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ON THE DIMENSIONING OF STEEL CONSTRUCTION CONSIDERING REVISION OF GERMAN STANDARDS

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ABSTRACT

Different procedures for dimensioning of structural steelwork are investigated in this paper. In the non-nuclear area, there has been made a change in the design concept for structural steelwork of surface constructions in Germany and the deterministic design concept of German DIN 18800 (03/81) [1] has been replaced by the semi-probabilistic design concept of DIN 18800 (11/90) [2]. Design codes established in nuclear safety standards for steelwork in German nuclear facilities are considered in the paper. Recent revisions of German nuclear safety standards and specifications are discussed with respect to regulations concerning design codes. The change in design concept for structural steelwork (non-nuclear) has not been adopted in recent revisions of nuclear safety standards and specifications for steelwork of components with importance to safety. The deterministic concept of DIN 18800 (03/81) [1] is compared with the semi-probabilistic concept of DIN 18800 (11/90) [2]. Differences between the standards [1] and [2] are investigated using an example of a steel construction. Results for dimensioning according to DIN 18800 (03/81) [1] are compared to those for dimensioning according to DIN 18800 (11/90) [2] putting the main emphasis on stress analysis and building structure interaction loads.

Keywords: structural steelwork, dimensioning

1. INTRODUCTION

Structural steelwork shall have adequate stiffness and mechanical strength. The structural analysis shall provide that the structure, its components, connections and supports demonstrate loadbearing capacity, serviceability and static equilibrium. Steel construction and supports are needed for safe support of active and passive components in nuclear power plants. Typical examples of steel constructions are platforms, operating platforms and support stays for pipes, valves or pumps. Besides the service loads resulting from the specified normal operation, different load cases have to be considered in design of structural steelwork, like non-specified operation (e.g. turbine trip out), thermal loads (e.g. loss-of-coolant accident), construction loads or design basis accidents (e.g. earthquake, aircraft crash).

The conception of the supports of components, e.g. the conception of piping supports in a pipe system defines essentially the mechanical behaviour of the component or system to be supported. This is given for static as well as for dynamic load cases. For all service conditions like normal or abnormal operation, accidental situations and extreme incidents, the steel construction has to ensure the safe support of active and passive components, for example in case of an earthquake or an aircraft crash, where no-collapse requirements have to be fulfilled by the structural steelwork. Ultimate limit state analysis provides proof that the structure and its component parts are not a risk of failure during the construction phase, in service or in accidental situations, based on the assumption that during service life of the structure there will be no occurrences which would have an adverse effect on its stability. Base joints usually require verification of static equilibrium. To ensure the serviceability of a structure limitations may need to be enforced, e.g. with regard to deformations or impact. This may affect design, particularly in plastic-plastic analysis.

In the non-nuclear area, there has been made a change in the concept for dimensioning of structural steelwork in Germany and the deterministic design concept of German DIN 18800 (03/81) [1] was replaced by the semi-probabilistic concept of DIN 18800 (11/90) [2]. The different procedures for dimensioning of structural steelwork are investigated in this paper. The deterministic concept of German DIN 18800 (03/81) [1] is compared with the semi-probabilistic concept of DIN 18800 (11/90) [2]. Differences between the standards [1] and [2] are investigated using an example of a steel construction. Results for dimensioning according to DIN 18800 (03/81) [1] are compared to those for dimensioning according to DIN 18800 (11/90) [2] putting the main emphasis on the stress analysis and on the building structure interaction loads.

Design codes in nuclear safety standards for steelwork in German nuclear facilities are considered in the paper. Recent revisions of German nuclear safety standards and specifications are discussed with respect to regulations concerning design codes for steelwork.

2. DESIGN CONCEPTS

Table 1 shows a comparison between old design concept DIN 18800 (03/81) [1] and new concept DIN 18800 (11/90) [2]. DIN 18800 (03/81) [1] is based on a deterministic procedure. The actual stress and the allowable stress are determined using elastic theory. A global safety coefficient is considered in the calculation of the allowable stress. Actual load values are used to determine the actual stress. A design stress intensity is considered in case of combined loading. This value is usually based on distortion-energy theory. The design stress intensity σ_v shall not exceed the allowable stress σ_{zul}.

In the semi-probabilistic concept of DIN 18800 (11/90) [2] equations are implemented, which were partly obtained in experiments. The analysis is based on design values. Design values are values which action parameter and resistance parameter are assumed to have for the purpose of the analysis. They are determined with respect to unfavourable effects of actions occur in structures having an unfavourable combination of properties. More critical situations are unlikely to happen in practice. The partial safety factors γ_F and γ_M take into account variations in actions (γ_F) and resistance parameters (γ_M). Furthermore, the probability of variable actions occurring simultaneously are considered by a combination value ψ. The design stress S_d is the parameter describing the state of a structure as a result of design actions F_d. The design resistance R_d is describing the state of a structure associated with its limit states. It is calculated using the design resistance parameter M_d or determined empirically. The design stress S_d shall not exceed the design resistance R_d in the calculation. Analyses may concentrate on stresses or internal forces and moments, covering the structure as a whole or parts of it, depending on context and type of the analysis selected. Stresses may also be a function of resistance parameters, e.g. of stiffness parameters where constraint occurs in hyperstatic structures.

The analysis according to DIN 18800 (11/90) [2] can take the form of one the three methods listed in table 2. One or more of the ultimate limit states: onset of yielding, plasticizing of cross section, formation of a chain of plastic hinges or rupture shall be considered, depending on the design model selected. The elastic-elastic analysis is on basis of stresses, while elastic-plastic analysis is a study of internal forces and moments, and plastic-plastic analysis is one of actions or internal forces and moments. Actions shall be classified according to their degree of permanence, into permanent actions, variable actions and accidental actions. The rules relating to elastic-plastic and plastic-plastic analysis only apply to structural steel with a ratio of tensile strength of more than 1.2. Linear elastic material behaviour (i.e. according to Hooke's law) shall be assumed in elastic analysis, and linear elastic-ideal plastic material behaviour in plastic analysis. Strain hardening of a material may be taken into account provided this is localized.

Table 1. Comparison of old and new design concept for structural steelwork (non-nuclear) according to [9]

old design concept	**new design concept**
safety coefficient	
γ (global safety factor, resistance) $\sigma_{zul} = \beta_s / \gamma$ $(\gamma \approx \gamma_F \cdot \gamma_M)$ β_s = yield strength	γ_F (partial safety factor, actions) γ_M (partial safety factor, resistances)
determination of force and moment components	
first order theory: actual values ($\gamma = 1$) second order theory: γ - factorization	γ_F - or $\gamma_F \cdot \gamma_M$ – factorization
Analytical model	
first order theory special case: second order theory	second order theory with imperfections special case: first order theory
verification	
$\sigma_v \leq \sigma_{zul}$ σ_v = stress intensity (equivalent stress) σ_{zul} = allowable stress	$S_d / R_d \leq 1$ S_d = design stress R_d = design resistance

old standards	**new standards**
DIN 18800-1 (03/81) [1]: dimensioning/construction Dast-Ri 008 [11]: limit analysis	DIN 18800-1 (11/90) [2]: dimensioning/construction
DIN 4114-1 [12], -2 [13]: component stability (buckling, tip over)	DIN 18800-2 [16]: component stability (beams, bars)
Dast-Ri 012 [14]: buckling (plates)	DIN 18800-3 [17]: component stability (plates)
Dast-Ri 013 [15]: buckling (shells)	DIN 18800-4 [18]: component stability (shells)

Table 2. Methods of analysis according to DIN 18800 (11/90) [2]

	Method	**stresses, S_d**	**resistances, R_d**
1	elastic – elastic	elastic theory	elastic theory
2	elastic – plastic	elastic theory	plastic theory
3	plastic – plastic	plastic theory	plastic theory

A complete description of the different concepts can not be given in this paper. Therefore, only a short description of the methods is given here. Further information can be found in standards [1-3] or in literature, e.g. in [8-10].

3. NUCLEAR SAFETY STANDARDS AND SPECIFICATIONS FOR STRUCTURAL STEELWORK

The nuclear safety standard KTA 3205, part 1-3 [4-7] deals with the design of steel construction, non-integral component supports and standard supports for primary system components and secondary system components of nuclear facilities. Table 3 gives an overview. The KTA standards refer to further standards. Dimensioning of steel construction for primary system components is dealed with in KTA 3205.1 [4], [5]. The supports of secondary system components are regarded in KTA 3205.2 [6]. KTA 3205.3 [7] deals with qualification procedures for standard supports, like variable support spring hangers, shock arresters, constant support hangers or pipe supports. KTA 3205.1 (6/91) [4], KTA 3205.2 [6] and KTA 3205.3 [7] refer mainly to DIN 18800 (03/81) [1] concerning the dimensioning of the steel parts and their connections. The revised KTA 3205.1 (6/02) [5] refers additionally to DIN 18800 (11/90) [2] (with limitations).

Table 3. Nuclear safety standards KTA 3205, part 1-3 [4-7] for steel components and non-integral supports

KTA	revision	steel construction for	Subject	refers to	revision
3205.1 [4]	06/91	primary system components	non-integral supports	DIN 18800 [1]	03/81
3205.1 [5]	06/02	primary system components	non-integral supports	DIN 18800 [1] * DIN 18800 [2] ** EC 3 [3] ***	03/81 * 11/90 ** 04/93***
3205.2 [6]	06/90	secondary system components	non-integral supports	DIN 18800 [1]	03/81
3205.3 [7]	06/89	primary and secondary system components	standard supports	DIN 18800 [1]	03/81

* DIN 18800 (03/81) [1] partly included in [5] as appendix "E"

** dimensioning according to DIN 18800 (11/90) [2] only in isolated case and only with approval of the authorized inspection agency

*** EC 3 [3] mentioned only informatively in [5] in appendix "G"

Plant-specific conventions have been established in the KS D-specifications for components in nuclear facilities in the eighties. The nuclear safety standards KTA are considered and precised in the KS D-specifications. Furthermore project-specific conventions have been made. Table 4 gives an overview about the KS D-specifications for the dimensioning of steel components and supports. The KS D standards are revised at planned intervals considering the state of the art. One aspect that was discussed during the last revision made in 2004 was the ending of the transition-period for the introduction of DIN 18800 (11/90) [2] in the conventional, non-nuclear field.

Table 4. KS D-specifications for dimensioning of steel components

KS D – No.	Subject	revision / date	refers mainly to	revision / date	refers mainly to
4570/50	rules for composition and classification of the KS D-specifications	C 09/93 [19]	KTA 3205.1 [4] KTA 3205.2 [6]	D 06/04 [20]	KTA 3205.1 [5] * KTA 3205.2 [6]
4571.1/50	pipe-supports S1 ****	0 09/94 [21]	DIN 18800 (03/81) [1] DIN 18800 (11/90) [2]** KTA 3205.1 [4]	A 06/04 [22]	KTA 3205.1 [5] * DIN 18800 (11/90) [2]**
4572/50	structural steelwork S2 and S3 with external / internal impacts	A 05/94 [23]	DIN 18800 (03/81) [1] DIN 18800 (11/90) [2]** KTA 3205.2 [6] ***	B 07/04 [24]	KTA 3205.2 [6] *** DIN 18800 (11/90) [2]**
4572.1/50	pipe supports S2 and S3 with external / internal impacts	A 05/94 [25]	KS D 4572/50 [23] KTA 3205.2 [6]	B 07/04 [26]	KS D 4572/50 [24] KTA 3205.2 [6]
4572.2/50	component supports S2 and S3 with external / internal impacts	A 05/94 [27]	KS D 4572/50 [23]	B 07/04 [28]	KS D 4572/50 [24]
4572.3/50	structures for safety measures and customized constructions S2 and S3 with external / internal impacts	A 05/94 [29]	KS D 4572/50 [23]	B 07/04 [30]	KS D 4572/50 [24]

* DIN 18800 (03/81) [1] partly included in [5] as appendix "E"

** dimensioning according to DIN 18800 (11/90) [2] only in isolated case and only with approval of the authorized inspection agency

*** refers to DIN 18800 (03/81) [1]

**** S = structural steelwork class

The hitherto valid KS D-specifications refer to DIN 18800 (03/81) [1] which will become invalid in the conventional, non-nuclear area (e.g. for bridge construction, steel construction for hydraulic engineering, crane gentry construction) with an European standard. Further reasons that made a revision of the KS D-specifications necessary were a number of changes in material codes, in the requirements to be met by manufacturers and the introduction of new DIN EN standards in the field of materials and manufacturing. In those parts of the KS D-specifications concerning calculation and design of components, changes in the referred version of the DIN 18800 were made if they had no effect on the dimensioning of steelwork respectively supports. In those sections ruling dimensioning of steel components, references to DIN 18800 (03/81) [1] concerning allowable stress, equivalent stress and so on were therefore replaced by references to the appropriate KTA-standard. When KTA-standard 3205.1 [5] was last revised in 2002, the chapters of DIN 18800 (03/81) [1] dealing with the dimensioning of steel construction were included into the KTA-standard itself as appendix “E”. References to DIN 18800 (03/81) [1] in the KS D-specification without influence on stress analysis of steel components (i.e. descriptive terms, proof of position permanence, buckling and constructive rules like placement of bolts) were changed to point to the appropriate section of the new DIN 18800 (11/90) [2]. Some of these changes had already been made in the previous revision of the KS D-specifications. References to merely descriptive terms that had no equivalent in the DIN 18800 (11/90) [2] were deleted. Table 4 gives an overview about on which standard dimensioning is based in KS D-specifications. The KS D 4570/50 [20] deals with classification of components and description of the composition of the KS D-specifications. The KS D-specifications still comply with the guidelines of the DIN 18800 (03/81) [1]. However, in the KS D-specifications as well as in the revised KTA-standard 3205.1 [5] paragraphs are implemented, that are giving possibility of using new DIN 18800 (11/90) [2] in isolated case if there is approval of the authorized inspection agency.

4. EXAMPLE

4.1 Static system

An example of a platform made of steel is presented in order to compare results obtained with the two different design concepts of DIN 18800 (03/81) [1] and DIN 18800 (11/90) [2]. The platform has the shape of a semicircle with a radius of approximately 4,80 m (Fig. 1). The calculations were performed with RSTAB [32].

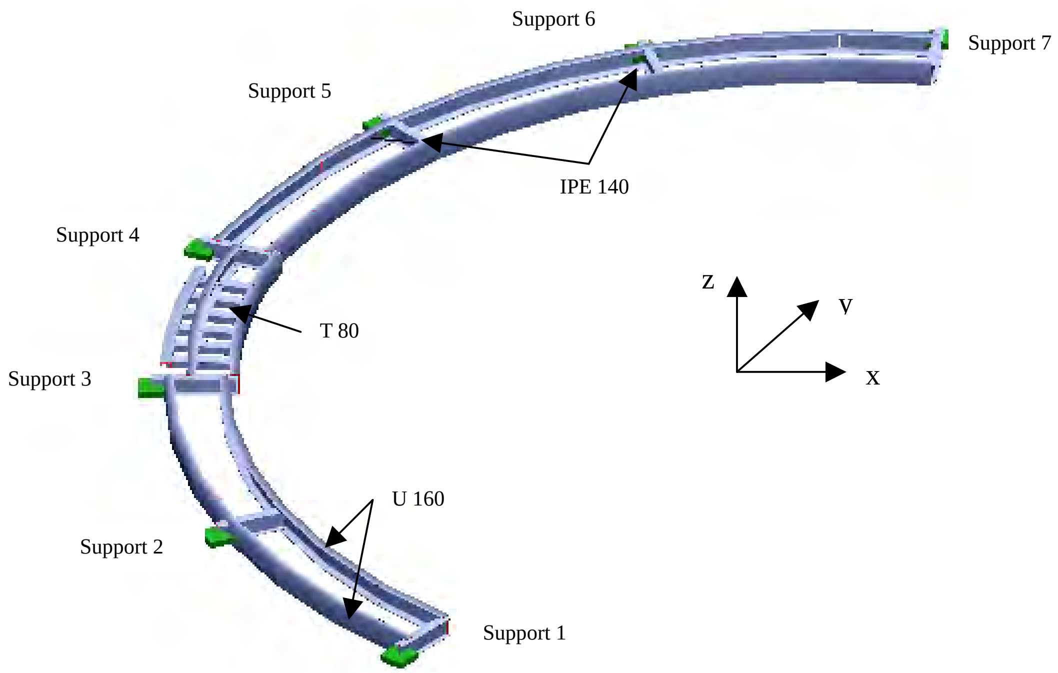

Fig. 1: Rendered view of the steel platform

The main parts of the supporting structure are made of profiles U 160 and IPE 140 from steel St 37-2 respectively S235JR. The U 160 transmit the actions of pipes and walkway on the IPE 140. The IPE 140 is a cantilever beam which is fastened to the building's wall with special anchors. In the middle area of the structure, T 80 profiles connected with the U 160 are used as pipe support stays. The structure's connections are welded.

4.2. Loading

Table 5 gives an overview about the actions on the platform. The actions in case of earthquake (load combination (LC) HS, AK) are shown in brackets. The Z-direction is contrary to the direction of the dead load.

Table 5. Actions on the platform

load combination	**x-direction**	**y- direction**	**z- direction**
dead load of the structure LC 1 (LC1S)	load is determined in computer program	load is determined in computer program	load is determined in computer program
dead load walkway LC 2 (LC2S)	0,0 / (±0,1) kN/m²	0,0 / (±0,14) kN/m²	-0,5 (-0,56, -0,45) kN/m²
dead load cable channel LC 3 (LC3S)	0,0 / (±0,4) kN	0,0 / (±0,54) kN	-2,0 / (-2,22, -1,78) kN
live load on the structure LC 4 (LC4S)	0,0 / (±1,0) kN/m²	0,0 / (±1,4) kN/m²	-5,0 / (-5,55, -4,45) kN/m²
pipe support stay 1 LC 5 (LC5S)	-0,11 / (-0,22) kN	-0,6 / (-0,88) kN	-0,99 / (-1,1) kN
pipe support stay 2 LC 6 (LC6S)	-0,11 / (-0,44) kN	-0,66 / (-0,77) kN	-1,43 / (-2,97) kN
pipe support stay 3 LC 7 (LC7S)	-0,11 / (-0,11) kN	-0,88 / (-1,21) kN	-0,27 / (0,0) kN
pipe support stay 4 LC 8 (LC8S)	-0,11 / (-0,11) kN	-0,44 / (-0,55) kN	-1,21 / (-1,32) kN
pipe support stay 5 LC 9 (LC9S)	-0,11 / (-0,11) kN	-0,44 / (-0,66) kN	-1,32 / (-1,32) kN
pipe support stay 6 LC 10 (LC10S)	-0,11 / (-0,11) kN	-0,33 / (-0,55) kN	-1,32 / (-1,32) kN
pipe support stay 7 LC 11 (LC11S)	-0,11 / (-0,11) kN	-0,44 / (-0,55) kN	-1,21 / (-1,32) kN
pipe support stay 8 LC 12 (LC12S)	0,2 / (0,3) kN	-0,7 / (0,0) kN	-1,2 / (-1,3) kN

As an example, the load cases LC 5 – 11 are shown in the structure detail below (Fig.2).

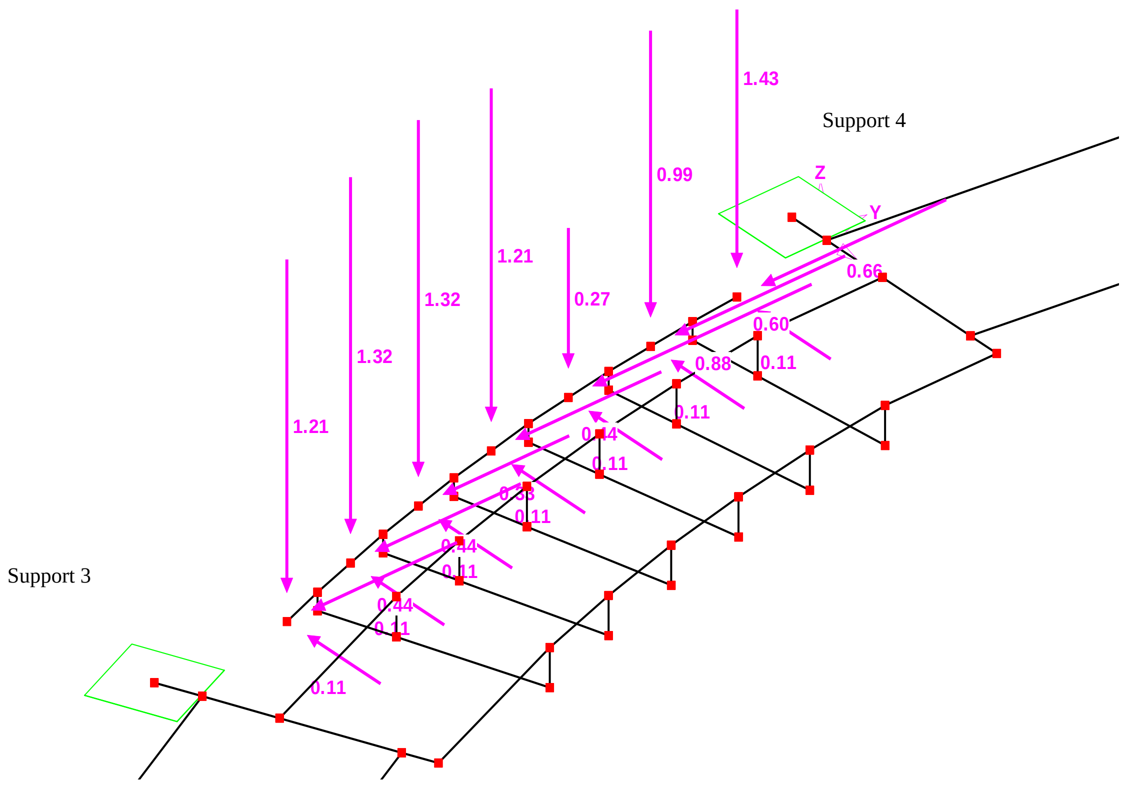

Fig. 2. Load cases LC 5-11

4.3 Verification analysis according to DIN 18800 (03/81) [1]

Load combinations (LC) H (permanent loads) and HS (permanent and accidental loads) are made by combination of the load cases above in the verification analysis. The dead loads, the live loads and the pipe loads are combined like shown below in load combination H .

LC H = LC1 + LC2 + LC3 + LC4 + LC5 + LC6 + LC7 + LC8 + LC9 + LC10 + LC11 + LC12

Two combinations will be considered in LC HS. First of all the combination of all actions under earthquake acceleration. Secondly the combination of dead loads under earthquake acceleration. Earthquake acceleration is considered as ± - acting in horizontal direction. In load combination HS_I, earthquake acceleration is regarded in negative z-direction and in LC HS_{II} in positive z-direction.

LC HS_I = LC1S + LC2S + LC3S+ LC4S + LC5S + LC6S + LC7S + LC8S + LC9S + LC10S + LC11S + LC12S

LC HS_{II} = LC1S + LC2S + LC3S

The stresses will be calculated by means of the profile cross section values. A comparison of maximum calculated stresses and allowable stresses is given in table 6-8. Thus the equivalent stress is not inevitably determined by the shown axial stresses and shear stresses. In this example the allowable axial stress is 16,0 kN/cm² and the allowable shear stress is 9,2 kN/cm² in LC H. The allowable axial stress is 23,5 kN/cm² and the allowable shear stress is 13,4 kN/cm² in LC HS.

Table 6. Comparison of calculated and limit stress in profile IPE 140

profile	load combination		calculated stress [kN/cm²]	limit stress [kN/cm²]	stress usage factor [%]
IPE 140	LC H	axial stress	5,9	16,0	36,9
		shear stress	4,1	9,2	44,6
		equivalent stress	7,2	16,0	45,0
	LC HS	axial stress	8,2	23,5	34,9
		shear stress	6,1	13,4	45,5
		equivalent stress	10,7	23,5	45,5

Table 7. Comparison of calculated and limit stress in profile U 160

profile	load combination		calculated stress [kN/cm²]	limit stress [kN/cm²]	stress usage factor [%]
U 160	LC H	axial stress	4,1	16,0	25,6
		shear stress	1,3	9,2	14,1
		equivalent stress	4,2	16,0	26,3
	LC HS	axial stress	5,3	23,5	22,6
		shear stress	1,7	13,4	12,7
		equivalent stress	5,4	23,5	23,0

Table 8. Comparison of calculated and limit stress in profile T 80

profile	load combination		calculated stress [kN/cm²]	limit stress [kN/cm²]	stress usage factor [%]
T 80	LC H	axial stress	5,9	16,0	36,9
		shear stress	1,0	9,2	11,1
		equivalent stress	5,9	16,0	36,9
	LC HS	axial stress	9,1	23,5	38,7
		shear stress	1,8	13,4	13,4
		equivalent stress	9,1	23,5	38,7

4.4 Verification analysis of the ultimate limit state according to DIN 18800 (11/90) [2]

In DIN 18800 (11/90) [2] a new design concept is brought into action as mentioned above where design values of actions (S_d) and resistances (R_d) are used to verify the ultimate limit state.

The design values contain the partial safety factor γ_F for actions and γ_M for resistances. The partial safety factors γ_F and γ_M make allowance for the variations in actions, F, and resistance parameters, M. It shall be verified that the stresses, S_d, do not exceed the resistances, R_d. Design stresses shall be calculated using the design values of actions, F_d. The appropriate combinations of actions shall be formed from permanent and variable actions.

The following actions shall be considered in combination for the ultimate limit state analysis:

- permanent actions G, together with all unfavourable variable actions, $Q_{i;}$
- permanent actions G, together with each in turn of the unfavourable variable actions, Q_i.

If there is an accidental action F_A like earthquake or vehicle impact, F_A shall be added to the combinations above. In the shown example, four combinations of actions (GK) and two combinations with accidental actions (AK) have to be considered. As a simplification, the internal forces and moments of the pipes (LC5 – LC12) shall be treated as variable actions.

GK1 = 1,35*(LC1 + LC2 + LC3)

GK2 = 1,35*(LC1 + LC2 + LC3) + 1,5*0,9*(LC4 + LC5 + LC6 + LC7 + LC8 + LC9 + LC10 + LC11 + LC12)

GK3 = 1,35*(LC1 + LC2 + LC3) + 1,5*LC4

GK4 = 1,35*(LC1 + LC2 + LC3) + 1,5*(LC5 + LC6 + LC7 + LC8 + LC9 + LC10 + LC11 + LC12)

One of the accidental combinations is made like GK2 under consideration of negative earthquake acceleration (z-direction). The other is made like GK1 under consideration of positive earthquake acceleration (z-direction). Earthquake acceleration is considered as ± - acting in horizontal direction.

AK1 = 1,0*(LC1S + LC2S + LC3S+ LC4S + LC5S + LC6S + LC7S + LC8S + LC9S + LC10S + LC11S + LC12S)

AK2 = 1,0*(LC1S + LC2S + LC3S)

The ultimate limit state analysis is made by the elastic – elastic method. The design resistance parameter M_d is calculated by dividing the characteristic resistance parameter M_k by the partial safety factor γ_M. In the example a steel S235JR is used. The yield strength of the material is 24,0 kN/cm² and γ_M being equal to 1,1. The design value of the limit of axial stress is:

$$\sigma_{R,d} = fy_{,d} = 24{,}0 \text{ kN/cm}^2 / 1{,}1 = 21{,}82 \text{ kN/cm}^2.$$

The design value of the limit of shear stress is:

$$\tau_{R,d} = \sigma_{R,d} / \sqrt{3} = 21{,}82 / \sqrt{3} = 12{,}6 \text{ kN/cm}^2.$$

In table 9 to 11 a comparison is given for the maximum existing stresses and the limit stresses of the profiles. Thus the equivalent stress is not inevitably determined by the shown axial stresses and shear stresses.

Table 9: Comparison of calculated and limit stress in profile IPE 140

profile	**load combination**		**load case**	**calculated stress [kN/cm²]**	**limit stress [kN/cm²]**	**stress usage factor [%]**
IPE 140	LC GK	axial stress	GK2	7,9	21,82	36,2
		shear stress	GK4	6,1	12,6	48,4
		equivalent stress	GK4	10,5	21,82	48,1
	LC AK	axial stress	AK1	8,1	21,82	37,1
		shear stress	AK1	6,1	12,6	48,4
		equivalent stress	AK1	10,7	21,82	49,0

Table 10: Comparison of calculated and limit stress in profile U 160

profile	**load combination**		**load case**	**calculated stress [kN/cm²]**	**limit stress [kN/cm²]**	**stress usage factor [%]**
U 160	LC GK	axial stress	GK2	5,6	21,82	25,7
		shear stress	GK2	1,7	12,6	13,5
		equivalent stress	GK2	5,6	21,82	25,7
	LC AK	axial stress	AK1	5,3	21,82	24,3
		shear stress	AK1	1,7	12,6	13,5
		equivalent stress	AK1	5,4	21,82	24,7

Table 11: Comparison of calculated and limit stress in profile T 80

profile	load combination		load case	calculated stress [kN/cm²]	limit stress [kN/cm²]	stress usage factor [%]
T 80	LC GK	axial stress	GK4	8,3	21,82	38,0
		shear stress	GK4	1,4	12,6	11,1
		equivalent stress	GK4	8,3	21,82	38,0
	LC AK	axial stress	AK1	9,1	21,82	41,7
		shear stress	AK1	1,8	12,6	14,3
		equivalent stress	AK1	9,1	21,82	41,7

4.5 Comparison of results

The stresses and stress usage factors obtained with the two design concepts are compared to each other in table 12. As expected, the stresses which result of LC GK are higher than the results of LC H, because of the partial safety factor γ_F = 1,35 / 1,50. Though, the profile efficiencies are not automatically higher. For example, the U 160 shows higher stresses, but lower profile efficiency in LC GK than in LC H. The stresses of the LC HS and the accidental combinations are equal. Here, the same actions are combined which each other and the partial safety factor for the accidental combinations is γ_f = 1,0. This means higher stress usage factor by using the combination according to DIN 18800 T.1 (11/90) [2]. This is determined by reduced yield strength based on the partial safety factor of steel γ_M = 1,1. Thus the maximum actions for the steel parts depend on the concept being used for design calculations.

Table 12. Comparison of maximum equivalent stress σV and stress usage factor

profile	load combination		DIN 18800 (03/81) [1]	DIN 18800 (11/90) [2]
IPE 140	LC H bzw. GK	max. σ_V [kN/cm²]	7,2	10,5
		max. stress usage factor [%]	45,0	48,1
	LC HS bzw. AK	max. σ_V [kN/cm²]	10,7	10,7
		max. stress usage factor [%]	45,5	49,0
U 160	LC H bzw. GK	max. σ_V [kN/cm²]	4,2	5,6
		max. stress usage factor [%]	26,3	25,7
	LC HS bzw. AK	max. σ_V [kN/cm²]	5,4	5,4
		max. stress usage factor [%]	23,0	24,7
T 80	LC H bzw. GK	max. σ_V [kN/cm²]	5,9	8,3
		max. stress usage factor [%]	36,9	38,0
	LC HS bzw. AK	max. σ_V [kN/cm²]	9,1	9,1
		max. stress usage factor [%]	38,7	41,7

4.6 Support actions

Figure 1 gives an overview about the structure regarded in this example. The platform is fixed by seven supports. One of these supports is shown in Fig. 3 with its local coordinate system. Besides the verification analysis of the steel structure the determination of the support actions is very important. The calculated support actions are used to verify the anchorage of the steel components at the building structure. Table 13 and 14 present the comparison of the support actions (forces and moments) of load combinations (LC) H and GK1-4. A minimum and a maximum support action are calculated for every direction because of the four load combinations GK1-4 in this example. In LC H there is only one force and one moment per direction.

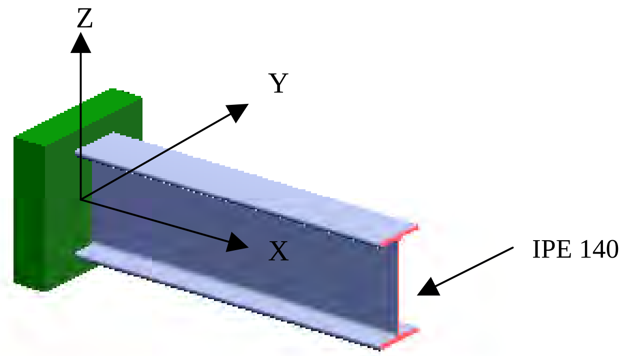

Fig 3. Example of support / connection to building with local coordinate system

In this example there is no difference between the support actions of LC HS and the accidental combinations LC AK. In both the same actions are combined and the partial safety factor for the accidental combination is $\gamma_f = 1{,}0$. It is obvious, that the support actions depend on the design concept used for the calculation. The support actions obtained with the new DIN 18800 (11/90) [2] are higher (except in LC HS) than those determined with the old design concept of DIN 18800 (03/81) [1]. This is due to the partial coefficients γ_F in the new design concept. This is of importance, if support actions of components with importance to safety are determined according to the old design concept and the stress analysis of anchorage and building are determined with the new design concept for concrete structures, DIN 1045-1 (2001) [31]. This design concept is as well based on partial safety factors.

Table 13. Comparison of support actions of load combination H resp. load combination GK1-4

		F_x [kN]		**F_y [kN]**		**F_z [kN]**	
		DIN 18800 (03/81) [1]	DIN 18800 (11/90) [2]	DIN 18800 (03/81) [1]	DIN 18800 (11/90) [2]	DIN 18800 (03/81) [1]	DIN 18800 (11/90) [2]
support 1	max.		0,0		0,01		-0,81
	min.	-0,01	-0,02	-0,02	-0,04	-2,01	-2,91
support 3	max.		0,01		0,07		-2,76
	min.	-0,42	-0,64	-0,89	-1,38	-9,01	-12,29
support 5	max.	0,74	1,12		0,0		-6,27

	min.		0,0	-1,42	-2,14	-10,72	-14,48

Table 14. Comparison of support moments of load combination H resp. load combination GK1-4

		M_x [kNm]		**M_y [kNm]**		**M_z [kNm]**	
		DIN 18800 (03/81) [1]	DIN 18800 (11/90) [2]	DIN 18800 (03/81) [1]	DIN 18800 (11/90) [2]	DIN 18800 (03/81) [1]	DIN 18800 (11/90) [2]
Support 1	max.		0,0	0,56	0,82	0,0	0,0
	Min.	-0,01	-0,01		0,24		-0,01
Support 3	max.		-0,01	1,94	2,67		0,01
	Min.	-0,01	-0,01		1,01	-0,08	-0,14
Support 5	max.	0,03	0,04	3,56	5,23		0,0
	Min.		0,01		2,71	-0,15	-0,23

5. CONCLUSIONS

The example presented shows that differences in the results (stress analysis and support actions) occur between the calculations performed according to DIN 18800 (03/81) [1] and DIN 18800 (11/90) [2]. Concerning the support actions, a procedure for conversion would be recommended, if the support actions are determined according to old design concept and the stress analysis of anchorage and building are determined with new design concept.

The summary and the comparison of the design concepts (non-nuclear), the nuclear safety standards and specifications for structural steelwork and pipe supports show, that the change in the design concept for structural steelwork, where the deterministic design concept of DIN 18800 (03/81) [1] has been replaced by the semi-probabilistic design concept of DIN 18800 (11/90) [2] in the non-nuclear field, has not been adopted in recent revisions of nuclear safety standards and specifications for steelwork of components with importance to safety. The usage of the new design concept has been restricted and limited to isolated cases and the approval of the authorized inspection agency will be necessary.

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