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Remarks on the Safety Concept of Tank Structures
Anmerkungen zum Sicherheitskonzept von Tankbauwerken

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Kurzfassung: Anmerkungen zum Sicherheitskonzept von Tankbauwerken

Betreiber, Genehmigungsbehörden, Abnahmeorganisationen und Fachplaner haben aufgrund verschiedenartiger Ausgangspositionen zum Teil sehr unterschiedliche Erwartungen an das Sicherheitskonzept von Tankbauwerken. Dies wird noch dadurch kompliziert, dass beim Brandschutz, im Gewässerschutz und in der Tragwerksplanung jeweils andere Konzepte zum Erreichen der Schutzziele angewandt werden. Hieraus entstehen zum Teil sehr emotional geführte Diskussionen, bei denen traditionelle und als bewährt angesehene Betrachtungsweisen auf neuere, semi-probabilistische Ansätze auf Grundlage des Eurocode 0 (EC0) treffen.

Im vorliegenden Beitrag werden diese unterschiedlichen Konzepte gegenübergestellt. Es werden Hintergrundinformationen gegeben, die die Grundlagen des semi-probabilistischen Konzepts erläutern. Es wird am Beispiel der Teilsicherheitsbeiwerte gezeigt, dass die Umsetzung der Prinzipien des EC0 im Behälterbau nur sehr unzureichend erfolgt ist, wobei das Bemessungsergebnis deutlich konservativer ausfällt, als nach den vorgegebenen Ausfallswahrscheinlichkeiten zu erwarten wäre.

Ziel ist es, aufzuzeigen, in welchen Fällen Festlegungen nach streng-rationalen Kriterien möglich sind, und in welchen Fällen eine Festlegung auf weniger rationaler Grundlage im Hinblick auf gesellschaftliche Akzeptanz angebracht ist.

1 Introduction

In this paper we try to make general remarks on the structural safety for tanks. We are aware, that this conference, organized by the TÜV, is very much focused on tanks for petro-chemical liquids. Still, we want to keep other liquids included in our considerations, such as fresh or waste water, liquid manure, etc.

In order to make it short and simple, we are not explicitly handling seismic design. However, seismic design follows the same principles of designing against an event with a target recurrence period.

2 Terms and Definitions

β — reliability index acc. to EC0 informative annex C.5, see Tab C.1 [7]
β is the argument of the reliability CDF for the target failure probability
$P_f = \Phi(-\beta)$
distance from CDF-mean to the target failure probability, normalised with CDF-SDEV
see Fig. 1; see Tab. 1 for example values
for an explicit example, how the reliability index β is linked to e.g. partial safety factor γF, see chapter 9 in [32]

Table 1: Selected pairs of values of the PDF Φ(x,0,1,z) in eq. 1,
failure probability in 50 years / 1 year

z	4,753	4,265	3,80	3,30	3	2	1	0
p(50)	$1{,}0\cdot10^{-6}$	$1{,}0\cdot10^{-5}$	$7{,}235\cdot10^{-5}$	$4{,}834\cdot10^{-4}$	$1{,}350\cdot10^{-3}$	$2{,}275\cdot10^{-2}$	0,1587	0,5000
p(1)	$2{,}0\cdot10^{-8}$	$2{,}0\cdot10^{-7}$	$1{,}4\cdot10^{-6}$	$9{,}7\cdot10^{-6}$	$2{,}7\cdot10^{-5}$	$4{,}6\cdot10^{-4}$	$3{,}2\cdot10^{-3}$	0,010

For comparison:
individual probability of winning 6/49: p = 1 / 140 millions
annual probability to get killed in an traffic accident in Germany
2017: 3.200 dead per 83 mil inhabitants: $p = 3{,}9\cdot10^{-5}$
1972: 18.800 dead per 62 mil inhabitants: $p = 3{,}0\cdot10^{-4}$
nuclear facilities are estimated to have a failure rate, which is another

	1-2 orders of magnitude lower than ordinary civil engineering structures who defines the value of public accepted failure rate?
CDF	cumulative density function, integral of PDF see eq. 1, which represents CDF of eq. 2

$$\Phi(x,\mu,\sigma,z) = \frac{1}{\sqrt{2\pi\cdot\sigma^2}} \int_{-\infty}^{-z} e^{-\frac{(x-\mu)^2}{2\sigma^2}}\,dx \qquad (1)$$

codes	technical delivery codes; product codes; design codes; fabrication codes; design codes split up in generic codes; application codes;
deterministic	known input values result in known output
DIBt	Deutsches Institut für Bautechnik; German Institute for Building Structures; highest German building authority
EC0; EC1; EC3; ...	short form for (DIN) EN 1990; (DIN) EN 1991; (DIN) EN 1993;
failure probability	see probability
Gaussian distribution	simplest, symmetric mathematical model for a PDF with 2 degrees of freedom, mean μ and SDEV σ; agreed to describe random variables in large number very well, see eq. 2.

$$f(x,\mu,\sigma) = \frac{1}{\sqrt{2\pi\cdot\sigma^2}}\, e^{-\frac{(x-\mu)^2}{2\sigma^2}} \qquad (2)$$

	boundaries $\pm 1\cdot\sigma$ around μ are including app. 68 % of the population boundaries $\pm 2\cdot\sigma$ around μ are including app. 95 % of the population boundaries $\pm 3\cdot\sigma$ around μ are including app. 99 % of the population
GMNIA	geometrically and materially nonlinear analysis with imperfections included, see EC3-1-6 chapter 8.7 [10]
Laziness Hypothesis	Knödel’s hypothesis no. 11 on laziness for structural engineers suggests, that we are investing only that amount of effort, which is needed to achieve the targeted result [31].
limit states	see ULS and SLS

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mean	denoted by μ if referring to the population; denoted by $\bar{x}$ if referring to a sample;
nominal stress design	design with stresses, with are assumed to be evenly distributed along the cross section;
PDF	probability density function
population	set of items with quasi-identical properties; e.g. all shell structures, that have been or will ever be designed according to EC3, see sample

probability (of not being exceeded)

measure for the reliability concept in EC0 4.1.2 (7)

Ramberg-Osgood: one-parameter model for a nonlinear stress-strain relationship with given yield limit fy, see eq. 3; low values for the exponent n describe strong hardening, high values n describe ideal plastic performance (see [38] chapter 5.2.2; EC3-1-4 eq. C.1 [10]);

n = 6 stainless steels;

n = 24 (almost) ideal plastic structural steels;

$$\varepsilon = \frac{\sigma}{E} + 0{,}002\left(\frac{\sigma}{f_y}\right)^n \quad (3)$$

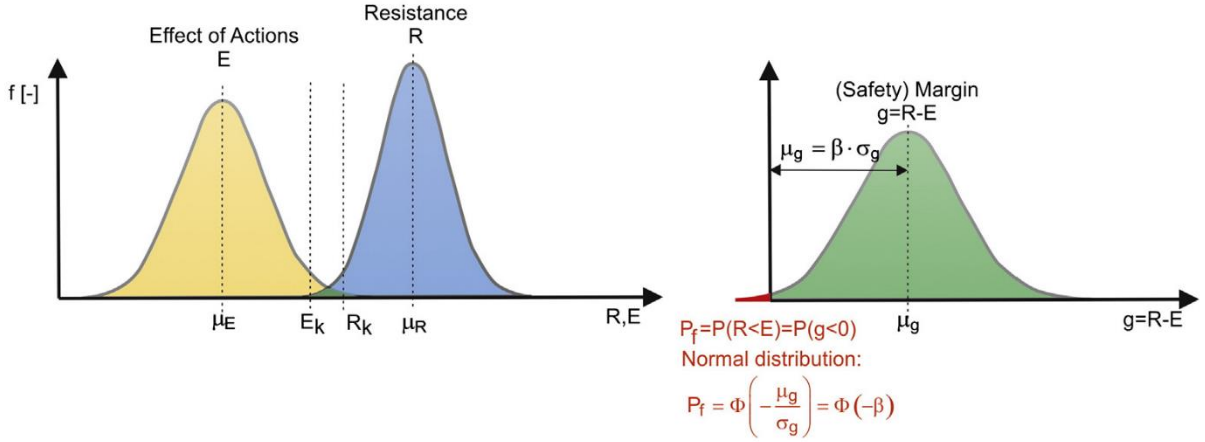

Figure 1. The reliability problem: failure occurs for E > R (or g = R – E < 0)
(used with permission of Prof. Taras, UniBW Munich, see [41])

reliability: see Fig. 1; failure is given for E (yellow) > R (blue);

the failure function g (green) is given by g = R – E;

failure occurs if g < 0

β describes the distance between failure and μg if SDEV is normalized

	to 1 E, R and g are probabilistic quantities, as such they are given by PDFs, characterized by their respective μ (mean) and σ (SDEV)
reliability index	see β
safety	for the common man: it will never happen for the engineer: it will happen, but only after a very long time see failure probability
sample	subset of a population
SDEV	see standard deviation
SLS	serviceability limit state often seen as limitation for deflections under nominal loads in civil engineering arbitrary in most cases: L/300 does not describe any functionality; however, it can be an indirect measure of the natural frequency of a component etc; in tank design deformation limits may be associated to the tightness of adjacent piping
standard deviation	denoted by σ, if referring to the population; denoted by s, if referring to a sample; square root of the variance; thus, SDEV has the same physical unit as the property under consideration
safety factor	intuitively set margin in order to cover the unforeseen; better be termed as un-safety factor
stochastic	based on random scatter
ULS	ultimate limit state per definition the ultimate load a structure can bear, independent of the magnitude of deformation; known only after a full scale test; if calculated, it's only a more or less precise estimate, depending on how much effort has been put in the calculation model (see laziness hypothesis);

utilization — η (eta) = Ed / Rd

Design value of effect (action) divided by design value of resistance; at 1,0, the structure is loaded exactly up to their design resistance, i.e. the global safety is the product of the load and resistance safety factors.

Weibull distribution — unsymmetric mathematical model for a PDF with 2 degrees of freedom, shape parameter k and scale parameter A; agreed to describe annular wind speed distributions very well; the notation in eq. 4 is taken from the European Wind Atlas [42], see chapter Actions – Wind.

$$f(x,k,A) = \frac{k}{A} \cdot \left(\frac{x}{A}\right)^{k-1} \cdot e^{-\left(\frac{x}{A}\right)^{k}} \qquad (4)$$

3 Points of View

- Operator
- Designer
- Manufacturer
- Building Authorities
- Environmental / Water Protection Authorities
- Inspection Bodies
- etc.

All of these have a different view on tanks. They might have a different system of rules (see below). In this article, we are looking from a designers point of view.

4 Systems of Rules

Structural design

meets mechanical requirements including strength, stability, ...

also durability including creep, fatigue, corrosion, wear, ...

As rules we have structural design codes (coming from CEN/TC 250), including

EC0 basis of design [7]

EC1 loads [8]

EC2 concrete (foundations) [9]

EC3 steel [10]

EC7 geotechnics [11]

EC8 seismic design [12]

EC9 Aluminium [13]

These codes are split up in generic codes, e.g. EC3-1-1, EC3-1-4 (additional rules for stainless steels); EC3-1-6 (shell structures)
and application codes, e.g. EC3-4-1 (silos) and EC3-4-2 (tanks)

Thus, if a designer wants to design a tank, he has to consult 3 different lengthy codes for steel design; if the tank happens to be made out of stainless steel, then there is a fourth one. Where in the old days, DIN 4119 [14] with two parts and a total of 24 pages seemed to contain everything what an engineer needs – including wind loads.

On the other hand: many of the former German application codes DIN 4xxx were regulating wind loads and strength design, which is very difficult in the sense of including new state of the art. Thus, although complicated, the structure of the Eurocodes may be regarded as step ahead.

Functional / Operational design

meets operational requirements such as retaining a liquid; access doors, piping, venting, controlling
As rules we have functional design codes (coming from CEN/TC 265) including

EN 14015 site fabricated welded tanks [18]

EN 12285 shop fabricated tanks [16]

EN 14620 liquefied gases [20]

Manufacturing

meets execution requirements such as welders qualification, manufacturing companies qualification
As rules we have execution codes (coming from CEN/TC 135), e.g. EN 1090 [6]

Fire and Explosion

meets requirements for safe handling of inflammable liquids and explosive vapours
Rules are coming e.g. from CEN/TC 191 Fixed firefighting systems or from CEN/TC 305 Explosion prevention, fulfilling e.g. the ATEX-Directive (Directive 2014/34/EU [4]).

Environmental protection

meets requirements for water and air pollution; in Germany rules are coming from WHG [2] and BImSchG [1]

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Work safety

meets requirements for safety and health protection of workers given in Directive 89/391/EEC [3]; rules are including e.g. ISO 14122 [19]

plus (many) others, e.g. lightning protection, etc.

5 Design Situations

Exemplary we look at three different design situations for the tank wall.

Tensile membrane hoop stress in the tank wall

relevant loads are hydrostatic pressure from liquid level and operational overpressure

the design for this action is decisive for the required wall thickness, given by a strength check

Apart from the circumferential membrane stress bending effects from stepped wall thickness and constraints at the tank's foot are considered. A typical example of the resulting membrane hoop stresses is given in Fig. 2.

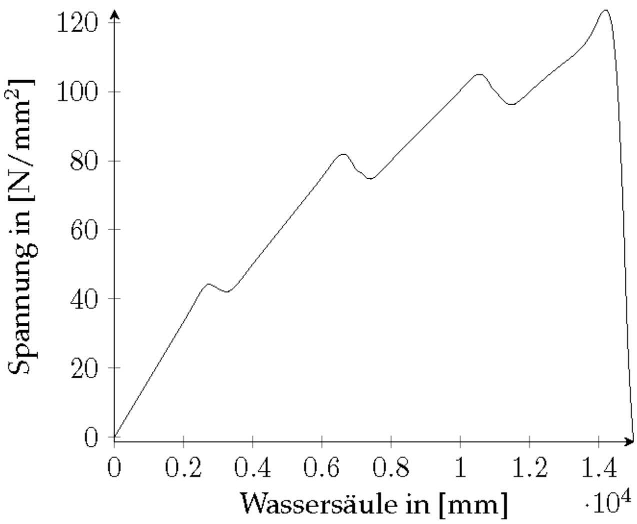

Figure 2. Hoop stresses in a tank vs. filling height, sum of membrane and bending action; D = 20 m, H = 15 m, T = 6 / 8 / 10 / 12 mm; ([25], see also [33])

Compressive membrane hoop stress in the tank wall

relevant loads are wind pressure on the luff meridian and operational under-pressure (with closed tanks) or partial evacuation by wind (with vented tanks)

the design for this action is decisive for the distance of the ring stiffeners

A typical example of the eigenmode developing with stepped wall thickness is given in Fig. 3.

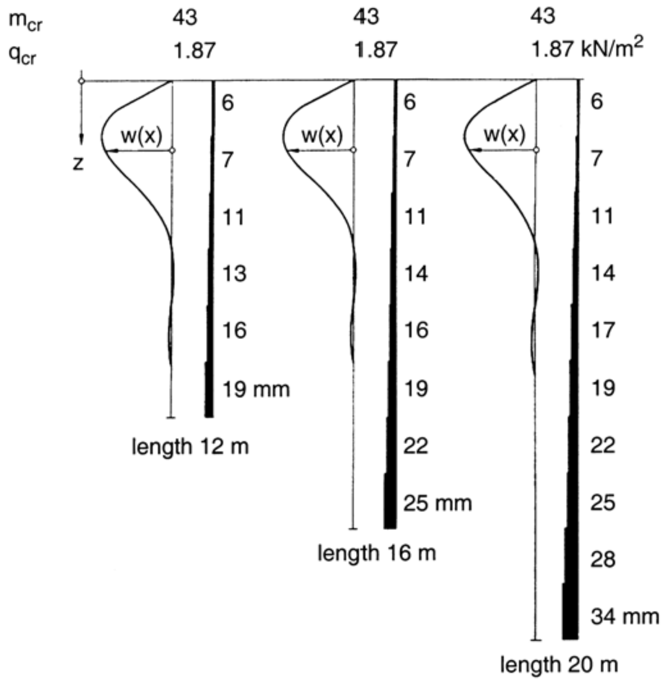

Figure 3. Eigenmode under external pressure for a 40-m-diameter tank
– independent of the tank's length;
([40], this method will be included in the next version of EC3-1-6; see also [32])

Compressive membrane meridional stress in the tank wall

relevant loads are (self weight is negligible) snow, under-pressure, bending due to wind or earthquake

the design for this action may be decisive for the wall thickness of the upper strakes in large-diameter tanks; often, it "kicks-off" the buckling-interaction-check along with hoop compression; Note, that the provisions of EC3-1-6 seem to bee over-conservative for tanks with R/T > 2000 ([26], [27]).

Typical failure modes are given in Fig. 4.

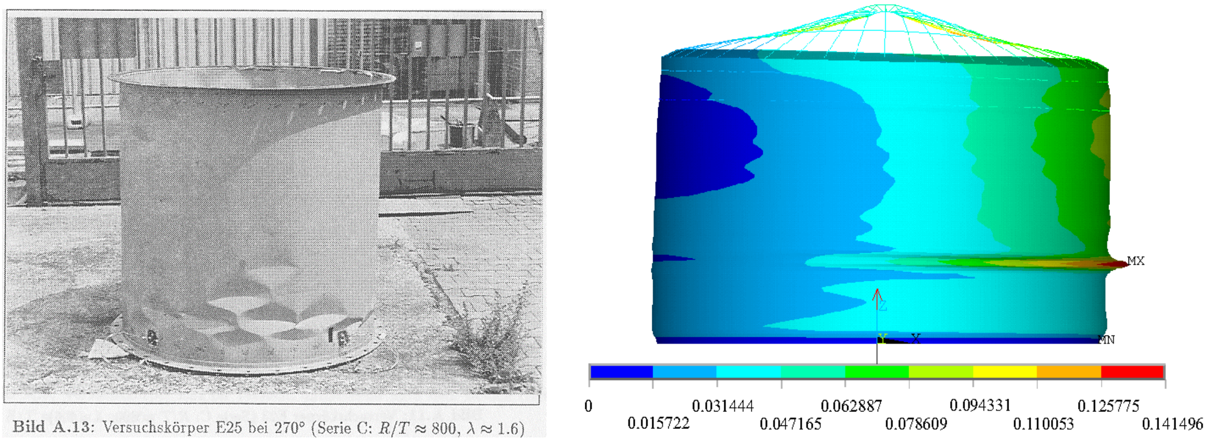

Figure 4. Failure modes of cylindrical shells under axial compression / bending
left: empty shell after experimental test [29]
right: FEA-displacement of a filled tank with stepped wall (PhD J. Rosin [39] Fig. 7.10)

6 Structural Safety Concept

6.1 Historical – deterministic

Historically, design of components was about nominal loads. If you had a 10-t-crane, then you were calculation all stress resultants and stresses for 10 t (app. 100 kN). Of course it was known, that by someone or somehow the crane could misused unintentionally. Therefor, in the strength check you would not allow your stresses to go up to yielding but only up to the "allowable stress", which commonly was the yield limit divided by e.g. 1,5. This factor was termed safety factor, and is was quite clear that a higher factor would give you more safety. Thus, you would receive for an ordinary structural steel S235 with fy = app. 240 N/mm^2 allowable stresses of 160 N/mm^2.

Stability issues were handled by ω-values, which were used to increase the nominal loads in order to account for second order effects. The "strength" check was still done against the allowable stresses, maybe these were reduced a bit down to 140 N/mm^2. The ω-values were given in tables depending on the slenderness λ and a stability check could be done in two lines, one for the slenderness and one for the check.

In tank or pressure vessel design, you might have had a safety factor of K = 2,0, but instead of 0,2 % proof stress you were allowed to go for 1,0 % proof stress.

We should note, that e.g. wind loads, although known to be highly irregular and unpredictable, were handled as if they were deterministic. This could be done in fixing a maximum value for a "design wind", which by experience was very unlikely to be exceeded.

From todays view, we should also note, that the safety factor gave you only a “look-alike-safety” because this term “safety” was completely unspecified in it’s magnitude.

Those were the good old days.

6.2 Modern – probabilistic

What we now know as “modern semi-probabilistic design concept” is based on several, quite different insights:

- There is no such thing as a safe technical structure (see chapter Terms and Definitions). Everything will fail, if you wait long enough.
- The loads (“actions”) are subjected to scatter; this holds also for the resistance side, e.g. geometric dimensions; material properties; thus, the values that we use, are “characteristic”, which means e.g. a 95 %-non-exceedance probability for a load and a 95 % survival probability of a steel coupon in a tensile test.
- Certain combinations of loads are unlikely to happen, e.g. design wind and design snow or design wind and design earthquake
- If load and resistance were known (“deterministic”), then a “safety factor” is not needed. In fact, what we did ever since with this factor, is to describe “uncertainty”. So, for common understanding, it would be much better to use the term un-safety factor or uncertainty factor. Of course, this holds for loads and resistances, because both of them are uncertain.
- The “semi” in semi-probabilistic comes from the fact, that in EC0 the contribution of actions and resistances to the total uncertainty is split up in fixed proportions – for reasons of simplification.
- In the context of EC0, the overall effect of the partial safety factors on the load and resistance side are described by means of a safety index β (see chapter Terms and Definitions). On first sight it is surprising, that β = 3,8, which describes a failure rate of 1/700.000 per year, corresponds on one hand to the familiar safety factors of (global) 1,7. On the other hand, a failure rate of 1 over almost a million seems to be fairly acceptable by society (see examples in chapter Terms and Definitions).
- This all might sound new, unfamiliar and strange, but in Germany we had in 1981 already a position paper “GRUSIBAU” [22], which contained e.g. principles for evaluating test results in order to obtain a “characteristic value” for a structural component. On European level, basis

of EC0 is the probabilistic model code of JCSS Joint Committee on Structural Safety, which was worked on since 1972 as given in the preface [23].

Summarizing and using precise terms, we are not talking about safety but about failure probability.

If you increase the safety factor for loads by 1,1, which is done if you increase the consequence class of your structure from "normal" (CC2) to "important" (CC3), then you are increasing the safety index β from 3,8 to 4,2, which result in an lowering of the failure rate from under 0,7 million p.a. to 5 million p.a.

6.3 Limit States

In the old days, very often ultimate load and serviceability criteria were mingled in the design concept.

According to EC0 we have a clear distinction between ULS and SLS (see chapter Terms and Definitions). Basically, you want to do only a ULS check. However, this state might be accompanied by huge deformations (remember: in the old days we were consequently designing against first yield). Consequently, this requires to do a double check in all cases, which in turn violates the Laziness Hypothesis. Thus, an experienced engineer will try to understand so far, that he can see, in which points a SLS would be relevant.

Note, that each individual check in tank analysis has it's own typical limit state, coming from a very specific load combination, see chapter Design Situations.

6.4 Hazardous Liquids

So far we were discussing only mechanical requirements, as if all possible liquids were water, and the issue of tank design was for mechanical function only.

As shown above, additionally to the civil engineering requirements, we have requirements from environmental protection.

Problems are twofold:

- Although we have a European Directive on environment protection, each country does their own technical rules, which are not concerted.
- In Germany, we have the principle of concern ("Besorgnisgrundsatz"). This does not include any weighing or measuring. It requires, that if something is thinkable to happen, action must

be taken. In the authors' opinion, this might be resulting from a naive legislation, which over-estimates the actual technical possibilities, and which is ignorant to the fact, that you will always have a remaining risk of failure.

- When you look at the "second barrier principle", you find a wonderful means of introducing probabilistics into water protection. You multiply the failure probabilities of both barriers and result in a square-million failure rate, which is by far higher than what we require for nuclear facilities.

Thus, when we try to top up the above pure structural design principles by additional environmental requirements, we have no sensible measure of how to do so. Currently, it is done like

- if the liquid is dangerous, step up one consequence class
- if the liquid is very dangerous, step up two consequence classes

The authors are sure, that this is a sensible approach. Also, it seems that the German environmental authorities are consent with this approach, because it feels good and sounds reasonable. But as a closer look, it contradicts the principle of concern.

6.5 Practical Issues in Tank Design

In conferring the above semi-probabilistic design concept, some things went well and some did not. So we are listing some good and bad news – open for discussion

Rather bad news

- Someone forgot to include in EC1, that all the loads resulting from the production or handling process of liquids or bulk solids, are very much controlled in narrow boundaries by redundant devices. Thus, we don't have such a scatter as with wind or snow loads.
- Someone forgot to include in EC1, that you can't fill an open tank higher than it's eaves. Also, you can't fill a tank with fixed roof higher than it's roof.

Rather good news

- There is a new draft of EC0, which allows to include deterministic loads in design.
  That means, you don't have to multiply "uncertainty" to your hoop stresses, if the tank cannot be overfilled.

Recommendations for designers

- Don’t consider a tank as such to be safe or unsafe:
  In a structural analysis of a 5.000 m^3 freshwater tank (constant wall thickness, conical roof with rafters) for Kremsmüller/Austria in 2016, we ended up in 43 utilizations for the different strength and stability checks. Thus, we have a ranking of 43 values, which describe the safety of individual components in individual design scenarios.

6.6 Structural Analysis

Classical stress design

Nominal Loads are acting on a structural system
A component as part of a structural system must resist nominal stress resultants
(virtual) “Fibres” at different parts of the cross section are subjected to nominal stresses
the yield limit of the material is divided by the safety factor to receive allowable stresses
nominal stresses are checked against allowable stresses

Multi-axial stresses are handled by the von Mises equivalent stress hypothesis

Stability and fatigue issues are handled by reducing the allowable stresses

Plastic design is done by use of a “plastic shape factor”, which is app. 1,15 for I-sections and 1,5 for rectangular sections. Again, you end up in stress design.

Eurocode way

Nominal loads are multiplied by a load safety factor to receive design loads
A component as part of a structural system must resist design stress resultants
the yield limit is divided by a resistance safety factor to receive design yield stress
Cross sections have a plastic design resistance stress resultant in tension, shear and bending (evaluated with the design yield stress)
acting stress resultants are checked against resistance stress resultants

Multi-axial stress resultants require interaction rules for combining plastic tension, shear and bending in the cross section

Summary:

- Basically, in each of the concepts you are doing the same thing. The major difference is, that you split up the global safety factor in one load and one resistance part, and you shift the load safety factor on the other side of the design equation.

- This gives on one hand opportunity to use different load safety factors, e.g. 1,5 for variable loads and 1,35 for self weight. On the other hand design is more complicated. Authors's recommendation for lazy designers: if you know, that you don't need to go for the last 15 % of saving between dead and live loads, then use a global load factor of 1,5 for all loads and make your life easier.
- Advantage of the Eurocode way is, that it is more general (from a philosophical point of view) and is can capture system stability more easily than stress design.
- Disadvantage of the Eurocode way is, that interaction rules can be very nasty. So it might be much easier to switch from stress resultants level to stress level and use von Mises equivalent stresses.
- When you are designing on basis of finite element analysis (FEA) then its pretty hard to go for stress resultants, because the software gives you only stresses (see [31]). Good news: soon (final draft in 2019) there will be another generic code EC3-1-14 [8]: Design by advanced computational analysis, which gives guidance on how to design with FEA in the context of EC3.

Remarks on plastic design:

If we want to capture the real ULS of a structure by analysis, we need to include the whole range of failure modes from plastic design on one end to elastic stability on the other. In terms of the EC3 rules, this is demonstrated in Fig. 5 for a structure under bending. It is obvious, that if you restrict your design to elastic-limit-stresses, you can miss a lot of your structures capability. In the European Foreword of EN 14015 (version 2005 [18] and draft 2017 [17]) it sounds, as if this was new in tank design and there were not enough experience. For the authors, this statement sounds strange, and is maybe only a big misunderstanding: The former DASt-Ri 013, which was among the leading and most modern shell buckling rules at that time, dates from as early as 1980 [5]. Of course, this guideline covered the whole parameter range for shells, from purely elastic buckling to purely plastic buckling. That means expressis verbis, that we have plastic shell design in Germany for almost 40 years.

Following the allowable stress method limits the perfomance of the structure to 1,0 on the y-axis in Fig. 5. What a waste!

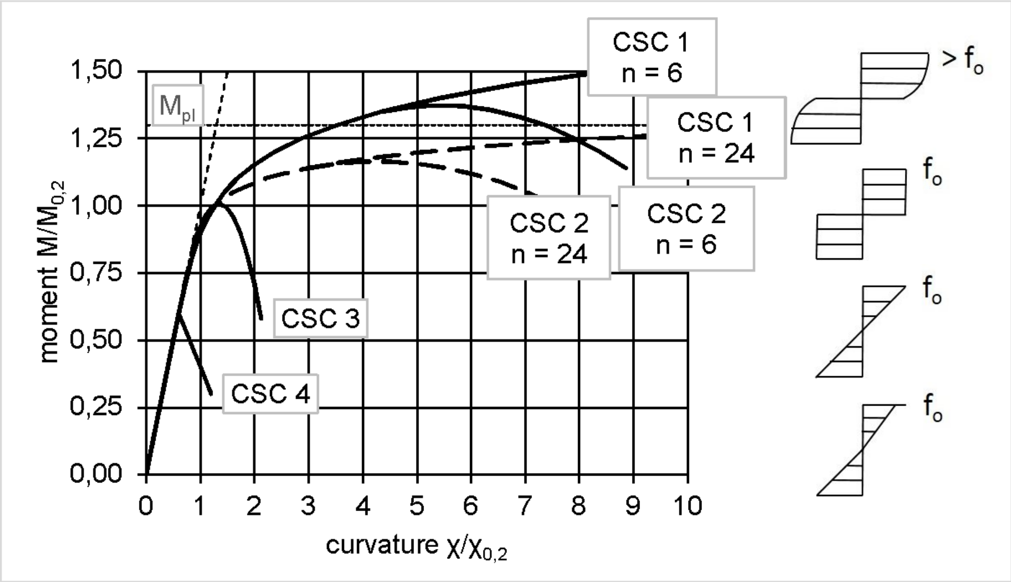

Figure 5. Moment-curvature relation of a metal structure for Ramberg-Osgood Parameters
n = 6 (stainless steels) and n = 24 (carbon steel, ideal plastic with only little hardening)
CSC are cross section classes according to EC3-1-1 (contribution for [38], see chapter 3 Fig. 14)

7 How precise do we know Actions

7.1 Wind

See EC1-1-4 [8].

According to JCSS 2.13.3 [23] the 10-min-mean velocity u of the wind speeds in 10 m height is following a Weibull distribution (see chapter Terms and Definitions).

Examples of the PDF and CDF are given in Fig. 6. In JCSS 2.13.3 [23] it is recommended to chose k "close to 2", so we used 2 ± 10 %. A is the standard deviation of the velocity fluctuation, arbitrarily chosen to be 5 m/s. With these parameters, a mean reference velocity of 25 m/s is given with 50 years recurrence.

Note, that these parameters are presumably describing Danish near-coast situations. Examples for German inland locations are given below (taken from the European Wind Atlas [42]):

Bremen airport	A = 4,9 m/s	k = 1,85
Frankfurt airport	A = 3,7 m/s	k = 1,60
München-Riem	A = 3,3 m/s	k = 1,28
Stuttgart airport	A = 2,8 m/s	k = 1,24

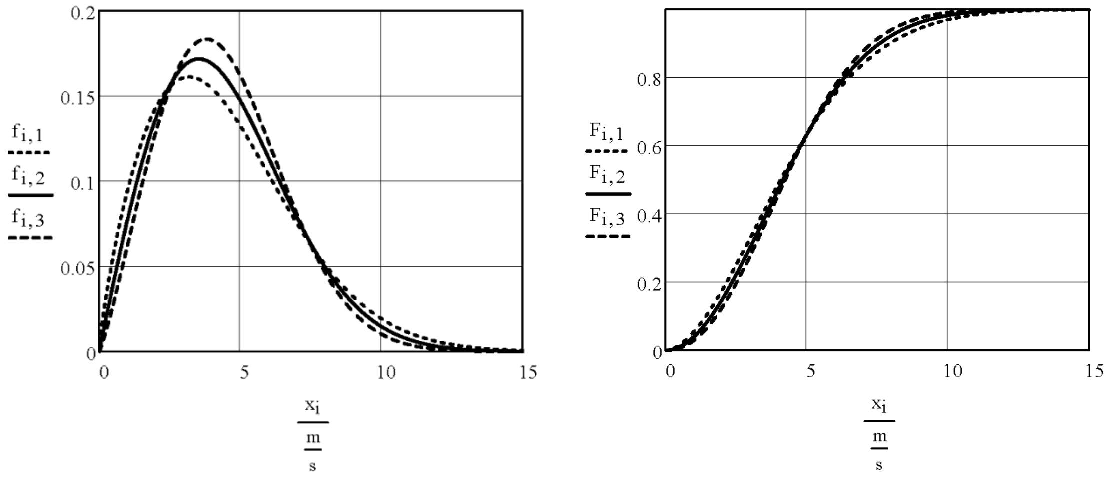

Figure 6. Weibull PDF and CDF for wind speeds [m/s] in one year according to the JCSS proposal [23] and the European Wind Atlas [42]

7.2 Filling level and operational pressure

As mentioned in this document on some other place, filling levels and operational pressures in industrial plants are controlled by redundant devices, so that a very smaller scatter is present. As demonstrated in [32], this allows to reduce the load-sided partial factor down to 1,03.

8 How precise do we know Resistances

8.1 Wall Thickness

The scatter of wall thicknesses is very narrow. Tolerances, which are describing minimum wall thicknesses are specified in the relevant technical delivery conditions. Data on statistical scatter are given in JCSS [23].

8.2 Material Constitutive Law

Characteristic values of the yield limit (or 0,2 % proof stress) and ultimate tensile strength are specified as 5 % quantile in the relevant technical delivery conditions (e.g. [15]). Additional statistical data are provided in [37].

For a extensive discussion on “real” material properties and different levels of modelling their constitutive law see [31] and [32].

8.3 Imperfection Pattern

The present design rules in EC3-1-6 are derived from many investigations on buckling tests. Although not exactly evaluated as 5 % quantile (95 % survival probability), it is a good engineering assumption. As we know from recent work of Jäger et al., the design curve seems to be over-conservative in the range of big R/T [27] (see also a recent discussion on imperfections in [34]).

An example for geometrical substitute imperfections used in "full" FEA is given in Fig. 7.

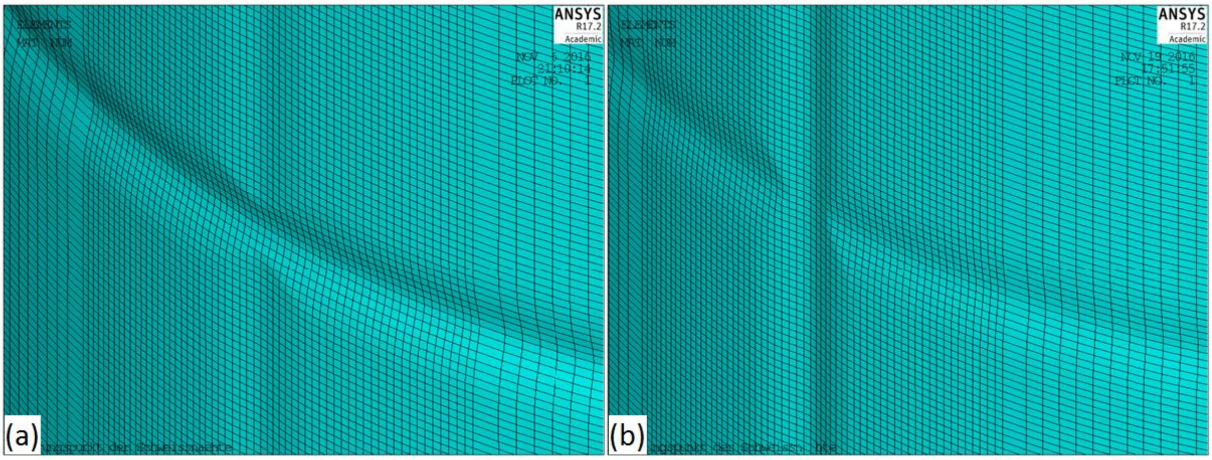

Figure 7. Substitute imperfections used in numerical GMNIA (Fig. 5.20 in [44])

9 Systems

In some cases, the splitting up of actions and resistances, which are treated separately and without interaction, is artificial and does not give a proper description of the structure. Due to space and time limitations this can not be discussed in detail here, but two examples are given.

- Structures, were 2nd order effects are significant.
  Stress resultants cannot be determined from the actions without taking into account the stiffness of the structure (which is the resistance part).
- Seismic design
  Natural frequencies of the structure are depending on the stiffness (resistance side) and the activated masses. Usually, the activated masses are described in terms of self weight, filling loads, snow, etc., which are on the actions side.

10 How can we be sure of the safety level

By definition, ULS is the maximum load or design load combination, that a structure can take. In most cases, this value can be determined only in a full scale test.

When this is not possible, we are using engineering models to make an estimate of the ULS. Since our models for loads and for the structure are conservative simplifications, we receive a calculatory ULS, which is systematically lower than the real ULS.

If we have a type of structure such as tanks, which is being built in about the same way very often, than we have a feedback from the failure rate (often referred to as having experience with a building type). However, the statement that none of the tanks failed does not exactly proof the quality of tank design, because it is not known, if the tanks were close before collapse or if they had the required “safety”. On the other hand, if we assume a realistic statistical scatter, than at least some of the tanks should have been failed, if the actual safety is close to 1.

In this context, DIN 4119 [14] tanks are accepted to be properly designed. There is no failure rate known, which is contributed to systematical errors in DIN 4119. Thus, at the Versuchsanstalt für Stahl, Holz und Steine, KIT, a research project for the DIBt Deutsches Institut für Bautechnik was performed [43], were we cross-calculated 3 different sizes of tanks according to DIN 4119 [14] and according to EC3-4-2 [10], see Fig. 8.

The three example tanks were initially designed according to EC3-4-2 (as well as according to the other relevant Eurocodes), and optimized with respect to wall thicknesses of the shell, anchors and roof. Then, for the optimized design, the utilization in the individual checks (strength, stability) were documented. Then, the chosen design was checked according to DIN 4119 [14], and again the individual utilizations were documented.

Comparing the utilizations according to EC3-4-2 [10] and according to DIN 4119 [14] a statement on the relative “safety” was possible. If the utilization according to EC3-4-2 is higher than the one from DIN 4119 [14], then EC3-4-2 [10] yields more conservative design. Sometimes this was the case and could be attributed to higher loading, which was required by EC1 [8]. In other cases, DIN 4119 [14] was more conservative, which was attributed to the fact, that in buckling design according to EC3-1-6 [10] different levels of imperfections can be prescribed, which was not possible in DIN 4119 [14] or DIN 18800-4 [21] respectively.

Generally we can state as a result, that there were no big differences found in the level of safety between these two codes. An example of the outcome is given in Fig. 9. We should note, that initially

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it was proposed to include EN 14015 [18] in this investigation as well, but which could not be done due to budget restrictions.

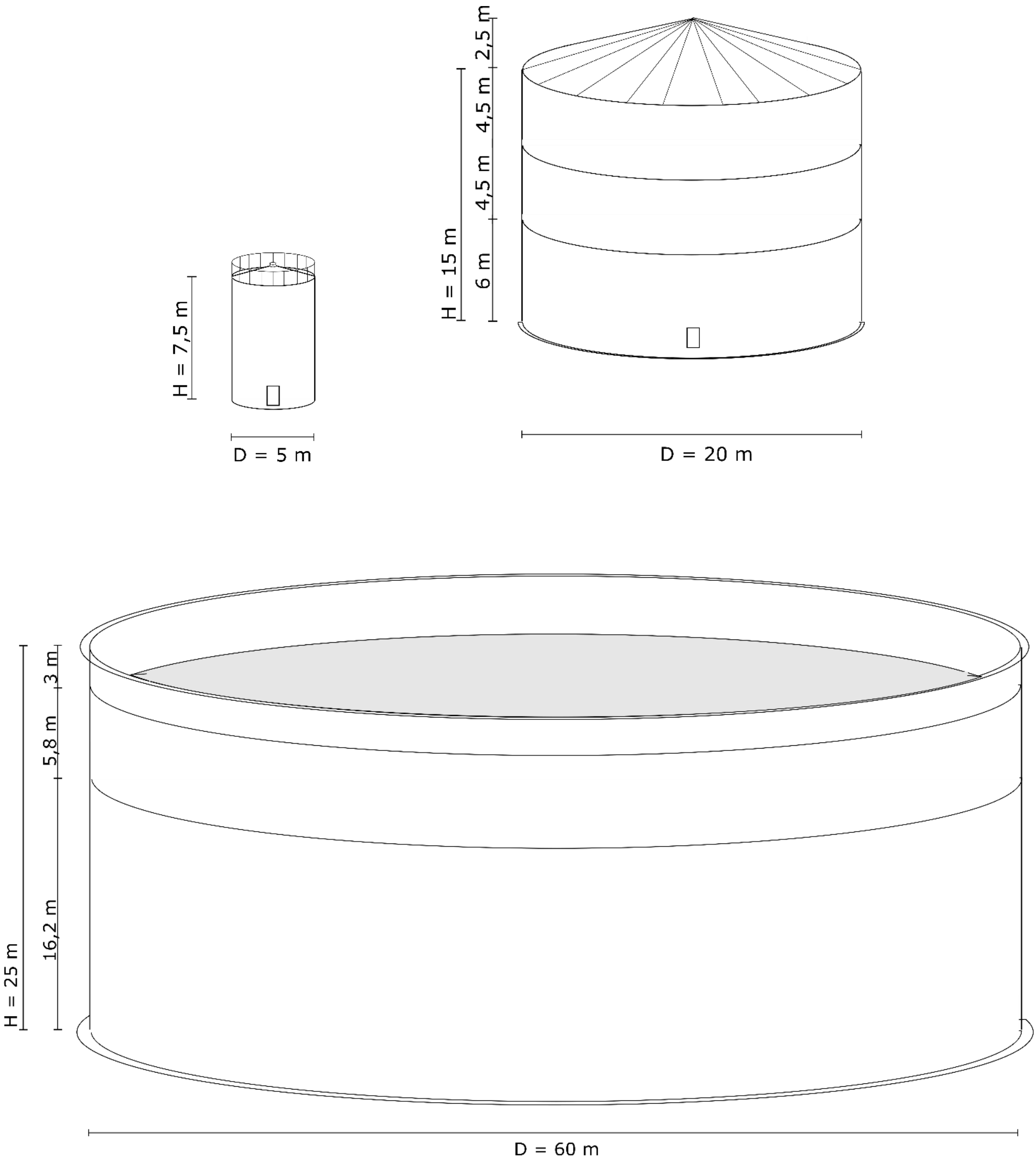

Figure 8. Three example tanks (not to scale), investigated in [43])

As one of the results of this research project a document was prepared, which the DIBt can recommend for temporary installation in the German States, until a new version of EC3-4-2 is available.

We recommend in this document, that the requirement for a FEA should not be linked to a high consequence class. We think, that for such a “simple” structure as an circular tank filled with liquid, a FEA cannot provide results, which are per se more accurate than a hand calculation (see a more elaborate discussion on this point in [32]).

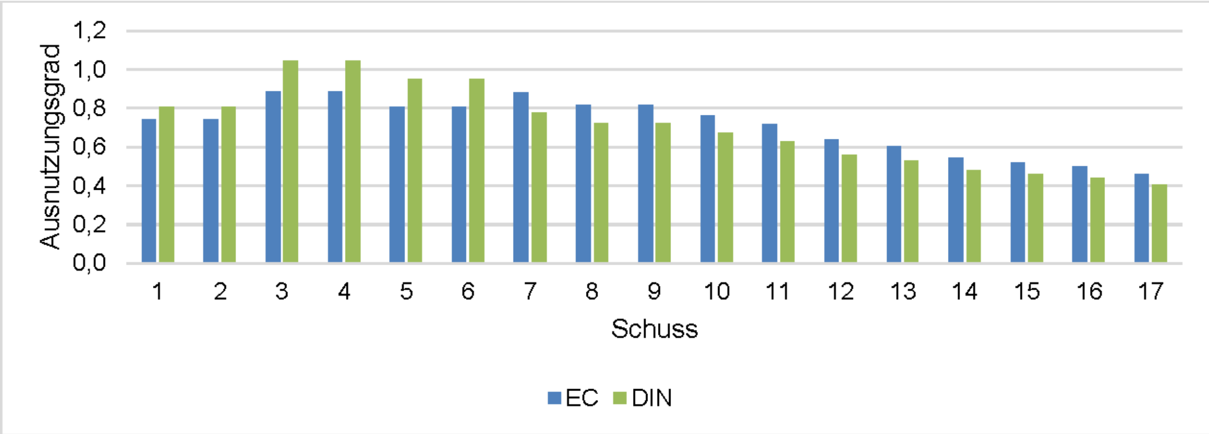

Figure 9. Example of the outcome in research project [43] (see Fig. 32):
utilization vs. strake number in the LS3 external pressure check for the 60 m tank
showing some configurations were DIN is more conservative and vice versa

Comment on Fig. 9:

In an external buckling check, every section of the shell between radial boundaries has it’s own critical external pressure, as overall performance of the involved strakes and their wall thicknesses. However, in each of the different strakes an individual hoop compression is generated by the critical external pressure. Thus, it is possible to evaluate a utilization for every strake’s wall thickness.

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12 References

12.1 Legal Norms

[1] Gesetz zum Schutz vor schädlichen Umwelteinwirkungen durch Luftverunreinigungen, Geräusche, Erschütterungen und ähnliche Vorgänge (Bundes-Immissionsschutzgesetz – BImSchG) vom 17.05.2013, zuletzt geändert durch Artikel 3 des Gesetzes vom 18.07.2017. BGBl. I S. 2771. (law to protect from harmful environmental impact, noise, vibrations and alike)

[2] Gesetz zur Ordnung des Wasserhaushalts (Wasserhaushaltsgesetz – WHG) vom 31.07.2009, zuletzt geändert durch Artikel 1 des Gesetzes vom 18.07.2017. BGBl I. S2771. (Federal Water Act)

[3] Richtlinie des Rates vom 12. Juni 1989 über die Durchführung von Maßnahmen zur Verbesserung der Sicherheit und des Gesundheitsschutzes der Arbeitnehmer bei der Arbeit (89/391/EWG). Amtsblatt der Europäischen Gemeinschaften Nr. L 183/2 vom 29.06.1989.
Council Directive of 12 June 1989 on the introduction of measures to encourage improvements in the safety and health of workers at work (89/391/EEC). Official Journal of the European Communities No. 183/2 as of 29.06.1989.

[4] Richtlinie 2014/34/EU des Europäischen Parlaments und des Rates vom 26. Februar 2014 zur Harmonisierung der Rechtsvorschriften der Mitgliedstaaten für Geräte und Schutzsysteme zur bestimmungsgemäßen Verwendung in explosionsgefährdeten Bereichen (Neufassung). Amtsblatt der Europäischen Union, 29.03.2014. (ATEX)
Directive 2014/34/EU of the European Parliament and of the Council of 26 February 2014 on the harmonisation of the laws of the Member States relating to equipment and protective systems intended for use in potentially explosive atmospheres (recast). Official Journal of the European Union, 29.03.2014. (ATEX)

12.2 Technical Codes

Remark: in order to save space codes are cited in short form, national annexes are not given; dates of issue are given for the latest amendment

[5] DASt Richtlinie 013: Beulsicherheitsnachweise für Schalen. August 1980.

[6] DIN EN 1090: Execution of steel structures and aluminium structures.
Part 1:2012-02 Requirements for conformity assessment of structural components.
Part 2:2018-09 Technical requirements for steel structures.
Part 3:2008-09 Technical requirements for aluminium structures.

[7] DIN EN 1990:2010-12 (EC0) Eurocode: Basis of structural design.

[8] DIN EN 1991 (EC1): Eurocode 1: Actions on structures.
Part 1-1:2010-12 General actions – Densities, self-weight, imposed loads for buildings.
Part 1-4:2010-12 General actions; Wind actions
Part 4:2013-08 Silos and tanks.

[9] DIN EN 1992 (EC2): Design of concrete structures.
Part 1-1:2011-01 General rules and rules for buildings.
Part 3:2011-01 Liquid retaining and containment structures.

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[10] DIN EN 1993 Eurocode 3 (EC3): Design of steel structures.
Part 1-1:2014-07 General rules and rules for buildings.
Part 1-6:2017-07 Strength and stability of shell structures.
Part 1-14: Design by advanced computational analysis. Unpublished draft V6.5:2018-09.
Part 4-1:2017-09 Silos.
Part 4-2:2017-09 Tanks.

[11] DIN EN 1997 Eurocode 7 (EC7): Geotechnical design.
Part 1: 2014-03 General rules.

[12] DIN EN 1998 (EC8): Design of structures for earthquake resistance.
Part 1:2013-05 General rules, seismic actions and rules for buildings.
Part 4:2007-01 Silos, tanks and pipelines.

[13] DIN EN 1999 (EC9) Eurocode 9: Design of aluminium structures.
Part 1-1:2014-03 General structural rules.

[14] DIN 4119: Above ground cylindrical flat bottom-tanks, constructed of metallic materials.
Part 1:1979-06 General regulations, construction, tests.
Part 2:1980-02 Structural analysis and design.

[15] DIN EN 10025: Hot rolled products of structural steels.
Part 1:2011-04 General technical delivery conditions (Draft).
Part 1:2005-02 General technical delivery conditions.
Part 2:2011-04 Technical delivery conditions for non-alloy structural steels (Draft).
Part 2:2005-04 Technical delivery conditions for non-alloy structural steels.

[16] DIN EN 12285: Workshop fabricated steel tanks.
Part 1:2016-10 Horizontal cylindrical single skin and double skin tanks for the underground storage of flammable and nonflammable water polluting liquids other than for heating and cooling of buildings; German and English version (Draft).
Part 2:2005-05 Horizontal cylindrical single skin and double skin tanks for the aboveground storage of flammable and non-flammable water polluting liquids.
Part 3:2016-10 Horizontal cylindrical single skin and double skin tanks for the underground storage of flammable and nonflammable water polluting liquids for heating and cooling of buildings; German and English version (Draft).

[17] DIN EN 14015:2017-12 Specification for the design and manufacture of site built, vertical, cylindrical, flat-bottomed, above ground, welded, steel tanks for the storage of liquids at ambient temperature and above (Draft).

[18] DIN EN 14015:2005-02 Specification for the design and manufacture of site built, vertical, cylindrical, flat-bottomed, above ground, welded, steel tanks for the storage of liquids at ambient temperature and above.

[19] DIN EN ISO 14122: Safety of machinery. Permanent means of access to machinery.
Part 1:2016-10 Choice of fixed means and general requirements of access.
Part 2:2016-10 Working platforms and walkways.
Part 3:2016-10 Stairs, stepladders and guard-rails.
Part 4:2016-10 Fixed ladders.

[20] DIN EN 14620: Design and manufacture of site built, vertical, cylindrical, flat-bottomed steel tanks for the storage of refrigerated, liquefied gases with operating temperatures between 0°C and –165°C.
Part 1:2006-12 General.

Part 2:2006-12 Metallic components.
Part 3:2006-12 Concrete components.
Part 4:2006-12 Insulation components.
Part 5:2006-12 Testing, drying, purging and cool-down.

[21] DIN 18800: Steel structures.
Part 1:2008-11 Design and construction.
Part 4:2008-11 Stability – Analysis of safety against buckling of shells.

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12.3 Technical Literature

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