The new RVS 09.01.42 (Tunnel structures in soft soil under built-up areas)
Design Concept and Benchmark

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Abstract

The new issue of RVS 09.01.42 follows the semi-probabilistic design concept of the Eurocode. Standard cases (2-D finite element analyses of tunnel cross sections applying non-linear constitutive relations for the ground and linear elastic material laws for the support) should be analysed using Design Approach DA 2* of EN 1997-1. Additional specifications deal with situations where Design Approach DA 3 has to be applied for ultimate limit state analyses. Finally, suggestions are made in RVS 09.01.42 how to tackle problems when the support is modelled using constitutive relations with implicit limitation of the stress level at high strains.

In order to gain insight into the effects of different design methodologies and Design Approaches a simple benchmark has been studied: The analysis of a bedded circular tunnel lining has been chosen and nonlinear constitutive relations for concrete have been adopted from EN 1992-1-1 and EN 1992-2.

The design methodologies of these codes for ultimate limit state analysis were compared with the procedure suggested in RVS 09.01.42 by means of a parameter study of the benchmark problem. The procedure specified in EN 1992-2 proved to be most conservative. EN 1992-1-1 is somewhat less conservative (by up to 15 % in terms of the maximum characteristic distributed load). With higher bedding stiffness and higher amount of reinforcement the differences decrease. The suggested procedure in RVS 09.01.42 is still less conservative, by another 5 to 15 %. This result is surprising given that the product of all safety factors is identical with EN 1992-1-1. A good match between RVS 09.01.42 and EN 1992-1-1 was obtained by reduction of the ultimate strain in addition to strength.

Motivation

The Austrian guideline RVS 09.01.42 “Tunnel structures in soft soil under built-up areas” 0 is currently being revised. At the time of the previous issue in 2004 0, Eurocode 7 0 was about to be published, but many aspects of the practical application of the semi-probabilistic safety concept were still under discussion. As partial safety factors were already introduced at that time for concrete and steel design this previous issue contained already some regulations concerning tunnel analysis and design for cyclic excavation based on the partial safety factor concept. It was already obvious that the concept proposed in Eurocode 7 0 has its limitations in connection with

numerical methods using nonlinear constitutive laws for both soil and support. In the meanwhile experience and expertise in the application of the partial safety factor concept in general, and the three design approaches of EN 1997-1 0 in particular, have increased. The strengths and limitations of each of the design approaches have become obvious.
Whereas nonlinear constitutive relations for soil have been used already for decades, combining them with nonlinear material laws for the support has been restricted to comparatively few applications. These applications have remained beyond the scope of standards and guidelines. Using nonlinear material laws for the support allows redistribution of forces from highly stressed parts of the support structure to less stressed parts. This may result in a more economic design. However, care has to be taken that a reasonable level of safety is maintained. With the re-design of RVS 09.01.42 0 these advanced techniques should be dealt with, maintaining an appropriate level of safety in accordance with the semi-probabilistic concept of the Eurocodes 0-0 being the first goal.
For retaining structures, which include tunnel linings in the broadest sense, Design Approach DA 2* is the most practical for standard cases. The asterisk indicates that effects of actions, and not the actions itself, are multiplied by a safety factor. It involves the least work in the transition to the Eurocodes. This procedure is also intended as standard in the Austrian guideline RVS 09.01.42 “Tunnel structures in soft soil under built-up areas” in the 2004 issue. However, it is not directly applicable in the case of constitutive laws for the support which limit the possible stress level. Possible remedies have been suggested, but their effect on the safety (in comparison with the established Design Approaches) still needs clarification. To this end, a simple benchmark has been devised and investigated to some depth.

Specifications for Analysis and Design in RVS 09.01.42

History of RVS 09.01.42

Up to 2004, use of the conventional safety concept with global safety factors was specified in RVS 9.32 (the former name of RVS 09.01.42). NATM tunnels were usually investigated by analyzing representative 2D plane strain sections, for the ground linear elastic – perfectly plastic constitutive laws like the Mohr-Coulomb-model were applied. For the shotcrete – frequently the most important means of support – beam elements with linear-elastic material behaviour were employed. For the friction angle and cohesion of the ground nominal values were utilized. Different values of stiffness of young and mature shotcrete were suggested. The reinforcement was designed according to 0 with a global safety factor on internal forces.
Starting with the 2004-issue 0, it was distinguished between partial safety factors on actions and partial safety factors on resistances for ultimate limit state design. Additionally, a separate serviceability limit state design was introduced. In accordance with the specifications for retaining structures in Eurocode 7 0 and the National Annex 0, use of Design Approach DA 2* has been specified. Failure of the soil is governed by strength parameters, which are hardly dependent on the dead load of the ground. Amplification of the dead load directly by a factor does not result in additional safety. Since active soil pressure and soil resistance is a result of the analysis, the boundary between active and passive regions cannot be known in advance. Compared with the pre-2004-issues of RVS 9.32 0 the required amount of reinforcement decreased slightly.
Experience with both the pre-2004-issues and the 2004-issue suggests a sufficient level of safety. The authors are not aware of any damage to a shotcrete lining which could be attributed to

insufficient safety margins in the design of the reinforcement.

Current version of RVS 09.01.42
In the current versions two aspects of the design have been paid more attention to:

- The stability of an unsupported face which is dominated by the shear parameters of the ground. Here application of Design Approach 3 is suggested in the guideline.
- Numerical tools in combination with non-linear material laws for support which implicitly confine the allowable stress level of the support do not allow the application of DA 2* any more: Internal forces cannot be increased beyond the inherent limits of the material law; amplification works only within the elastic range of the material description. As a remedy, one can either use Design Approach 3 where the partial safety factor on permanent (geotechnical) actions is 1.0. (Variable actions can be increased directly by a factor of 1.5/1.35 = 1.11.) This option is also specified in Appendix A of ÖNORM B 1997-1-1 0, with the restriction that relatively conservative values for the partial safety factors on friction angle and cohesion of the ground are used. This option is primarily focused on failure of the ground (GEO) 0. If possible failure of the support is predominant (STR) the only remaining option appears to be applying an overall safety factor on the strength parameters of the support.

In the current issue of RVS 09.01.42 0 it has been specified, that both types of failure should be investigated, and that the global safety factor on the support strength parameters should be chosen as $\gamma_R \cdot \gamma_E$, where γ_R is the partial safety factor on the support strength parameters and γ_E is the partial safety factor on effects of actions.
These specifications are plausible and appear to guarantee a reasonable safety level. In order to gain more insight into the effects of different assumptions, the RVS-specifications had to be compared with specifications directly in the Eurocodes. As a first step, a simple benchmark has been devised.

Benchmark

General Design
It is not an easy task to find an example which allows straight-forward comparison of different design methodologies if nonlinear constitutive relations for both ground and support are used. (Either DA 3 cannot be used because the shear strength of the ground is not considered, or the failure criterion in shear prevents direct application of partial safety factors on dead load.) A simple way to cover at least some ground-support-interaction is a collapse load analysis for the secondary lining of a typical tunnel cross section. In the chosen example the lining is circular, and bedded by radial springs with constant stiffness in compression and no stiffness in tension. The lining is loaded by a constant distributed load in vertical direction, see Figure 1. Stiffness and strength of the support (i.e. the secondary tunnel lining) interact with the stiffness of the ground. The strength parameters of the ground do not affect the results. The analysis is performed using different software packages.

Geometry
The geometry of the structural model is given in Figure 1.
Geometric parameters are given in Table 1.

[3] Table 1: Geometry / Parameters

Geometry / Parameters	**Unit**	**Value**
Secondary lining thickness h	cm	40
Tunnel radius R (to lining centerline)	m	5.0
Reinforcement a_s per face	cm^2	3.85
Concrete cover (to center of reinforcement)	cm	5.0

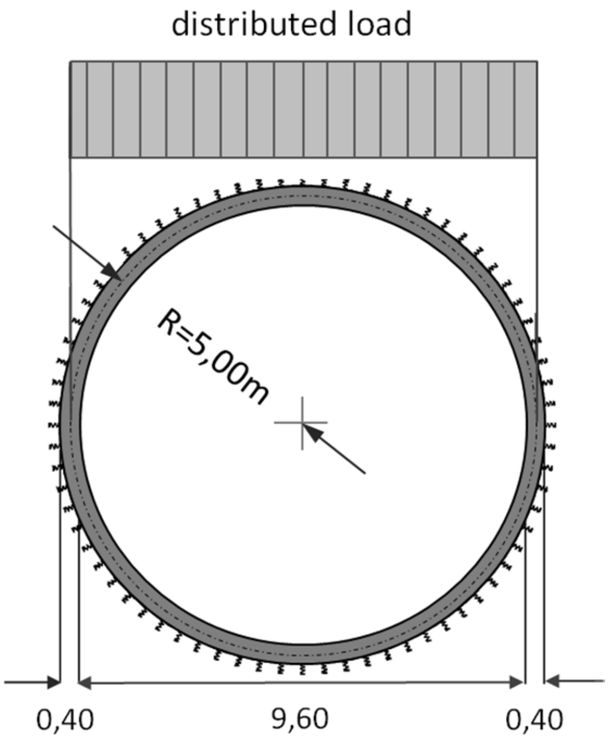

Figure 1: Structural model, bedding and ground pressure

Material and bedding properties
Used material properties for concrete grade C25/30 are given in Table 2.

[4] Table 2: Material properties for concrete grade C25/30

Properties	**Unit**	**Value**
Young's modulus E_{cm}	GPa	31
Mean compressive strength f_{cm}	MPa	33
Characteristic cylinder strength f_{ck}	MPa	25
Design strength f_{cd}	MPa	16.67
Mean tensile strength f_{ctm}	MPa	2.6

Both, the concrete tensile strength and the effects of tension stiffening are neglected in a first run. Material properties for reinforcement (steel grade B550B) are given in Table 3.

[5] Table 3: Material properties for reinforcement B550B

Properties	Unit	Value
Young's modulus E_s	GPa	200
Yield strength f_{yk}	MPa	550
Design yield strength f_{yd}	MPa	478
Min value of ratio k $(=f_t/f_y)_k$	-	1.08
Strain at maximum load ε_{uk}	%	≥ 5.0

The lining is bedded with springs in radial direction having a bedding stiffness of k_S=20 MN/m³.

Load

Only a constant, vertical load is applied to the structure, acting over the tunnel diameter on the projected length (see Figure 1). Dead loads are neglected.

Design methodologies

Design Approach DA3 according to Eurocode 7 0 is not suitable for this problem type (no effect of ground shear strength on the results). As a consequence, mainly design methodologies suggested by Eurocode 2 are applied for the analysis, such as:

Methodology according to EN1992-1-1, clause 5.8.6 0 (the only clause in this standard which deals with ultimate limit states in conjunction with non-linear analysis)

Nonlinear procedure according to EN1992-2, clause 5.7 0. In this procedure an overall safety factor γ_O = 1.27 is used.

Both procedures are based on the stress-strain-relation of 0, clause 3.1.5, and not on the parabola-rectangle-diagram, 0, clause 3.1.7.
Additionally, the method stipulated in RVS 09.01.42 0 (double reduction of material strength) is investigated. In this case, the procedure according to 0, clause 5.8.6, is applied, but with further reduction of the strength parameters. For details see 0-0.

Stress-strain relationship of concrete

Depending on the design methodology different stress-strain relationships for concrete are needed:

- Stress-strain relationship for structural analysis according to EN 1992-1-1 0, clause 5.8.6. (σ_{NL1} in Figure 2). In equation (3.14) and for the determination of the ratio k, the mean compressive strength f_{cm} is replaced with the design compressive strength f_{cd}. Furthermore, E_{cm} will be replaced with $E_{cd} = E_{cm}/\gamma_{CE}$ (see Table 4 for safety factors).
- Stress-strain relationship according to EN 1992-2 0, clause 5.7 (σ_{NL2} in Figure 2).The methodology according to EN 1992-2, clause 5.7 is based on the stress-strain relationship following equation (3.14) with replacement of f_{cm} in equation (3.14) and the k-value by γ_{cf} f_{ck} ($\gamma_{cf} = 1.1\ \gamma_S/\gamma_C$).
- Doubly reduced strength parameters according to RVS 09.01.42 0, chapter 5 (σ_{NL3} in Figure 2). The characteristic support properties are reduced with factor $\gamma_R \cdot \gamma_E$ where γ_R represents the partial factor of safety (material) according to relevant standards and γ_E a factor according to RVS 09.01.42, clause 5.2.

[6] Table 4: Safety factors applied for stress-strain relationship of concrete

Safety factors		**Value**
γ_{CE}		1.2
$\gamma_{cf} = 1.1\ \gamma_S/\gamma_C$		0.843
with	γ_S (partial factor of safety for reinforcement)	1.15
	γ_C (partial factor of safety for concrete)	1.5
$\gamma_R \cdot \gamma_E$		2.025
with	$\gamma_R = \gamma_C$ (partial factor of safety for concrete)	1.5
	γ_E (partial factor of safety for permanent actions)	1.35

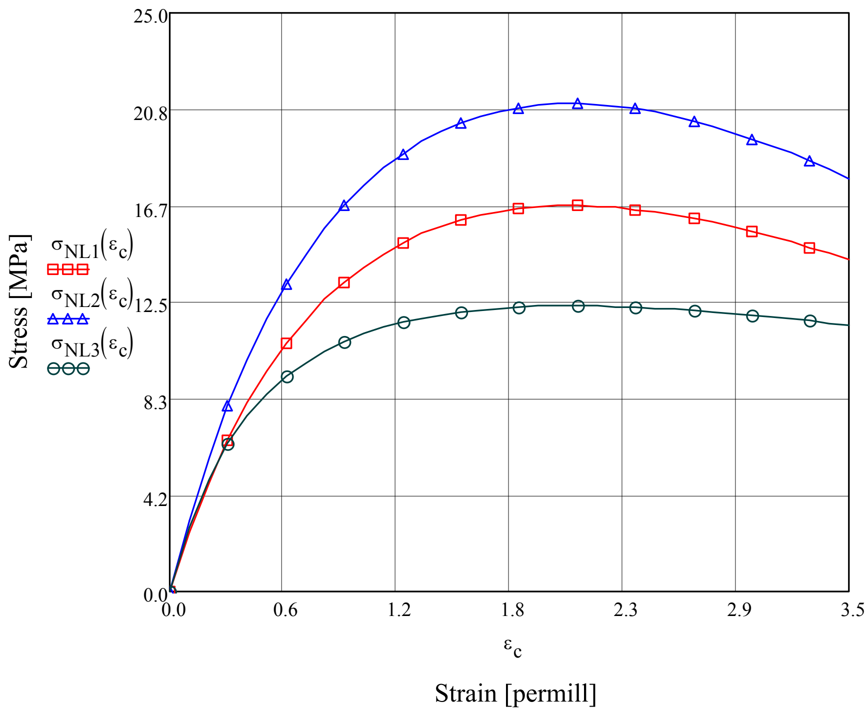

Figure 2: Stress-strain relationship of concrete under compression.

The benchmark is checked for ultimate limit state (ULS) only - serviceability state checks (such as limitations for crack width, stresses, deformation) are not accounted for.

Software packages, discretization

Four finite element programs have been employed to carry out the test. In all of them the model is discretized with beam elements. Bedding is simulated using radial springs (active only in compression).

Two of the packages handle material nonlinearities by numerically integrating the stresses in thickness direction over a number of layers. Another two packages use a flexibility based approach, handling the nonlinearity based on the relationship between moment and curvature. To ensure that nonlinearity of concrete and steel is treated correctly in all programs, a simple test on a cantilever beam with a moment and an axial force at the free end has been carried out. The programs used in this benchmark are listed below; all of them passed this check.

- Program M (layered beams)
- Program Z (layered beams)
- Program S (flexibility based approach)
- Program C (flexibility based approach)

In all packages two-node beams are chosen, except in program M, which offers three-node beam elements as well. The model has an equal amount of nodes.
The number of layers and the layer thicknesses for the benchmark are different in programs M and Z and have been chosen according to previous experience with those packages (17 concrete layers with varying thickness in program M, 20 concrete layers with constant thickness in program Z). In both programs the reinforcement layers have been modelled as thin steel layers. The results obtained with programs M and Z were close enough to each other that one can assume that the integration over the thickness is accurate enough. All analyses are load driven.

First Results

In a first step results had to be obtained for the case without tension stiffening. It was found that the resulting ultimate load strongly depends on the mesh size which was chosen in a first guess according to experience. It turned out that the mesh dependency is mainly caused by the distance of integration points from the location of extreme bending moments. To achieve better comparability afterwards the secondary lining has always been divided into 60 beam elements (programs Z, S and C) and 30 beam elements (program M) and 60 springs. Care was taken that in all models one integration point is situated at the uppermost point of the ceiling (extremum of the bending moment).

With the adjusted mesh defaults, the following ultimate limit loads have been determined for characteristic material strength (f_{cm}, f_{yk}):

[7] Table 5: Ultimate Distributed Load without Tension Stiffening

Program	Unit	Ultimate Load
M	kPa	540
Z	kPa	530
S	kPa	298
C	kPa -	500

It appears that programs with a flexibility based approach have problems to overcome a certain point when approaching the ultimate limit state. The results of programs which are based on layered beam formulation are similar.

In a next step tension stiffening was taken into account. The left graph in Figure 3 shows a first definition of tension vs. strain, which was confirmed by all participants to be a practical assumption.

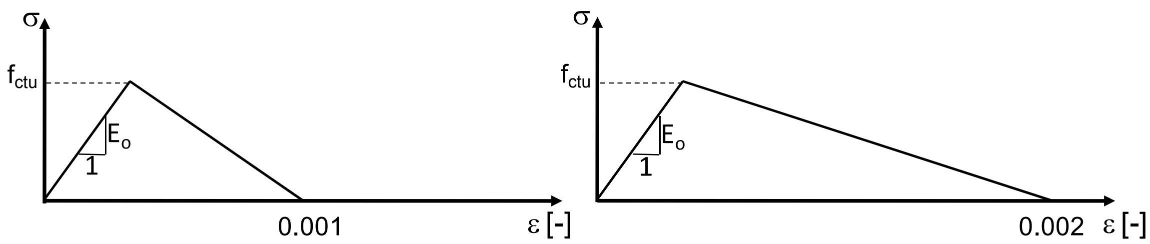

Figure 3: Assumptions for tension softening

Surprisingly all of the programs had problems to obtain convergence and it was necessary to change the tensile region of the stress strain curve according to the curve depicted in the right graph of Figure 3.

A detailed examination of the development of the relation between moment and axial force showed that all the programs had difficulties to overcome the beginning of fracturing. This seems to be a difficulty for structures with low reinforcement in general.

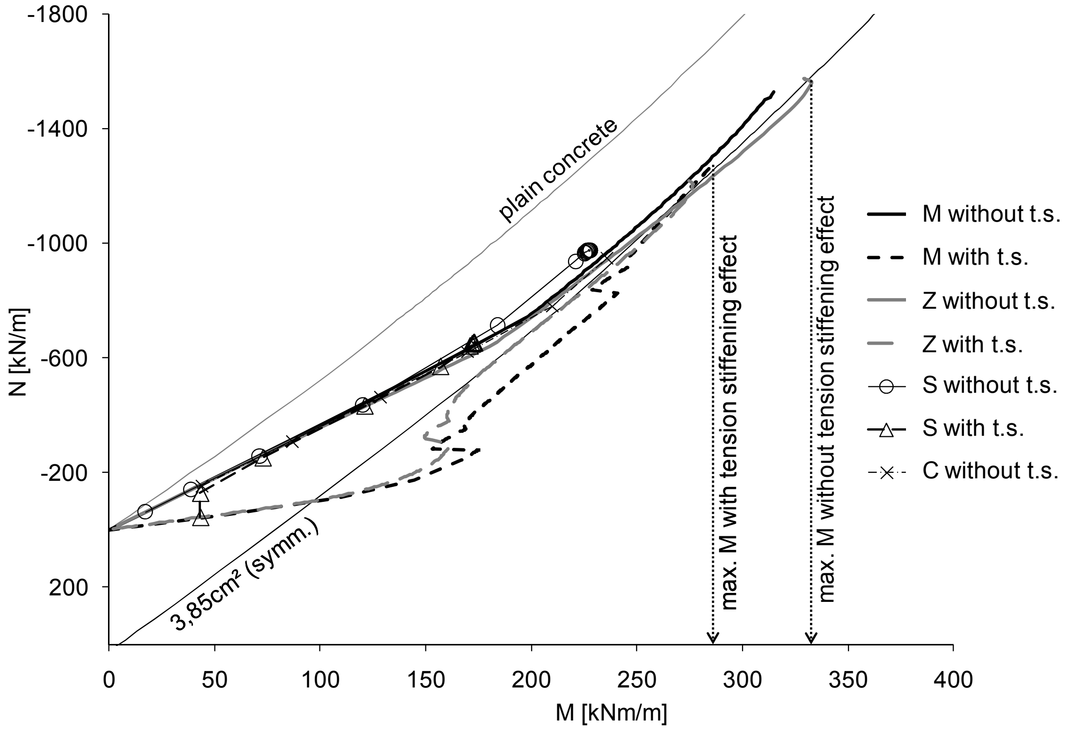

Figure 4: Development of M-N relation with and without tension stiffening (t.s.) for different FE codes M, Z, S and C.

Normally one would assume that with tension stiffening the capacity of structures will increase, but in this case without tension stiffening a higher ultimate load has been obtained (Figure 4). With no physical explanation at hand the authors assume that the differences are caused by numerical instability only. It was decided to neglect tension stiffening in the following benchmark analyses.

Another interesting aspect is that the load-displacement curves do not show a typical plateau due

to yielding; they have only a slight curvature and obviously exhibit brittle failure characteristics. This view is corroborated by the fact that failure occurs in the relatively small compression zone: As soon as all layers in the compression zone have reached the descending branch of the stress-strain-relation the failure load of the beam is reached.
It should be mentioned, that shear failure was not investigated, since in all programs shear is handled linearly.

Comparison of Design Methodologies

In order to follow the primary purpose of the benchmark a comparison of the requirements of EN 1992-1-1 (section 5.8.6), EN 1992-2 (Section 5.7) and the suggested variant of DA 2 specified in the new version of RVS 09.01.42 was conducted.
To get an overview, a range of possible values of the bedding stiffness and of the amount of reinforcement has been studied. The two programs M and Z with a layered-beam-approach have been used and mean values of their results have been calculated; tension stiffening effects have been neglected. Table 6 summarizes results obtained with the three approaches. For easier assessment, the characteristic values of the applied pressure p_k have been compared, and not the obtained (average) pressure just before failure, p_{ult}:

EN 1992-1-1: $p_k = p_{ult} / \gamma_E = p_{ult} / 1.35$
EN 1992-2: $p_k = p_{ult} / \gamma_E / \gamma_O = p_{ult} / 1.35 / 1.27$
RVS: $p_k = p_{ult}$

[8] Table 6: Calculated maximum characteristic distributed load in [kN/m²]

AA_S [cm²/m]	3.85			7.7			20.0
Bedding [kN/m³]	4000	20000	100000	4000	20000	100000	20000
EN 1992-1-1	124	255	481	149	282	537	362
EN 1992-2	108	220	429	127	248	483	330
RVS	142	287	515	162	318	565	396
RVS, red.ε_{cu}	129	256	492	140	285	544	368
Linear elastic	12.3	25.1	74	24.2	49	136.5	118

There is an additional line referring to "RVS, red. ε_{cu}" which will be discussed later.
As already mentioned, the values in the table result from the average of results of codes Z and M. The maximum difference between the results was approx. 5 percent.
From Table 6 and Figure 5 it is obvious, that the procedure according to EN 1992-2 yields the most conservative results. A possible cause for the difference might be, that the different value for α_{CC} (the recommended value is 1.0 in EN 1992-1-1 and 0.85 in EN 1992-2) does not affect the procedure described in clause 5.7.
Surprisingly, the results of EN 1992-1-1 and RVS differ (up to around 15 %) despite using the same product of partial safety factors.

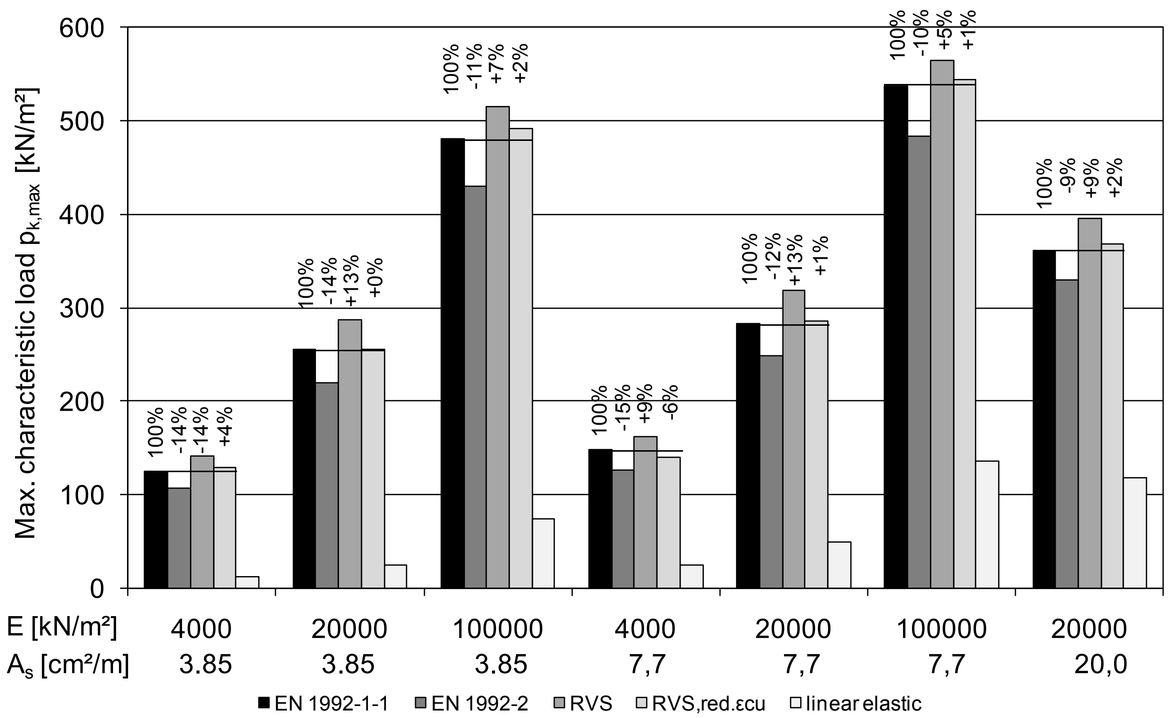

Figure 5: – Comparison of results.

The analyses for the RVS-procedure were repeated with the same safety factors, but additional reduction of the ultimate compressive strain of the concrete, ε_{cu}. Again the factor $\gamma_E = 1.35$ was applied to ε_{cu} , resulting in a limitation of 2.59 ‰ instead of 3.50 ‰. Application of this value resulted in the distributed loads in the sixth line of Table 6 which are in the range of 5% of the results for the procedure of EN 1992-1-1. A possible explanation for the relative strong influence of ε_{cu} (whereas a reduction of ε_{c1} had almost no effect on the ultimate load) is the effect of the shape of the stress-strain-relation in compression on the moment carrying capacity of each cross section (larger lever arm of the resultant compressive force). With increasing stiffness of the bedding the relative differences between the procedures decrease (except for one case), as well as the eccentricity M/N in the cross section. The differences also have a decreasing tendency with increasing amount of reinforcement.
The calculated values were also compared with the design according to EN 1992-1-1 using relation 3.1.7 for cross sectional forces and moments from a linear elastic analysis using the same amount of reinforcement as in the nonlinear analyses. As can be seen from Table 6 and Figure 5 the difference between linear and nonlinear analyses is huge: With low reinforcement, the linear elastic analysis allows only about 10 to 20 percent of the load. With higher reinforcement and higher bedding stiffness the differences decrease, but are still in the range of 70 % and above.

Conclusions

The new issue of RVS 09.01.42 is closer related to the Eurocodes than the previous issue of 2004. New specifications deal with the applicability of the Design Approaches of EN 1997-1, also in connection with nonlinear constitutive models for support. Because experience with the

new specifications and suggestions is limited, a simple benchmark problem has been defined. It allows the comparison of design procedures specified in EN 1992-1-1 and EN 1992-2 for nonlinear analyses with the corresponding specifications in RVS 09.01.42.
Some of the most interesting or surprising results are summarized below:

1. Despite the simplicity of the benchmark considerably different ultimate loads (under load control) are obtained if different engineers tackle the problem with different software packages. The differences in ultimate load can be easily 50 % and more. Only after scrutiny and elimination of all sources of discrepancies the results of programs M and Z (both using layered beams) agreed within about 5 %.
2. The descending branch of the stress-strain-relation of concrete in compression governs the overall load-displacement curve: Only a slightly nonlinear curve can be observed, the failure of the structure appears brittle in the analyses. It should be pointed out that no horizontal load was applied.
3. Modelling Tension Stiffening with the help of a softening branch of the stress-strain-relation in tension appears to be very difficult to handle with finite element simulations. Neglecting Tension Stiffening resulted in higher ultimate loads although the opposite had to be expected.
4. The result obtained by the design procedure specified in EN 1992-1-1 and EN 1992-2, respectively differ by up to 15 %. Results with EN 1992-2 are more conservative, the differences increase with increasing eccentricity of the axial force.
5. The procedure suggested in RVS 09.01.42 is even less conservative than EN 1992-1-1 unless not only the strength parameters, but also the ultimate strain (before failure) are reduced. Again, the differences increase with increasing eccentricity of the axial force.
6. Compared with a linear elastic analysis and design according to EN 1992-1-1, clause 3.1.7, the ultimate load is at least 70 % higher using the fully nonlinear model. This was true for all variants of the parameters of the benchmark studied.

Limitations of the benchmark problem are

1. No effect of the shear strength of the soil, therefore no comparison with DA 3 of EN 1997-1 is possible.
2. Just different design methodologies and approaches based on the semi-probabilistic concept of the Eurocodes have been compared. A fully probabilistic investigation is missing.
3. Time dependent behaviour of support and ground has been neglected.

Other types of benchmark, e.g. for face stability analysis or for shotcrete as support, have to be undertaken in order to generalise the findings.

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