4 Diubal

Version April 2015

Program

## **PLATE-BUCKLING**

Plate Buckling Analysis for Stiffened and Unstiffened Plates According to EN 1993-1-5 and DIN 18800-3

# Program **Description**

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## . Introduction

## 1.1 About PLATE-BUCKLING

The European standard Eurocode 3 (EN 1993-1-5:2010-12 + NA 2010-12) describes design and construction of plate-like structural steel components used in the member states of the European Union. With the add-on module PLATE-BUCKLING, the DLUBAL company provides a powerful tool for designing plate-like structural components. Country-specific regulations are taken into account by National Annexes (NA). In addition to the parameters included in the program, you can define your own limit values or create new National Annexes.

PLATE-BUCKLING can be used as a stand-alone program or as an add-on module in RSTAB or RFEM. In the add-on module, you can import design-relevant input data and internal forces from the current RSTAB or RFEM model.

Finally, the design process can be documented in the global printout report, from input data to design.

PLATE-BUCKLING performs all typical stability, stress, and deformation analyses as well as the torsional buckling safety check for stiffeners. The Stability analysis is carried out according to the method of stresses reduced by the interaction criterion. Furthermore, analytical equations for calculation of critical buckling stresses from Annex A are implemented, thus allowing for a calculation using the eigenvalue solution.

We hope you will enjoy working with PLATE-BUCKLING.

Your DLUBAL-Team





### 1.2 PLATE-BUCKLING Team

The following people were involved in the development of PLATE-BUCKLING:

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### 1.3 Using the Manual

Topics like installation, graphical user interface, results evaluation, and printout are described in detail in the manual of the main program RSTAB or RFEM. The present manual focuses on typical features of the add-on module PLATE-BUCKLING.

The descriptions in this manual follow the sequence of the module's input and results windows as well as their structure. The text of the manual shows the described **buttons** in square brackets, for example [View mode]. At the same time, they are pictured on the left. **Expressions** appearing in dialog boxes, windows, and menus are set in *italics* to clarify the explanations.

At the end of the manual, you find the index. However, if you do not find what you are looking for, please check our website **www.dlubal.com** where you can go through our comprehensive *FAQ* pages by selecting particular criteria.

## 1.4 Opening the PLATE-BUCKLING Module

There are several possibilities to start the add-on module PLATE-BUCKLING.

### Menu

۲

To start the program in the RSTAB or RFEM menu bar, click

Add-on Modules  $\rightarrow$  Design - Steel  $\rightarrow$  PLATE-BUCKLING.

Add	d-on Modules Window	<u>H</u> el	р	
<b>*</b> 0	Current Module	Q	> <u> </u> 🕺	🎫 🖌 🖓 🖓 🐜 🗱 🗱 🕸 🕼 🎘 👘 🖓 🔶 🕼
	Shape Properties	• <mark>8</mark> -	M - 🕲 -	17 🖘   J. 🔻 🗸 🗸 MT My Mz 🍺 🔁   🚼 🔡
	Design - Steel	P	STEEL	General stress analysis of steel members
	Design - Concrete	Fe	STEEL EC3	Design of steel members according to Eurocode 3
	Design - Timber	Auso	STEEL AISC	Design of steel members according to AISC (LRFD or ASD)
	Design - Aluminium	LIS	STEEL IS	Design of steel members according to IS
	Dynamic	SIA	STEEL SIA	Design of steel members according to SIA
	Connections	1 <sub>BS</sub>	STEEL BS	Design of steel members according to BS
	Stability	1 <sub>G8</sub>	STEEL GB	Design of steel members according to GB
	Towers	Is.	STEEL CS	Design of steel members according to CS
	Others	1	KAPPA	Flexural buckling analysis
	1	13	LTB	Lateral-torsional and torsional-flexural buckling analysis
		4	FE-LTB	Lateral-torsional and torsional-flexural buckling analysis by FEM
		14	EL-PL	Elastic-plastic design
			C-TO-T	Analysis of limit slenderness ratios (c/t)
		1	PLATE-BUCKLIN	IG Plate buckling analysis
		<b>₽</b> ₿	VERBAND	Design of wind bracings for roofs

Figure 1.1: Menu: Add-on Modules  $\rightarrow$  Design - Steel  $\rightarrow$  PLATE-BUCKLING



### Navigator

As an alternative, you can start the add-on module in the Data navigator by clicking

```
Add-on Modules \rightarrow PLATE-BUCKLING.
```

Project Navigator - Data	×
i	*
🚋 🚈 🛅 Results	
Printout Reports	
🗄 🗉 🛅 Guide Objects	
📥 🛁 Add-on Modules	
SHAPE-THIN 7 - Design of thin-walled cross-sections	
STEEL - General stress analysis of steel members	
	Ξ
📲 ALUMINIUM - Design of aluminium members according to Eurocod	
C-TO-T - Analysis of limit slenderness ratios (c/t)	
PLATE-BUCKLING - Plate buckling analysis	
BRACING - Design of wind bracings for roofs	
CONCRETE Columns - Design of concrete columns	Ŧ
4	
🔁 Data 🖀 Display 🔏 Views	

Figure 1.2: Data navigator: Add-on Modules  $\rightarrow$  PLATE-BUCKLING



## 2. Input Data

After you have opened the module, a new window appears with a navigator on the left showing all selectable windows. Above the navigator, you see a drop-down list with the possibly already available design cases.

You can select a window either by clicking the corresponding entry in the PLATE-BUCKLING navigator or by using the buttons shown on the left. You can also use the function keys to select the next [F2] or previous [F3] window.

The animated graphics in the info field help visualize your input. In addition to that, you can click [Graphics] to visualize and manage your input data.

To save your specifications and exit PLATE-BUCKLING, click [OK]. If you click [Cancel], you exit the module but without saving the data.

After you have entered all relevant data, click [Calculate] to generate the structural system defined in PLATE-BUCKLING and to calculate it with RFEM or RSTAB and RF-STABILITY/RSBUCK. Then, the results of the eigenvalue analysis are evaluated in PLATE-BUCKLING in order to carry out the relevant designs and to show them in the input windows.

## 2.1 General Data

In the window *General Data*, you have to enter the plates to be analyzed, their geometry, the material properties, and boundary conditions. Furthermore, you have to define the standard according to which you want to perform the plate buckling analysis.



Figure 2.1: Window 1.1 General Data



-

÷.

Graphics

Cancel

Calculate

OK



### Material

This section allows you to select one of the materials stored in the program. You can select from the steel materials that are allowed by the selected standard. The respective steel grades and their properties are stored in the program's library. To access the library containing the steel grades, click [Material Library]. To import the material to Window 1.1, select it and confirm the selection by clicking [OK].

When you have entered the panel dimensions a, b, and t, the Euler critical stress is computed.

According to EN 1993-1-5:

$$\sigma_{E} = \frac{\pi^{2} \cdot E}{12 \cdot (1 - \upsilon^{2})} \cdot \left(\frac{t}{b}\right)^{2}$$

According to DIN 18800-3:

$$\sigma_{\mathsf{E}} = \frac{\pi^2 \cdot \mathsf{E}}{12 \cdot (1 - \mu^2)} \cdot \left(\frac{\mathsf{t}}{\mathsf{b}}\right)^2$$

### **Standard / National Annex**

In the drop-down lists, you can select the standard and the National Annex (NA) for the design. Both the standards EN 1993-1-5 and DIN 18800-3 are available.

If you choose to design according to EN 1993-1-5, you can select in the list the National Annex whose parameters you want to apply for the design.

To check and, if necessary, adjust the preset parameters of the current National Annex or standard, click the [Edit] button. Those parameters are mainly partial safety factors used for the design.

To create a user-defined National Annex, click [New].

You can also delete a National Annex by clicking [Delete].

### **Panel Dimensions**

In the dialog section *Panel Dimensions*, you can enter geometrical specifications for the respective surface. According to the sketch, you have to specify the length of *Panel side a, b,* and plate *thickness t*. The side ratio  $\alpha$  is determined from these input parameters.



Panel	Panel Dimensions					
Panel	Panel side					
a :	4000.0 🜩 [mm]					
b :	4647.0 🔶 [mm]					
Side n	atio					
α:	0.861 🔶 🕨 [-]					
Thick	ness					
t :	27.0 <table-cell-rows> [mm]</table-cell-rows>					
Figure 2.2: Panel Dimensions						



```
DIN 18800 Germany
EN 1993-1-5 European Union
CEN CEN European Union
CEN CEN European Union
```

NBN Belgium

NEN Netherlands

- 🎦 🔄

🔽 🔟 CEN

💿 EN 1993-1-5



### **Boundary Conditions**

This dialog section allows you to define the support of the buckling panel. You can select *Hinged*, *Built-in*, *Unsupported*, or *Hinged - Elastic*.

The boundary conditions to be considered in this calculation depend on the characteristics of the plate edges and are influenced by the connection of adjacent parts. Fully hinged or built-in edges do not exist in practice, since the plates usually form flanges and webs of beams. Use the option *Hinged - Elastic* to consider the real support by entering spring stiffness resulting from the adjacent parts.

A common simplified assumption is that the plates have hinged supports along their edges. With this assumption you are on the save side. In PLATE-BUCKLING, you have to specify these geometric boundary conditions for displacement, rotation, and warping on the nodes of the four plate edges of the entire plate. The following is assumed:

- Built-in edge (rotation restrained)
- Hinged edge (freedom of rotation)
- Free edge (rotation and displacement are possible perpendicular to the plate plane)
- Hinged elastic edge (rotation is partially restrained)

Boundary Conditions	
Edge A-B:	
Hinged 🗸	1
Hinged	
Built-in Unsupported Hinged - Elastic	
Edge A-C:	
Hinged 🔹	
Edge B-D:	
Hinged -	

Figure 2.3: Boundary Conditions

In case you perform a design according to DIN 18800-3, the following option is available:

• Uniform Edge Displacement u according to Table 1, row 5

With this option, you decide how to calculate the reduction factor  $\kappa$  for buckling panels supported on three sides.

### Comment

In this input field, you can enter user-defined notes.

Comment	
Design acc. to DIN EN 1993-1-5	*
	~

Figure 2.4: Comment



## 2.2 Stiffeners

le Settings Help							
A1 - Plate buckling analysis 🛛 🔻	1.2 Stiffer	ners					
nput Data	Lonaitudi	nal Stiffeners					
General Data		A	В	C	D	E	
Stiffeners	Stiffener		Position [cm]		Stiffener	-	
- Loads	No.	z	×1	×2	Туре	Parameters [mm]	
LC1	1	3098.00	0.00	4000.00	Flat plate	300/30 R	
	2						
	3						
	4						
	5						
	6						
	7						
	8						
	9						
	10						
	12						
	12						
	13						
	Transver	se Stiffeners					
	~ "	A	B	C	D	E ^	
	Stiffener		Position [cm]		Stiffener	Demonstra front	
	110.	X	21	22	Type	Farameters (mm)	
	2						
	3						
	5						
	6						
	7						
	8						
	9						
	10						
	11						
	12					-	
	14						
	12						
	12						
	12						

Figure 2.5: Window 1.2 Stiffeners

In this two-part input window, the upper part *Longitudinal Stiffeners* contains the stiffeners arranged in longitudinal direction and to be included in the calculation. The lower part offers the same input possibilities for *Transverse Stiffeners*. Both tables are identical except for the mentioned difference and will therefore be described together.

The various additional functions facilitate the work in this window. The buttons are reserved for the following functions:

Button	Description	Function
	Regular stiffener positions	Distributes the existing stiffeners over the buckling panel depth uniformly
-	Copy row	Copies the currently selected row to the next row
*	Delete row	Deletes the selected row
<b>1</b>	Export to Excel	Exports the current table to MS Excel
<b>B</b>	Import from Excel	Imports the existing input from MS Excel

Enter the *Position* of the longitudinal and transverse stiffeners in the columns A to C. According to **EN 1993-1-5**, these are the positions *z*, *x*1, *x*2 or *x*, *y*1, *y*2 and according to **DIN 18800-3**, the positions *y*, *x*1, *x*2 or *x*, *y*1, *y*.



In column D Stiffener Type, specify the cross-section of the stiffeners. You can choose from the following stiffener types.

F	lat plate
E	Bulb Flat Steel
A	Ingle
٦	-Stiffener
٦	Trapezoidal stiffener
F	Rolled L-section
F	Rolled T-section
F	Rolled C-section

Figure 2.6: Stiffener Types

After you have selected the type, you can enter the cross-section parameters in a dialog box.

Stiffene	er Material		
Steel S	S 355 JR (EI	N 10025-2:2004-11)	7
Stiffene	r Parameter	S	÷
Dimensi	ions		t., i .,t
h:	80.0	[mm]	
b:	50.0	[mm]	Provide a construction of the construction of
s:	12.0	[mm]	
t	10.0	[mm]	
Position	1		
z:	3098.00	[mm]	
Arrange	ement:		
💿 Sho	rt leg		
C Long	g leg		
Clos	ed		
Орм Орм	vard ,		1
	vriward		
Left	at		
<ul> <li>Both</li> </ul>	n sides		
0.000			

1

0

In this dialog box, you can specify the type and orientation of the stiffeners. PLATE-BUCKLING allows you to define various materials for the stiffeners and buckling panel. You can select one of the materials that depend on the selected standard and are stored in the program's material list. The corresponding steel grades and their properties are stored in the library. To open the data base containing the steel grades, click [Library]. When you have selected a material, click [OK] to transfer it to Window 1.2.

To receive information on the cross-section values of the selected stiffener, click [Info].



### 2 Input Data

To save the specified data in a stiffener data base, click [Save]. The data can be reimported by clicking [Load].

User-Defined Stiffener Library		×
Stiffener Type	Stiffener	
Flat plate	Flat plate 300/30 B	<u> </u>
		-
		-
		+
		X
$\mathfrak{D}$	ОК	ancel

Figure 2.8: User-Defined Stiffener Library

To transfer a saved stiffener to the dialog box *Flat Plate Stiffener* and, if necessary, modify it, double-click the relevant stiffener in the library.

Click [OK] to exit the *Flat Plate Stiffener* dialog box and to transfer the stiffener to the PLATE-BUCKLING input Window 1.2. If you do not want to transfer the stiffener to the input table, click [Cancel].

In column E *Parameters* of Window 1.2, the properties of the specified stiffener are displayed. To edit the input data, click the button [...].

When you have defined the loads in Window 1.3 *Loads*, the layout in Window 1.2 *Stiffeners* changes as follows.



Figure 2.9: Window 1.2 Stiffeners

Program PLATE-BUCKLING © 2015 Dlubal Software GmbH

Cancel

....



### **Effective flange widths**

This table part is shown only after you have entered the loading in Window 1.3.

The effective flange widths of the stiffeners are used to determine the critical buckling stresses and calculate the critical buckling stress of the stiffeners.

You have to specify whether or not to calculate the effective width according to the standard (EN 1993-1-5 or DIN 18800). When you have cleared the check box, you can define the effective flange widths in columns F to I manually.

According to EN 1993-1-5, the Table 4.1 or 4.2 is used, according to DIN 18800-3, chapter (4). Note that you have to consider the *Boundary Conditions* defined in Window 1.1 *General Data*.

Moreover, the respective normal stress resulting from the provided loads is displayed for the respective stiffener under the normal stresses.

If several load cases are defined, the effective flange widths are calculated and displayed separately. Then you can use the drop-down list to choose the individual load cases.

## 2.3 Loads

In this window, you can specify the loads (stresses) on the buckling panel.



Figure 2.10: Window 1.3 Loads

### Load Case

Assign a *No*. to the new load case and enter a *Description*. To show the descriptions already used, click [▼]. To create a new load case No., click [New]. To delete a current load case, click [Delete].

### **Boundary Stresses**

In this dialog section you have to specify the effective normal stresses (*Normal stresses in x-direction*), shear stresses, and transverse stresses (*Normal stresses in z-direction*). Compressive stresses are specified as positive, tensile stresses as negative, and shear stresses as positive.





#### Note on Normal stresses in z-direction

In PLATE-BUCKLING, you can combine transverse stresses and local transverse stresses. The superposition principle is used here. Thus, the stress resulting from the superposition is the governing loading of the buckling panel.

### Import stresses from RSTAB or RFEM

from RSTAB ..

3

Click [from RSTAB]/[from RFEM] to import the buckling panel stresses from RSTAB resp. RFEM. A dialog box opens where you can select the *Member*, the *c/t-Part*, and the relevant *Load Cases*.

mport panel from RSTAB						×
Import Geometry from				IPE 360		
Member No.: 44 € c/t-Part No.: 5 ▼	Position of plate in me Start x1: End x2:	mber 0.00 🖈 (cr 627.40 🖈 (cr	n] n]		1	
Import Loads from Load Ca	se				· <del>5</del> · - · - · - ·	····••y
Coll Cases Coll - Swt+s+wx+p+l Coll - Swt+s+lmp (ch Coll - Swt+wx+lmp (c Coll - Swt+wy+lmp (c Coll - Swt+wind liftin Coll - Swt+wy+lmp Coll - Swt+wy+p+l Coll - Swt+s+wy+p+l Coll - Swt+s+wy+p+l Coll - Swt+s+wy+p+l Coll - Extreme design V RC1 - Extreme charac RC3 - Fire	mp (char. val rar. values) rar. values) g+lmp (char shar. values) (char. values) (char. values) mp (char. val +lmp (char. v values teristic values		III		3 ! 4 ¥ z	الس الله
					ОК	Cancel

Figure 2.11: Import panel from RSTAB

To select a member in the work window graphically by clicking it, use the button [<sup>k</sup>].



Figure 2.12: Graphical selection of members

Immediately after you have selected a member by clicking it, the number of the member is entered in the dialog box. Furthermore, when exiting the dialog box, a query appears as to whether or not you want to adjust the panel dimensions in Window 1.1 *General Data* to the geometrical conditions of the member.

The relevant panel can be selected from the list *c/t-Part No*. or in the cross-section graphic. Thus you can adjust the geometrical parameters of the panel in Window 1.1 *General Data*.



To receive further information on the buckling panel data, click [Details about c/t-parts].

/t-Parts o	of IPE 360   DIN	1025-5:199	4								×
	A	B	C	DÍ	E	F	G	H (	1	J	IPE 360
c/t-Part	Restrained	С	t	c/t	Coordinat	es Start	Coordinat	tes End	Average Stati	cal Moments	
No.	Shape	[mm]	[mm]	[-]	y [mm]	z [mm]	y [mm]	z [mm]	Qy [cm <sup>3</sup> ]	Q <sub>z</sub> [cm <sup>3</sup> ]	
1	One Side	63.0	12.7	4.96	-22.0	-180.0	-85.0	-180.0	69.47	25.60	
2	One Side	63.0	12.7	4.96	22.0	-180.0	85.0	-180.0	69.47	25.60	
3	One Side	63.0	12.7	4.96	-22.0	180.0	-85.0	180.0	69.47	25.60	
4	One Side	63.0	12.7	4.96	22.0	180.0	85.0	180.0	69.47	25.60	1 2
5	Both Sides	298.6	8.0	37.33	0.0	-149.3	0.0	149.3	480.28	0.00	and a second sec
											3 4 z
2											Close

Figure 2.13: c/t-parts

In the dialog section *Import Loads from Load Case* in the dialog box *Import panel from RSTAB* (see Figure 2.11), you can select the loading of the panel from all RSTAB load cases.

After exiting the dialog box *Import panel from RSTAB* by clicking [OK], the RSTAB calculation of the load cases not calculated yet is started automatically. To close the dialog box without importing the data, click [Cancel].

### **Graphic window**

To select the view mode of the graphic, click [Show figure or rendering]. In addition to the panel figure with the stress graphics, 3D rendering of the panel is possible.

You can control the rendering view of the panel selected for analysis by using the buttons shown on the left. If your pointer is in the graphic window, you can use the zoom and rotation functions. For more information, see [3], chapter 3.4.9.



Details.



## 3. Calculation

Before you start the [Calculation], it is recommended to check the design details. The corresponding dialog box can be accessed in all windows of PLATE-BUCKLING by clicking [Details].

## 3.1 Details DIN 18800

E-Model for Stiffeners		FE-I	Discretization	
3D using surface elements		Nur	nber of finite elements	
2D using members (without ecce	ntricity)	• T	otal minimum:	327
3D using members (with eccentri	city)	• T	otal maximum:	10000
Eigenvalues		• M	inimum on the plate	
Solver method:		de	epth:	8
Method by Lanczos		Cun	rent number of finite	
Roots of the characteristic polyn	omial	eler	nents:	26 X 13 = 338
Subspace iteration method		Nur	nber of buckling modes to	0
ICG iteration method		Cal	uidle.	0
Calculation for all eigenvalues		Det	ermination of Buckling Va	lue
		0	Calculate buckling value	es for unstiffened plates accordin
Solver version			to standard formulas if po (DIN 4114 - 1, Tab. 6)	ossible
🔘 32-bit			Always calculate using f	inite element analysis
64-bit			, majo odlodidio delligi	
Critical Global Buckling Stress with	Regard to Local Bu	ckling Effects		
Consider buckling effects accord	ling to Paragraph (5	i03). Ea. (13)		
Cross-section:			1	
		6	ļ	
Buckling perpendicular to axis v			Buckling perpendicu	ular to axis z
(about the major axis)			(about the minor axis	3)
Effective length	sĸ,y:	0.00 [cm]	s K,z : 0.0	<b>DO</b> [cm]
Radius of gyration	iy:	0.00 [cm]	iz : 0.0	<b>DO</b> [cm]
Buckling curve	BCv: a -		BCz: a 🔻	
-				

Figure 3.1: Dialog box Details - DIN 18800-3

### **FE-Model for Stiffeners**

In this dialog section, you have to specify according to which conventions the stiffeners are to be considered in the calculation of the buckling shape. In the settings *3D using surface elements* and *3D using members*, the real stiffnesses of the stiffeners are included in the calculation. The option *2D using members* considers the stiffener only in relation to the centroid as a line element with increased stiffnesses in the plate plane. Thus, the advantages of the stiffener's eccentric connection are lost. Use the *3D* options in order to better consider the effectiveness of the cross-section in the design ratio. If, however, you use the *2D using members* option, the calculation time will be reduced significantly.

### **Eigenvalues**

The determination of a buckling shape of a plate is performed as an eigenvalue calculation of the buckling panel. Here, the program calculates the ideal plate buckling values  $\sigma_x$ ,  $\sigma_y$ ,  $\tau$  as well as the ideal buckling value for the simultaneous occurrence of all stress components. To do this, you can use one of the three direct solver methods (*Method by Lanczos, Roots of the characteristic polynomial, Subspace iteration method*) or the iterative solver (*ICG iteration method*).



The direct solver methods are recommended for small and medium-sized models. However, the RAM memory should be large enough for the files of the triangular decomposition, or else this method will result in longer computing times. The method by Lanczos is preset because it is suited for most models. For further information on that method, see http://en.wikipedia.org/wiki/Lanczos\_algorithm.

The ICG iteration method should be applied if none of the direct methods is successful, or if it takes really much time to calculate large models. The advantages of that method are minimum requirements for the size of RAM memory as well as the output of accurate results in case of poorly convergent models, that is, systems that are close to instability.

To perform the plate buckling analysis for each selected eigenmode and its eigenvalues, select the option *Calculation for all eigenvalues*. If the check box is cleared, the first eigenmode is considered as governing.

### Solver Version

This dialog section controls whether the 32-bit or 64-bit solver method is to be applied.

### **FE-Discretization**

The fields *Number of finite elements* control the degree of refinement of the FE mesh. To obtain a good approximation solution, it might be necessary to increase the number of elements of the FE mesh. However, a large number of finite elements requires a longer computing time.

To carry out the calculation, at least four elements must be created Minimum on the plate depth.

PLATE-BUCKLING determines the most unfavorable buckling shapes, whereby the lowest buckling shape corresponds with the governing buckling shape. A large *Number of buckling modes to calculate* will affect the computing time.

### **Determination of Buckling Value**

In this dialog section, you decide which method to use for the computation of buckling values: analytically according to *standard formulas* or according to the *finite element* method. If there are stiffeners in the model, FE-BEUL uses the option *Always calculate using finite element analy-sis* to calculate the buckling value. The applied calculation method is documented in the result window.

### **Critical Local Buckling Stresses with Regard to Global Buckling Effects**

If a buckling analysis is required for the structural component containing the buckling panel, select the check box *Consider Buckling Effects according to Paragraph (503), Eq. (13)*. This is the case if the buckling panel is, for example, part of a compression member. Thereby, a reciprocal influence of plate buckling and local buckling is given: If single cross-section elements of the compression member buckle before the critical compressive force is reached, a reduction of stiffness will be the result for the compression member. The bearable compressive force falls to a value smaller than the critical compression force.

If you select the check box, the fields below become available for the input of the parameters.



To use the cross-section library from RSTAB or RFEM, click [Library]. To import the cross-section properties and the length of a member from the work window in RSTAB or RFEM, use the [<sup>5</sup>] button. The parameters for *Buckling perpendicular to axis y* are then entered automatically. The cross-section's effective length, radius of gyration, and buckling curve can be defined manually, too.



### 3.2 Details EN 1993-1-5

EM-Model for Stiffeners	FEM-Discretization
<ul> <li>Surface elements</li> </ul>	Number of finite elements
Beam elements (without eccentricity)	• Total minimum: 10
Beam elements (with eccentricity)	• Total ma <u>xi</u> mum: 10000
igenvalue Solver Method	• Minimum on the plate
Solver method:	depth: 8
Method by Lanczos	Current number of finite
Roots of the characteristic polynomial	elements: 40 x 20 = 800
Subspace iteration method	Number of buckling modes to
ICG iteration method	calculate: 8
Calculation for all eigenvalues	Determination of Reduction Factors
oading on Longitudinal Stiffeners	Contribution from the web $\chi_W$ acc. to Tab. 5.1
Longitudinal stiffeners subject to direct stresses	<ul> <li>Rigid end post</li> </ul>
Conservation of internal forces (M.N)	Non-rigid end post
Cimplified critical buckling lead factor method	Help values acc. to Tab. B.1
	Welded or cold formed
Determination of Buckling Curve Shape	Hot rolled
Various buckling curves	Determination of Buckling Value
Generalized buckling curve	
Critical Plate Buckling Stresses for Stiffened Plates	to standard formulas if possible
<ul> <li>Eigenvalue analysis</li> </ul>	(Tab. 4.1 or Tab. 4.2)
According to Annex (A. <u>1</u> ; A.2; A.3)	Always calculate using finite element analysis
Solver Version	Distance Between Stiffeners
🗇 32-bit	Minimum distance between sitffeners: 30 ε t
64-bit	

Figure 3.2: Dialog box Details - EN 1993-1-5

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The dialog sections *FEM-Model for Stiffeners*, *Eigenvalue Solver Method*, *Solver Version*, *FEM-Discretization*, and *Determination of Buckling Value* are described in chapter 3.1.

### **Loading on Longitudinal Stiffeners**

If the *Longitudinal stiffeners subject to direct stresses* box is checked, the stresses defined in the longitudinal direction are also applied to the longitudinal stiffeners and will be considered in the calculation of the eigenvalues. If not, the stresses act on the panel only.



Figure 3.3: Longitudinal stresses applied to panel and stiffeners (left) or panel only (right)



### Longitudinal stiffeners subject to direct stresses

If the *Longitudinal stiffeners subject to direct stresses* check box is selected, the stress is distributed along the stiffeners according to the stresses defined in Window 1.3 *Loads*.

Loading on Longitudinal Stiffeners

Longitudinal stiffeners subject to direct stresses

Conservation of internal forces (M,N)

Simplified critical buckling load factor method



Figure 3.4: Stress distributed along the stiffeners

### Loading on stiffeners with conservation of internal forces

Loading on Longitudinal Stiffeners

Longitudinal stiffeners subject to direct stresses

Conservation of internal forces (M,N)

Simplified critical buckling load factor method



Figure 3.5: Stress distributed according to Conservation of internal forces (M, N)

In order to maintain the original internal forces (M, N) in the cross-section, the stress must be recalculated. Only in this way is it possible to observe the relationship between external loading and internal forces of the initial model where the stiffeners have not yet been implemented. Adding stiffeners to the panel without recalculating the stress will result in an imbalance between the external loads and the internal forces.

4 Diubal

To recalculate the stress, a linear stress distribution is considered as shown in Figure 3.5, using the following equations.

$$\oint_{A} \sigma(\overline{a}) d\overline{a} = \oint_{A} \sigma(a) da = N$$
$$\oint_{A} \sigma(\overline{a}) z \cdot d\overline{a} = \oint_{A} \sigma(a) da \cdot z = M$$

### Loading on stiffeners with simplified critical buckling load factor method



Figure 3.6: Stress distributed according to Simplified critical buckling load factor method

With this method, the horizontal stresses are completely concentrated in the stiffeners. An advantage of this method is that only few local extremes of the buckling modes have to be calculated. The stress in the stiffener corresponds to a proportional part of the length of the panel adjacent to the stiffener. This method is described in [18] at page 65.

$$F_{1} = \int_{L_{1}/2}^{L_{2}/2} \sigma(x) t dz + A_{Stiff_{-1}} \sigma_{2,3}$$

$$F_{2} = \int_{L_{2}/2}^{L_{3}/2} \sigma(x) t dz + A_{Stiff_{-2}} \sigma_{4,5}$$

$$\underline{\sigma_{2}} = \underline{\sigma_{3}} = F_{1} / A_{Stiff_{-1}}$$

$$\underline{\sigma_{4}} = \underline{\sigma_{5}} = F_{2} / A_{Stiff_{-2}}$$

5

It is possible to combine the *Simplified critical buckling load factor* and the *Conservation of internal forces* methods.



### **Determination of Buckling Curve Shape**

For the interaction formula and the reductions factors, EN 1993-1-5 provides the possibility either to use a *Generalized buckling curve* for the entire analysis or *Various buckling curves* for each existing stress.

### **Critical Plate Buckling Stresses for Stiffened Plates**

The *Annex* to EN 1993-1-5 provides analytical methods for the determination of critical plate buckling stresses. The following variants are included in the standard:

- one or two stiffeners in the compression area of the buckling panel
- three or more stresses in the buckling area

If the check box According to Annex is selected, PLATE-BUCKLING analyzes the stiffeners and load situation and calculates the critical plate buckling stress according to the governing variant. If the required restrictions are not met, the buckling values are determined according to the *Eigenvalue analysis* (FE method), thus computing the buckling stress.

### **Determination of Reduction Factors**

According to *Table 5.1* of EN 1993-1-5, you can select a rigid end-post or non-rigid end-post to determine  $\chi_w$  (web contribution). To make use of the standard's possibilities, choose between the two options.

When you determine the reduction factors for the plate buckling, the program, according to EN 1993-1-5, Annex B.1, *Table B.1*, distinguishes between welded or cold-formed and hot-rolled products. Use the check boxes to select one of these variants.









Immediately after the calculation, the Window 2.1 Governing Load Case appears. The Windows 2.1 to 2.5 display the designs including the explanations for each structural component. Every window can be opened from the PLATE-BUCKLING navigator. As an alternative, you can use the function keys to select the next [F2] or previous [F3] window.

Click [OK] to save the results. Thus you exit PLATE-BUCKLING and return to the RSTAB or RFEM work window.

Chapter 4 Results describes the results windows one by one. Evaluating and controlling results is described in chapter 5 Results Evaluation, page 27 ff.

#### Governing Load Case 4.1



Figure 4.1: Window 2.1 Governing Load Case

### Description

This column provides information on the description of load cases, load combinations, and result combinations governing for the respective designs.

### **Eigenvalue No.**

For each designed load case, load combination, and result combination, the number of the eigenvalue (buckling shape) with the highest design ratio is displayed.

### Design

For each type of design as well as for each load case or load combination and result combination, the design conditions are displayed according to EN 1993-1-5 or DIN 18800.

The colored scales visualize the ratios due to individual load cases.





### **Design According to Formula**

This column lists the code's equations by which the designs have been performed.

### Details

In the lower dialog section *Details*, the intermediate result for the performed designs are shown in a comprehensible form with references to the selected standard.

### **Graphic window**

The graphic window shows the designed panel using 3D rendering. The different view modes can be controlled by using the buttons bellow the graphic window. For more information, see [3], chapter 3.4.9.

### 4.2 Design by Load Case



Figure 4.2: Window 2.2 Design by Load Case

This window lists the maximum ratios and the corresponding governing designs of all load cases, load combinations, and result combinations defined for the design.



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## 4.3 Design by Eigenvalues

	2.3 Desig	in by Eigen	values								
Input Data			B		C [	D	I F	1		F	
General Data		Eigenvalue			bsol	Desig	1				
Stiffeners	No.	No.	Description		Case	Ratio			Design	gn According to Formula	
- Loads	1	1	Load Case 1		LC1	1.777	>1	104) Interaction according	to Ch. 10, Eq.	1. 10.5	
- LC1 - Load Case 1	2	2				1.719	>1	104) Interaction according	to Ch. 10, Eq.	1. 10.5	
LC2 - Load Case 2	3	3				1,391	>1	104) Interaction according	to Ch. 10, Eq.	10.5	
late Buckling Analysis	4	4				1.340	>1	104) Interaction according	to Ch. 10, Eq.	10.5	
- Governing Load Case	5	5				1.289	>1	104) Interaction according	to Ch. 10, Eq.	10.5	
— Design by Load Case	6	6				1.223	>1	104) Interaction according	to Ch. 10, Eq.	10.5	
<ul> <li>Design by Eigenvalues</li> </ul>	7	7				1.097	>1	104) Interaction according	to Ch. 10, Eq.	1. 10.5	
Design by All	8	8				1.097	>1	104) Interaction according	to Ch. 10, Eq.	a. 10.5	
Critical Buckling Load Factors											
					Max	1.777	>1	8			
	Geoge Stresses										
	H Edge Stresses										
	E Desig	n Ratio		_					_		
	Str	ess ratio		Ψx	-0.881			[1], Tab. 4.1	_		
	Str	ess ratio		Ψz	0.000			[1], Tab. 4.1	_		
	Bu	ckling load t	actor	o.cr,x	1.910			Determined using FEM	_		
	Bu	ckling load t	actor	α.or, τ	1.514			Determined using FEM	_		-
	Crit	ical load fac	tor	0.or	1.178			Determined analytically	_		
	BU	cking coefficient	cient	KX	88.691			Determined using FEM	_		
	Bu	cking coefficients	cient	KT -	28.228	NI /mm 2	_	Determined using FEM			
		ical plate bu	cruing stress	o or,p,x	568.2/6	N/mm <sup>2</sup>		Determined by FEM	_		
	0.4	ioni olato hu		Vor.p. t	180.867	111/10/04		Annex A (A. I)			
	Crit	ical plate bu	cking stress		2002 110	NI/mm2					
	Crit	ical plate bu ical plate bu	cking stress ckling stress	σcr.c.x	2093.110	N/mm <sup>2</sup>		[1]; Ch. 4.5.3 (3)	_		
	Crit Crit Eq	ical plate bu ical plate bu uivalent Stre	cking stress ckling stress ss Mises	σor,c,x σeqv,Ed	2093.110 362.500	N/mm <sup>2</sup> N/mm <sup>2</sup>		[1]; Ch. 4.5.3 (3) [1];Eq. 10.3			
	Crit Crit Eq Los	ical plate bu ical plate bu uivalent Stre ad amplifier te elendeme	cking stress ckling stress ss Mises	σor.c.x σeqv.Ed αult,k	2093.110 362.500 0.952	N/mm <sup>2</sup> N/mm <sup>2</sup>		[1]; Ch. 4.5.3 (3) [1];Eq. 10.3 [1];Eq. 10.3			
	Crit Crit Eq Los Pla	ical plate bu ical plate bu uivalent Stre ad amplifier te slendeme te slendeme	ckling stress ckling stress ss Mises ss for shear	σor,c,x σeqv,Ed αult,k λp	2093.110 362.500 0.952 0.899	N/mm <sup>2</sup> N/mm <sup>2</sup>		[1]; Ch. 4.5.3 (3) [1];Eq. 10.3 [1];Eq. 10.3 [1];Eq. 10.2 [1]:Eq. 10.2			
	Crit Crit Eq Los Pla Pla	ical plate bu ical plate bu uivalent Stre ad amplifier te slendeme te slendeme te buckling	cking stress ckling stress ss Mises iss iss for shear reduction factor	Ger,c,x Geqv,Ed αult,k λp λw	2093.110 362.500 0.952 0.899 0.899	N/mm <sup>2</sup> N/mm <sup>2</sup>	<1	[1]; Ch. 4.5.3 (3) [1];Eq. 10.3 [1];Eq. 10.3 [1];Eq. 10.2 [1];Eq. 10.2 [1];Eq. 10.2			
	Crit Crit Eq Los Pla Pla Pla Pla Re	ical plate bu ical plate bu uivalent Stre ad amplifier te slendeme te slendeme te buckling i duction facti	cking stress ckling stress ss Mises iss ss for shear reduction factor rid ue to column buckline.	Ger.c.x Geqv.Ed αult.k λp λw Pp.x	2093.110 362.500 0.952 0.899 0.899 0.968 0.583	N/mm <sup>2</sup> N/mm <sup>2</sup>	≤1	[1]; Ch. 4.5.3 (3) [1];Eq. 10.3 [1];Eq. 10.3 [1];Eq. 10.2 [1];Eq. 10.2 [1];Ch. 4.4 (2) [2]; Ch. 4.4 (2)			
	Crit Crit Eq Pla Pla Pla Re Sh	ical plate bu ical plate bu uivalent Stre ad amplifier te slendeme te slendeme te buckling i duction facti ear buckling	cking stress ckling stress ss Mises :ss for shear reduction factor or due to column buckling reduction factor	σcr.c.x           σeqv.Ed           αult,k           λp           λw           ρp.x           χc.x           γw	2093.110 362.500 0.952 0.899 0.899 0.968 0.583 0.923	N/mm <sup>2</sup> N/mm <sup>2</sup>	≤1	[1]; Ch. 4.5.3 (3) [1];Eq. 10.3 [1];Eq. 10.3 [1];Eq. 10.2 [1];Eq. 10.2 [1];Ch. 4.4 (2) [2]; Ch. 6.3.1.2 [1];Tab 5.1			
	Crit Crit Eq Pla Pla Pla Re Sh Re	ical plate bu ical plate bu uivalent Stre ad amplifier te slendeme te slendeme te buckling i duction faction ear buckling duction faction	cking stress cking stress ss Mises ss Sonshear reduction factor or due to column bucking reduction factor or	σcr.c.x           σeqv.Ed           αult,k           λp           λw           ρp.x           χc.x           χw           ρc.x	2093.110 362.500 0.952 0.899 0.899 0.968 0.583 0.923 0.583	N/mm <sup>2</sup> N/mm <sup>2</sup>	≤1	(1); Ch. 4.5.3 (3) (1);Eq. 10.3 (1);Eq. 10.3 (1);Eq. 10.2 (1);Eq. 10.2 (1);Eq. 10.2 (1);Eq. 10.2 (1);Eq. 10.2 (1);Eq. 10.3 (1);Eq. 10	-	हिंद्र (इ.	<b>1</b>

Figure 4.3: Window 2.3 Design by Eigenvalues

This results window will be shown if you select the check box *Calculation for all eigenvalues* in the dialog box *Details* (see Figure 3.2, page 19). PLATE-BUCKLING calculates the designs for the selected amount of eigenmodes. Then, the governing load case with the respective design is displayed in Window 2.3.

## 4.4 Design by All

∆1 - Plate buckling analysis ▼	2.4 Desig	n by All										
nnut Data								1				
- General Data		A	В		C	D	I E			F		
Stiffeners	No.	Case	Description		No	Design			Denic	n According to Ex	mula	
Loads	1		Lead Care 1		1	1 777	× 1	104) Interaction according	to Ch. 10. E	a 10.5	inuia	
- LC1 - Load Case 1	2	LUI	Load Case 1			0.922	21	107) Toreional buckling of	etiffenere ac	c to Ch 921(8)		
LC2 - Load Case 2					2	1 719	\$1	104) Interaction according	to Ch. 10 F	a 10.5		
ate Buckling Analysis	4					0.932	<1	107) Torsional buckling of	stiffeners ac	c to Ch 921(8)		
Governing Load Case	5				3	1 391	51	104) Interaction according	to Ch. 10. E	a 10.5		
Design by Load Case	6					0.932	≤1	107) Torsional buckling of	stiffeners ac	c. to Ch. 9.2.1 (8)		
<ul> <li>Design by Eigenvalues</li> </ul>	7				4	1.340	>1	104) Interaction according	to Ch. 10, E	Eq. 10.5		
Design by All	8					0.932	≤1	107) Torsional buckling of	stiffeners ac	c. to Ch. 9.2.1 (8)		
<ul> <li>Critical Buckling Load Factors</li> </ul>	9				5	1.289	>1	104) Interaction according	to Ch. 10, E	Eq. 10.5		
	10					0.932	≤1	107) Torsional buckling of	stiffeners ac	c. to Ch. 9.2.1 (8)		
					Max:	1.777	>1	8				
	E Design Stre Stre Buc Buc	n Ratio ess ratio ess ratio kling load kling load	factor factor	Ψx Ψz α <sub>or,x</sub> α <sub>cr,τ</sub>	-0.881 0.000 1.910 1.514			[1], Tab. 4.1 [1], Tab. 4.1 Detemined using FEM Detemined using FEM				
	Criti	cal load fa	ctor	0.or	1.178			Determined analytically				
	Buc	kling coeff	icient	koc	88.691			Determined using FEM	_			+
	Buc	kling coeff	icient	kτ	28.228			Determined using FEM	=			
	Cnti	cal plate b	ucking stress	σcr.p.x	568.276	N/mm <sup>2</sup>		Determined by FEM	_		+	<b></b>
	Criti	cal plate b	ucking stress	Gor.p. t	180.867	N/mm <sup>2</sup>		Annex A (A. I)	_			
	Eau	cal plate b	ucking stress	0 or.o.x	2093.110	N/mm <sup>2</sup>		[1]; Un. 4.5.3 (3)	_			
	Lon	d amplifier	Coo Mioco	Olegy,Ed	302.000	14/1111-		[1],Eq. 10.3	_			
	Plat	e elendem	000	A n	0.002			[1].Eq. 10.3				
	Plat	e slendem	ess for shear	λ.p. λ.w.	0.000			[1]:Eq. 10.2				
	Plat	e buckling	reduction factor	Onx	0.000		<1	[1]: Ch 44(2)				
	Rec	duction fac	tor due to column buckling	70.8	0.583			[2] Ch 6312				
	She	ar bucklini	reduction factor	2w	0.923			[1]:Tab. 5.1				
	Rec	duction fac	tor	Po,x	0.583			[1]:Eq. 4.13	-		<b>I</b> 🕅	] 🚺 🚺 [

Figure 4.4: Window 2.4 Design by All

### Calculation for all eigenvalues



This window displays an overview of all results with reference to EN 1993-1-5, EN 1993-1-1 and Design of Plated Structures ISBN (ECCS Design Manual): 978-92-9147-100-3 for European Standards or DIN 18800-3 for the German Standard.

According to *DIN 18800-3*, the calculation results are displayed separately for the actions of only one edge stress and to an action due to simultaneous occurrence of all edge stresses.

The designs according to *EN 1993-1-5* include the interaction design of the buckling panel and the other designs that are required for the possibly existing stiffeners.

## 4.5 Critical Buckling Load Factors

File Settings Help							
CA1 - Plate buckling analysis 🔹	2.5 Critical Buckling Load Factor	s					
Input Data		Finenvalue		Critical Buckling I	oad Factors for		
General Data	Load Case	No.	σx [-]	τ[-]	g <sub>7</sub> [-]	Total [-]	
Stiffeners	1 - Load Case 1	1	1 90953	1 51353		1 61016	
🖻 - Loads		2	1.91622	1,70366		1.83806	
LC1 - Load Case 1		3	2.54880	5.37226		2.47366	
- LU2 Load Case 2		4	3.19128	5.44548		2.89536	
Plate Buckling Analysis		5	3.99269	6.67572		3.20605	
- Governing Load Case		6	6.46244	8.14101		3.86984	
Design by Eigenvelves		7	13.76030	14.76870		3.94697	
Design by All		8	14.66430	16.40570		6.41515	
Entire I Rusking Load Factors	2 - Load Case 2	1	81.56790	180.86700	46.98000	29.39140	
chical backing coad ractors		2	86.02140	203.58700	92.90890	52.71950	
		3	146.01400	641.98500	158.03000	87.10880	
		4	270.04500	650.73500	188.62100	100.56900	
		5	340.25900	797.74900	196.91300	132.86800	
		6	361.42800	972.85100	271.39500	142.05800	
		7	435.34000	1764.86000	301.76200	184.69100	
		8	467.82400	1960.48000	432.70000	199.15300	
	Calculate Details		G	raphics			Car

Figure 4.5: Window 2.5 Critical Buckling Load Factors

The last result window displays the critical buckling load factors resulting from  $\sigma_x$ ,  $\tau$  and  $\sigma_z$  ( $\sigma_y$ ) for all load cases. They are listed by action for all buckling shapes.



## 5. **Results Evaluation**

You can evaluate the design results in various manners.

## 5.1 Results Windows

The buttons at the end of the upper table facilitate the evaluation in the results windows.



Figure 5.1: Buttons for results evaluation

The buttons are reserved for the following functions:

Butt	ton	Description	Function
L		Show Color Bars	Turns on and off the colored reference scales in the results tables
<b>7</b> >1		Exceeding	Displays only the rows where the ratio is greater than 1, and thus the design is failed

Table 5.1: Buttons in results windows 2.1 to 2.5



## 5.2 Visualization of Buckling Shapes

To display the buckling shapes graphically, click [Graphics]. To this end, a PLATE-BUCKLING window opens.



Figure 5.2: Graphic Buckling mode

In this window, you can visualize different actions on the panel, different load cases and buckling shapes. For load cases that are not calculated yet, the loads on the panel are shown. The graphical representation allows for a quick check of the buckling shapes and load data.

The graphic can be controlled by using the drop-down menu or the functions of the toolbar. The buttons shown on the left allow you to view the buckling panel from different angles.

The grab button is a special feature: By left-clicking and pressing the [Shift] key simultaneously while moving the mouse up or down, you can zoom the view in or out. By left-clicking and pressing the [Ctrl] key at the same time, you can rotate the view. You can "play" with this function in order to better understand how it works.

The field [Factor] allows you to show small deformations as elevated.

The [Animation] of the buckling shapes often helps to understand the buckling behavior of the stiffened plates.



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Graphics



## 6. Printout

### 6.1 Printout report

First, the program generates a printout report for the PLATE-BUCKLING results, to which graphics and descriptions can be added. In the printout report, you can select the data to be included in the printout.



The printout report is described in detail in the RSTAB or RFEM manual [3]. In particular, chapter 10.1.3.4 *Selecting Data of Add-on Modules* provides information on the selection of input and output data in add-on modules for the printout.

For large structural systems with many design cases, it is recommended to split the data into several printout reports, thus allowing for a clearly-arranged printout.

## 6.2 PLATE-BUCKLING Graphic Printout

Graphics

13

In PLATE-BUCKLING, every picture that is displayed in the program's work window can be transferred to the printout report or sent directly to a printer.

Printing graphics is described in [3], chapter 10.2.

### **PLATE-BUCKLING model with loads**

To print the current PLATE-BUCKLING graphic, click

### File ightarrow Print Graphic

or the respective button in the toolbar.



Figure 6.1: Button *Print* in the toolbar of the main window

\_\_\_ Dlubal

The following dialog box appears:

iraphic Printout		×
General Options		
Graphic Picture         Directly to a printer         Image: To a printout report:         PR1         To the Clipboard	Window To Print Current only More Mass print	Graphic Size <ul> <li>As screen view</li> <li>Window filling</li> <li>To scale 1: 100 ▼</li> </ul>
Graphic Picture Size and Rotation ✓ Use whole page width ✓ Use whole page height ④ Height: 50 ← [% of page]	Options Show results for selected x diagram Lock graphic picture (witho	clocation in result put update)
Rotation: 0 _ [*] Header of Graphic Picture Panel with Edge Stresses		
		OK 🔍

Figure 6.2: Dialog box Graphic Printout, tab General

This dialog box is described in [3], chapter 10.2. The RSTAB manual also describes the *Options* and *Color Spectrum* tab.

A graphic can be moved anywhere within the printout report by using the drag-and-drop function.

To adjust a graphic subsequently in the printout report, right-click the relevant entry in the navigator of the printout report. The option *Properties* in the context menu opens the dialog box *Graphic Printout*, offering various options for adjustment.

iraphic Printout	×
Properties Options Color Spectrum	
Header	
Panel with Edge Stresses	
Additional Comment	
Show	
Graphic Picture Size	Options
Use whole page width	Lock graphic picture (without update)
V Use whole page width Height: 50 + [% of page]	Cock graphic picture (without update)  Rotation:
Vie whole page width Height: 50 🔶 [% of page]	Lock graphic picture (without update) Rotation:
Use whole page width Height: 50 + [% of page]	Cock graphic picture (without update) Rotation:

Figure 6.3: Dialog box Graphic Printout, tab Options

Remove from Printout Report Start with New Page	
Selection	
Properties	



## 7. General Functions

The final chapter describes useful menu functions as well as export options for the designs.

## 7.1 Design Cases

With the design cases you can, for example, group the buckling panels from the model or check variants.

### **Create a New Design Case**

To create a new design case, use the PLATE-BUCKLING menu and click

```
\textbf{File} \rightarrow \textbf{New Case}.
```

The following dialog box appears:

New PLAT	E-BUCKLING-Case
No.	Description Plate buckling analysis
D	OK Cancel

Figure 7.1: Dialog box New PLATE-BUCKLING-Case

In this dialog box, enter a *No*. (one that is still available) for the new design case. The corresponding *Description* will make the selection in the load case list easier.

Click [OK] to open the PLATE-BUCKLING Window 1.1 *General Data* where you can enter the design data.

### **Rename a Design Case**

To change the description of a design case, use the PLATE-BUCKLING menu and click

#### $\textbf{File} \rightarrow \textbf{Rename Case}.$

The following dialog box appears:

Rename Pl	LATE-BUCKLING-Case
No.	Description Plate buckling analysis
٢	OK Cancel

Figure 7.2: Dialog box Rename PLATE-BUCKLING-Case

In this dialog box, you can define a different *Description* as well as a different *No*. for the design case.



### Copy a Design Case

To copy the input data of the current design case, use the PLATE-BUCKLING menu and click

```
File 
ightarrow Copy Case.
```

The following dialog box appears:

Copy PLA	TE-BUCKLING-Case	X
Copy fro	m Case	
CA1 - PI	ate buckling analysis	<b>•</b>
New Cas	e	
No.:	Description:	
2		•
		OK Cancel

Figure 7.3: Dialog box Copy PLATE-BUCKLING-Case

Define the No. and, if necessary, a Description for the new case.

### **Delete a Design Case**

To delete design cases, use the PLATE-BUCKLING menu and click

```
\textbf{File} \rightarrow \textbf{Delete Case}.
```

The following dialog box appears:

elete C	ases	23
Availat	ole Cases	
No.	Description	-
1	Plate buckling analysis	
		-
		_
۲		-
9		31

Figure 7.4: Dialog box Delete Case

The design case can be selected in the list *Available Cases*. To delete the selected case, click [OK].



## 7.2 Units and Decimal Places

Units and Decimal Places are managed in one dialog box for RFEM/RSTAB and the add-on modules. In PLATE-BUCKLING, you can use the menu to define the units. To open the corresponding dialog box, click on the menu

#### Settings $\rightarrow$ Units and Decimal Places.

The program opens the following dialog box that you already know from RSTAB or RFEM. The module PLATE-BUCKLING is preset.

Units and Decimal Places					×
Program / Module	PLATE-BUCKLING				
RSTAB	Character			Oritical Durables Load Fee	1
STEEL	Stresses			Critical Buckling Load Fac	tor
STEEL EC3		Unit	Dec. places		Unit Dec. places
STEEL AISC	Stresses:	N/mm^2 👻	2 🌲	Factors:	% 👻 5 荣
	Stress ratios:	97 -	3 📥		
STEEL BS	Chess raise.	/o *			
-STEEL GB					
STEEL CS	Dimensions			Stiffener Values	
ALUMINIUM	Panels:	mm 🔻	1 ≑	Cross-sectional area:	
KAPPA	Cross soctions:		1	Second memory of areas	
LTB	Cross-sections.	•		Second moment of alea.	
FE-LTB				Warping constant:	cm^6 ▼ 2
EL-PL	Elastic Foundations				
С-ТО-Т	Forces:	kN 🔻	3 ≑		
PLATE-BUCKLING	Lengths for moments:		3 📥		
CONCRETE	Longina for momenta.				
TIMBER Pro	Lengths:	m 🔻	3 ≑		
TIMBER	Angles:	rad 🔻	3 🌩		
DYNAM					
JOINTS					
FRAME-JOINT Pro					
FRAME-JOINT					
DOWEL					
FOUNDATION					
FOUNDATION Pro					
HSBUCK					
	۲				
					OK Cancel

Figure 7.5: Dialog box Units and Decimal Places



The settings can be saved as user profile und reused in other models. These functions are described in [3], chapter 11.1.3.

## 7.3 Export of Results

The PLATE-BUCKLING results can also be used in other programs.

### Clipboard

To copy cells selected in the results tables to the Clipboard, press the keys [Ctrl]+[C]. To insert the cells, for example in a word-processing program, press [Ctrl]+[V]. The headers of the table columns will not be transferred.

### **Printout report**

The data of the PLATE-BUCKLING add-on module can be printed into the global printout report (see chapter 6.1, page 29) to export them subsequently. Then, in the printout report, click

 $\textbf{File} \rightarrow \textbf{Export to RTF}.$ 

This function is described in [3], chapter 10.1.11.



### **Excel / OpenOffice**

PLATE-BUCKLING provides a function for the direct data export to MS Excel, OpenOffice.org Calc, or the file format CSV. To open the corresponding dialog box, click

```
\textbf{File} \rightarrow \textbf{Export Tables}.
```

The following export dialog box appears.

export - MS Excel	×
Table Parameters	Application
📝 With table header	Microsoft Excel
Only marked rows	OpenOffice.org Calc
	CSV file format
Transfer Parameters	
Export table to active workbook	
Export table to active worksheet	
🔽 Rewrite existing worksheet	
Selected Tables	
<ul> <li>Active table</li> </ul>	Export tables with details
All tables	
🔽 Input tables	
Result tables	
1	

Figure 7.6: Dialog box Export - MS Excel

When you have selected the relevant parameters, you can start the export by clicking [OK]. Excel or OpenOffice will be started automatically, that is, the programs do not have to be opened first.

А	В	С	D	E	F	G
	Load		Eigenvalue	Design		
No.	Case	Description	No.	Ratio		Design According to Formula
1	LC1		1	0,981	≤1	104) Interaction according to Ch. 10, Eq. 10.5
2				0,932	≤1	107) Torsional buckling of stiffeners acc. to Ch. 9.2.1 (8)
з			2	0,977	≤1	104) Interaction according to Ch. 10, Eq. 10.5
4				0,932	≤1	107) Torsional buckling of stiffeners acc. to Ch. 9.2.1 (8)
5			3	0,877	≤1	104) Interaction according to Ch. 10, Eq. 10.5
6				0,932	≤1	107) Torsional buckling of stiffeners acc. to Ch. 9.2.1 (8)
7			4	0,828	≤1	104) Interaction according to Ch. 10, Eq. 10.5
8				0,932	≤1	107) Torsional buckling of stiffeners acc. to Ch. 9.2.1 (8)
9			5	0,788	≤1	104) Interaction according to Ch. 10, Eq. 10.5
10				0,932	≤1	107) Torsional buckling of stiffeners acc. to Ch. 9.2.1 (8)
11			6	0,728	≤1	104) Interaction according to Ch. 10, Eq. 10.5

Figure 7.7: Result in Excel



## 8. Theoretical Background

### 8.1 DIN 18800-3

Buckling means that plane thin-walled plates whose plate thickness t is significantly smaller than the surface geometry  $a \cdot b$  and that are subjected to normal and shear stresses deflect perpendicular to the plate plane. Rectangular plates prone to buckling are called buckling panels.

When you analyze a buckling problem, you must consider the plate's states of stress and deformation. For this, you have to consider the following parameters:

- Position of the web and flange zones most prone to buckling
- Dimensions of the buckling panels
- Supports of the buckling panel edges
- Loading in the form of the stresses acting upon the edge surfaces

The program PLATE-BUCKLING is based on the finite element method and can be used to determine critical buckling load factors. The following is assumed for the calculation (linear buckling analysis):

- At the beginning of the loading, the plate is completely plane.
- The buckling deformations rectangular to the plate plane are small.
- The loading acts only on the plate's center plane.
- The material is assumed to behave in an ideal linear elastic way.

With these assumptions for plate buckling, a bifurcation problem arises. Linear buckling analysis is only used to determine a plate slenderness ratio. The reduction factors  $\kappa$  required for plate buckling analysis depend on the plate slenderness.

### 8.1.1 Terms and Definitions

### **Critical plate buckling stress**

Under this loading, the plate can still remain in its original position. If the loading is increased further, the plate buckles.

 $\sigma_{xPi} = k\sigma_x * \sigma_E$  Critical buckling stress with sole action of edge stresses  $\sigma_x$ 

$$\sigma_{\mathsf{E}} = \frac{\pi^2 \cdot \mathsf{E}}{12 \cdot (1 - \mu^2)} \cdot \left(\frac{t}{b}\right)^2 \qquad \text{Euler's critical stress}$$

With these input values, you can determine the critical plate buckling stress for the sole action of  $\sigma_x$ ,  $\sigma_x$  and  $\tau$ . The smallest critical plate buckling stress and, therefore, the smallest buckling value are governing for buckling. The buckling value and thus the critical plate buckling stress depend on the following influences:

- Boundary conditions (support conditions)
- Type of action
- Side ratio α
- Type and position of stiffeners

In PLATE-BUCKLING, the buckling values are usually determined by using the FE method, solving the eigenvalue problem.



### **Critical plate buckling stress**

With reference to the linear buckling analysis, the reduction factors  $\kappa$  for the critical plate buckling stress are determined in relation to the panel dimensions, the support and load conditions, as well as the plate slenderness ratio. The following factors also influence the calculation of the critical plate buckling stress:

- Structural components without local buckling effects (pure bending)
- Structural components with local buckling effects (bending beams with compressive force / compression columns with bending moments)
- Plates without local buckling behavior
- Plates with local buckling behavior

#### Critical plate buckling stress without buckling effects

The critical plate buckling stresses are determined according to the following equations.

 $\sigma_{xP,R,d} = \kappa_x \cdot f_{y,k} / \gamma_M$  $\sigma_{yP,R,d} = \kappa_y \cdot f_{y,k} / \gamma_M$  $\tau_{P,R,d} = \kappa_\tau \cdot f_{y,k} / (\sqrt{3} \cdot \gamma_M)$ 

#### Critical plate buckling stresses with buckling effects

If the buckling panel is part of a compression member, the reciprocal influence of plate buckling and local buckling must be considered. This is achieved by reducing the critical plate buckling stress with the reduction factor  $\kappa_{\kappa}$  for local buckling.

 $\sigma_{xP,R,d} = \kappa_{K} \cdot \kappa_{x} \cdot f_{y,k} \, / \, \gamma_{M}$ 

To determine  $\kappa_{\kappa}$  see DIN 18 800 part 2, el. (304) equation (4a) - (4c).

### Critical plate buckling stress with local buckling behavior

Local buckling behavior is found in a plate which is pushed in longitudinal direction and that has too small a side ratio. Then, a support of the central plate zones on the plate edges is not given. Thus, the plate, like buckling members - has no more supercritical load capacity and has to be classified between the failure modes *global buckling* and *local buckling*. This is done by using the weighting factor  $\rho$ . If weighting factor  $\rho > 0$ , the critical plate buckling stress must be determined by using the reduction factor  $\kappa_{PK}$ .

$$\rho = \frac{\Lambda - \sigma_{xPi} \, / \, \sigma_{xKi}}{\Lambda - 1} \ge 0$$

In PLATE-BUCKLING,  $\sigma_{xki}$  is determined analytically. It is the Euler buckling stress of the buckling panel with unsupported longitudinal edges. For the reduction factor for local buckling behavior, according to DIN 18 800, part 3, element (603):

$$\kappa_{\mathsf{PK}} = \left(1 - \rho^2\right) \cdot \kappa_{\sigma} + \rho^2 \cdot \kappa_{\mathsf{K}}$$

The reduction factor  $\kappa_{\kappa}$  is determined according to DIN 18 800 part 2, Eq. (4a), (4b), or (4c) according to buckling curve b.

The critical plate buckling stresses are calculated according to DIN 18 800, part 3, element (502) as follows.

$$\begin{split} \sigma_{P,R,d} = & \frac{\kappa_{PK} \cdot f_{y,k}}{\gamma_M} \leq 1 \\ \tau_{P,R,d} = & \frac{\kappa_\tau \cdot f_{y,k}}{\gamma_M \cdot \sqrt{3}} \leq 1 \end{split}$$



### Interaction condition

If several stress components  $\sigma_x$ ,  $\sigma_x$ , and  $\tau$  act simultaneously, you must perform the interaction design. These are always stresses that are assigned to each other.

Deviating from this provision, the maximum value is imported from every stress type when you import these stresses from RSTAB or RFEM,

$$\begin{aligned} \mathbf{e}_{1} &= 1 + \kappa_{\sigma_{x}}^{4} \\ \mathbf{e}_{2} &= 1 + \kappa_{\sigma_{y}}^{4} \\ \mathbf{e}_{3} &= 1 + \kappa_{\sigma_{x}} \cdot \kappa_{\sigma_{y}} \cdot \kappa_{\tau}^{2} \\ \mathbf{V} &= (\kappa_{x} \cdot \kappa_{y})^{6} \\ &\left(\frac{\left|\sigma_{x}\right|}{\sigma_{x^{P,R,d}}}\right)^{e_{1}} + \left(\frac{\left|\sigma_{y}\right|}{\sigma_{y^{P,R,d}}}\right)^{e_{2}} - \mathbf{V}\left[\frac{\left|\sigma_{x} \cdot \sigma_{y}\right|}{\sigma_{x^{P,R,d}} \cdot \sigma_{y^{P,R,d}}}\right] + \left(\frac{\tau}{\tau_{P,R,d}}\right)^{e_{3}} \leq 1 \end{aligned}$$

### 8.2 EN 1993-1-5

For every plate buckling analysis according to EN 1993-1-5, the reduced stresses are implemented in PLATE-BUCKLING. The Eurocode offers two methods for the plate buckling analysis.

- Effective width method (EN 1993-1-5, chapter 4-7)
- Reduced stress method (EN 1993-1-5, chapter 10)

The method of reduced stresses compares the stresses acting on the buckling panel with a limit stress condition reduced accounted for the VON MISES yield condition. The plate buckling analysis is performed on the basis of the entire stress field. This approach corresponds with the one from DIN 18800-3, however, with the significant difference that in EN 1993-1-5 a single global slenderness ratio is determined on the basis of the entire stress field. Thus, the analysis of the single loading and the subsequent merging via the interaction criterion is omitted.

As the determination of buckling values is based on numerical calculation in EN 1993-1-5, in PLATE-BUCKLING the input parameters can easily be determined by using the eigenvalue method.

In EN 1993-1-5, chapter 9, the designs for the possibly existing stiffeners are still required. For longitudinal and transverse stiffeners, the following analyses must be performed successfully:

- Elastic stress analysis with internal forces according to second order analysis
- Deformation analysis
- Torsional buckling analysis

The design procedure is described in detail in [18].



### 8.2.1 Determination of Critical Plate Buckling Stresses

The Annex to EN 1993-1-5 provides analytical formulas for calculation of the critical buckling stresses of unstiffened and stiffened buckling plates. In general, the following applies:

 $\sigma_{cr,p} = k_{cr,p} * \sigma_e$  Annex A, (A.1)

For **unstiffened buckling plates**, the buckling values are computed according to Table 4.1 or Table 4.2 based on the existing edge stress condition.

Buckling plates supported on two sides	Table 4.1
--	-----------

Buckling plates supported on one side
 Table 4.2

For **stiffened buckling plates**, the following variants of the arrangement of the stiffeners within the buckling plate are distinguished:

<ul> <li>Three or more longitudinal stiffeners in buckling panel</li> </ul>	Annexes (A.1), (A.3)
One longitudinal stiffener in compression zone of buckling panel	Annexes (A.1), (A.2.2.2), (A.3)
• Two longitudinal stiffeners in compression zone of buckling panel	Annexes (A.1), (A.2.2.1), (A.3)

These formulas or their application are bound to provisions that must be considered. If these conditions are not met, PLATE-BUCKLING automatically calculates the critical plate buckling stresses by using the eigenvalue method.

### 8.2.2 Interaction Design

The stresses acting on the buckling panel are compared with a limit stress state reduced taking into account the VON MISES yield condition. In EN 1993-1-5, only a single global slenderness ratio is determined on the basis of the entire stress field. The following parameters are relevant for the interaction design:

#### Slenderness ratio of sheet metal plate

$$\overline{\lambda}_{p} = \sqrt{\frac{\alpha_{\text{ult,k}}}{\alpha_{\text{cr}}}} \qquad \qquad Eq. 10.2$$

$$\frac{1}{\alpha_{ult,k}} = \left(\frac{\sigma_{x,Ed}}{f_y}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y}\right) \cdot \left(\frac{\sigma_{z,Ed}}{f_y}\right) + 3 \cdot \left(\frac{\tau_{Ed}}{f_y}\right)^2$$
 Eq. 10.3

$$\frac{1}{\alpha_{\rm cr}} = \frac{1+\psi_x}{4\cdot\alpha_{\rm cr,x}} + \frac{1+\psi_z}{4\cdot\alpha_{\rm cr,z}} + \left[ \left(\frac{1+\psi_x}{4\cdot\alpha_{\rm cr,x}} + \frac{1+\psi_z}{4\cdot\alpha_{\rm cr,z}}\right)^2 + \frac{1-\psi_x}{2\cdot\alpha^2_{\rm cr,x}} + \frac{1-\psi_z}{2\cdot\alpha^2_{\rm cr,z}} + \frac{1}{\alpha^2_{\rm cr,\tau}} \right]^{0.5} \qquad Eq. \ 10.6$$

#### Reduction factors p<sub>i</sub>

There are two possible approaches to determine the reduction factors: The individual factors can be calculated for every existing stress component. Alternatively, a single reduction factor is determined globally for all existing stresses. This approach is related to the selection of the corresponding buckling curve.

For various buckling curves, the reduction factors are as follows:

Reduction factor for x-direction	$ ho_{\rm x}$ according to chapter 4.4 (2)
Reduction factor for z-direction	$ ho_z$ according to Annex B.1
Reduction factor for shear buckling	$\chi_w$ according to Table 5.1
If you use the <i>generalized</i> buckling curve:	
Reduction factor	$ ho_i$ according to Annex B.1



### Interaction between plate and local buckling behavior

To obtain the final reduction factor of the respective direction, an interaction between plate type and local buckling behavior must be determined.

 $\rho_{i} = (\rho - \chi_{c}) \cdot \xi \cdot (2 \cdot \xi) + \chi_{c}$ 

according to Eq. 4.13

where:  $\chi_c$  according to chapter 4.5.3 (5)

 $\xi$  according to chapter 4.5.4 (1)

Interaction criterion according to Eq. 10.5:

$$\left(\frac{\sigma_{x,Ed}}{\rho_x \cdot f_y / \gamma_{M1}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{\rho_z \cdot f_y / \gamma_{M1}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{\rho_x \cdot f_y / \gamma_{M1}}\right) \cdot \left(\frac{\sigma_{z,Ed}}{\rho_z \cdot f_y / \gamma_{M1}}\right) + 3 \cdot \left(\frac{\tau_{Ed}}{\chi_w \cdot f_y / \gamma_{M1}}\right)^2$$

### 8.2.3 Elastic Stress Design of the Stiffeners

EN 1993-1-5 requires an elastic-elastic stress design for transverse stiffeners according to second order analysis. Here, you must consider a precamber of the transversal stiffener, the skew loads on the adjacent partial panels as well as the stress on the buckling panel. The existing stresses may not exceed the yield stress  $f_y/y_{M1}$ .

These formulas are mainly analytical. They are described in detail in [18].

### 8.2.4 Deformation Analysis of the Stiffeners

The maximum deformation of the stiffener is limited to the value b/300. Here, you have to consider that the deformation results from the precamber and the actual deformation of the stiffener.

These formulas are also described in [18].

### 8.2.5 Torsional Buckling Design

To avoid torsional buckling of transverse and longitudinal stiffeners with open cross-sections, the following designs must be carried out successfully:

$$\frac{J}{I_p} \ge 5.3 \cdot \frac{f_y}{E}$$

according to Eq. 9.3

where: J St. Venant torsional constant stiffener cross-section

*I<sub>p</sub>* Polar moment of inertia

If the warping stiffness of the stiffener is considered, Eq. 9.3 or Eq. 9.4 should be satisfied.

 $\sigma_{cr} \geq \theta \cdot f_y$ 

according to Eq. 9.4

where:  $\theta$  Factor used to ensure elastic behavior ( $\theta = 6$ )



## 9. Examples

9.1 DIN 18800

9.1.1 Unstiffened Buckling Panel with Local Buckling Behavior

Material:	Steel S	Steel St 37			
	Yield	strength f <sub>y</sub>	$k_{k} = 240 \text{ N/mm}^{2}$		
Partial safety factor:	γ <sub>M</sub> = 1	γ <sub>M</sub> = 1,1			
Parameters of the structural sys	stem:				
Length of the buckling panel	а	=	1000 mm		
Width of the buckling panel	Ь	=	1200 mm		
Plate thickness	t	=	10 mm		
$\rightarrow$ Side ratio	$\alpha = \frac{a}{b} = \frac{1000}{1200}$	- = 0.833			
Governing stresses:					
Axial compressive stress	$\sigma_1$	=	80 N/mm <sup>2</sup>		
Axial compressive stress	$\sigma_2$	=	80 N/mm <sup>2</sup>		
Shear stress	τ	=	12 N/mm <sup>2</sup>		

Edge stress ratio related to the maximum compression stress:

$$\psi = \frac{\sigma_2}{\sigma_1} = \frac{80}{80} = 1.0$$

Euler critical stress:

$$\sigma_{\rm E} = \frac{\pi^2 \cdot {\rm E}}{12 \cdot (1 - 0.3^2)} \cdot \left(\frac{{\rm t}}{{\rm b}}\right)^2 = \frac{3.14^2 \cdot 210000}{12 \cdot (1 - 0.3^2)} \cdot \left(\frac{10}{1200}\right)^2 = 13.18 \, {\rm N/mm^2}$$

Calculation of the buckling values according to DIN 4114, Table 6, rows 3 and 5:

$$\alpha = 0.833 < 1 \text{ and } \psi = 1$$

$$k_{\sigma} = \left(\alpha + \frac{1}{\alpha}\right)^{2} \cdot \frac{2,1}{\psi + 1.1} = \left(0.833 + \frac{1}{0.833}\right)^{2} \cdot \frac{2,1}{1 + 1.1} = 4.4134$$

$$k_{\tau} = 4.00 + \frac{5.34}{\alpha^{2}} = 4,00 + \frac{5.34}{0.833^{2}} = 11.69$$

Critical plate buckling stress if edge stresses  $\sigma$  according to DIN 18 800 part 3, El. (113) are effective:

 $\sigma_{Pi} = k_{\sigma} \cdot \sigma_{E} = 4.13 \cdot 13.18 = 54.43 \text{ N/mm}^{2}$ 

Critical plate buckling stress if edge stresses  $\tau$  are effective:

 $\tau_{Pi} = k_{\tau} \cdot \sigma_{E} = 11,69 \cdot 13,18 = 154.07 \, \text{N/mm}^2$ 

**Reference slenderness ratio:** 

$$\lambda_a = \pi \cdot \sqrt{\frac{E}{f_{y,k}}} = 3.14 \cdot \sqrt{\frac{210000}{240}} = 92.93$$

Plate slenderness (axial stress):

$$\lambda_{\rm P} = \pi \cdot \sqrt{\frac{E}{\sigma_{\rm Pi}}} = 3.14 \cdot \sqrt{\frac{210000}{54.49}} = 195.03$$

Plate slenderness (shear stress):

$$\lambda_{p} = \pi \cdot \sqrt{\frac{E}{\tau_{Pi} \cdot \sqrt{3}}} = 3,14 \cdot \sqrt{\frac{210000}{154.07 \cdot \sqrt{3}}} = 88.13$$

Plate slenderness ratio (axial stress) according to DIN 18 800 Part 3, El. (113):

$$\overline{\lambda}_{p} = \frac{\lambda_{p}}{\lambda_{a}} = \frac{195.03}{92.93} = 2.098$$

Plate slenderness ratio:

$$\overline{\lambda}_{\rm P} = \frac{\lambda_{\rm P}}{\lambda_{\rm a}} = \frac{88.13}{92.93} = 0.948$$

Reduction factor for plate buckling according to DIN 18 800 Part 3, Table 1:

$$\kappa_{\sigma} = c \cdot \left( \frac{1}{\overline{\lambda}_{p}} - \frac{0.22}{\overline{\lambda}_{p}^{2}} \right) \text{ where } c = 1.25 - 0.25 \cdot \psi, \text{ but } c \le 1.25$$
$$c = 1.25 - 0.25 \cdot 1.0 = 1.00$$
$$\kappa_{\sigma} = 1.0 \cdot \left( \frac{1}{2.098} - \frac{0.22}{2.098^{2}} \right) = 0.427$$
$$\kappa_{\tau} = \frac{0.84}{\overline{\lambda}_{p}} = \frac{0.84}{0.948} = 0.886$$

Plate buckling with local buckling behavior according to DIN 18 800 Part 3, El. (602):

$$\rho = \frac{\Lambda - \sigma_{\text{Pi}} / \sigma_{\text{Ki}}}{\Lambda - 1} \ge 0 \qquad \Lambda = \overline{\lambda} \rho^{2} + 0.5, \text{ but } 2 \le \Lambda \le 4$$
$$\sigma_{\text{Pi}} / \sigma_{\text{Ki}} = k_{\sigma} \cdot \alpha^{2} = 4.134 \cdot 0.833^{2} = 2.838$$
$$\Lambda = 2.098^{2} + 0.5 = 4.902 > 4 \Longrightarrow \Lambda = 4$$
$$\rho = \frac{4 - 2.868}{4 - 1} = 0.377$$



According to DIN 18 800 Part 3, El. (603):

$$\begin{split} \lambda_{p} > 0.2 & \Rightarrow \kappa_{K} = \frac{1}{\left(k + \sqrt{k^{2} - \lambda_{p}^{2}}\right)} \\ k = 0.5 \cdot \left[1 + 0.34 \cdot (\lambda_{p} - 0.2) + \lambda_{p}^{2}\right] \\ k = 0.5 \cdot \left[1 + 0.34 \cdot (2.098 - 0.2) + 2.098^{2}\right] = 3.023 \\ \kappa_{K} = \frac{1}{\left(3.023 + \sqrt{3.023^{2} - 2.098^{2}}\right)} = 0.192 \end{split}$$

Reduction factor with local buckling behavior

$$\kappa_{PK} = (1 - \rho^2) \cdot \kappa_{\sigma} + \rho^2 \cdot \kappa_{K} = (1 - 0.377^2) \cdot 0.427 + 0.377 \cdot 0.192 = 0.393$$

Calculation of critical plate buckling stresses according to DIN 18 800 Part 3, El. (502):

$$\sigma_{P,R,d} = \frac{\kappa_{PK} \cdot f_{y,k}}{\gamma_M} = \frac{0.393 \cdot 240}{1.1} = 85.88 \,\text{N/mm}^2$$
$$\tau_{P,R,d} = \frac{\kappa_\tau \cdot f_{y,k}}{\gamma_M \cdot \sqrt{3}} = \frac{0.886 \cdot 240}{1.1 \cdot \sqrt{3}} = 111.5 \,\text{N/mm}^2$$

Analysis of the buckling panel according to DIN 18 800 Part 3, El. (501):

$$\frac{\sigma}{\sigma_{P,R,d}} = \frac{80}{85.88} = 0.931 < 1$$
$$\frac{\tau}{\tau_{P,R,d}} = \frac{12}{111.5} = 0.107 < 1$$

Analysis for simultaneous occurrence of edge stresses (interaction) according to DIN 18 800 Part 3, El. (504):

$$e_{1} = 1 + \kappa_{\sigma}^{4} = 1 + 0.393^{4} = 1.023$$

$$e_{3} = 1 + \kappa_{\sigma} \cdot \kappa_{\tau}^{2} = 1 + 0.393 \cdot 0.886^{2} = 1.308$$

$$\left(\frac{\sigma}{\sigma_{P,R,d}}\right)^{e_{1}} + \left(\frac{\tau}{\tau_{P,R,d}}\right)^{e_{3}} \le 1$$

$$\left(\frac{80}{85,82}\right)^{1,023} + \left(\frac{12}{111,5}\right)^{1,308} = 0.984 \le 1$$

The plate buckling safety is sufficient!



### 9.1.2 Stiffened Panel

The following example is extracted from [11]. It describes the plate buckling analysis for a stiffened buckling panel.



Figure 9.1: Sketch of structural system including dimensions and loads

Material:		Steel St 37				
	Yield sti	rength f <sub>y,k</sub>	= 240 N/mm <sup>2</sup>			
Partial safety factor:	$\gamma_M = 1.1$	$\gamma_M = 1.1$				
Parameters of the structural system:						
Length of the buckling panel	а	=	2500 mm			
Width of the buckling panel	Ь	=	1940 mm			
Plate thickness	t	=	12 mm			
$\rightarrow$ Side ratio $\alpha = \frac{a}{b}$	$=\frac{2500}{1940}$	= 1.29				
Stiffener:						
Height:	h	=	150 mm			
Length:	а	=	2500 mm			
Thickness:	t	=	12 mm			
Stiffener position:	у	=	485 mm (from upper edge)			
Stiffener parameters::	<b>I</b> y,Stiffener	=	3040 cm <sup>4</sup>			
	<b>A</b> y,Stiffener	=	36 cm <sup>2</sup>			
	$\delta = 0.15$	5	acc. to DIN 18800, part 3, el. (114)			
	$\gamma = 99$		acc. to DIN 18800 part 3, el. (114)			
We obtain from these initial values according to [13]:						
	k <sub>ox</sub>	=	84			
	kτ	=	12			



#### **Governing stresses:**

Axial compressive stress	$\sigma_1$	=	130 N/mm <sup>2</sup>
Axial tensile stress	$\sigma_2$	=	- 130 N/mm²
Shear stress	τ	=	52 N/mm <sup>2</sup>

Edge stress ratio related to maximum compressive stress:

$$\Psi = \frac{\sigma_2}{\sigma_1} = \frac{13}{-13} = -1.0$$

**Euler critical stress:** 

$$\sigma_{\rm E} = \frac{\pi^2 \cdot {\rm E}}{12 \cdot (1 - 0.3^2)} \cdot \left(\frac{t}{b}\right) = \frac{3.14^2 \cdot 210000}{12 \cdot (1 - 0.3^2)} \cdot \left(\frac{1.2}{194}\right)^2 = 0.73 \, \text{kN/cm}^2$$

Calculation of the buckling values according to DIN 4114, Table 6, rows 3 and 5:

 $\alpha = 1.29 > 1$  and  $\psi = -1$ 

Critical plate buckling stress if edge stresses  $\sigma$  according to DIN 18 800 Part 3, El. (113) are effective:

 $\sigma_{Pi_{x}} = k_{\sigma} \cdot \sigma_{E} = 84 \cdot 0.73 = 61.3 \, kN/cm^{2}$ 

Critical plate buckling stress if edge stresses  $\tau$  are effective:

 $\tau_{Pi} = k_{\tau} \cdot \sigma_{E} = 12 \cdot 0.73 = 8.8 \text{ kN/cm}^{2}$ 

Related slenderness ratio:

$$\lambda_a = \pi \cdot \sqrt{\frac{E}{f_{y,k}}} = 3,14 \cdot \sqrt{\frac{210000}{240}} = 92.93$$

Plate slenderness ratio (axial stress):

$$\lambda_{P_{\sigma_x}} = \pi \cdot \sqrt{\frac{E}{\sigma_{Pi}}} = 3.14 \cdot \sqrt{\frac{21000}{61.3}} = 58.12$$

Plate slenderness ratio (shear stress):

$$\lambda_{p} = \pi \cdot \sqrt{\frac{E}{\tau_{p_{i}} \cdot \sqrt{3}}} = 3,14 \cdot \sqrt{\frac{21000}{8.8 \cdot \sqrt{3}}} = 116.55$$

Relative plate slenderness ratio (axial stress) according to DIN 18 800 Part 3, El. (113):

$$\overline{\lambda}_{P_{\sigma_{x}}} = \frac{\lambda_{p}}{\lambda_{a}} = \frac{58.12}{92.93} = 0.625$$

Relative plate slenderness ratio (shear stress):

$$\overline{\lambda}_{\rm P} = \frac{\lambda_{\rm P}}{\lambda_{\rm a}} = \frac{116.55}{92.93} = 1.254 < 1.38$$



Reduction factor for plate buckling according to DIN 18 800 Part 3, Table 1:

$$\begin{aligned} \kappa_{\sigma_{x}} &= c \cdot \left( \frac{1}{\overline{\lambda_{p}}} - \frac{0.22}{\overline{\lambda_{p}}^{2}} \right) \quad \text{where } c = 1.25 - 0.25 \cdot \psi, \text{but } c \leq 1.25 \\ c &= 1.25 - 0.25 \cdot -1.0 = 1.50 = > 1.25 \\ \kappa_{\sigma_{x}} &= 1.25 \cdot \left( \frac{1}{0.625} - \frac{0.22}{0.625^{2}} \right) = 1.296 = > 1.0 \\ \kappa_{\tau} &= \frac{0.84}{\overline{\lambda_{p}}} = \frac{0.84}{1.255} = 0.669 \end{aligned}$$

Calculation of critical plate buckling stresses according to DIN 18 800 Part 3, El. (502):

$$\sigma_{xP,R,d} = \frac{\kappa \cdot f_{y,k}}{\gamma_M} = \frac{1.0 \cdot 240}{1.1} = 218 \,\text{N/mm}^2$$
$$\tau_{P,R,d} = \frac{\kappa_\tau \cdot f_{y,k}}{\gamma_M \cdot \sqrt{3}} = \frac{0.669 \cdot 240}{1.1 \cdot \sqrt{3}} = 84 \,\text{N/mm}^2$$

Analysis of the buckling panel according to DIN 18 800 Part 3, El. (501):

$$\frac{\sigma_{x}}{\sigma_{xP,R,d}} = \frac{13}{21.8} = 0.60 < 1$$
$$\frac{\tau}{\tau_{P,R,d}} = \frac{5.2}{8.4} = 0.62 < 1$$

Analysis for simultaneous occurrence of edge stresses (interaction) according to DIN 18 800 Part 3, El. (504):

$$e_{1} = 1 + \kappa_{\sigma_{x}}^{4} = 1 + 1^{4} = 2$$

$$e_{3} = 1 + \kappa_{\sigma_{x}} \cdot \kappa_{\tau}^{2} = 1 + 1.0 \cdot 0.669^{2} = 1.447$$

$$\left(\frac{|\sigma_{x}|}{\sigma_{xP,R,d}}\right)^{e_{1}} + \left(\frac{\tau}{\tau_{P,R,d}}\right)^{e_{3}} \le 1$$

$$\left(\frac{13}{21.8}\right)^{2} + \left(\frac{5.2}{8.4}\right)^{1.447} = 0.73 \le 1$$

The plate buckling safety is sufficient!



### 9.2 EN 1993-1-5

### 9.2.1 Unstiffened Buckling Panel with Local Buckling Behavior

The following example is extracted from [6]. It describes the plate buckling analysis for an unstiffened buckling panel.



Figure 9.2: Sketch of structural system including dimensions and loads

Material:		Steel S355		
		Yield	strength f	$_{k,k} = 355 \text{ N/mm}^2$
Partial safety factor:		<i>γ</i> <sub>M</sub> = 1	.1	
Parameters of the structural s	ystem:			
Length of the buckling panel		а	=	600 mm
Width of the buckling panel		b	=	1000 mm
Plate thickness		t	=	12 mm
$\rightarrow$ Side ratio	$\alpha = \frac{a}{b}$	$=\frac{600}{1000}$	- = 0.60	
Governing stresses:				
Axial compressive stress		$\sigma_1$	=	100 N/mm <sup>2</sup>
Axial compressive stress		$\sigma_2$	=	100 N/mm <sup>2</sup>
Shear stress		τ	=	50 N/mm <sup>2</sup>

### Edge stress ratio related to the maximum compressive stress:

$$\psi = \frac{\sigma_2}{\sigma_1} = \frac{100}{100} = 1.0$$

### **Euler critical stress:**

$$\sigma_{E} = \frac{\pi^{2} \cdot E}{12 \cdot (1 - 0.3^{2})} \cdot \left(\frac{t}{b}\right)^{2} = \frac{3.14^{2} \cdot 210000}{12 \cdot (1 - 0.3^{2})} \cdot \left(\frac{12}{1000}\right)^{2} = 27.33 \text{ N/mm}^{2}$$

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Calculation of buckling values according to EN 1993-1-5, Table 4 and Annex A.3/A.5:

 $\alpha = 0.6 < 1$  and  $\psi = 1$   $k_{\sigma} = 4.0$  $k_{\tau} = 4.00 + \frac{5.34}{\alpha^2} = 4.00 + \frac{5.34}{0.60^2} = 18.83$ 

Critical plate buckling stress if edge stresses  $\sigma$  according to EN 1993-1-5, Annex A.1 are effective:

 $\sigma_{cr,p,x} = k_{\sigma,x} \cdot \sigma_E = 4.0 \cdot 27.33 = 109.32 \text{ N/mm}^2$ 

Critical plate buckling stress if edge stresses  $\tau$  are effective:

 $\tau_{cr} = k_{\tau} \cdot \sigma_{E} = 18,83 \cdot 27.33 = 514.75 \text{ N/mm}^{2}$ 

Yield criterion according to EN 1993-1-5, Eq. (10.3):

$$\alpha_{ult,k} = \frac{f_y}{\sigma_{v,Ed}} = \frac{355}{132.29} = 2.6835$$

where: 
$$\sigma_{v,Ed} = \sqrt{\sigma_{x,Ed}^2 + 3 \cdot \tau_{Ed}^2} = \sqrt{100^2 + 3 \cdot 50^2} = 132.29 \text{ N/mm}^2$$

Eigenvalues of the stress components according to EN 1993-1-5, Eq. (10.6):

$$\alpha_{\rm cr,x} = \frac{\sigma_{\rm cr,p,x}}{\sigma_{\rm x,Ed}} = \frac{109.32}{100} = 1.0932$$
$$\alpha_{\rm cr,\tau} = \frac{\tau_{\rm cr}}{\tau_{\rm Ed}} = \frac{514.75}{50} = 10.295$$

Critical load factor according to EN 1993-1-5, Eq. (10.6):

$$\frac{1}{\alpha_{\rm cr}} = \frac{1 + \psi_{\rm x}}{4 \cdot \alpha_{\rm cr,x}} + \sqrt{\left(\frac{1 + \psi_{\rm x}}{4 \cdot \alpha_{\rm cr,x}}\right)^2 + \frac{1 - \psi_{\rm x}}{2 \cdot \alpha_{\rm cr,x}^2} + \frac{1}{\alpha_{\rm cr,\tau}^2}}$$
$$\alpha_{\rm cr} = 1.081$$

Plate slenderness ratio according to EN 1993-1-5, Eq. (10.2):

$$\overline{\lambda}_{P} = \overline{\lambda}_{w} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} = \sqrt{\frac{2.6835}{1.081}} = 1.576$$

Reduction factors for plate buckling according to EN 1993-1-5, section 4.4 and Table B.1 Check:

$$\begin{split} \overline{\lambda}_{P} &\geq 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ 1.576 &\geq 0.673 \\ \rho_{p} &= \frac{\overline{\lambda}_{P} - 0.055(3 + \psi)}{\overline{\lambda}_{P}^{2}} \leq 1.00 \\ \rho_{p} &= \frac{1.576 - 0.055(3 + 1.0)}{1.576^{2}} \leq 1.00 \\ \rho_{p} &= 0.546 \leq 1.00 \end{split}$$

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Using Table 5.1 and the option "Non-rigid end post":

$$\chi_{\rm w} = \frac{0.83}{\overline{\lambda_{\rm p}}} = \frac{0.83}{1.576} = 0.527$$

Plate buckling with local buckling behavior according to EN 1993-1-5, section 4.5.4(1):

$$\xi = \frac{\sigma_{cr,c}}{\sigma_{cr,c}} - 1 \le 1$$
  
where:  $\sigma_{cr,c} = \frac{\pi^2 \cdot E \cdot t^2}{12 \cdot (1 - v^2) \cdot a^2} = \frac{\pi^2 \cdot 210000 \cdot 12^2}{12 \cdot (1 - 0.3^2) \cdot 600^2} = 75.92 \text{ N/mm}^2$   
 $\xi = \frac{109.32}{75.92} - 1 = 0.44 \le 1$ 

The total buckling panel shows a local buckling behavior.

### Reduction factor with local buckling behavior:

$$\rho_{c} = \left(\rho_{p} - \chi_{c}\right) \cdot \xi \cdot \left(2 - \zeta_{x}\right) + \chi_{c} = \left(0.546 - 0.342\right) \cdot 0.44 \cdot \left(2 - 0.44\right) + 0.342 = 0.482$$
  
where:  $\chi_{c} = \frac{1}{\left(\theta_{p} + \sqrt{\theta_{p}^{2} - \overline{\lambda_{p}^{2}}}\right)} = \frac{1}{\left(1.886 + \sqrt{1.886^{2} - 1.576^{2}}\right)} = 0.342$   
 $\theta_{p} = 0.5 \cdot \left(1 + 0.21 \cdot \left(1.576 - 0.2\right) + 1.576^{2}\right) = 1.886$ 

Analysis (interaction condition) according to EN 1993-1-5, Eq. (10.5):

$$\left(\frac{\sigma_{x,Ed}}{\rho_{c} \cdot \frac{f_{y}}{\gamma_{M1}}}\right)^{2} + 3 \cdot \left(\frac{\tau_{Ed}}{\chi_{w} \cdot \frac{f_{y}}{\gamma_{M1}}}\right)^{2} \le 1$$
$$\left(\frac{100}{0.482 \cdot \frac{355}{1.1}}\right)^{2} + 3 \cdot \left(\frac{50}{0.527 \cdot \frac{355}{1.1}}\right)^{2} = 0.672 \le 1$$

The plate buckling safety is sufficient!



### 9.2.2 Stiffened Buckling Panel

The following example is extracted from [18]. It describes the plate buckling analysis for a stiffened buckling panel.



Figure 9.3: Sketch of structural system including dimensions and loads

Material:	Steel S355			
	Yield strength $f_{y,k}$ = 345 N/mm <sup>2</sup> (for $t$ = 30 mm)			
Partial safety factor:	$\gamma_M = 1.1$	$\gamma_M = 1.1$		
Parameters of the structural system:				
Length of the buckling panel	а	=	4000 mm	
Width of the buckling panel	b	=	4647 mm	
Plate thickness	t	=	27 mm	
$\rightarrow$ Side ratio $\alpha = \frac{a}{b}$	$=\frac{4000}{4647}$	= 0.861		
Stiffener:				
Height:	h	=	300 mm	
Length:	а	=	4000 mm	
Thickness:	t	=	30 mm	
Stiffener position:	Ζ	=	3098 mm (from upper edge)	
Governing stresses:				
Axial compressive stress	$\sigma_2$	=	297.6 N/mm <sup>2</sup>	
Normal shear stress	$\sigma_1$	=	- 262.1 N/mm <sup>2</sup>	
Shear stress	τ	=	119.5 N/mm <sup>2</sup>	



### Determination of critical plate buckling stresses according to EN 1993-1-5, Annex A:

For a longitudinal stiffener in the compression zone it follows according to Annex A.2.2:

Critical buckling stress  $\sigma_{cr,p}$ 

$$\begin{array}{c}
\frac{3 - \psi_1}{5 - \psi_1} b_1 \\
0,4 b_{2c}
\end{array}$$

Figure 9.4: Figure A.1, Annex A

• Determination of effective widths:

$$\psi_{1} = \frac{\sigma_{sl,1}}{\sigma_{1}} = \frac{111.03}{297.6} = 0.373 \ge 0 \quad \rightarrow \text{ okay}$$

$$b_{1} = 1549 \text{ mm}$$

$$b_{1,inf} = \frac{3 - 0.373}{5 - 0.373} \cdot 1549 = 879.45 \text{ mm}$$

$$b_{2,c} = \frac{\sigma_{sl,1}}{m} = \frac{111.03}{0.1204} = 922.76 \text{ mm}$$

$$b_{2,sup} = 0.4 \cdot 922.76 = 369.10 \text{ mm}$$

$$\Rightarrow \underline{A_{sl,1}} = 42711 \text{ mm}^{2}$$

$$\Rightarrow \underline{I_{sl,1}} = 2.549 \cdot 10^{9} \text{ mm}^{4}$$

• according to Eq. (A.4), Annex A:

$$a_{c} = 4.33 \cdot \sqrt[4]{\frac{I_{sl,1} \cdot b_{1}^{2} \cdot b_{2}^{2}}{t^{3} \cdot b}} = 12241 \text{ mm} > a = 4000 \text{ mm}$$

$$\sigma_{cr,sl} = \frac{\pi^{2} \cdot E \cdot I_{sl,1}}{A_{sl,1} \cdot a^{2}} + \frac{E \cdot t^{2} \cdot b \cdot a^{2}}{4 \cdot \pi^{2} \cdot (1 - \upsilon^{2}) \cdot A_{sl,1} \cdot b_{1}^{2} \cdot b_{2}^{2}}$$

$$\sigma_{cr,sl} = \frac{\pi^{2} \cdot 210000 \cdot 2.549 * 10^{9}}{42711 \cdot 4000^{2}} + \frac{210000 \cdot 27^{2} \cdot 4647 \cdot 4000^{2}}{4 \cdot \pi^{2} \cdot (1 - 0.3^{2}) \cdot 42711 \cdot 1549^{2} \cdot 3098^{2}}$$

$$\sigma_{cr,sl} = 796.1 \text{ N/mm}^{2}$$

The critical buckling stress  $\sigma_{cr,p}$  is obtained by extrapolating the edge subjected to pressure:

$$\sigma_{\rm cr,p} = \sigma_{\rm cr,sl} \cdot \frac{\sigma_1}{\sigma_{\rm sl,1}} = 796.1 \cdot \frac{297.6}{111.03} = 2134.41 \, \text{N/mm}^2$$



### Critical buckling stress $\tau_{cr}$

• Determination of effective widths

According to EN 1993-1-5, Figure 5.3, the minimum widths are used for the effective widths.

$$\min b = 15 \cdot \varepsilon \cdot t_w$$
  

$$\min b = 15 \cdot \sqrt{\frac{235}{345}} \cdot 27 = 334.26 \text{ mm}$$
  

$$\Rightarrow \underline{A_{sl,1}} = 2.786 \cdot 10^4 \text{ mm}^2$$
  

$$\Rightarrow \underline{I_{sl,1}} = 2.315 \cdot 10^9 \text{ mm}^4$$

• Determination of buckling value according to A.3, Eq. (A.6):

$$\alpha = \frac{a}{h_{w}} = \frac{4000}{4647} = 0.861 < 3$$

$$k_{\tau} = 4.1 + \frac{6.3 + 0.18 \cdot \frac{l_{sl}}{t_{w}^{3} * h_{w}}}{\left(\frac{a}{h_{w}}\right)^{2}} + 2.2 \cdot \sqrt[3]{\frac{l_{sl}}{t_{w}^{3} * h_{w}}}$$

$$k_{\tau} = 4.1 + \frac{6.3 + 0.18 \cdot \frac{2.315 * 10^{9}}{27^{3} * 4647}}{\left(\frac{4000}{4647}\right)^{2}} + 2.2 \cdot \sqrt[3]{\frac{2.315 * 10^{9}}{27^{3} * 4647}} = 16.22$$

The critical buckling stress  $\tau_{cr}$  if edge stresses  $\tau$  are exerted is as follows:

$$\tau_{cr} = k_{\tau} \cdot \sigma_{E} = 16.22 \cdot \frac{\pi^{2} * 210000}{12 \cdot (1 - 0.3^{2})} \cdot \left(\frac{27}{4647}\right)^{2} = 103.9 \text{ N/mm}^{2}$$

Yield criterion according to EN 1993-1-5, Eq. (10.3)

$$\alpha_{ult,k} = \frac{f_y}{\sigma_{v,Ed}} = \frac{345}{362.5} = 0.952$$
  
where:  $\sigma_{v,Ed} = \sqrt{\sigma_{x,Ed}^2 + 3 \cdot \tau_{Ed}^2} = \sqrt{297.6^2 + 3 \cdot 119.5^2} = 362.5 \text{ N/mm}^2$ 

Eigenvalues of the stress components according to EN 1993-1-5, Eq. (10.6):

$$\alpha_{cr,x} = \frac{\sigma_{cr,p,x}}{\sigma_{x,Ed}} = \frac{2134,41}{297.6} = 7.172$$
$$\alpha_{cr,\tau} = \frac{\tau_{cr}}{\tau_{Ed}} = \frac{103,9}{119.5} = 0.869$$

Critical load factor according to EN 1993-1-5, Eq. (10.6):

$$\frac{1}{\alpha_{\rm cr}} = \frac{1 + \psi_{\rm x}}{4 \cdot \alpha_{\rm cr,x}} + \sqrt{\left(\frac{1 + \psi_{\rm x}}{4 \cdot \alpha_{\rm cr,x}}\right)^2 + \frac{1 - \psi_{\rm x}}{2 \cdot \alpha_{\rm cr,x}^2} + \frac{1}{\alpha_{\rm cr,\tau}^2}}$$
$$\alpha_{\rm cr} = 0.86$$

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Plate slenderness ratio according to EN 1993-1-5, Eq. (10.2):

$$\overline{\lambda}_{\mathsf{P}} = \overline{\lambda}_{\mathsf{w}} = \sqrt{\frac{\alpha_{\mathsf{ult},k}}{\alpha_{\mathsf{cr}}}} = \sqrt{\frac{0.952}{0.86}} = 1.052$$

Reduction factors for plate buckling according to EN 1993-1-5, Eq. (4.2) and Table 5.1:

$$\rho_{p} = \frac{\overline{\lambda_{p}} - 0.055 \cdot (3 + \psi)}{\overline{\lambda_{p}}^{2}} = \frac{1.052 - 0.055 \cdot \left(3 + \left(\frac{-262.1}{297.6}\right)\right)}{1.052^{2}} = 0.845$$

According to Table 5.1, the contribution of the web in the case of a ridged end post is given by:

$$\chi_{\rm w} = \frac{0.83}{\overline{\lambda_{\rm p}}} = \frac{0.83}{1.052} = 0.789$$

Buckling with local buckling behavior according to EN 1993-1-5, section 4.5.4, Eq. (1):

$$\xi = \frac{\sigma_{cr,c}}{\sigma_{cr,c}} - 1 = \frac{2134.41}{2109.8} - 1 = 0.011 \le 1$$
  
where: 
$$\sigma_{cr,c} = \frac{\pi^2 \cdot E \cdot I_{sl,1}}{A_{sl,1} \cdot a^2} \cdot \frac{\sigma_1}{\sigma_{sl,1}} = \frac{\pi^2 \cdot 210000 \cdot 2.549 \cdot 10^9}{42711 \cdot 4000^2} \cdot \frac{297.6}{111.03} = 2109.8 \,\text{N/mm}^2$$

The entire buckling field shows a local buckling behavior.

#### **Reduction factor with local buckling behavior:**

$$\rho_{c} = \left(\rho_{p} - \chi_{c}\right) \cdot \xi \cdot (2 - \zeta_{x}) + \chi_{c} = (0.845 - 0.496) \cdot 0.011 \cdot (2 - 0.011) + 0.496 = 0.487$$

$$\chi_{c} = \frac{1}{\left(\theta_{p} + \sqrt{\theta_{p}^{2} - \overline{\lambda_{p}^{2}}}\right)} = \frac{1}{\left(1.282 + \sqrt{1.282^{2} - 1.052^{2}}\right)} = 0.496$$
where:
$$\theta_{p} = 0.5 \cdot \left(1 + \alpha_{e} \cdot (1.052 - 0.2) + 1.052^{2}\right) = 1.282$$

$$\alpha_{e} = \alpha + \frac{0.09}{i/e} = 0.49 + \frac{0.09}{\sqrt{\frac{2.549 \cdot 10^{9}}{42711}}} = 0.537$$

### Analysis (interaction condition) according to EN 1993-1-5, Eq. (10.5):

$$\left(\frac{\sigma_{x,Ed}}{\rho_{c} \cdot \frac{f_{y}}{\gamma_{M1}}}\right)^{2} + 3 \cdot \left(\frac{\tau_{Ed}}{\chi_{w} \cdot \frac{f_{y}}{\gamma_{M1}}}\right)^{2} \le 1$$
$$\left(\frac{297.50}{0.487 \cdot \frac{345}{1.1}}\right)^{2} + 3 \cdot \left(\frac{119.50}{0.789 \cdot \frac{345}{1.1}}\right)^{2} = \underline{4.50 > 1}$$

The buckling safety is <u>not</u> sufficient!



## **A** Literature

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