DESIGN OF REINFORCED AND PRESTRESSED CONCRETE STRUCTURES IN NUCLEAR FACILITIES REGARDING THE NEW GERMAN STANDARDS

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ABSTRACT

The current German standards and regulations for civil works of nuclear facilities are based on obsolete versions of German design standards DIN 1045 and DIN 1055. Corresponding to the new generation of German standards DIN 1045 and DIN 1055 based on the European standards Eurocode and Eurocode 2 a revision of the respective nuclear technical standards and regulations has become necessary. With regard to an adjustment to these new standards DIN 1045 and DIN 1055 KTA-recommendations have been developed (KTA-report KTA-GS-78), which result in a revision of DIN 25449. DIN 25449 represents the basis for the design of reinforced and prestressed concrete structures in nuclear facilities. In the revised DIN 25449 published in 2006 the main topics are given by the external and internal incidents, the consideration of the partial safety concept and the specified design approaches.

INTRODUCTION

Since 2001 new versions of German standards for actions and for different construction materials have been published. For the design of new reinforced and prestressed concrete structures now the new DIN 1045-1[1] is obligatory. Like the other material specific standards DIN 1045-1[3] is based on a uniform partial safety concept according to the new generation of European codes. This safety concept as given in DIN1055-100[22] considers different partial safety factors for the actions as well as for the material properties. Furthermore essential differences between the new DIN 1045-1[3] and its previous standard are given by the possible application of non-linear design analysis, the modified punching shear design and the increased requirements with respect to durability.

These new design standards require an adjustment of regulations and standards concerning nuclear facilities. For such an adjustment the KTA-committee developed some recommendations which resulted in the KTA-report KTA-GS-78[29]. In addition to the KTA-report KTA-GS-78[28] the new standard DIN 25449[25] represents an essential design basis for reinforced and prestressed concrete structures in nuclear facilities.

STANDARDS FOR CIVIL WORKS OF NUCLEAR FACILITIES

The new generation of European standards

As shown in Fig. 1 the outdated German standards for the design of reinforced and prestressed concrete structures have been replaced by new standards regarding a transitional period up to the end of 2004. Corresponding to the new generation of European standards like Eurocode[23] and Eurocode 2[24] the new versions of German standards have been released: DIN 1045 “Concrete, reinforced concrete and prestressed concrete structures” with the different parts from 1 to 4 [3 – 7] and DIN 1055 “Actions on structures” with the different parts form 1 to 10 and part 100 [12 – 22].

The comparison between the outdated and the new German standards shows the following essential differences (see Fig. 2):

- Instead of a global safety concept (one global safety factor γ) a partial safety concept will be used in the new standards. This safety concept considers different partial safety factors for the actions (γ_E) as well as for the material properties of steel and concrete (γ_R respectively γ_C for concrete and γ_S for steel).
- According to the new DIN 1045 non-linear analyses are allowed to be used for the design alternatively to linear analyses. Thus the structural behavior of statically undetermined structures can be analyzed in a realistic manner using non-linear material laws for steel and concrete [30].
- Regarding indirect actions like temperature E_T reduced section stiffness can be taken into account by application of linear analyses. On condition that structural failure will not be influenced by E_T the outdated DIN 1045 additionally provides a reduction of the safety factor. Such a reduction is not allowed according to the new DIN 1045 and so for the consideration of E_T non-linear analyses seem to be recommendable for an economic design.
- In the new DIN 1045 demands on durability are higher than those of the outdated DIN 1045. Exemplary for the concrete cover an allowance of 15 mm instead of 10 mm has to be taken into account.

outdated standards [1, 2, 8 - 11]	**new standards** [3 – 7, 12 - 22]
DIN 1045 (1988) Reinforced concrete structures **DIN 4227 (1988)** Prestressed concrete structures	**DIN 1045-1 (2001)** b) Concrete, reinforced concrete and prestressed concrete structures part 1: design ⇔ Eurocode 2 [24]
DIN 1055 (1971 - 1976) a) Design loads for buildings	**DIN 1055-100 (2001)** c) Actions on structures part 100: basis of design, safety concept and design rules ⇔ Eurocode [23]

8 / 2002 ⟷ transitional period **12 / 2004**

a) several parts: DIN 1055-1...4

b) several parts: DIN 1045-1...4, DIN EN 206-1

c) several parts: DIN 1055-1...10, 100

Fig. 1 The development of the German design standards

outdated standards [1, 2, 8 - 11]	**new standards** [3 – 7, 12 - 22]
DIN 1045 (1988) DIN 4227 (1988) DIN 1055 (1971 - 1976)	DIN 1045-1 (2001) DIN 1055-100 (2001)
global safety concept $\gamma \cdot \sum E_i \leq \sum R_k$	**partial safety concept** $\sum (\gamma_{E,i} \cdot E_i) \leq \sum (R_k / \gamma_{R,k})$
design analyses linear	**design analyses** linear / non-linear
indirect action E_T - red. stiffness ⇒ red E_T a) - red. safety ⇒ red E_T / γ b)	**indirect action E_T** - red. stiffness ⇒ red E_T a)
a) using linear analyses b) on condition that structural failure will not be influenced by E_T	**durability** higher requirements (e. g. concrete cover)

Fig. 2 The new generation of German design standards

Adjustment to the new German standards

Because the new standards DIN 1045[3 - 7] or DIN 1055[12 - 22] are obligatory since 2005 the German regulations and standards for the design of nuclear technical buildings have to be revised. Especially the German “KTA-rules” need an adjustment to the new partial safety concept given in the new standards. This partial safety is a common basis for concrete structures as well as for steel structures. So in addition to the standards for reinforced and prestressed concrete also the standards for steel structures have to be considered.

For the transitional period up to a complete adjustment of all relevant standards the KTA-report KTA-GS-78[29] has been developed. The recommendations of this report can be used for the non–adjusted standards to fulfil the new design requirements as well as for the adjustment of these standards.

With respect to these recommendations a revision of DIN 25449[25] dealing with the reinforced concrete design of nuclear power plants subjected to internal incidents has become necessary. The new standard DIN 25449 [26] is based on DIN 1045-1[3] and DIN 1055-100[22] and regards external incidents as well as internal incidents. Furthermore prestressed concrete is considered. The essential topics of this new standard are given as follows:

- Definition of the internal and external accidental actions which are relevant for nuclear facilities like piping rupture (pressure build-up, jet loads, temperature effects), airplane crash, earthquake, explosion pressure wave or site flooding,
- Specifications of the safety concept: partial safety and combination factors for the actions of the design situations and partial safety factors for the design resistance depending on different requirements,
- Calculation methods and verification methods,
- Design advices including specifications for the values of strength, shear design and punching shear resistance.

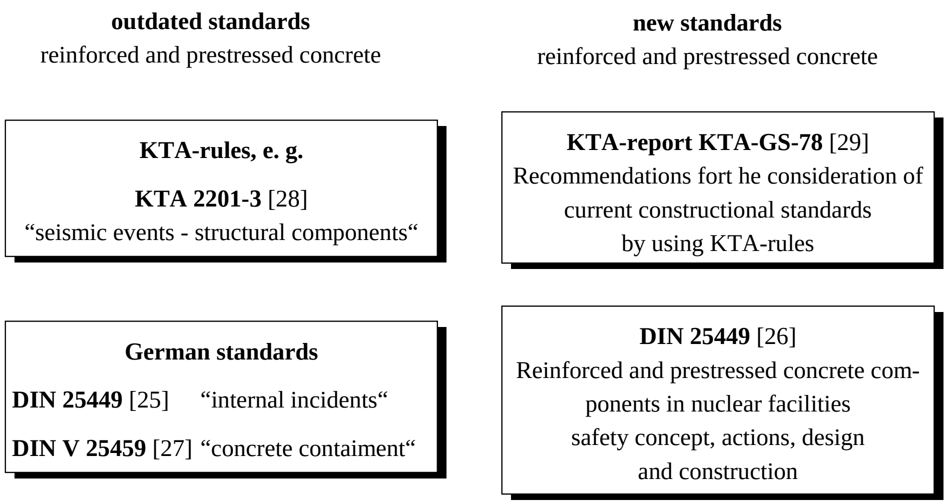

Fig. 3 Adjustment of the nuclear engineering standards to the new standards of civil engineering

SAFETY CONCEPT

Design procedure

The new generation of European standards and German standards alike is based on a partial safety concept to verify ultimate limit states (ULS) and serviceability limit states (SLS). The verifications of the ultimate limit states are necessary to avoid the structural failure whereas the verifications of the serviceability limit states contribute to guarantee the durability and the utility of buildings (see Fig. 4).

For verifications of the ultimate limit states linear analyses may be applied as well as non-linear analyses. Using linear analyses a section design with the resulting internal forces deliver the reinforcement quantities. Such a section design is also possible using non-linearly calculated internal forces. An alternative design procedure results from an ultimate load analysis where the ultimate load represents the global safety of a structure [30].

The verifications of serviceability limit states aim at the limitation of crack width, displacements and stresses. Generally these values will be calculated linearly.

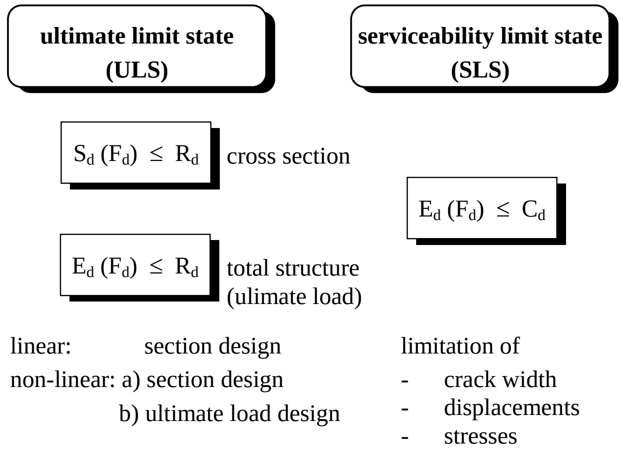

Fig. 4 Design limit states

Actions and combination rules

Corresponding to DIN 1055-100[22] the following different actions have to be considered:

- independent permanent actions G_k,
- independent actions due to prestressing P_k,
- dominant independent variable actions Q_{k1},
- other independent variable action Q_{ki} (i > 1),
- accidental actions A_d,
- seismic actions A_{Ed}.

In opposite to other actions (described by characteristic values; index k) seismic actions or accidental actions may be derived from design values (index d). So the design values of seismic actions and accidental action imply a partial safety factor of 1.0. For the design of nuclear facilities accidental actions (including seismic actions) will be separated into internal and external incidents.

Internal incidents result from rupture or leaks of high pressurized components (for instance vessels or pipes). Therefore pressure build-up or jet loads with specified load-time-histories have to be considered. Additionally temperature effects, anchorage forces or impact forces can be relevant.

The different external incidents are subdivided in seismic actions, airplane crash, explosion pressure wave and site flooding. Corresponding to DIN 25449 [26] in combination with the relevant KTA-rules these actions will be defined regarding to a very low level of occurrence rate. So an occurrence rate of $1 \cdot 10^{-5}$/a for the design earthquake or $1 \cdot 10^{-4}$/a for site flooding will be assumed. The airplane crash is defined by a load-time-history-function with a peak value of 110 MN (impact area of 7 m^2) and for the explosion pressure wave a load-time-history-function with a peak value of 45 kN/m^2 has to be considered [26].

With respect to the different limit states of the design procedure the actions have to be combined differently. The verifications of the ultimate limit states result in the following combinations of actions:

- permanent and temporary design situation:

$$E_d : \quad \gamma_G \bullet G_k + \gamma_P \bullet P_k + \gamma_{Q,1} \bullet Q_{k1} + \Sigma(\gamma_{Q,i} \bullet \psi_{0,i} \bullet Q_{ki}) \tag{1}$$

- accidental design situation:

$$E_{dA} : \quad G_k + P_k + A_d + \psi_{1,1} \bullet Q_{k1} + \Sigma(\psi_{2,i} \bullet Q_{ki}) \tag{2}$$

- seismic action:

$$E_{dAE} : \quad G_k + P_k + A_{Ed} + \Sigma(\psi_{2,i} \bullet Q_{ki}) \tag{3}$$

For the serviceability limit states (limitation of crack width, displacements and stresses) the following combinations of actions have to be considered:

- rare combination:

$$E_{d,rare} : G_k + P_k + Q_{k1} + \Sigma(\psi_{0,i} \bullet Q_{ki}) \tag{4}$$

- frequent combination:

$$E_{d,frequ} : \quad G_k + P_k + \psi_{1,1} \bullet Q_{k1} + \Sigma(\psi_{2,i} \bullet Q_{ki}) \tag{5}$$

- quasi-permanent combination:

$$E_{d,perm} : \quad G_k + P_k + \Sigma(\psi_{2,i} \bullet Q_{ki}) \tag{6}$$

Safety coefficients

The partial safety factors for prestressing, permanent and temporary action generally can be assumed according to DIN 1045-1[3]: γ_P = 1.00 (favourable or unfavourable effects), γ_G = 1.35 (favourable effects: 1.00) and γ_Q = 1.50 (favourable effects: 0). Detailed information about the partial safety coefficients and additionally recommended combination values are given in Table 1.

Table 1 Reference partial safety and combination values

actions Q_k		partial safety factor γ_G, γ_Q	combination value ψ_0	ψ_1	ψ_2
G	dead load	1.35[a]	—	—	—
variable actions Q	quasi-permanent imposed loads	1.50[b]	1.0	1.0	1.0
	variable imposed loads	1.50[b]	0.9	0.8	0.8
	crane loads	1.35	1.0	0.9[d]	0
	indirect actions (settlements)	1.50[c]	1.0	1.0	1.0

a 1.00 for favourable effect of action

b 1.35 for extremly ascertainable actions

c 1.00 using linear analysis (considering reduced stiffness)

d for requirement category A3 crane loads have not be considered as variable loads: $\psi_1 = 0$

Table 2 Partial safety factors of resistance (ULS)

requirement categories	**A1**	**A2**	**A3**
materials and resistances			
partial safety factor of concrete	γ_c = 1.50	γ_c = 1.3	γ_c = 1.0
partial safety factor reinforcement / prestr. steel	γ_s = 1.15	γ_s = 1.0	γ_s = 1.0
non-linear analysis			
structural resistance [1)] according DIN 1045-1 section 8.5.1	γ_R = 1.30	γ_R = 1.1	γ_R = 1.0
design data concrete compressive strength f_{cR}[2)]	0.85 α f_{ck}	0.85 α f_{ck}	1.0 α f_{ck}
design data steel yield strength f_{yR}	1.1 f_{yk}	1.1 f_{yk}	1.0 f_{yk}
design data 0.1 %-proof stress prestr. steel $f_{p0.1R}$	1.1 f_{pk}	1.1 f_{pk}	1.0 f_{pk}

1) system-resistance

2) reduction coefficient α according DIN 1045-1

The safety coefficients of the structural resistance depend on the different design limit states. For limit states of serviceability the safety coefficients of the structural resistance may be assumed to 1.0.

For the ultimate limit states the consideration of internal and external incidents requires three different design requirements given by the categories A1, A2 and A3. With respect to the KTA-report KTA-GS-78[29] these categories depend on different combinations of actions and are defined for reinforced and prestressed concrete structures as well as for steel structures:

A1: combinations of actions which rank among permanent and temporary design situations of DIN 1055-100[22]

A2: combinations of accidental actions with a repeated occurrence during the service life

A3: combinations of accidental actions with a minor probability of occurrence ($\leq 10^{-4}$ /a; actions with a single occurrence during the service life)

Depending on these requirement categories the partial safety factors of resistance are defined as shown in Table 2. Considering these different partial safety factors the ultimate limit states have to be verified in such a way that the design values of the action effects will not exceed the design values of the structural resistance. These verifications can be carried out by linear or nonlinear design analyses.

In case of linear design analyses generally a section design shall be applied comparing the acting and resisting internal forces. As a result the reinforcement quantities will be obtained directly. Using analyses including non-linear reinforced concrete models two different concepts are possible as shown in Fig. 5 [30]: a section design according to Eurocode 2[24] or an ultimate load design according to DIN 1045-1 [3].

The section design with non-linearly calculated internal forces S^{NL} provides two different material laws – one with mean values f_{cm}, f_{ym} for the calculation of internal forces and another one with design values f_{cd}, f_{yd} for the section analysis. The alternative ultimate load design provides a global safety factor γ_R and the different design material values like f_{cR} and f_{yR} (see Table 2).

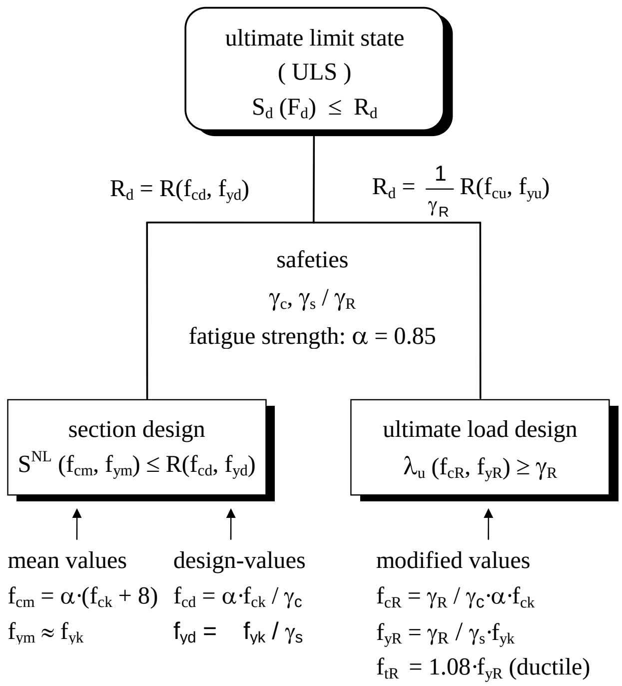

Fig. 5 Non-linear design procedure of ultimate limit states (ULS

DESIGN ASPECTS

Generally

The design in the ultimate limit states considers the design value for the uniaxial compressive strength $f_{cd} = \alpha\cdot f_{ck} / \gamma_c$, where α represents a factor regarding the fatigue strength. Generally $\alpha = 0.85$, but for short-term loading $\alpha = 1.00$. Furthermore strength increasing due to post-hardening can be assumed as well as the strength increasing due to multi-axial stress-states. The design value for the steel strength is given by $f_{yd} = \alpha\cdot f_{yk} / \gamma_s$. The partial safety factors γ_c and γ_s. depend on the different design requirements categories A1, A2 and A3 given in Table 2. With regard to possible damage effects due to cyclic action effects a possible decreasing of the strength values has to be considered.

Shear resistance

The verifications for the shear resistance can be realized according to DIN 1045-1[3]. So shear reinforcement will be necessary if

$$V_{Ed} > V_{Rd,ct} \tag{7}$$

where V_{Ed} represents the design value of the applied shear force in comparison to the design value of the shear resistance $V_{Rd,ct}$. The shear resistance is characterized by the concrete strength values and by a coefficient c_d which depends on the different design requirements categories A1, A2 and A3 [26].

The shear reinforcement has to be determined on the basis of a truss model according to DIN 1045-1 [3, 26]. Such a truss model implies a limitation of the truss inclination as well as a limitation of the compressive strut strength.

Punching shear resistance

Punching shear reinforcement will be necessary if the applied shear force in the critical section v_{Ed} (per united length) is greater than the shear resistance $v_{Rd,ct}$, that means

$$v_{Ed} > v_{Rd,ct} \tag{8}$$

The shear resistance corresponds to DIN 1045-1[3] extended by a coefficient c_d which depends on the different design requirements categories A1, A2 and A3 [26].

For the necessary punching shear reinforcement a differentiation between the structural members becomes necessary:

- structural members with indirect load actions which are typical conventional buildings (columns of plates or foundations) and
- structural members with direct load actions which are typical for nuclear technical buildings with design requirements categories A2 and A3 (for instance impact areas of an airplane crash).

Regarding structural members with indirect load the punching reinforcement can be calculated according to DIN 1045-1[3]. To avoid concrete spalling of the columns edges the maximum shear resistance is limited by

$$v_{Rd,max} = 1.5 \cdot v_{Rd,ct} \tag{9}$$

For structural members with direct load actions a new verification for the punching shear resistance has been formulated on the basis of DIN 1045-1[3] and additional experimental results. In this case spalling of the concrete cover generally must not be avoided in the categories A2 or A3. But to activate the stirrups the compressive strut strength $V_{Rd,max}$ has to be limited by the resisting force of the effective zone of the punching shear cone which is defined in DIN 25449[26] on the basis of adequate experimental results mentioned before.

Considering $V_{Rd,max}$ the necessary reinforcement quantities result from the resistance of two parts: a concrete part considering the longitudinal reinforcement with the design value $V_{Rd,c}$ and a steel part considering the transverse reinforcement with the design value $V_{Rd,s}$ [26]. So the applied punching shear V_{Ed} has to be verified as follows:

$$V_{Ed} \leq V_{Rd,sy} = V_{Rd,c} + V_{Rd,s} \tag{10}$$

CONCLUSIONS

Caused by the new generation of German design standards based on the corresponding European standards an adjustment of the current regulations and standards concerning nuclear facilities – especially KTA-rules - has become necessary. Recommendations for such an adjustment are given in the KTA-report KTA-GS-78[29].

Regarding these recommendations with respect to reinforced and prestressed concrete structures a revised DIN 25449[26] has been published. In this revised standard the following aspects were to be considered:

- adaptation of the partial safety concept regarding ultimate limit states and serviceability limit states,
- definition of relevant internal and external incidents,
- consideration of the different design requirements categories A1, A2 and A3,
- methods for structural analysis and design; particularly the application of non-linear analysis,
- ensuring the durability including the demands for concrete properties

In addition to these aspects the revised standard DIN 25449[26] comprises special indication for the design approach. So for structural members with direct load actions a new verification for the punching shear resistance has been formulated. Exemplary applications especially with respect to local areas of an airplane crash impact have demonstrated that the new verification represents an acceptable design approach.

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