

# Results of Deterministic Investigations of Reinforced Concrete Structures for the Safety of Non-linear Calculations

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## SUMMARY

*At the beginning this paper describes the design suggestion of the new DIN 1045 for physical and geometrical non-linear structural design. Some select examples of designed beams and frames will be analyzed to show, that this suggestion is on the right track.*

## 1 INTRODUCTION

In the two contributions [1] and [2], which were published up to now, the suggestion for physical and geometrical non-linear structural design of reinforced concrete structures for DIN 1045 was developed, because the verification concept in EC2 was not applicable to this newly allowed design method [3], [4].

In the author's opinion other approaches, like [5] with a partial safety coefficient depending on the building component, are too complicated or, like the favoured probabilistic design in [6], not yet enough determined with regard to the calculating method. Further suggested continuous consideration with mean values or design values results in too less deformation caused by the high stiffness or too much deformation caused by underestimated stiffness. Therefore the following described design algorithm orientates by characteristic material properties, as prescribed in [7] and [8].

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## 2 DESIGN CONCEPT

Corresponding to the design philosophy for reinforced and prestressed concrete structures according to [7], [8] the following design equation (1) must be met.

$$S_d \leq R_d \tag{1}$$

In case of non-linear FE–design  $S_d$  is the design value of actions and  $R_d$  the corresponding design value of structural resistance.

The design value of actions is not changed according to the definition in [8], to keep the level of safety on the side of action which is considered as sufficient from experience. Therefore is valid :

$$S_d = \sum \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P_k + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{k,i} \tag{2}$$

The partial safety and combination coefficients for the single action are defined in [8]. The design value of the structural resistance is defined as follows:

$$R_d = \frac{R_{sys}}{\gamma_R} \tag{3}$$

In this case  $R_{sys}$  is the bearing capacity of the system with material properties acc. eq. (4) .

The partial safety coefficient  $\gamma_R$  on the side of resistance of the system is mainly influenced by the material properties, which are input values in the FE–design , and the kind of actions (mainly bending or axial force) .

Considering the partial safety coefficients for steel and concrete in [9] following safety coefficients are necessary to achieve the same reliability on the side of resistance.

$$\begin{aligned} f_{yR} = 1,1 \cdot f_{yk} ; \quad \gamma_S = 1,15 \Rightarrow \quad \gamma = 1,1 \cdot 1,15 = 1,265 \\ f_{cR} = 0,85 \cdot f_{ck,Zyl} ; \gamma_S = 1,5 \Rightarrow \quad \gamma = 0,85 \cdot 1,5 = 1,275 \end{aligned} \tag{4}$$

Up to now investigations [10] show that with material properties acc. (6) a partial safety coefficient of 1,3 on the side of resistance guarantees on average a reliability, which is considered as sufficient from experience for mainly under bending stress structures.

This indicates a wise modification of material properties for design investigation.

Caused by previous investigations it is suggested to introduce a partial safety coefficient for structural resistance with a value of

$$\gamma_R = 1,3 . \tag{5}$$

The verification of bearing capacity for the whole system results in following equation :

$$\left( \sum \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P_k + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \right) \leq \frac{R_{sys}}{\gamma_R} \tag{6}$$

$$R_{sys} = f(System, A_c, f_{cR}, A_s, f_{yR}, A_p, f_{pR}, P)$$

A verification for the important cross-sections is not planned. By this way the inconsistency between structural analysis and design of cross-sections is removed [9]. Now a general application of nearly real material properties for the design of reinforced and prestressed concrete structures is possible.

### 3 CHECK OF THE SUGGESTION BY DETERMINATION OF LIMIT LOAD WITH NON-LINEAR FE-CALCULATIONS

A great number of calculations was realized to confirm the suggestion of the defined calculating values acc. (4) as input values for non-linear FE-calculations. Two examples will be described to limit the extent of this report. Further calculation results can be taken from [11] . The investigation uses the FE-program FEMAS, which considers the geometrical and physical non-linearities.

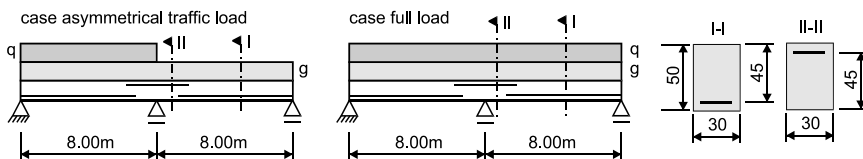


Fig. 1: Two-span beam with actions

The first example is the two-span beam which is shown in fig.1. The design load was chosen so that with linear design method calculated internal forces and moments result in a geometrical percentage of reinforcement of 0,5% (fig.2) or 1,2% (fig.3) for the cross-section above the support. The corresponding reinforcement for the span was calculated with the static equilibrium conditions of the corresponding span moments. The load is increased in the non-linear FE-calculation. The load factor is defined as the relation between current load and design load. A negative redistribution means a redistribution from span to support. A positive one means the inverted course.

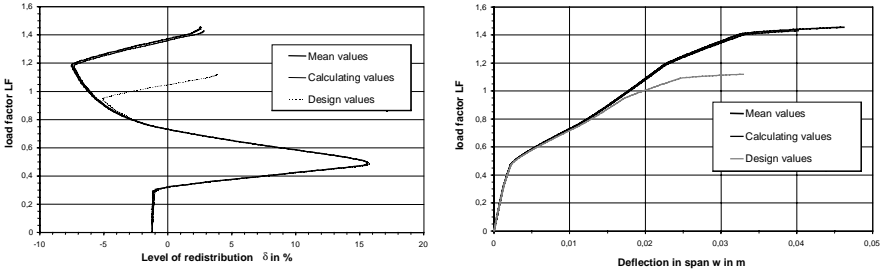


Fig.2: Level of redistribution and deflection with percentage of reinforcement of 0,5% in cross-section above support

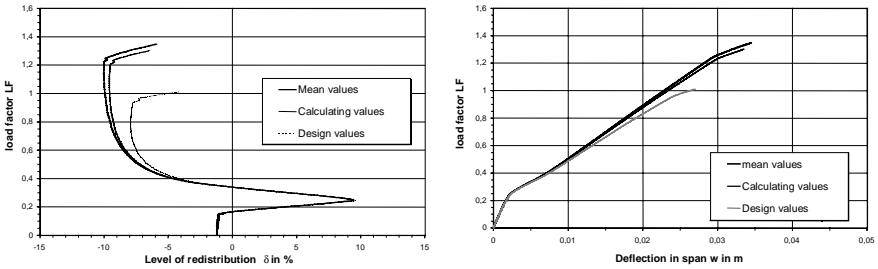


Fig.3: Level of redistribution and deflection with percentage of reinforcement of 1,2% in cross-section above support

Figures 2 and 3 show clearly that the difference concerning the behaviour of deformation and redistribution, which is caused by the application of material properties acc. to the mean values and eq. (4), is negligibly small. By application of design values acc. EC2 as input values however the difference is greater. Furthermore the results show impressively the essential advantage of non-linear methods, that is the recording of the real behaviour of the structure for an arbitrary level of load.

Like in every cross-section design in this example it is presumed for the calculation of the percentage of reinforcement for the cross-sections of span and support that both cross-sections achieve their calculated bearing capacity with the same load step. This is not realistic. The different percentages of reinforcement in span and support cross-sections result in different stiffnesses and this causes a real distribution of internal forces and moments over the whole spectrum of load which is unlike the distribution of the linear method. At first a small redistribution

happens from span to support. After formation of cracks in the region of the support happens a redistribution from support to span. As soon as cracks occur in the span the way of redistribution turn around until the reinforcement in the cross-section of support achieve its yield strength. Now redistribution happens from support to span again. According to the linear method the relation between moment of support and span is independent of load. In reality this relation changes permanently.

Figures 2 and 3 show also the influence of the percentage of reinforcement on the behaviour of the structure. While with a reinforcement level of 0,5% the level of redistribution changes permanently, what points to a permanent change of the relation of stiffness between span and support zone, this level nearly remains constant in that example with a reinforcement level of 1,2% between load factors 0,8 and 1,2 . The reason for this behaviour is the forming of a complete crack pattern from a load factor of 0,8 . The present level of redistribution changes again just with achieving the yielding of steel in the cross-section of support. The maximum load factor of the first example is determined by achieving of bearing capacity of span and support cross-section (forming of an hinge chain), what is shown by the great increasing of deflection with small increasing of the load. In the second example however the achieved bearing load is limited by the deformation capacity of the cross-section of the support. That is why the achieved load factor is lower than the one in the first example. In calculations with design values acc. EC2 the reinforcement in the cross-section of support already achieve the yield limit with a load factor of less than 1,0 . If a redistribution would be impossible after yielding of steel the system would fail immediately. Then we get a calculated deficit of safety which does not correspond with the reality.

As the second example the system in fig.4 is investigated. It is a sway structure. To show the influence of 2<sup>nd</sup> order theory the force P is chosen as different values. The design loads for the investigated cases are shown in table 1 . The eccentricity caused by imperfection is realized by the skew sway frame. The design of the important cross-sections was performed with internal forces and moments of the linear calculation and application of the so-called “model column method”.

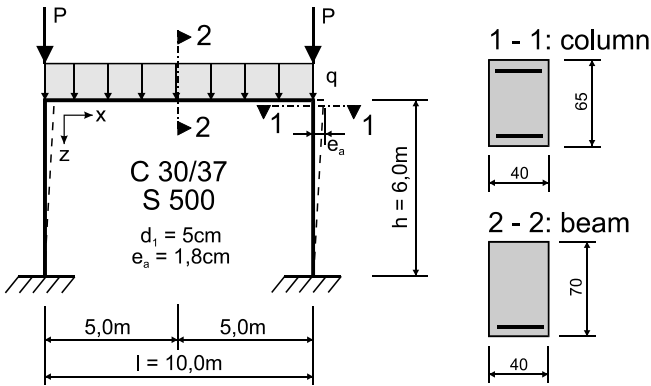


Fig.4: Sway frame with actions

Case	P in kN	q in kN/m
1	1000	100
2	2500	100
3	4000	100
4	8000	100

Table 1: Loads of the investigated cases for the sway frame in fig.4

The results of all 4 cases are shown in figures 5 – 8 . Also it can be recognized in this example that the difference, caused by different material properties acc. to the mean values and eq. (4), is a small one. For all investigated cases the difference of the deformation in the limit state of serviceability is negligibly small. With the increasing of the single load  $P$  the deviation, according to the achieved load factor, between calculations with design values acc. EC2 and calculations with mean values increases also. But the deviation between calculations with material properties acc. mean values and eq.(4) remains nearly constant.

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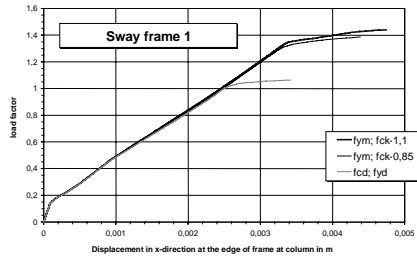
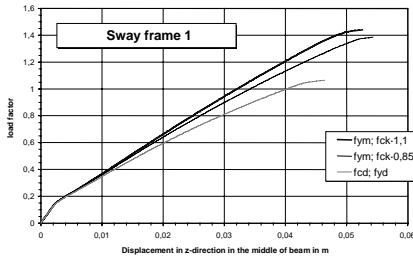


Fig.5: Relation between load factor and deflection of the beam and horizontal displacement for case 1

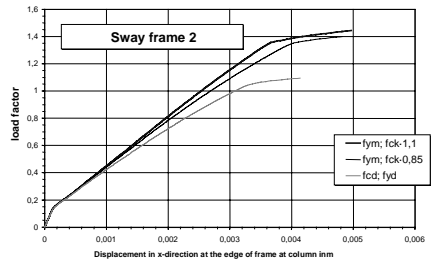
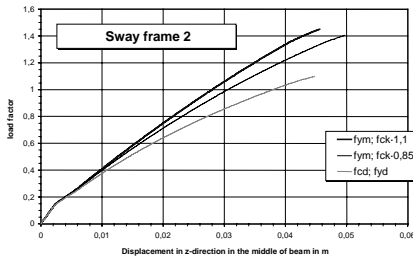


Fig.6: Relation between load factor and deflection of the beam and horizontal displacement for case 2

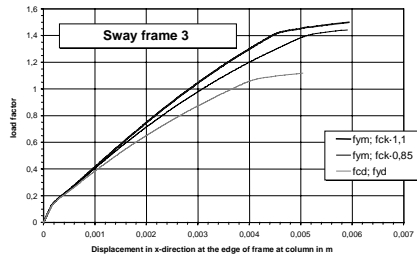
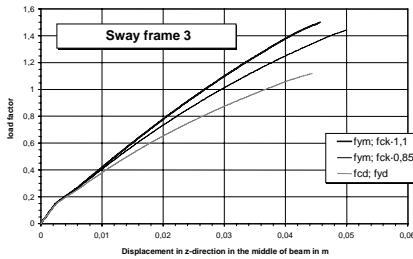


Fig.7: Relation between load factor and deflection of the beam, and horizontal displacement for case 3

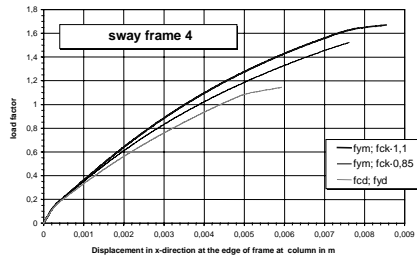
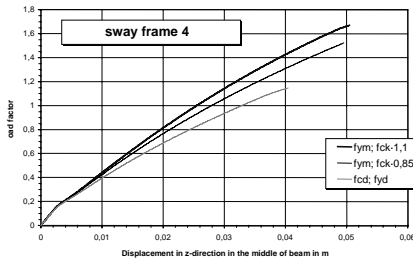


Fig.8: Relation between load factor and deflection of the beam and horizontal displacement for case 4

The examples which are described show that with material properties acc. eq. (4) as input values for non-linear FE-calculations it is possible to reproduce the real behaviour of reinforced and prestressed concrete structures. In comparison to calculations with mean values the deformation and the capacity of redistribution are only marginal influenced by application of the characteristic structural strength of concrete. Furthermore the results show that a load factor of about 1,3 for mainly under bending stress structures result in a sufficient safety margin, if the material properties acc. eq.(4) are used.

#### **4 CONCLUSIUN**

The calculations in chapter 3 showed that with material properties acc. eq. (4) a comparatively insensitive behaviour contrary to the changing of the percentage of reinforcement in the important cross-sections could be achieved. Also the deformation and the capacity of redistribution in the limit state of serviceability of the structure is changed only marginal in comparison to the calculations with “real” mean values of the material properties. The essential advantage of non-linear FE-calculations is maintained.

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