BAUINGENIEURWESEN
Prof. Dr.-Ing. Jürgen Schnell
Fachgebiet Massivbau und Baukonstruktion

Erwin-Schrödinger-Straße
67663 Kaiserslautern
Gebäude 14, Zimmer 515
Telefon (0631) 2 05 - 21 57
Telefax (0631) 2 05 - 35 55
e-mail: jschnell@rhrk.uni-kl.de

Application of Partial Safety Factors on Existing Structures

- Outline Report-

This research report was funded by the
Federal Office for Building and Regional Planning (BBR).

(File number: Z 6 – 10.08.18.7 – 06.8 / II 2 – F20-06-019)

The author takes responsibility for the content of the report.

Project Manager: Prof.-Dr.-Ing. Jürgen Schnell

Referees: Dipl.-Ing. Alexander Fischer
Dipl.-Ing. Markus Loch

Kaiserslautern August 2008

Content

1. Introduction

1.1. Motivation of the research project

“Building in existing structures” means all kind of activities like repair, strengthening or modification of existing construction.
Designers and builders are deemed to decide which technical building regulations should be applied and in which cases the principle of maintaining the existing conditions is prevailing.

This is especially important in case the requirements in the current technical building codes restrict the design based at that time the existing structures were erected and no regulations for existing buildings have been defined. The participants have to clarify on which basis the structural analyses have to be done.

The project was suggested by designing and building engineers who have been involved in controversial conclusions between contractors and approval authorities on structural safety questions.
With the present research project application rules for the assessment of existing buildings in the ultimate limit state of structural capacity (proof of structural stability) should to be developed. They are the prerequisites for an available scientific approach to avoid safety deficits as well as over dimensioning.

1.2. Destination and implementation of the research project

Basically the following situations arise in case a new statical analysis for the existing structure has to be done as a result of conversion, refurbishing, service load increase etc. existing structural parts also claimed:

I. Structural verification of existing structures without having additional information from survey of the existing structure

The structure has been designed according to former applicable standards. A sufficient extensive sampling of the building materials is not possible - for example, at the time of the retrofit design the building is fully occupied. For the preliminary design the required material data can be selected from the material information given in the original design documents.

While the recalculation always has to be done using current codes, the material data of former codes has to be adapted to current reference values. In the present report a conversion factor is used to verify, for example, the different specimen geometries in the different standards. The application of semi probabilistic safety concept in the new generation of standards also require that the properties of the materials used as so-called characteristic parameters have become available, i.e. that they occur with a certain probability by assuming a statistical distribution (quartile data).

Based on a comprehensive literature research for the materials concrete and reinforcement the conversion factors mentioned above as well as design data for the characteristic parameters of the relevant mechanical properties of materials have been determined. The presentation of results is given on tables, from which the characteristic values of the material can be concluded for specific time periods. These theoretical assumptions, however, can only serve for an initial dimensioning. For final assessment of the structural safety of buildings, such assumptions made for the initial approach have to be verified in performing an adequate survey of the existing structure by a competent engineer for further planning activities.

II. Structural verification of existing structures having available additional information from survey of the existing structure

If a survey of the as-built conditions is accomplished before starting off with building measures, reliable information on the type of operation and the structural attributes can usually be gathered in such extent that a large part of the above listed uncertainties need not to be covered by safety surcharges. Thereby advantage can be taken in calibrating partial safety factors for the resistance of components.
Often the verification of existing structures succeed without additional strengthening measures by introducing decreased partial safety factors without lowering the acceptable reliability level.

The partial safety factors for new buildings have to cover all fields of uncertainty shown in figure 2.1

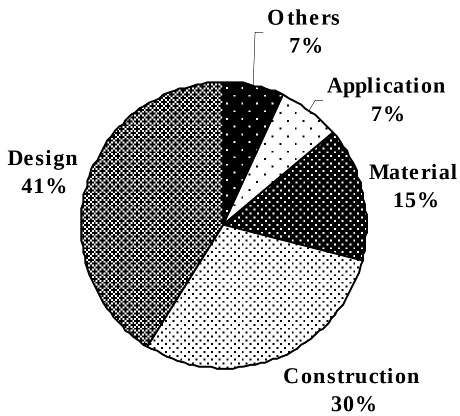


Figure 2.1: reason for quality defects on structures according to *Hansen 2004*

Especially the ratio of dead load and life load in reinforced concrete constructions is going to be a point of departure for reduction of partial safety factors of steel and concrete. In addition, there are lower material safety coefficients if the variation of material strength values encountered is lower than based in the corresponding standards.

All additional information discovered about the structure can be used in the probabilistic parameter studies to calibrate modified safety factors for the material.

2. Summary of Results

2.1. Design data for the characteristic concrete compressive strength

2.1.1. Concrete grade and strength classes applicable from 1916 to 1972

The following table provides data for the characteristic concrete compressive strength for concrete grade or classes during particular periods from 1916 to 1972 (respectively up to 1980 for concrete in accordance with the standards valid on the territory of the former German Democratic Republic).

Different specimen geometries and storage conditions of the previous regulations were considered by conversion factors.

It has been differentiated between a lower and upper limit of the characteristic concrete compressive strength (f_{ck}^{l}, f_{ck}^{u}), the values of which represent different quality levels of concrete production associated with the different variation of concrete compressive strength. The lower limit of compressive strength is expected for smaller sites with acceptable supervision and mix of concrete at the construction site without sampling. The upper limit can be assumed for concrete manufactured at medium-sized construction sites with good supervision (3-10 concrete samples) or large construction sites with good supervision (more than 30 concrete samples during production).

Period	Former Concrete Strength Grade or Class	Lower Limit f_{ck}^{l} [N/mm²]	Upper Limit f_{ck}^{u} [N/mm²]	f_{ck}^{l} ; f_{ck}^{u} Current Concrete Strength Class (*DIN EN 206-1)*
1916-1925	W_{28}=150 kg/cm²	6,3	8,5	- ; C8/10
	W_{28}=180 kg/cm²	7,5	10,2	- ; C8/10
1925-1932	W_{b28}=100 kg/cm²	4,2	5,7	- ; -
	W_{b28}=130 kg/cm²	5,4	7,4	- ; -
	W_{b28}=180 kg/cm²	7,5	10,2	- ; C8/10
1932-1943	W_{b28}=120 kg/cm²	5,0	6,8	- ; -
	W_{b28}=160 kg/cm²	6,7	9,1	- ; C8/10
	W_{b28}=210 kg/cm²	9,0	11,1	- ; C8/10
1943-1972 (GDR until 1980)	B 120	5,0	6,2	- ; -
	B 160	6,7	8,3	- ; C8/10
	B 225	10,1	12,3	C8/10 ; C12/15
	B 300	15,5	18,0	~C16/20 ; C16/20

Chart 2.1: Concrete strength assignment of different concrete strength grades and classes from 1916 to 1972 (respectively to 1980 in GDR) in the strength classes according to *DIN EN 206-1; 2001-07* taking into account the quality level of concrete production

2.1.2. Concrete strength classes from 1972 to 2001

The nominal compressive strength β_{WN} is defined as the 5%-quantile of the populations in DIN 1045 since 1972 so only the conversion factors of the different sample form and the different storage has to be considered at the conversion to the characteristic compressive cylinder strength.

The same is true for concrete strength classes from 1981 to 1990 in accordance with the regulations of the concrete construction works on the territory of the former GDR.

Period	Former Concrete Strength Class	f_{ck} [N/mm^2]	Current Concrete Strength Class (*DIN EN 206-1; 2001-07*)
1972-1978	Bn 50	3,9	-
	Bn 100	7,7	~C8/10
	Bn 150	11,6	~C12/15
	Bn 250	19,3	C16/20
	Bn 350	27,1	C25/30
	Bn 450	34,8	~C35/40
	Bn 550	42,5	C40/50
1978-2001	B 5	3,9	-
	B 10	7,7	~C8/10
	B 15	11,6	~C12/15
	B 25	19,3	C16/20
	B 35	27,1	C25/30
	B 45	34,8	~C35/45
	B 55	42,5	C40/50
1981-1990 (GDR)	Bk 5	3,7	-
	Bk 7,5	5,5	-
	Bk 10	7,4	~C8/10
	Bk 15	11,0	~C12/15
	Bk 20	14,7	~C12/15
	Bk 25	18,4	C16/20
	Bk 35	25,8	C25/30
	Bk 45	33,1	C30/37
	Bk 55	40,5	C40/50

Chart 2.2: Assignment of concrete strength for various classes from 1972 to 2001 on the concrete strength classes according to *DIN EN 206-1; 2001-07*

2.2. Design data for the characteristic material properties of reinforcement

The mechanical properties of reinforcement demanded in the specific generations of standards of the last century are comparable concerning their test conditions since 1936. This is the result of a comparison of former and current standards.

The following tables give information about the characteristic yield point of reinforcement, put in order by their names and by the different time periods. The elastic limit and 0.2% proof stress are taken as the yield point depending on the steel grade.

The assignment of reinforcing steel bars to the ductility classes of *DIN 1045-1* is based on the findings by screening the literature and is explained in sections 2.3.1 and 2.3.4.4 of the final report.

General recommendations to classify reinforcing steel mesh by ductility classes can not be given.

Reinforcing steel before 1972

The recommendations of the *DB-Richtlinie 805* are adopted for the characteristic yield point f_{yk} because no sufficient statistical data is available for reinforcement produced before 1972.

Reinforcing steel since 1972

The nominal value of yield point is defined in the respective regulations and standards as 5%-quantile of the populations since 1972 (since 1968 for deformed steel bars with an approval in the former West German States). So it corresponds to the current definition of the characteristic design data of the yield point f_{yk}.

2.2.1. Reinforcing Steel Bars

Name	Grade of Steel [Ductility Class]	Usage	Year	Charac. Yield Point [N/mm^2]
plain round bars	Mild steel (from 1925: St 00.12) [B]	1860-1937	-	130
	Mild steel (from 1925: St 37, St 37.12) [B]	1860-1972	before 1943	210
			from 1943	245
	Concrete steel I (from 1943) [B]	1943-1972	1943	245
	BSt 220/340 GU (DIN 488) [B]	1972-1984	1972	220
	St 52 [B]	1932-1972	1932	260
	Concrete steel IIa (from 1943) [B]	1943-1972	1943	315
	St A-0 (GDR) concrete steel I [B]	1965-1985	from 1965	245
			from 1972	220
	St A-I (GDR) concrete steel I[B]	1965-1990	from 1965	245
			from 1972	240
	St B-IV / St B-IV S (GDR) [-]	1970-1990	1972	490
ribbed steel bars DIN 488	BSt 220/340 RU (I) [B]	1972-1984	1972	220
	BSt 420/500 RU (III) [B]			420
	BSt 420/500 RK (III) [A]			
	BSt 420 S (III) [B]	since1984	1984	420
	BSt 420 S (III) twisted [A]			
	BSt 500 S (IV) [B]			500
	BSt 500 S (IV) twisted [A]			
ribbed steel bars (GDR) TGL 101-054 TGL 12530 TGL 33403	St A-III [B]	1965-1990	from 1965	315
			from 1972	390
	St T-III [B]	1972-1985	1972	400
	St T-IV (since 1981) [B] St B-IV RDP [-] St B-IV S-RDP [-]	1977-1990	1977	490

Chart 2.3: Design data for the characteristic yield point and ductility classes of reinforcing steel bars during different time periods, according to *Fingerloos und Becker 2008*

2.2.2. Deformed Steel Bars with Approval

<table>
<tr><th>Name</th><th>Grade of Steel
[Ductility Class]</th><th>Usage</th><th>Year</th><th>Charac. Yield Point
[N/mm^2]</th></tr>
<tr><td>“Isteg”-steel</td><td>min. St 37, cold-drawn by twisting [-]</td><td>1933-1942</td><td>1933</td><td>210</td></tr>
<tr><td rowspan="2">“Drillwulst”-steel</td><td>St 52 [B]</td><td rowspan="2">1937-1956</td><td>1937</td><td>260</td></tr>
<tr><td>Concrete steel IIIa [B]</td><td>1943</td><td>315</td></tr>
<tr><td rowspan="2">“Nocken”-steel</td><td>St 52 [B]</td><td rowspan="2">1937-1962</td><td>1937</td><td>260</td></tr>
<tr><td>BSt IIa, IIIa, IVa [B]</td><td>1943</td><td>315</td></tr>
<tr><td rowspan="2">“Tor”-steel</td><td>St 37 [-]</td><td rowspan="2">1938-1960</td><td>1938</td><td>210</td></tr>
<tr><td>Concrete steel IIIb [-]</td><td>1943</td><td>315</td></tr>
<tr><td rowspan="3">Transverse ribbed deformed concrete steel</td><td>BSt I [B]</td><td rowspan="3">1952-1972</td><td rowspan="3">1952</td><td>245</td></tr>
<tr><td>BSt IIa [B], III a[B], IV a [A]</td><td rowspan="2">315</td></tr>
<tr><td>BSt IIb; IIIb; IVb [-]</td></tr>
<tr><td>QUERI-steel</td><td>Concrete steel IVa [-]</td><td>1952-1972</td><td>1952</td><td rowspan="10">315</td></tr>
<tr><td>cold-drawn, ribbed deformed concrete steel</td><td>Concrete steel IIIb, IVb [-]</td><td>1956-1962</td><td>1956</td></tr>
<tr><td>“Rippen-Torstahl”</td><td>Concrete reinforcing bar IIIb [-]</td><td>1959-1972</td><td>1959</td></tr>
<tr><td>FILITON-steel</td><td>Concrete steel IIIb [-]</td><td>1965-1969</td><td>1965</td></tr>
<tr><td>HI-BOND-A-steel</td><td>Concrete steel IIIa [B]</td><td>1959-1972</td><td>1959</td></tr>
<tr><td>NORI-steel</td><td>Concrete steel IIIa, IVa [B]</td><td>1960-1972</td><td>1960</td></tr>
<tr><td>NORECK-steel</td><td>Concrete steel IIIb [-]</td><td>1960-1967</td><td>1960</td></tr>
<tr><td>Inclined ribbed deformed concrete steel</td><td>with “Einheitszulassung”
BSt IIIa [B]</td><td>1964-1972</td><td>1964</td></tr>
<tr><td>DIROC-Stahl</td><td>Concrete steel IIIa [B]</td><td>1964-1969</td><td>1964</td></tr>
<tr><td>Stahl Becker KG</td><td>Concrete steel IIIa [B]</td><td>1964-1969</td><td>1964</td></tr>
<tr><td rowspan="2">GEWI-steel</td><td>BSt 420500 RU (III) [B]</td><td>since 1974</td><td>1974</td><td>420</td></tr>
<tr><td>BSt 500 S (IV) [B]</td><td>since1984</td><td>1984</td><td>500</td></tr>
<tr><td rowspan="2">Deformed concrete steel from coils</td><td>BSt 500 WR (IV) [B]</td><td rowspan="2">since1984</td><td rowspan="2">1984</td><td rowspan="2">500</td></tr>
<tr><td>BSt 500 KR (IV) [A]</td></tr>
<tr><td>Deformed concrete steel (nuclear power engineering)</td><td>BSt 1100 [-]</td><td>since1988</td><td>1988</td><td>500</td></tr>
<tr><td rowspan="4">Deformed concrete steel</td><td>BSt 420/500 RUS [B]
BSt 420/500 RTS [B]</td><td>since1977</td><td>1977</td><td>420</td></tr>
<tr><td>BSt 500/550 RU (IV) [B]</td><td rowspan="2">1973 -1984</td><td rowspan="2">1973</td><td rowspan="2">500</td></tr>
<tr><td>BSt 500/550 RK (IV) [A]</td></tr>
<tr><td>BSt 500/550 RUS [B]
BSt 500/550 RTS [B]</td><td>1976-1984</td><td>1976</td><td>500</td></tr>
<tr><td>Deformed concrete steel from coils with special shaped ribs</td><td>BSt 500 WR [A]</td><td>since1991</td><td>1991</td><td>500</td></tr>
</table>

Chart 2.4: Design data for the characteristic yield point and ductility classes of deformed concrete steel bars with an approval during different time periods, according to *Fingerloos und Becker 2008*

2.2.3. Reinforcing Steel Mesh

Reinforcing Steel Mesh[1]	Grade of Steel [Ductility Class]	Usage	Year	charac. Yield Point [N/mm^2]
"Baustahlgewebe B.St.G." with plain bars	ST 55 (IVb)	1932-1955	1932	315
- with deformed bars N, Q, R-meshs	Concrete steel IV B	1957-1973	1957	315
Meshs with joints of plastic material		1964-1969	1964	
- with special shaped deformaton [3]		1968-1973	1968	
- with ribs				
- with plain ribs	BSt 500/550 GK (IVb)	1972-1984	1972	500
	BSt 500 G (IV)	since1984	1984	
- with deformed bars	BSt 500/550 PK (IVb)	1972-1984	1972	
	BSt 500 P (IV)	since1984	1984	
- with ribs	BSt 500/550 RK (IV)	1972-1984	1984	
	BSt 500 M (IV)	since1984	1984	
	BSt 630/700 RK	1977	1977	630
	BSt 550 MW	1989	1989	550

[1] Name of meshs by geometry
since 1955: Q – quadratic (Q 92 up to Q 377); R – rectangular (R 92 up to R 884); N – non static (N47 up to N 141);
since 1961: A 92, B 131 – boundary meshs
since 1972: Q – (Q 84 up to Q 513); R – (R 131 up to R589), K – rectangular (K 664 up to K 884); N – (N 94 and N 141);
since 1984: Q – (Q 131 up to Q 670); R – (R 188 up to R 589); K – (K 664 up to K 884)
[2] since 1957 two ranks of ribs; since 1962 with three ranks of ribs
[3] six ranks of ribs

Chart 2.5: Design data for the characteristic yield point of reinforcing steel mesh during different time periods, according to *Fingerloos und Becker 2008*

2.3. Safety factors for existing structures

The resistance of components and the actions on the components, are linked through the limit state function [14] defining the failure conditions. In these limit states all statistical information of resistance and actions are assimilated. The required statistical parameters can be traced from, for example, *Rackwitz, R. 1996, JCSS 2000* and *Spaethe, G*. 1992. Additional information about impacting actions on structures can also be found in *Rackwitz, R. 1996, CIB W81 1991* and *CIB W81 1996*.

In accordance with *DIN 1055-100; 2001* the safety index β_T = 4.7 for the reference period of one year was chosen as target value of reliability. All probabilistic investigations described in this section have been done with the program *COMREL 2007*.

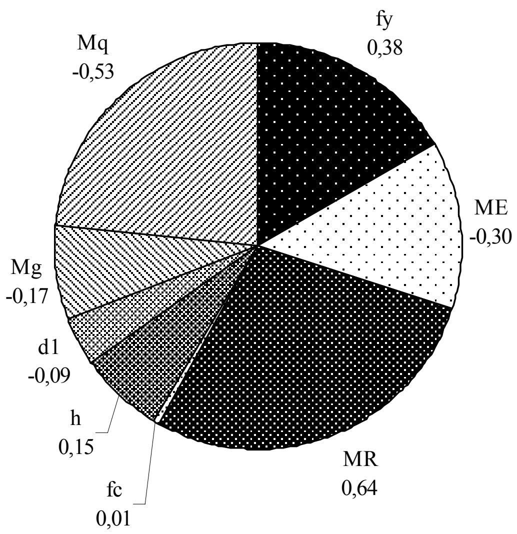
2.3.1. Assessment in case of bending of reinforced concrete elements

Looking on concrete components, lowly reinforced only, the ruling factors for the structural design in case of bending (i.e. flexural failure of the beam is decisive) are discussed subsequently.

Therefore the statistical parameters are taken from the literature. These characteristics reflect the average quality of execution, having been likely to be valid for the period beginning in the 1980s.

As a result of a probabilistic analysis using the First Order Reliability Method (FORM) according to Appendix B of DIN 1055-100 the reliability index β_T and sensitivity factors α_i can be concluded. The sensitivity factors are also known as importance factors, because they reflect the impact of each basis variable concerning to the reliability of the observed failure criterion of the component. Figure 2.2 shows a typical distribution of the sensitivity factors for flexural failure of weak reinforced concrete slabs, as a result of residential and office space usage

The influence of each variable concerning the reliability of the component becomes evident from the distribution of the different sensitivities of the basic variables. In general terms, the bigger the absolute value of the basic variables, the more it affects the reliability of the component.



with the basic variables

f_c	medial concrete compressive strength
f_y	medial steel tensile strength
M_R	model uncertainty factor resistance
M_E	model uncertainty factor actions
M_g	Moment Dead Load
M_q	Moment Life Load
h	Slab thickness
d_1	Edge distance of reinforcement

Fig. 2.2: Diagram depicting of the distribution of the sensitivity factors for flexural failure of low reinforced concrete slabs for g / q = 70 / 30

Figure 2.2 shows that the uncertainty of the mechanical model and the moment of the variable action of live load M_q provide an significant impact on the target reliability β_T. Similarly, quite large an impact of the reliability is related to the steel strength fy and the model uncertainties on the load side. These values strongly affect the reliability relatively in contrast to the concrete compressive strength f_c which has almost no influence. The component height h, the distance of reinforcement to the component surface d_1 and the moment due to dead load M_g give a moderate peripheral affect on the reliability only.

The sensitivity factors of the basic variables have a negative sign in case the basic variables influence the reliability of the component negatively considering the failure criterion.

2.3.1.1. Influence of the ratio of dead and life load

Fluctuating actions possess - because of their large coefficient of variation - a significant influence on the component reliability. As limiting type of case, assumed in the parameter studies it is deemed that the maximum life load is as large as the dead load.

The surplus load counting for partition walls may be assigned for the load ratio g/q on the side of the permanent actions.

Figure 2.3 shows the considerable lower reliability for the higher live load portion adopted for a percentage of longitudinal reinforcement of 0.06% for instance.

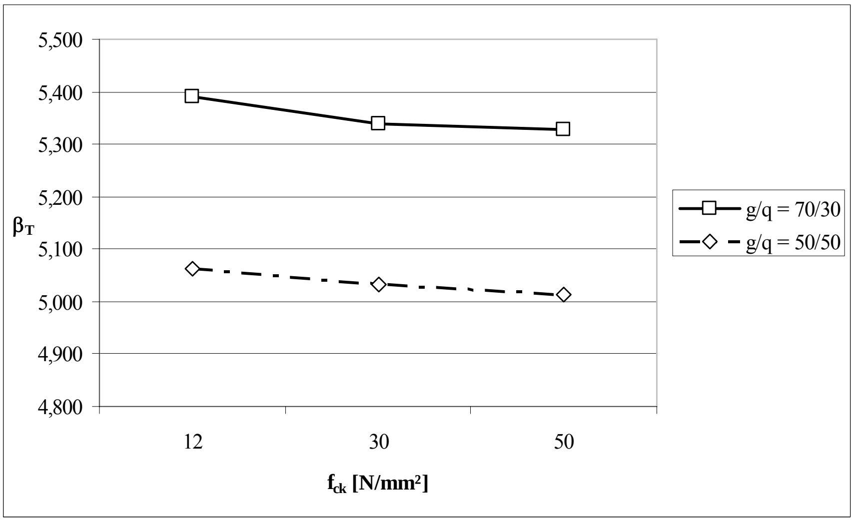


Fig. 2.3: Reliability index β_T depending on the concrete compressive strength for different ratios of dead load versus live load applying the partial safety factors γ_c = 1,50 and γ_s = 1,15 (percentage of longitudinal reinforcement of 0.06%)

2.3.1.2. Influence of concrete compressive strength

It should be noted that for bending elements the reliability declines with an increasing concrete compressive strength grade. Looking at the reliability on cross-sectional level for flexural failure of slabs and beams, the values of the reliability index β_T are shown in the figure 2.4.

2.3.1.3. Impact of percentage of longitudinal reinforcement

Figure 2.5 reveals clearly that the percentage of longitudinal reinforcement of components impart a decisive influence on its reliability. The greater the degree of reinforcement, the greater the reliability. The justification among other things is found in the small variation of steel tensile strength. Extensive studies on this feature are illustrated in the bulletin.

Studying the impact of percentage of longitudinal reinforcement on the component reliability, the variation of the partial safety factor for concrete γ_c addicts no further significant reduction in reliability as expected.

2.3.1.4. Optimized partial safety factors for flexural failure

For flexural failure of floors in residential construction the following findings have been identified:

- The higher the concrete strength class of the component is supposed, the lower the reliability of the floor slab for flexural failure to cope with.
- The component reliability increases with the growth of the percentage of longitudinal reinforcement ρ_l.
- The reliability of the floor slab declines significantly with the increase of the ratio of imposed live load.
- The variation of concrete compressive strength f_c has a minor impact on the reliability of floor slabs under flexural failure.

Even with the increase of component thickness a slight growth of reliability is recognized. If the as-built stocktaking confirmed the dimensions, the scope of reinforcement scope, the material data and the structural system as well as a state of structure free of damages state of structure, a reduction of partial safety factors for concrete to γ_c = 1.20 and for reinforcement steel to γ_s = 1.10 is suggested.

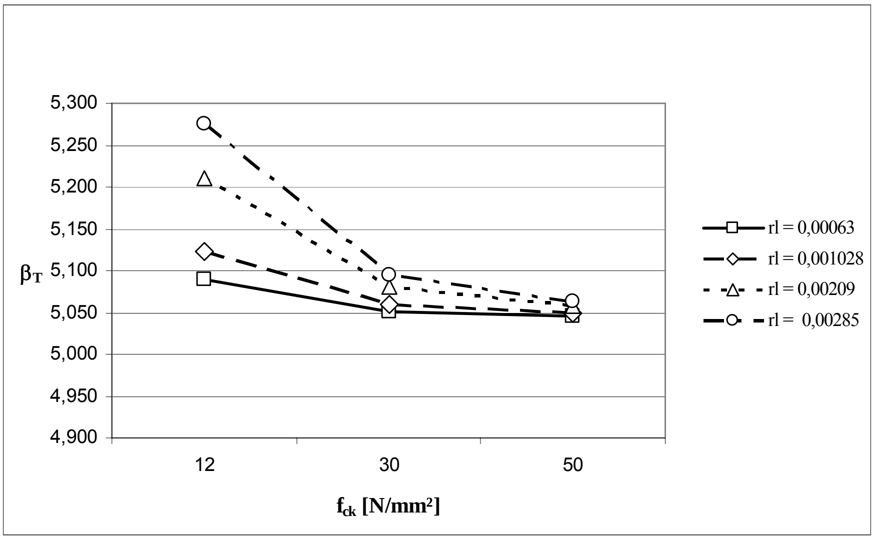


Fig. 2.4: Reliability index β_T depending on the concrete strength for different percentages of longitudinal reinforcement ρ_l with the load ratio g / q = 70 / 30, using the partial safety factors γ_c = 1.20 and γ_s = 1.10

Figure 2.4 shows the reliability index for reduced partial safety factor. The reliability value will never fall below β_T = 4.7.

The study was made for load ratio g / q = 70 / 30, only. Previous studies have shown that for the load ratio of g / q = 50 / 50 the reliability index decreases from about 0.3. Therefore even in case of equal load fractions for dead load and live load the reduced safety coefficients of γ_c = 1.20 and γ_s = 1.10 are justified.

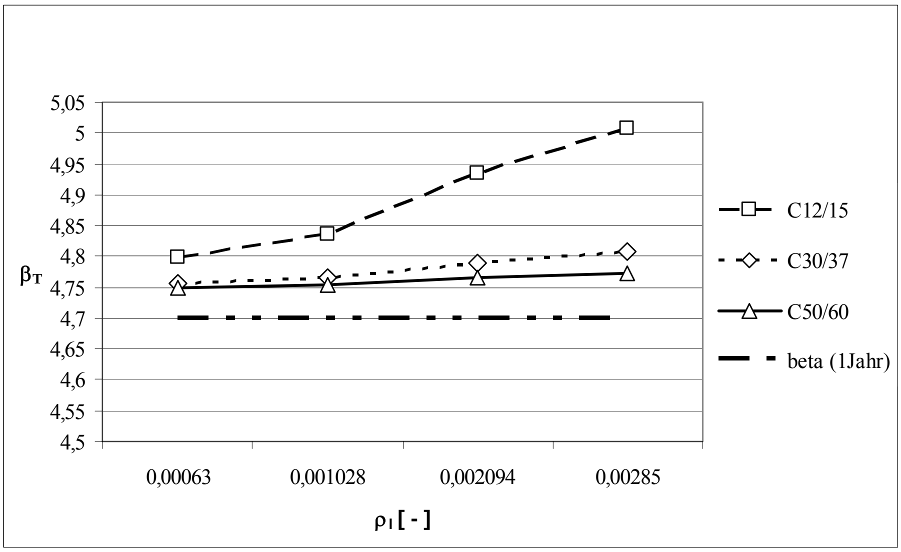


Fig. 2.5: Reliability index β_T depending on longitudinal percentage of reinforcement ρ_l for different concrete strength classes with the load ratio g / q = 70 / 30 using the partial safety factor γ_c = 1.20 and γ_s = 1.05

For the load ratio g / q = 70 / 30, in any case a scope for a further reduction of the material factor for reinforcing bars is still available. In figure 2.5 it can be recognized that for reduced safety coefficient of γ_c = 1.20 and γ_s = 1.05 the reliability does not undergo β_T = 4.7.

2.3.2. Implementation in practice

The results of the parameter study for residential and office usage have been merged in a flowchart (figure 2.6). Lots of parameter studies for normal solid concrete C12/15 up to C50/60 with different coefficients of variation were made. Likewise the dispersion of reinforcing steel strength was investigated.

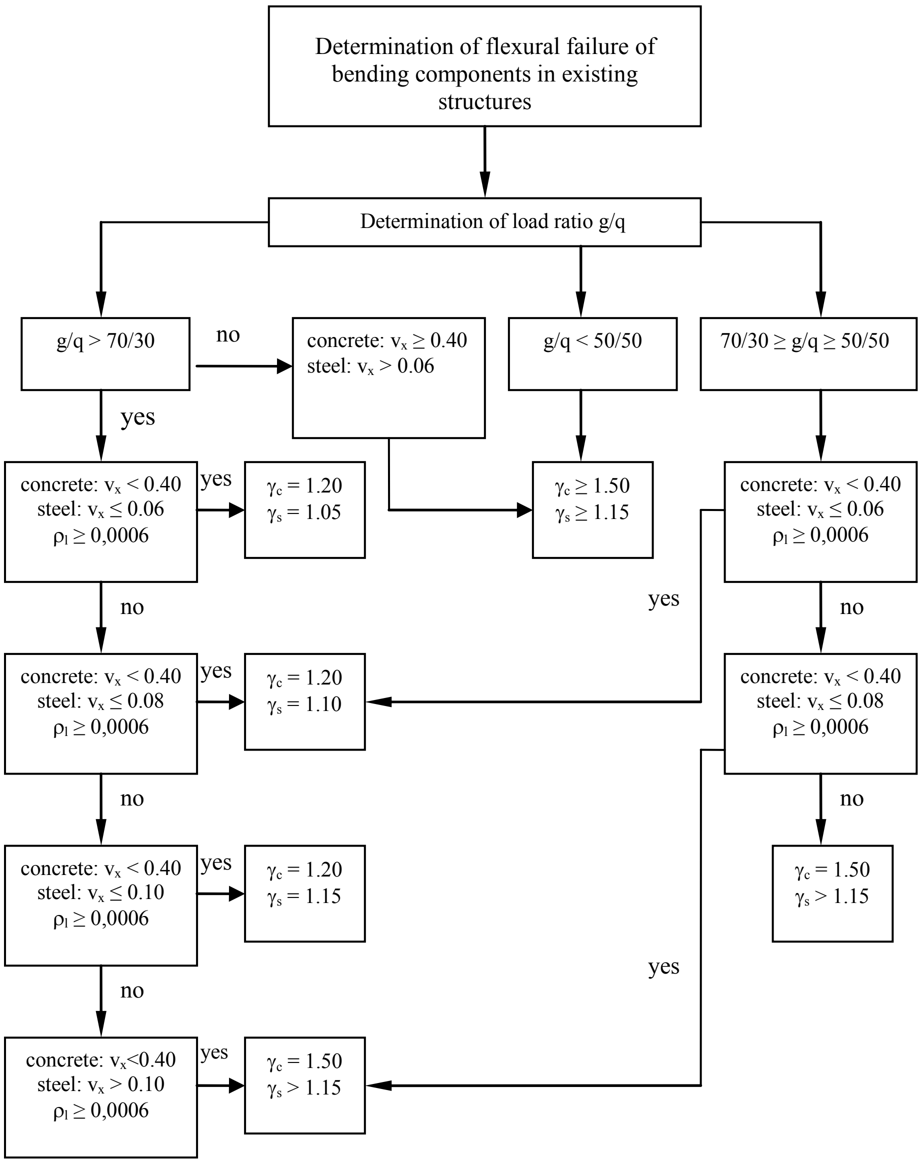


Abb. 2.6: Compilation of modified partial safety factors for the analysis of reinforced concrete components in bending of existing structures after as-built survey

It should also be pointed out that for very large dispersion of material properties it might be necessary to use a larger partial safety factor for reinforcement steel as fixed in the current standard. Also different ratios of actions especially if the live load is bigger than the dead load, quite large partial safety factor on the material side may occur.

It should also be pointed out that with the proposed modification of partial safety factors on the material side, no reduction of the safety coefficients on the action side is allowed.

An analogous approach for the verification of shear load capacity and also for the proof of compact compression members is inclosed. The following recommendations can be given if the variation of concrete compressive strength is of $v_x < 0.40$ and for reinforcing steel $v_x \leq 0.06$ has been established:

For concrete slabs without shear reinforcement and without risk of shear failure due to punching, a modified partial safety factor of concrete $\gamma_{c,mod} = 1.20$ can be scheduled when a load ratio $g / q \geq 70 / 30$ is existent. For compression members which are verified after the theory of first order modified partial safety factors lying on the safe side for concrete $\gamma_{c,mod} = 1.20$ and reinforcing steel $\gamma_{s,mod} = 1.05$ can be used and concrete compressive strength showing a variation of $v_x < 0.20$. However, this is applicable only if the existing stirrups comply with the minimum diameter and maximum distance according to code DIN 1045-1.

3. Literature und Code of Practices

CIB W81 1991: Actions on Structures, Snow Loads, Report No. 141, Rotterdam, September 1991.

CIB W81 1996: Actions on Structures, General Principles, Report No. 201, Rotterdam, November 1996.

Hansen, M. 2004: Zur Auswirkung von Überwachungsmaßnahmen auf die Zuverlässigkeit von Betonbauteilen. Dissertation, Uni Hannover, 2004

JCSS 2000: Probabilistic Model Code, Joint Committee on Structural Safety, www.jcss.ethz.ch, 2000.

Rackwitz, R. 1996: Einwirkungen auf Bauwerke. In: Mehlhorn, G. (Hrsg.): Der Ingenieurbau: Tragwerkszuverlässigkeit, Einwirkungen, Ernst & Sohn Verlag, Berlin, 1996.

RCP 2004: STRUREL, A Structural Reliability Analysis Program-System, Modul: COMREL, RCP GmbH. München, 2004.

Spaethe, G. 1992: Die Sicherheit tragender Baukonstruktionen. 2., neubearbeitete Auflage. Springer-Verlag, Wien, 1992.

DB-Richtlinie 805; 2002: Tragsicherheit bestehender Brückenbauwerke. Deutsche Bahn, gültig ab 01.09.2002.

DIN 1045-1; 2001-07: Tragwerke aus Beton, Stahlbeton und Spannbeton - Teil 1: Bemessung und Konstruktion.

DIN 1055-100 2001: Einwirkungen auf Tragwerke - Teil 100: Grundlagen der Tragwerksplanung, Sicherheitskonzept und Bemessungsregeln; Deutsches Institut für Normung e.V., Fassung März 2001.

DIN EN 206-1; 2001-07: Beton - Teil 1: Festlegung, Eigenschaften, Herstellung und Konformität.