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New Design Concept for Reinforced Concrete Columns in Buildings

Approche nouvelle dans l'analyse des colonnes de bâtiment

Neues Bemessungsverfahren für Stahlbetonstützen in Gebäuden

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SUMMARY
Buildings are very often horizontally stabilized by means of a core or by shear walls. The columns in these buildings serve primarily to transfer the vertical loads. The columns are nevertheless subjected to imposed end deformations. A new design concept for reinforced concrete columns is proposed based on theoretical and experimental studies.

RESUME
Comme les colonnes de bâtiments sont avantageusement stabilisées dans le sens horizontal par des noyaux ou refends, ils ne servent qu'à reprendre les charges verticales tout en subissant des déformations imposées à leurs extrémités. A partir d'études théoriques et expérimentales, un nouveau concept de dimensionnement des colonnes est présenté.

ZUSAMMENFASSUNG
Da Gebäudestützen mit Vorteil durch Kerne oder Wände horizontal ausgesteift werden, dienen sie nur zur Übertragung der vertikalen Lasten, wobei ihnen an den Enden Verformungen aufgezwungen werden. Von theoretischen und experimentellen Untersuchungen ausgehend, wird ein neues Konzept für den Entwurf von Stützen dargestellt.

1. REASONS FOR A NEW CONCEPT

Two completely different attitudes can be observed in practice when civil engineers design reinforced concrete columns for buildings:

a) considering only the normal force, which considerably simplifies the problem,

b) regarding the flexural moment and the normal force, which leads to a frame analysation, sometimes considering the influence of the second order effects.

The first attitude leads often to a very good design provided that the engineer has sufficient design experience and feeling for design. The second attitude is one of the conscientious engineer who belives the correct solution can only result from complicated calculations.
Columns in buildings are a good example where the gain in knowledge which has been established over the last 20 years in the desing of reinforced concrete structures could be put into practice.
Theoretical studies and tests, based on the theory of plasticity, have proven the high ductility of reinforced concrete members, despite a certain reluctance to accept this theory, which is diminishing each year.
Moreover, the service behaviour of reinforced concrete members under compression has shown to be more favourable in reality what the crack problem concerns than predicted. Hence there is a need to adopte column design to current knowledge of column behaviour.
The high ductility of the columns often allows the complete suppression of expansion joints or large spacings. Due to the presence of cores, staircases or elevator cages, a multistorey building with columns can be conceived in the following manner (Fig. 1):

- spacing between the expansion joints between every 50 to 100 m,
- core and/or shear walls in reinforced concrete to take the horizontal loads,
- columns with a high degree of reinforcement ($\rho = 1$ to 15%) to take the vertical loads.

In conceiving a building in this way, it is necessary to consider the behaviour of the columns under imposed deformations. The columns follow deformations which are imposed on them by the bending or contracting of the floor slabs (Fig. 2).
It is not useful to calculate the moments in the frame joints. The calculation is made uncertain due to various effects such as crack formation in the slabs, the effective width of the slab and the adopted live load pattern etc.. It is preferable to verify that the columns have a sufficient axial load capacity to take the normal force and that they are ductile enough to withstand the imposed end deformations. It is evident that this ductility is higher the more slender the column is.

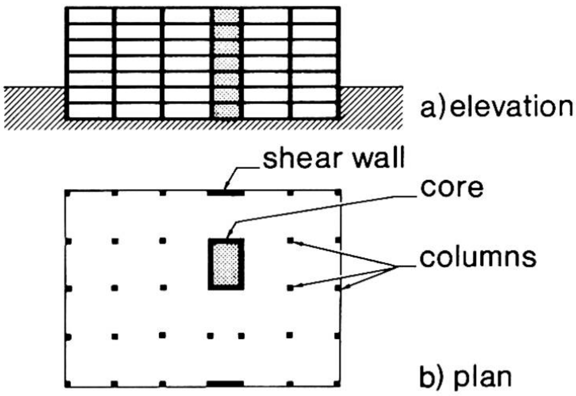


Fig. 1 Building with Core and Shear Walls

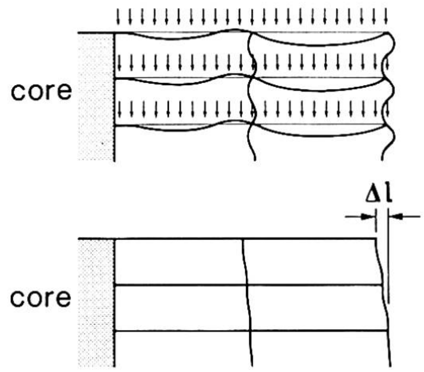


Fig. 2 Imposed Deformations on Building

This may be achieved by choosing a high degree of reinforcement which allows a reduction of the dimensions of the columns leading to an economy of material.

2. DESIGN CRITERIA

The proposed column design method is based on limit state considerations.
The ultimate and the serviceability limit states of a column are verified by comparing the estimated imposed angles at the column end with the respective limit angles. The latter depend on the level of the applied normal force. Four different deformation cases can principally be distinguished (see Fig.3).

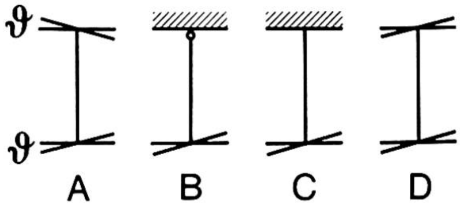


Fig. 3 Deformation Cases

2.1 Ultimate Limit State

Braced columns with a moderate slenderness ratio do not fail when their maximum flexural resistance is reached. They simply start to develop plastic hindges, which allow further large deformations to occur. The normal force starts to centre itself at the rotated column end after the maximum flexural resistance has been reached. The column fails when the deflection within the plastic hinge produces a moment which becomes equal to the remaining resistance. The imposed end deformations and the applied normal force are important for the design of a column and not the maximum flexural resistance.

Reinforced concrete columns with closely spaced stirrups show ductile behaviour under imposed deformations. Fig. 4 shows schematically the moment-curvature relationship of a column with a constant normal force.
The strains on the compression face are around 0.4 to 0.5 % when the column reaches the maximum flexural resistance. The concrete cover starts then to spall on the compression face.
Tests [1] show that strains of 2 % and more can be reached in the core if

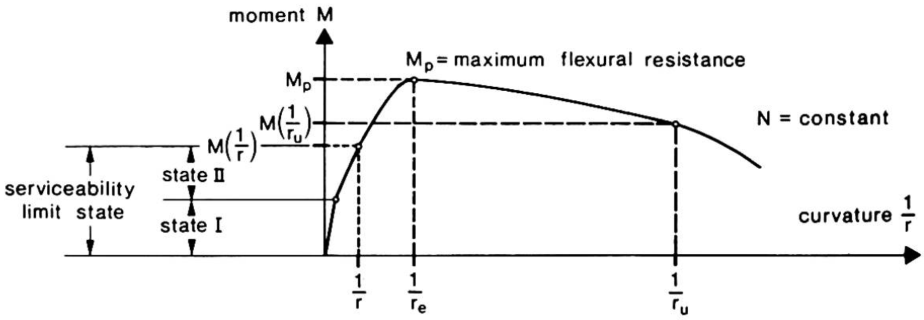
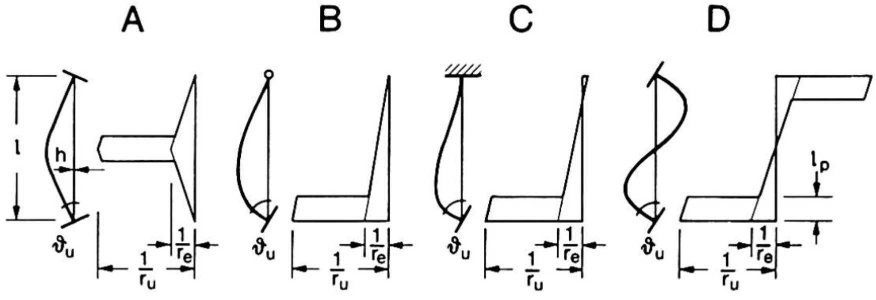


Fig. 4 Normal Force-Moment-Curvature Relationship : Admissible ($1/r$), Plastic ($1/r_e$) and Ultimate ($1/r_u$) Limit Curvatures



$\xi_e =$ $\frac{1}{4}\cdot\frac{l}{h}$ $\frac{1}{3}\cdot\frac{l}{h}$ $\frac{1}{4}\cdot\frac{l}{h}$ $\frac{1}{6}\cdot\frac{l}{h}$

$\xi_p =$ $\frac{1}{2}\cdot\lambda_p$ $\left(1-\frac{1}{2}\cdot\frac{h}{l}\cdot\lambda_p\right)\cdot\lambda_p$ $\left(1-\frac{3}{4}\cdot\frac{h}{l}\cdot\lambda_p\right)\cdot\lambda_p$ $\left(1-\frac{h}{l}\cdot\lambda_p\right)\cdot\lambda_p$

Fig. 5 Ultimate Limit Angle : Curvature Diagrams and Coefficients ξe, ξp

the concrete is adequately confined by stirrups. An ultimate limit curvature can be fixed to $h/r_u = 10 \cdot \varepsilon_{yk}/\psi = 0.02/\psi$ (see [2]), with ψ equal to the ratio of the distance between the longitudinal reinforcement of opposite faces and the column width, h (see Fig. 14). A column with closely spaced stirrups (e.g. 50 mm) in the zone where the plastic hinge will occur can reach this curvature without buckling of the longitudinal bars and without any significant loss of resistance in the core concrete. The strains in the reinforcement bars on the compression face lie between 0 and 2% (10 times the yield strain of steel ε_{yk}) for that curvature, depending on the applied normal force.
A column with a very small or no normal force may reach larger curvatures in a plastic hinge as there is no danger of buckling of the reinforcement bars. The proposed limit is nevertheless considered to be sufficiently large to allow large plastic rotations. h/r_u equal the rotation of the plastic hinge if one assumes the length of the plastic hinge to be equal to the column width h and also if one assumes a constant curvature in the plastic hinge. For example with $\psi = 0.8$, the rotation of the plastic hinge is equal to 0.025 rad.
Fig. 5 shows the assumed curvature distribution for the columns which have reached the ultimate limit curvatures in the plastic hinge.
The elastic curvature distribution of deformation case A (Fig. 5) has been assumed to be triangular in order not to overestimate the ultimate limit angle as the maximum flexural resistance is reached at midspan. The ultimate limit angle is equal to

$$\vartheta_u = \xi_e \cdot h/r_e + \xi_p \cdot (h/r_u - h/r_e) \tag{1}$$

Design charts give the values for the curvatures h/r_e as a function of the applied normal force.
The value λ_p in coefficient ξ_p is defined as $\lambda_p = \ell_p/h$ with ℓ_p equal to the length of the plastic hinge. $\ell_p = 0.5 \cdot h$, h or $2 \cdot h$ respectively can be used in the estimation of the ultimate limit angle provided that the column has closely spaced stirrups over h, $1.5 \cdot h$ or $2.5 \cdot h$ respectively in the zone where the plastic hinge will appear. A numerical example is given in table 1 for the ultimate limit angles.

ϑ_u [rad]	Deformation case	A	B	C	D
$\ell/h = 5$	$\ell_p = h$	0.014	0.024	0.022	0.020
	$\ell_p = 2 \cdot h$	0.026	0.040	0.035	0.029
$\ell/h = 10$	$\ell_p = h$	0.018	0.030	0.027	0.024
	$\ell_p = 2 \cdot h$	0.029	0.049	0.045	0.040

Table 1 Ultimate angles for $\ell/h = 5$ and $\ell/h = 10$ with $\psi = 0.8$, $h/ru = 0.0250$ and $h/r_e = 0.0025$

In most practical cases the slenderness ratio of reinforced concrete columns in buildings is such that the second order effects do not have to be considered. The deflection of the column axis is normally so small that the remaining resistance in the plastic hinge is sufficiently large to balance the moment due to the deflection. When not considering any second order moment, the maximum normal force is equal to

$$N_u = (A_c' - A_s) \cdot f_{ck} + A_s \cdot f_{yk} \tag{2}$$

with A_c' equal the section area of the column without any concrete cover.

2.2 Serviceability limit state

It is evident that the verification of the serviceability has to be based on conventional simple criteria. There are two criteria for the serviceability limit state as follows

a) limitation of the crack widths,
b) prevention of the spalling of the cover concrete.

In order to simplify the verification of these criteria it is sensible to examine only the normal force due to the permanent loads. The imposed angle on the other hand will be estimated with respect to the admissible deflections of the slab and the relatif change in the length of the slab due to the influence of temperature and shrinkage. These imposed deformations are assumed to be time dependent and proportional to that of creep, i.e.increasing from zero at time $t = 0$ to a final value at time $t = \infty$.
The second criteria may be neglected since the stresses are in fact positively influenced with time by the nonlinear creep of the concrete which greatly diminishes the danger of concrete spalling at the edge or corner of the column.

The first criteria, the limitation of the crack width, will now be discussed. In this case, the curvature distribution over the column length has to be determined. It can be distinguished between cracked zones (state II) and uncracked zones (state I) in a column. The creep coefficient φ and the age coefficient χ have to be known in order to determine the curvatures. Furthermore, shrinkage (ε_{cs}) has to be considered in state II and a suitable moment-curvature relationship for a constant normal force, such as given in [3] and shown in Fig. 6, has to be used for the analysis. The curvature distribution at time $t = \infty$ in a column (Fig. 7) can thus be established with the help of design charts.

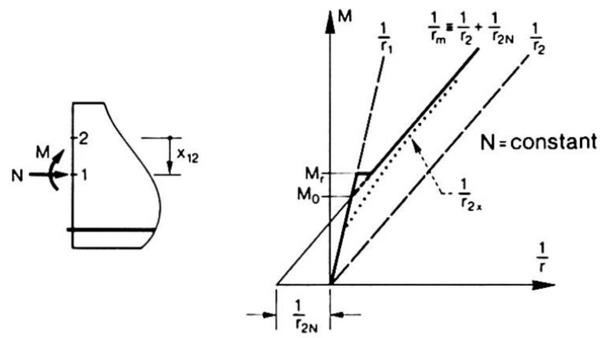


Fig. 6 Normal Force-Moment- Curvature Relationship :
(Serviceability State)

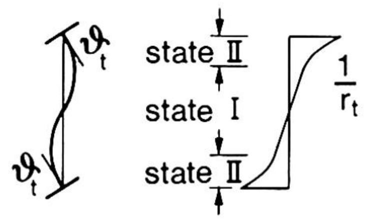


Fig. 7 Imposed Angle ϑ_t and Curvature $1/r_t$ at time $t = \infty$

The estimation of the maximum crack width is difficult in a column subjected to a constant normal force and an imposed curvature which reaches at time $t = \infty$ the value $1/r_t$. It is important to note that the cracks open considerably further with time. A whole series of tests on reinforced concrete slabs subjected to permanent loads, which has been made at the Swiss Federal Institute of Technology, Lausanne, has shown that the crack widths increase by a factor 2 during one year [4], which is a problem of pure bending. Tests have also been made on columns (see chapter 3) in order to study their crack behaviour. A constant normal force with a fixed eccentricity was applied to the columns of the long term load test series. Once again, a net increase of the crack widths was observed. Most existing formulae for the calculation of crack widths do not take into account this phenomenon or if so, very slightly. Furthermore the steel strain ε_{s2} remains more or less constant, at least in the case of pure pending, which makes it even more

difficult to explain adequately the increase in the crack widths. The reasons may lie in the shrinkage of the concrete which is under tension, in the diminishing adherence due to creep, or in the complexe, three dimensional behaviour of the concrete between two cracks.
The following approach, described in detail in [5] is proposed in order to take into account the increase of the crack width with time:
The curvature distribution $1/r_t$ is determined from the given constant normal force N and the given end rotation ϑ_t. With N and $1/r_t$, one determines the moment M. Assuming that M acts already at time t = 0, one determines then $1/r_o$, x_o and $\varepsilon_{sm,o}$, which allows the estimation of the mean crack width

$$w_o = \varepsilon_{sm,o} \cdot (h - x_o)$$

where $\varepsilon_{sm,o}$ is equal to the mean strain in the steel at time t = 0 [3], x_o is equal to the compression zone in the cross section at time t = 0 and $h - x_o$ is equal to the distance between two cracks, assumed to be equal to the length of the tensile zone (St.Venant's principle).
The maximum mean crack width at time t = ∞ is equal to:

$$w_t = \frac{h - x_t}{h - x_o} \cdot \frac{1/r_t}{1/r_o} \cdot w_o$$

The index 0 indicates time t = 0, the index t time = ∞

3. LABORATORY TESTS

Two series of 9 columns of 2.30 m length and with a rectangular section of 0.30 x 0.30 m have been tested under long term loads at EPF Lausanne in order to establish the normal force-moment-curvature relationships at the service limit state. The columns of series A had a constant reinforcement ratio of $\rho = 1\%$. The columns were subjected to a constant normal force of 262 kN to 785 kN with a uni-axial eccentricity varying from 65 mm to 198 mm (Table 2). The deformations of the columns have been observed over a period of one year. Fig. 8 and 9 show the test assembly. The columns of serie B had a reinforcement ratio varying between $\rho = 0.50\%$ and 4.72%. All columns were loaded with the same normal force of 523 kN. The eccentricity varied from 100 mm to 221 mm.
The test results of the two series A and B will be published in a test report [6] and in spezialised papers. Fig. 10 shows as an example without any detailed comment the increase of the curvatures in the columns with time.
Other tests have been made on prefabricated centrifuged columns in order to study the ultimate limit behaviour of columns with a high degree of reinforcement. The columns have been tested at EPF Lausanne in collaboration with GRAM SA, Villeneuve, Switzerland, which produces and sells such columns. The 10 columns which have been tested have a diameter of 0.29 m. Six columns have a longitudinal reinforcement of 8 bars of 34 mm ($\rho = 12.4\%$) and four have a HEM 140 steel section in the interior ($\rho = 18.0\%$). The columns have been tested in a 10'000 kN press (Fig. 11 and 13) which allows deformation controlled loadings. The load was applied either using inclined steel plates (imposed angle on the columns: 0.01 to 0.02 rad) or linear knife edges (eccentricity: 30 or 60 mm).
The test results [6] demonstrate the high load capacity of centrifuged columns as well as a high degree of ductility which allows to accommodate the imposed deformations.

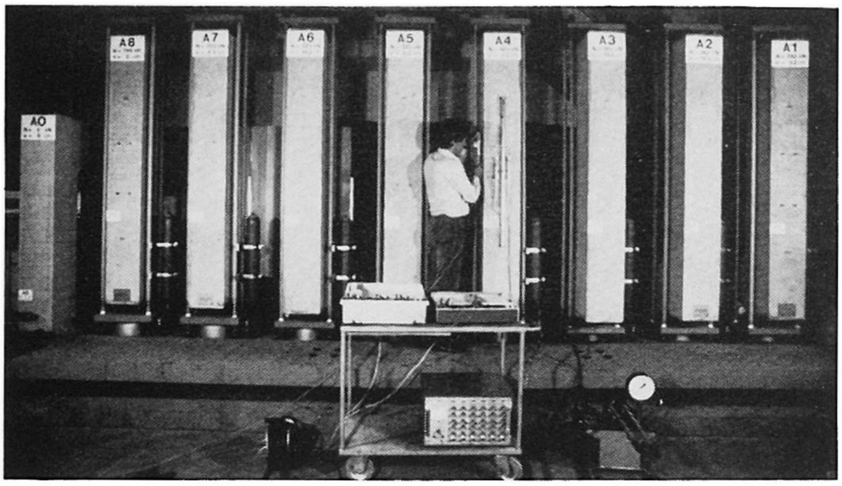


Fig.8 Tests under Long Term Loads

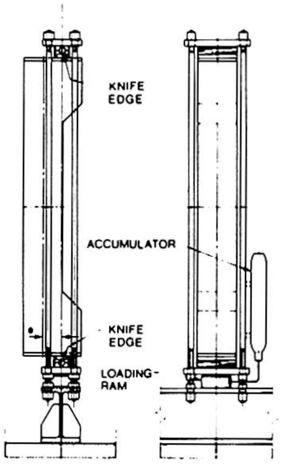


Fig.9 Testing Arrangement

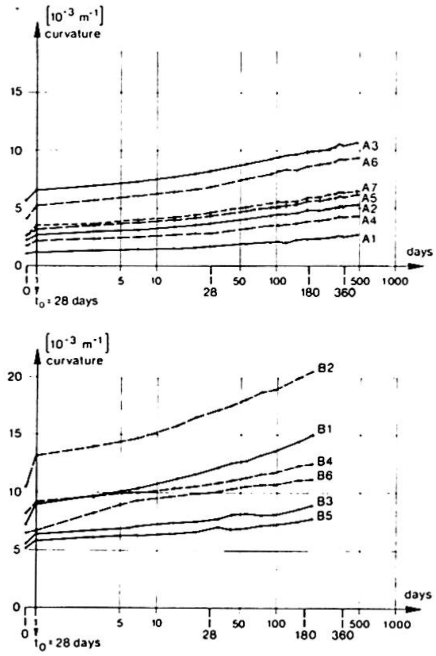


Fig.10 Curvatures against Time

SERIE A		A0	A1	A2	A3	A4	A5	A6	A7	A8
REINFORCEMENT	%	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NORMAL FORCE	KN	-	262	262	262	523	523	523	785	785
ECCENTRICITY	MM	-	92	145	198	72	93	113	65	-
B. MOMENT	KNM	-	24.1	38.0	51.9	37.7	48.6	59.1	51.0	-

SERIE B		B0	B1	B2	B3	B4	B5	B6	B7	B8
REINFORCEMENT	%	-	0.50	0.50	2.26	2.26	4.72	4.72	-	-
NORMAL FORCE	KN	-	523	523	523	523	523	523	523	1134
ECCENTRICITY	MM	-	109	123	129	156	161	221	100	-
B. MOMENT	KNM	-	57.0	64.3	67.5	81.6	84.2	115	52.3	-

Table 2 Loads of the Test Columns

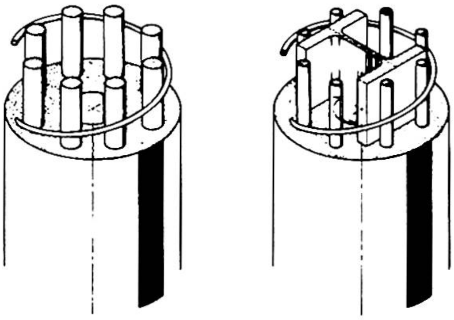


Fig.12 Two Column Types

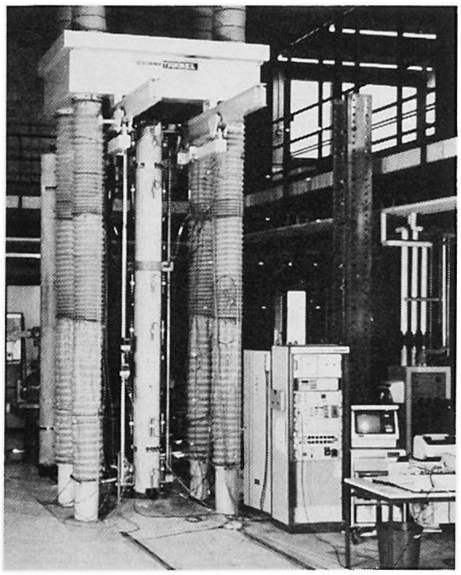


Fig.11

Test assembly

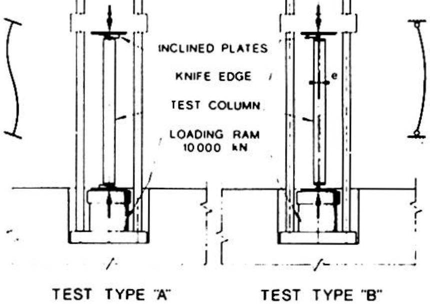


Fig.13 Deformation Cases

4. EXAMPLE

Consider a column with a length of 3.50 m, adequtately confined with reinforcement as shown in Fig. 14. The slabs impose on the column the usual type of deformation, type D, Fig. 3.

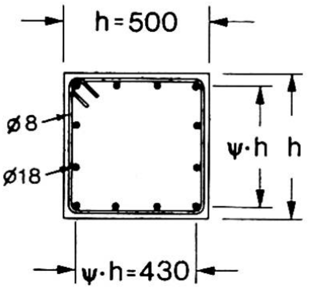


Fig. 14 Cross Section

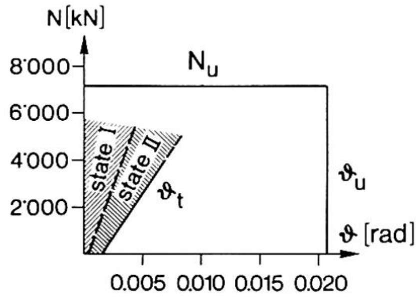


Fig. 15 Ultimate and admissible limit angle

The degree of reinforcement is equal to 12 bars of Ø 18, ρ = 1.2 %, where A_s = $3 \cdot 10^3$ mm^2. The stirrups of Ø 8 have a spacing of 50 mm over a length of 750 mm at the column ends and of 200 mm elsewhere. The concrete grade is C 30 with a compressive strength f_{ck} = 30 N/mm^2 and a tensile strength of 2 N/mm^2. The creep coefficient is equal to φ = 2.5, the age coefficient χ = 0,8 and the value for shrinkage may be taken as $\varepsilon_{cs} = 25 \cdot 10^{-5}$.
The determination of the limit angles is required as a function of the normal force N which the column is able to support at the ultimate and serviceability limit state.
Solution : The slenderness ratio is with $\lambda = \sqrt{12} \cdot \ell/h = 24$ very small. The influence of second oder does not need to be checked. Equations (1) and (2) give

$$\vartheta_u = 3.5/6 \cdot 0.5 \cdot 0.0025 + (1 - 0.5/3.5) \cdot (0.0233 - 0.0025) = 0.0207 \text{ rad.}$$

$$N_u = (450^2 - 3000)\ mm^2 \cdot 30\ N/mm^2 + 3000\ mm^2 \cdot 400\ N/mm^2 = 7185\ kN$$

with $\lambda_p = \ell_p/h = 1$, $h/r_e = 0.0025$ (from design chart) and $h/r_u = 0.0233$.
An even higher ultimate angle could be reached if one provides the column with closely spaced stirrups over a larger length than has been assumed.
No failure occurs in a column subjected to a normal force N smaller than N_u = 7185 kN if the imposed angle is smaller than ϑ_u = 0.0207 rad (see Fig. 15). The assumption of a maximum mean crack width w_m = 0.2 mm leads to the limit angle ϑ_t for the serviceability state. The dotted line in Fig. 15 represents the limit angle so that no cracks occur in the column assuming $\sigma_c = f_{ct} = 2\ N/mm^2$.

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