Beton- und Stahlbetonbau 05/2008
No. 614

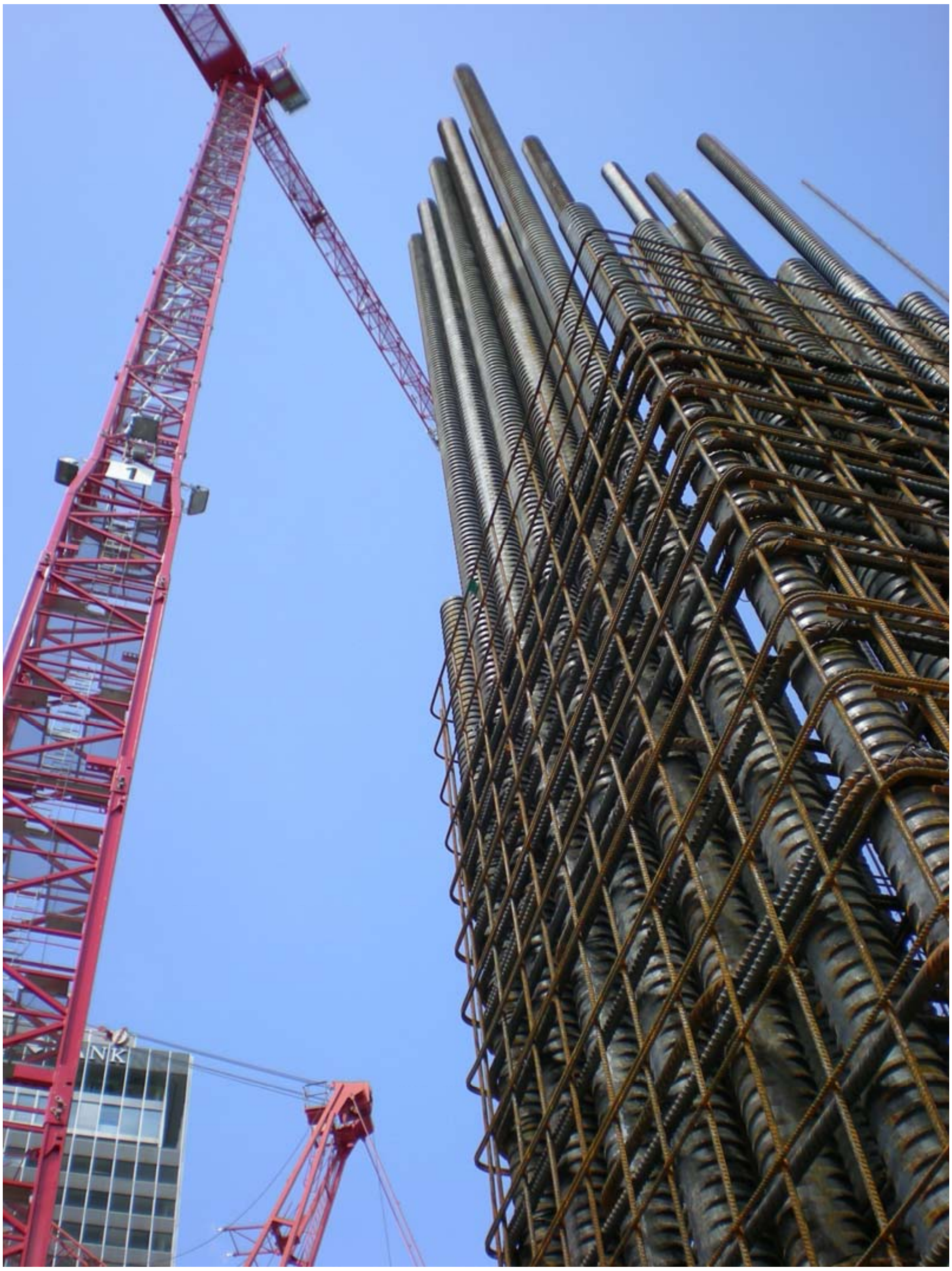
Main issue
DOI: 10.1002/best.200800614

The new reinforcement system; compression members with SAS 670 high-strength reinforcement steel

Special reprint

Part I:

Development, testing, design, construction



SAS SYSTEMS

SAH Stahlwerk Annahütte

Part I:

Development, testing, design and construction

**Prof. Dr.-Ing., Dr.-Ing. E.h. Horst Falkner, Dr.-Ing. Dominique Gerritzen,
Prof. Dr.-Ing. Dieter Jungwirth, Univ.-Prof. Dipl.-Ing. Dr. techn. Lutz Sparowitz**

Beton- und Stahlbetonbau 05/2008 DOI: 10.1002/best.200800614
Main Issue

THE NEW REINFORCEMENT SYSTEM; COMPRESSION MEMBERS WITH SAS 670/800 HIGH-STRENGTH REINFORCEMENT STEEL

PART I: DEVELOPEMNT, TESTING; DESIGN AND CONSTRUCTION

Horst Falkner, Dominique Gerritzen, Dieter Jungwirth, Lutz Sparowitz

ABSTRACT

The scope of conventional reinforced concrete construction is being extended through the use of new and innovative reinforced concrete compression members: SAS 670/800 threadbars with diameters of up to 75 mm, couplings and end anchorages, developed by the Annahütte steelworks. These high-quality compression members can be produced economically in small dimensions and with reinforcement percentages of up to 20%. Taking creep and shrinkage of the concrete and/or the use of ductile fibre-reinforced concretes into consideration, the yield strength of SAS 670/800 high-strength reinforcement steels, whose yield strength exceeds the compression yield point ε_{c2} , can be fully exploited.

For BST 500 reinforcement steel, the valid standard only allows an ultimate compressive strain of ~2‰, with diameters of up to ≤ 40 mm and permissible reinforcement percentages in the overlapping zone of max. 9%.

Part I discusses modified design and construction concepts based on the Eurocodes as well as the tests performed. Part II deals with the mega columns of the Frankfurt Opernturm constructed using this concept under a construction approval on an individual basis.

At present, national technical approvals for the system have already been issued or are in the process of being issued in several countries such as USA, Russia, Austria and other European countries.

DOI: 10.1002/best.200800614

1. Historical development

In 1972 [1], *Leonhardt/*Teichen, followed by *Falkner* [2], [3], already tested prestressing bars used as compression reinforcement. They installed and exploited the bars up to the compression yield point (section 5.1).

According to the Eurocode and national design regulations derived from it, the ultimate compressive strain of columns, assuming high loading rates, may only be ~2‰. Bar diameters are limited to 40 mm, the reinforcement percentages in the coupling zone to 9%. In the USA the ACI-Code 318-05 allows an ultimate compressive strain of 3‰.

The Annahütte steelworks have continued to develop BSt 500 threadbar steel. SAS 670/800 threadbars, also known as S 670, Ø18 to 75 mm, have been successfully used for years in geotechnical applications as piles, nails or anchors. The next obvious step, with the support of the authors of this report, was to promote the use of this system in combination with coupler splices and end anchors in reinforcement applications and thus to further advance reinforcement technology (Fig. 1).

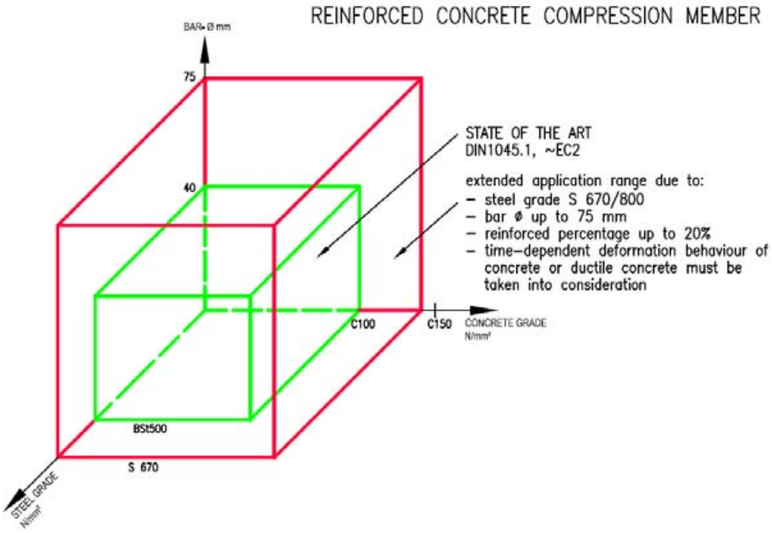


Fig. 1: Extending the State of the Art

If redistributions due to shrinkage and creep are taken into consideration or fibre-reinforced concrete is used, S 670 high-strength reinforcement steels may be exploited up to a compression yield point of 670 N/mm^2.

 DOI: 10.1002/best.200800614

2. The new reinforcement system; advantages, load bearing behaviour and application range

2.1 Advantages

Fig. 2 shows the economical advantages of S 670 as a substitute or with cross section reduction.

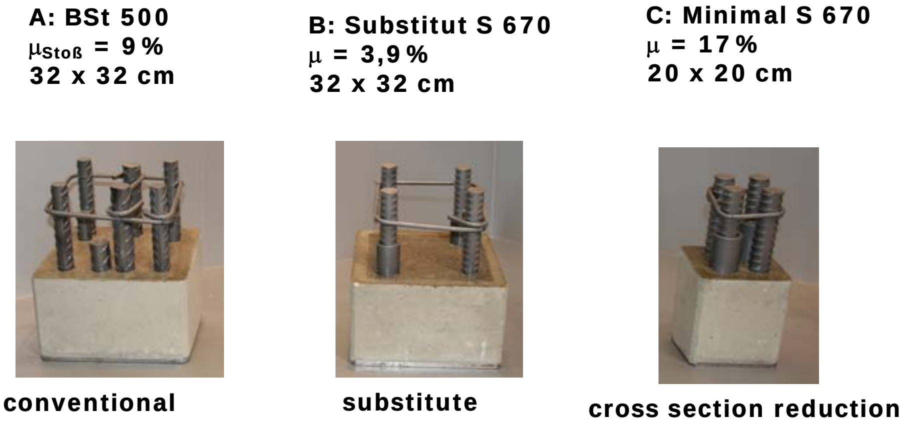
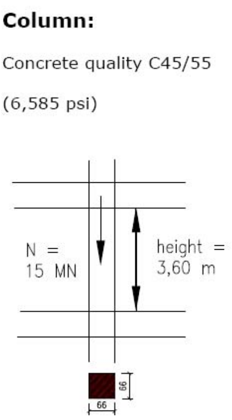


Fig. 2: Cross section reduction by using S 670 and increasing the reinforcement percentage, while retaining the same load bearing capacity; C35/45; 50% splicing

The easily mounted threadbar, preferably spliced using compression couplers, has a more favourable cost-benefit ratio than BSt 500 reinforcement steel with overlap splices. Thus, for the specified reinforced concrete column, the use of S 670 as a *substitute* reduces the costs of the columns by approx. 10% (Table 1a).

Table 1a: Savings of steel and wage costs for equal dimensions (S 670 replaces BSt 500)



Steel (reinforcement percentage)	BSt 500 S (6 %)	SAS 670/800 (3,7 %)
Column cross section a x a [cm]	66 x 66 12 Ø 40 mm 14 Ø 32 mm 50 % overlap splicing	66 x 66 4 Ø 63,5 mm 2 Ø 43 mm 3 couplers (easy pourable)
	233 kg/m	125 kg/m + 3 couplers
Longitudinal reinforcement [%]	100	54
Installation period [%]	100	43
Total costs per column €; €/MNm	100 % 1.030; 20,4	90 % 930; 18,4

If the S 670 reinforcement percentage limit of approx. 20% is fully exploited, the costs are approx. 10% higher as compared to a column reinforced using BSt 500 with a reinforcement percentage of 6% (in the unspliced area); the weight, however, decreases by approx. 50% and the cross-sectional area by approx. 55% (Table 1b).

Table 1b: Cost savings per column with cross section reduction

Concrete	**C45/55** **Standard concrete**	**C45/55** **Standard concrete**	**C 100/115** **High-strength concrete with fibres**	**C 100/115** **High-strength concrete with fibres**
Steel **Reinforcement percentage [%]**	BSt 500 S 6	SAS 670/800 18	BSt 500 S 6	SAS 670/800 18
Cross section a x a [cm]	66 x 66	40 x 40	50 x 50	36 x 36
Cross sectional area [m²] [%]	**0.436 100**	**0.160 37**	**0.250 57**	**0.130 30**
Production costs per column [€]	**1,030**	**1,100**	**960**	**1,050**
Increased floor space [m²] [%]	**0 0**	**0.28 63**	**0.19 43**	**0.31 70**
Cost savings with 10,000 €/m² [€	**0**	**2,800**	**1,900**	**3,100**
Weight [t/m] [%]	**1.14 100**	**0.53 46**	**0.66 58**	**0.44 39**

Columns constructed using SAS 670/800 offer considerable cost advantages and more design freedom.

If, due to the 55% *increase in floor space,* the rental income for 30 years is assumed to be 10,000 €/m², cost savings for a calculated high-rise building comparable to the Frankfurt Opernturm would amount to more than 3%.

Another great advantage is that an already minimized column constructed using high-strength concrete and reinforced with BSt 500 may be replaced economically by a column constructed using more inexpensive normal concrete of the same dimensions but reinforced with S 670.

In addition, the architect has more design freedom due to the improved use of identical floor plans over several floors. Avoiding overlap splices facilitates pouring of S 670 columns.

The absence of bursting forces and compressive forces at the bar splice increases the quality.

2.2 Load bearing behaviour

One possibility to fully exploit the steel up to the compression yield point in the ultimate limit state is to take concrete creep and shrinkage, already present in most cases, into consideration. Fig. 3 shows the resulting typical time-dependent internal force flow within the compression member.

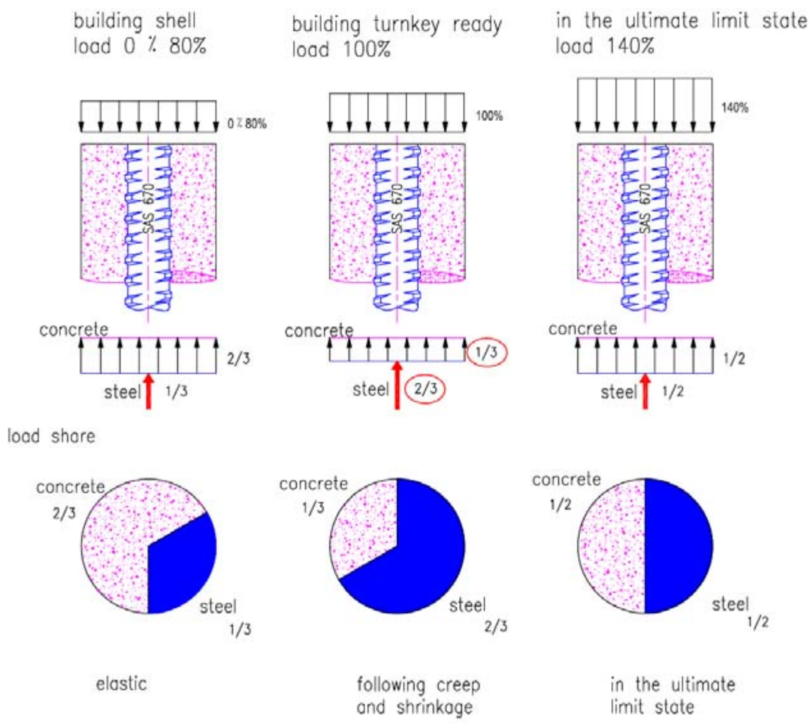


Fig. 3: Redistribution due to creep and shrinkage from the building shell up to the ultimate limit state (C45/55, μ~13%, φ = 1.5, ε_s = 0.3‰) – Load shares of concrete and steel

Initially, upon load introduction, the concrete bears 2/3 and the steel 1/3 of the load. Following redistribution due to creep and shrinkage, 1/3 of the load remains in the concrete and 2/3 remain in the steel of the building which will by that time already be in operation. In the calculated ultimate limit state, the concrete and steel bear an approximately equal share of the fracture action effects. Due to unloading in the serviceability limit state, the concrete gains reserves in regard to e.g. earthquakes.

 DOI: 10.1002/best.200800614

2.3 Application range

The application range primarily includes every type of compression member used with concrete strengths of C25/30 to C100/115 or .higher and subjected predominantly to static and non-static loading as well as earthquake loading. Thick plates as well as structural elements subjected to moderate tensile and bending stresses may be reinforced using S 670. Due to large deformations and inadmissible crack widths, full exploitation of the yield strength under tension, particularly with large diameters, is only possible in a limited number of cases [4].

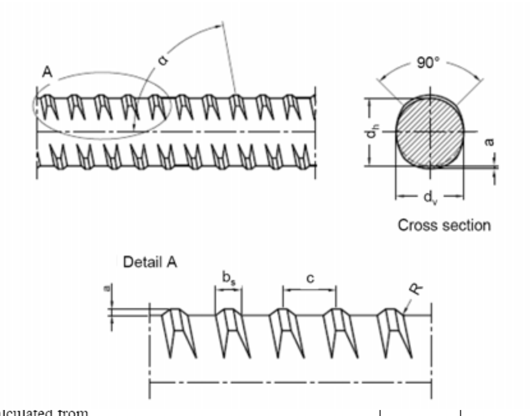
3. S 670 steel and accessories

3.1 Steel[1]

The robust, tempered, water quenched S 670 reinforcement steel complies with DIN 488 rsp. EN 10080 as well as with the requirements for reinforcement steel specified in the Eurocode. The required approval tests according to EN ISO 15630-1 have been carried out. Diameters range from Ø18 to 75 mm. The mechanical parameters are summarized in Table 2.

Table 2: Mechanical parameters of S 670 (no likelihood of confusion with other threadbars due to right-hand thread and different pitch)

Parameters and requirements					
	Nominal diameter d_S	Nominal cross section deviation ± 4.5%	Characteristic		Nominal mass
			Yield load F_e	Ultimate load F_m	
	[mm]	[mm²]	[kN]	[kN]	[kg/m]
1	18	254	170	204	2.0
	22	380	255	304	2.98
	25	491	329	393	3.85
	28	616	413	493	4.83
	30	707	474	565	5.55
	35	962	645	770	7.55
	43	1,452	973	1,162	11.40
	57.7	2,597	1,740	2,077	20.38
	63.5	3,167	2,122	2,534	24.86
	75	4,418	2,960	3,535	34.68



2	Ch		)
3	Ch		)
4	Ra		10
5	To (calculated from		0
6	Relative rib area f_R		≥ 0.075
7	Fatigue strength $2 \cdot \sigma_A$ (under an upper load of $\sigma_o = 0.7 \cdot R_{e,nom}$ and $N = 2 \cdot 10^6$ load cycles	[N/mm²]	d_s = 18-43: 150 d_s = 57.5-63.5: 120 d_s = 75: 100
8	Bendability	up to 43 mm	
9	Weldability	weldable	
1) 5% fractile value 2) R_e corresponds to yield strength $R_{p0.2}$ 3) E ~ 205 000 N/mm²			

[1] Technical data of the steel and methods (accessories) as well as the approvals may be requested from Annahütte D-83404 Ainrnig

S 670 steel has a relative rib area of $f_R \geq 0.075$ which is larger than that of reinforcement steel according to DIN 1045-1. The bond characteristics of the S 670 threadbar in the favourable bond zone are shown in Fig. 4. Based on these, the load introduction lengths and steel/concrete displacement may be determined according to [5] using differential equations rsp. the mean bond stresses f_{bd} complying with EC 2 and DIN 1045-1, see Fig. 4, Table. Since the influence of the diameters is limited, the mean value of all diameters was taken as a basis with a variation coefficient of 20%.

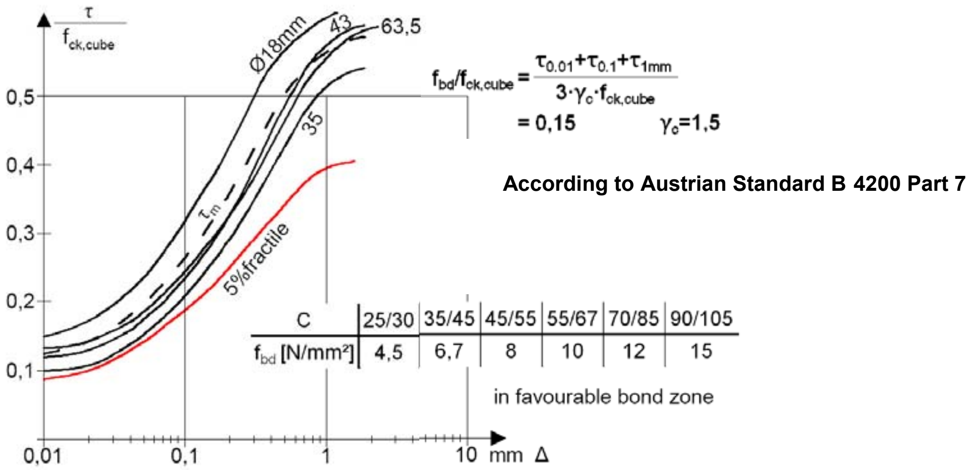
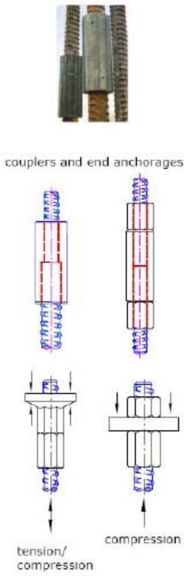


Fig. 4: Bond stresses dependent on relative displacement according to [6]

3.2 Accessories[1]

As a compression splice, a hand-tightened contact coupler is used with full static and dynamic load bearing capacity at a slip of < 0.2 mm in the serviceability limit state. The bar can also transfer its load to a steel plate designed according to steel construction standards or DIN 1045-1 through a butt joint. Locked anchor pieces (end anchorages) transfer tensile rsp. compressive forces into the concrete. Centre and edge distances are shown in Fig. 5.

[1] Technical data of the steel and methods (accessories) as well as the approvals may be requested from Annahütte D-83404 Ainrnig



End anchorage with anchor piece free length 10 Ø C20/25			
Ø	Centre distance A [mm]	Edge distance R [mm]	Additional reinforcement
35	360	185 + c	6 Ø 12/80
43	480	230 + c	8 Ø 12/80
57[5]	640	310 + c	13 Ø 12/55
63[5]	710	345 + c	15 Ø 12/50
75	840	410 + c	5 Ø 20/160
End anchorage with anchor piece free length 5 Ø C80/95			
Ø	Centre distance A [mm]	Edge distance R [mm]	Additional reinforcement n x Ø/a [mm]
35	125	55 + c	6 Ø 6/45
43	155	70 + c	6 Ø 8/40
57[5]	210	95 + c	7 Ø 6/40
63[5]	230	105 + c	8 Ø 8/37
75	270	125 + c	10 Ø 8/37

Fig. 5: Couplers and end anchorages

Under moderate tensile loading, locked and largely slip-free coupler splices are used. Transition couplers enable load transfer to smaller diameters.

The end anchorages and locked coupler splices can be subjected to seismic loading according to EC 2 and EC 8. The specified accessories were tested according to German and European guidelines such as ETAG013, ISO/DIS 15835-1 to 2 (e.g. low cycle fatigue). Several national approvals have been issued.

4. Basic principles of the load bearing behaviour of composite concrete/steel compression members

4.1 Concrete

The limit curves of the stress-strain curves for normal concrete for various loading rates and fatigue loading (creep) are shown in Fig. 6 [7].

 DOI: 10.1002/best.200800614

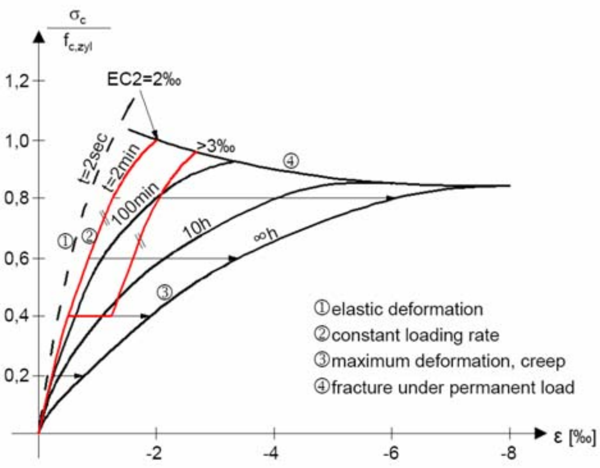


Fig. 6: Stress-strain curves for normal concrete according to [7]

As parameters, graphs representing a high loading rate of t = 2 min resulting in an ultimate compressive strain of 2‰ and a low loading rate of t = 100 min resulting in an ultimate compressive strain of > 3‰ have been plotted. Quick loading with an extended creep phase at 0.4 $\sigma_c/f_{c,zyl}$ also leads to an ultimate compressive strain of ~3‰.

Fibre-reinforced concretes do not have considerably higher ultimate compressive strains than normal concrete. They are however more ductile.

Data on the fire behaviour are defined in EN 1992-1-2 and DIN 4102.

4.2 Steel

Showing bi-linear behaviour in the relevant area, the yield strength/compression yield point is reached at 670/205000 = 0.00327 = 3.27‰. The stress-strain graph under compression equals the stress-strain graph under tension (Fig. 7).

The fire behaviour corresponds to that of Tempcore steel and is also defined in EN 1992-1-2 and DIN 4102. For more specific values see [8].

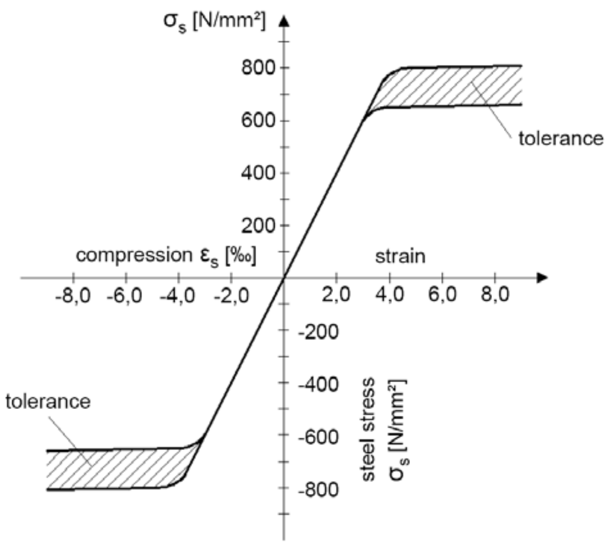


Fig. 7: Stress-strain/compressive strain curve for S 670

Centrically loaded reinforced concrete compression member

Under short-term loading, n_0-fold concrete stresses σ_C occur in the steel σ_S (plane cross sections according to *Bernoulli*).

$$n_0 = E_s/E_c \qquad (1)$$

Based on the ideal area of $A_i = A_c + n_0 \cdot A_s$ (2)

the concrete and steel stresses can be determined

$$\sigma_c = P/A_i \qquad (3)$$

$$\sigma_s = n_0 \cdot \sigma_c \qquad (4)$$

The time-dependent redistribution of the stresses from concrete to steel can be determined using creep and shrinkage functions [9] according to *Trost* or, for more simplified calculation, using the deformation modulus:

$$E_{ct} = E_c/(1+\varphi) \qquad (5)$$

where $n_t \cdot \sigma_c$

$$n_t = E_s \cdot (1+\varphi)/E_c \qquad (6)$$

Where φ = creep coefficient

In the calculated ultimate limit state, the ultimate loads of concrete and steel are added. If the cross sections are fully precompressed, the compressive strain of the steel may, according to

DIN 1045-1, not exceed 2 to 2.2‰ irrespective of the steel grade, e.g. $\sigma_{su} = 0.002 \cdot 205000 = 410$ N/mm^2.

As a rule, the ultimate bearing capacity is therefore determined as follows:

permissible concrete load bearing capacity $= \alpha \cdot f_{ck} \cdot A_c/(\gamma_c \cdot \gamma_{c'})$ (7.1)

plus steel load bearing capacity $= \varepsilon_{c2} \cdot E_s \cdot A_s/\gamma_s$ (7.2)

Where:

α	=	Long-term influence 0,85
f_{ck}	=	Characteristic cylinder compressive strength of concrete
A_c	=	Net - concrete cross section
A_s	=	Steel cross section
ε_{c2}	=	Ultimate compressive strain of precompressed concrete = 2.0‰, from C55 to C100 increasing proportionally to 2.2‰
E_s	=	Steel E-modulus 205000 N/mm^2
E_c	=	Concrete E-modulus (acc. to DIN 1045-1)
γ_c	=	Concrete partial safety factor 1.5
$\gamma_{c'}$	=	Additionally as of concrete = C55 = $1/(1.1 - f_{ck}/500))$
γ_s	=	Steel partial safety factor 1.15

If the time-dependent redistribution in the serviceability limit state is verified or concrete with demonstrably improved post-failure behaviour is used, the yield strength of higher-strength steels may also be fully exploited under compression. During redistribution, the steel stresses increase whereas the concrete stresses decrease. This corresponds to "compressive preloading" in the reinforcement which sets in independently during the creep process.

Creep and shrinkage redistributions delay the ultimate compressive strain of the concrete. The concrete is de-stressed gaining load-bearing reserves.

Fig. 8 shows the respective loads borne by the concrete and steel (reinforcement percentage μ~13%, C55) from the serviceability to the ultimate limit state under quick loading (dotted

 DOI: 10.1002/best.200800614

line) and under slow loading with creep phase (continuous line), see also section 5, Testing. The different deformations $\varepsilon_1 + \varepsilon_2 + \varepsilon_3$ which can be used to increase the ultimate compressive strain in the steel are clearly discernible: $\Delta\varepsilon_s = \varepsilon_1+\varepsilon_2+\varepsilon_3$ (8).

ε_1= Creep and shrinkage deformation

ε_2= Restressability of the concrete following creep and shrinkage decrease/breathing

ε_3= Deformation difference due to varying load rates (subsequently generally disregarded)

Similar conditions prevail in relaxation tests (section 5.2).

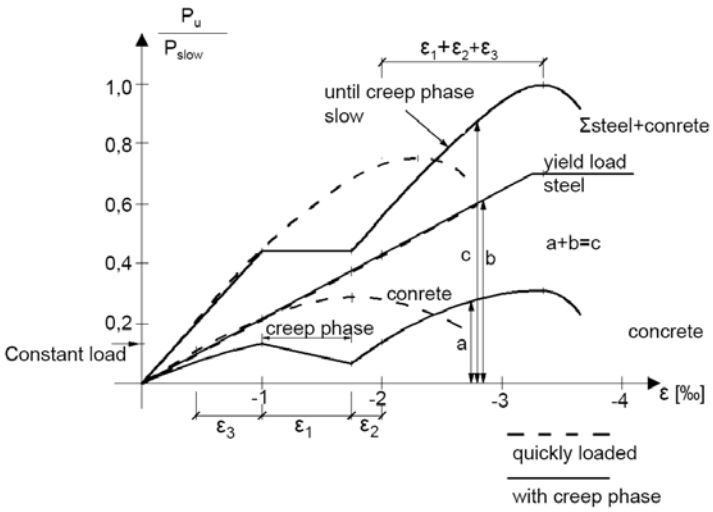


Fig.8. Load graphs covering the entire load history under centric compression until failure of a quickly loaded column and a slowly loaded column with creep phase

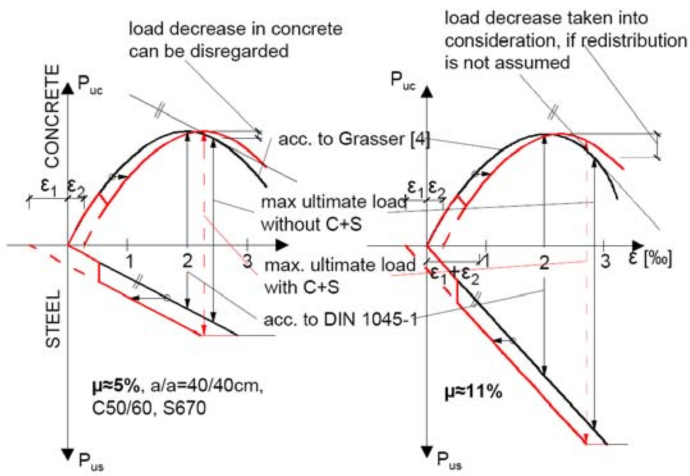


Fig. 9. Ratio of concrete - steel forces under centric compression with different S 670 reinforcement percentages (post-failure behaviour)

Fig. 9 clearly shows the post-failure behaviour of concrete described above. Its effect is dependent on the reinforcement percentage. With low μ~5%, failure sets in shortly after attainment of the maximum concrete load-bearing capacity, if the ultimate load in the concrete decreases more rapidly than it increases in the steel.

With high reinforcement percentages, this maximum state is reached later, so that the yield load in the steel is attained and thus a high overall fracture load even without creep redistribution. This is generally described as the supporting effect of reinforcements with high reinforcement percentages. In both cases, the overall fracture load could of course still be further increased if the creep redistributions are taken into consideration, as is shown by the red lines.

With BSt 500 reinforcement steel, these mechanisms have a less pronounced effect. Fibres in the concrete increase this effect.

4.4 Creep redistribution dependent on reinforcement percentage μ

Since the new reinforcement technique is also intended to be used in areas with reinforcement percentages of μ > 6%, their influence on redistribution is of interest. Fig. 10 shows this dependency.

With low μ, the yield strength/compression yield point is quickly reached, the redistribution load is however low. With increasing μ, the steel stresses increase more slowly, the concrete however is quickly de-stressed and the redistribution load increases.

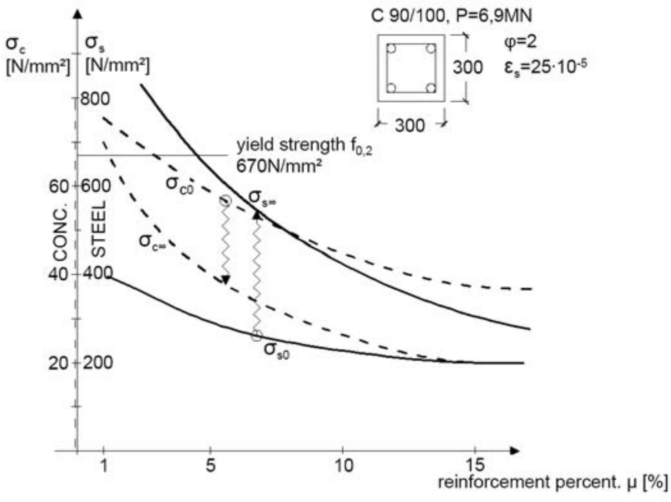


Fig 10: Influence of the reinforcement percentage on redistribution under constant external load P (as compared to Fig. 16)

DOI: 10.1002/best.200800614

4.5 Eccentrically loaded compression members

The above statements apply to centric compression loading (system statically indeterminate to the first degree, see also Fig. 15). In the case of additional bending loading and more complicated cross-sectional shapes (statically indeterminate to the second degree or multiple degrees of indeterminacy), the system may be decoupled using the creep fibre method according to *Busemann* [9] and reduced to two centrically loaded columns of two creep fibres, KI and KII (Fig. 11). Similar to the method of core point moments, the two fibres/columns do not influence each other. Concrete and steel areas as well as the action effects of nominal force and moment (load pair M/c) are proportionally assigned to fibres KI and KII according to the lever principle. The time-dependent redistributions $\Delta\varepsilon_S$ in the serviceability limit state within the respective centrically loaded columns can again be easily determined and be included as compressive prestress in fracture design calculation.

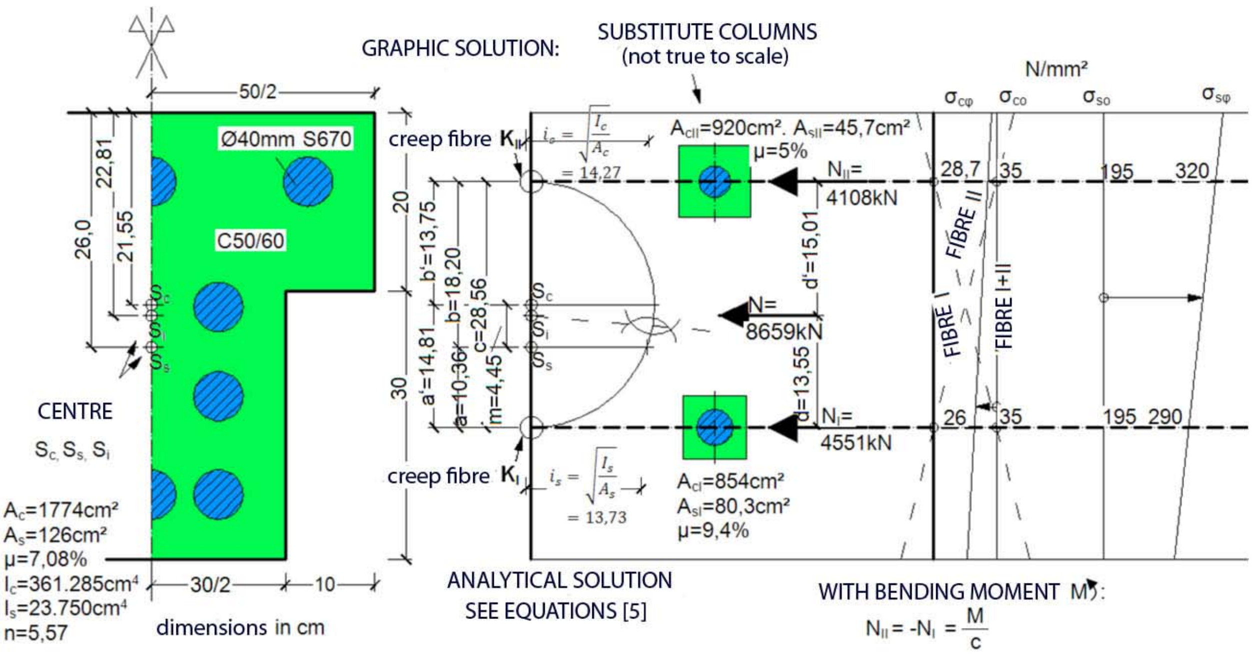


Fig 11: Decoupling according to Busemann into two centrically loaded compression members [9] KI and KII

In addition to this analytically clear and consistent solution, simplifications are recommended, e.g. redistribution $\Delta\varepsilon_S$ induced only by the centric normal load and displacement of the parable/rectangle stress diagram according to DIN 1045-1 by $\Delta\varepsilon_S$, see section 6.2.3. A more precise and higher utilization of redistribution is not necessary (safe side), since concrete is compressed up to 3.5‰ even without redistribution in the moment compression zone. This means that in most cases the steel reaches the compression yield point in any case. If the steel does not reach the compression yield point under an assumed concrete edge strain of 3,5‰, the redistributions should be verified in more detail. The required reinforcement decreases relative to the yield points as compared to column design using BSt 500.

5 Testing

5.1 Previous tests

In 1972, *Leonhardt and Teichen* already tested columns consisting of concrete βw = 50 N/mm^2 and St 600/900 high-strength steel [1]. The test columns had reinforcement percentages between 13 and 16%. In this case, as with the columns constructed using S 670, the time-dependent behaviour of the concrete was taken into consideration. *Leonhardt and Teichen* subjected column sections of 20 × 20 × 250 cm to centric and eccentric dead and permanent loads and held these loads for an extended period between 70 days and one year. The subsequent fracture test revealed that the concrete could be loaded up to the yield strength of the high-strength steel without any damage such as cracking or spalling. In the design the concrete was not taken into consideration. It was however used to ensure corrosion protection of the reinforcement, protect the reinforcement bars and stirrups against buckling and increase the fire resistance periods to the required fire resistance ratings.

A similar column concept was used in the year 2000 for the *Herriot's* tower block in Frankfurt. In this case, *Falkner* et al. used C100/115 high-strength concrete with reinforcement percentages of approx. 10% in combination with special steel St 750/1200. For the required approval on an individual basis tests were carried out to determine the service and ultimate behaviour as well as the fire resistance period [2].

The columns' increased concrete ultimate compressive strain of 2.75‰, as compared to DIN 1045, still remained below the steel's yield strength of 750/205 = 3.66‰. The time-dependent behaviour of the concrete was disregarded. Instead, the concrete was made more ductile by adding a fibre cocktail of polypropylene and steel fibres. Thus, even under quick loading in the ultimate compressive strain zone of the concrete, spalling or scaling could not occur. In the design, the slightly downward sloping stress-strain/compressive strain curve was taken into consideration above the compression yield point of the concrete. The tests revealed that due the high reinforcement percentage the load bearing behaviour of the columns was still ductile even with an already downward sloping concrete stress-strain/compressive strain curve (see also section 4.3, Fig. 9).

5.2 Tests on columns with S 670 and prism tests

To test the new compression members, eight 1m-long column sections were examined taking previous tests into consideration. Thereby, the load holding periods resp. load histories of the test columns were modified. In addition, various prism tests were carried out to obtain additional criteria on the load bearing behaviour of the concrete for test analysis. Below, the most significant tests are described in more detail.

The columns were reinforced using a centred S 670 reinforcing bar with a diameter of 75 mm and four external BSt 500 reinforcing bars with diameters of 10 mm. 14 stirrups with diameters of 6 m were installed (Fig. 12). The concrete strength was C50/60. To ensure fire protection, 1.5 kg of polypropylene fibres per m^3 of concrete had been added. The concrete also contained quartziferous aggregates with grain sizes of 0/16 mm and cement CEM II/A – S 42.5 R.

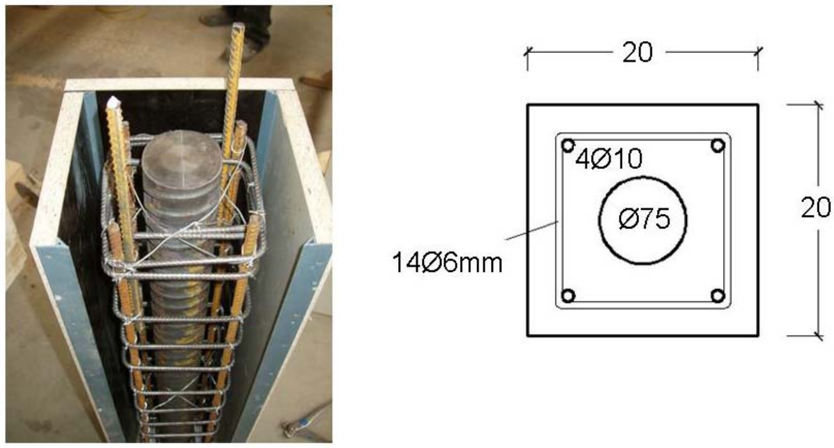


Fig. 12: Reinforcement and cross section (20 x 20 cm) of a test column

During these tests the load was held constant at various high load steps, displacement and force controlled, to simulate the time-dependent behaviour of the concrete, i.e. the creep and shrinkage redistributions. They were performed in order to clarify the following central questions:

- Is it possible, using appropriate creep resp. relaxation and shrinkage redistributions, to fully exploit the yield strength of the steel?
- How does the composite system of high-strength steel, high reinforcement ratio of μ = 13.4% and normal strength concrete behave after the concrete has exceeded its

ultimate compressive strain and is therefore already in the post-failure phase ?

- How does the system behave under quick loading, if time-dependent creep and shrinkage redistributions cannot occur?

The most important column tests performed are summarized in Table 3 below. The tests are documented in detail in [8].

Table 3: The most important key data of the tests

Test specimen	Cross section Reinforcement	Relaxation load/load steps Holding times test procedure	Ultimate compressive strain of concrete /cross section Ultimate load	Test type
S5	See Fig. 12	≈ 3300 kN = 70% F_u 480 hours → displacement controlled fracture test	Analytically determined: 3.3 ‰ F_a = 5160 kN 3.39 ‰ (test) **F_u = 5,492 kN**	Relaxation test + displacement controlled compression test – centric –
S6	See Fig.12 spliced using contact coupler	≈ 3300 kN = 70% F_u 24 hours displacement controlled fracture test	Analytically determined: 3.3‰ F_a = 5160 kN 3.92‰ (test) **F_u = 5,219 kN**	Relaxation test + displacement controlled compression test – centric–
S7	See Fig. 12	Quickly loaded until failure, displacement controlled 0.002 mm/s displacement controlled fracture test	Analytically determined: 3.3‰ F_a = 5160 kN 2.8‰ **F_u = 5,120 kN** (test)	displacement controlled compression test
S8	See Fig. 12	Loaded in steps in the course of one day 639 kN – 2h 1278 kN – 2h 1917 kN – 2h 2555 kN – 2h 3194 kN – 2h 3833 kN – 2h → fracture test	Analytically determined: 3.3‰ F_a = 5160 kN 3.2‰ **F_u = 5,320 kN** (test)	Relaxation and. displacement controlled compression test

For test specimen S 5, a relatively high permanent load and a 20-day relaxation period were chosen to simulate creep redistributions comparable to those occurring in practical applications that are under lower loads but have longer holding periods. The test showed that the concrete's ultimate compressive strains considerably exceeded the required value of 670 (205 × γ_s) = 2.84‰ (Fig. 13). Even at 1‰ above this ultimate compressive strain, almost no concrete scaling occurred. Only under very high compressive strains of approximately 4.5 to 7‰ did concrete scaling set in (Fig. 14). The compression coupler of test column S6 only showed an additional deformation of 0.4 mm upon attainment of the yield strength.

All tests revealed that by exploiting the time-dependent behaviour of concrete, the yield strength of S670 high-strength steel can be fully exploited and that the two materials steel and concrete are very compatible even when using the largest steel bar diameter of 75 mm. Under high loading rates the concrete had an ultimate compressive strain of 2.8‰. In this case as well, both bearing components performed well together in the post-failure phase of the concrete.

In all performed creep and. relaxation tests the yield strength of the high-strength steel could be fully exploited; the load bearing behaviour proved to be extremely ductile.

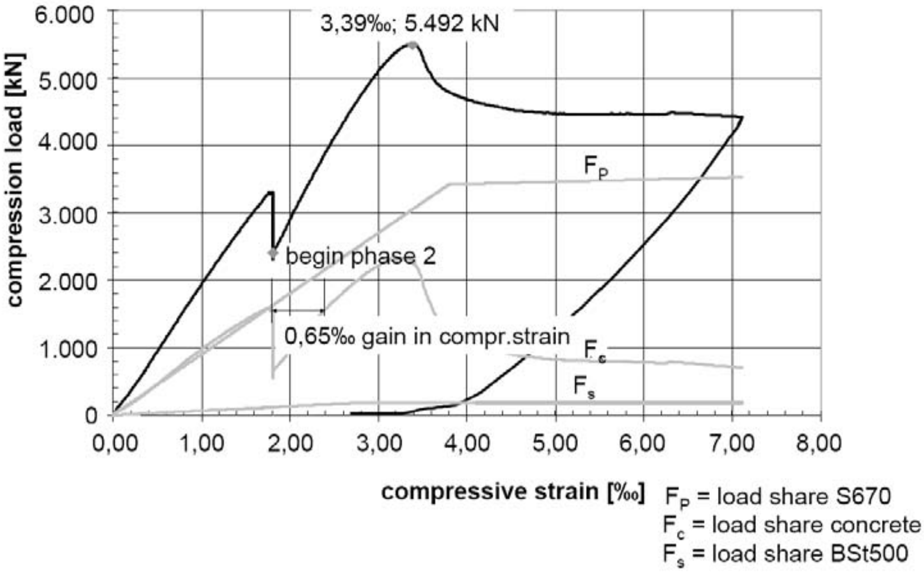


Fig. 13: Ultimate load and post-failure behaviour of column S5 (example)

Fig. 14: Conditions of column S5 under a compressive strain of 4.5‰, first signs of scaling bottom left (encircled)

 DOI: 10.1002/best.200800614

Short-term, creep and relaxation comparison tests were carried out on prisms of the same concrete as used for the columns [10]. Thereby, upon reloading after a holding period, the concrete behaved somewhat stiffer than in the other tests. On the one hand, this can be attributed to continuing hydration processes in the relatively green concrete specimen. On the other hand, under permanent loading, compaction of the concrete structure could have set in to some extent thereby contributing to the slight increase of the E modulus. Therefore, to be on the safe side, in the design of the reloading share ε_2 (section 4.3) is only assumed to be 75% (section 6.2).

5.3 Fire tests

Due to the different parameters as compared to columns according to DIN 1045-1, e.g. reinforcement diameter and increased reinforcement percentage, the fire behaviour of columns with high-strength S 670 had to be tested both experimentally and analytically [11]. For their first practical application in the Frankfurt Opernturm special investigations had to be carried out, particularly in regard to the large slendernesses and column lengths as well as the very high load utilization.

Since the performed tests are directly related to the application in the Frankfurt Opernturm, Part II of this report due be published in a few months will provide more detailed information. At this point it shall only be noted that S 670 behaves like Tempcore reinforcement steel Due to the large reinforcement diameters the test columns with 1.5 kg of polypropylene fibres per m^3 of concrete were expected to have an increased risk of spalling. This, however was not the case. Neither could a reduced fire resistance period, that might have resulted from the increased reinforcement percentages and thermal conductivity of the steel, be observed.

6. Serviceability and ultimate limit state design of the compression members

6.1 Design according to EC 2 and DIN 1045-1

In the most simple case of a pendulum column of length ℓ without intended moment stresses, a moment due to unintended eccentricity e_a must, according to DIN 1045-1, sections 8.6.3 and 8.6.4, under certain conditions be taken into consideration in the design in order to allow for imperfections.

$$e_a = \alpha_{a1} \cdot \ell/2 \qquad (9)$$

$$\alpha_{a1} = 1/(100 \cdot \sqrt{\ell}) \leq 1/200 \qquad (10)$$

 DOI: 10.1002/best.200800614

With small slendernesses or $\lambda = \ell/i$ (i = radius of inertia) up to 25, bending and additional loading resulting from the second-order theory play only a minor role in the design of pendulum columns. The cross section is fully precompressed, the relevant parameter for column design is the centric compressive load.

For columns that are to a large extent centrically precompressed, the compressive strain ε_{c2} must be limited. This compressive strain must not be exceeded in point C (DIN 1045-1, Fig. 30) of the cross section.

Under a compressive strain ε_{c2} of 2‰ when using concrete up to strength C50/60 ?), increasing to 2.2‰ up to strength class C100/115, it is not easily possible, up to concrete strength class C80/95, to fully exploit BSt 500 rebar reinforcement up to the yield strength $\varepsilon_{yd} = 500/(205 \times 1.15) = 2.17‰$.

With fully precompressed cross sections and small eccentricities of $e_d/h = 0.1$, $\varepsilon_{c2} = 2.2‰$ is permissible for normal concrete, enabling full exploitation of the yield strength of conventional BSt 500 reinforcement steel. The basis for this slightly larger compressive strain value ε_{c2} are the compressive creep and shrinkage strains that are always present.

For larger slendernesses or λ_{crit} between 25 and 75, the influence of unintended eccentricity becomes increasingly important depending on the buckling length resulting from the column bearing conditions. The interaction between normal load and bending moment is, therefore, just as relevant for column design as in the case of intended additional moment loading.

The stress-strain/compressive strain curves and deformations according to the second-order theory defined in DIN 1045-1 must be taken into consideration in the design. Usually, long-term effects such as creep and shrinkage are thereby disregarded. In the forthcoming new version of DIN 1045-1 creep eccentricity k_φ must be taken into account if $\lambda > 50$. Based on EC 2, all strain values of the concrete must be multiplied by1 + φeff, where φeff = φ × $M_{1perm}/M1_{Ed}$.

M_{1perm} = Bending moment according to first-order theory when subjected to quasi continuous combined actions incl. imperfection (serviceability).

M_{1Ed} = Bending moment according to first-order theory when subjected to the design combined actions incl. imperfection (ultimate bearing capacity).

 DOI: 10.1002/best.200800614

When using S 670 high-strength steel, to ensure full economic exploitation of the steel, higher concrete compressive strains are required due to the increased yield strength (Fig. 7), in particular with small slendernesses under largely centric compression. Below, as already indicated, the design concept of EC2 and DIN 1045-1 has therefore been adapted accordingly.

Thereby, if the time-dependent influence is favourable, rather lower creep and shrinkage coefficients are to be expected. Non-linear creep is discussed in issue 525 of DAfStb.

6.2 Modified design for the use of S 670 including C + S and/or taking the post-failure behaviour with increased ductility of the concrete into consideration

6.2.1 General

On the basis of the information discussed above, the conditions required for full exploitation of the yield strength of S 670 may be summarized as follows:

- Determination of redistributions $\Delta\varepsilon_S$ of the concrete in the serviceability limit state during the construction phase (sections 4.3, 4.5 and 6.2.2).
- Redistribution $\Delta\varepsilon_S$ is included as compressive prestressing/precompression (analogous to tensile prestressing in prestressed concrete construction, see [12]) in fracture design calculation.
- Alternatively, the increased compressive strain $\Delta\varepsilon_S$ may be added to the concrete compression (Fig. 18). Otherwise, conventional reinforced steel design applies.
- Should it not be possible to fully exploit the yield strength of S 670 following calculation of redistribution $\Delta\varepsilon_S$, the supporting effect of the steel on the concrete's post-failure behaviour may, with high reinforcement percentages, additionally be taken into consideration (section 4.3). This applies all the more, if the ductility of the concrete is increased by fibres.
- The calculation sequence includes time steps and is conveniently carried out using a calculation programme (section 6.2.3).

 DOI: 10.1002/best.200800614

6.2.2 Creep and shrinkage utilization in the design, including a numerical example

For a high-rise building, several floors are combined to determine the respective time and load steps. Below, equations and a simplified numerical example for only one load and time step of a centrically loaded column are defined (Fig. 15).

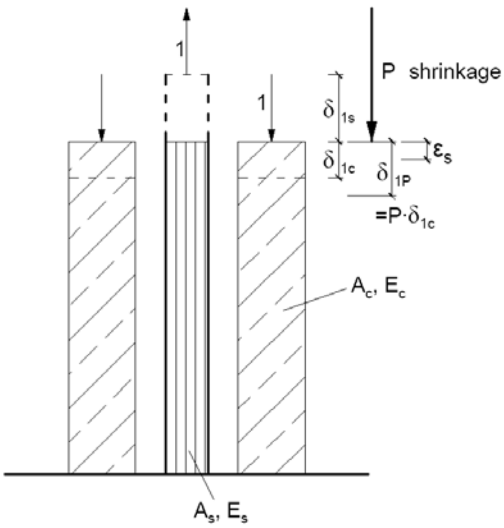


Fig. 15: Compression member statically indeterminate to the first degree

Parameters: $\delta_{1s} = 1/(A_sE_s)$ (11); $\delta_{1c} = 1/(A_cE_c)$ (12); $n = E_s/E_c$ (1); $\mu = A_s/A_c$ (13);
$\alpha = \delta_{1c}/(\delta_{1c}+\delta_{1s}) = n\cdot\mu/(1+n\mu) = n\cdot A_s/A_i$ (14); $A_i = A_c+n\cdot A_s$ (2)

Serviceability limit state prior to c+s:
$N_s = \alpha\cdot P$ (15); $\sigma_{Ps} = N_s/A_s = n\cdot\sigma_{Pc}$ (16)
$N_c = (1-\alpha)P$ (17); $\sigma_{pc} = N_c/A_c = P\cdot/A_i$ (18)
$\sigma_{\varepsilon ss} = (1-\alpha)\cdot\varepsilon_s\cdot E_e$ (19)
$\sigma_{\varepsilon sc} = -\sigma_{\varepsilon s}\cdot\mu$ (20)
Following k+s:
Redistribution according to *Trost* [5]:

Numerical example: P = 25138 kN
C 50/60, A_s = 319cm², A_c = 4681cm²
n = 205000/36800 = 5,57
μ = 319/4681 = 0,0681
A_i = 6458cm², α = 0,275
redφ = 1, redε_s = $15\cdot10^{-5}$
Redistribution with deformation modulus:

At time t = 0:

$$\boldsymbol{\sigma_{c0} = 25138\text{kN}/6458\text{cm}^2 = 38{,}93\ \text{N/mm}^2}$$

$$\boldsymbol{\sigma_{s0} = \sigma_{c0}\cdot n = 216{,}84\ \text{N/mm}^2}$$

c_d = 1+(1-α)·φ/(1+α·ρ·φ) = 1,594 (21)

ρ = 0,8 relaxation coefficient.

c_s = 1/(1+α·ρ·φ) = 0,82 (22)

$E_{c\varphi}$ = E_c/(1+φ) = 18400 N/mm²

n_φ = (1+φ)·n = 11,14

$A_{i\varphi}$ = 4681+11,14·319 = 8235cm²

α_φ = 11,14·319/8235 = 0,43

$\sigma_{st} = \sigma_{s0}\cdot c_d+\varepsilon_s\cdot E_s\cdot c_s(1-\alpha)$ (23)

= 345,6+18,3 = 363,9 N/mm²

$\sigma_{ct}=(P-\sigma_{st}\cdot A_s)/A_c$= 30,2-1,3 = 28,9 N/mm² (24)

σ_{ct} = 25138/8235 - 17,5·0,0681 =

= 30,53-1,19 = 29,34 N/mm²

σ_{st} = 11,14·30,53+(1-0,43)·15·10^{-5}·205000 =

= 340,1+17,5 = 357,6 N/mm²

The accordance between the two calculation methods is very satisfactory:

Redistribution steel

$\varepsilon_1 = (\sigma_{st}-\sigma_{s0})/E_s = 0{,}000717$ (25)

Restressability/breathing concrete

$\varepsilon_2 = 0{,}75 \cdot (\sigma_{c0}-\sigma_{ct})/E_c = 0{,}000204$ (26)

Compressive prestressing/redistribution

$\Delta\varepsilon_s = \varepsilon_1+ \varepsilon_2 = 0{,}000717+0{,}000204 = 0{,}000921 \rightarrow 0{,}921‰$

Based on the compressive strain Δεs referred to above, the safety against fracture can be verified using conventional methods. Under a centric concrete compression of 2‰ according to DIN 1045-1, the yield strength of the steel is fully exploited, attaining an ultimate compressive strain of 2.921‰ corresponding to 205 · 2.921 = 599 N/mm^2 > 670/1.15 = 583 N/mm^2.

Fig. 16 exemplarily shows the redistributions and thus the exploitation of the yield strength of S 670 for various reinforcement percentages. It confirms that the yield strength of full high-strength reinforcement steel can be almost fully exploited using c + s with no other measures being necessary. Unlike Fig. 10, the yield strength of the cross sections is fully exploited.

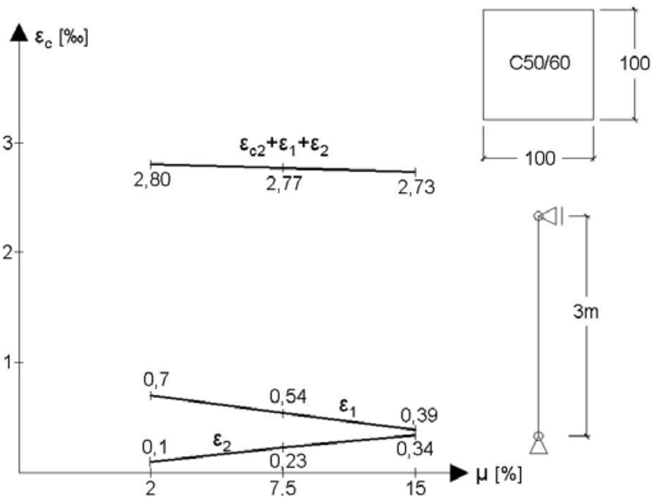


Fig. 16: Influence of the reinforcement percentage on redistribution $\Delta_{\varepsilon S}$, example Opernturm

In the serviceability limit state, columns of high-strength reinforcement steel with high reinforcement percentages may cause high concrete stresses prior to creep and shrinkage. In such cases the serviceability stresses must be verified and the concrete stress limited to $0.45 \times f_{ck}$.

For buckling verification DIN 1045-1 is applicable taking the compressive strain due to compressive prestressing $\Delta_{\varepsilon S}$ into consideration.

6.2.3 Calculation programme, simplification

For successive loading during the construction phase various load steps must be defined (Fig. 17). For each of these, the compressive strain increase until construction completion must be calculated separately. The superposition principle applies. However, to simplify the calculation process calculation should be carried out using a computer programme.

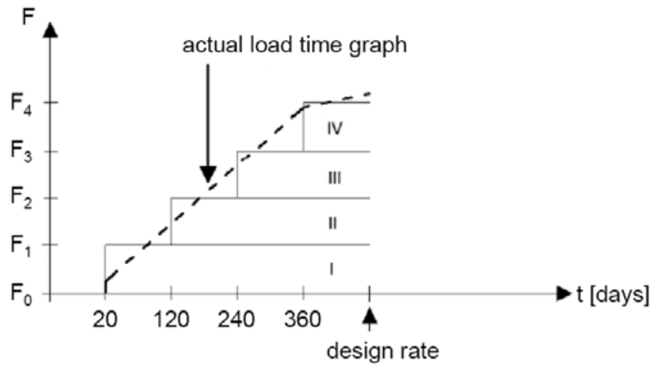


Fig. 17: Exemplary load time graph and definition of load steps for calculation

 DOI: 10.1002/best.200800614

The use of suitable templates facilitates data input.
In the future, based on this programme, a matrix will be prepared showing the redistributions dependent on the following parameters:

- Reinforcement percentage
- Permanent load level
- Duration of redistribution
- Concrete grade

This information makes additional redistribution calculations in certain "window" areas unnecessary. In addition, data will be provided in order to determine as of which larger slendernesses the design can be carried out using conventional column design programmes if the reinforcement is reduced in relation to the yield strengths.

Design tables will be prepared, based on which the reinforcement can be calculated using simple manual calculation for largely centrically loaded columns and using the usual column design programmes for more eccentrically loaded columns.

As simplification (see section 4.6) the programme only takes redistribution induced by centric normal loads into consideration and not redistribution $\Delta\varepsilon_S$ which varies with the cross section height (Fig. 11). $\Delta\varepsilon_S$ resulting from compressive prestressing is also not taken into consideration, but is added to the concrete compression value, see Fig. 18, which has the same effect.

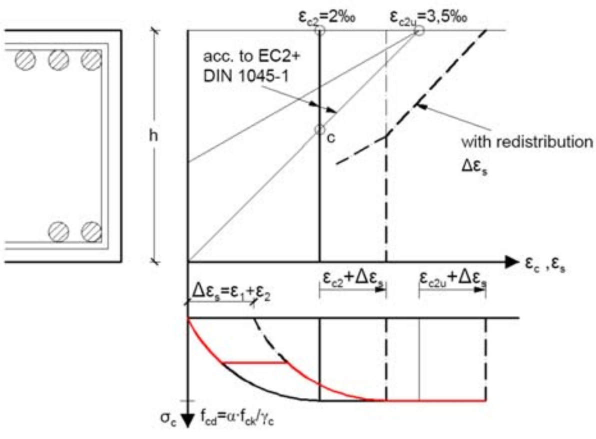


Fig. 18: Modified concrete stress-strain/compressive strain curve taking the compressive strain increase $\Delta\varepsilon_S$ into consideration for full exploitation of the yield strength of S 670 high-strength steel

DOI: 10.1002/best.200800614

6.2.4 Increasing the ductility of the concrete

As has already been noted, the ductility of the concrete can be increased, even without taking redistribution into consideration, by adding fibres. This also improves the effect of the concrete's post-failure behaviour (Fig. 9). The respective tests will be carried out in the future. In addition, tests should be carried out to determine to what extent the ultimate compressive strain of the concrete increases also under quick loading, without the influence of creep and shrinkage and dependent on various high reinforcement percentages. This actively supporting effect could delay premature decrease of the concrete's load bearing capacity shown in Fig. 9.

6.3 Hot-rolled steel design

Like cold-formed steel design, hot-rolled design also plays an important role. As indicated above, S 670 behaves just like normal reinforcement steel. Therefore, the design principles for normal reinforced concrete construction apply, see [13]. The fire behaviour of columns with S 670 was investigated by Prof. *Hosser a*nd Dr. *Richter* [11] both experimentally and analytically in connection with the required approval on an individual basis for the Frankfurt Opernturm. The columns showed very favourable fire behaviour. More detailed information can be found in Part II of this report dealing with the Frankfurt Opernturm.

With high fire resistance classes, however, small quantities of PP-fibres should always be added to avoid concrete spalling.

7 Structural design

7.1 Introduction

Basically EC 2 applies resp. the national adaptations. The following Eurocode-based restrictions and modifications must be observed:

- Compression or tensile overlap splices are not to be used, bond stresses remain therefore low and their effects minimal
- Moderate tension with large diameters means reduced yield strength exploitation as compared to BSt 500 due to excessive deformation and excessive crack widths.
- Concrete cover due to risk of longitudinal cracking resulting from varying transverse strains between steel and concrete and due to fire protection $\geq 0{,}8 \cdot Ø$.

- Max. reinforcement percentages of μ ≤ 20%, bar diameters up to 75 mm, steel grade St 670/800
- Reduced stirrup diameter < 1/4 of longitudinal reinforcement for non-buckling S 670 diameters (section 7.5).

7.2 Compression and tensile splices

Although designed for ultimate bearing capacity, it is recommended that splices be staggered by approx. 15 · Ø when using C45/55. For higher concrete grades staggering may be reduced by √ $(45/f_{ck})$(27), resulting in 50% bar splicing. The contact splice may not be subjected to tensile loading. Due to the specified splice staggering one bar remains unspliced, its yield strength can therefore be fully exploited.

7.3 End anchorages

With sufficient concrete dimensions, tensile and compressive forces can be transferred with permissible slip (< 0.2 mm) through the anchor piece or anchor plate and free length. For high reinforcement concentrations, installation of a compression butt splice (similar to contact splice) on a thick steel plate to be designed according to steel construction and reinforced concrete principles is recommended.

7.4 Bond stresses resulting from load introduction and creep redistributions

Usually the stirrup spacings below and above the slab are reduced to 60% of the free length according to DIN 1045-1. This ensures accommodation of the concrete/reinforcement load-distribution disturbances and the load-transfer disturbances from the slab.

The in this case very important redistributions cause bond stresses that may not be disregarded. These are rather low from one column to the next, since only the differential redistribution must be transferred. At the ends of a column row these bond stresses may also be larger resulting in steel/concrete displacements and bursting forces. They should therefore be verified separately.

Due to the displacement induced by the load transfer, the stresses in the steel decrease while the concrete in this zone is overstressed. However, as summarized in [14], the concentrated stirrup reinforcement to be installed in this zone increases the concrete

 DOI: 10.1002/best.200800614

strength in the longitudinal direction through multiaxial loading to a larger extent than the load increases due to load reduction of the flexible longitudinal reinforcement. This effect occurs to a lesser degree in conventional reinforced concrete construction as well.
Therefore, a bursting/bond reinforcement must be installed in the introduction zone in the form of stirrups that can absorb the bond stresses caused by the slab loads and redistributions.

7.5 Stirrup diameters to prevent buckling large diameters

The standards specify stirrup diameters of ≥ $Ø_L/4$, as deduced from the buckling values for longitudinal reinforcement diameters $Ø_L$ which are usually smaller. The yield strength-buckling length according to Euler case I for S 670 $Ø_L$ = 75 mm is 1.04 m and for BSt 500 $Ø_L$ 40 mm= 0.64 m. It suffices therefore to assume stirrup diameters of $Ø_L/4$ for S 670 up to $Ø_L$ 43 mm, linearly decreasing to $Ø_L/6$ for $Ø_L$ 75 mm. This even ensures a larger effective stirrup quantity along the buckling length than according to DIN 1045-1.

7.6 Penetration high-strength columns/low-strength slab concrete

In this case as well, the multiaxial stress state in the slab has a favourable effect so that according to [15] the strength class of the slab concrete may amount to up to 1/3 of the strength class of the column concrete without requiring special verification. Alternatively, the load transfer via continuous reinforcement may be favourably assumed in design calculations, Fig. 19.

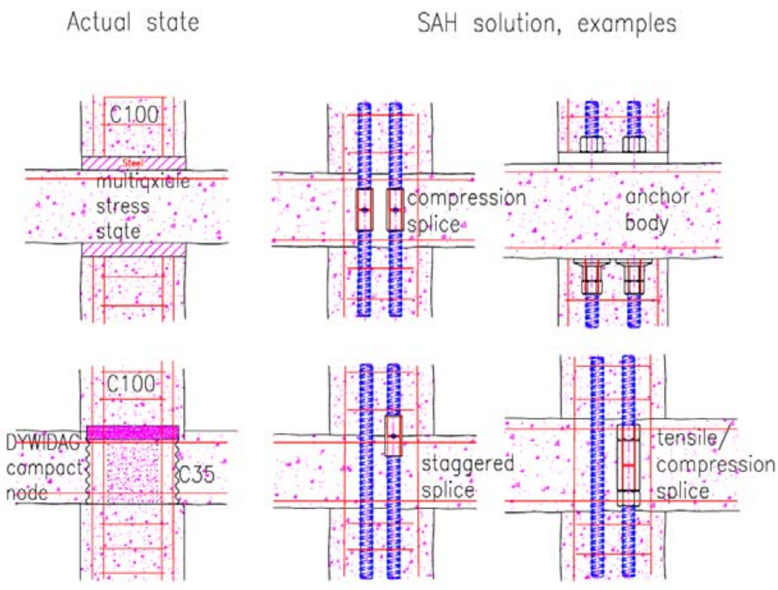


Fig. 19: Penetration high-strength column – low strength slab concrete

 DOI: 10.1002/best.200800614

7.7 Further structural details, e.g. punching

The usual structural details of state-of-the-art reinforced concrete technology apply. Since columns reinforced with S 670 have rather smaller cross sections than usual, special attention must be paid to punching.

8 Installation, references

The threadbars and components are subject to internal and external control. The interfaces of the contact splices must be plane-parallel with a maximum deviation of 0.5°. Transport, storage and quality-assured installation as well as pouring and post-treatment must be specified in manuals and be carried out by qualified personnel. Part II of this report provides more detailed information. To date, two tower blocks in the USA, the Opernturm in Frankfurt (see also Part II) as well as some smaller projects in Moscow have been constructed using this new reinforcement technique. Various projects are in the planning stage.

9 Summary, future prospects

The new reinforcement system for compression members with SAS 670/800 presented in this report extends the scope of EC2 and DIN 1045-1 in regard to improved quality and cost efficiency. This report has shown that columns constructed of high-strength steel have up to ~ 50% smaller cross sections. Thereby, the existing regulations for concrete and prestressed concrete construction (EC2, DIN 1045-1, etc.) are taken as a basis. Time-dependent redistributions in the construction state due to creep and shrinkage of the concrete, the post-failure behaviour and/or ductile behaviour of fibre-reinforced concrete are taken into consideration. By now several practical applications have been carried out. In addition, approvals have been granted in various countries. A European technical approval taking the effects of fire into consideration is in preparation.

 DOI: 10.1002/best.200800614

References

[1] Leonhardt, F., Teichen, K.-T.: Druckstoße von Bewehrungsstäben und Stahlbetonstützen mit hochfestem Stahl St90; Heft 222 des D.A.f.Stb, Berlin 1972

[2] Eierle, B., Gabel, N., Stenzel, G.: Fertigteilstützen aus Hochleistungsbeton B125 für das Hochhaus HERRIOT's in Frankfurt am Main; Betonwerk + Fertigkeit-Technik, Heft 3/2003, 69 Jahrgang

[3] Martin, H.: Zusammenhang zwischen Oberflächenbeschaffenheit, Verbund und Sprengwirkung von Bewehrungsstählen, D.A.f.Stb. Heft 228, Berlin 1973

[4] Rüsch, H.: Stahlbeton Spannbeton, Band 1, Werner Verlag 1972

[5] Rüsch, H., Jungwirth, D., Hilsdorf, H.: Creep and Shrinkage, Springer Verlag 1983, New York/ Heidelberg

[6] Hegger, J.: Pull Out Versuche N° 119/2004 vom 22.12.2004, TU Aachen

[7] Rußwurm, D.: Beton und Stähle für den Stahlbetonbau, Bauverlag 1993

[8] Falkner, H., Gerritzen, D., Grunert, J.: Stützenversuche mit SAS 670 ø 75mm, Untersuchungsbericht N° 06S1315-a, 2007, TU Braunschweig, sowie vergleichende Prismenversuche

[9] Hosser, D.: Brandversuche TU Braunschweig mit SAS 670, N° G 07011, 2007

[10] Bemessung von Stahlbeton- und Spannbetonbauteilen: Zilch, Rogge, Betonkalender 2002, T1 oder Grasser, Kupfer, Pratsch, Feix Betonkalender 1996, T1

[11] Fingerloos, F.: Heißbemessung von Stahlbetonstützen nach DIN 4102, Fortbildungsveranstaltung Deutsche Beton- und Bautechnikverein, Heft 14, 2007

[12] Jungwirth, F.: Untersuchung zur Krafteinleitung über Zwischenverankerungen bei externen Spanngliedern, Dissertation TU Leipzig 2003

[13] Weiske, R.: Durchleitung hoher Stützlasten bei Stahlbetonflachdecken; Dissertation TU Braunschweig 2004, ISBN 3-89288-161-8

[14] Falkner, H.: Trends und Entwicklungen im Bauwesen - HH Stützen -, Braunschweiger Bauseminar 2000, ISBN 3-89288-131-6

[15] Wechtitsch, M.: GEWI-Stahl SAS 670 als Betonstahl; Technische Universität Graz, Lehrstuhl für Massivbau, 2006

DOI: 10.1002/best.200800614

SAH THREAD BAR SYSTEMS - INNOVATIONS FOR CONSTRUCTION ENGINEERING

The innovative SAS thread bar systems of Stahlwerk Annahütte are wellknown world wide. For building and bridge constructions, civil engineering, tunneling or mining, we produce and deliver for all applications the suitable thread bar systems in diameter 12 up to 75 mm of steel grade SAS 500, SAS 670 and SAS 1050. *With our thread bar SAS 670 important design and economic advantages for high-strength compression elements are achieved.*

Design advantages:

- Less weight [kg/m] compared to conventional reinforcing steel under the same load
- Easy concreting with the increasing of the maximal reinforcement percentage
- Minimization of the cross section of columns due to the higher steel grade SAS 670

The consequences of the design advantages are

Economic advantages:

- Faster installation of the thread bars saves wages and time
- Minimization of construction time for precast segements and site-mixed concrete constructions
- Reduction of construction costs as a result of smaller cross sections of the columns and due to an enlargement of the effective area per floor, the rental profitability can be increased

SAS SYSTEMS

Stahlwerk Annahütte Max Aicher GmbH & Co. KG, D-83404 Hammerau / Germany
Tel. +49 (0)86 54 / 4 87-0 • Fax +49 (0)86 54 / 4 87-964
stahlwerk@annahuette.com • www.annahuette.com

2008 SI

SAH Stahlwerk Annahütte