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PROGRESSIVE COLLAPSE ANALYSIS OF A „LAY-DOWN" STRUCTURE UNDER FIRE CONDITIONS

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ABSTRACT

The scope of this paper is to present the strength capability under fire action of a typical naval top side structure (lay down area structure) of a FSO (Floating, Storage and Offloading Unit).

Where mechanical resistance in the case of fire is required, steel structures shall be designed and constructed in such a way that they maintain their load bearing function the relevant fire exposure. The present study covers the yielding and buckling checks of the top side structure fire exposure. The analysis tools used for the structural analysis of the FSO are DNV Nauticus Hull and FEMAP with NASTRAN NX ver. 11.4.

KEYWORDS: strength, fire, finite element, yielding

1. INTRODUCTION

1.1. Problem definition

The scope of this paper is to present the strength capability under fire action of a typical naval top side structure (lay down area structure) of a FSO (Floating, Storage and Offloading Unit).

Where mechanical resistance in the case of fire is required, steel structures shall be designed and constructed in such a way that they maintain their load bearing function the relevant fire exposure.

The scenario proposed for the structure verification is following:

1. Making the preliminary 3D CAD model;
2. Dimensioning of the structure in terms of the mechanical strength following the requirements to which it is subjected:
   -Hull girder (bending of the ship);
   -Inertial forces (acceleration due to wave motion);
   -Hydrostatic pressures due to sea water action on the structure;
   -Wind forces applied to the structure.
3. Strength verification of the structure under the fire action according to a possible scenario with a thermal flow of 150kW /m^2 (value of the heat flux for a "pool fire", calculated according to the type of fuel) applied to one foot of the structure over a period of 5 minutes (the minimum time for people evacuating before the structure is collapsed);
4. Re-dimensioning the structure if it does not withstand thermal-mechanical stresses

2. STRENGTH ANALYSIS OF THE STRUCTURE

2.1. Methodology

The analyses are performed by means of FE models and by hand calculations based on formulas given by rules and standards.

A structural fire design analysis should take into account the following steps as relevant:

- o selection of the relevant design fire scenarios;
- o determination of the corresponding design fires;
- o calculation of temperature evolution within the structural members;
- o calculation of the mechanical behaviour of the structure exposed to fire.

According to [5] Sec. 9, B300, the structure that is subjected to a fire shall maintain sufficient structural strength before evacuation has occurred.

The following fire scenarios shall be considered:

- jet fires;
- fire inside or on the hull;
- fire on the sea surface.

For this analysis only „jet fire" scenario has been considered.

Assessment of fire may be omitted provided fire protection requirements made in DNV-OS-D301 are met.

The analyses are performed on two steps:

- o Transient Heat Transfer Analysis when the heat flux acts on a part of the structure a time period (for this study it was considered a heat flux of 150kW/m^2 @ 5 min.);

- Static Analysis when the input loads are the temperatures gradient and the rest of the static loads (environmental, operational, etc).

2.1. 3D FE MODEL

Figure 1 shows the FE model including both the lay-down structure and the adjacent hull structure.

The limits of the local 3D FE model are as follows:

- Fr.73 to Fr.77 on x direction;
- SB-PS on y-direction;

The model was built based on existing hull drawings. The 3D FE model is sufficiently large to ensure that the FE analysis results are not significantly affected by the assumptions made for boundary conditions and loads.

The mesh size and the types of elements have also been used to model the lay down structure and its hull supporting structures.

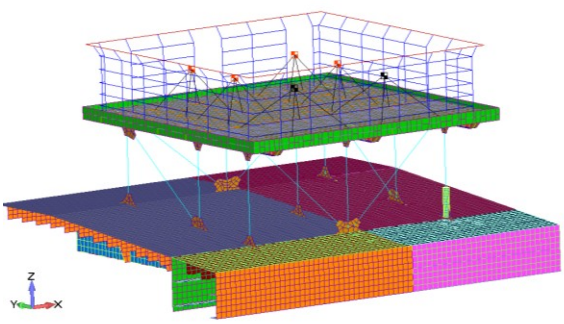


Fig. 1. *Full 3D-FE model*

2.2. LOADS

Mechanical behaviour of a structure is depending on thermal actions and their thermal effect on material properties and indirect mechanical actions, as well as on the direct effect of mechanical actions.

2.2.1. Weight and Inertia Loads

The inertia loads acting on the corresponding top side structure and hull supporting structure, on X, Y and Z direction, are taken into account by using the longitudinal (a_x), transversal (a_y), and vertical accelerations (a_z).

For transit conditions, the accelerations have been adopted according to Ref. [7], Ch.5.7.2 and have been modelled as body accelerations (see Table 1).

Supplementary, on Z direction, the gravitational acceleration, g=9.81 [m/s^2] has been added in order to take into consideration the steel weight.

Table 1. Long term accelerations for 20 years return period in transit conditions

a_x[m/s^2]	a_y[m/s^2]	a_z[m/s^2]
1.802	5.747	4.612

A uniform distributed pressure, p_{dk}, of 5 kN/m^2 has been adopted on the upper deck in order to take into consideration the mass of the equipment placed on the deck.

2.2.2. Wind Loads

The wind loads are considered according to Ref. [1], Sec.4, C300 and modelled as normal pressure for plate elements and distributed load for bar elements. The wind loads are variable on height.

The mean wind speed over an averaging period T at a height z above sea level, $U(T,z)$[m/s], is determined according to Ref. [5], Ch.2.3.2.12.

$$U(T,z) = U_0 \cdot \left\{1 + C \cdot \ln\frac{z}{H}\right\} \cdot \left\{1 - 0.41 \cdot I_U(z) \cdot \ln\frac{T}{T_0}\right\} \quad (1)$$

where:

$$C = 5.73 \cdot 10^{-2}\sqrt{1 + 0.148U_0} \quad (2)$$

and

$$I_U = 0.06 \cdot (1 + 0.043U_0) \cdot \left(\frac{z}{H}\right)^{-0.22} \quad (3)$$

- U_0 = 37.3 [m/s] - the one hour mean wind speed at 10 m above sea level with a return period of 100 years;
- T_0=1 hour - one-hour wind reference period;
- T = 1 minute for the wind action (Ref. [1], Sec. 4, C 301).

The wind pressure applied on the plate elements of the structure has been calculated using the formula:

$$p = C_p \cdot \frac{\rho_a \cdot U(z,T)^2}{2} \cdot 10^3 [N/mm^2] \quad (4)$$

where,

- C_p = pressure coefficient; C_p = 1.0;

The input data for the calculation of $U(T,z)$ are:

- ρ_a [kg/mm^3] = mass density of air at -7 C°; ρ_a = 1.332.10^{-9} [kg/mm^3].

The wind action has been modelled as a distributed wind load along the length of the bar members. The values of the distributed loads have been calculated using the formula:

$$q = C_s \cdot \frac{\rho_a \cdot U(z,T)^2}{2} \cdot D \cdot 10^3 [N/mm] \quad (5)$$

where,

- C_s = shape coefficient; C_s = 1.2 for the tubular structures covered with ice and C_s = 2.1 for H profile;
- D [mm] = diameter of bar.

The angle between the direction of the wind and the axis of the exposed member or surface, α, is conservatively considered to be equal with 90°.

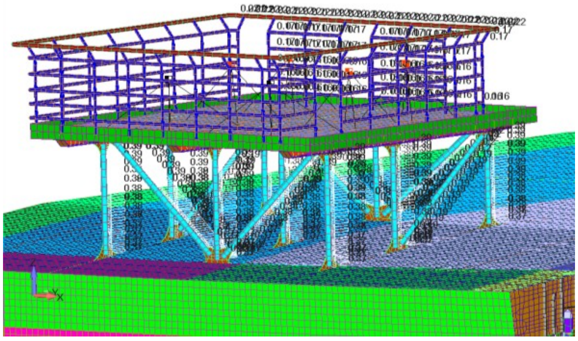


Fig. 2. *Wind forces applied on the structure*

2.2.3. External Sea Pressure

According to Ref. [1], Sec.5, A105 the external sea pressure is to be considered on the hull structure.

For LC-S, no external pressure has been considered (the sea level is below the lower limit of the FE model). For LC-H, the external sea pressure on side has been determined, based on the values given for the long term sea pressure.

The maximum sea pressure for Fr.54 is 159.093 kN/m^2 at z=13.435 m ABL; the sea pressure decreases linearly above 13.435 m. The following law is applied to determine the sea pressure on sea side of the FE model (see Fig. 3):

$$p_{zmax} = p_{zmin} - \rho_{water} g(z_{max} - z_{min}) \quad (6)$$

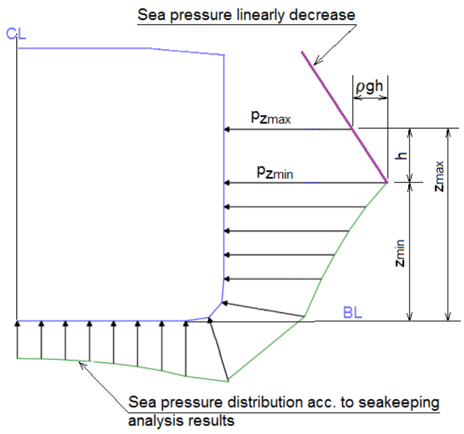


Fig. 3. *Sea pressure distribution on side shell*

The lower limit of the FE model is 16.5m (greater than 13.435) therefore the sea pressure on the model sides has been determined using the above law (for the linearly decreasing sea pressure).

The minimum and maximum values of the external sea pressure obtained for the minimum and maximum height of the FE model at side, are presented in Table 2.

Table 2. External sea pressure on side

	Sea pressure [kN/m^2]	
	z=16.5 m ABL	z=22.0 m ABL
LC-H	128.274	72.970

The external sea pressure acting on the weather deck, determined for the considered loading condition, is calculated according to Ref. [2], Pt.3, Ch.1, Sec.8, B100 using DNV Nauticus Hull, software. The external sea pressure on deck decreases linearly from side, at B/2, to B/4 being constant from B/4 to the CL (see Fig. 4).

The external sea pressure at y=B/2 (in side) and y=B/4 are presented in Table 3 and is modelled as normal pressure for the plate elements.

Table 3. External sea pressure on weather deck

	Sea pressure [kN/m^2]	
	$p_{y=0} = p_{y=B/4}$	$p_{y=B/2}$
LC-H	6.24	16.0

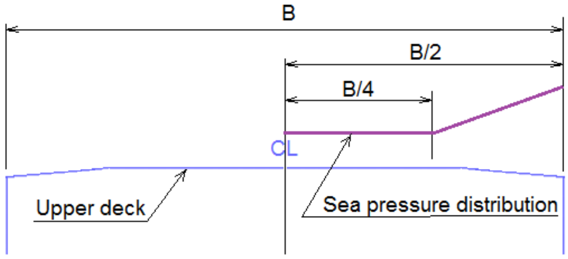


Fig. 4. *Sea pressure distribution on upper deck*

2.2.4. Loads on Lay-Down Platform

The top side platform should be designed for a distributed load, set equal with 15 [kN/m^2] and modelled as normal pressure for the plate elements.

In order to consider the wind and inertia loads on containers, the most unfavourable containers position has been selected (see Fig. 5).

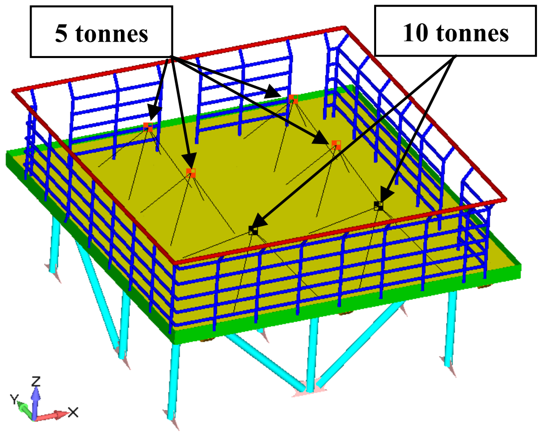


Fig. 5. *Positions of containers*

The inertia loads of the containers, on X, Y and Z direction, are taken into account by using mass elements defined in the centre of gravity of each container and the longitudinal (a_x), transversal (a_y), and vertical accelerations (a_z).

2.3. Load Combinations

The lay down structure shall be analysed based on the load combinations presented in Table 4.

Table 4. The load combination factors

Load	LC1	LC2	LC3	LC4
a_x	1	1	-1	-1
a_y	1	-	1	-1
a_z	-1	-1	-1	-1
g	-1	-1	-1	-1
Wind loads	+X,+Y	+X,-Y	-X,+Y	-X,-Y
q	1	1	1	1
p_{dk}	1	1	1	1
Temperature	1	1	1	1

2.4. Results of the strength analysis

The allowable stress was obtained by reporting the yielding stress to the safety coefficient. The safety factor, c = 0.8, was calculated according to the DNV naval register.

$$\sigma_{allowable} = c \cdot \sigma_{yielding} = 0.8 \cdot 355 = 284\,N/mm^2 \qquad (6)$$

In order to remain in the elastic domain, the effective stress values will need to be lower than the allowable stress. Having two types of elements (plates and bars) in the FEM model, the results are tabulated, depending on the type of element being analyzed. Table 5 presents the results of the plate elements and table 6 the results of the bar element.

Table 5. Plate elements results

Load case	σ_{VM} [MPa]	σ_a [MPa]	Load case	σ_{VM} [MPa]	σ_a [MPa]
LC-H1	91		LC-S1	82	
LC-H2	90		LC-S2	80	
LC-H3	91	284	LC-S3	83	284
LC-H4	144		LC-S4	78	
LC-H5	**203**		LC-S5	85	

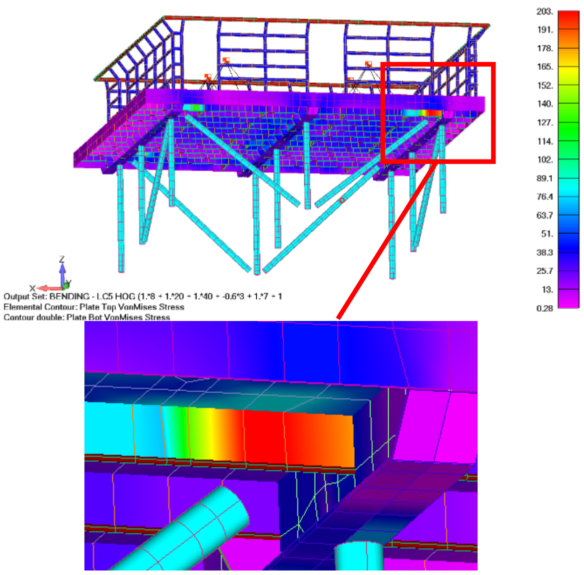


Fig. 6. *Plate elements VM stress*

Table 6. Bar elements results

Load case	$\sigma_{comb.}$ [MPa]	σ_a [MPa]	Load case	σ_{comb} [MPa]	σ_a [MPa]
LC-H1	132		LC-S1	110	
LC-H2	141		LC-S2	112	
LC-H3	126	284	LC-S3	101	284
LC-H4	238		LC-S4	121	
LC-H5	**287**		LC-S5	109	

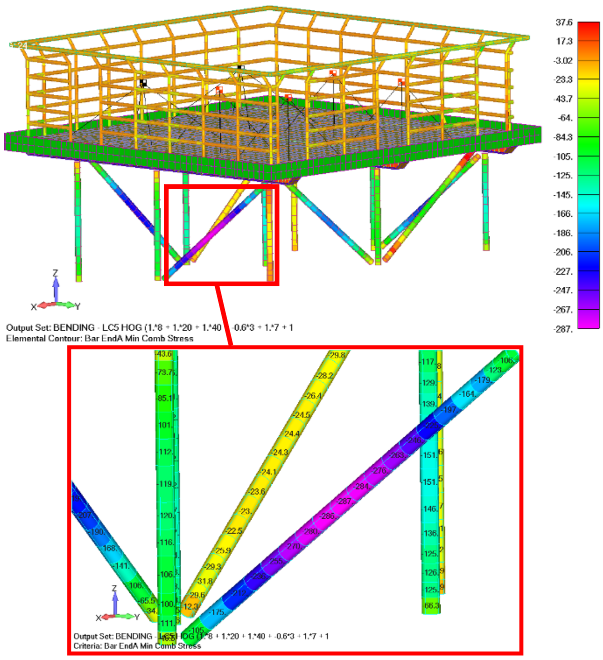


Fig. 7. *Bar elements max.comb. stress*

Because the effective stress value of the platform pillar exceeds the value of the allowable stress, it was decided to replace the respective profile Φ168.3x10 with Φ219.1x12.7. As a result of this change, the calculation was recalled and the new values are presented in Table 7.

Table 7. Bar elements results

Load case	$\sigma_{comb.}$ [MPa]	σ_a [MPa]	Load case	σ_{comb} [MPa]	σ_a [MPa]
LC-H1	100		LC-S1	84	
LC-H2	107		LC-S2	81	
LC-H3	97	284	LC-S3	85	284
LC-H4	154		LC-S4	87	
LC-H5	**162**		LC-S5	98	

3. THERMO-MECHANICAL ANALYSIS

3.1. Heat transfer analysis

The variations of the physical and mechanical properties of the temperature according to the European standard EN 1993-1-2 were considered, these variations being presented in the following figures.

3.1.1. Thermal elongation

-for 20°C ≤ T <750°C

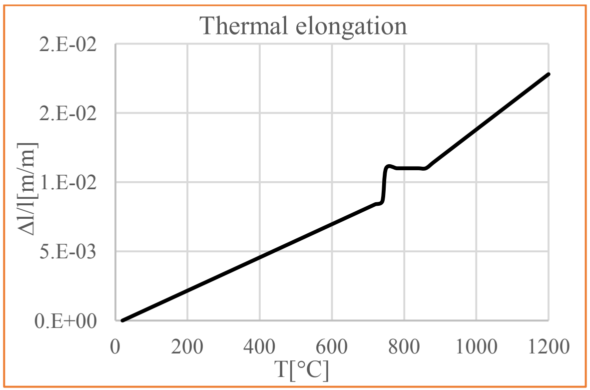
$$\frac{\Delta l}{l} = 1.2 \cdot 10^{-5} \cdot T + 0.4 \cdot 10^{-8} \cdot T^2 - 2.416 \cdot 10^{-4}$$

-for 750°C ≤ T < 860°C (7)

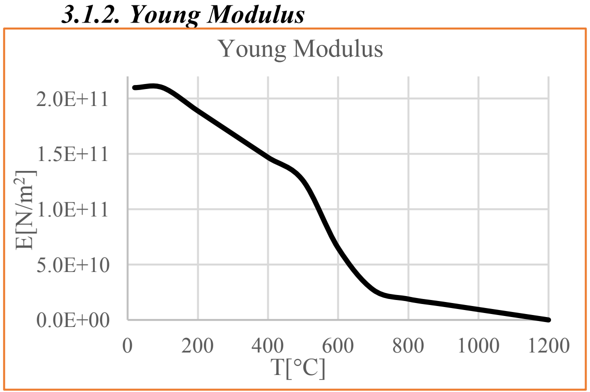
$$\frac{\Delta l}{l} = 1.1 \cdot 10^{-2}$$

-for 860°C ≤ T <1200°C

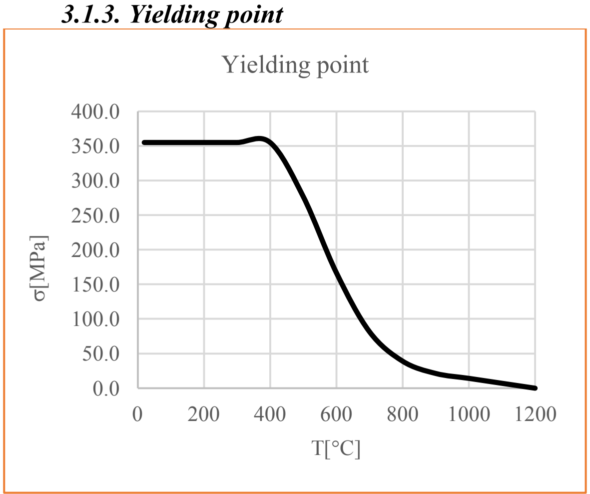
$$\frac{\Delta l}{l} = 2 \cdot 10^{-5} \cdot T - 6.2 \cdot 10^{-3}$$



3.1.2. Young Modulus



3.1.3. Yielding point



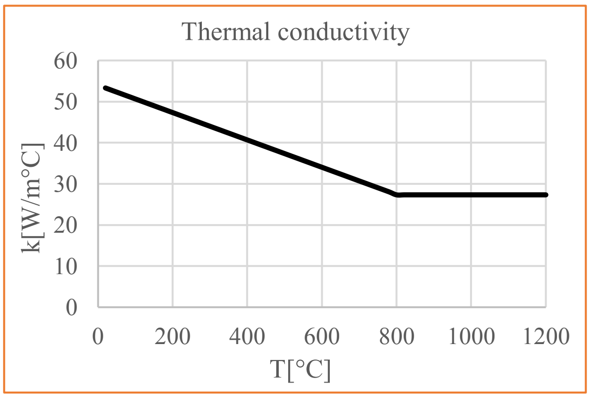
3.1.4. Thermal conductivity

-for 20°C ≤ T < 800°C

$$k = 54 - 3.33 \cdot 10^{-2} \cdot T \quad (8)$$

-for 800°C ≤ T < 1200°C

k=27.3



3.1.5. Specific heat

-for 20°C ≤ T < 600°C

$$c_s = 425 + 7.73 \cdot 10^{-1} \cdot T - 1.69 \cdot 10^{-3} \cdot T^2 + 2.22 \cdot 10^{-6} \cdot T^3$$

-for 600°C ≤ T < 735°C

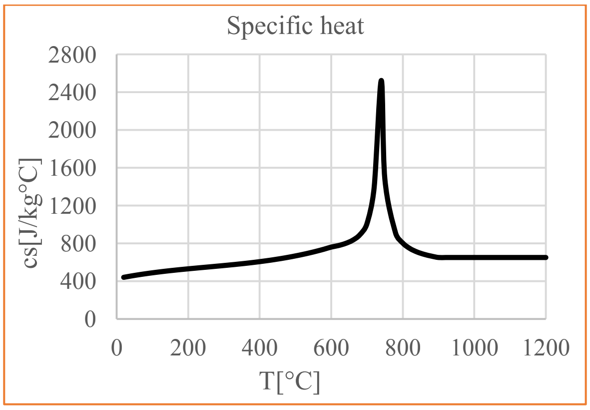
$$c_s = 666 + \frac{13002}{738 - T} \quad (9)$$

-for 735°C ≤ T <900°C

$$c_s = 545 + \frac{17820}{T - 731}$$

-for 900°C ≤ T <1200°C

$$c_s = 650$$



Thermo-mechanical analysis has two phases:

-the model loaded with thermal load;

-the model loaded with physical loads (shown in Ch.2.4) plus the loads results from thermal analysis.

3.1.6. Thermal Loads

With a nominal temperature-time curve, the temperature analysis of the structural members is made for a specified period of time, without any cooling phase.

The heat flux of 150 kW/m^2 is applied on a leg of the laydown structure for 5 min.

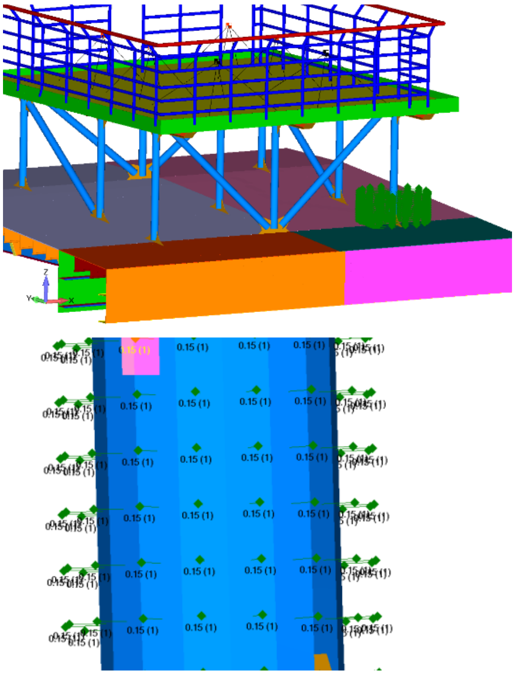


Fig. 8. *Heat flux on the pillar*

The coefficient of heat transfer by convection should be taken as $\alpha_c = 35$ [W/m^{2}C] and the emissivity coefficient $\varepsilon_m = 0.8$ acc. to [2].

The maximum temperature in pillar, exposed to heat flux of 150 kW/m^2 for 5 min, is 720°C (see Fig. 9).

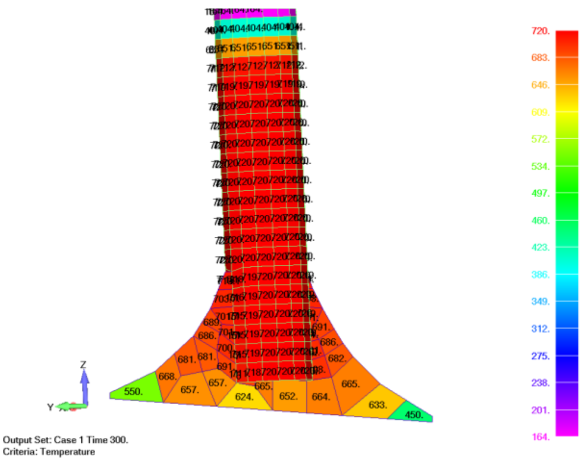


Fig. 9. *The maximum temperature in pillar*

This temperature value has been used as input data for the strength analysis shown in the next chapter.

3.2. Yielding Check

The yielding stress results are presented just for the part of the structure exposed to fire.

According to [6], Sec. 7, B, load-bearing structures shall maintain integrity for the required period of time when exposed to the defined dimensioning accidental loads as defined in [7].

Normally the critical temperatures with respect to structural integrity are as given in [6], Sec. 7, B, Table B1.

Table B1 Critical temperatures	
Material	*Temperature*
Structural steel and ordinary reinforcing steel	400 to 450°C
Pre-stressed reinforcing steel	350°C
Aluminium dependent on type of alloy	200°C

Other critical values may be used as long as corresponding changes are taken into account concerning the thermal and mechanical properties.

Table 8. Reduction factors for stress-strain relationship of carbon steel temperatures

Steel Temperature θ_a	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20°C		
	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
20°C	1,000	1,000	1,000
100°C	1,000	1,000	1,000
200°C	1,000	0,807	0,900
300°C	1,000	0,613	0,800
400°C	1,000	0,420	0,700
500°C	0,780	0,360	0,600
600°C	0,470	0,180	0,310
700°C	0,230	0,075	0,130
800°C	0,110	0,050	0,090
900°C	0,060	0,0375	0,0675
1000°C	0,040	0,0250	0,0450
1100°C	0,020	0,0125	0,0225
1200°C	0,000	0,0000	0,0000
NOTE: For intermediate values of the steel temperature, linear interpolation may be used.			

The material characteristics have been modified, for this analysis, according to the temperature value obtained from the thermic analysis using the following formulas:

$E_{a,\theta} = E_a \cdot k_{E,\theta}$ $\quad f_{y,\theta} = f_y \cdot k_{y,\theta}$ (see Table 8)

The Young module, E, is:

- 700°C - $E_{a,700} = 210000 \cdot 0.13 = 27300$ MPa
- 800°C - $E_{a,800} = 210000 \cdot 0.09 = 18900$ MPa

For 720°C - $E_{a,720} =$ **25620** MPa

The stress level in the lay down pillar exposed to the fire action must be compared with temperature dependent allowable stress (see Table 8).

- 700°C - $f_{y,700} = 355 \cdot 0.23 = 81.65$ MPa
- 800°C - $f_{y,800} = 355 \cdot 0.11 = 39.05$ MPa

For 720°C - $f_{y,720} =$ **73** MPa

Table 9. Max. yielding stress results

Lay down area structure NV-36 exposed to fire		
Load Cases	f_{VM} [N/mm^2]	$f_{y,720}$ [N/mm^2]
LC1	130	73
LC2	144 (Figure 10)	
LC3	114	
LC4	131	

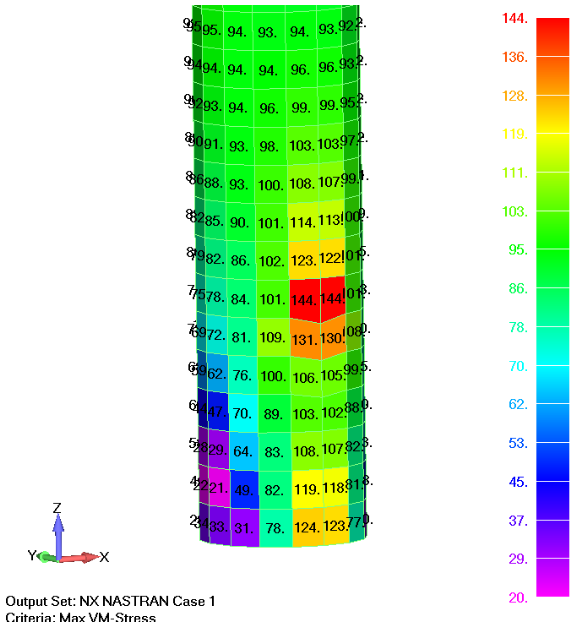


Fig. 10. *The maximum yielding stress in the pillar exposed to fire*

3.3. Buckling Check

The Nauticus Hull spreadsheet "Buckling of Bars and Beams" has been used for the buckling verification of the axially compressed legs of the lay down supporting structure.

The axial forces (N_A) and bending moments (M_Y and M_Z) have been determined using FE model (see Table 10).

Table 10. Input data for buckling check

	N_A [N]	M_Y [Nmm]	Effective length factor of M_Y	M_Z [Nmm]	Effective length factor of M_Z	Bar length [m]
Legs exposed to fire	-309495	4084403	1.0	942637	1.0	3.1

The maximum allowable usage factor was adopted 0.8 because the axial forces and bending moments have been determined for combined static and dynamic load cases.

Based on the results presented in Table 11, the maximum usage factor for the axial compression is 0.844, higher than the allowable value (0.8), therefore it has been concluded that the pillar exposed to fire must be replaced with another profile because not verify buckling criteria.

Table 11. Buckling check – Summary

Profile Id: / Dimensions:	L / σ_F	E / ν	Load N_A	Moments M_y / M_z	eff.b.leng. Strong / Weak	Curve Strong / Weak	Crit.stress σ_{acr} / σ_{bcr}	Usage fact. ηa / η_{tot}
Legs exposed to fire	3.1	26		4.08	1.000	a	61.6	0.610
219.1 / 12.7	73	0.300	309.5	0.94	1.000	a	73	0.844

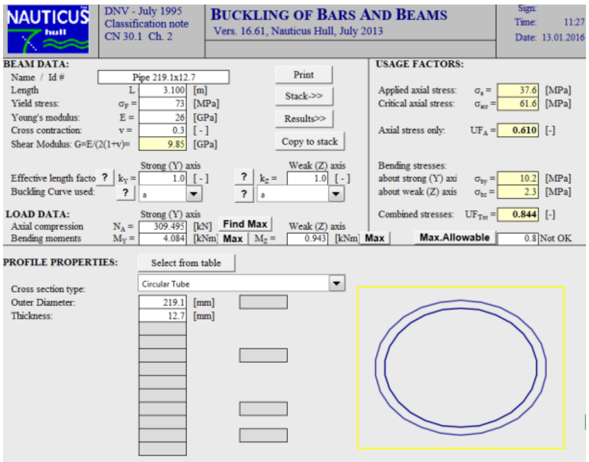


Fig. 11. *Buckling check – Input data and results*

5. CONCLUSIONS

The heat flux for this analysis 150kW/m^2 for 5 min is hypothetical, the heat flux value and the acting time is in general greater than these values which will lead to higher values of the temperature. In this case the strength of the structure is compromised and no technical solution cannot be applied to make the structure to resist.

The only way is to use fireproof solutions.

REFERENCES

[1] EN 1993-1-2: Eurocode 3: *Design of steel structures – Part 1-2: Structure fire design*, 2005;
[2] EN 1991-1-2: Eurocode 1: *Actions on structures – Part 1-2: Actions on structures exposed to fire*, 2002;
[3] DNV-OS-B101 *Metallic Materials*, October 2012;
[4] DNV-RP- C205 *Environmental Conditions and Environmental Loads*, October 2010;
[5] DNV-OS- C102 *Structural Design of Offshore Ships*, October 2011;
[6] DNV-OS- D301 *Fire Protection*, October 2008;
[7] DNV-OS- A101 *Safety Principles and Arrangements*, April 2011