping the correct temperature level to the same degree as in the test at 200 ^{0}C. In a few locations the temperature exceeds 250 ^{0}C, but these is likely to be due to direct contact between the thermocouples and the electrical heating elements. If the material should have been subjected to excessive heating, this could have a significant impact on the results, because the strength is very sensitive to the temperature at this level.

NUMERICAL ANALYSES

Numerical analyses are carried out with the non-linear FE codes ABAQUS and USFOS. The analyses with ABAQUS are based upon the 4-node shell element S4R. Satisfactory accuracy is obtained with 40 x 40 elements in the web plate between the stiffeners. The material model assumed is a continuous true stress-strain relationship, The mesh adopted in USFOS analysis is cruder, namely 16 x 16. A stress-resultant material model is adopted for the shell element, which obeys a linear elastic-linear hardening law. The effect of HAZ is simulated by attributing reduced stress values for elements adjacent to the welds. In the analysis the web plate is assumed to have an initial deflection compatible with governing buckling mode with a maximum amplitude of 2 mm. The imperfection is included to trigger buckling of the web, the actual magnitude is believed to have moderate significance.

Figure 9 shows the plate girder in deformed configuration at ultimate collapse at normal temperature. The simulations are done with USFOS, but ABAQUS yields similar results. The agreement with the true deformation mode, shown in Figure 5, is evident.

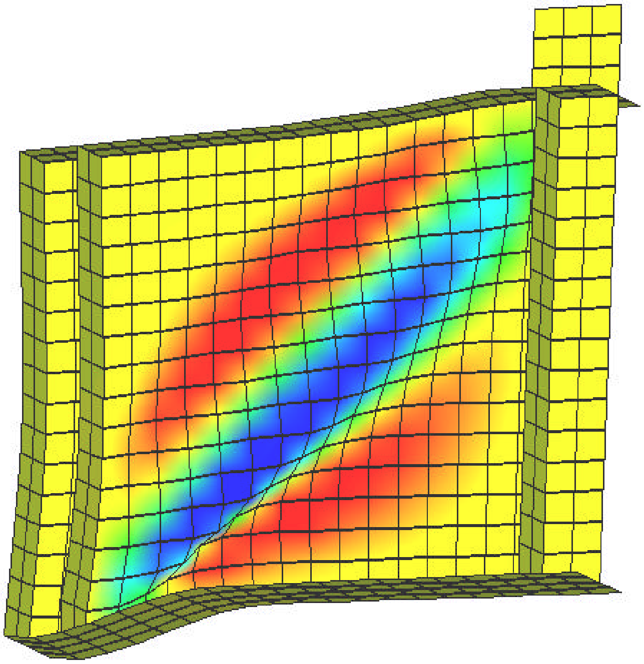

Figure 9 Plate girder at ultimate capacity-normal temperature – simulation with USFOS.

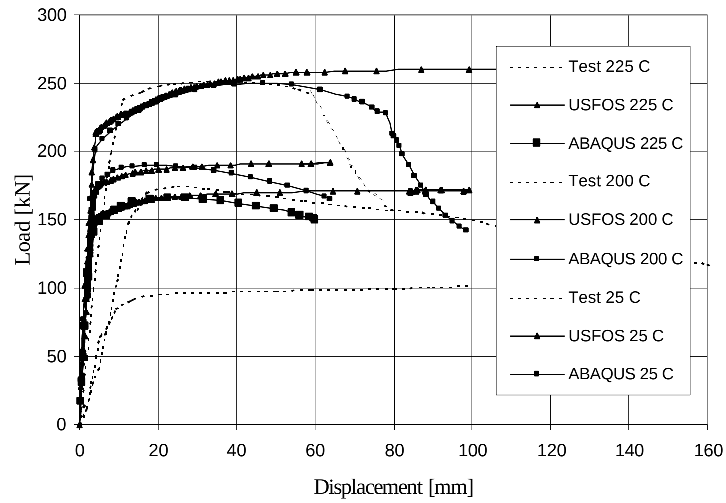

Figure 10 Load displacement characteristics and numerical analyses for specimen tested at room temperature (no HAZ effect)

Figure 10 shows simulated behaviour along with test results. It is observed that USFOS and ABAQUS yield fairly similar results as concerns ultimate strength. The unloading in the post-collapse range is not captured by USFOS, because global unloading was not implemented for analysis with shell elements at the time of executing the analyses. The more sudden loss of capacity in the test compared to ABAQUS analysis is due to crack initiation and growth.

At normal temperature without the effect of HAZ, the ultimate strength is very close to the recorded values. If HAZ is included, assuming a strength reduction factor of 20% in the web, the ultimate capacity is predicted to be 225 kN.

At 200 ^{0}C both programs overpredict the ultimate strength by approximately 8%, neglecting HAZ effects. If the strength in HAZ is reduced by 20%, as in the normal temperature analyses, the capacity is found to be virtually identical to the test value.

This may be a coincidence, because the HAZ softening is expected to be less severe at high temperatures.

At 225 ^{0}C the ultimate capacity comes out to be 165 kN, while the load is only slightly above 100 kN at termination of the test. The large deviation may probably be attributed to tension field discrepancies. Similar to the 200 ^{0}C simulation, a significant part of the tension field is anchored in the flanges in the simulations, whereas this beneficial effect seems to be almost absent in the test.

COMPARISON WITH EUROCODE 9

The capacity of the plate girders is calculated according to Eurocode 9, Part 1.1 and 1.2. Calculations are carried out both for plate girders with stiffeners at supports (assuming rigid end posts) and for webs with intermediate stiffeners. The assessment is based upon measured material properties as well as code specifications. When code material properties are used at elevated temperatures, the reduction factors are applied separately to the contribution from the web and flanges, respectively. In all calculations the partial safety factor is assumed equal to unity. Since material properties are not specified for EN AW-5754, reduction factors for EN AW-5454 H32 are used.

Using measured values for 0.2% proof strength the capacity at normal temperature is accurately predicted by means of the tension field theory (intermediate web stiffeners). The requirement for end stiffeners only is very conservative. At high temperatures tension field theory overpredicts the strength, and the capacity approaches the code requirement for plate girders with end stiffeners only at 225 ^{0}C.

There may be several reasons for the discrepancy between tension field theory and test results at high temperature:

The temperature field exceeds the nominal values locally, notably at 225 ^{0}C. Since the proof strength degrades rapidly in this temperature range, the ultimate strength may be severely affected.

Creep may have had some effect: The loading phase of the test was completed within 10-15 minutes, which is a very short period for creep to play a significant role. On the other hand, the creep is exponentially dependent on the stress level, which is close to or equal to the yield stress for a very high temperature.

Thermal expansion reduces the effective strain levels.

If code specifications for material properties are used the capacity is underestimated both at room temperature. At 200 ^{0}C , tension field theory gives a correct value, but is too optimistic at 225 ^{0}C.

Table 1 Plate girder test results and capacity according to design codes

Temp. [^{0}C]	Measured $f_{0.2}$ [MPa] EN AW-5754	EN AW-5083	Cap. [kN] End stiff.	Cap. [kN] Intm. stiff	Cap. [kN] Test
25	191	265	152	255	252
200	167	153	139	205	175
225	140	147	116	188	102
Temp. [^{0}C]	Eurocode $f_{0.2}$ [MPa] ENAW-5754	EN AW-5083	Cap. [kN] End stiff.	Cap. [kN] Intm. stiff	Cap. [kN] Test
25	1.0 / 160	1.0 / 250	135	225	252
200	0.78	0.78	105	176	175
225	0.57	0.705	77	136	102

CONCLUSIONS

The laboratory tests confirm the adequacy of the tension field concept according to Eurocode 9 for plate girders working at normal temperature. At 200 ^{0}C the theory yields somewhat optimistic strength predictions and at 225 ^{0}C it is very non-conservative. It is tentatively suggested that the capacity of plate girder with intermediate stiffeners be assessed on the basis of the tension field concept to 180 ^{0}C - 190^{0}C, but should not be assumed fully effective for higher temperatures.

It is difficult to explain the reason for the discrepancy in the high temperature range. Local hot-spots or non-uniform temperature distribution in the test specimen may be the primary cause, but thermal expansion and creep may also play an important role.

Non-linear FE analysis shows good agreement with tests at normal temperature and at 200 ^{0}C, but fails to capture the dramatic reduction at 225 ^{0}C. The reason for the discrepancy may be the same as quoted in the case of code comparison.

In view of the very limited experimental evidence it is suggested that the SOLAS requirement, allowing a temperature rise of 200 ^{0}C above ambient temperature, be modified to a accept a temperature of maximum 200 ^{0}C.

ACKNOWLEDGMENTS

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Table 2 Tensile coupon tests of EN AW-5754 H34 – 4mm – 20 min. heating period

Temperature [^{0}C]	Proof strength $f_{0.2}$ [MPa]	Ult. strength f_u [MPa]	Ult. strain $\varepsilon_{u\,[\%]}$	Fract. elong. [%]
25	192	257	11.3	15
25	191	255	10.1	14
170	175	199	3.9	43
173	171	195	4.3	41
197	167	181	2.3	29
197	164	182	2.1	31
220	151	164	1.5	25
221	149	158	1.1	30

Table 3 Tensile coupon tests of EN AW-5083 H34 – 15 mm
– 20 min. heating period

Temperature [^{0}C]	Proof strength $f_{0.2}$	Ult. strength f_u	Ult. strain $\varepsilon_{u\,[\%]}$	Fract. elong. [%]
25	262	352	13.5	15
25	267	363	13.9	15
170	227	252	4.9	37
171	228	251	4.6	43
190	223	240	3.4	33
193	220	234	2.7	35
225	192	204	1.1	40
225	195	200	1.1	43

Table 4 Tensile coupon tests of EN AW-57543 H34 –4mm and EN AW-5083 H34 – 15 mm
- varying heating period

Temperature [^{0}C]	Heating period + constant temp.	Proof strength $f_{0.2}$	Ult. strength f_u	Ult. strain $\varepsilon_{u\,[\%]}$	Fract. elong. [%]
EN AW-5754 H34-4mm 200	20	166	182	2.2	30
	60	167	182	2.1	26
	60+60	167	183	2.3	28
EN AW-5754 H34-4mm 225	20	150	161	1.3	28
	60	138	157	1.3	31
	60+60	141	157	1.5	28
EN AW-5083 H34 –15mm 200	20	222	237	3.0	34
	60	153	190	10.9	52
	60+60	153	187	7.8	60
EN AW-5083 H34 –15mm 200	20	194	202	1.1	42
	60	147	164	5.7	44
	60+60	193	209	1.5	33