Design of Fibre Reinforced Polymer structures – load combinations and partial factors to be used together with JRC document on 'Design of FRP'

– with Fiberline products

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Contents

1 Introduction

This report describes the specification for application of the JRC (2016) report for design of FRP load bearing structures, incl. the load combinations to be used and the basis for selecting material partial factors for design according to the principles in the Eurocodes as described in EN 1990 (2002). Especially, load combinations and the model for material partial factors for obtaining design values for resistances in the Danish national annex to the Eurocodes are described, DK NA EN 1990.

Section 2 describes the load combinations and load partial factors to be used following the Danish National Annex to EN1990.

Section 3 describes the models for obtaining design values for load bearing capacities following the Danish National Annex to EN1990. Also described is the statistical basis for estimating material partial factors for design of fibre reinforced polymer structures obtained from tests with typical fibre reinforced polymer and calculation models.

Section 4 describes the specifications for use of the JRC (2016) report for design of FRP structures in connection with the Eurocodes EN1990 and EN1991, and the corresponding Danish national annexes.

Finally Annex A describes the background for the material partial factors in chapter 4.

2 Load combinations in EN1990 and in DK NA EN1990

Generally all limit states in EN1990 (2002) have to be checked. However, only one design situation for one limit state is considered in this this report, namely STR for Persistent and transient design situations:

SLS – Serviceability Limit States

SLSs are not considered in this report.

ULS – Ultimate Limit States

In accordance with EN1990 (2002) the following ultimate limit states (ULS) have to be verified:

EQU: Loss of static equilibrium.
EQU is not considered in this report.

STR: Internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs.
The following design situations have to be checked:

- Persistent and transient design situations.
- Accidental. These design situations are not considered in this report.
- Seismic. These design situations are not considered in this report.

GEO: Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance.
GEO is not considered in this report.

FAT: Fatigue failure of the structure or structural members.
FAT is not considered in this report.

The design value of the action effect E_d for STR can be determined by the following three models where "+" indicates 'to be combined with' when determining action effects (e.g. deformations or cross-sectional forces):

1) Verification of eq. (6.10):

STR (6.10): $$E_d \text{ '=' } \sum_j \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_{Q,1} Q_{k,1} \text{ "+" } \sum_{j>1} \gamma_{Q,j} \psi_{0,j} Q_{k,j}$$

where

$G_{k,j}$ characteristic value of permanent load from source no. *j*

$Q_{k,1}$ characteristic value of the dominating variable load

$Q_{k,j}$ characteristic value of non-dominating variable load no. *j*

$\gamma_{G,j}$ partial factor for permanent load from source no. *j*

$\gamma_{Q,1}$ partial factor for the dominating variable load

$\gamma_{Q,j}$ partial factor for non-dominating variable load no. *j*

$\psi_{0,j}$ load combination factor for variable load no. *j*

2) Verification of both eq. (6.10a) and (6.10b):

STR (6.10a): $E_d\,\text{'='}\sum_j \gamma_{G,j} G_{k,j}\,\text{"+"}\,\gamma_{Q,1}\psi_{0,1}Q_{k,1}\,\text{"+"}\sum_{j>1}\gamma_{Q,j}\psi_{0,j}Q_{k,j}$

STR (6.10b): $E_d\,\text{'='}\sum_j \xi_j \gamma_{G,j} G_{k,j}\,\text{"+"}\,\gamma_{Q,1}Q_{k,1}\,\text{"+"}\sum_{j>1}\gamma_{Q,j}\psi_{0,j}Q_{k,j}$

where

ξ reduction factor for unfavourable permanent actions G

3) Verification of both eq. (6.10a) and (6.10b), but no variable loads in eq. (6.10a):

STR (6.10a): $E_d\,\text{'='}\sum_j \gamma_{Gj} G_{k,j}$

STR (6.10b): $E_d\,\text{'='}\sum_j \xi_j \gamma_{Gj} G_{k,j}\,\text{"+"}\,\gamma_{Q,1}Q_{k,1}\,\text{"+"}\sum_{j>1}\gamma_{Q,j}\psi_{0,j}Q_{k,j}$

The recommended values for the partial factors in EN 1990, Annex A1 for **buildings** are

$\gamma_{G,\sup}$ = 1.35 where unfavourable

$\gamma_{G,\inf}$ = 1.00 where favourable

γ_Q = 1.5 where unfavourable (0 where favourable)

ξ = 0.85 (so that ξ x $\gamma_{G,\sup}$ = 0.85x1.35 ≈ 1.15)

$\psi_{0,j}$ see EN 1990, Annex A

The recommended values for the partial factors in EN 1990, Annex A2 for **bridges** are

$\gamma_{G,\sup}$ = 1.35 where unfavourable

$\gamma_{G,\inf}$ = 1.00 where favourable

γ_Q = 1.35 when Q represents unfavourable actions due to road or pedestrian traffic (0 when favourable)

γ_Q = 1,45 when Q represents unfavourable actions due to rail traffic, to groups of loads 11 to 31 (except 16, 17, 26 and 27), load models LM71, SW/0 and HSLM and real trains, when considered as individual leading traffic actions (0 when favourable)

γ_Q = 1.20 when Q represents unfavourable actions due to rail traffic, to groups of loads 16 and 17 and SW/2 (0 when favourable)

γ_Q = 1.50 for other traffic actions and other variable actions

ξ = 0.85 (so that ξ x $\gamma_{G,\sup}$ = 0.85x1.35 ≈ 1.15)

In the Danish National annexes for buildings DK NA EN1990 (2013) and bridges EN 1990/A1 DK NA (2009) model 3) is chosen because it gives a more uniform reliability level with respect to different combinations of permanent and variable loads than using (6.10). The following load combinations have to be checked in ULS according to the Danish National annex

STR (6.10b): $E_d\,\text{'='}\sum_j \gamma_{GA,j} G_{k,j}\,\text{"+"}\,\gamma_Q Q_{k,1}\,\text{"+"}\sum_{j>1}\gamma_Q\psi_{0,j}Q_{k,j}$

STR (6.10a): $E_d\,\text{'='}\sum_j \gamma_{GB,j} G_{k,j}$

For buildings the following values are applied, see DK NA EN1990 (2013):

$\gamma_{GA,\sup}$ partial factor for permanent load in (6.10b) = 1.0

$\gamma_{GB,\sup}$ partial factor for permanent load in (6.10a) = 1.2

γ_Q partial factor for variable load =1.5

$\psi_{0,j}$ load combination factors, see DK NA EN1990

For bridges the following values are applied, see EN 1990/A1 DK NA (2009):
ξ = 1.00
$\gamma_{GA,\text{sup}}$ = 1.00 for permanent load from structural elements, soil and ground water
$\gamma_{GA,\text{inf}}$ = 0.90 for permanent load from structural elements
$\gamma_{GA,\text{inf}}$ = 1.00 for permanent load from soil and ground water
γ_{Q} = 1.40 for traffic load on bridges
γ_{Q} = 1.20 for heavy special transports on rail (SW2)
γ_{Q} = 1.40 for imposed loads during execution
γ_{Q} = 1.50 for al other variable loads (climatic loads etc.)
$\gamma_{GB,\text{sup}}$ = 1.25 for permanent load from structural elements
$\gamma_{GB,\text{sup}}$ = 1.00 for soil and ground water (1.25 in cases where the structure is loaded by soil or (ground) water)
$\gamma_{GB,\text{inf}}$ = 1.00 for permanent load from structural elements, soil and ground water

Consideration of consequence classes are done by multiplying all unfavorable loads with the factor K_{FI}, see Table 1. The consequence classes in the Danish national annex are defined in DK NA EN1990:2013.

Consequence class	Low – CC1	Medium – CC2	High – CC3
K_{FI}	0.90	1.00	1.10

Table 1. K_{FI} - factor depending on consequence class.

According to DK NA EN 1990/A1:2009 for bridges CC1 is not used for bridges and CC2 is only applied in special cases.

3 Safety format for resistances and materials

3.1 Design values of resistances and materials

In DK NA EN1990 (2013), annex E it is described how partial factors for materials and resistances are to be obtained. The same material partial factors are applied for buildings and bridges. The description follows the overall principles in EN1990 (2002) but gives a more detailed model to determine the material partial factors depending on the level of uncertainty measured by the coefficient of variation (COV).

It is assumed that uncertainties related to strengths and resistances can be modelled by LogNormal distributions. This is in accordance with the recommendations in EN 1990 (2002) and JCSS' 'Probabilistic Model Code' (2002). Characteristic values are determined as 5%-quantiles following EN 1990 Annex D. In assessment of the material partial factors it is important to account for 'hidden safety' or bias in the calculation models. Annex D in EN 1990 contains a method to quantify this bias by comparing test results with the calculation model.

Three models may be applied to determine the design value of the resistance. Model 1 and 2 are based on a calculation model, whereas model 3 is based on tests performed to obtain the resistance directly.

Note: the sub-partial factor γ_3 in DK NA EN1990 (2013) is not used, since the effect of control is assumed to be accounted for by γ_2 and γ_4.

Model 1

The design value of the resistance R_d is determined by:

$$R_d = \frac{R(X_d, d_d)}{\gamma_R} \tag{1}$$

where

$R(X,d)$ calculation model for resistance *R* as function of strength parameters *X* and geometry *d*

γ_R partial factor related to uncertainty in calculation model (model uncertainty) - including uncertainty associated with the transfer from laboratory conditions to conditions in an actual structure

d_d design value of geometrical parameters

X_d design value of strength parameters

If the calculation model depends on more than one strength parameter then design values are applied for each of the strength parameters.

The design value of the strength parameter, X_d is determined by:

$$X_d = \eta \frac{X_k}{\gamma_m} \tag{2}$$

where

η modification factor for e.g. load duration, temperature and scale effects, see below

X_k characteristic value of strength parameter

γ_m $= \gamma_4$: partial factor for strength parameter - including uncertainty associated with the transfer from laboratory conditions to conditions in an actual structure, see below

The partial factor γ_R related to the resistance model is determined by:

$$\gamma_R = \frac{\gamma_1 \gamma_2}{b} \tag{3}$$

where

γ_1 depends on type of failure (with or without warning), see below

γ_2 depends on the uncertainty related to the calculation model, see below

b bias (additional safety) related to the calculation model

The following simplification can be applied for a calculation model linear in the strength parameters:

$$R_d = R\left(\eta \frac{X_k}{\gamma_M}, d_d\right) \tag{4}$$

where

$$\gamma_M = \gamma_m \gamma_R = \frac{\gamma_1 \gamma_2 \gamma_4}{b} \tag{5}$$

Model 2

The design value of the resistance R_d is obtained by:

$$R_d = \eta \frac{R_k}{\gamma_M} \tag{7}$$

where

η modification factor for e.g. load duration, temperature and scale effects

R_k characteristic value of resistance determined by

$$R_d = R(X_k, d_d) \tag{4}$$

γ_M partial factor determined by

$$\gamma_M = \frac{\gamma_1 \gamma_4}{b} \tag{8}$$

Model 3

The design value of the resistance R_d is obtained by:

$$R_d = \eta \frac{R_k}{\gamma_M} \tag{7}$$

where

η modification factor for e.g. load duration, temperature and scale effects

R_k characteristic value of resistance determined by tests

γ_M partial factor determined by

$$\gamma_M = \gamma_1 \gamma_4 \tag{8}$$

3.2 Material partial factors

The sub-partial factors γ_2 and γ_4 are calibrated corresponding to an annual reliability index equal to 4.3 for buildings and 4.8 for bridges. It is noted that the same material factors are used for buildings and bridges whereas the load partial factors are different due to the different reliability levels.

Table 2 shows the sub-partial factor, γ_1 which depends on the type of failure of the structure.

No warning of failure refers to failure that occurs without prior warning (e.g. in the form of increased crack formation or deformation) and significant reduction of resistance immediately after a failure (e.g. in the event of stability failure or brittle fracture).

Warning of failure without residual resistance refers to failure where a warning is given of exhausted resistance (e.g. in the form of increased crack formation or deformation) and the resistance is retained for some time after the warning.

Warning of failure with residual resistance refers to failure where the resistance increases (e.g. as a result of strain hardening) after a formal failure has occurred (e.g. in the event of the permissible strain being exceeded). If the residual resistance is utilized in the calculation models, the failure type is to be taken as "Warning of failure without residual resistance".

Type of failure	Warning of failure with residual resistance	Warning of failure without residual resistance	No warning
γ_1	0,90	1,00	1,10

Table 2. γ_1 - Sub-partial factor depending on type of failure.

Table 3 shows the sub-partial factor γ_2 which depends on the coefficient of variation of the model uncertainty related to the calculation model. The coefficient of variation can be established by comparing resistances determined by testing the structural members and by applying the calculation model, with the use of measured/given strength parameters and geometric dimensions. As an exception, the coefficient of variation may be determined as an estimate. The coefficient of variation includes uncertainty associated with the transfer from laboratory conditions to conditions in an actual structure.

Coefficient of variation of the calculation model, V_δ	≤ 5 %	10 %	15 %	20 %	25 %
γ_2	1.05	1.10	1.15	1.20	1.25

Table 3. γ_2 - Sub-partial factor γ_2 for uncertainty of the calculation model.

Table 4 shows the sub-partial factor γ_4 depends on the coefficient of variation for the measured strength parameter or resistance. The coefficient of variation is to include the uncertainty associated with the transfer from laboratory conditions to conditions in an actual structure.

Coefficient of variation for measured strength parameter or resistance, V_r	≤5 %	10 %	15 %	20 %	25 %	30 %
γ_4	1.15	1.20	1.25	1.30	1.35	1.40

Table 4. γ_4 - Sub-partial factor for measured strength parameter or resistance.

4 Application to Fibre Reinforced Polymer (FRP) structures

This section describes the application of JRC (2016), PART II for design of fibre reinforced polymer (FRP) load bearing structures in Denmark using FRP structures from Fiberline. The design is assumed to be performed using the Eurocodes EN 1990 and EN1991 with Danish National Annexes. Chapter 2 describes the load combinations and partial factors for loads to be applied. For resistances and materials the safety format described above is applied.

NOTE – the following specifications can be considered as a 'Fiberline' annex for application of CEN (2016), PART II for design of FRP structures, since specific requirements are included related to Fiberline manufacturing processes.

In general the following specifications only apply to pultruded components and related connections/joints, and not to e.g. sandwich panels.

Material strength parameters and resistance of structural elements are assumed to be modelled by LogNormal stochastic variables with characteristic values determined according to EN 1990, Annex D.

Chapter 1 shall be used.

Chapter 2 shall be used with the following changes:

- Section 2.3.2: characteristic and design values shall be determined using DK NA1990 and section 3.1 above.
- Section 2.3.3: design values of capacities shall be determined using section 3.1 above.
- Section 2.3.4: Table 2.1 is exchanged with, see Annex A:

  Table 2.1 NA – values of γ_{M2}

ULS (strength)	Local stability	Global stability
1.5	1.65	1.65

  2.3.4.1(3) and (4) can not be used.
  2.3.4.2(1) can not be used
- Section 2.3.6.4 can not be used.

Chapter 3 shall be used with the following additions:

- A quality control system shall be established to verify the characteristic values of the materials used in design and the coefficients of variation (COV) used in annex A for determination of the material partial factors, see also chapter 9.
  Characteristic values and COVs for the following strength parameters shall be documented:
  - Tensile strength – parallel and perpendicular
  - Compressive strength – parallel and perpendicular
  - Pin-bearing strength – parallel and perpendicular

  The quality control system shall at least document on a regular basis that the characteristic values and COV for the flexural strength are satisfactory and fulfils the assumptions in annex A using the statistical control requirements in EN1990, Annex D.

Chapter 4 shall be used.

Chapter 5 shall be used.

Chapter 6 shall be used – except sections 6.3, 6.4 and 6.5

Chapter 7 shall be used.

Chapter 8 shall be used – except section 8.4.
- Additional information to section 8.2 can be found in Oppe (2009)

Chapter 9 shall be used.

Chapter 10 (Annex A) shall be used.

Chapter 11 shall not be used. Refer to annex A

Chapter 12-15 (Annex C-F) shall be used.

5 References

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DS INF 172 (2009) Sørensen, J.D., J. Munch-Andersen, S.O. Hansen, F.O. Sørensen, H.H. Christensen, P. Lind & A. Poulsen: Background investigations in relation to the drafting of National Annexes to EN 1990 and EN 1991 – Reliability verification formats, combination of actions, partial coefficients, fatigue, snow load, wind load, etc. Dansk Standard, 2009.

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Trumpf, H. (2006) Stabilitätsverhalten ebener Tragwerke aus pultrudierten faserverstärkten Polymerprofilen. Shaker Verlag, Aachen 2006.

Annex A – Background for material partial factors

This annex describes the background for the material partial factors in Table 2.1 NA in chapter 4.

According to section 2.2 the following sub-material partial factors are to be chosen:

- If a calculation model is applied to determine the resistance:
  - If model 1 is applied:
    - b depending on calculation model (determined using EN 1990, Annex D)
    - γ_1 depending on failure mode
    - γ_2 depending on uncertainty related to calculation model
    - γ_4 depending on uncertainty related to material parameters
  - If model 2 is applied:
    - b depending on calculation model (determined using EN 1990, Annex D)
    - γ_1 depending on failure mode
    - γ_4 depending on uncertainty related to calculation model incl. uncertainty related to material parameters
- If a calculation model is not applied, but the resistance is determined based on tests:
  - Model 3 is applied:
    - γ_1 depending on failure mode

  This method is not being used in the following.

Table A1 and A2 show examples of mean values, 5% fractile values and corresponding COV (Coefficient Of Variation) values from FBD 300 (2008).

			Average values		5% fractile characteristic values		COV	
Material characteristics	Unit	Test standard	Top flange	Bottom flange	Top flange	Bottom flange	Top flange	Bottom flange
Modulus-axial Tension Etx	N/mm²	EN ISO 527-4	27.000	27.000			-	
Modulus-axial Compression Ecx	N/mm²	EN ISO 14126	27.000	27.000			0.07***	
Modulus-transverse Tension Ety	N/mm²	EN ISO 527-4	14.000	14.000			-	
Modulus-transverse Compression Ecy	N/mm²	EN ISO 14126	14.000	14.000			0.19***	
Strength-axial Flexural ffx	N/mm²	EN ISO 14125	400	*	300	*	0,17	
Strength-axial Tension ftx	N/mm²	EN ISO 527-4	450	450	350	350	0,15	0,15
Strength-axial Compression fcx	N/mm²	EN ISO 14126	240	240	205	205	0,10	0,10
Strength-transverse Flexural ffy	N/mm²	EN ISO 14125	120	*	100	*	0,11	
Strength-transverse Tension fty	N/mm²	EN ISO 527-4	130	130	100	100	0,16	0,16
Strength-transverse Compression fcy	N/mm²	EN ISO 14126	115	115	100	100	0,08	0,08
Interlaminar shear strength-axial tm	N/mm²	EN ISO 14130	23	23	20	20	0,08	0,08
In-plane shear modulus G **	N/mm²	ISO 15310	3.000	3.000				

Table A1. Average- and characteristic values for the local analysis – flanges. ***) indicative numbers.

			Average values		5% fractile characteristic values		COV	
Material characteristics	Unit	Test standard	Outer web	Inner web	Outer web	Inner web	Top flange	Bottom flange
Modulus-axial Tension Etx	N/mm²	EN ISO 527-4	20.000	20.000			-	
Modulus-axial Compression Ecx	N/mm²	EN ISO 14126	15.000	15.000			-	
Modulus-transverse Tension Ety ***	N/mm²	EN ISO 527-4	18.000	18.000			-	
Modulus-transverse Compression Ecy ***	N/mm²	EN ISO 14126	18.000	18.000			0.04***	
Strength-axial Tension ftx	N/mm²	EN ISO 527-4	450	450	350	350	0,15	0,15
Strength-axial Compression fcx	N/mm²	EN ISO 14126	250	250	200	200	0,14	0,14
Strength-transverse Tension fty ***	N/mm²	EN ISO 527-4	90	120	68	100	0,17	0,11
Strength-transverse Compression fcy ***	N/mm²	EN ISO 14126	130	150	80	100	0,30	0,25
In-plane shear strength tm **	N/mm²	ASTM D 5379	65	65	61	61	0,04	0,04
In-plane shear modulus G **	N/mm²	ASTM D 5379	3.000	3.000			-	

Table A2. Average- and characteristic values for the local analysis – webs. *) These tests shall not be carried out in accordance to "Test Program FBR 300, 12. Dezember 2006". **) Tests made at RISØ. ***) Tests made at IMA. ***) indicative numbers.

These values indicate that in general COV for the material strength parameters are below 15-20% except ‘Strength-transverse Compression fcy’ for webs.

Table A3 shows examples of estimation of material partial factors for failure modes where the dominating failure mode is stability failure. The statistical data are from Trumpf (2006). The Table shows the coefficient of variation related to the calculation model, V_δ, the coefficient of variation related to the material parameters in the calculation model, V_r and the bias *b* related to the calculation model. This means that these uncertainty measures are related to the calculation models described in Trumpf (2006).

	Local buckling		Global buckling			
			Flexural buckling			Lateral-torsional buckling
	Web	Flange	Square profile	I and U-profile Weak axis	I and U-profile Strong axis	I profile
V_δ	0,11	0,07	0,10	0,15	0,13	0,17
V_r	0,13	0,10	0,18	0,13	0,28	0,17
b	1,08	1,31	1,26	1,30	1,69	1,15
γ_1	1,10	1,10	1,10	1,10	1,10	1,10
γ_2	1,11	1,07	1,10	1,15	1,13	1,17
γ_4	1,23	1,20	1,28	1,23	1,38	1,27
$\gamma_1\gamma_2\gamma_4$	1,50	1,41	1,55	1,56	1,72	1,63
$\gamma_1\gamma_2\gamma_4/b$	1,39	1,08	1,23	1,20	1,01	1,42

Table A3. Uncertainties related to calculation models from Trumpf (2006) and associated material partial factors.

The following material partial factor γ_M is based on the following assumptions:

- The calculation models in Trumpf (2006) or similar models are used to estimate the characteristic load bearing capacity (defined as a 5% quantile) using characteristic (5% quantiles) of the material strength parameters and the bias *b* is not accounted for when calculating the characteristic load bearing
- The failure mode is brittle
- The material partial factor γ_M (incl. an additional uncertainty factor 1.05) is applied to the characteristic load bearing capacity

Using the sub-partial factors in section 3.2:

- Local buckling: γ_M = 1.5
- Global buckling: γ_M = 1.5

V_δ	0,05
V_r	0,06
b	0,95
γ_1	1,00
γ_2	1,05
γ_4	1,16
$\gamma_1\gamma_2\gamma_4$	1,22
$\frac{\gamma_1\gamma_2\gamma_4}{b}$	1,28

Table A4. Uncertainties related to calculation models from Feldmann & Oppe (2007) and associated material partial factors.

Table A4 shows examples of estimation of material partial factors for failure modes where the dominating failure modes such as bending. The statistical data are from Feldmann & Oppe (2007). It is noted that these uncertainty measures are related to the calculation models described in Feldmann & Oppe (2007).

The following material partial factor γ_M (incl. an additional uncertainty factor 1.05) is based on the following assumptions:

- The calculation models in Feldmann & Oppe (2007) or similar models are used to estimate the characteristic load bearing capacity (defined as a 5% quantile) using characteristic (5% quantiles) of the material strength parameters and the bias *b* is not accounted for when calculating the characteristic load bearing
- The failure mode is ductile
- The material partial factor γ_M is applied to the characteristic load bearing capacity

Using the sub-partial factors in section 3.2 the following partial factor for ULS (strength) is obtained:

- γ_M = 1.35

The above material partial factors are related to the computational models in Trumpf (2006). However, these models are different from the models in JRC (2016). Figures A2-A5 show the ratios

χ_T / χ_{WG4} between the load bearing capacities using the models in Trumpf (2006) and the models in JRC (2016) developed by TC250 WG4.

Model in Trumpf for <u>Flexural buckling</u> with α = 0.45 for Square profile, 0.75 for weak axis, 0.5 for strong axis:

$$\phi(\lambda,\alpha) := 0.5\cdot\left[1 + \alpha\cdot(\lambda - 0.5) + \lambda^2\right]$$

$$\chi_T(\lambda,\alpha) := \mathrm{if}\left[\lambda < 0.5, 1, \left(\phi(\lambda,\alpha) + \sqrt{\phi(\lambda,\alpha)^2 - \lambda^2}\right)^{-1}\right]$$

Model in JRC/WG4:

$$\varnothing(\lambda) := 0.5\cdot\left(1 + \lambda^2\right)$$

$$\chi_{WG4}(\lambda) := \frac{1}{0.65\cdot\lambda^2}\cdot\left(\varnothing(\lambda) - \sqrt{\varnothing(\lambda)^2 - 0.65\cdot\lambda^2}\right)$$

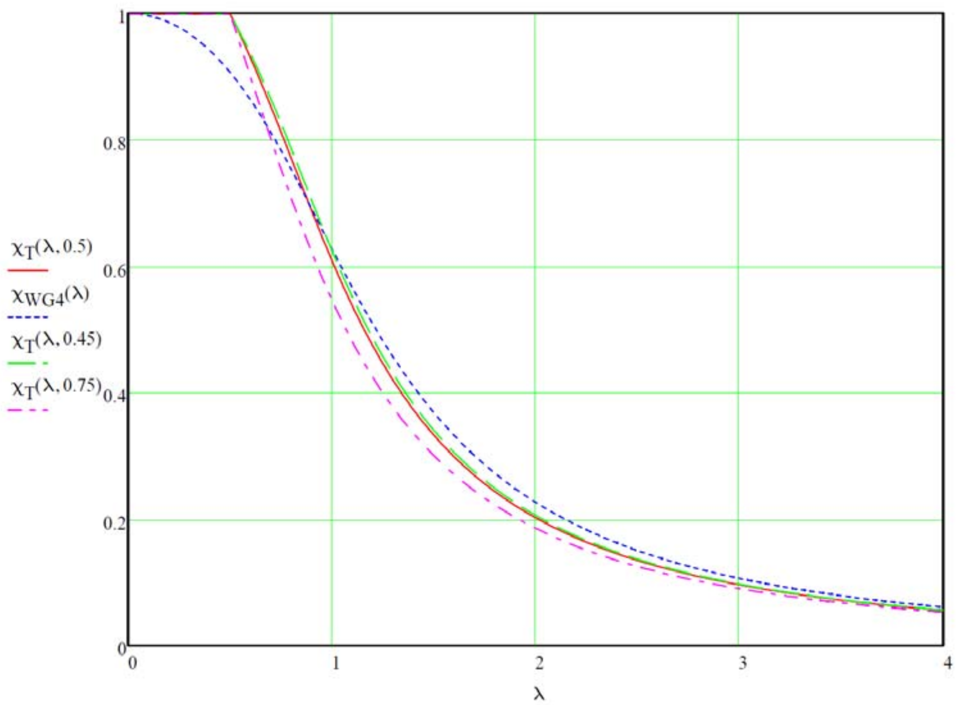

Figure A2. χ_T and χ_{WG4} for <u>Flexural buckling</u>: with α = 0.45 for Square profile, 0.75 for weak axis, 0.5 for strong axis for Trunpf model, from Plum (2015).

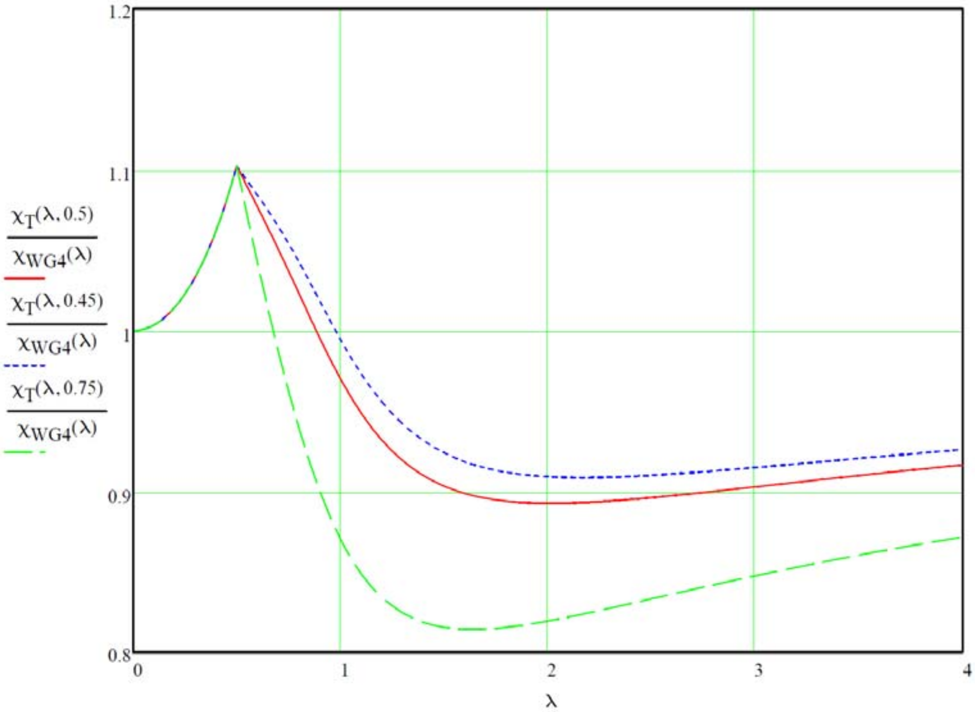

Figure A3. χ_T / χ_{WG4} for Flexural buckling: with α = 0.45 for Square profile, 0.75 for weak axis, 0.5 for strong axis for Trunpf model, from Plum (2015).

Model in Trumpf Lateral-torsional buckling with α = 0.5:

$$\phi(\lambda,\alpha) := 0.5\cdot\left[1 + \alpha\cdot(\lambda - 0.5) + \lambda^2\right]$$

$$\chi_T(\lambda,\alpha) := \text{if}\left[\lambda < 0.5, 1, \left(\phi(\lambda,\alpha) + \sqrt{\phi(\lambda,\alpha)^2 - \lambda^2}\right)^{-1}\right]$$

Model in JRC/WG4:

$$\text{Ø}(\lambda) := 0.5\cdot\left(1 + \lambda^2\right)$$

$$\chi_{WG4}(\lambda) := \frac{1}{0.70\cdot\lambda^2}\cdot\left(\text{Ø}(\lambda) - \sqrt{\text{Ø}(\lambda)^2 - 0.70\cdot\lambda^2}\right)$$

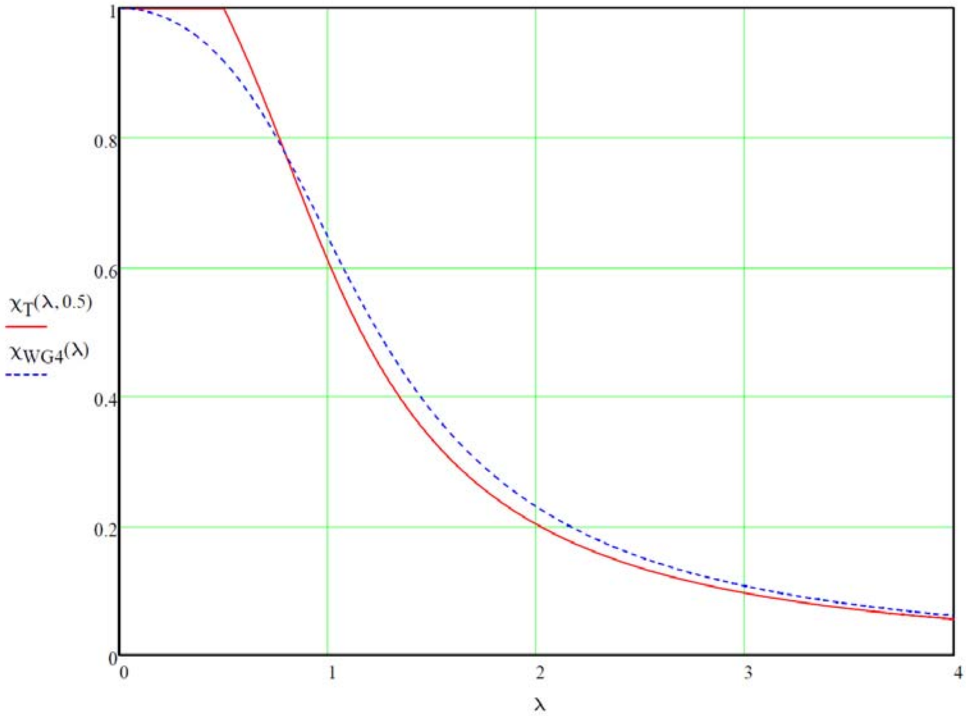

Figure A4. χ_T and χ_{WG4} for Lateral-torsional buckling with α = 0.5, from Plum (2015).

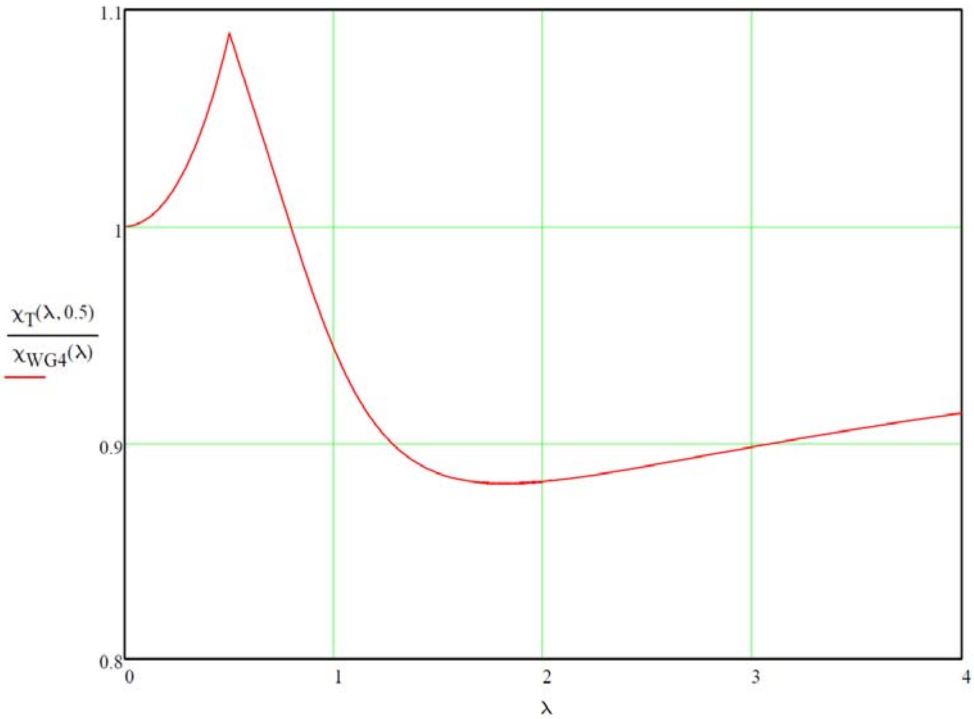

Figure A5. χ_T / χ_{WG4} for Lateral-torsional buckling with α = 0.5, from Plum (2015).

Based on the results in Figures A2 and A4 it is seen that the ratio χ_T / χ_{WG4} can approximately and on the safe side be put equal to 0.9 except for the case with flexural buckling with I- and U-profiles, Strong axis where the ratio can be put to 0.8. This implies that the material partial factors to be modified to

- Local buckling: $\gamma_M = 1.65$
- Global buckling: $\gamma_M = 1.65$

In a similar way to account for the sparse documentation for the uncertainties related to the computational model for ULS (strength) the partial factor is modified to:

- $\gamma_M = 1.5$

resulting in the values in Table 2.1 NA.