TALAT Training in Aluminium Application Technologies

TALAT Lectures 2711

Design of A Helicopter Deck

11 pages, 7 figures

Advanced Level

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Update from the TAS Project :

TAS

Leonardo da Vinci program
Training in Aluminium Alloy Structural Design

Remark

The design of the main structural parts of an aluminium alloy helicopter deck is shown in this document. The design of a bolted connection on the supporting structure is also presented. The calculations carried out hereafter are based on the European Standard prENV 1999 (version April 1997).

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2711 Design of A Helicopter deck

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1. INTRODUCTION

The structure under consideration consists of a helideck to be built on an off-shore platform (fig. 1). Its main bearing structure is made of three I-girders with welded built-up cross section (figs 2 and 3). The top deck is made with an extruded profile whose design has been performed in such a way to give place to a continuous deck without welding (fig. 4). The span of the main girders, assumed simply supported, is 8000mm. The upper deck may be considered as an orthotropic plate (having two 3000mm long spans) simply supported to the girders.

2. MATERIALS

The structure is entirely made of aluminium alloy members. The alloys used are listed in tab. 1.

	Denomination	$f_{0.2}$ (N/mm^2)	f_t (N/mm^2)	Min. elong. (%)
Extrusions of support structure	EN AW-6082 T6	255	300	8
Web plate of support structure	EN AW-5083 H24	240	340	4
Deck extrusion	EN AW-6005 T6	215	255	8

Tab. 1 – Mechanical properties of alloys employed in the structure.

The design values of the basic properties assumed for all alloys are shown in Tab. 2. The partial safety factor for material is assumed equal to γ_M = 1.10 for members and γ_M = 1.25 for bolted connections.

Modulus of elasticity	E = 70 000 (N/mm^2)
Shear modulus	G = 27 000 (N/mm^2)
Poisson's ratio	ν = 0.3
Coefficient of linear thermal expansion	$\alpha = 23 \times 10^{-6}$ per °C
Density	ρ = 2 700 kg/m^3

Tab. 2 – Design values of material coefficients.

3. LOADS

3.1. Permanent loads

Nominal values of permanent loads consist of structure selfweight. They are summarised in tab. 3.

Support structure	34.88 kg/m (H = 450 mm)
Deck extrusion	10.55 kg/m

Tab. 3 – Selfweight of structural members.

3.2. Live loads

The live load is represented by the helicopter landing load, referred to in tab. 4. Distinction is made between values for ultimate limit state and serviceability limit state, Elastic analysis is performed for checking both limit states.

	Ultimate limit state	Serviceability limit state
Deck extrusion	F_{deck} = 116kN, distr. on 300 × 300 mm^2	F_{deck} = 70kN, distr. on 300 × 300 mm^2
Support structure	F_{supp} = 151 kN	F_{supp} = 91 kN

Tab. 4 – Live loads on deck extrusion and support structure.

3.3 Load combination

Rules provided in EC 1 have been followed for evaluating the partial safety factors γ_G and γ_Q. The resulting values are shown in tab. 5. The value of γ_Q has been increased from 1.5 to 1.65 in order to take into account a possible overload in emergency conditions.

	Ultimate limit state		Serviceability limit state	
	γ_G	γ_Q	γ_G	γ_Q
Support structure	1.35	1.65	1.00	1.00
Deck extrusion	1.00	1.65	1.00	1.00

Tab. 5 – List of the partial safety factors for applied loads.

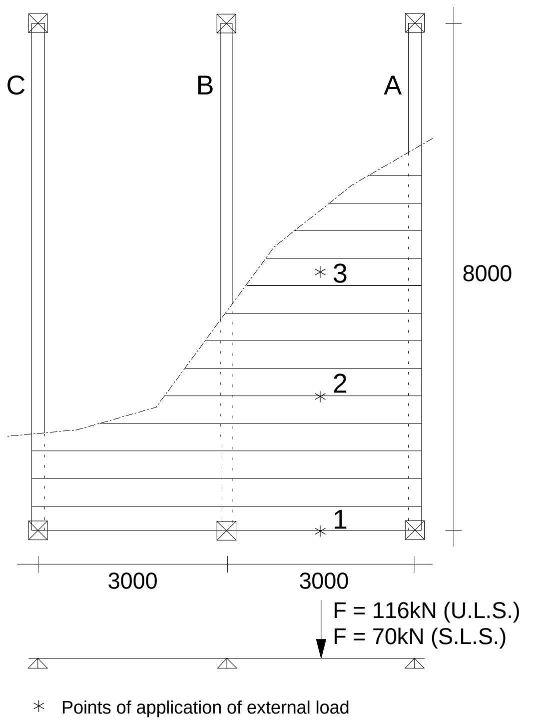

Fig. 1 – Structural scheme of the helideck.

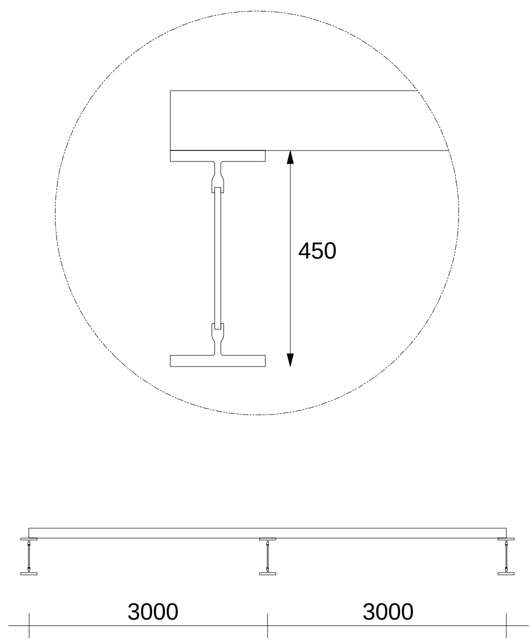

Fig. 2 – Detail of the girder-to-deck assembly.

4. ANALYSIS OF THE DECK EXTRUSION

The elastic analysis of the deck structure has been carried out by means of the F.E.M. code ABAQUS. Specific allowance has been made for the effect of plate orthotropy due to the particular arrangement of the extruded profiles. In order to investigate the structural behaviour for different load conditions, the concentrated force shown in tab. 4 has been considered acting on the deck in

three different points, as shown in fig. 1. Because of its very small value, the deck selfweight has been neglected in this analysis.
For the above load conditions, the results in terms of transverse displacements and internal actions are graphically shown in figs 1-6 of the Appendix for both serviceability and ultimate limit state. The relevant numerical results are also provided in tabs 1-6 of the Appendix.
In order to check the ultimate limit state the cross section has to be classified. According to EC9 Part 1-1, Chapter 5.4.3, the slenderness parameter β must be evaluated. In the case of bending β = 0.40b/t for the extrusion webs, where the stress gradient results in a neutral axis at the section center, and β = b/t for the compressed flange, b and t being the width and the thickness of the section element, respectively.

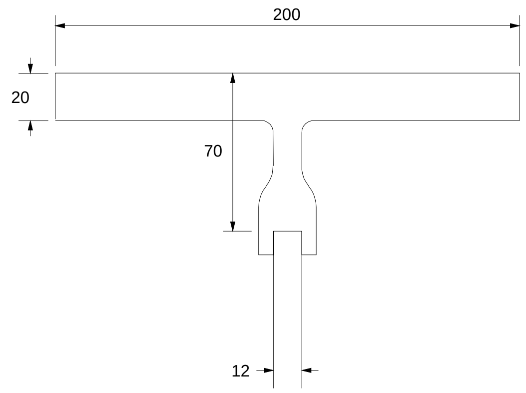

Fig. 3 – Detail of the girder extrusion

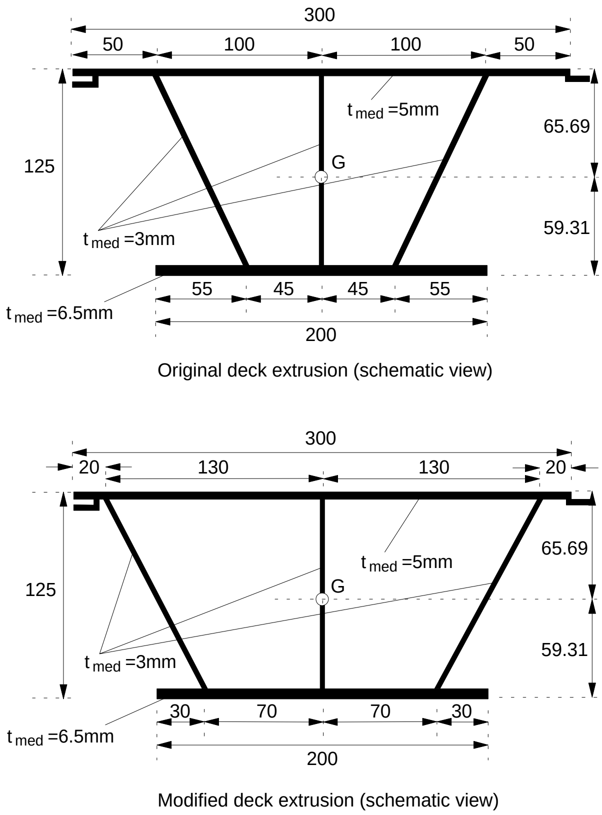

Fig. 4 – Deck extrusion (original and modified)

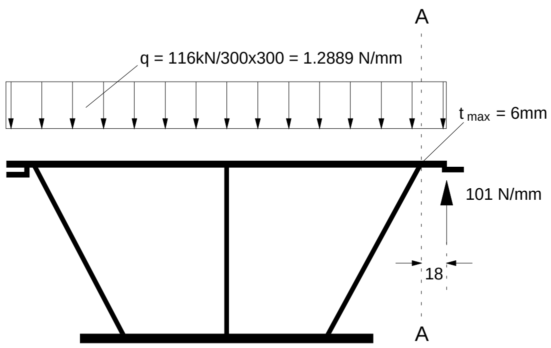

Fig. 5 – Calculation model for deck cantilever outstands.

For the assumed cross sections the values evaluated for β are summarised in tab. 6. Since the values of the slenderness parameter β fall into the Class 1, 2 and 4 regions (see tab. 5.1 of EC9 Part 1-1, $\varepsilon = \sqrt{250 / f_{0.2}} = 1.0783$), a specific allowance for the effect of local buckling should be made. In addition, the value of the shear forces evaluated by means of the F.E.M. analysis would be too much high for the cantilever outstand of the original section (fig. 4). For this reason, it is convenient to modify the original extruded profile according to fig. 4, where the relevant modifications of the section are schematically indicated. Such modifications are intended in order to keep the same value of the inertia moment of the original section (I_{xc} = 10 918 999 mm^2). In this way, all the section elements have a slenderness parameter β falling into Classes 1, 2 and 3, as shown in tab. 7. As a consequence, the cross section is allowed for a value of the maximum bending moment corresponding to the elastic limit moment $M_e = f_{0.2}W/\gamma_M$.

Webs	Upper flange	Lower flange internal	Lower flange outstand
$14.82 < \beta_2 = 17.25$ Class 2	$16 < \beta_2 = 17.25$ Class 2	$5.38 < \beta_1 = 11.86$ Class 1	$7.69 > \beta_3 = 6.47$ Class 4

Tab. 6. – Values of the slenderness parameter β for the original deck extrusion.

Webs	Upper flange	Lower flange internal	Lower flange outstand
$14.82 < \beta_2 = 17.25$ Class 2	$22 < \beta_3 = 23.72$ Class 3	$9.23 < \beta_1 = 11.86$ Class 1	$3.85 < \beta_2 = 4.85$ Class 2

Tab. 7. – Values of the slenderness parameter β for the modified deck extrusion.

Referring to fig. 4, for a value of the inertia moment I_{xc} = 10 918 999 mm^4, one can obtain the value of $W_{min} = I_{xc} / C_y$ = 10918999 mm^4/ 65.69 = 166220 mm^3. Hence:

$$M_e = f_{0.2}W/\gamma_M = 215\times166\,220/1.10 = 32\,488 \text{ Nm}$$

This value is higher than the maximum bending moment evaluated in the numerical analysis for ultimate limit state (M = 22 548 Nm, load condition 1, F = 116 kN, see tab. 1 in the Appendix).
The effect to shear forces can be evaluated approximately by means of the scheme shown in fig. 5, where the calculation model assumed for the evaluation of the bending moment in the section A-A is represented. This model is intended in order to provide an estimation of the maximum bending action induced on the deck extrusion by a concentrated load applied on an area of 300 × 300 mm^2. Under the assumption of fig. 5, the design bending moment per unit length in the section A – A is given by:

$$M_d = T \times l - q \times d^2 /2 = 101 \times 18 - 1.2889 \times 18^2 /2 = 1818 - 219 = 1\,609 \text{ Nmm/mm}$$

Where (see fig. 5):

T = 101 N/mm, is the maximum value of the design shear force
(load condition 1, F = 116 kN, see tab. 1 in the Appendix);
d = 18 mm, is the distance of the section A – A from the point of application of shear force;
q = 1.2889 N/mm^2, is the value of the distributed load on a deck area of 300 × 300 mm^2.

The ultimate bending moment per unit length is equal to:

$$M_u = \alpha_0 \times W \times f_{0.2} /\gamma_M = 1.5 \times 6 \times 215 / 1.1 = 1\,759 \text{ Nmm/mm}$$

Where a geometrical shape factor α_0 = 1.5 (rectangular section) has been considered. A section depth t = 6mm has been assumed and the resistance modulus W has been evaluated accordingly. From the above equations it results $M_d < M_u$.
The analysis under serviceability conditions (P_{deck} = 70kN) has provided values of the maximum deflection u_{max} for the deck ranging from 8.41 to 15.39, corresponding to L/357 and L/195, respectively (see Figs 4-6 and Tabs 4-6 in the Appendix). These deflection values have been calculated from the numerical results by subtracting to the displacements of the deck the displacements of the girder on the corresponding alignment.

5. ANALYSIS OF THE SUPPORT STRUCTURE

The support structure consists of a welded built-up girder with I-section. The dimensions of the extruded flanges, as well as the web thickness are given in fig. 3; the web depth has to be determined according to the design limit state requirements. The solutions considered, different from each other for the girder depth value, are shown in tab. 8, where the relevant geometrical and mechanical properties are also shown for each solution. Even though built-up with two different alloys, for the sake of simplicity the girder plastic modulus Z has been evaluated by assuming the cross section as made of one alloy only, that of the flanges. This has been possible since both alloys have a similar value of the proof stress $f_{0.2}$ (255 and 240 N/mm^2) and, in addition, the contribution of the web to the ultimate resistance is very small.

Girder depth (mm)	I (mm^4)	W (mm^3)	Z (mm^3)	α_0
400	335 722 667	1 678 613	1 908 800	1.1371
450	438 987 667	1 951 056	2 224 300	1.1400
500	558 402 667	2 233 610	2 554 800	1.1438

Tab. 8 – Main geometrical properties of the examined solutions.

The most unconservative load condition is obtained when the helicopter landing load is applied in a concentrated way at the beam midspan. The design value of the bending moment at beam midspan is given by:

$$M_d = \frac{\gamma_G q L^2}{8} + \frac{\gamma_Q F L}{4} = \frac{1.35 \times (34.88 + 10.55) \times 9.81 \times 8^2}{8} + \frac{1.65 \times 91000 \times 8}{4} = 306811 Nm$$

where γ_G = 1.35 and γ_Q = 1.65 are the partial safety factor for dead load and for live load, respectively. The selfweight of the intermediate solution (H = 450mm, P = 342 N/m, see tab. 8) has been considered, the differences with respect the other depth values being negligible. The deck selfweight (10.55 N/m) has been also considered.
For the assumed cross sections the values evaluated for β are summarised in tab. 9. In all cases the slenderness parameter β falls into the Class 1 or 2 region (see tab. 5.1 of EC9 Part 1-1, $\varepsilon = \sqrt{250 / f_{0.2}} \cong 1$). As a consequence the cross section is allowed to withstand an ultimate bending moment equal to the plastic moment $M_p = f_{0.2}Z/\gamma_M = \alpha_0 \, f_{0.2} \, W/ \gamma_M$. The geometrical shape factor α_0 may be evaluated through the relationship:

$\alpha_0 = Z/W$

Z being the plastic resistance modulus of the cross section (see tab. 8).

Girder depth (mm)	Web	Flanges
400	$8.67 < \beta_1 = 11$ (Class 1)	$4.5 = \beta_2$ (Class 2)
450	$10.33 < \beta_1 = 11$ (Class 1)	$4.5 = \beta_2$ (Class 2)
500	$12.00 < \beta_2 = 16$ (Class 2)	$4.5 = \beta_2$ (Class 2)

Tab. 9 – Values of the slenderness parameter β for the support structure.

Under the above assumption, the ultimate bending moment M_u is equal to:

$M_u = M_p = f_{0.2}Z/\gamma_m = \alpha_0 \, f_{0.2} \, W/ \gamma_m$

in which the $f_{0.2}$ value of the flanges has been considered $f_{0.2}$ = 255 N/mm^2. The values of the ultimate bending moment M_u for the selected solutions are given in tab. 10, together with the ratio M_u/M_d, meaning the extra value of the safety coefficient at failure.

Girder depth (mm)	M_u (Nm)	M_u/M_d
400	439 578	1.4327
450	512 226	1.6695
500	588 362	1.9177

Tab. 10 – Values of M_u and M_u/M_d for the support structure.

Since $M_u/M_d > 1$ for all solutions, all the analysed schemes are safe with regard to the structural plastic collapse. The choice of the design solution is made on the basis of the serviceability requirements. For this purpose, the serviceability limit states for the support structure are investigated, namely the maximum deflection and the maximum stress values under the service loads. A design value of the concentrated load equal to 91 kN is assumed. The maximum midspan deflection is given by:

$$v_{\max} = \frac{5}{384}\frac{\gamma_G q l^4}{EI} + \frac{1}{48}\frac{\gamma_Q F l^3}{EI}$$

where the partial safety value γ_G and γ_Q have been put equal to unity (see tab. 5). The values of v_{max} are provided in tab. 11, together with the values of σ_{max} and τ_{max} evaluated via the expressions:

$$\sigma_{\max} = \frac{M}{W};$$

$$\tau_{\max} = \frac{T}{b_w \times t_w};$$

The design values M and T of the maximum bending moment and shear are evaluated as follows:

$$M = \frac{qL^2}{8} + \frac{FL}{4} = 184737382 Nmm$$

$$T = \frac{qL}{2} + \frac{F}{2} = 46869 N$$

Girder depth (mm)	v_{max} (mm)	v_{max}/l	σ_{max} (N/mm^2)	τ_{max} (N/mm^2)
400	42.0805	1/190.1	110.05	10.85
450	32.1817	1/248.6	94.69	9.53
500	25.2996	1/316.2	82.71	8.49

Tab. 11 – Values of the relevant displacement and stresses for the support structure at serviceability limit state.

6. GIRDER CONNECTION

A connection between two semi-girders is designed, consisting of a lap joint with gusset plates, as shown in fig. 5. Since it is placed at the girder midspan, the connection is conceived in order to withstand the maximum design bending moment (M_d = 305 695 Nm) with zero shear.
With reference to fig. 6, the moment of inertia of the gusset plates evaluated by accounting for holes (∅21) is given by:

$$I = 2\times(200–42)\times14\times(225+7)^2 + 4\times(85–21)\times14\times(225-27)^2 + 2\times8\times280^3/12 - 4\times21\times8\times100^2 =$$

$$= 401\,173\,845 \text{ mm}^4$$

The maximum tensile stress at the outside gusset plate is equal to:

$$\sigma_{max} = M/I \times H/2 = 305\,695\,000/401\,173\,845 \times 478/2 = 182.12 \text{ N/mm}^2$$

By assuming for the gusset plate the same alloy as for the flange extrusion ($f_{0.2}$ = 255 N/mm^2, f_u = 300 N/mm^2), it results:

$$\sigma_{max} = 182.12 \text{ N/mm}^2 < f_{0.2}/\gamma_M = 255/1.25 = 204 \text{ N/mm}^2$$

Similarly, the moment of inertia of the girder net section is given by:

$$I_{net} = 438\,987\,667 - 4\times 21 \times 14 \times 215^2 – 2 \times 21 \times 12 \times 100^2 = 379\,587\,067 \text{ mm}^4$$

Hence, the resistance modulus:

$$W_{net} = I_{net} / (H/2) = 379\,587\,067 / 225 = 1\,687\,054 \text{ mm}^3$$

The ultimate bending moment is equal to:

$$M_u = M_p = \alpha_0\, f_{0.2}\, W/ \gamma_m = 1.14 \times 255 \times 1\,687\,054 / 1.25 = 392\,341 \text{ Nm} > M_d = 306\,811 \text{ Nm}$$

The maximum tensile action transmitted by the extrusion flange may be conservatively calculated by assuming that the bending moment acting on the joint is carried entirely by the flanges. Under this assumption it follows that:

$$T_{tot} = M/d' = 305\,695\,000/430 = 710\,919 \text{ N}$$

d' = 430mm being the distance between the flange centroids (fig. 6). By adopting M20 bolts, the maximum shear stress in the flange bolts is evaluated as follows:

$$\tau_{max} = T_{tot} / (2\times n\times A_b) = 710\,919/(2\times6\times245) = 241.8 \text{ N/mm}^2$$

n and A_b being the number of bolts and the bolt net cross sectional area, respectively. By considering a material partial factor γ_M = 1.25, the minimum shear ultimate resistance of the bolts must be:

$$\tau_u = \tau_{max} \times 1.25 = 241.8 \times 1.25 = 302.2 \text{ N/mm}^2$$

A suitable bolt type is represented by Class 8.8 zinc coated steel bolts, having $f_{d,v}$ = 385 N/mm^2.
The bearing resistance of the flange is evaluated through the formula:

$$\sigma_{b,Rd} = 2.5\ \alpha\ f_u / \gamma_M = 2.5 \times .794 \times 300 / 1.25 = 476.2 \text{ N}$$

The factor α = .794 has been calculated according to Chapter 6, Table 6.4 of EC 9.
The maximum stress in the flange is equal to:

$$\sigma_{max} = T_{tot} / (n \times d \times t) = 710\ 919/(6\times20\times14) = 423.2 \text{ N/mm}^2 < \sigma_{b,Rd} = 476.2 \text{ N/mm}^2.$$

n, d and t being the number of bolts, the bolt diameter and the flange thickness, respectively.
The check of the gusset plates bearing resistance may be neglected because their total thickness (28mm) is greater than the flange thickness (20mm).
As far as the web gusset plates are concerned, the stress in the bolts is evaluated assuming that each bolt is subjected to a force proportional to the distance from the central bolt (see detail of fig. 6). The share of bending moment carried by the web gusset plates is evaluated as follows:

$$M_{web} = M_d\ (I_{web}/I) = 305\ 695\ 000 \times 22\ 549\ 333 / 401\ 173\ 845 = 17\ 183 \text{ Nm}$$

Under these assumptions, the shear force in the most stressed bolt is equal to:

$$F_{max} = F_1 = \frac{M}{4d_1 + 4d_2^2 / d_1} = 20270N$$

from which the shear stress in the bolt is obtained:

$$\tau_{max} = F_{max} / (2\times A_b) = 20270/(2\times245) = 41.4 \text{ N/mm}^2 < f_{d,v} = 385 \text{ N/mm}^2$$

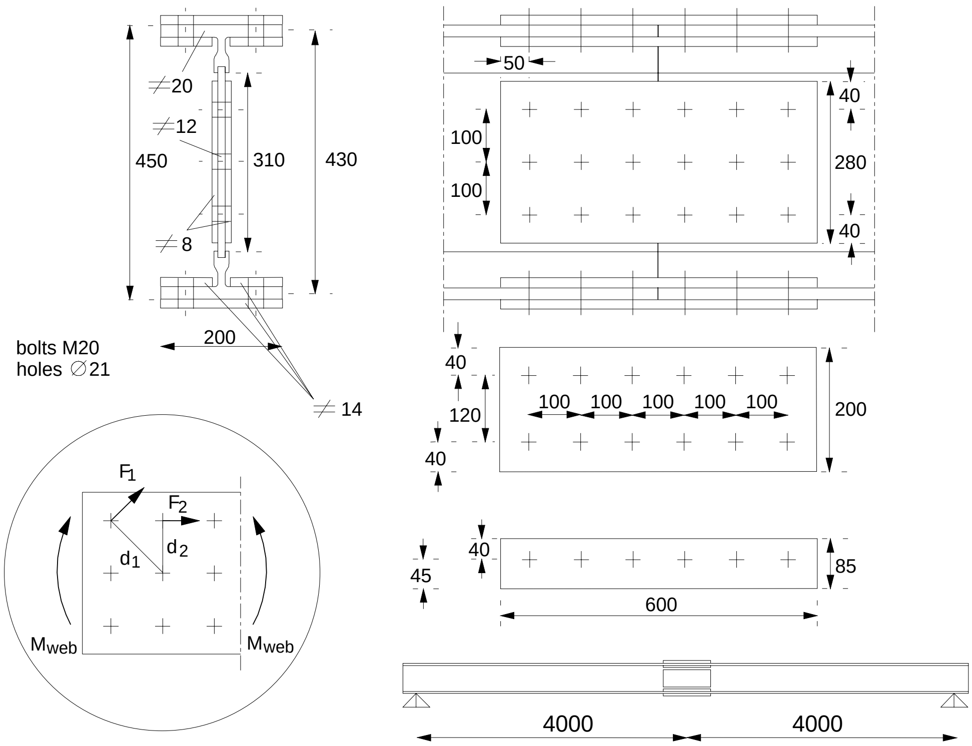

Fig. 6 – Detail of the girder connection

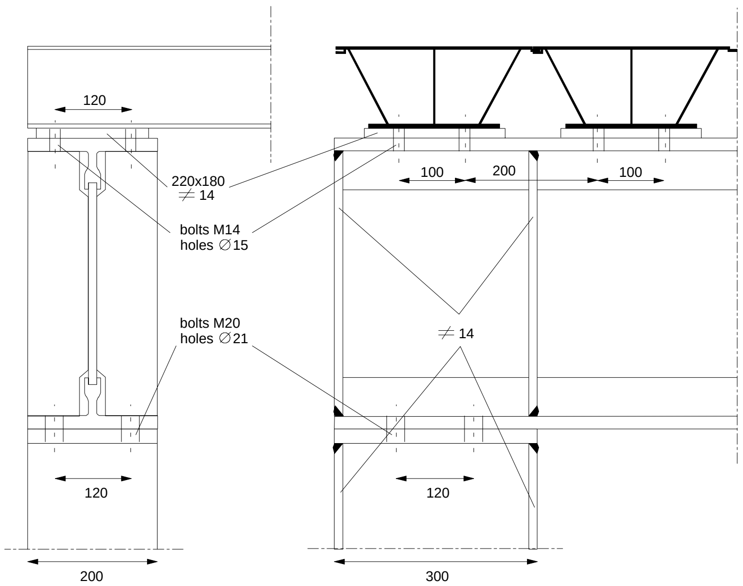

Fig. 7 – Detail of girder-to-deck connection

The representation of the deck-to-girder connection is shown in fig. 7, together with a possible arrangement of the connection to the supporting structure. This connection is made by bolting the girder bottom flange directly to a flush end plate welded to the support point. Because of the very little resisting moment developed by this joint, it can be considered as nominally pinned. A 14mm thick rectangular plate is placed between the top flange of the girder and each deck element in order to compensate the overthickness introduced by the gusset plate of the midspan joint.