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Design rules for lateral torsional buckling of channel sections subject to web loading

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Channel sections are widely used in practice as beams. However, design rules for eccentrically loaded (not through shear centre) beams with channel cross-sections are not available in Eurocode 3. In this paper five proposed design rules are summarised, explained and their validity is checked by Finite Element analyses. The design rules yield ultimate loads that are compared to ultimate loads from geometrical and material nonlinear analyses of imperfect (GMNIA) beams with channel cross-sections. A parameter study is performed by varying the dimensions of the cross-section, the span length to section height ratio of the beams, the type of loading and the point of load application but is limited to load application through the web of the channels. Based on one of the proposed design rules, this study has led to a new design rule which conforms to Eurocode 3.

Bemessungsregeln für das Biegedrillknicken von U-Querschnitten unter Belastung in Stegebene. *In der Praxis kommen häufig U-Querschnitte zum Einsatz. Eurocode 3 enthält jedoch keine Bemessungsregeln für exzentrisch, d. h. nicht im Schubmittelpunkt, belastete Träger mit U-Querschnitt. In diesem Beitrag werden fünf Bemessungsvorschläge vorgestellt, erläutert und ihre Anwendbarkeit mit Hilfe von Finite-Element-Berechnungen überprüft. Die berechneten Tragfähigkeiten der Bemessungsmodelle werden mit Traglasten verglichen, die auf Grundlage von geometrisch und materiell nichtlinearen Berechnungen an Trägern mit Imperfektionen (GMNIA) ermittelt wurden. In der numerischen Parameterstudie werden die Querschnittsabmessungen, das Verhältnis von Spannweite zur Trägerhöhe, der Lasttyp und der Lastangriffspunkt variiert. Die Belastung wirkt jedoch lediglich in Stegebene. Basierend auf einem der bisherigen Bemessungsvorschläge wird im Rahmen dieser Untersuchung ein neuer Bemessungsvorschlag formuliert, der die Regelungen des Eurocode 3 widerspruchsfrei ergänzt.*

1 Introduction

Steel channel sections are often used in the building practice. The structural behaviour of mono-symmetric channel sections is different from that of double symmetric cross-sections, such as solid or I-shaped cross-sections as shown in figure 1. This difference exists because the shear centre (S) and centre of gravity (C) do not coincide. If the applied load goes through the shear centre of a channel section (figure 1c), the load is called 'centric'. It has been shown [1] that standard code requirements for lateral torsional buckling of double symmetric cross-sections can be used for the design of centrically loaded channel sections. In a 1st order analysis, the cross-section does not rotate. In a 2nd order analysis, the cross-section will rotate due to compression in one of the flanges which causes that flange to buckle sideways.

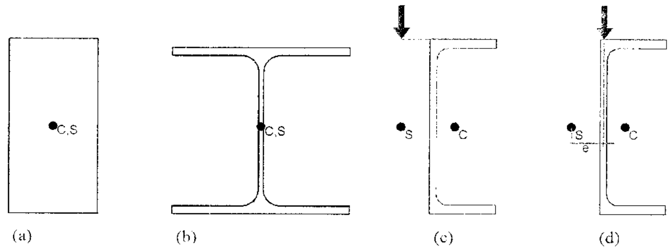


Fig. 1. Cross-sections: (a) solid, (b) double symmetric, (c) mono symmetric centrically loaded and (d) mono symmetric eccentrically loaded
Bild 1. Querschnittsarten: (a) voll, (b) doppelt-symmterisch, (c) einfach symmetrisch, zentrisch belastet, (d) einfach symmetrisch, exzentrisch belastet

When the applied load does not go through the shear centre (figure 1d), the load is called 'eccentric' and in addition to bending, 1st order torsion and rotation of the cross-section will occur. In a 2nd order analysis, the cross-sectional rotation is enhanced by sideways buckling of the compressed flange.

In practice channel sections are most frequently eccentrically loaded. However, no specific design rules are available in Eurocode 3 [2] for lateral torsional buckling of eccentrically loaded channel sections used as beams. In the literature the authors have found five proposed design methods for lateral torsional buckling of channel sections. Ultimate loads from these design rules have been compared to the ultimate loads obtained from geometrical and material nonlinear analyses of beams with imperfections (GMNIA) [3]. On the basis of a parameter study, a new design rule is proposed, which is in line with design rules in Eurocode 3.

2 Design rules

At Eindhoven University of Technology earlier studies on lateral torsional buckling of channel sections focussed on experiments [4] and Finite Element simulations [5]. This has led to a proposal for a design rule [1] based on the *Merchant-Rankine* formula.

In a joint project by several German universities research was carried out on the „Influence of torsion on the ultimate resistance of cross-sections and structural elements – Investigations into the influence of torsion effects on the plastic cross-sectional resistance and the ultimate resistance of steel profiles" [6-15] („Einfluss der Torsion auf die Grenztragfähigkeit von Querschnitten und Bauteile – Untersuchungen zum Einfluss der Torsionseffekte auf die plastische Querschnittstragfähigkeit und die Bautragfähigkeit von Stahlprofilen"). This yielded three proposals for the design of beams of hot-rolled steel channel sections.

In Eurocode 3 [2] the so called 'General Method' can also be used for the design of beams of channel sections.

2.1 Modified Merchant-Rankine method

The method of design [1] is based on the *Merchant-Rankine* formula which is modified to yield the ultimate load for eccentrically loaded channel beams:

$$F_{MR} = \frac{1}{\frac{1}{F_{cr}} + \frac{1}{F_{pl}}} + \mu \cdot F_{pl} \qquad (1)$$

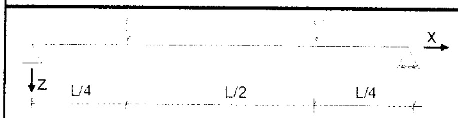
where

F_{pl} is the first order plastic limit load of the beam shown in table 1

F_{cr} is the elastic critical load of the beam shown in table 1

μ is a correction factor given in table 1.

Table 1. Correction factors of the Modified Merchant-Rankine Method
Tabelle 1. Korrekturfaktoren für das modifizierte Merchant-Rankine-Verfahren



Load case	Point of load application		μ [-]
	A		0.06
	B		0.11
	C		0.15

2.2 Modified κ_M-method

The design rule for lateral torsional buckling of beams in bending, as given in DIN 18800 [16] is as follows:

$$\frac{M}{\kappa_M \cdot M_{pl}} \leq 1.0 \qquad (2)$$

where

M is the design bending moment due to the applied loading

M_{pl} is the plastic moment capacity of the cross-section

κ_M is the reduction factor for lateral torsional buckling given by:

$$\kappa_M = \left(1 + \bar{\lambda}_M^5\right)^{-0.4} \qquad (3)$$

This design rule is analogous to the procedure given in Eurocode 3 [2] except for the lateral torsional buckling curve as represented by equation (3). The relative slenderness $\bar{\lambda}_M$, which is required to obtain the reduction factor κ_M, must be computed first:

$$\bar{\lambda}_M = \sqrt{\frac{M_{pl}}{M_{cr}}} \qquad (4)$$

where

M_{cr} is the elastic critical moment.

For channel sections the reduction factor κ_M should not be determined as presented above. The relative slenderness $\bar{\lambda}_M$ must first be adjusted to account for torsion. This has been achieved [6] by adding a term $\bar{\lambda}_T$ to $\bar{\lambda}_M$ which results in a relative slenderness that includes the influence of torsion $\bar{\lambda}_{MT}$. This is relevant only in case of a relative slenderness $\bar{\lambda}_M$ greater than 0.5 because the influence of second order effects is limited in case of compact beams. This method results in a modified reduction factor which can be obtained as follows:

$$\kappa_{MT} = \left(1 + \bar{\lambda}_{MT}^5\right)^{-0.4} \qquad (5)$$

Where the relative slenderness including the influence of torsion is defined as:

$$\bar{\lambda}_{MT} = \bar{\lambda}_M + \bar{\lambda}_T \qquad (6)$$

The torsion term $\bar{\lambda}_T$ depends on the relative slenderness as follows:

$$\begin{aligned} \bar{\lambda}_T &= 1.11 - \bar{\lambda}_M && \text{if } 0.5 \leq \bar{\lambda}_M < 0.75 \\ \bar{\lambda}_T &= 0.69 - 0.44\bar{\lambda}_M && \text{if } 0.75 \leq \bar{\lambda}_M < 1.14 \\ \bar{\lambda}_T &= 0.19 && \text{if } \bar{\lambda}_M \geq 1.14 \end{aligned} \qquad (7)$$

The reduction factor κ_M in equation (2) is replaced by the reduction factor κ_{MT} given by equation (5). This design rule is valid for channel sections subjected to a uniformly distributed load on the web.

2.3 Modified χ_{LT}-method

Instead of using equation (5) to obtain the reduction factor according to DIN, $\bar{\lambda}_{MT}$ could also be used in combination with the lateral torsional buckling curves of Eurocode 3 to obtain a reduction factor χ_{LT}:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{MT}^2}} \qquad (8)$$

with

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}(\bar{\lambda}_{MT} - 0.2) + \bar{\lambda}_{MT}^2\right] \qquad (9)$$

where
$\bar{\lambda}_{MT}$ is the modified relative slenderness according to equation (6)
α_{LT} is the imperfection factor corresponding to the relevant buckling curve.

This reduction factor can then replace κ_M in equation (2) to determine the ultimate load. Then, the design rule becomes:

$$\frac{M}{\chi_{LT} \cdot M_{pl}} \leq 1.0 \tag{10}$$

In this way, the effect of torsion is included in the Eurocode 3 design rule. This will be called the Modified χ_{LT}-method.

2.4 Second order theory, the α_ϑ-method

In this proposed design method [7], second order theory is employed to calculate internal forces for the situation shown in figure 1d where a beam is eccentrically loaded on the web. This introduces a torsional moment. The internal forces are checked on the basis of their resulting stresses. The calculation procedure for this method is quite long and omitted here.

2.5 Simplified design rule

In Germany, a large investigation consisting of experimental testing and simulations with several Finite Element programs was carried out. This investigation [13] has led to a simplified design rule for lateral torsional buckling of beams with channel cross-sections. The original formulae were presented for double bending and are simplified here for single bending:

$$\frac{M}{\chi_{LT} \cdot M_{pl}} + k_w \cdot \alpha \cdot \frac{M_\omega}{M_{pl,\omega}} \leq 1.0 \tag{11}$$

$$k_w = 0.7 - 0.2\frac{M_\omega}{M_{pl,\omega}} \tag{12}$$

$$\alpha = \frac{1}{1 - \frac{M}{M_{cr}}} \tag{13}$$

where
α is the amplification factor
M_ω is the applied bi-moment
$M_{pl,\omega}$ is the plastic bi-moment capacity
k_w is a factor representing the influence of the torsional moment.

This design rule is applicable to all load cases, all points of load application and all magnitudes of eccentricity.

2.6 General Method

In Eurocode 3 [2], the General Method may also be used for the design of beams with a channel cross-section subject to eccentric loading. Summarised, the General Method requires the following check:

$$\frac{\chi_{op} \cdot \alpha_{ult,k}}{\gamma_{M1}} \geq 1.0 \tag{14}$$

where
γ_{M1} is a partial factor for element instability: $\gamma_{M1} = 1$
χ_{op} is the lower value of the reduction factors for lateral buckling χ and lateral torsional buckling χ_{LT}, based on the relative slenderness $\bar{\lambda}_{op}$:

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} \tag{15}$$

where
$\alpha_{ult,k}$ is the resistance based on in-plane behaviour without lateral or lateral torsional buckling but including the effects of in-plane geometrical deformations and imperfections
$\alpha_{cr,op}$ is the elastic critical resistance with respect to out-of-plane instability.

3 Finite Element Model

The ultimate loads from the proposed design rules are compared to ultimate loads obtained from the Finite Element Method (FEM). The FEM-program used is Ansys release 10.0.

3.1 Elements

The thickness of the flanges and the web are relatively small compared to the other dimensions. Therefore, shell elements are used: four node three dimensional shell (SHELL181) elements with 7 integration points over the thickness of the shell-element, based on *Mindlin* shell theory.

3.2 Model

In this particular investigation, the cross-section (figure 2a) is divided into several elements, 12 elements in the web and 8 elements in a flange. For beam spans shorter than 2.8 meter, the number of elements over the length is 28 in order to avoid an increase of the discretisation error, see figure 2b. If the span exceeds 2.8 meter, the length of an element is kept to 100 mm to retain the same depth-to-length ratio. It was decided to neglect the fillets at the intersection of web and flanges. This requires that all cross-sectional properties in the previously introduced design rules must be determined for this adjusted cross-section.

The end supports of the beam are also shown in figure 2b. At mid height of the web the displacements in the x-, y- and z-directions are zero at one end of the beam (hinged support) and at the other end only the displacements in the y- and z-direction are zero (roller support). In the corners of the cross-section the displacements in the y-direction are zero.

Additional beam elements with large flexural stiffness in both directions are introduced along the edges of the

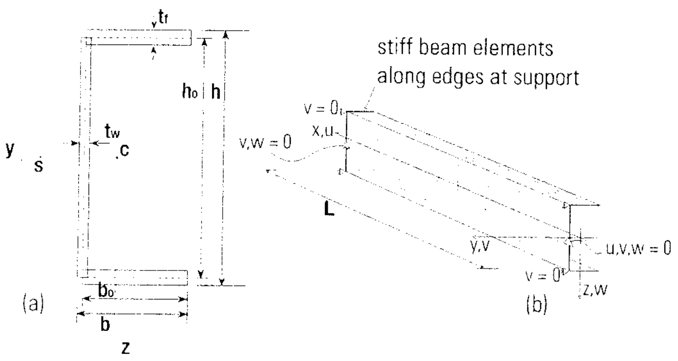


Fig. 2. FE Model: (a) cross-section, (b) length direction and boundary conditions
Bild 2. FE Modell: (a) Querschnitt, (b) Längsrichtung und Lagerung

section at the supports and allow the ends free to warp but not twist. These stiff beam elements are given a very small sectional area to enable *Poisson's* contraction. Assigning very small torsional stiffness to these elements disables the channel section beam to indirectly acquire extra torsional stiffness from these beam elements.

3.3 Imperfections

Geometric and material nonlinear analyses of imperfect (GMNIA) beams are performed to obtain the ultimate loads. The imperfections to be included in these analyses consist of geometrical imperfections and residual stresses. The imperfection mode is taken to be equal to the lateral torsional buckling mode, which is in line with Eurocode 3. The maximum imperfection is assumed to be $L/1000$, which is a commonly used value for the situation where residual stresses are explicitly taken into account [17]. The direction of the imperfection is chosen unfavourably in the direction of the eccentricity.

Measurements of residual stresses in channel sections are not available in literature. For more common IPE-sections, the residual stresses are known and shown in figure 3a. Applying the same magnitude of stresses to a channel section would yield the residual stresses shown in figure 3b. Since a channel section is not double symmetric, the stresses in figure 3b are not in equilibrium. Therefore, the theoretically obtained residual stresses [13] as shown in figure 3c are used.

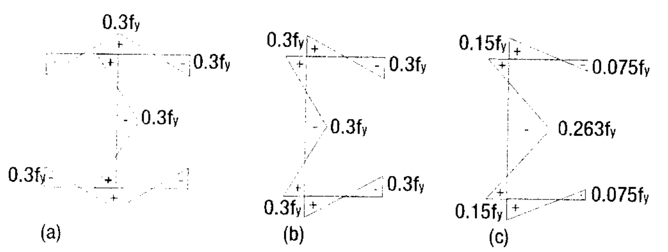


Fig. 3. Residual stresses: (a) IPE-section, (b) Channel without equilibrium (c) Channel in equilibrium
Bild 3. Eigenspannungsverteilungen: (a) IPE-Querschnitt, (b) U-Querschnitt ohne Gleichgewicht, (c) U-Querschnitt mit Gleichgewicht

3.4 Material

The steel grade employed in all analyses is S235 with a yield strength of 235 N/mm². A bilinear stress-strain curve is used with a *Young's* modulus of elasticity of 210 kN/mm². The *Von-Mises* yield criterion is applied.

4 Example calculations

In this section, the calculations to obtain the ultimate loads from the proposed design rules and the results from the FEM are given for the channel section beam shown in figure 4. The cross-sectional dimensions represent an adjusted UPE 160, i. e. without fillets. The beam spans 2800 mm and is subjected to a uniformly distributed load on the top flange.

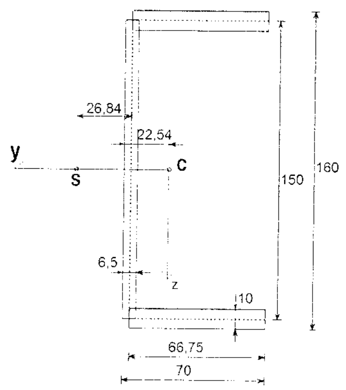


Fig. 4. Cross-sectional dimensions
Bild 4. Querschnittsabmessungen

4.1 Design calculations

4.1.1 Modified Merchant-Rankine method

This method was originally developed for a beam with two point loads (see table 1). As the bending moment distribution from two point loads is inherently different from that of a uniformly distributed load, the design method given by equation (1) for point loads must be adjusted to make it suitable for a uniformly distributed load, i. e.

$$q_{MR} = \frac{1}{\frac{1}{q_{cr}} + \frac{1}{q_{pl}}} + \mu \cdot q_{pl} \tag{16}$$

where

q_{MR} is the *Merchant-Rankine* based ultimate uniformly distributed load
q_{pl} is the plastic uniformly distributed load
q_{cr} is the elastic critical uniformly distributed load
μ is a correction factor given in table 1: for this case $\mu = 0.06$.

The elastic critical buckling moment is determined numerically. For a uniformly distributed load applied on the top flange this yields $M_{cr} = 35.56$ kNm and thus $q_{cr} = 36.29$ N/mm.

The plastic moment capacity is also determined numerically: $M_{pl} = 32.03$ kNm. This results in a plastic uniformly distributed load of q_{pl} = 32.68 N/mm. The ultimate uniformly distributed load according to this design rule then becomes:

$$q_{MR} = \frac{1}{\frac{1}{36.29} + \frac{1}{32.68}} + 0.06 \cdot 32.68 = 19.16 \text{ N/mm}$$

4.1.2 Modified κ_M-method

Using equation (4), the relative slenderness for buckling becomes:

$$\bar{\lambda}_M = \sqrt{\frac{M_{pl}}{M_{cr}}} = \sqrt{\frac{32.03}{35.56}} = 0.95$$

The influence of torsion is given by equation (7):

$$\bar{\lambda}_T = 0.69 - 0.44\bar{\lambda}_M = 0.69 - 0.44 \cdot 0.95 = 0.27$$

The relative slenderness including the effect of torsion from equation (6) is:

$$\bar{\lambda}_{MT} = \bar{\lambda}_M + \bar{\lambda}_T = 0.95 + 0.27 = 1.22$$

Then, the reduction factor is obtained from equation (5):

$$\kappa_{MT} = \left(1 + \bar{\lambda}_{MT}^5\right)^{-0,4} = \left(1 + 1.22^5\right)^{-0,4} = 0.59$$

Using equation (2), the ultimate bending moment $M_{u,Mod-\kappa}$ can then be expressed as follows:

$$M_{u,Mod-\kappa} = \kappa_{MT} \cdot M_{pl} = 0.59 \cdot 32.03 = 18.90 \text{ kNm}$$

This yields a uniformly distributed load:

$$q_{Mod,\kappa} = \frac{8 \cdot M_{u,Mod-\kappa}}{L^2} = \frac{8 \cdot 18.90 \cdot 10^6}{2800^2} = 19.29 \text{ N/mm}$$

4.1.3 Modified χ_{LT}-method

Alternatively, the Modified χ_{LT}-method can be used, where $\bar{\lambda}_{MT}$ is used in combination with buckling curve 'a' of Eurocode 3 [2]. The reduction factor χ_{LT} can be obtained using equations (9) and (8) respectively:

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{MT} - 0.2\right) + \bar{\lambda}_{MT}^2\right] = 0.5\left[1 + 0.21(1.22 - 0.2) + 1.22^2\right] = 1.35$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{MT}^2}} = \frac{1}{1.35 + \sqrt{1.35^2 - 1.22^2}} = 0.52$$

The ultimate bending moment $M_{u,Mod-\chi}$ then becomes with equation (10):

$$M_{u,Mod-\chi} = \chi_{LT} \cdot M_{pl} = 0.52 \cdot 32.03 = 16.66 \text{ kNm}$$

This results in:

$$q_{Mod,\chi} = \frac{8 \cdot M_{u,Mod-\chi}}{L^2} = \frac{8 \cdot 16.66 \cdot 10^6}{2800^2} = 17.00 \text{ N/mm}$$

4.1.4 Second order theory, the α_ϑ-method

The design rule of the α_ϑ-method is complex to use. The ultimate load according to this method was obtained by a trial and error process [3] and yields an ultimate uniformly distributed load q_{2nd} = 15.25 N/mm based on *Terrington's* Bi-moment [18].

4.1.5 Simplified design rule

The design rule of this proposal is laborious as well and the ultimate load must also be obtained by an iterative procedure. Earlier calculations [3] yielded an ultimate uniformly distributed load q_{Simple} = 14.50 N/mm.

4.1.6 General Method

The relative slenderness is obtained from equation (15):

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} = \sqrt{\frac{M_{pl}}{M_{cr}}} = \sqrt{\frac{32.03}{35.56}} = 0.95$$

With buckling curve 'd' for channel sections, this results in a reduction factor:

$$\Phi_{LT} = 0.5\left[1 + 0.76\left(0.95 - 0.2\right) + 0.95^2\right] = 1.24$$

$$\chi_{op} = \chi_{LT} = \frac{1}{1.24 + \sqrt{1.24^2 - 0.95^2}} = 0.49$$

Using equation (14) and taking γ_{M1} = 1, the ultimate bending moment M_{GM} can be calculated as follows:

$$\alpha_{ult,k} = \frac{1}{\chi_{op}} = \frac{M_{pl}}{M_{GM}}$$

and so:

$$M_{GM} = \chi_{op} \cdot M_{pl} = 0.49 \cdot 32.03 = 15.69 \text{ kNm}$$

This results in:

$$q_{GM} = \frac{8 \cdot M_{GM}}{L^2} = \frac{8 \cdot 15.69 \cdot 10^6}{2800^2} = 16.01 \text{ N/mm}$$

4.2 Finite Element results

With a GMNIA calculation the ultimate uniformly distributed load is q_{FEM} = 21.39 N/mm. This is for the situation where residual stresses are explicitly taken into account and geometrical imperfections are introduced according to the buckling mode with an amplitude of L/1000. For comparison, an additional GMNIA calculation was carried out without taking residual stresses into account directly but with a larger amplitude of the geometrical imperfection of L/150, as given by Eurocode 3. In that case the ultimate load is $q_{FEM,150}$ = 19.40 N/mm

4.3 Comparison of results

Figure 5 shows a comparison of results for the adjusted UPE160 section with a span of 2800 mm subjected to a uniformly distributed load applied on the top flange. The load-displacement diagram includes both curves for an imperfection of L/1000 with residual stresses and L/150 without residual stresses.

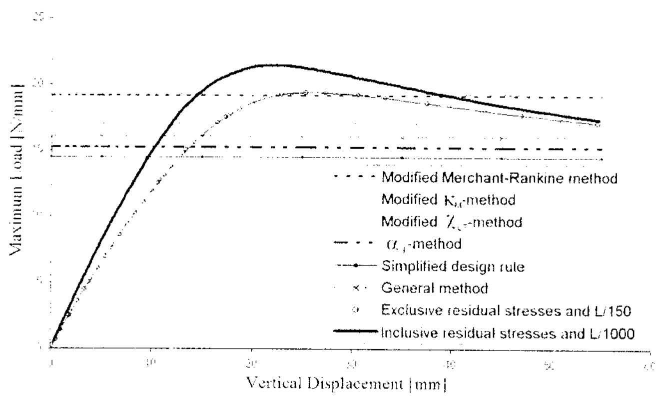


Fig. 5. Load-displacement diagram with maximum uniformly distributed loads
Bild 5. Last-Verfomungs-Diagramm mit Tragfähigkeiten

The results are also compared in table 2. It is clearly shown that all ultimate loads from the proposed design rules underestimate the loads obtained with GMNIA, even for an imperfection magnitude of L/150. The Modified *Merchant-Rankine* method and the Modified κ_M-method yield the smallest underestimation.

Table 2. Comparison of ultimate loads (UPE 160, span 2800 mm, uniformly distributed load on the top flange)
Tabelle 2. Vergleich der Tragfähigkeiten (UPE 160, Länge 2800 mm, Gleichstreckenlast am Druckflansch)

	Maximum load	Error $\left[\frac{q_{FEM} - q}{q_{FEM}} * 100\right]$
GMNIA Imperfection L/1000 with residual stresses q_{FEM}	21.39 N/mm	0.00 %
GMNIA Imperfection L/150 without residual stresses	19.40 N/mm	9.30 %
1 – Modified *Merchant-Rankine* method	19.16 N/mm	10.42 %
2 – Modified κ_M-method	19.29 N/mm	9.82 %
3 – Modified χ_{LT} -method	17.00 N/mm	20.52 %
4 – Second order theory, the α_ϑ-method	13.18 N/mm	38.38 %
5 – Simplified design rule	14.50 N/mm	32.21 %
6 – General method	16.01 N/mm	25.15 %

Since the ultimate loads from the „α_ϑ-method" and the „Simplified design rule" show differences of ± 30 % when compared to the ultimate loads from GMNIA, they will be left out of the parameter study.

4.4 Summary of remaining design rules

The remaining design rules can be expressed in terms of buckling curves which relate a reduction factor χ_{LT} to the relative slenderness $\bar{\lambda}_{LT}$. The results are shown in figure 6.

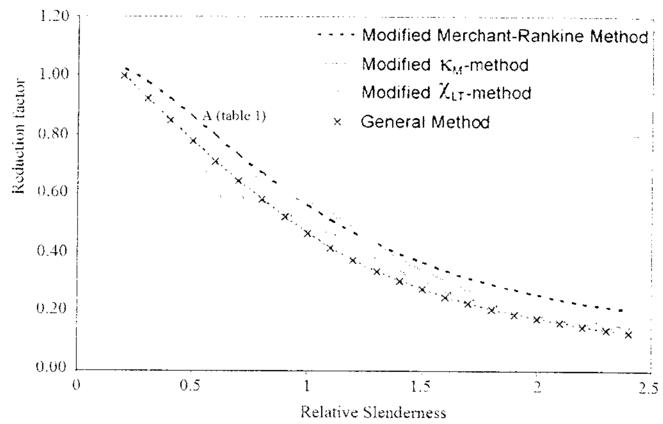


Fig. 6. Comparison of buckling curves studied
Bild 6. Vergleich der untersuchten Biegedrillknickspannungslinien

5 Parameter Study

For a thorough evaluation and comparison of the remaining design rule proposals, an extensive parameter study has been performed.

5.1 Cross-sections

Only adjusted cross-sections, i. e. without fillets, will be used with the dimensions of nominal UPE-sections. UPE-sections are available with a height of 80, 100, 120, 140, 160, 180, 200, 240, 270, 300, 330, 360 and 400 mm. Not every cross-section will be investigated. Table 3 shows which cross-sections are included in the parameter study.

5.2 Span lengths

A restriction to the span length to section height ratio of $15 \leq L/h \leq 40$ is taken as range of application. This results in a number of spans per cross-section as shown in table 3. For all span lengths, the number of elements used over the length is presented in brackets.

For cross-sections with a height of 80, 120 and 160 mm, the span increases with steps of 400 mm. For cross-sections with a height of 200, 270, 330 and 400 mm, the span increases with steps of 1000 mm. This results in a minimum of 6 and a maximum of 11 spans per cross-section.

Table 3. UPE-sections with spans considered
Tabelle 3. Untersuchte UPE-Querschnitte und Spannweiten

UPE	Span [m]	(*)	UPE	Span [m]	(*)	UPE	Span [m]	(*)	UPE	Span [m]	(*)	UPE	Span [m]	(*)
80	1.2	(28)	160	2.4	(28)	200	3	(30)	330	5	(50)	400	6	(60)
	1.6	(28)		2.8	(28)		4	(40)		6	(60)		7	(70)
	2	(28)		3.2	(32)		5	(50)		7	(70)		8	(80)
	2.4	(28)		3.6	(36)		6	(60)		8	(80)		9	(90)
	2.8	(28)		4	(40)		7	(70)		9	(90)		10	(100)
	3.2	(32)		4.4	(44)		8	(80)		10	(100)		11	(110)
120	2	(28)		4.8	(48)	270	4	(40)		11	(110)		12	(120)
	2.4	(28)		5.2	(52)		5	(50)		12	(120)		13	(130)
	2.8	(28)		5.6	(56)		6	(60)		13	(130)		14	(140)
	3.2	(32)		6	(60)		7	(70)					15	(150)
	3.6	(36)		6.4	(64)		8	(80)					16	(160)
	4	(40)					9	(90)						
	4.4	(44)					10	(100)						
	4.8	(48)					11	(110)						

(*) Number of elements over the length

5.3 Load cases

The loads are eccentrically applied at the centre line of the web. Two types of loading are considered: a uniformly distributed load over the full length of the beam and a point load at mid span. For each type of load, three points of load application are investigated. Load application points A, B and C are positioned respectively at the centre line of the upper flange, in the middle of the web and at the centre line of the lower flange (see figure 7).

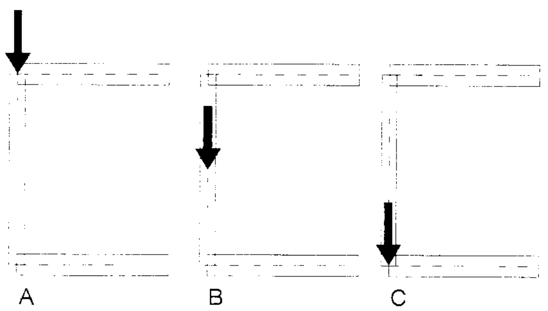


Fig. 7. Points of load application
Bild 7. Lastangriffspunkte

5.4 Results

The results from GMNIA are compared with the results obtained from the proposed design rules in terms of ultimate loads in a buckling curve presentation similar to figure 6. In the figures 8 to 10 the results are shown for a uniformly distributed load applied at the top flange, at web mid height and at the lower flange respectively. In the figures 11 to 13 the results are shown for a point load applied at the top flange, at web mid height and at the lower flange respectively. All results for one type of loading applied at one location are plotted in one graph for all sections included in the study.

5.5 Discussion of results

It can be seen in figures 8 to 13 that the ultimate loads from the Modified *Merchant-Rankine* method, are not always on the safe side when compared to the GMNIA ulti-

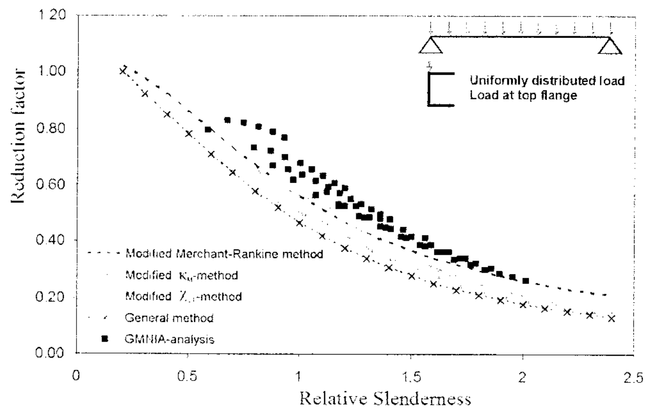


Fig. 8. Results for uniformly distributed load at the top flange
Bild 8. Ergebnisse bei Belastung des Druckflansches durch eine Gleichstreckenlast

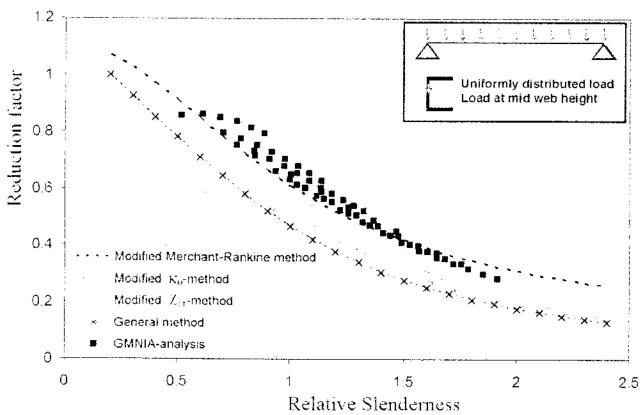


Fig. 9. Results for uniformly distributed load at web mid height
Bild 9. Ergebnisse bei Belastung im Stegschwerpunkt durch eine Gleichstreckenlast

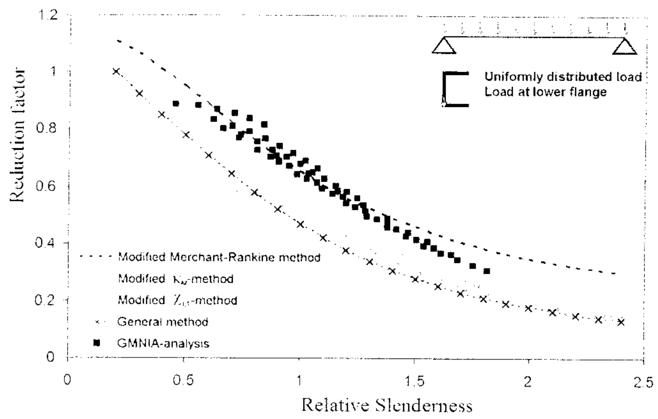


Fig. 10. Results for uniformly distributed load at the lower flange
Bild 10. Ergebnisse bei Belastung des Zugflansches durch eine Gleichstreckenlast

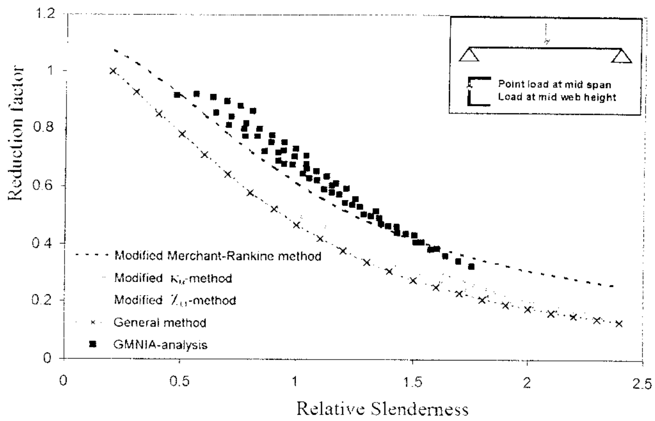


Fig. 12. Results for point load at web mid height
Bild 12. Ergebnisse bei Belastung im Stegschwerpunkt durch eine Einzellast

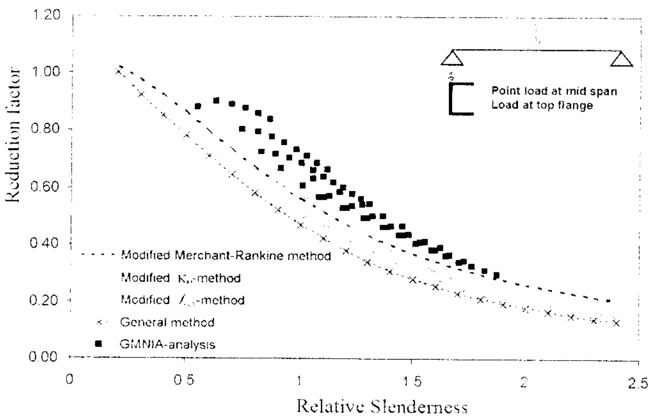


Fig. 11. Results for point load at the top flange
Bild 11. Ergebnisse bei Belastung des Druckflansches durch eine Einzellast

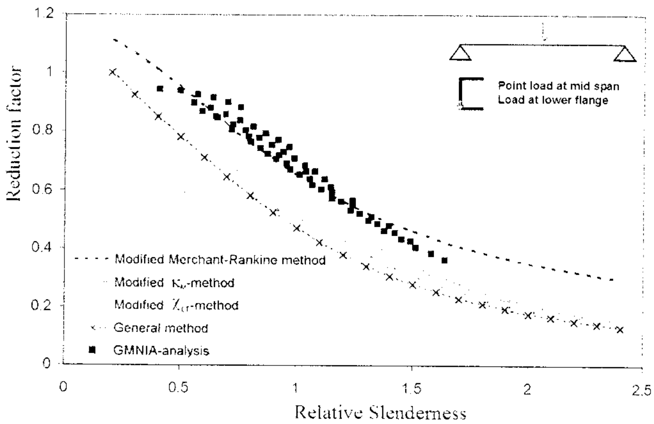


Fig. 13. Results for point load at the lower flange
Bild 13. Ergebnisse bei Belastung des Zugflansches durch eine Einzellast

mate loads. However, the ultimate loads from the Modified κ_M-method are always on the safe side. In many cases they are excessively so. The Modified χ_{LT}-method is even more conservative. For all cases the General Method yields the safest results.

It can be observed in figures 8 to 13 that the design curves for the Modified κ_M-method and the Modified χ_{LT}-method show a levelling off of the reduction factors for relative slenderness values smaller than 0.8. The same can be observed when studying the GMNIA results in detail. This is caused by the influence of warping restrained torsion on the plastic section strength under combined bending, shear and torsion. The strength requirement is governing the behaviour of the beams in this slenderness region.

The wide investigation into the design of eccentrically loaded channel section beams allows a proposal for a new design rule.

6 New design rule

Among the studied proposals, the best design rule appears to be the Modified κ_M-method. However, this method uses the DIN lateral torsional buckling curves and its results can be improved. In order to incorporate the Eurocode 3 buckling curves into the design of eccentrically loaded channel section beams, it is suggested to use the Modified χ_{LT}-method as a basis for a new design rule.

Figure 14 shows all GMNIA results from the parameter study and the design curve from the Modified χ_{LT}-method. There are two basic ways for the Eurocode 3 buckling curves to yield larger reduction factors for eccentrically loaded channel sections. The first way is to algebraically redefine the buckling curve, e. g. by adding a constant value to the equation for the reduction factor as presented in Eurocode 3. This would introduce a new curve. To avoid this introduction, it is suggested to reduce the relative slenderness $\bar{\lambda}_{MT}$ for lateral torsional buckling as given in equation (6) by adjusting the torsion term $\bar{\lambda}_T$ in this equation.

The complete newly proposed design rule is then as follows:

$$\frac{M}{\chi_{LT} \cdot M_{pl}} \leq 1.0 \qquad (17)$$

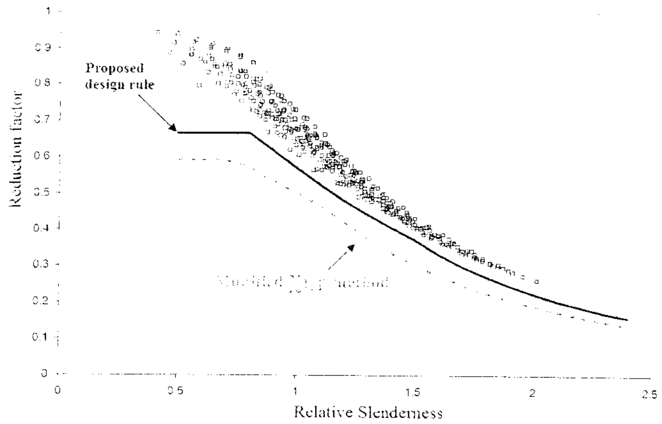


Fig. 14. Proposal for new design rule against all GMNIA results
Bild 14. Neu vorgeschlagene Bemessungsgleichung verglichen mit gesamten GMNIA Ergebnissen

The relative slenderness neglecting the effect of torsion is

$$\bar{\lambda}_M = \sqrt{\frac{M_{pl}}{M_{cr}}} \tag{18}$$

The adjusted torsion term reads:

$$\begin{aligned} \bar{\lambda}_T &= 1.0 - \bar{\lambda}_M && \text{if} \quad 0.5 \leq \bar{\lambda}_M < 0.80 \\ \bar{\lambda}_T &= 0.43 - 0.29\bar{\lambda}_M && \text{if} \quad 0.80 \leq \bar{\lambda}_M < 1.5 \\ \bar{\lambda}_T &= 0 && \text{if} \quad \bar{\lambda}_M \geq 1.5 \end{aligned} \tag{19}$$

The expression for the relative slenderness including the effect of torsion still is:

$$\bar{\lambda}_{MT} = \bar{\lambda}_M + \bar{\lambda}_T \tag{20}$$

The remaining procedure for obtaining the reduction factor follows Eurocode 3. The reduction factor for lateral torsional buckling including the effect of torsion becomes:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{MT}^2}} \tag{21}$$

where

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{MT} - 0.2\right) + \bar{\lambda}_{MT}^2\right] \tag{22}$$

with $\alpha_{LT} = 0.21$ for buckling curve 'a'.
This adaptation of the torsion term is based on the following observations.
– For relative slenderness values $\bar{\lambda}_M$ larger than 1.5, buckling curve 'a' yields reasonably good results. This allows the torsion term to be ignored in this region of the diagram, i. e. $\bar{\lambda}_T = 0$ for $\bar{\lambda}_M \geq 1.5$.
– The Modified κ_M-method gives a maximum value for the reduction factor of 0.67. Buckling curve 'a' yields a reduction factor $\chi_{LT} = 0.67$ for $\bar{\lambda}_{LT} = 1.0$. By taking the torsion term to be $\bar{\lambda}_T = 1.0 - \bar{\lambda}_M$ for $0.5 \leq \bar{\lambda}_M < 0.80$, the maximum value for the reduction factor will also be 0.67.
– For intermediate values of the relative slenderness, i. e. $0.80 \leq \bar{\lambda}_M < 1.5$, the value of the torsion term is adjusted such that a gradual transition is obtained.

The effect of adjusting the torsion term is shown in figure 14.

7 Conclusions and recommendations

The ultimate loads of five proposed design rules for channel section beams subject to eccentric loading have been compared to ultimate loads obtained from geometrical and material nonlinear analyses of imperfect (GMNIA) beams. From an extensive study it appears that:
– The Modified *Merchant-Rankine* method [1] does not always lead to safe results.
– The Modified κ_M-method [6] yields acceptable results but is based on DIN buckling curves.
– A design method using second order theory, the α_ϑ-method [7] and a Simplified design rule [13] give rather conservative ultimate loads and require trial and error calculation procedures.
– The General Method presented in Eurocode 3 gives very conservative results.

On the basis of a thorough numerical parameter study a new design rule is proposed for lateral torsional buckling of channel cross-section beams with a span length to section height ratio between 15 and 40. The newly proposed design rule is based on the Modified κ_M-method [6] and is adjusted to conform to Eurocode 3. The new design rule is derived for eccentric load application along a line through the centre of the web.

It is suggested by the authors that the new design rule can be safely applied for all vertical load application lines between the shear centre and the centre of the web.

It is recommended to investigate more eccentricities than the one corresponding to load application along a line through the centre of the web. Also, it is recommended to investigate the plastic strength of a channel section under combined bending, shear and torsion.

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Kreatives Planen und Bauen mit Stahlbau-Hohlprofilen: MSH-Wettbewerbe ausgelobt

Zum dritten Mal lobt im Jahr 2008 VALLOUREC & MANNESMANN TUBES seine MSH-Wettbewerbe für Architekten und Studenten aus. Während es bei den einzureichenden Architektenprojekten um „echte", nach 2003 realisierte Objekte geht, werden unter den Studenten der Fachrichtungen Architektur und Bauwesen Ideen für einen Aussichtsturm für die BUGA 2011 in Koblenz gesucht. In beiden Fällen entscheidet der kreative Einsatz des „Ideenprofils MSH".

In der jüngeren Vergangenheit ist die Kombination eines Architekten- mit einem Nachwuchswettbewerb bereits zweimal mit großer Resonanz von V & M DEUTSCHLAND durchgeführt worden. Bewertet wird auch in diesem Jahr wieder der kreative Einsatz der kreisförmigen, quadratischen und rechteckigen MSH-Profile. Dabei soll von den Teilnehmern die Stärke der breiten Palette an Stahlbau-Hohlprofilen ausgenutzt werden, die u. a. in der Kombination konstruktiver Ästhetik, variabler statischer Leistungsfähigkeit und werkstoffspezifischer Flexibilität liegt.

Architekten, die ein Objekt geplant haben, das nach dem 1. Januar 2003 und unter dem Einsatz von MSH-Profilen fertig gestellt wurde, sind im Rahmen des MSH-Preises für Architektur 2008 eingeladen, ihre Projektunterlagen einzureichen. Unabhängig vom Bauwerk sind sowohl nationale als auch internationale Objekte zur Teilnahme zugelassen.

Anders beim MSH-Förderpreis 2008 für Studierende der Fachrichtungen Architektur und Bauwesen: Hier sollen die Teilnehmer für die 2011 in Koblenz stattfindende BUGA einen Aussichtsturm entwerfen, der in exponierter Lage und weithin sichtbar von Autobahn und Schienenverkehr zugleich ein Entrée für die Stadt Koblenz darstellen wird. Auch hier ist Entwurfsvoraussetzung, dass MSH-Profile eine ästhetisch und technisch „tragende Rolle" spielen.

Auch bei diesem Wettewerb werden die beiden MSH-Wettbewerbe zentral über die Düsseldorfer „Bauen mit Stahl e. V." organisiert, wo sowohl die Registrierung als auch die Abgabe der Wettbewerbsbeiträge erfolgt. Nach vorheriger Anmeldung ist der Abgabetermin auf den 30. September 2008 festgelegt. Beide Wettbewerbe bewertet eine qualifizierte Jury aus Architekten, Bauingenieuren, Fachjournalisten und Anwendungstechnikern.

Für die teilnehmenden Studenten dürfte neben der Aufgabe, den Preisen und Teilnahmeurkunden besonders die Abschlussveranstaltung mit der feierlichen Preisverleihung für die Gewinner beider Wettbewerbe von Interesse sein. Dort bietet sich die Möglichkeit zum regen Informationsaustausch und Networking zwischen dem Nachwuchs und etablierten Architekten und Ingenieuren.

Weitere Informationen zum MSH-Preis für Architektur 2008 und zum MSH-Förderpreis 2008:
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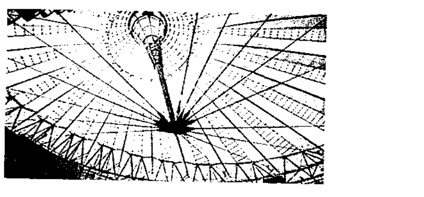

Das Dach über dem SONY Center in Berlin wird von einer ringförmigen Stahlkonstruktion aus kreisförmigen MSH-Profilen getragen