Ribs

Schemes & Checks

The rib schemes are more numerous and open to a wider range of materials and cross-sections compared to the spar elements. Multiple ribs will be required for the MWP, VTP and HTP, so it will be important to optimise their design to avoid excess weight.

Referring to the Loads notes for rib loading, we can now check selected trial rib schemes.

Contents

4. Ribs

- Schemes
- Checks
  - Stiffness
  - Stength
  - Stability

After a quick recap on Rib loading, these notes lay out some example rib schemes, followed by checks for stiffness, strength and stability.

LOADS -Ribs - recap

AVDASI 2 UAV 2022-2023 RIB CALC ILLUSTRATION

JRF

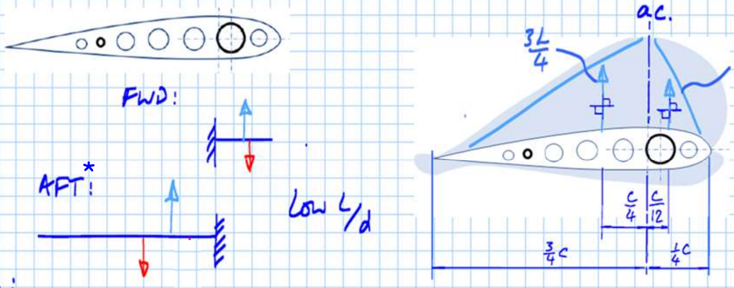
Note Title 16/10/2021

05.10.2022

1.

From the loading Calcs:

MODEL: Fwd and aft cantilevers each side of ac.



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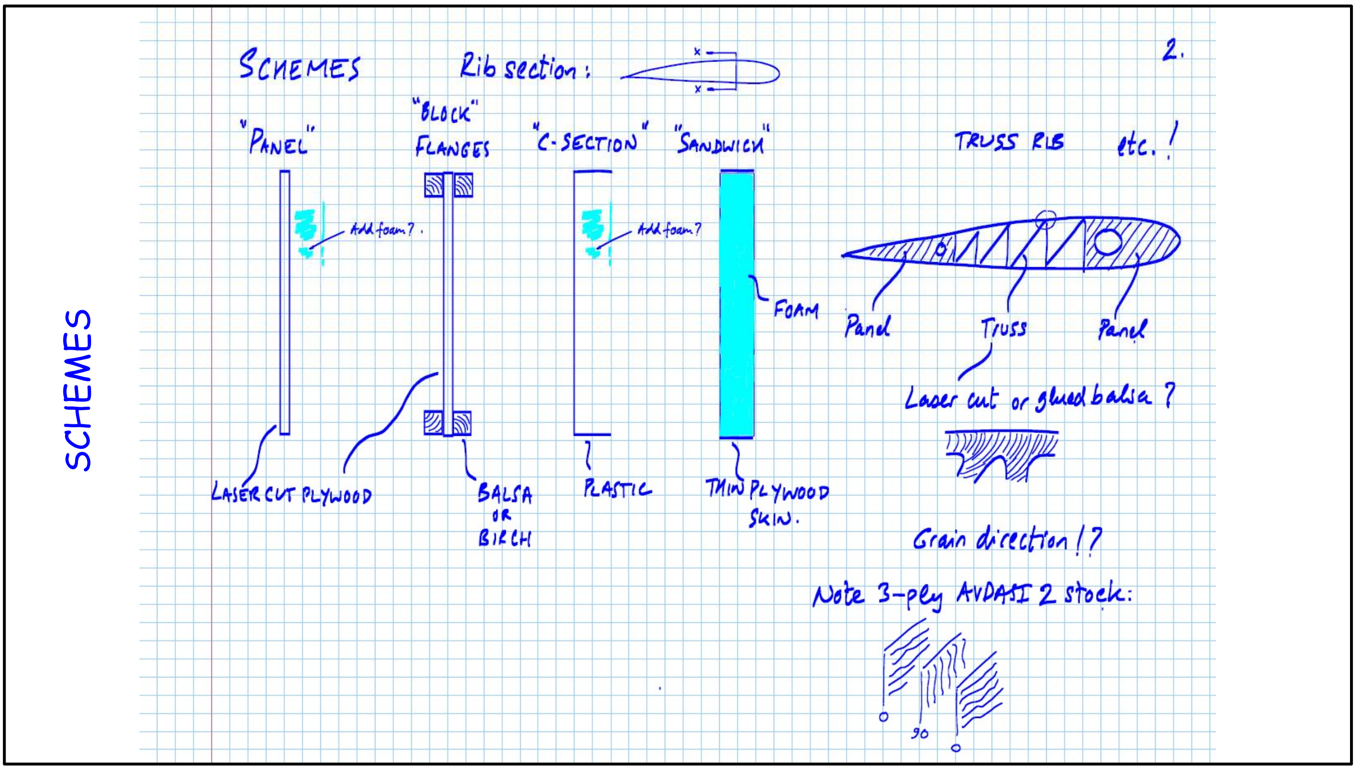
Note:

- These rib beams are short and deep so they do not fully obey the assumptions of simple bending, resulting in some error which should be corrected.
- Bending deflection will be small and shear deflection will no longer be negligible in comparison, so we should account for shear deformation here.

The ribs can be modelled as “stubby” i.e., short, deep beams, which are cantilevered off the main spar and loaded by the share of lift and inertia loading from the rib-bay on each side of the rib. From the Loads notes, the aerodynamic loading is simplified to point loads at the centre of triangular chordwise distributions and the inertia load can initially be assumed to act typically around 40% of the chord from the nose.

Simple bending analysis is only directly applicable to long slender beams, so for stubby rib beams, we will need to apply some corrections to the analysis. E.g.: 1) shear deflection (neglected in simple beam theory) should now be accounted for, and 2) assumption of linear bending stress distribution and effective shear areas need to be corrected for short, deep beams.



I have suggested a few different rib section schemes here, but other schemes or combinations of these schemes may also be considered.

Note The plywood supply is “3-ply” and as such will have orthotropic properties as a wooden composite laminate. Also note the subtlety of plywood grain direction, particularly at the details within some of the schemes.

CHECK - Stiffness

STIFFNESS

3.

Bending and Shear

└ Account for shear deformation of short deep beams.

Second mmt of areas and effective shear areas: see X1 MAPP

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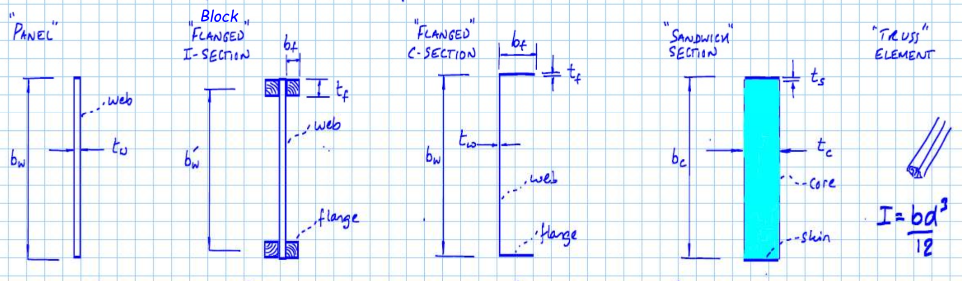

$I = \frac{t_w b_w^3}{12}$ | $I \approx \frac{t_w b_w'^3}{12} + 4 \times t_f b_f \times \left(\frac{b_w'}{2}\right)^2$ | $I = \frac{t_w b_w^3}{12} + 2(t_f b_f)\left(\frac{b_w}{2}\right)^2$ | $I \approx 2(t_s t_c) \times \left(\frac{b_c}{2}\right)^2$

$A_s = t_w b_w$ ($\times \frac{5}{6}$) | $A_s = t_w b_w'$ | $A_s = t_w b_w$ | $A_s = t_c b_c$

see X1 MAPP

$G = G_{FOAM}$

To check the stiffness of the different schemes, we need to identify the area properties. Note, some of the second moments of areas quoted on this slide are initial estimates which neglect the second-order terms in the parallel axis sum. The effective shear areas are also simplified initial estimates, but acceptable for initial design.

Deflection checks: 4.

Bending: $v_b = \frac{kPL^3}{EI}$
where I from p3.

Bending deflections of deep beams will be relatively low, so that shear deflection, neglected in simple bending, must be considered as a more relevant component of deflection

Shear: $v_s = \frac{PL}{GA_s}$

where A_s = effective shear area. See X1 MAPP.

Compare total deflection with the allowable or acceptable value

i.e. Check: $v_b + v_s < v^*$ @ LIM

* Ref: Roark. for beams with low "L/d"

For the rib scheme checks, you should consider the same critical load case as the main spar which carries the rib.

For the stiffness check, you should also account for the shear deflection and add this to the bending deflection to give the total rib deflection. You can then check whether the deflection is reasonable at limit load (remember, we check stiffness at limit loading).

CHECK - Strength

STRENGTH

5.

#

As a first approximation:

- for unflanged panel ribs
  we need to consider combined bending and shear loading on the web, where lateral stability may be an issue.

- for flanged or sandwich panel ribs
  we shall assume that elements carry <u>either</u> bending <u>or</u> shear and check only the primary loading for each.

  i.e. – For flanges and skins: check axial stress due to bending
  – For webs or cores: check shear stress due to transverse load

For an unflanged panel rib scheme, you will need to consider the combined bending and shear stresses, whereas for flanged or sandwich rib schemes, you can simplify by assuming that the particular elements of the cross section only carry their predominant loading. I.e., in a flanged or sandwich section, the flanges or skins can be assumed to carry only axial stress due to bending and the webs or cores can be assumed to carry only shear stress due to transverse loading. This will be quite reasonable for initial design.

Truss ribs can be assumed to act as simple trusses, assuming effective pin joints at the bonded ends of slender truss elements where the trusses' ability to transfer bending is neglected due to its flexibility.

Unflanged panel web. - Combined stress check

6.

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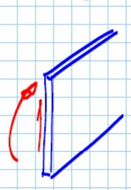
For an unflanged web we need to consider combined bending and shear using a failure criterion based on failure indices.



I.e. similar to our consideration for spars.

Check: $\left(\frac{\sigma}{\sigma^*}\right)^2 + 2\left(\frac{\tau}{\tau^*}\right)^2 = FI < 1$ @ ULT

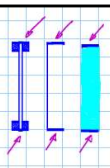
See X1 folder MAPP.

Where σ and τ are the applied bending and shear stress at ultimate and σ^* and τ^* as the ultimate failure strengths of the panel under pure direct and shear stress

e.g.: $\sigma^* = \sigma_x^*$ and $\tau^* = \tau_{xy}^*$ for plywood $\hookrightarrow$ $\boxed{RF = \frac{1}{\sqrt{FI}}}$

For an unflanged panel rib, as a web only, the combined bending and shear stresses must be accounted for by using a failure criterion.

Flanges + Skins - Bending stress check. 6.



Check: $\sigma = \frac{MY}{I} \times k_{DB} < \sigma^*$ @ ULT

where k_{DB} = bending stress factor for short deep beam

= 1.03^* for our UAV rib dimensions

$Y = \frac{b'_w}{2}$ Where $b'_w \approx b_w$ for thin flanges or skins #

and I from p3

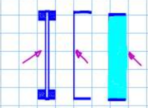
$$\boxed{RF = \frac{\sigma^*}{\sigma}}$$

For flanged or sandwich ribs, the second moments of area can be simplified as just the contribution from the flanges or skins, i.e., assuming only the flanges or skins react bending.

Note, when calculating the bending stress for the ribs as stubby cantilever beams, we need to apply a correction factor (since simple bending theory is based on long slender beams).

7.

Webs + Cores - Shear stress check.



Check: $\tau = \frac{S}{A_s} \cdot k_{DS} < \tau^*$ @ ULT

where k_{DS} = shear stress factor for short deep beam

= $1{\cdot}59^*$ for our UAV rib dimensions.

* Ref: Roark. for beams with low "l/d"

A_s = effective shear area.

$\boxed{RF = \frac{\tau^*}{\tau}}$

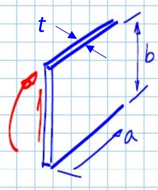
As a reasonable initial estimate, the effective shear area of flanged or sandwich ribs can be taken as just the web or core alone. Also, as a stubby beam, a shear stress correction factor must be applied.

CHECK - Stability

STABILITY

9.

Unflanged panel web - buckling check



Bending: $\sigma^*_{b_{CR}} = K_b E \left(\frac{t}{b}\right)^2$

where $K_b = \frac{k_b \pi^2}{12(1-\nu^2)}$

Shear: $\tau^*_{CR} = K_s E \left(\frac{t}{b}\right)^2$

where $K_s = \frac{k_s \pi^2}{12(1-\nu^2)}$

Elastic panel buckling formulae for inplane bending + shear

For typical a/b of AVDASI 2 UAV ribs. #

$\left.\begin{array}{l} k_b = 24.0 \\ k_s = 5.5 \end{array}\right\}$ for simply supported edges i.e. flanged. #

But ÷ 10 for free edges! *

Using combined loading failure criterion:

Check: $R_b^2 + R_s^2 = FI$: $\left(\frac{\sigma}{\sigma^*_{bCR}}\right)^2 + \left(\frac{\tau}{\tau^*_{CR}}\right)^2 = FI < 1$ @ ULT: #

where σ, τ are applied stresses. ↳ $\boxed{RF = \frac{1}{\sqrt{FI}}}$

*Unflanged panel buckling strengths low! → Foam stabilise?

An unflanged panel rib is particularly susceptible to buckling since all edges are free and the panel carries all bending and all shear stress as a web alone.

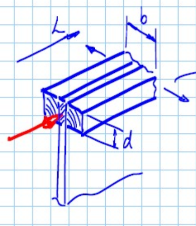
The panel buckling equations given on this slide include “buckling constants” for bending and shear at typical a/b panel aspect ratios with simply supported edges. Note for free edges, the simply supported buckling constant values should be reduced by an order of magnitude, i.e. => K/10.

The interaction of bending and shear stress is accounted for by a failure criterion, similarly to previous calculations.

Note, to counter the inherently low stability of unflanged panels, you could consider the use of foam for stabilisation.

10.

"Block" flanges – Lateral (Euler) buckling check.



lateral stability

Check: $\sigma = \frac{MY}{I} < \sigma_{CR}^{*}$ @ ULT

where $\sigma_{CR}^{*} = \frac{\pi^2 E I}{L^2 / A}$

$I = \frac{d b^3}{12}$

$A = b.d.$

Using "Euler buckling" as a conservative estimate of lateral stability.

Accounting for the "block" flanges and intermediate web area

$\hookrightarrow \boxed{RF = \frac{\sigma_{CR}^{*}}{\sigma}}$

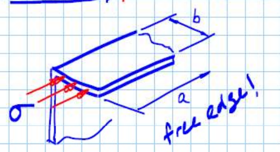
For panels with "block-flanges", we can consider the lateral stability of the blocked section by approximating it as a Euler strut, buckling sideways to the rib plane.

11.

Slender Flanges - Panel buckling check

Check: $\sigma = \frac{MY}{I} < \sigma_{CR}^{*}$ @ ULT

where $\sigma_{CR}^{*} = K_c E \left(\frac{t}{b}\right)^2$ for compression loading.



$K_c = \frac{k_c \pi^2}{12(1-\nu^2)}$ $k_c = 0.426$ for flange a/b ratio with a free edge + simply supported edges

↳ $\boxed{RF = \frac{\sigma_{CR}^{*}}{\sigma}}$

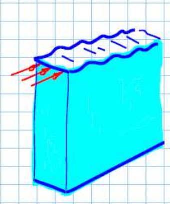
Note, the buckling model is linear, i.e. based on elastic modulus, E
so if $\sigma_{CR}^{*} > \sigma_y^{*}$ then use $\sigma_{CR}^{*} = \sigma_y^{*}$ as a preliminary limit.
i.e truncate predicted buckling strength at the yield strength of the material

For ribs with slender flanges, we can consider the stability of the flanges as panels. For simplicity, the flanges can be considered as only carrying end-loading due to bending as the predominant loading, i.e., neglecting shear. The buckling strength of a flange under end-compression loading due to bending is then given by the classic panel buckling equation.

As a general note, remember that the buckling models are based on an elastic modulus, so if a prediction of buckling strength exceeds the proof strength of the material, then the buckling strength should be rejected since the material will have failed earlier by plastic deformation, i.e., by yielding before the predicted elastic buckling strength can be achieved.

Skins - skin wrinkling check

12.



Check $\sigma = \frac{MY}{I} < \sigma_{CR}^*$

Where $\sigma_{CR}^* = \frac{1}{2}\left[\frac{G_c . E_c . E_s}{(1-\nu_s^2)}\right]^{1/3}$ semi-empirical formula

G_c = Shear modulus of foam core.

E_c = Compressive modulus of foam core

E_s = Modulus of skin $= \sqrt{E_x . E_y}$ for ply skin

ν_s = Poisson's ratio of skin $= \sqrt{\nu_{xy} . \nu_{yx}}$ " "

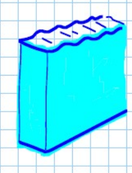
$\hookrightarrow \boxed{RF = \frac{\sigma_{CR}^*}{\sigma}}$

For thin skin sandwich panels subjected to bending, "skin wrinkling" will be the most susceptible instability on the compression side. A semi-empirical expression is given here to predict the skin wrinkling buckling strength. As you would expect, the elastic moduli of the skin and core materials all play a part in the resulting value.

As an initial estimate we can use "equivalent isotropic" values, given by the square root of the product of orthogonal values for orthotropic materials, such as plywood or laminated cfrp.

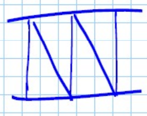
Cores - "Yielding" check

13.



For thick styrofoam core assume "yielding" failure before buckling

Truss elements - Euler buckling check



Check: $\sigma = \frac{P}{A} < \sigma_{CR}^{*}$ @ ULT

where P = truss element load.

$$\sigma_{CR}^{*} = \frac{\pi^2 E I}{L^2} \Big/ A$$

I = min' 2nd mmt of area of truss element

assume pinned ends for slender truss elements and truncate predicted σ_{CR}^{*} value @ σ_y^{*}

$$\boxed{RF = \frac{\sigma_{CR}^{*}}{\sigma}}$$

The Styrofoam used in this exercise will be more likely to "yield" before collapsing by buckling when used in sandwich cores.

Rib truss schemes can be modelled as a simple truss, assuming that the truss elements are slender so that moment transfer is secondary at the ends of the truss elements. I.e., assuming that the ends of the elements are effectively pin-jointed. The most critical failure to be checked will then be the Euler buckling of a truss element in compression.