B8 - Prestressed Concrete Girder - Verifications

! ! This documentation is part of the manual describing the B8 software

Contents

Reference example: You can find an extensive example of an output at
www.frilo.eu B8-Ref-BS-Eng.pdf

Abbreviations used in this document:

EN 01/01/1992:	EN2
DIN EN 1992-1-1/NA:	NA_D
PN EN 1992-1-1/NA:	NA_PN
ÖNORM B 02/01/1992:	NA_A
NA to BS EN 1992-1-1	NA_GB

Verifications

Calculation of the creep factor and the shrinkage strain

Depending on the selected options, the calculation is performed either on a single defined cross-section or on each examined cross-section.

Creep factor and shrinkage strain as per EN2, NA_D, NA_PN, NA_A

The calculation is performed for each creep stage as per 3.1.4 and Annex B, based on the following parameters:

Creep

α	exponent in equation B.9, depends on the type of cement
t0T	age of the concrete at the beginning of the creep stage. If heat treatment was applied in the stressing bed, the concrete age is increased at the beginning of the storage in accordance with equation B.10. Otherwise, *t0T* is determined by *tT* of the previous creep stage.
t0TA	concrete age at the beginning of the creep stage, modified in line with the cement type as per equation B.9, to be used in equation B.5
tT	concrete age at the end of the creep stage; for periods with temperatures unequal to 20° C, modified concrete age as per equation B.10.
βH	coefficient for the effective component thickness h0 and the relative humidity RH in % as per equation B.8 a, b
h0	effective component thickness in [mm] with the cross-sectional area Ac and the cross- sectional perimeter exposed to fresh air as per equation B.6
βtt0	coefficient to describe the creep behaviour over time as per equation B.7, modified as per reference /22/ p. 261, equation 3.104 to β (t - tk) - β (t0 - tk) with tk referring to the start of the creep
φRH	coefficient to consider humidity in the determination of the basic creep factor as per equation B.3 a, b
β(t0)	coefficient to consider the concrete age in the calculation of the basic creep factor as per equation B.5
β(fcm)	coefficient to consider the concrete strength in the determination of the basic creep factor as per equation B.4
φ(t,t0)	creep factor as per equation B.1 in the examined creep stage

Shrinkage strain

βt0ts	coefficient to describe the shrinkage behaviour over time as per equation 3.10 until the beginning of the creep stage
β▯t0ts	coefficient to describe the shrinkage behaviour over time as per equation 3.10 until the end of the creep stage
βRH	coefficient to consider the relative humidity as per equation B.12
εcds0	coefficient to consider the influence of the cement type and of the compressive concrete strength on the drying shrinkage as per equation B.11
βas	coefficient to consider the age during shrinkage as per equation 3.13
εas	coefficient to consider the compressive strength and the cement type for the shrinkage as per equation 3.12
ε(t,t0)	shrinkage strain as per equation 3.8 in the considered creep stage, modified in accordance with reference /11/, 2.6

Non-linear creep

If the limit compressive concrete stress cannot be complied with under the loads of the quasi-permanent load combination, an increased creep factor for non-linear creep is used in the calculation (equation 3.7). This factor is put out in the table ‘Internal forces by prestressing’ if a detailed output was selected.

Cross-section properties

First, the cross-section properties (area, moment of inertia and centre of gravity) of the gross concrete cross-section are calculated with consideration of the current girder height, of possible support reinforcements and recesses.

After this, the ideal cross-section properties are determined with consideration of the net concrete cross-section and pre-tensioned or untensioned reinforcement.

For cast-in-place concrete complements, the cross-section properties of the composite cross-section are determined in addition. The cast-in-place concrete cross-section is considered with its effective width.

The cross-section properties determined this way are specified in the detailed output of the cross-sections.

Support reactions and internal forces

Support reactions

The support reactions are determined with all loads acting at the time $t = \propto$ on the support.

The following values are put out for the left and right column:

- G:	due to permanent loads (characteristic values)
- min Q, max Q:	due to variable loads (characteristic values)
- min R, max R:	resultant forces *G+Q*

If the option ‘All sections detailed’ is checked in the output profile, the characteristic support reactions are put out for all load cases and load components.

Internal forces

The internal forces for the self-weight and the defined imposed loads are calculated on the effective structural system.

During the storage of the girder, the same structural system is assumed as in the installed state with the supporting distance LST.

During the erection, the structural system is determined by the location of the suspension points.

Longitudinal forces caused by cable-stayed supports are not considered.

During the casting of the cast-in-place complement, auxiliary supports are considered, if applicable.

It is assumed that the columns are placed underneath the girder that is deformed by its self-weight and pre-tensioning. This means that the weight of the cast-in-place concrete and a possibly existing concreting load act on this structurally undetermined system. The internal forces and the supporting forces under the load GE+BL are determined on a two-span or three-span girder with a constant supporting distance and with rigid supports. If the girder is continuously supported, no internal forces are generated through the concreting load and the weight of the cast-in-place concrete until the support is removed.

After removal of the support, the supporting forces of the auxiliary support act as a load on the girder to which the cast-in-place complement was applied.

In connection with cast-in-place complements, the specification of unequal distances to the adjacent girders may produce an extremely asymmetrical cross-section, which must be designed under oblique bending.

Oblique bending and possible torsion effects cannot be considered in the current version of the software.

Combinations of actions by external loading

The internal forces are determined in accordance with the theory of elasticity. Therefore, the superposition law applies to the internal forces and you can combine the internal forces instead of the actions.

For shear force, the combination criterion is the absolute amount. For the moments, Mmax (tension on bottom) and Mmin (tension on top) are determined.

The internal forces determined this way are put out in the table 'Internal forces due to external loading', when the detailed output of the cross-sections was selected:

- See the output example: Internal forces due to external loading

Effective prestress

Transfer of the pre-stressing force

The effective prestress at the time when the anchoring is released (t= tA,Lag) is specified in the table "Effective Tendons" in the detailed output of the cross sections.

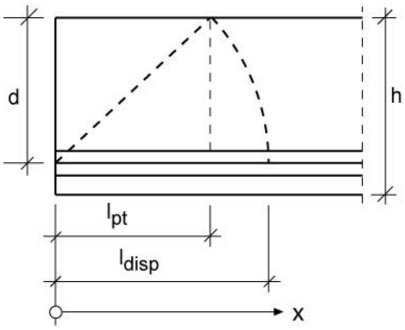
Only tendons with bond, i.e. tendons that are not stripped at the corresponding point are considered.

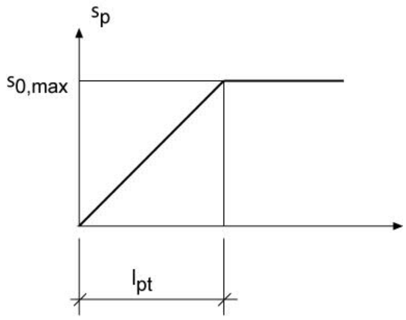
Within the transfer length l_{pt}, you are allowed to assume that the pre-stressing force has been totally transferred to the concrete. A linear transfer may be assumed.

If the cross-section to be verified is in the area of the transfer length of individual tendons, the effective prestress is reduced accordingly.

According to 8.10.2.2. (3), an unfavourable design value should be used in the respective verification (l_{pt2}= 1.2 * l_{pt} or l_{pt1}= 0.8 * l_{pt}), i.e. in the area of the transfer length, a maximum and a minimum prestress occurs after releasing the anchoring.

Outside of the force application length l_{disp}, you are allowed to assume a linear behaviour of the concrete stresses. The tensile splitting forces generated within l_{disp} due to the non-linear behaviour of the stresses must be compensated with a tensile splitting reinforcement.





	EN 1992-1-1	
Transfer length	l_{pt}	eq. 8.16
Force application length	l_{disp}	eq. 8.19

$l_{pt} = \alpha 1 \cdot \alpha 2 \cdot \phi \cdot \sigma pm0 / fbpt$ eq. 8.16

$\alpha 1$: coefficient prestressing force application suddenly: 1.25 gradually: 1.0

$\alpha 2$: coefficient prestressing steel type strands 0.19 others: 0.25

ϕ: rated diameter of prestressing steel

$\sigma pm0$ stress in the prestressing steel for t = tA,Lag caused by prestressing

$fbpt = \eta 1 \cdot \eta p1 \cdot fctd(t)$ bond stress as per eq. 8.15

$\eta 1$: bond coefficient (according to 8.4.2: good bond 1.0; otherwise 0.7; definition according to Figure 8.2)

$\eta P1$: coefficient prestressing steel type

NA_D: $\eta p1 = 2.85$

Otherwise: wires: 2.7 strands: 3.2

$fctd(t) = \alpha ct \cdot 0{,}7 \cdot fctm(t) / \gamma c$ design value of the tensile strength for t= tA,Lag

fctm(t) as per eq. in table 3.1 with fck(t)

NA_D: lt. /56/ p. 324 instead of fck(t) with fcm(t)

αct: NA_D: 0.85

Otherwise: 1.0

or user-defined *lpt*

$$l_{disp} = \sqrt{lpt^2 + d^2}$$ eq. 8.19

Losses until the release of the anchoring of the prestressing reinforcement

Until the release of the anchoring, losses due to a short-term relaxation of the prestressing steel as per 5.10.3. (3) (see the paragraph prestressing steel relaxation) and possible losses due to a heat treatment before the release as per 10.5.2. (1) must be considered.

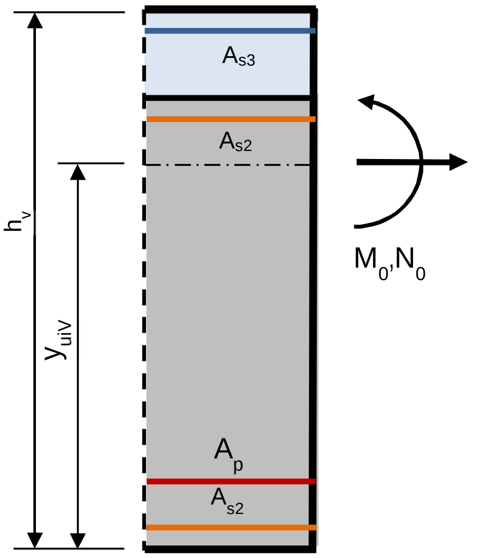
Losses after the release of the anchoring of the prestressing reinforcement

After the release of the anchoring and the transfer of the prestressing force to the concrete, creep and shrinkage begin and additional losses occur due to the relaxation of the prestressing steel.

The equation 5.46 of DIN EN 1992-1-1, which is used in many examples in expert literature, is only appropriate for a very limited range of border conditions. NCI to 5.10.6 (2) in DIN EN 1992-2/NA recommends more accurate calculations if multi-strand prestressing, high longitudinal reinforcement ratios with slag reinforcement or composite cross-sections of different concrete types are used.

The calculation in the software is performed in accordance with the method described by Abelein in reference /13/. We explain the method briefly below.

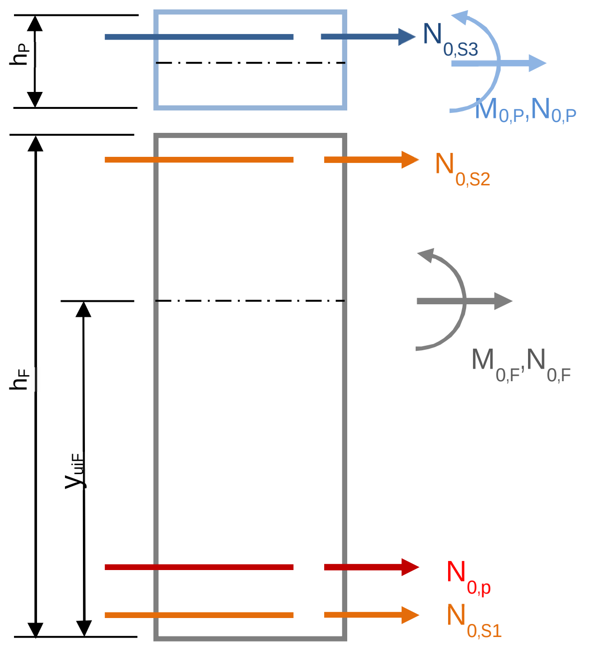
1) Creep-generating loads N_0, M_0



The creep-generating loads M_0 and N_0 consist of all effective permanent loads (characteristic values) as well as of the prestress (average value). Quasi-permanent load portions of variable loads are only considered if they do not have a relieving effect i.e. do not counteract prestress.

First, the resulting forces M_0 and N_0 of these loads that apply at the centre of gravity of the composite cross section are distributed over the k partial cross sections (pre-cast concrete, cast-in-place concrete, if applicable, prestressing steel layers and reinforcing steel layers).

2) Load portions $N_{0,k}$, $M_{0,k}$ of the partial cross-sections



In accordance with equation 3a, 3b, the following results:

M0,k = nk * Ik/Ii* M0

N0,k = nk * Ak/Ai* N0 + nk * Sk/Ii * M0

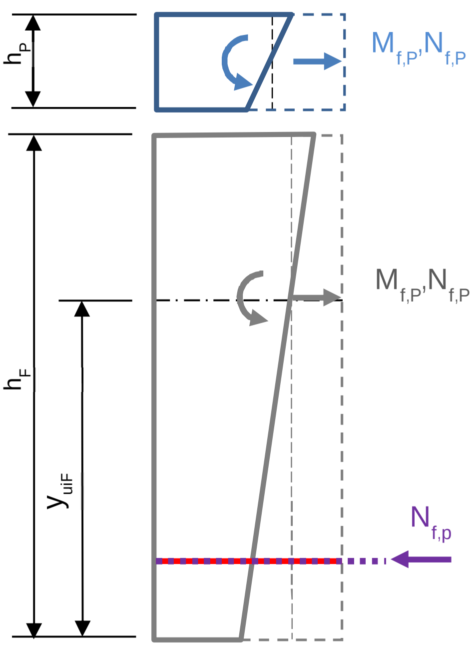
nk= Ek/Ei	relation modulus of elasticity of partial cross-section to *Ecm* (precast component)
Ak, Ik, yuk:	Cross-sectional properties of partial cross-section
Ai, Ii, yuik:	Cross-sectional properties of composite cross-section

Each partial cross-section would be affected by the following strains and deformations due to creep, shrinkage and relaxation. These effects are compensated by the retaining forces *Nf,k* and *Mf,k* for the composite cross-section.

ε_{csr}=ϕk * N0,k/(Ak * Ek)+εs,k + 0.8 * εp,r   χcsr= ϕk * M0,k/(Ik * Ek)   (eq. 5a, 5b):

The factor 0.8 that is applied to the portion from relaxation considers the effect that the lowering of the stress in the prestressing steel due to creep and shrinkage strain also reduces the relaxation losses.

3) Retaining forces $N_{f,k}$, $M_{f,k}$



The general relationship for time-dependent deformations of rigid composite systems based on equation 2 is as follows: 2

$\varepsilon(t) = \sigma_0/E * (1+\phi)+\Delta\sigma(t)/E * (1+\rho * \phi)+\varepsilon_s(t)$

The resulting deformations due to *Nf,k* or *Mf,k* as per equation 6a, 6b are as follows:

$\varepsilon_f = (1+\rho k * \phi k) * Nf,k/(Ak * Ek)$

$\chi_f = (1+\rho k * \phi k) * Mf,k/(Ik * Ek)$

If $\varepsilon_{csr} = \varepsilon_f$ and $\chi csr = \chi_f$, the retaining forces as per equation 7a, 7b are as follows:

$Nf,k=(N\phi,k + Ns,k + Nr,k)/(1+\rho k * \phi k)$

$Mf,k= M\phi,k /((1+\rho k * \phi k)$

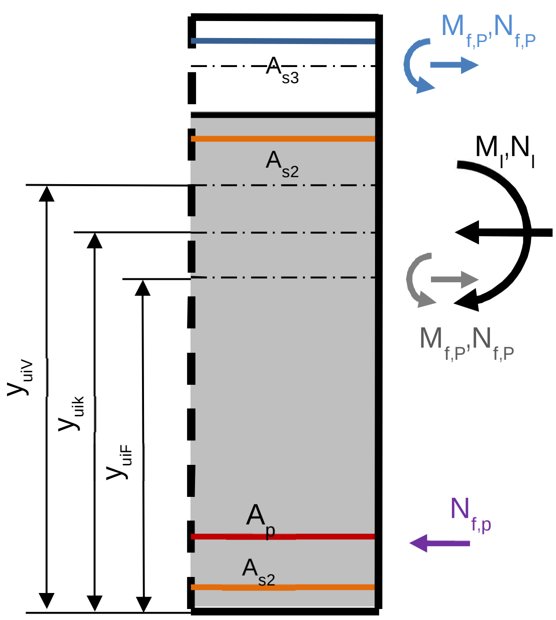
with the portions from creep and shrinkage strain (only on concrete cross-sections)

$N\phi,k = N0,k * \phi k$ $\quad$ $Ns,k= Ak * \varepsilon s,k * Ek$

$M\phi,k = M0,k * \phi k$

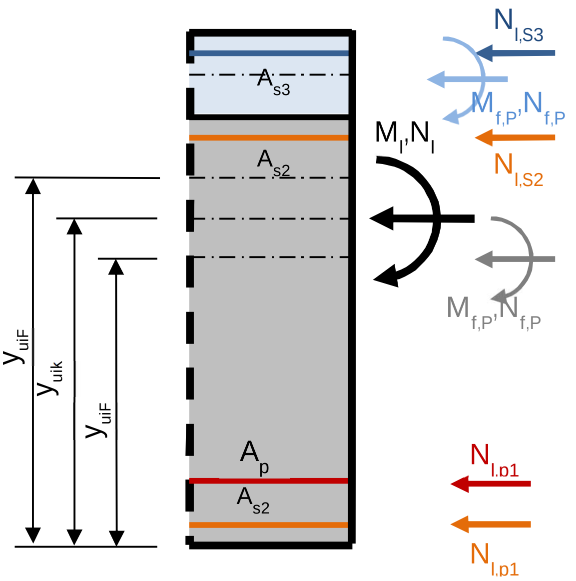
and the portion from relaxation (only prestressing steel) $Nr,k= Ak * \varepsilon k,r * Ek$

4) Releasing forces N_l, M_l



The resulting retaining forces are applied in the reverse direction to the centre of gravity of the total cross-section as so-called releasing forces *N_l, M_l*. After creep, this cross-section assumes modified ideal cross-sectional properties, based on the cross-sectional properties of the partial cross-sections reduced by $Ek/(1+ \rho k * \phi k)$, and is therefore also referred to as creep cross-section. This modification also includes the distance of the centre of gravity to the lower edge y_{uik}.

5) Releasing forces $N_{f,k}$, $M_{f,k}$



The releasing forces on each partial cross-section *Nl,k* and *Ml,k* are determined analogously to *M0,k* and *N0,k*. Instead of *nk*, *nk*=nk/(1+ρk * ϕk)* is to be considered, however.

The loss due to creep, shrinkage and relaxation for each partial cross section is determined by the sum of the retaining and releasing forces (eq. 10a, 10b) of the respective partial cross-section. The resulting stresses in the prestressing steel layers and the reinforcing steel layers are converted to the prestressing bed condition (/10/, eq. 45b).

Note: *Due to the creep stresses in the reinforcing steel that are considered in the verification as per Eurocode/DIN 1045-1 because of the higher reinforcing steel portion (see also /22/p. 248), the effective prestress referenced to the prestressing bed condition can be affected by considerably higher losses caused by creep, shrinkage and relaxation than we know from experience, e.g. when applying equation 5.46 in DIN EN 1992-1-1, especially with low prestressing rates.*

If the compressive concrete stresses in the quasi-permanent load combination are not complied with, a higher creep factor as per /30/ 11.1.1.2 (2) must be used in the calculation. This factor is put out in the table 'Internal forces by prestressing' if a detailed output was selected.

See the output example: Losses due to creep, shrinkage and relaxation

Measures to reduce creep and shrinkage losses:

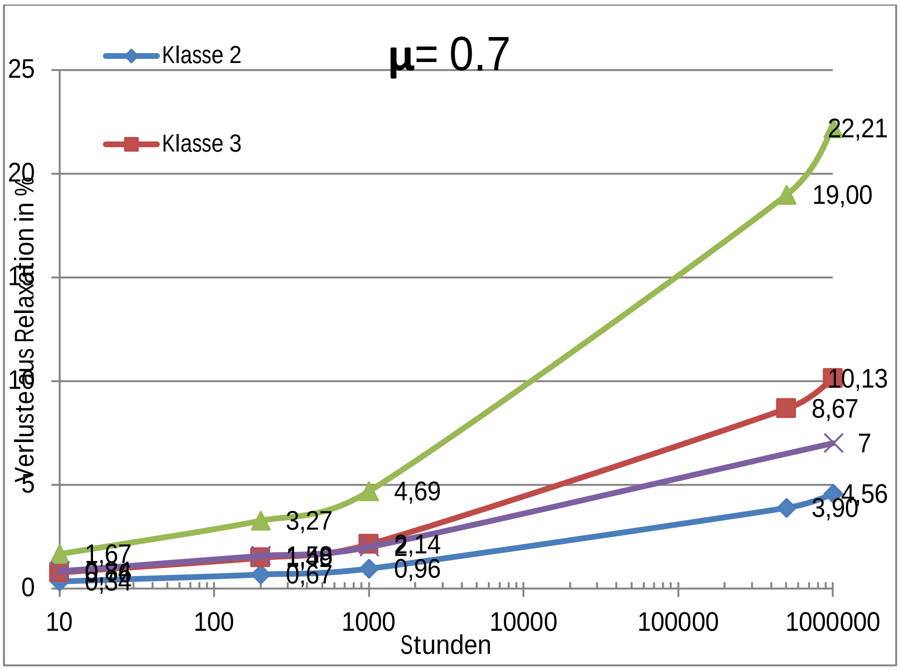
- Higher concrete age when releasing the anchoring (heat treatment, fast curing cements or specification of a later time)
- Earliest possible installation of the girder
- Selection of a higher concrete class
- Low reinforcing steel utilization (lower relaxation)

Prestressing steel relaxation

The relaxation losses are determined by time-dependent and load-dependent functions and by the selected prestressing steel.

EN2, NA_A, NA_PN, NA_GB:	equations as per 3.3.2 (6) in accordance with the relaxation class
NA_D:	logarithmic interpolation with tabular values specified in the approval

The percentage of stress losses *Δσpr/σpi* occurring on a prestressing steel with a very low relaxation as per 3.3.2(6) or as per approval are illustrated exemplary in the chart below for a loading rate *μ = σpi/ fpd= 0.7* (*σpi*: prestressing steel stress, *Δσpr*: stress change in the prestressing steel due to relaxation).

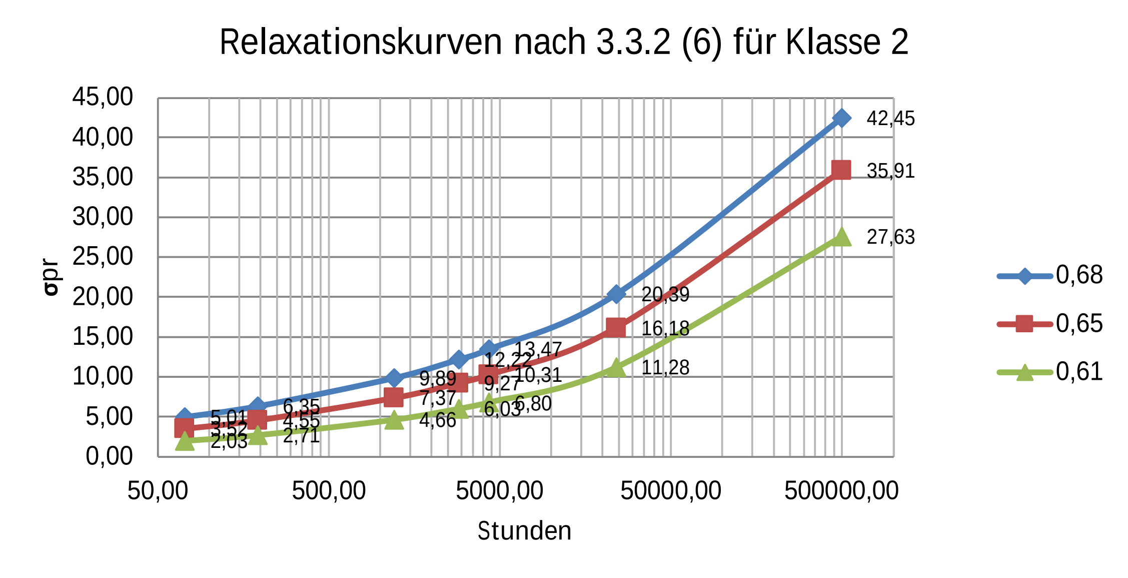


The loss due to short-term relaxation is determined by the loading rate resulting from the full prestress in the prestressing bed immediately before the release of the anchoring (t= tA,Lag).

The relaxation loss in the creep stage is determined with the help of the relaxation curve for the loading rate *μ= σpi/ fpk* at the beginning of the creep stage. *σpi* is the prestressing steel stress that is determined by all permanent loads acting at that time.

Δσpr is determined by the difference between the value on the curve for the end (t= tE) and the value on the curve for the beginning (t=tA) of the creep stage.

The chart below shows exemplary curves for relaxation class 2 for three loading rates as per 3.3.2 (6): the curve for the short-term relaxation with μ=0.68, the curve for the 'storage' creep stage with μ=0.65 and the curve for the 'usage' creep stage with μ=0.61.



Because the order of magnitude of relaxation losses is considerably smaller than that of losses due to creep and shrinkage, the quasi-permanent load portions of variable loads are only considered in the determination of *σpi*, if they do not have a load-relieving effect for the creep in total.

According to 3.3.2 (8), the final values of the relaxation losses (end of the 'usage' creep stage) may be calculated with *t* = 500,000 h.

If heat treatment is applied before releasing the anchoring, an equivalent period *teq* as per 10.3.2.1 is added to the times to be considered.

NA_D: In accordance with the *general approval* of the prestressing steel by the building authorities, the relaxation loss over the total service life is anticipated as short-term relaxation if a heat treatment is applied in the prestressing bed. If the prestress in the prestressing bed is smaller than 0.65 * *Rm* and smaller than 0.8 * *Rp,0.1*, you may anticipate a loss of 4 % for prestressing strands with very low relaxation.

Verifications of the load-bearing capacity

The analyses in the ultimate limit state include the following individual verifications:

- Verification of the bending resistance and verification of the resisting tensile force coverage
- Verification of the shear resistance (verifications of the stirrup reinforcement and the compression strut)
- Verification of the shear transfer in the composite joint
- Verification of the prestressing steel anchoring
- Verification of the tensile splitting reinforcement
- Verification of the lateral buckling stability

The verifications are performed in different design situations that depend on the applying actions:

- Permanent and transient design situation, abbreviated to PT
- Accidental design situation, abbreviated to A
- Accidental design situation earthquake (seismic situation), abbreviated to Ae

Depending on the design situation, different rules apply to the load combinatorics and different material parameters need to be defined. They are described in the following chapters.

Combination for the permanent and transient situation in the ultimate limit state

The combination is based on the STR limit state (failure of the load-bearing structure or excessive deformation)

NA_D, NA_A:	eq. 6.10
EN2, NA_PN, NA_GB	more unfavourable value as per 6.10a and 6.10b
	NA_GB: $\xi = 0.925$, otherwise $\xi = 0.85$

Permanent actions

NA_D: Partial safety factors γ_G as per DIN EN 1990/NA table NA.A.1.2(B)

EN2, NA_D, NA_PN, NA_A, NA_GB: Partial safety factors γ_G as per EN 1990 table A.1.2(B)

According to the interpretation of 2.4.2 in reference /52/, the simplifying regulation for permanent actions, which allows using the same γ_G value for the top and the bottom in all spans, is only applicable for a relation of the variable and permanent loads of $q/g > 0.2$ and for smaller cantilevers.

In other cases, the decisive combination must be sought after by applying the most unfavourable γ_G in each span. This rule is implemented as a standard in the software, the simplification described above can be selected optionally.

By activating the option ‘Do not combine permanent actions span-wise’ in the menu ‘Optional settings’, the combination is performed in accordance with the rule described above, which was applicable until recently and is now a user-defined special case.

Variable actions

NA_D: partial safety factors as per DIN EN 1990/NA tab.
combination coefficients as per DIN EN 1990/NA table NA.A.1.1

NA_GB: partial safety factors as per NA to BS EN 1990 , table NA.A1.2 (B),
combination coefficients as per DIN EN 1990, table NA.A1

EN2, NA_A, NA_PN: partial safety factors γ_Q as per EN 1990 table A.1.2(B)
combination coefficients as per EN 1990 table A.1.1

Variable actions with an unfavourable effect are included in the combination with characteristic values modified by the partial safety factor and the combination coefficient $\psi 0$.

In contrast to other variable actions, the dominant independent action (leading action) is not reduced by the corresponding combination coefficient.

When assigning another consequence class than CC2 (EN 1990 table B.1), the partial safety factors of the actions are modified via an adjustment factor KFI (EN 1990 Tab. B.3).

NA_D: according to the model list of technical construction regulations MLTB 9/2014, Annex B (KFI factors) must not be applied.

NA_GB: Annex B (KFI factors) must not be applied

If different actions due to imposed and/or live loads apply, they are treated by default as correlating actions, i.e. as a single action. The action with the greatest combination coefficient is decisive $\psi 0$ (cf. /41/ p.19, 28, 38). You can cancel the dependency in the design settings, if there is no correlation between these actions.

NA_D (NDP to A1.2.1(1) Note 2):

- If wind and snow are accompanying actions of a non-climatic action, you need only consider one of the two climatic actions in altitudes not higher than 1000 metres above MSL.
- If wind and snow is combined in regions of wind zone 3 or 4 and if wind is the leading action, snow can be dispensed with as an accompanying action.

Construction states:

NA_D: NCI to 10.2 allows the inclusion of γG= 1.15

Combination for the accidental situation in the ultimate limit state

EN 1990, eq. 6.11

NA_D:	partial safety factors as per DIN EN 1990/NA table NA.A.1.1
NA_GB:	combination coefficients in accordance with NA to BS EN 1990 table NA.A1
EN2, NA_PN, NA_A:	combination coefficients as EN 1990 table A.1.1

For each independent accidental action, a separate combination is to be examined. The accidental combination is to be included with its calculated value *Ad*.

In the current version of the software, you cannot consider several independent accidental actions in the same item. An accidental action can consist of several components, however.

Permanent loads are considered with their characteristic values.

The prevailing independent variable action is reduced by the combination coefficient $\psi 1$, all other variable actions are reduced by the combination coefficient $\psi 2$.

NA_A: Also the prevailing action is reduced by the combination coefficient $\psi 2$.

If different actions due to imposed and/or live loads apply, they are treated by default as correlating actions, i.e. as a single action. The action with the greatest combination coefficient is decisive $\psi 2$ (cf. /41/ p.19, 28, 38). You can cancel the dependency in the design settings, if there is no correlation between these actions.

NA_D:

In the current software version, it is always assumed that the accidental action is caused by a vehicle impact or an explosion.
According to NDP to A1.3.2, you may use the combination coefficient $\psi_{2,1}$ instead of $\psi_{1,1}$ in the equations 6.11 in this case.

The construction regulations of some German federal states, mainly in Northern Germany, require the consideration of an accidental snow load in addition to the normal snow load. In the current version of the software, you cannot map this correctly unless you calculate two separate items.

Combination for the seismic situation in the ultimate limit state

EN 1990, eq. 6.11

NA_D:	partial safety factors as per DIN EN 1990/NA table NA.A.1.1
NA_GB:	combination coefficients in accordance with NA to BS EN 1990 table NA.A1
EN2, NA_PN, NA_A:	combination coefficients as EN 1990 table A.1.1

For each independent seismic action, a separate combination is to be examined. The action is to be included with its calculated value *Aed*.

In the current version of the software, you cannot consider several independent seismic actions in the same item. A seismic action can consist of several components, however.

Permanent loads are considered with their characteristic values.

Variable actions with an unfavourable effect are included in the combination with characteristic values reduced by the quasi-permanent coefficient $\psi 2$.

NA_D:

If the option 'accidental earthquake with snow' is ticked, the combination coefficient $\psi 2$= 0.5 instead of $\psi 2$= 0 is considered for snow. This is required by the Construction Codes of some federal states in Germany (Baden-Württemberg).

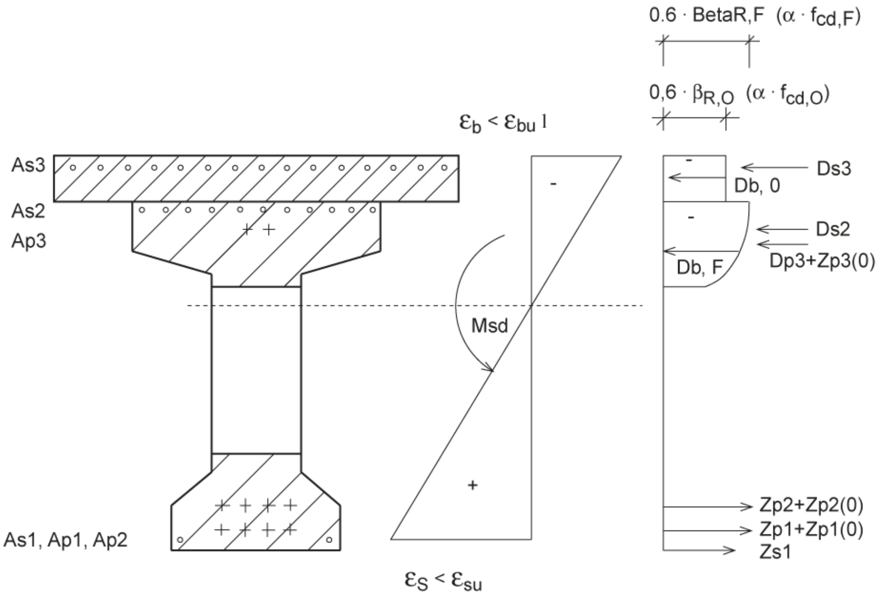
Bending with longitudinal force and resisting tensile force coverage

The resisting moments *MRd* are determined at the beginning and the end of each creep stage for a tension zone assumed on top or on bottom.

If prestress generates tension on top, the verification on bottom is performed at the end of the creep stage and the verification on top is performed at the beginning of the creep stage. In other cases, this is done the other way round.

MRd is compared to the calculated value of the decisive applying internal moment *MEd*. *MEd,max* becomes decisive for the verification on bottom and *MEd,min* for the verification on top.

The verification is always performed for the permanent and transient situations. If corresponding actions apply, the verification is also performed for the accidental and seismic situations.



Verification not successful

If *Eta* is not complied with, the cross section must be increased under normal conditions if the pressure zone fails. Otherwise, the tensile reinforcement must be increased.

Determination of the ultimate moment

The ultimate strains and compressive strains of the cross-section are determined by iteration with consideration of the limit strains as per 6.1 (6). The state is sought after in which the resultant force of the concrete and steel stresses is in equilibrium with the resultant forces of the tensile steel stresses and one of the following failure criteria is met:

Limit strain as per 6.1 (6), figure 6.1) 1

	EN2 NA_A, NA_PN, NA_GB	NA_D
Ultimate compressive concrete strain [‰] $\varepsilon c2$ and $\varepsilon cu2$	tab.3.1	= EN2
Limit steel strain [‰] εud	0.9 * εuk	25

The compressive concrete force is determined with the help of the internal action curve of the concrete used. Recesses are considered, if applicable.

Parabola rectangle stress chart as per figure 3.3

fcd: calculated value of the compressive concrete strength as per eq. 3.15 $fcd = \alpha cc \cdot fck/\gamma c$

For $t = tA,Lag$, the calculation is based on the early strength $fck(t)$

γc: partial safety factor of the concrete corresponding to the examined design situation (PT: permanent/transient, A: accidental, Ae: seismic (earthquake) PT and A as per 2.4.2.4 (1), Ae as per EN 1998-1 para. 5.42.4.(3))

αcc: coefficient to consider the long-term effect

εc2: limit strain for the transition from the parabolic to the rectangular area as per table 3.1

εcu2: limit strain as per table 3.1

n exponent n as per table 3.1

	EN2	NA_D	NA_A	NA_PN	NA_GB
γc(PT):	1.5	= EN2	= EN2	1.4	= EN2
γc(A):	1.2	1.3	= EN2	= EN2	= EN2
γc(Ae):	=γc(PT):	= EN2	1.3	= EN2	= EN2
αcc	1.0	0.85	= EN2	= EN2	0.85

If the precast component was manufactured in the factory, possible reduction of γc as per annex A

	A2.1 reduced geometric deviations due to control γc,Red1	A2.2 (1) measured or reduced geometric data γc,Red2	A2,2 (2) variation coefficient of concrete strength < 10 % γc,Red3	A2.3 concrete strength in the mixing plant determines the reductionη (γc,Red* η)	A2.3 Minimum γc (γc,Red4)
EN	1.4	1.45	1.35	0.85	1.30
NA_D	1.5	1.5	1.5	0.9	1.35
NA_GB	= EN	= EN	= EN	= EN	= EN
NA_A	= EN	= EN	= EN	= EN	= EN
NA_PN	1.35	Not allowed	Not allowed	Not allowed	1.35

Bi-linear internal action curve as per figure 3.8:

fyk: characteristic value of the yield point

ftk: characteristic value of the tensile strength (maximum value of the inclined branch at εuk)

εεuk characteristic strain under maximum load

γs: partial safety factor in accordance with the examined design situation
(PT: permanent/transient, A: accidental, Ae: seismic (earthquake)
PT and A as per 2.4.2.4 (1), Ae as per EN 1998-1 para. 5.2.4.(3))

fyd: design value of the yield strength
fyd= fyk/γs

ftd: design value of the maximum stress of the inclined upper branch at εuk
ftd= ftk/γs

εud: calculated ultimate strain as per 3.2.7 (2)

Es: modulus of elasticity of reinforced concrete,
as per 3.2.7 (4) is Es = 200,000 N/mm²

	EN2	NA_D	NA_A	NA_PN	NA_GB
εuk (o(oo)	Annex C:	= 25	= EN2	= EN2	= EN2
ftk	= (ft/fy)k*fyk	ftk,cal = 525 N/mm2	= EN2	= EN2	= EN2
γs(PT)	1.15	= EN2	= EN2	= EN2	= EN2
γs(A)	1.0	= EN2	= EN2	= EN2	= EN2
γs(Ae)	=γs(PT)	= EN2	1.0	= EN2	= EN2
εuk (o(oo)	0.9* εuk	25	= EN2	= EN2	= EN2

If the precast component was manufactured in the factory, possible reduction of γs as per annex A

	A2.1 reduced geometric deviations due to control γs,Red1	A2.2 (1) measured or diminished geometric data γc,Red2
NA_EN	1.10	1.05
NA_D	1.15	1.15
NA_GB	= EN2	= EN2
NA_A	= EN2	= EN2
NA_PN	= EN	= EN

Bi-linear internal action curve as per figure 3.10

fpk0,1k: characteristic value of the strain limit of 0.1 %

fpk: characteristic value of the tensile strength (maximum value of the inclined branch at *εuk*)

εεuk characteristic strain under maximum load

γs: partial safety factor in accordance with the examined design situation
(PT: permanent/transient, A: accidental, Ae: seismic (earthquake) PT and A as per 2.4.2.4 (1), Ae as per EN 1998-1 para. 5.42.4.(3))

fpd: design value of the stress at the beginning of the inclined upper branch
fpd= fpk0,1k/γs

fpk/γs: calculated value of the maximum stress of the inclined upper branch at εuk

εud: calculated ultimate strain as per 3.3.6 (7)

Ep : modulus of elasticity of the prestressing steel
as per 3.3.6 (3) 195,000 N/mm² for strands, 205,000 N/mm² for bars and wires

	EN2	NA_D	NA_A	NA_PN	NA_GB
fp0.1k	0.9* fpk	Approval	= EN2	= EN2	=0.88* fpk
fpk	EN 10138	Approval	ÖNORM B 4758	= EN2	BS 5896 [2012]
εuk	EN 10138	Approval	ÖNORM B 4758	= EN2	BS 5896 [2012]
γs(PT):	1.15	= EN2	= EN2	= EN2	= EN2
γs(A)	1.0	= EN2	= EN2	= EN2	= EN2
γs(Ae)	=γs(PT)	= EN2	= EN2	= EN2	= EN2
fpk/γs:	at εuk	at εud	= EN2	= EN2	= EN2
εuk (o(oo)	0.9* εuk	$\varepsilon_p^{(0)}$+25 o/oo < 0.9* εuk	= EN2	= EN2	= EN2

The effect of the effective prestress at the time of examination is considered as pre-strain of the reinforcement. According to 5.10.8 (1) the calculated value of the prestress $P_{d,t} = \gamma_p \cdot P_{m,t}$ is to be used in the verifications in the SLS.

EN2,NA_D, NA_A, NA_PN: $\gamma_{p,fav}$ = 1.0 $\gamma_{p,unfav}$ = 1.0

NA_GB: $\gamma_{p,fav}$ = 0.9 $\gamma_{p,unfav}$ = 1.1

The size and the location of the resultant tensile and compression forces are determined for the failure state to be examined. The resisting ultimate moment is the product of the resultant tensile force and the distance of the resultant tensile force to resultant compressive force.

‣ See the output example: Bending with longitudinal force in the ULS

Specialities regarding cast-in-place complements

In verifications that are performed on cross-sections with cast-in-place complements after the creation of the bond (end of the creep stage “casting of cast-in-place complement”), the cross-section is assumed as being complete right from the beginning (EN2: 10.9.3 (8)), the specific internal action curve of the cast-in-place concrete is considered.

Resisting tensile force coverage

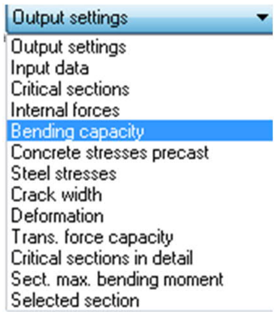
The tensile force diagram of each cross-section is the result of *TEd(x) = MEd(x)/z(x)*. The resisting tensile force diagram *Td(x)* is determined by displacing the tensile force diagram by the offset dimension in the more unfavourable direction in each case.

Td(x) at the grid points is determined by linear interpolation inside the displaced polygonal chain. The resisting tensile force is determined by the relation *TRd(x)= MRd(x)/z(x).*

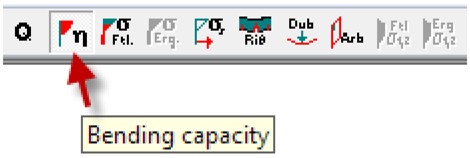
Each verification is to be performed in the ultimate limit state. Because of the linear-elastic determination of the internal forces, you can dispense with a verification in the serviceability limit state.

In accordance with 9.2.1.3(2), the offset dimension *al* is determined by the compression strut angle θ for vertical stirrups (shear force resistance verification) and the lever arm *z* of the internal force (verification of bending with longitudinal force) and is expressed by the following relation $al(x) = z/2 \cdot \cot \theta$. At the same time, the condition $al >= 0.5 \cdot d$ applies as recommended in reference /27/ p. 720.

The tensile force coverage can be displayed in the form of a table



or graphically.



In the tabular representation, the safety condition *Eta = TRd(x) / Td(x)*, the offset dimension and the reason why a verification is required are displayed.

As per 8.10.2.2, the maximum tensile force of the tendons is limited in the anchoring area *lbpd* as shown in figure 8.17DE.

It may happen that resisting tensile force coverage can only be achieved by adding slag reinforcement or by increasing the projection.

Detailed information about the anchoring area is displayed in the list selection of the text view.

The verification is successful, when the following condition is true: *Eta = TRd(x) / Td(x)* > 1.0.

Shear resistance

The verification is performed at the beginning and the end of each creep stage for the maximum shear force and the associated moment as well as for the maximum moment and the associated shear force. The shear reinforcement is calculated for vertical stirrups.

In the current version of the software, you cannot verify sections in the area of recesses.

The design of the shear reinforcement is based on the method using a variable compression strut inclination. Moreover, the load-bearing capacity of the compression struts is to be verified.

VEd0: calculated value of the shear force applied by external loads for the corresponding design situation.

VEd: design value of the shear force (6.2.1 (1), (2), (3), (5))

‣ See also reference /54/ eq.7.99b and NA_D NCI to 6.2.1 (1))

VEd = VEd0 - Vpd - Vccd - Vtd

Vpd: component with inclined tendons (*dV* from *pd*),

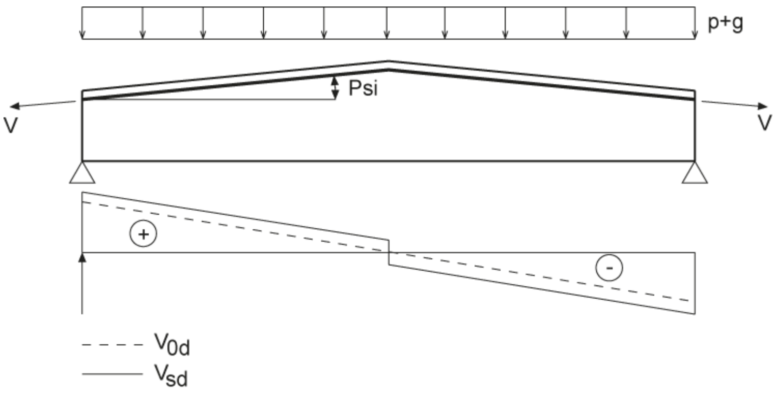
Vpd = -sin(Psi) · Fpd

Psi: angle between the tendon and the horizontal axis

Fpd: calculated value of prestress

Fpd = Pmt < = Ap · fp0,1k /Gams /31/ p.338 eq. 4.56

Pmt: average value at the time of examination

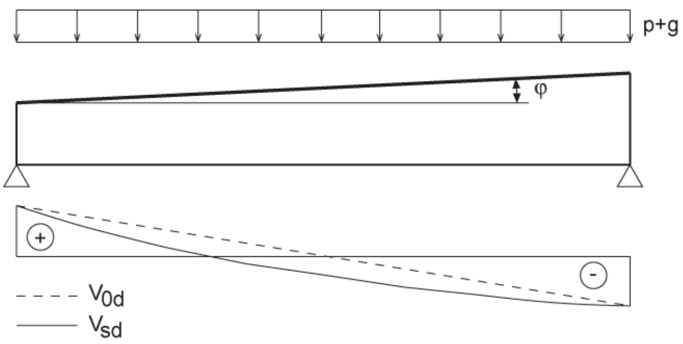


With variable cross-section height (*dV* from *cc*):

Vtd: component with inclined bottom chord

Vtd = 0, as the bottom chord is always horizontal

Vccd: component with inclined top chord
Vccd = Myd · tan φ/zII
Myd: associated moment
φ: inclination of the top chord in relation to the horizontal axis
zII: lever arm of the internal forces
‣ see Bending with longitudinal force



Sign convention: reducing, when *z* and |*Myd*| increase or decrease simultaneously.

VEd,Red shear force portions of concentrated loads applying at a distance to the support edge *av* < 2.0 · *d* may be reduced by *ß= max(av*, 0.5 · *d*)/(2 * *d*) if a direct support was defined in accordance with 6.2.3 (8).

VRd,c shear force resistance without reinforcement, determined with equation 6.2a+b.

Crucial parameters are:

- concrete strength,
- longitudinal reinforcement ratio ρl of the tensile reinforcement extended beyond the area in which reinforcement is required by the dimension *lbd+d*
- concrete stress σcp caused by longitudinal forces in the centre of gravity
- effective height d and scale factor *k* for the component height
- smallest cross-section width inside the tension zone *bw*
- pre-factor CRdc in accordance with the examined design situation

	EN2	NA_D	NA_A	NA_PN	NA_GB
CRdc	0,18 / γc	0,15 / γc	= EN2	= EN2	= EN2 > C50 test or as C50
K1	0.15	0.12	= EN2	= EN2	= EN2
νmin d<=600 mm d > 800 mm	$0.035 * k^{3/2} \cdot fck^{1/2}$	$(0.0525 / \gamma_C) * k^{3/2} \cdot fck^{1/2}$ $(0.0375 / \gamma_C) * k^{3/2} \cdot fck^{1/2}$	= EN2	= EN2	= EN2

For *CRdc* and *νmi*', the partial safety factors of the examined design situation are used (see the paragraph 'Bending with longitudinal force').

Cot Θ inclination angle of the compression strut

The best possible stirrup reinforcement results when selecting the greatest possible value, at which the compression strut is still resisting.

- Minimum and maximum (NDP)

EN2, NA_GB: $1 \le \cot \theta \le 2.5$

NA_PN: $1 \le \cot \theta \le 2.0$

NA_D: as per eq. 6.7aDE, $0.58 \le \cot \theta \le 3.0$

NA_A: 0.6... 0.4 (fyd ≤ σsd ≤ 0) ≤ tan θ ≤ 1.0
0.4 also applies when the tensile reinforcement is constant over the total length of the girder.

asw if the design value *VEd,Red* is greater than *VRdc* (6.2.1 (5) and 6.2.2(6)), a shear reinforcement *asw* is calculated as per equation 6.8 1 that meets the condition *VRd,s* = *VEd,Red*. Otherwise, the software checks whether a minimum shear reinforcement for beams as per 9.2.2 or for slabs as per 9.3.3 is required.

The decisive area for the determination of the shear reinforcement ends at a distance *d* to the support edge in accordance with 6.2.1 (8) if direct supports have been defined and if the loads are uniformly distributed. In other cases, this area ends at the edge of the support. If any concentrated load applies between the edge of the support and the area border, the area border is displaced to the concentrated load that has the lowest distance to the support edge.
For the design value of the yield strength of the stirrups, the partial safety factors of the examined design situation are to be used (see the paragraph Bending with longitudinal force)

z: lever arm of the internal forces, is determined in the verification of the bending resistance.

NA_D: NCI: z < d -2 · cv or z < d - cv - 3.0 cm

The user can optionally disable this condition in the design settings to avoid very small cantilevers in combination with thin slabs.

Minimum shear reinforcement min. asw/s= $\rho \cdot bw \cdot \sin \alpha$

	ρ (beams) as per 9.2.2:	ρ (slabs) as per 9.3.2:	Comment
EN	0.08 · √fck/fyk	0	
NA-D:	0.16 · fctm/fyk Prestressed tension chord of flanged cross-sections: 0.256 fctm/fyk	0 if VEd < VRdc Otherwise 0.6 *ρ	Junction area 4 < b/h < 5: Interpolation between 0 and the simple value (VEd < VRdc) or between 0.6 and the simple value (VEd > VRdc)
NA-GB	= EN	= EN	
NA-A	0.15 · fctm/fyd	= EN	
NA-PN	= EN	= EN	

VRd,max the compression strut resistance is determined as per equation 6.9.

VRd,max = αcw · bw · z · v1 · fcd / (cot θ + tan θ)

acw: EN2, NA_A, NA_PN, NA_GB:

αcw =1 not prestressed

αcw =(1 + σcp / fcd) for 0 < σcp ≤ 0.25fcd

αcw =1.25 for 0.25fcd < σcp ≤ 0.5fcd

αcw =2.5 · (1 - σcp / fcd) for 0.5fcd < σcp < 1.0fcd

NA_D: αcw =1.0

v1: EN2, NA_A, NA_PN, NA_GB: v1= 0,6* (1- fck/250) eq. 6.6N 6.6N

NA_D: v1 = 0,75 · v2

1 = 1.1 - fck/500 > 1.0

bw: slowest width in the cross-section as shown in figure 6.5.

fcd design value of the compressive strength of the concrete as per equation 3.15, see the chapter Bending with longitudinal force...,

For *t= tA,Lag*, *fcd* is calculated with the early strength *fck(t).*

NA_GB: fcd as per PD 6687:2006 with α_{cc}=1.0

In accordance with 6.2.3 (8), the verification is performed at the support edge and considers the shear force *VEd* without reduction (6.2.1 (8)). The resistance of the compression strut increases with the steepness of the strut angle and reaches its maximum at θ=45 degrees.

In the detailed output for a single section, the mentioned intermediate results are put out for each creep stage at its beginning and its end.

Moreover, you can display the layout of the shear reinforcement and the compression strut resistance in the form of tables or graphically.

Verification of the shear force transfer in the joint as per DIN EN 1992-1-1/NA

vEd <= vRdi

vEd shear force to be transferred per length unit in the joint

vEd = ß · VEd / (z · bi) eq 6.24

VEd: design value of the shear force

z: lever arm of the internal forces ▸ see Verification of the shear resistance

NA_D: If *VRd,c* > *VEd*, the lever arm limitation by *cv* is dispensed with.

ß: relation of axial force in the cast-in-place concrete to the total compressive force. The software always assumes a value of 1.0 on the safe side.

bi: effective joint width, reduced total width due to prefabricated formwork, if applicable.

vRdi design value of the shear force resistance of the joint as per eq. 6.25

vRdi = c · fctd + μ · σn + ρ · fyd · (μ · sin α + cos α) < 0.5 · ν · fcd

NA_D:

vRdi = c · fctd + μ · σn + ρ · fyd · (1.2 · μ · sin α + cos α) < 0.5 · ν · fcd

σn axial stress perpendicular to the joint with σn = < 0.6 · fcd
tension is negative

c roughness factor in accordance with the surface roughness as per 6.2.5 (2)

If σn is tension than c= 0

c	Very smooth	Smooth	Rough	Interlocked
	0,025 NA_D: 0	0.20	0.40	0.50

fctd design value of the compressive strength as per eq. 3.16

$f_{ctd} = \alpha_{ct} \cdot f_{ctk;0,05} / \gamma_c$

$\alpha_{ct:}$ EN2, NA_A, NA_PN, NA_GB: 1.0
NA_D: 0.85

$f_{ctk;0,05}$ characteristic tensile strength as per table 3.1, 5% quantile

γ_c partial safety factor,
see the chapter Bending with longitudinal force...

μ · σn portion from axial force normal to the joint, is not considered by the software

μ friction coefficient in accordance with the surface roughness

as per 6.2.5 (2)

μ	Very smooth	Smooth	Rough	Interlocked
	0.5	0.6	0.7	0.9

ν · reduction of the joint roughness as per 6.2.2 (6)

ν	Very smooth	Smooth	Rough	Interlocked
EN2	v = 0.6(1 – fck / 250)			
NA_D	0	0.2	0.5	0.70
	>C50/60: v = v · (1.1 – fck / 500)			

fcd analogous to *VRd,max*

As *vEd* = *vRdi* and ρ= *As/Ai* (*Ai* is the area of the joint) is used to calculate the required reinforcement quantity.

NA_A: *As* results with *fyd*, a verification of the anchors is not performed.

If a calculated required reinforcement results, the software checks whether a minimum reinforcement is required.

For the design values of the compressive and tensile strength of the concrete as well as for the yield strength of the shear reinforcement, the partial safety factors of the examined design situation are to be used (see the paragraph Bending with longitudinal force)

Verification of the lateral buckling stability

The lateral buckling stability in the installed state can be verified with methods described by Stiglat /16/ or Mann /17/. In addition to this, the lateral buckling stability can be verified for the erection with a lifting beam and/or a suspension gear.

Reference /4/ 1.3 allows to change over to the global safety concept and this concept is used here ▸ see also example 5 in reference /9/ and /58/.

The existing maximum moment is determined with the characteristic values of the actions without consideration of load factors. If accidental actions or seismic actions apply, they are considered with their full value.

The summary safety factors of the respective method apply.

Cast-in-place complement

If a cast-in-place complement is added subsequently to a cross-section, the state of the prefabricated component during the casting of the cast-in-place concrete is examined.

In combination with a cast-in-place complement of the ‘solid slab’ type, the state after the creation of the bond is not examined, because the slab increases the lateral buckling stability.

In other cases, it is assumed in the verification after the creation of the bond that the composite cross-section for all loads has existed right from the beginning. The material parameters are weighted in accordance with the respective area portions.

Cantilevers

The common stabilizing effect of cantilevers with respect to the lateral buckling moment cannot be considered in the current version of the software. You should use the software BTII (Second-order buckling torsion analysis) to determine the ideal lateral buckling moment.

Comparison to the methods described by Mann and Stiglat

The verification based on the method described by Stiglat normally delivers greater referenced lateral buckling stabilities than the verification method described by Mann. However, it may happen that the lateral buckling stability is provided in accordance with Stiglat but not in accordance with Mann. For fully or partially prestressed girders, the method described by Stiglat produces results that have been proven by tests (/16/) and turned out to be on the safe side. If the initial imperfections are not extraordinarily great, lateral buckling safety proven in accordance with Stiglat will be sufficient.

In combination with untensioned reinforcement or weakly prestressed girders, we recommend using the method described by Mann.

Lateral buckling stability verification in the installed state in accordance with Stiglat

▸ See reference /14/ and /16/ and /36/

The method described by Stiglat is based on the lateral buckling analysis known from the theory of elasticity. It is adjusted to the concrete via the load-bearing stresses of a compression member with the same comparison slenderness.

The influence of the reinforcement is neglected. Because initial imperfections cannot be considered, an increased safety with a factor of 2.0 is required.

To consider a possibly existing state II in verifications as per EN2/ DIN 1045-1, only 60 % of the torsional moment of inertia are considered in the calculation, if *fctk0.05* is exceeded in the infrequent load combination.

The assumptions the method is based on (parabolic moment behaviour, load application at the top edge of the girder, fork support, centre of gravity = shear centre) are on the safe side under normal conditions.

Explanations concerning the output data

hc: distance between the centres of gravity of the compression chord and the tension chord

Beta1: auxiliary value for the calculation of *k2*

Beta2: auxiliary value for the calculation of *k3*

k1: factor to consider the supporting conditions and the moment behaviour (assumption in the software: fork support and parabolic moment behaviour, *k1* = 3.54)

k2: factor to consider flange bending (warping resistance), approx. 1.0

k3: coefficient to consider the load application height above the shear centre, *k3* < 1 if the load applies above *M* (assumption in the software: load application at the girder top edge, shear centre = centre of gravity)

AK: intermediate value in the calculation of the elastic lateral buckling moment

AK = Eb · Iy · Gb · It · Ix / (Ix Iy)

The stiffnesses for girders with variable height are averaged in accordance with Rafla /15/ .

Gb: shear modulus of the concrete, corresponds to 0.4 · Eb

It: torsional moment of inertia

MK: lateral buckling moment with ideally elastic material behaviour

$$MK = k1 \cdot k2 \cdot k3 \cdot \frac{\sqrt{AK}}{L}$$

Wxo: section modulus of the compression chord at the cross-section at the distance *x*

x: distance of the cross-section with the highest concrete stresses due to external loading including the self-weight of the girder beginning

SigmaB: calculated concrete stress at the compression edge *MK'/Wxo; MK'* results from *MK* when assuming a behaviour of *MK* affine to the internal moment function.

LambdaV: comparison slenderness of the buckling girder

$$LambdaV = \frac{Pi}{\sqrt{Eb/SigmaB}}$$

For cross-sections with cast-in-place complement, the concrete class is averaged in accordance with the portions of the pre-cast concrete and the cast-in-place concrete.

SigmaT: buckling stress of a compression member with pinned supports on both sides and with the same comparison slenderness with consideration of the non-linear stress-strain relation (equation 3.14, tangent modulus) of the concrete. For cross-sections with cast-in-place complement, the concrete class is averaged in accordance with the portions of the pre-cast concrete and the cast-in-place concrete (*FakBN*: factor for pre-cast concrete).

Mkipp: calculated lateral buckling moment of the reinforced concrete girder

Mkipp = MK · SigmaT / SigmaB

Required lateral buckling stability:

Eta = Mkipp / Mvor > 2.0

Mvor: maximum span moment

Improvement of the lateral buckling stability

Enlarging the cross section, especially the compression chord, can increase the resisting lateral buckling moment.

Verification of the lateral buckling stability in the installed state in accordance with Mann (/17/ and /18/, required specifications on the “Lateral buckling” tab)

The verification of the lateral buckling stability is based on the flexural buckling of a compression chord idealized to an equivalent cross-section.

The increased stiffnesses in comparison to reinforced concrete caused by prestressing are not considered. The influence of initial imperfections and of the reinforcement can be taken into account, however. As the procedure described by Mann assumes a constant distance between the tension chord and the compression chord over the total length of the girder, it can be used for girders with a variable height only under restricted conditions.

The assumptions the method is based on (parabolic moment behaviour, fork support) are on the safe side under normal conditions.

Explanations concerning the output data

EpsB: compressive stress on the compression edge

EpsS: strain at the height of the centre of gravity of the tensile reinforcement

EpsB and EpsS are determined by iteration via the equilibrium of the internal forces.

heff: distance of the centre of gravity of the tensile reinforcement from the compression edge in the ridge

xconcrete: height of the compression zone under the determined strain conditions

Itconcrete: torsional moment of inertia in the compression zone of the concrete

IyConcrete: moment of inertia in the compression zone of the concrete about the vertical axis

xconcrete’: height of the equivalent compression zone under the assumption of constant stresses.

Aconcrete': area of the equivalent compression zone = equivalent compression chord

z': lever arm of the internal forces

kapa: auxiliary value:

Iy': moment of inertia of the equivalent compression chord about the vertical axis

Lamda': slenderness of the equivalent compression chord

b': width of the compression chord

ev': ideal eccentricity of the compressive force due to the initial imperfection of the equivalent chord

mo': referenced ideal eccentricity of the compressive force

BetaR': load-bearing stresses on the ideal equivalent chord (/19/)

EN 2: the concrete is assigned to a concrete class as per DIN 4227 which specifies this rated strength as a maximum

Db: flexural buckling load on the ideal equivalent chord = resultant force of the compressive concrete stresses at which the girder starts buckling.

SysFaktor: system factor for the consideration of a variable girder height

myli = myre: reinforcement ratio of the ideal equivalent chord (*As,k / Aconcrete'*)

Mkipp: calculated lateral buckling moment of the girder *Mkipp = Db · z'*

Required lateral buckling stability:

Eta = Mkipp/Mvor > 1.75

Mvor: maximum span moment

Improvement of the lateral buckling stability

How to correct the defined parameters for the verification described by Mann:

The initial imperfections at the top and the bottom side should be in approximately the same order of magnitude because an inclination of the girder reduces the lateral buckling stability considerably.

An increase of the 'lateral buckling reinforcement' is reasonable, especially, when the compressive concrete zone is fully loaded due to a compressive strain *EpsB* of 0.35%. The stiffness of the compression chord is to be increased.

Data entered for the structural system (exit the verification of the lateral buckling stability):

An increase of the reinforcement on bottom is only reasonable when the steel strain *Eps* has reached 0.5 %. Due to the lower position of the neutral axis, the compression chord becomes larger and, therefore, also stiffer.

Lateral buckling stability verification in the erection state in accordance with Stiglat

The ideal elastic buckling moment is modified for the suspended beam by an auxiliary value in accordance with Lebelle. This value depends on the horizontal and vertical position of the suspension points. The non-linear concrete behaviour is considered in the erection state in accordance with the method described by Stiglat.

The assumptions the method is based on (parabolic moment behaviour, load application at the centre of gravity = shear centre) are on the safe side under normal conditions.

Due to an inclined suspension position, the girder is loaded by axial forces and bending moments in addition to bending moments caused by self-weight.

The influence of this loading combination is estimated by applying the formula specified by Dunlerley.

The same cable angles are assumed on the left and the right. This is not the case with asymmetric girders or an asymmetric layout of the suspension points. There are different cable angles on the left and the right that provide for the equilibrium of the horizontal components of the cable forces.

Explanations concerning the output on the screen

With lifting beam:

beta4, delta, gamma: auxiliary values for the calculation of *qk1*

f:	vertical distance between the suspension points and the centre of gravity of the entire girder
p:	distance of the suspension eyes/girder length
j(alpha):	auxiliary values for the calculation of *qk1*
qk1:	line load under which the girder starts to buckle

AK, Wxo, x, SigmaB, SigmaT, LamdaV

MK:	moment due to qk1 at the point *x*
Mkipp:	calculated lateral buckling moment

Mkipp = MK · SigmaT / SigmaB

Mvor:	maximum span moment

Required lateral buckling stability: Eta = Mkipp / Mvor > 2.5

With rope gear:

Nk2:	buckling load if only axial forces apply
Mk3:	buckling moment if only moments apply
qk:	line load under which the girder starts buckling due to a combined loading caused by *qk, Mk* and *Nk*
Nk:	axial force due to *qk* (inclined suspension position)

MK: moment due to *qk* at the point *x*

Ab: cross-sectional area of the girder at the critical section

AK, Wxo, x, SigmaB, SigmaT, LamdaV

Mkipp: calculated lateral buckling moment

Mkipp = MK · SigmaT / SigmaB

Mvor: maximum span moment + additional moment due to the cable force

Required lateral buckling stability: Eta = Mkipp/Mvor > 2.5

Improvement of the lateral buckling stability

How to correct the data entered for the erection state:

Place the suspension point at a higher level

Increase the angle of the erecting cable

Data entered for the structural system (exit the verification of the lateral buckling stability):

Displace the suspension eyes in the direction of the quarter points of the girder length.

Modify the cross-section geometry (enlarge the compression chord)

Determination of the tensile splitting reinforcement

Tendons with the same stripping are summarized to a force application area. For each area, a tensile splitting reinforcement is calculated as per reference /10/ p. 666, 11.2 for a final anchoring by bond.

The concrete stresses and reinforcing steel stresses are calculated at the end of the force application area (force application length l_{disp} as per equation 8.19). In this calculation, the cross-section properties are included without consideration of existing recesses.

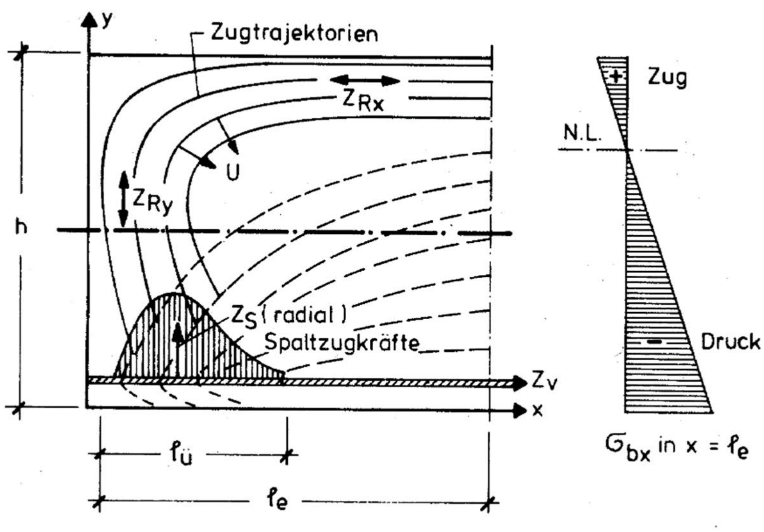
The resultant of the prestressing steel stresses Np and the resultant of the concrete stresses Nc are formed at the end of the cross-section below the uppermost tendon layer of the force application area. Multiplying the shear force T= Np- Nc by the factor k results in the splitting tensile force.

The factor *k* allows you to consider whether the tensile force applies to the edge (*k*=1/3) or in the middle (*k* = ½).

In accordance with the location of the centre of gravity of the prestressing steel layer of the considered area, the factor *k* is determined by interpolation.

If another load application area exists in front of the current area when looking from the girder end, the concrete force and the reinforcing steel force are determined by the growth of the respective resultant for the prestressing steel and the concrete.

If prestressing steels are located at the top of the cross-section, the splitting tensile force is determined from above.



Prestress is to be considered with its calculated value and the factor $\gamma_{p,unfav}$ is to be applied for local effects in accordance with paragraph 2.4.2.2 (3).

The tensile splitting reinforcement is obtained by dividing the result by the calculated value of the reinforcing steel stress *fyd*.

EN 2, NA_PN, NA_A: $\gamma_{p,unfav}$ =1.2

NA_D: $\gamma_{p,unfav}$ =1.35 (NCI to 2.4.2.2. (3)).

The tensile splitting reinforcement is to be laid in over a shortened application length (strands ¾* l_{disp} or bars 0.5 * l_{disp}).

Anchoring of the pre-stressing reinforcement

The anchoring length is obtained in accordance with figure 8.17 first from the transfer length *lpt2* where *σpm∞*, the full prestress minus all tensile force losses due to creep, shrinkage and relaxation is reached, an additional area up to full utilization of the prestressing steel strength $\sigma pd = fpk/\gamma s$.

lbpd = lpt2 + α2 φ · (σpd – σpm∞) / fbpd equation 8.21

σpm∞: prestressing steel stress obtained from the effective prestress for *t = infinite* at *x=lpt2*

The distance of the first bending crack *lr* is obtained when the tensile concrete stresses (tensile edge stress σ_R and/or main tensile stress σI) exceed the tensile concrete strength *fctk0.05* under loading in the ULS.

If the crack is inside the anchoring length (*lr* < *lbpd*), the anchors are to be verified.

The anchoring area of the prestressing steel is examined on a grid of 30 cm and the anchoring is verified via the tensile force coverage.

The resisting tensile force *TEd* on the cross-section x is determined via the expression *TEd= MEd/z* on the cross-section displaced by the offset dimension; *MEd* is determined in line with the design situation and the lever arm *z* is obtained in the verification of the bending strength.

The possible maximum tensile force in the prestressing steel is obtained in accordance with figure 8.17. It reaches the calculated strength of the prestressing steel only at the distance l_{bpd}.

NA_D:
If the crack occurs inside the transfer length (lr < lpt2), l_{bpd} is to be determined via equation NA.8.21.1 and the maximum possible tensile force in the prestressing steel as per figure 8.17DE.

lbpd = lr + α2 · φ · [σpd – σpt(x = lr)] / fbpd equation 8.21.1

σpt(x = lr): prestressing steel stress obtained from the effective prestress for *t = infinite* at *x=lr*

Due to the less steep run of the σp-line and the increased length of l_{bpd}, lower values are obtained for the maximum possible tensile force in the prestressing steel. The value $\sigma pt(x=lr)$ is the stress in the prestressing steel due to the prestress at the time $t = \propto$ at the point *x= lr*.

The design value of the transfer length *lpt2* included in the anchoring length l_{bpd} depends on the coefficient $\eta p1$ for the kind of prestressing steel, the coefficient $\eta 1$ for the bond conditions, the tensile concrete strength at the time of the cancellation of the tensile force *fctd(t)*, the coefficient $\alpha 1$ for the type of tensile force application, the coefficient $\alpha 2$ for the shape of the prestressing steel as well on the prestressing steel diameter ϕ.

$lpt2 = 1.2 \cdot lpt$ equation 8.18

$lpt = \alpha1 \cdot \alpha2 \cdot \varphi \cdot \sigma pm0 / fbpt$ equation 8.16

$\alpha1$:	gradual application of the prestressing force	1.0;
	sudden application of the tensile force	1.25
$\alpha2$:	prestressing steel; round bar	0.25;
	strand	0.19
$\sigma pm0$:	tensile steel stress due to the prestress directly after applying the prestressing force	

$fbpt = \eta p1 \cdot \eta1 \cdot fctd(t)$ equation 8.15

$\eta1$:	good bond	1.0
	otherwise	0.7
	The bond condition is determined for all prestressing steels according to 8.4.2(2) - Figure 8.2.	
$\eta p1$:	profiled wires	2.7
	strands	3.2
	NA_D:	2.85

$fctd(t) = \alpha ct \cdot 0.7 \cdot fctm(t)/\gamma c$

EN2, NA_A, NA_PN, NA_GB:	$\alpha ct = 1.0$	fctm(t) as per equation 3.4
NA_D:	$\alpha ct = 0.85$	$fctm(t) = 0{,}3 \cdot fcm(t)^{2/3}$ as per /56/ p. 324

The bond stress *fbpd* included in the anchoring length l_{bpd} depends on the tensile concrete strength *fctd* (equation 3.16), the coefficient $\eta p2$ for the type of prestressing steel and the coefficient $\eta 1$ for the bond conditions.

$fbpd = \eta p2 \cdot \eta1 \cdot fctd$ equation 8.20

$\eta p1$:	profiled wires	1.4
	strands	1.2
NA_D:	$\eta p2 = 1.4$	
	fbpd applies only if $Ap \leq 100\ mm^2$	

Improved anchoring:

- addition of untensioned reinforcement in the anchoring area
- creation of more favourable conditions in the anchoring area
  - e.g. less stripping, longer projection
  - higher concrete strength at the transfer of the prestressing force
  - elimination of causes for cracking in the anchoring area e.g. support reinforcement

Minimum reinforcement for longitudinal tension

The reinforcement to be calculated as per 9.2.1.1 is intended to prevent failure without signs caused by the failure of tendons and corresponds to the reinforcement for the absorption of the crack moment of the untensioned cross-section.

EN 2, NA_A, NA_PN, NA_GB: As,min = 0,26 · fctm/fyk · bt · d > 0.0013 bt · dequation 9.1N

NA_D:

As,min = McR / (z · fyk) - Ap'

McR = fctm · Wb crack moment

Wb: resisting moment on the tension side

fctm: average tensile strength of the concrete as per table 9

fyk: characteristic value of the yield strength

z: lever arm, approximation z= 0.9 · d
d: effective height, reinforced concrete reinforcement and accountable prestressing reinforcement in the tension zone with consideration of the distance of the centre of gravity

Ap': accountable prestressing reinforcement (at least two tendons)
up to a third of the existing prestressing reinforcement the distance of which to the reinforcing steel layer is smaller than 250 mm and/or smaller than 0.2 · h.

Verification of recesses

The verification of the recesses is carried out either according to DAfStb Booklet 399 in combination with recommendations from Leonhardt (/21/ Chapter 9.12) and engineering assumptions or alternatively according to DAfStb Booklet 599.

The same verification is carried out for round and rectangular recesses.

The points in time at which use begins (tA = after application of external loads, but possibly without any additional loads) and the point in time at which use ends (tE) are examined. Construction and assembly states are currently not taken into account when dimensioning the recess!

At both points in time, the load case combinations at maximum and minimum moment as well as at maximum shear force with positive and negative moment are considered in the middle of the recess.

This results in the following 8 load combinations to be investigated:

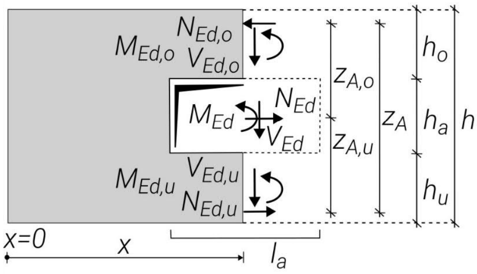
LC1 – N(tA), maxM, tensV	LC5 – N(tE), maxM, tensV
LC2 – N(tA), max|V|/tensM	LC6 – N(tE), max|V|/tensM
LC3 – N(tA), minM, tensV	LC7 – N(tE), minM, tensV
LC4 – N(tA), max|V|/tens(M<0)	LC8 – N(tE), max|V|/tens(M<0)

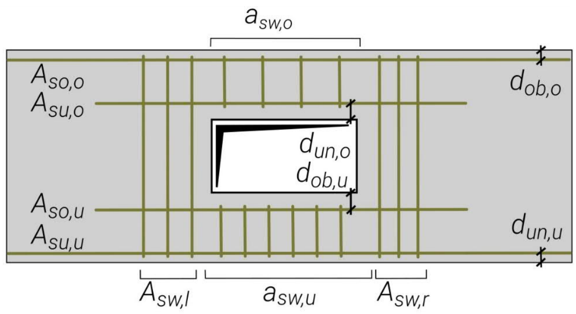
Hints:

- *To ensure the most compressed output possible, only the relevant LC are output. To see partial results, select "Outputs with intermediate results" in the "output settings"*
- *To better understand the internal forces, please place a user-defined section in the axis of the recess and select "sel. sections" in the output section*
- *In the printout log, notes are marked with a "~" and errors with a "#"*

The internal forces on the overall cross-section are divided into a resulting normal force, bending and shear force component in the top and bottom chords based on a strut-and-tie model. The chords must be verified for bending with tension/compression as well as for shear. In addition, the required suspension reinforcement to the left and right of the recess must be determined.

The basic reinforcement of the cross-section is taken into account for the calculations on the overall cross-section (table of design internal forces: compression zone height x_0, static effective height d and internal lever arm z_A (booklet 599)). The upper and lower reinforcement layers are determined from the center of gravity of the specified reinforcing steel and prestressing steel reinforcement of the overall cross-section.





Design of top and bottom chords

The resulting internal forces in the top and bottom chords are calculated according to DAfStb-H. 399 and 599 (considering the special features listed) as follows:

Compressive force in the top chord: $N_{Ed,O} = N_{Ed} \cdot \frac{z_{A,u}}{z_A} - \frac{M_{Ed}}{z_A}$

Tensile force in the bottom chord: $N_{Ed,U} = N_{Ed} \cdot \frac{z_{A,o}}{z_A} - \frac{M_{Ed}}{z_A}$

Shear force component in the top chord: $V_{Ed,O} = V_{Ed} \cdot \frac{\alpha_O \cdot I_O}{\alpha_O \cdot I_O + \alpha_U \cdot I_U}$

Shear force component in the bottom chord: $V_{Ed,U} = V_{Ed} \cdot \frac{\alpha_U \cdot I_U}{\alpha_O \cdot I_O + \alpha_U \cdot I_U}$

Moment in the top chord: $M_{Ed,O} = V_{Ed,O} \cdot \frac{l_a}{2}$

Moment in the bottom chord: $M_{Ed,U} = V_{Ed,U} \cdot \frac{l_a}{2}$

Relative normal force ν [-]	Reinforcement ratio ρ_l [-]	α_O or α_U
$\nu <$ −0,15	all ρ_l	1,0
$\nu \leq$ |0,15|	$\rho_l \leq$ 0,6	1,0
	$\rho_l >$ 0,6	0,65
$\nu >$ 0,15	all ρ_l	0,2 + 6(ρ_{l1} + ρ_{l2})
$\nu_o = \frac{N_{Ed,O}}{b_o \cdot d_o \cdot f_{cd}}$; $\rho_{l,o} = \frac{A_{s,o}}{d_o \cdot b_o}$; $\nu_u = \frac{N_{Ed,U}}{b_u \cdot d_u \cdot f_{cd}}$; $\rho_{l,u} = \frac{A_{s,u}}{d_u \cdot b_u}$		

The design of the chords is carried out for these internal forces on bending with longitudinal force and shear force. For the reinforcement distance in the top chord, the smallest defined reinforcement distance of the basic reinforcement on the top of the member is considered. For the bottom chord, the smallest distance on the bottom is considered.

Tip: *The existing longitudinal reinforcement in the overall cross-section can be considered for the results of the required longitudinal reinforcement of the chords.*

Suspension reinforcement

Due to bending, only if $x_O > h_O$: $Z_M = 0{,}4 \cdot F_{cd} \frac{x_0 - h_o}{d}$

Due to normal force (pressure pos.): $Z_N = 0{,}25 \cdot N_{Ed} \frac{h_a}{h}$

Due to shear force top chord: $Z_{V+\Delta M,o} = V_{Ed,o} \cdot \left(1 + 0{,}1 \cdot \frac{l_a}{d} + \frac{l_a}{3 \cdot h_o}\right)$

Due to shear force bottom chord: $Z_{V+\Delta M,u} = V_{Ed,u} \cdot \left(1 + 0{,}1 \cdot \frac{l_a}{d} + \frac{l_a}{3 \cdot h_u}\right)$

Right of the recess: $Z_{V,r} = Z_M + Z_N + Z_{V+\Delta M,o}$

Left of the recess: $Z_{V,l} = Z_M + Z_N + Z_{V+\Delta M,u}$

Suspension reinforcement left/right: $A_{sw,r/l} = Z_{v,r/l} / f_{yd}$

For comparison, the suspension reinforcement according to Leonhardt is also given (in Booklet 399):

Leonhardt: $Z_{V,r/l} = 0{,}8 \cdot V_{Ed}$ $A_{sw,r/l} = Z_{v,r/l} / f_{yd}$

Special features according to DAfStb Booklet 399

- The shear forces are distributed between the top and bottom chords in proportion to the stiffness of the uncracked concrete cross-section (α_o=α_u =1). The following boundary conditions apply:
  For positive moments, at least 70% and a maximum of 90% of the shear force is assigned to the top chord. If the dimensions of the bottom chord are less than 8 cm, the full shear force is assigned to the top chord and the bottom chord only acts as a tension chord. For negative moments, these conditions apply in reverse.
- The effective flexural tensile or bending compressive force is applied at the center of gravity of the respective chord.
- The flexural design of the top and bottom chords is always carried out with the specified symmetrical reinforcement ratio As1=As2
- For T-beams, the following width is considered in the area of the openings to determine the chord stiffnesses: $b_{fo/u} \leq 3\, b_w$ (with b_w – web width).

Special features according to DAfStb Booklet 599

- Distribution of the shear forces between the top and bottom chords according to the α_o/α_u values according to DAfStb Booklet 240 (comparison, Booklet 599, see table shown). For the stiffness coefficients α_o/α_u, the longitudinal reinforcement ratios are taken from the chord design, the basic reinforcement is not used.
- The lever arm (flexural tensile and bending compressive force) is not applied at the respective center of gravity of the chords but is taken from the flexural design of the overall cross-section.
- The static effective height of the total cross-section results from the cross-sectional height minus the center of gravity of the flexural tension reinforcement.
- The flexural design of the top and bottom chords is carried out using the kd method. If no result is achieved using the kd method, the design is carried out using the specified symmetrical reinforcement ratio As1=As2
- For T-beams, according to DAfStb-Booklet 599, the following width is considered in the area of the openings to determine the chord stiffnesses: $b_{fo/u} \leq 2 \cdot 0{,}2\, l_a + b_w$ (with l_a – length of the opening and b_w – web width).

The following restrictions and design instructions must be observed when dimensioning the recess:

- The verifications are currently only carried out for the permanent and temporary design situation (not for earthquakes or exceptional combinations).
- At present, recess design for cast-in-place concrete layers is not yet possible.
- Recesses whose edge distance to the support is smaller than the beam height cannot be verified using the method according to DAfStb Booklet 399 or 599. From an engineering point of view, recesses should not begin or end at a distance of < 0.10 · span length.
- In addition, the clear distance between the recesses must be greater than twice the static effective height of the member so that the interference areas of the truss models do not overlap.
- Within the program, the minimum chord height was set to 2.5 times the reinforcement distance to have sufficient lever arm for the design.
- Furthermore, due to the cross-sectional weakening, the length of a recess should be limited to a maximum of 1/3 of the span length.
- In the case of concentrated loads directly above the recess or at a distance ≤ beam height, the load transfer must be subsequently checked in the case of slender residual cross-sections (beam load-bearing effect).
- Afterwards, it must also be checked whether the anchoring lengths of the additional longitudinal reinforcement are adhered to.
- Due to the minor impact on the load-bearing behavior, no verification is carried out for recesses with dimensions L and h < beam height / 10.

Verifications in the **serviceability limit state (SLS)**

The analyses in the serviceabilty limit state include the following individual verifications:

- Verification of the limitation of the stresses for the concrete, the reinforcing steel and the prestressing steel
- Verification of the limitation of the crack width for loading and minimum reinforcement
- Verification of decompression for specific exposure classes
- Verification of the limitation of the deformation (deformation upwards, downwards and deflection after erection)

Specific requirements, which depend partly on the type of construction (pre-tensioned concrete in the software) and the exposure classes, apply to the verifications in view of the necessity to perform the verification, the limit values to be verified and/or the load combinations to be used.

Infrequent (= characteristic combination)

EN 1990, eq. 6.14

NA_D:	partial safety factors as per DIN EN 1990/NA table NA.A.1.1
EN2, NA_PN, NA_A:	combination coefficients as EN 1990 table A.1.1
NA_GB:	combination coefficients in accordance with NA to BS EN 1990 table NA.A1

Permanent actions are included in the combinations with their characteristic values.

In contrast to other variable actions, the dominant independent action (leading action) is not reduced by the corresponding combination coefficient $\psi 0$.

Frequent combination

EN 1990, eq. 6.15

NA_D:	partial safety factors as per DIN EN 1990/NA table NA.A.1.1
EN2, NA_PN, NA_A:	combination coefficients as EN 1990 table A.1.1
NA_GB:	combination coefficients in accordance with NA to BS EN 1990 table NA.A1

Permanent actions are included in the combinations with their characteristic values.

The dominant independent action is reduced by the combination coefficient $\psi 1$. All other variable actions are reduced by the combination coefficient $\psi 2$.

Quasi-permanent combination

EN 1990, eq. 6.16

NA_D:	combination coefficients as per DIN EN 1990/NA table NA.A.1.1
EN2, NA_PN, NA_A:	combination coefficients as EN 1990 table A.1.1
NA_GB:	combination coefficients in accordance with NA to BS EN 1990 table NA.A1

Permanent actions are included in the combinations with their characteristic values.

Variable actions with an unfavourable effect are included in the combination with their characteristic value reduced by the quasi-permanent coefficient $\psi 2$.

Note for different actions:

If different actions due to imposed and/or live loads apply, they are treated by default as correlating actions, i.e. as a single action. The action with the greatest combination coefficient is decisive $\psi 0$ (in the rare combination), $\psi 1$ (in the frequent combination) or $\psi 2$ (in the quasi-permanent combination) (cf. /41/ p.19, 28, 38). You can cancel the dependency in the design settings, if there is no correlation between these actions.

Limitation of the concrete edge stresses and the steel stresses

The verification is performed at the beginning and the end of each creep stage with the respective effective prestress.

As the influence of creep must be considered via a reduced modulus of elasticity of the concrete $E_{c,eff} = E_{cm}/(1+ \varphi)$, which reduces (higher creep factor φ) the concrete stresses but increases the steel stresses with time, the combinations of the maximum moment and the minimum moment of external loads must be examined at the end and the beginning of each creep stage.

Individual verifications

Compressive concrete stresses under the infrequent load combination as per 7.2 (2):

Sigc < k1 · fck

Compressive concrete stresses under the quasi-permanent load combination as per 7.2 (3):

Sigc < k1 · fck

This verification simply provides a criterium for non-linear creep which is automatically considered by the software in this case.

Compressive concrete stresses under the infrequent load combination as per 7.2 (5):

Sigs < k3 · fyk

Compressive concrete stresses under the infrequent load combination as per 7.2 (5):

Sigp < k5 · fpk

EN 2:	k1= 0.6	k2= 0.45	k3= 0.8	k5= 0.75

Departing from this regulation:

NA_PN: k1= 1.0

NA_D: k5= 0.65

NCI: infrequent load combination: Sigp < 0.8 · fpk and Sigp < 0.9 · fp0.1k

Compressive concrete stress at the time of the prestressing force application *t = tA,Lag* as per 5.10.2.2 (5)

fck(t): The resulting compressive strength *fck(t)* at the time of the prestressing force application is *fcm(t) - 8* with *fcm(t)* as per equation 3.1.

In accordance with 10.3.1.1.(3), the heat treatment in the prestressing bed the concrete age matched to the temperature *tT* as per equation B.10 is considered instead of *t* and *ßcc(t)* in equation 3.1 is limited to 1.0.

Sigc < k1 · fck

EN2, NA_A, NA_PN, NA_GB: k6= 0.7

NA_D: k6= 0.6

0.7 is only permissible, if specific prerequisites are met, see reference /52/ p. 63.

The verification is performed with the internal forces in the erecting system also for the case “Lifting the girder up from the mould”.

Sigc < 0.45* fck(t)

This verification simply provides a criterium for non-linear creep which is automatically considered by the software in this case. See the chapter determination of the creep factor and the shrinkage strain.

Stress analysis

Internal forces: external loading in accordance with the required load combination

Prestress: as per 5.10.9 with the characteristic value of the prestress still acting at the respective time; for the verification of the steel stresses, however, as per 7.2 (5) with the average value of the prestress

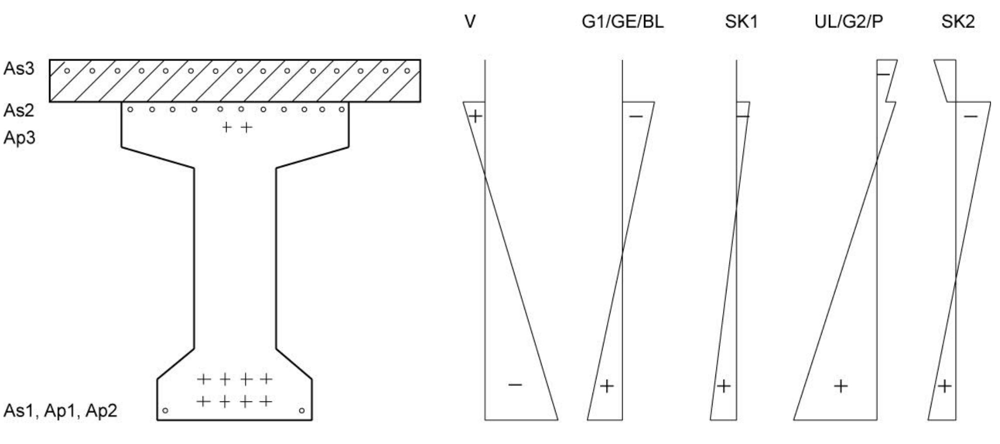
Cross section: the cross section is considered cracked from the time, when the tensile edge stress in the infrequent load combination in state I exceeds *fctk0.05*. For this time and all later times at which tensile stress would be generated in state I, the stresses are determined in state II (recommendation of reference /54/ p. 404).

Deflection in state I

All stresses resulting from external loads are determined with the ideal cross-sectional properties.

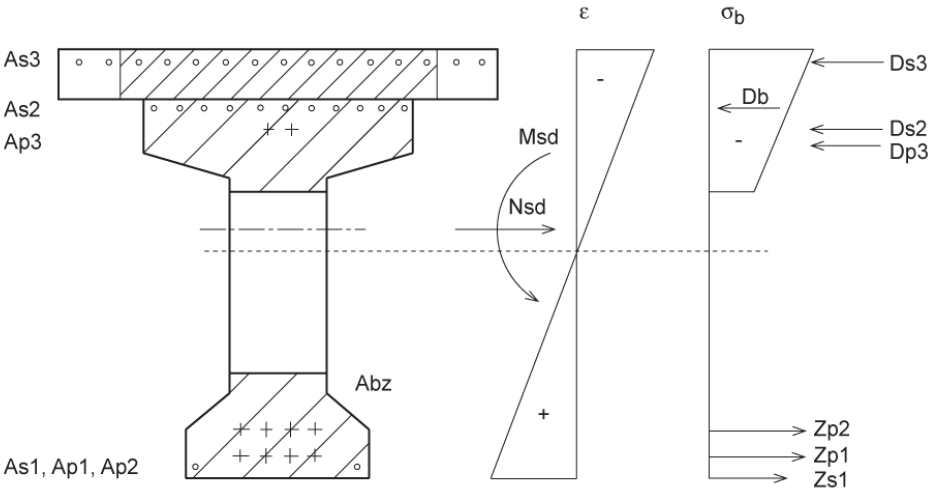
If a cast-in-place complement was added, the ideal composite cross-section is used for all loads after the creation of the bond. The concrete stresses of the added layers calculated this way are additionally multiplied by the relation of the moduli of elasticity of the concretes.

Stresses due to prestress as well as creep, shrinkage and relaxation are determined with the method described by Abelein using the respective partial internal forces and partial cross-sections.



Concrete edge stresses in state II

As the stresses cannot be superimposed, the internal forces caused by prestress and external load are combined to a maximum or minimum moment together with the associated longitudinal force. The edge strain and compressive strain in state II is determined for these combinations.



A linear elastic behaviour of the concrete with failure of the tension zone is assumed. The influence of creep is considered via a reduced modulus of elasticity of the concrete $E_{c,eff} = E_{cm}/(1+ \varphi)$.

The creep factor φ is the sum of the creep factors of the creep stages completed at the respective time.

For the infrequent and frequent load combinations, for the time *t = tE,Nut, ϕeff = ϕ * Mqp,k / ME0,k* is used instead of ϕ for the 'usage' creep stage (ÖNorm B 1922-1-1 10.1.1 d).

Mqp,k: quasi-permanent load combination including prestress

ME0,k: load combination including prestress required for the verification

NA_A: The increase of the steel stresses due to state II is obtained in the following expression $\Delta\sigma = \xi_1^2 * \varepsilon(yp) * Ep$ (10.1.1. c)

The neutral axis position and edge strain at which the internal and external forces are in balance is sought after by iterative approximation.

Specialities in connection with cast-in-place complements:

For cast-in-place complements, the width of the added layers that is used in the calculation is modified in accordance with the relation of the moduli of elasticity of the cast-in-place concrete and the pre-fabricated concrete.

For verifications after the creation of the bond (from the end of the creep stage 'casting of cast-in-place complement', it is assumed that the global cross-section has the same effect as if it had been fully casted right at the beginning. This means that the moment from prestress is referenced to the centre of gravity of the composite cross section in the calculation.

The considered creep factor is averaged in accordance with the area portions.

Prestressing steel and reinforcing steel stresses

The steel stresses result from strain at the height of the steel fibre that is calculated assuming a plain strain state. A stress portion in the prestressing bed condition due to prestress, creep, shrinkage and relaxation is added to the considered strain.

Limitation of the crack width

The verification is performed at the beginning and the end of each creep stage with the respective effective prestress. If prestress generates tension on top as this is generally the case, the verification on the upper edge is performed at the beginning and the verification on the lower edge at the end of the creep stage. In other cases, it is done the other way round.

In addition, the maximum moment at the lower edge and the minimum moment at the upper edge resulting from external loads are considered.

The decisive internal forces combination, the permissible crack width and an additional verification of the decompression are determined based on the exposure classes in accordance with table 7.1.

For reinforced concrete components:

	X0, XC1	XC2/XC3/XC4	XS1-3, XD1-3	Comment
EN	0.4 + Qc	0.3 + Qc	0.3 + Qc	Tab. 7.1N
NA_D	= EN	= EN	= EN	Tab. 7.1DE
NA_GB	0.3 + Qc	= EN	= EN	
NA_A	= EN	= EN	= EN	
NA_PN	= EN	= EN	= EN	

Pre-tensioned concrete

	X0, XC1	XC2/XC4	XS1-3, XD1-3	Comment
EN	0.2 + Fc	0.2+ Fc Dec. Qk	Dec. Fc	Tab. 7.1N
NA_D	= EN	= EN	0.2+ Ic and Dec. Fc	Tab. 7.1DE
NA_GB	= EN	= EN	= EN	
NA_A	= EN	= EN	0.2+ Ic and Dec. Fc	
NA_PN	= EN	= EN	= EN	

Qc quasi-permanent combination
Fc frequent combination
Ic infrequent combination
Dec. verification of the decompression

If no tensile forces greater than fctk0.05 result for the infrequent load combination in state I at the current time, the verification of the limitation of the crack width can be dispensed with.

If the edge stress under the decisive load combination is no tensile stress, the verification of the limitation of the crack width can also dispensed with.

The verification is based on a direct calculation of the crack width, which must be smaller than the permissible crack width.

$$wk = s_{r,max} \cdot \left(\varepsilon_{sm} - \varepsilon_{cm}\right) \qquad \text{Gl. 7.8}$$

The existing crack width *wk* results from the maximum crack spacing $s_{r,max}$ and the average strain difference (ε_{sm}-ε_{cm}) of concrete and steel.

Max. crack spacing as per equation 7.11

$$s_{r,max} = k_3 \cdot c + \frac{k_1 \cdot k_2 \cdot k_4 \cdot \phi}{\rho_{p,eff}}$$

k_1: coefficient for reinforcement for the bond quality

k1= (φs* ns * k1s + ϕp* np* k1p) / (φs* ns + ϕp* np)
k1s= 0.8 good bond quality
1.6 poor bond quality

k_2: coefficient of expansion distribution

Bending: 0.5

Tension: 1.0

Bending + tension (ε1 + ε2) / (2 · ε1)

c: concrete cover on longitudinal reinforcement

ϕ: mean diameter of the tensile reinforcement as per equation 7.12

k3, k4: additional coefficients, NDP

$\rho_{\rho eff}$: effective reinforcement ratio in the effective tension zone as per equation 7.10

Aceff: area of the effective tension zone

hc,ef: height of the effective tension zone as per 7.3.2 (3)

Ap: reinforcement ratio in the effective tension zone

ξ1: reduction factor for the bond strength of the prestressing steel as per equation 7.5

ξ: coefficient of the bond strength of the prestressing steel as per table 6.2

ϕp equivalent diameter of the prestressing steel as per 6.8.2

As: reinforcing steel in the effective tension zone

EN 2, NA_PN, NA_GB: k3= 3.4 k4= 0.425

NA_A, NA_D: $s_{r,max} = \phi/(3.6 \cdot \rho_{\rho eff}) < \sigma s \cdot \phi/(3.6 \cdot fcteff)$

Mean strain difference as per equation 7.9

$\varepsilon_{sm}-\varepsilon_{cm} = (\sigma_s - kt \cdot fcteff/\rho_{\rho eff} \cdot (1+ \alpha e \cdot \rho_{\rho eff}))/Es >=0.6\ \sigma_s /Es$

kt: 0.4 long-term load action

fcteff: fctm as per table 3.1

αe = Es/Ecm

σs: stress in the tensile reinforcement when assuming a cracked cross-section

NA_D: effective steel stress with consideration of the different bond properties of the reinforcing steel and the prestressing steel 7.3.3 (2) NCI, equation NA 7.5.3

In the calculation of the crack spacing, it is distinguished between a single crack and the final cracking state.

Where a single crack is concerned, the entire tension zone of the concrete cross-section is involved in the cracking process and the steel strain recedes except for the concrete strain (cf. /33/ figure 2a). The crack spacing $s_{r,max}$ results on the left side of equation 7.11, the mean strain difference on the right side of equation 7.9 (cf. /33/ 5.2).

In the final cracking state, the bond deteriorates and is everywhere smaller than the steel strain. Where thick components are concerned, only a part of the tension zone of the concrete cross-section is involved in the cracking process (cf. /33/ figure 2b). The crack spacing $s_{r,max}$ results accordingly on the left side of equation 7.11, the mean strain difference on the right side of equation 7.9 (cf. /33/ 5.2).

See the output example: Limitation of cracking

Verification not successful

Inadmissible crack widths are marked with an asterisk (*). The reinforcement of the concerned side must be increased, or the diameter of that side must be reduced if possible. The undesired effect of the prestress creates cracks especially on top. This effect can be prevented by stripping the insulation.

On bottom, you can reduce the crack width by increasing the prestressing force or by reducing the creep and shrinkage losses.

Minimum reinforcement for the limitation of the crack width

The minimum reinforcement is used to limit the crack width of the constraint internal forces and residual stresses. It is calculated for the crack internal forces.

If you can exclude the mentioned causes, e.g. for statically determined pre-fabricated components supported without constraint (cf. /35/ p. 5-18), the calculation of the minimum reinforcement is not required. You can optionally exclude this calculation in the design settings.

Otherwise, a minimum reinforcement as per 7.3.2 is calculated for the top and the bottom face, if the decisive extreme concrete stress in the infrequent combination is greater than the following limit value on the respective cross-section side (7.3.2 (4):

EN 2, NA_A, NA_PN, NA_GB: σ > fcteff

NA_A: σ > -0 N/mm²

NA_D: σ > -1 N/mm²

The minimum reinforcement for flanged cross-sections is calculated separately for the web and the flange, whereby the rectangle over the total cross section height is considered as the web and the remaining parts of the plate as the flange.

$A_{s,min} \cdot \sigma s = kc \cdot k \cdot f_{ct,eff} \cdot A_{ct}$ (equation 7.1)

k coefficient for non-linearly distributed internal stresses

1.0 (h <= 300 mm)... 0.65 (h >= 800 mm)

h: web height or flange width

NA-D: h is the smaller value of the partial cross section

NA-D, NA-A: with internal constraint, $k \cdot 0.8$ applies

$f_{ct,eff}$ tensile strength, f_{ctm} (t <= 28d)

NA_D: >= 2.9 N/mm² if t >= 28 d

k_c coefficient for the stress distribution

$kc = 0.4 \cdot (1 - \sigma c / (k_1 \cdot f_{ct,eff} \cdot h/h'))$

σc: concrete stress (state I) under internal crack forces in the centre of gravity of the partial cross section

Chords hollow box, T-cross sections, for internal crack forces completely under tension

$kc = 0.9 \cdot F_{cr} / (A_{ct} \cdot f_{ct,eff}) >= 0.5$

Fcr: tensile force in the chord loaded by internal crack forces (state I)

σs: Tab. 7.2N with Ds1, derivation see /54/ p. 7-6

$D_{s1} = D_s \cdot f_{ct0} / f_{ct,eff} \cdot 2 \cdot (h\text{-}d)/ (k_c \cdot h_{cr})$

NA_D, NA_A:

As is determined via the relationships of the direct crack width calculation.

The force *Fs* the reinforcing steel must resist to is determined by the resultant force of the tension wedge under internal crack forces in state I *Fcr* on the left side of equation 7.1N.

With $Fs = k \cdot k_c \cdot f_{cteff} \cdot A_{ct}$, the equation is as follows:

$$As = \sqrt{\frac{ds \cdot (1-\beta t) \cdot Fs \cdot Fs}{3.6 \cdot Es \cdot wk \cdot fcteff}}$$

If Fs > Fcre (tensile concrete force in the effective tension zone Fcre= $A_{ceff} \cdot f_{cteff}$), the following may be assumed:

$$As = \sqrt{\frac{ds \cdot Fcre \cdot (Fs - \beta t \cdot Fcre)}{3.6 \cdot Es \cdot wk \cdot fcteff}}$$

Verification of the decompression

The verification of decompression requires that the tendon must at least have a defined distance *a* to the overcompressed concrete under the action of the load combination with the unfavourable characteristic values of the prestress (*rinf, rsup*) that is determined by the exposure classes.

XC2, XC3, XC4:	quasi-permanent load combination
XD1, XD2, XD3, XS1, XS2, XS3:	frequent load combination

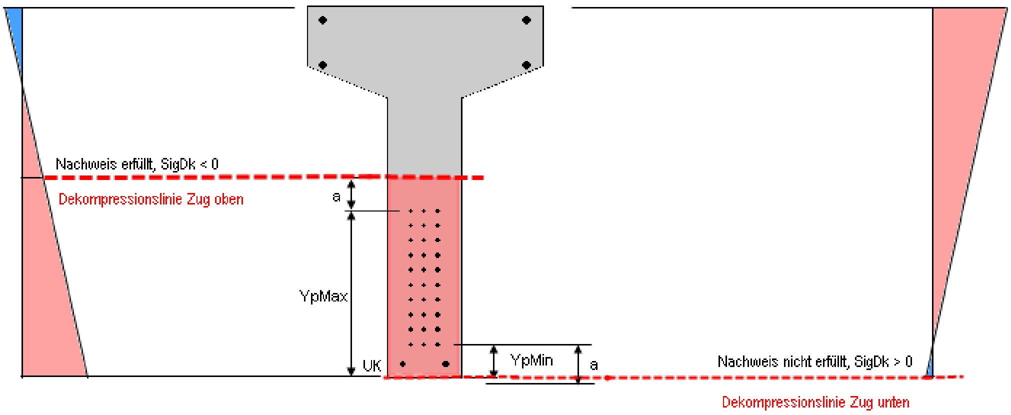
EN2, NA_A, NA_PN, NA_GB:

a= 25 mm

NA_D:

a= 100 mm >= h/10

As per /52/ p. 120, you can dispense with the verification of the end areas (x < ldisp or x > LBI - ldisp application length *ldisp*, see the paragraph effective prestress)



The verification is successful when the neutral tension axis coincides with the decompression line defined by the distance *a* or runs behind it. *a* is referenced to the tendon axis.

If tensile stresses occur at the examined cross-section edge, state II becomes decisive because of the accompanying reduction of the compression zone height.

The verification is performed for the states ‘tension on top’ and ‘tension on bottom’. For tension on top, the resulting decompression line runs at the distance *a* from the topmost tendon layer, i.e. *YpMax+a*. For tension on bottom, the resulting decompression line runs at the distance *a* from the lowest tendon layer, i.e. *YpMin-a*.

If the decompression line projects beyond the cross-section, a verification for the cross-section edge is sufficient.

For illustration purposes, the concrete stresses *σDk* at the decompression line and/or at the edge of the cross section are indicated in the software. They are determined with the strain in state II, if applicable, and, if tensile strain applies, they are a fictive value to demonstrate the exceeded verification limits.

Limitation of deformation

Time and decisive loading:

Deformation is calculated as per DIN EN 1990 A1.4.3 (1) with a moment from external loads, a load combination of the serviceability limit state (quasi-permanent, frequent or infrequent combination) as well as with the prestressing forces highly active at the time of examination (as per 5.10.9 (1), characteristic value).

Especially when physical reasons play a role in addition to aesthetical criteria (e.g. protection of partition walls or glass facades) or when variable loads would not be considered because of *ψ2= 0*, it might be necessary to assume a less favourable load combination than the quasi-permanent combination specified in 7.4.1 (4).

Variable loads are only considered if their superposition with the prestress is unfavourable. The sag is calculated at the beginning and at the end of the ‘usage’ creep stage.

Beginning of the creep stage:

The maximum prestress in the time segment applies and is considered accordingly with its upper characteristic value (factor *rsup*). If the prestress generates tension on top (typical case) the loads of the *min M* load case are considered.

End of the creep stage:

After deduction of the losses from creep and shrinkage, the minimum tensile stress in the time segment applies and is considered with its lower characteristic value (factor *rinf*). If the prestress generates tension on top (typical case) the loads of the *max M* load case are considered.

From the difference of the sags at the beginning of the usage and its end, a deflection after erection is determined. This deflection must not exceed a permissible value in accordance with 7.4.1(5).

You can configure the permissible values for the deflection and the deflection after erection. They are preset to L/250 and L/500.

Basis of calculation

The sag results from the integral of the curvatures and the moment due to a virtual force applying at the point of the deformation to be determined over the member length (/27/ T.1, equation 8.21).

In the software, the curvatures ρ and the virtual moment *M'* are calculated at the pertaining grid points and are integrated over the member section length.

The mapping accuracy of discontinuities such as concentrated loads, stripping, cantilever etc. depends on the number of grid points.

The curvature consists of a portion due to bending including creep and a portion due to shrinkage. You can optionally eliminate the portion due to shrinkage in the design settings. When calculating the curvatures, a tensile stiffener is considered.

Curvature due to bending:

State I: $\rho M_1 = M_{Ed} * k\phi \ / \ (E \cdot I_i)$

M_{Ed}: internal moment of the quasi-permanent combination including the moment from effective prestress (characteristic value)

I_i: ideal moment of inertia
Cast-in-place complement:
After creation of the composite cross section ($t > t_{E,Bet}$), deformation is calculated with the stiffnesses of the composite cross section I_{iV}. All previously acting load portions (*G1+V+SKR1+GE*) are therefore increased with the factor $kI = I_{iV}/I_{iF} - 1$.

E: modulus of the concrete, secant modulus *Ecm*
→ see Concrete properties, coefficient αe
Cast-in-place concrete complement:
Modulus of elasticity of the pre-fabricated concrete (the modulus of elasticity of the cast-in-place concrete is considered in the ideal moment of inertia of the composite cross section).

kϕ: the resilience of the concrete under compression load is considered by including a factor, that is determined by the action over time of the participating actions (cf. /48/, p. 405 et seq. or /53/ p. 349 et seq.).

$k\phi = (\Sigma(Mi * (1+\phi i(ti,tE)) - \Delta Mcsr) / MEd$

ϕi(ti,tE): the effective creep factor for the load *i*

ti: start of the load action

prestress V, self-weight G1: ti= tA,Lag (releasing the anchors)

Imposed loads Q, subsequent permanent load G2: ti= tA,Nut (start of usage) tE: end of the examined creep stage:

Effective creep factor means in the sense of paragraph 5.8.4 that the partial creep factor 'usage' is reduced by the factor *k= (Myd+ Mv)/(My+Mv).*
(*My*: external loads of the selected combination, *Mv*: characteristic prestress,
Myd: external loads of the quasi-permanent combination). If the deflection is calculated with the frequent or infrequent combination, the resulting $k < 1$, in the quasi-permanent combination $k = 1$.

In combination with cast-in-place complements, the creep factor of the composite cross section is determined by the creep factors of the partial cross sections weighted in accordance with the transformed strain stiffnesses (cf. /53/ p. 354)

ΔMcsr: additionally, the gradual occurrence of creep and shrinkage losses are considered in connection with the prestress. This is achieved via

the correction factor *ΔMcsr= Mcsr *(1- ρ * ϕik)*.

Mcsr: moment caused by creep and shrinkage

ρ: aging coefficient (fullness) of the creep function

ϕik creep factor of the examined creep stage

kI* Σ(Mj) only with cast-in-place complements after creation of the bond
see explanatory notes on I_i

State II $\rho M_2 = (\varepsilon 2 - \varepsilon 1) / h$

ε2, ε1: edge strain under quasi-permanent loads in state II with a linear elastic concrete action curve creep is considered via the reduced modulus of elasticity *Ecm/(1+φ(t0,t))*. A weighting of the factor (1+φ) in accordance with the load history is not possible in this case.

φ(t0,t) effective creep factor from the release of the anchoring *t0 = tA,Lag* until the considered time *t*. It is the sum of the creep factors of the corresponding creep sections. (effective creep factor: see explanations above)

In combination with cast-in-place complements, the creep factor of the composite cross section is determined by the creep factors of the partial cross sections weighted in accordance with the transformed strain stiffnesses (cf. /53/ p. 354)

h: component height

Curvature due to shrinkage:

State I:

$\rho S_1 = -\varepsilon_{cs}(t0,t) \cdot \alpha_E \cdot Sx_1 / I_1$ (EN 1992-1-1: 7.21)

in /54/ p 399 the expression rephrased to:

$\rho S_1 = -M_{cs1} / (EI_1(t))$

M_{cs1} moment due to shrinkage impeded by the reinforcement

$M_{cs1} = -\varepsilon_{cs}(t0,t) \cdot Es \cdot Sx_1$

ε_{cs} (t0,t): Shrinkage strain before releasing the anchoring *t0= tA,Lag* up to the examined time *t*

For cast-in-place complements, the shrinkage strain of the pre-fabricated component is assumed for reasons of simplification.

$Sx_1 = \Sigma(A_{si} \cdot z_{si})$ static moment of the reinforcement,

z_{si}: distance of the reinforcement to the centre of gravity of the ideal cross section, positive if located below the centre of gravity

$EI_1(t)$=	$E_{ceff,t}*I_i$	stiffness in state I at the time *t*
	$E_{ceff,t}$=	$E_{cm}/(1+\varphi(t0,t))$
	I_i	ideal moment of inertia

State II:

$\rho S_2 = -\varepsilon_{cs}(t0,t)*\alpha_E*Sx_2/I_2$ (EN 1992-1-1: 7.21)

in /54/ p. 399 the expression rephrased to:

$\rho S_2 = M_{cs2}/(EI_2(t))$

M_{cs1} moment due to shrinkage impeded by the reinforcement

$M_{cs2} = -\varepsilon_{cs}(t0,t)*Es*Sx_2$

$Sx_2 = \Sigma(A_{si} * z_{si})$ static moment of the reinforcement,

z_{si}: distance of the reinforcement to the neutral axis, positive if the laxer is below the neutral axis:

$EI_2(t)$= $MEd/\rho M_2$ stiffness in state II at the time *t*

Tension stiffener and overall curvature:

A distribution coefficient ζ that depends on the degree of cracking is used to consider the contribution of the concrete between the cracks.

The total curvature is obtained from the curvatures in state I and II weighted with ζ.

$\rho = \zeta\cdot \rho(ZII) + (1 - \zeta) \cdot \rho(ZI)$

EN2, NA_A, NA_PN, NA_GB:

$\zeta = 1 - \beta \cdot (\sigma_{sr} / \sigma_{s2})^2$ (equation 7.19)

β: coefficient of the load application period 0.5 (long-term action)

σ_{sr}: steel stress in state II obtained from the internal crack forces with f_{ctm}

σ_{s2}: steel stress under load in state II

$\sigma_{s2} < \sigma_{sr}$ $\zeta = 0$

EN2-D: (cf. /54/ p. 393)

$\sigma_{s2} < \sigma_{sr}$

$\zeta = 0$ (/54/ p. 404 crack criterion for the deformation calculation f_{ctm})

$\sigma_{sr} < \sigma_{s2} < 1.3* \sigma_{sr}$

$\zeta = 1- (\beta_t *(\sigma_{s2}-\sigma_{sr})+(1.3*\sigma_{sr}-\sigma_{s2}))/(0.3*\sigma_{sr})* (\varepsilon_{sr2} - \varepsilon_{sr1})/ \varepsilon_{s2}$

$1.3 * \sigma_{sr} < \sigma_{s2} <$ fy:

$\zeta = 1-\beta_t* (\varepsilon_{sr2} - \varepsilon_{sr1})/ \varepsilon_{s2}$

ε_{sr2}, σ_{sr}: steel strain and steel stress in state II obtained from the internal crack forces with f_{ctm}

ε_{sr1} steel strain in state I obtained from internal crack forces

The crack moment is determined with f_{ctm} and with *Ned* in accordance with the effective prestressing force; crack strains are determined with consideration of $\phi(t0,t)$.

ε_{s2}, σ_{s2}: steel strain and steel stress determined under load in state II with consideration of $\phi(t0,t)$

β_t coefficient of load application period 0.25 (long-term action)

It is recommended to determine the tensile stiffening with the characteristic load combination for high demands on the deformation calculation (see /52/ p.134), as actions from wind and snow are not taken into account in the quasi-permanent load combination due to $\psi 2 = 0$.

Optional: tension stiffening with the load combination selected for the calculation of the pre-deformation (e.g. to compare them with the results in former software versions)

Verification not successful

If the permissible deformations are exceeded at the bottom, you should either increase the bending stiffness of the component by increasing the cross section or select a higher concrete class or increase the prestress. The latter is limited by the higher negative sag before and after the installation, however.

By checking the corresponding option in the output profile, you can put out intermediate results from the curvature calculation.

Modification of the length of the girder at the supports

The length of the girder changes due to temperature, creep and shrinkage. This change produces horizontal support reactions in combination with longitudinal displacement impediments.

When activating the option for the verifications on the supports in the data-entry menu, the change in length is calculated for each creep stage.

When working through the grid to determine the critical sections, the change in length due to creep and shrinkage at the top and bottom face of the girder is determined for each grid section. This allows you to consider parameters that might change over the length of the girder such as stiffness, creep factors, shrinkage strain, creep-generating stresses as well as stress due to creep and shrinkage in line with the density of the grid.

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