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Serviceability and the Nonlinear Design of Concrete Structures

Aptitude au service de constructions en béton armé dimensionnées suivant des méthodes non linéaires

Gebrauchstauglichkeit von nichtlinear dimensionierten Stahlbetontragwerken

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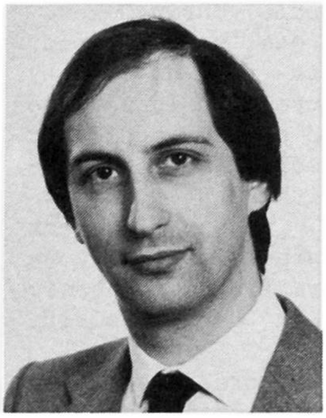

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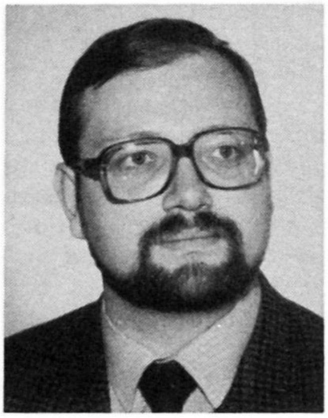

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SUMMARY

This paper deals with the serviceability of concrete structures designed by nonlinear methods. The authors consider a series of two-span reinforced beams, designed for rather extreme moment redistributions, but capable of carrying the same ultimate load. Their short-term serviceability is assessed by tests up to failure. An additional numerical analysis provides useful information about the behaviour under long-term sustained loading.

RÉSUMÉ

Cet article étudie l'aptitude au service de constructions en béton armé dimensionnées suivant des méthodes non linéaires. On envisage une série de poutres continues à deux travées, où l'armature est largement redistribuée entre sections critiques, mais qui possèdent toutes la même capacité portante. Leur comportement en service instantané est évalué par des essais jusqu'à rupture. Une analyse numérique complémentaire apporte des indications utiles au sujet de leur comportement à longterme sous charge de service.

ZUSAMMENFASSUNG

In diesem Aufsatz wird die Gebrauchstauglichkeit von nichtlinear dimensionierten Stahlbetontragwerken analysiert. Die Autoren untersuchen eine Reihe von Zweifelddurchlaufbalken mit jeweils gleichen Traglasten. Die Verteilung der Bewehrung entspricht verschiedenen Momentverläufen. Ihre kurzzeitige Gebrauchstauglichkeit wird in Versuchen bis zum Bruch gemessen. Eine zusätzliche numerische Analyse ergibt wertvolle Informationen über ihr Verhalten unter Dauerlast.

1. INTRODUCTION

Although all codes for the structural use of concrete allow the nonlinear design of statically indeterminate structures, this approach is not widely applied in common design practice, except for some countries. Probably this may be attributed to insufficient awareness of the potential benefits of the method and the concern for additional checks and calculations. However, nowadays several approaches with varying degree of sophistication are available :

a) limited moment redistribution within certain ductility conditions but without check of plastic rotation capacity ;
b) moment redistribution without specified limitations but requiring verification of plastic rotation capacity ;
c) nonlinear analysis involving computer programs based on realistic structural and material models.

Most methods are based on the plastic hinge concept as used in limit design of structures. The general principle of all nonlinear design methods is that in the ultimate limit state, a moment distribution is adopted which is different from the elastic distribution for all, or some, loading patterns. During several decades, a tremendous amount of experimental evidence has been gained, which justifies the application of this approach at least as far as safety related to failure is concerned. However, generally less attention is paid to the problem of serviceability for structures designed according to nonlinear methods. This aspect is of particular concern if large redistributions are adopted. A thorough analysis of this problem is important with respect to the practical applicability of simple nonlinear design methods. Moreover, crack widths and deflections also need to be evaluated under sustained loading conditions. Indeed, due to the effects of delayed concrete deformations, two basic phenomena have to be expected which could impair long term serviceability requirements :

a) The stress redistribution in a cross section leads to an increase of the steel stress in the tensile reinforcement. If in a critical section the margin between the bending moment under service loading and the yielding moment is rather small, the yield stress in the tensile reinforcement could be reached at long-term.
b) The initial redistribution of bending moments between critical sections, due to different stiffness characteristics along the structural members, will continue in course of time. The trend of the time-dependent redistribution will be determined by the heterogeneity of the viscoelastic properties of the structure. Due to this phenomenon, the steel stress in certain sections might increase considerably compared to the instantaneous value.

The number of papers dealing with long-term tests on continuous reinforced concrete beams appears to be rather limited ([1],[2]), which is probably due to the considerable experimental effort required for this kind of testing.

In order to investigate the phenomena quoted above in more detail, a combination of experimental and numerical results is presented in this paper. The instantaneous behaviour of a series of two-span beams, designed for rather extreme moment redistributions, is used as a starting point for the numerical calculation of the time-dependent behaviour under sustained loading. This dual approach results in useful conclusions concerning both instantaneous and long-term serviceability of statically indeterminate reinforced concrete beams.

2. ANALYTICAL APPROACH

Before dealing with the experimental and the numerical part of this paper, a simplified analytical approach of some aspects of the structural behaviour of

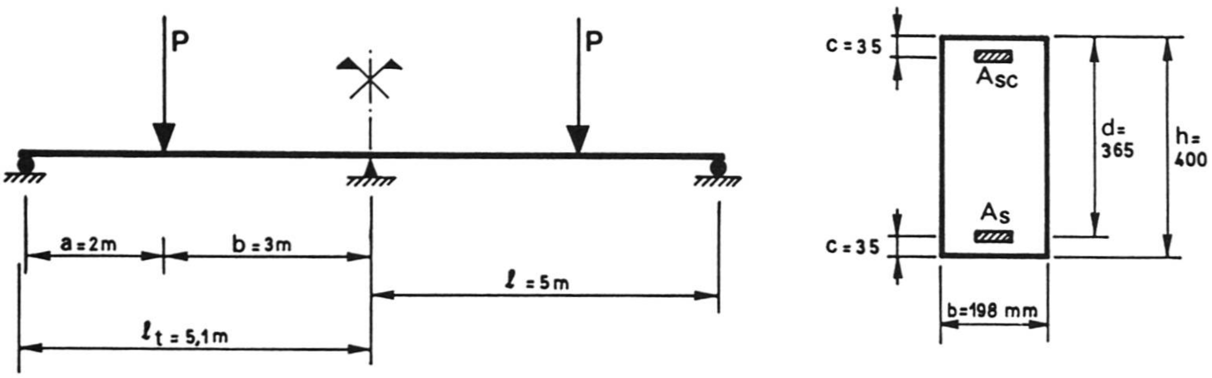


Fig. 1 Test set-up and dimensions of the beams tested

the tested beams is given. The dimensions of the beams, and the notations used are indicated in fig. 1. In the following, subscripts "sup" and "span" respectively refer to the critical support and span sections.

2.1. Ultimate load

Neglecting the beam's dead weight, the ultimate load P_u follows from the collapse mechanism :

$$P_u = (M_{u,span} \cdot L/a + M_{u,sup})/b \qquad (1)$$

For a fairly extensive range of reinforcement ratios, M_u is approximately proportional to A_s and hence

$$P_u \div (A_{s,span} \cdot L/a + A_{s,sup})/b \qquad (2)$$

From this relationship it follows that for a series of geometrically identical beams, designed for the same ultimate load, the amount of reinforcement in the critical sections may be chosen freely as long as the right hand member of eq. (2) remains constant. Of course, this conclusion holds from a merely statical point of view and restrictions related to rotation capacity and serviceability have to be checked additionally.

2.2. Moment distribution

In order to study the influence of an arbitrary arrangement of reinforcement sections on the moment distribution, we make use of a simplified structural model. It is considered that the beam has variable stiffness along its length, however limited to the assumption that the stiffness is constant between every two consecutive sections with zero bending moment (fig. 2). Introducing the following parameters

$$m = M_{sup}/M_{span} \quad \text{and} \quad k = K_{sup}/K_{span} \qquad (3)$$

where K stands for the bending stiffness, and expressing the compatibility of rotations, the following equation in m is found for a/b = 2/3 :

$$36\ m^3 + (45-8\ k)m^2 - 34\ km - 35\ k = 0 \qquad (4)$$

This relationship is depicted in fig. 3. The case k = 1 corresponds to the linear elastic moment distribution and yields m = 0.9722. By introducing the representative values of k, the moment redistributions for different loading stages are obtained, at least as long as the yielding moment is nowhere exceeded and within the restrictions of the model.

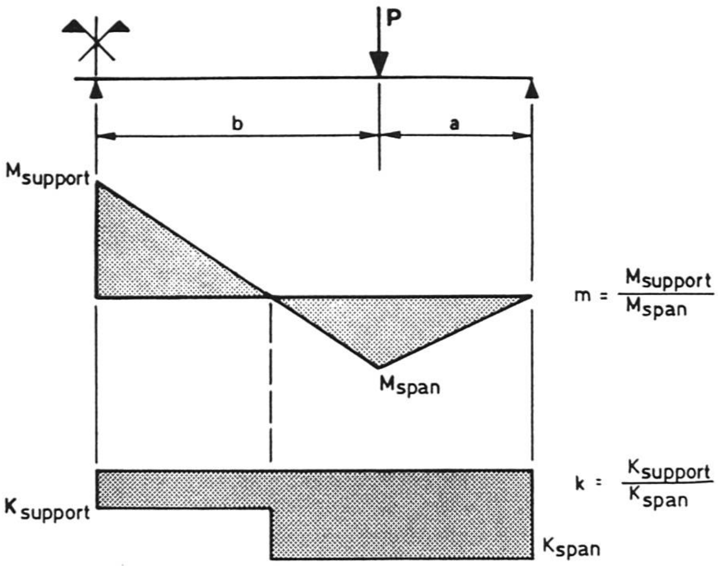


Fig. 2 Simplified analytical model

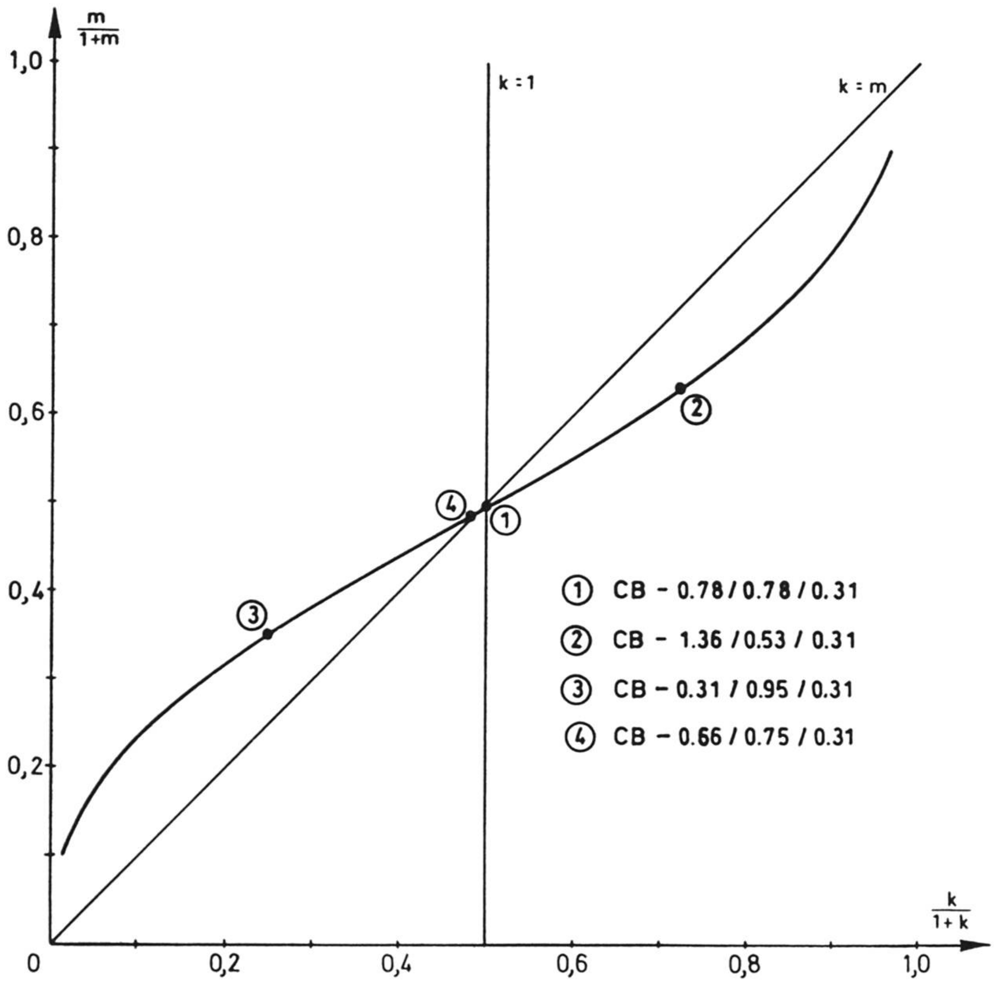


Fig. 3 Relationship between k and m for a/b = 2/3

In the fully cracked stage we may assume that $K \div A_s$ and hence

$$k \simeq A_{s,sup}/A_{s,span} \tag{5}$$

is a good approximation in this case.

2.3. Deflections

We consider a series of beams with arbitrary ratio $A_{s,sup}/A_{s,span}$, the right-hand side of (2) however remaining constant, in order to provide the same ultimate load. By using the simplified model presented in section 2.2, the deflection y of the sections under the point loads is calculated assuming the fully cracked state and the expression (5) for k. From the foregoing assumptions it can be shown that the quantity

$$C = K_{span}.(L/a + k)/b \tag{6}$$

is representative for a specified ultimate load P_u. From static equilibrium conditions it follows that

$$Pab = M_{span}\ (a.m + L) \tag{7}$$

Finally the deflection y, for a given load level P is given by

$$y = \frac{PL^2}{2C(1+\lambda)^2} \cdot \frac{1+\lambda(1+k)}{1+\lambda(1+m)} \cdot \left[\frac{m}{k}.\zeta(1-\frac{\zeta}{3}) - \frac{1}{3}(1-\zeta)^2\right] \tag{8}$$

where : - k and m are related through (4)
- $\lambda = a/b$
- $\zeta = m/(1+m)$

In fig. 4, the ratio y/y_{el} is shown, where y_{el} is the deflection corresponding to the elastic moment distribution i.e. k = 1. The curve indicates that for the same bearing capacity, the deflection of the beam is very little influenced by the particular arrangement of the reinforcement in the critical sections. This conclusion is important with respect to serviceability under short term static loading of continuous beams, designed by nonlinear methods. A discussion on approximate methods for the computation of short-term deflections may be found in [3].

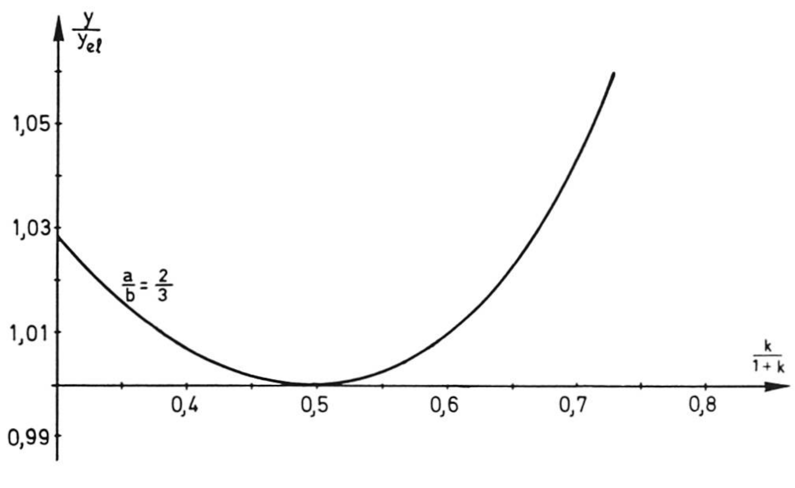


Fig. 4 Deflection of the beams in function of the stiffness ratio k

In the next sections, these preliminary findings will be explored in more detail by means of both an experimental and a numerical analysis which will yield more quantitative results.

3. EXPERIMENTAL PROGRAM

3.1. Survey of the test program

The experimental program was executed by the first author in the Magnel Laboratory for Reinforced Concrete, Ghent State University [4]. The five test specimens have the dimensions indicated in fig. 1. The beams are cast in contact with their final supports. At an age of 7 days they are subjected to their dead weight and at 28 days the static loading test is executed.

The concrete mix used has a W/C-ratio equal to 0.55. The strength values determined at 28 days are given in table 1. The reinforcing steel is of the type S400.

TABLE 1

Concrete strength data, ultimate loads, values of k and m

Beam	f_c (N/mm^2)	f_{ccub} (N/mm^2)	f_{ct}* (N/mm^2)	P_u (kN)	k**	m***
CB-0.78/0.78/0.31	28.3	33.9	3.63	115	1.00	0.97
CB-1.36/0.53/0.31	29.4	35.1	3.70	112	2.59	1.68
CB-0.31/0.95/0.31	27.7	33.8	3.66	112	0.33	0.53
CB-0.66/0.75/0.31	28.6	34.7	3.69	108	0.89	0.91
CB-0.66/0.75/0.87	29.4	36.8	3.46	115	0.89	0.91

* f_{ct} : flexural tensile strength determined on prisms 150x150x600 mm
** $k = A_{s,sup}/A_{s,span}$
*** m according to (4)

The loading arrangement is indicated in fig. 1. The point loads are applied by means of hydraulic actuators. The support reactions are measured with calibrated electronic load cells.

The beam designations are composed of the notation CB for "continuous beam" and the values of 100 $\rho_{s,sup}$, 100 $\rho_{s,span}$ and 100 ρ_{sc} respectively.

Beam CB-0.78/0.78/0.31 is reinforced according to the linear elastic moment distribution. For the particular loading arrangement adopted, the maximum support and span moments are almost equal, which means that the cracking moment and the stiffness in the different stages, are constant along this beam.

For the other beams, the ratio $A_{s,sup}/A_{s,span}$ is varied between rather wide limits, but the absolute values are determined in such a way that the same ultimate load for the different beams is reached according to eq. (2). For beam CB-1.36/0.53/0.31 the longitudinal reinforcement is concentrated at the central support while for beam CB-0.31/0.95/0.31 more reinforcing steel is placed in the span sections. Beams CB-0.66/0.75/0.31 and CB-0.66/0.75/0.87 differ only in the amount of compression reinforcement which is quite large in the latter case.

The values of k, calculated by the approximation (5), and the corresponding values of m obtained by solving (4) are mentioned in table 1. These values allow to locate the characteristic point of each beam on the curve depicted in fig. 3.

Over a length of about 2d to the left and to the right of the three critical sections, closed stirrups (steel S220, Ø 10 mm) are provided with a spacing of 150 mm. Outside these regions, stirrup spacing is adapted to the local magnitude of the shear force.

The choice of the beam dimensions and the location of the point loads result in very low shear effects on the structural behaviour.

3.2. Test results

The ultimate loads of the beams are mentioned in table 1. The values are almost identical, as intended. Fig. 5 shows beam CB-0.78/0.78/0.31 at the load level P = 110 kN. The distribution of the bending moments is given in figs. 6 and 7 for beams CB-1.36/0.53/0.31 and CB-0.31/0.95/0.31. For these cases, the redistribution of bending moments compared to the linear elastic behaviour is quite considerable and the formation of the first plastic hinge is clearly visible in figs. 6 and 7, as well as in fig. 8 where the deflections of the beams are compared. For the central section of beam CB-1.36/0.53/0.31, the load at which the reinforcement starts yielding is only 16 % higher than the service load P = 45 kN. This service load is obtained from the design values of the resisting moments in the critical sections of beam CB-0.78/0.78/0.31. For all beams, an excellent agreement is obtained between the experimental ultimate loads and those calculated by introducing in (1) the calculated ultimate moments. This indicates that for all cases a complete moment redistribution could take place, which was confirmed by an analytical verification of the occuring plastic rotations. In all cases the tensile steel is in the strain-hardening range at P_u.

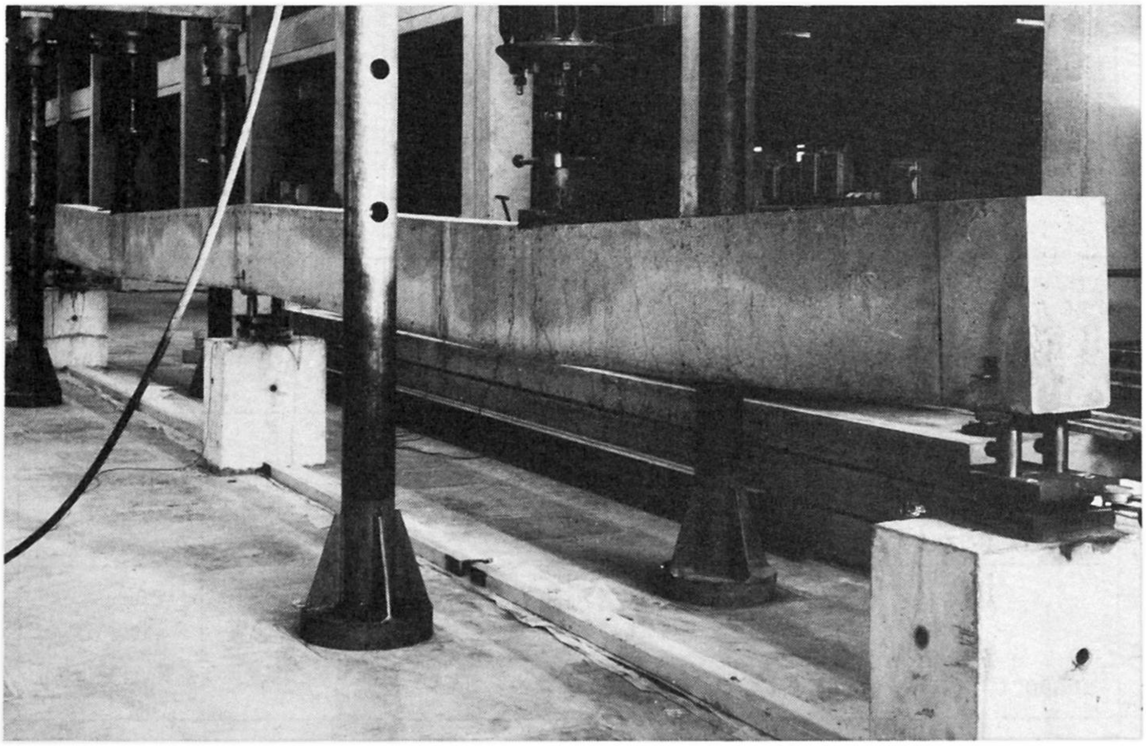


Fig. 5 Beam CB-0.78/0.78/0.31 at P = 110 kN

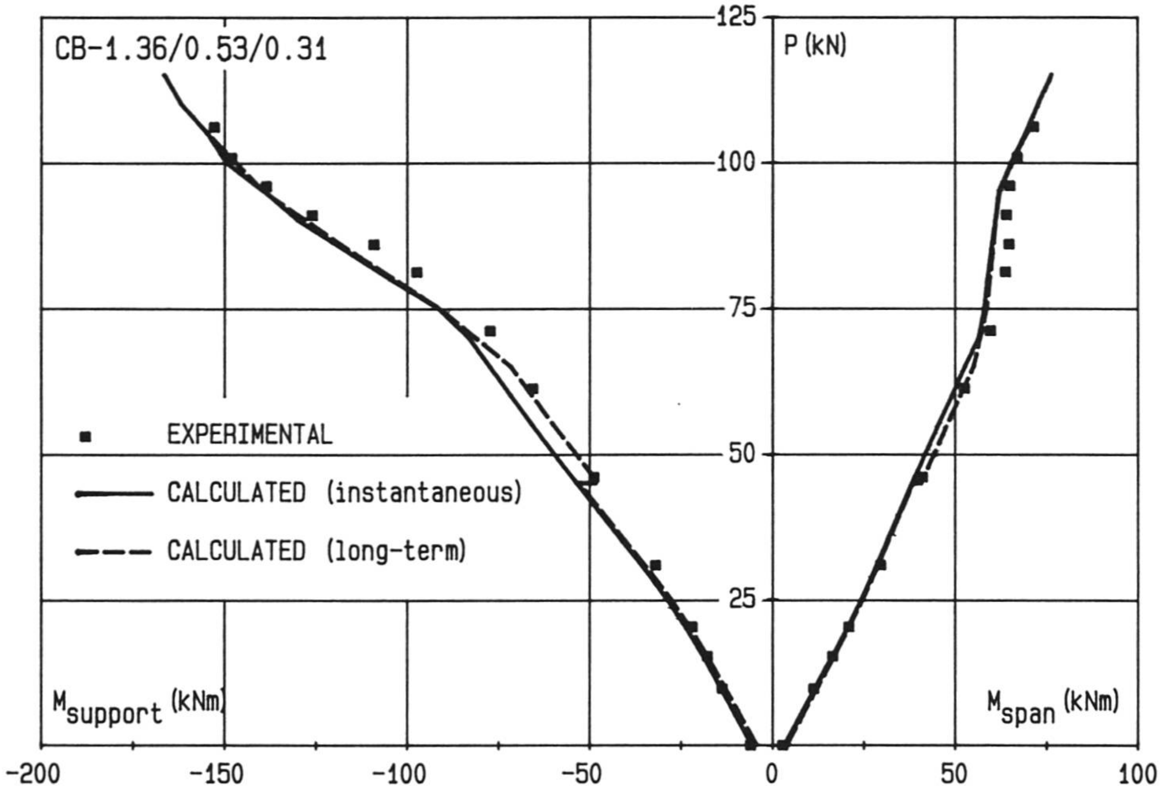


Fig. 6 Evolution of bending moments for beam CB-1.36/0.53/0.31

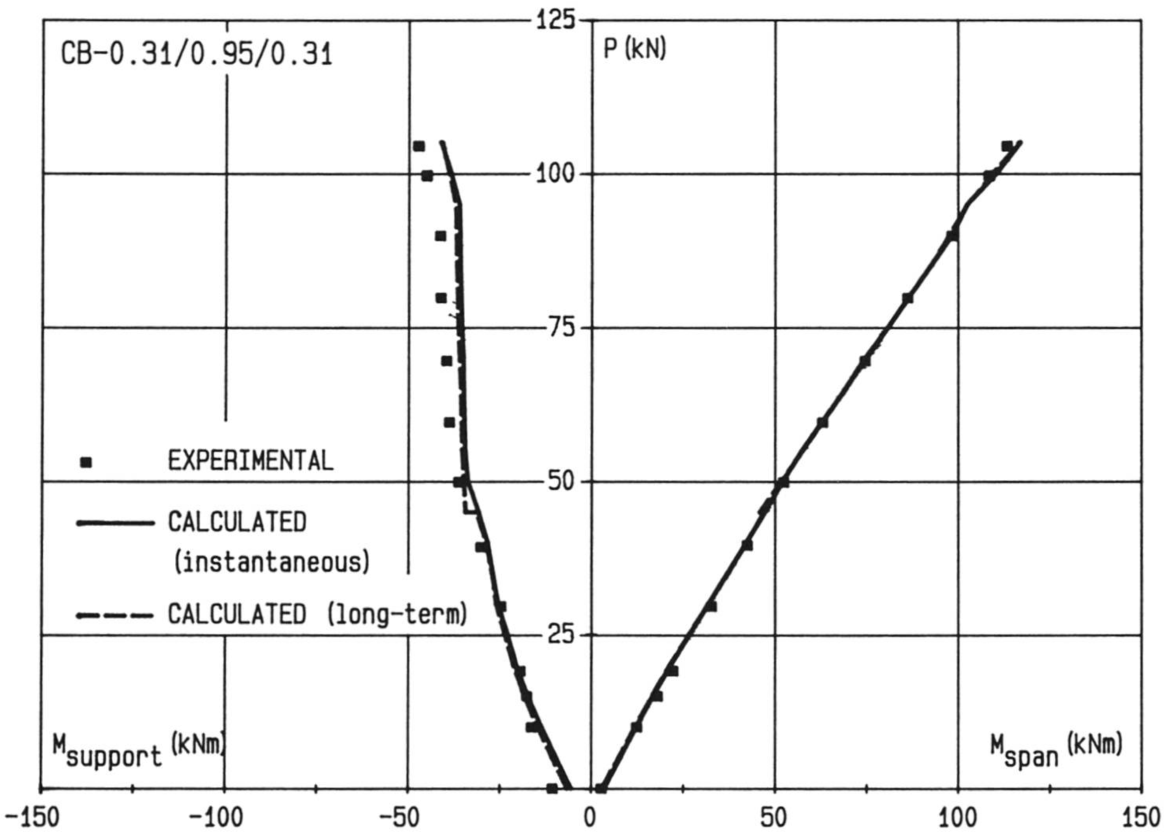


Fig. 7 Evolution of bending moments for beam CB-0.31/0.95/0.31

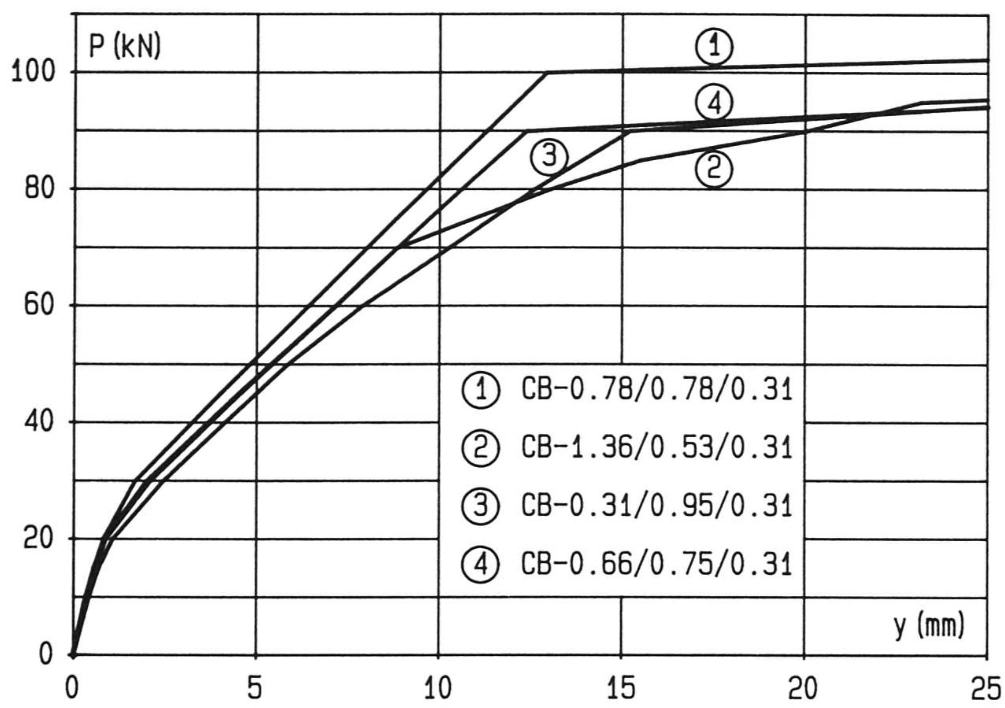


Fig. 8 Comparison of experimental deflection curves

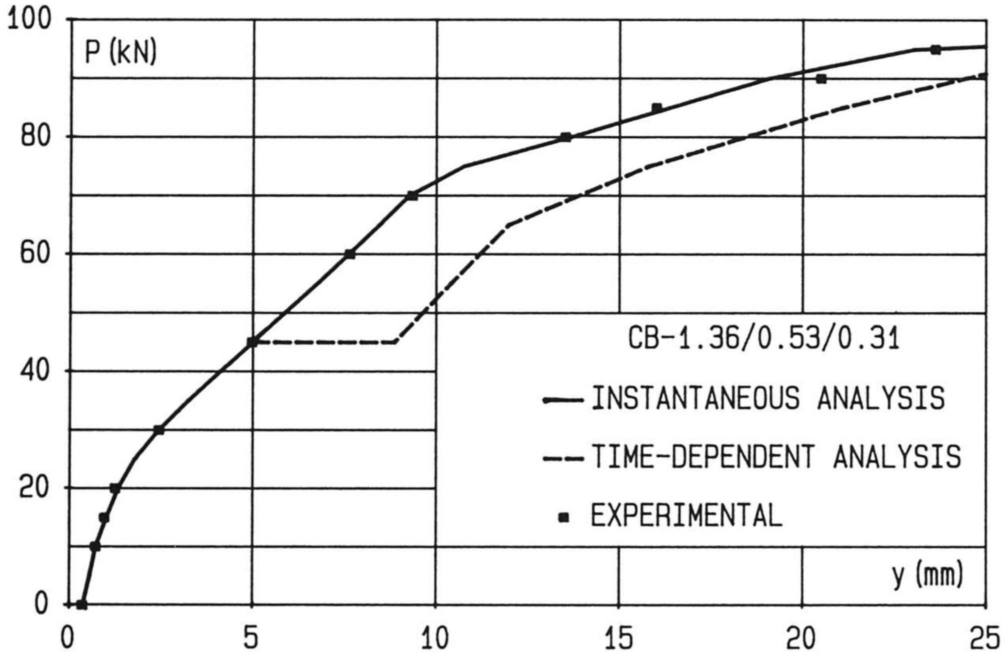


Fig. 9 Deflection of beam CB-1.36/0.53/0.31

TABLE 2

Observed maximum crack widths and calculated steel stresses

Beam	Section	w_{max} (mm)	σ_s (N/mm^2) t_o (28 days)	t_o+1 (29 days)
CB-0.78/0.78/0.31	support	0.10	195	204
	span	0.08	185	195
CB-1.36/0.53/0.31	support	0.10	137	144
	span	0.14	268	282
CB-0.31/0.95/0.31	support	0.25	414	431
	span	0.11	172	180
CB-0.66/0.75/0.31	support	0.13	229	240
	span	0.12	199	209
CB-0.66/0.75/0.87	support	0.14	230	240
	span	0.12	189	198

As appears from fig. 8, the deflections of the different beams are fairly comparable as long as $M < M_y$, which was predicted in section 2.3. In all cases they appear to be lower than L/1000 at service load.

In table 2 the observed crack widths at P = 45 kN are mentioned. All these values are acceptable for most applications.

4. NUMERICAL ANALYSIS

4.1. Numerical model

The numerical model used to study the behaviour of reinforced concrete structures is a nonlinear plane frame analysis program based on the standard displacement type finite element method. The beam elements are of the Navier-Bernoulli type with 9 degrees of freedom (fig. 10). The general description and some applications of this program may be found in [5], [6] and [7]. The program was developed by the second author at the Free University of Brussels.

Material nonlinearities are modelled in the concrete compression zone by the adoption of the usual parabola-rectangle stress-strain relationship (concrete strengths f_c reported in table 1 reached at $\epsilon_c = 0.002$).

In the tensile zone, tension stiffening effects are modelled with an average stress-strain relationship (smeared crack approach) where the tensile strength of concrete is an apparent value that falls with increasing tensile steel strain. This method, originally suggested by Quast [8], is now proposed in Chapter 14 of the next edition of the CEB-FIP Model Code [9]. In this study, concrete is assumed to crack under a tensile strain equal to 0.000125.

The actual behaviour of the reinforcement is represented by a multisegmental σ-ϵ law for the steel with : $f_y = 440$ N/mm^2, $E_s = 200$ kN/mm^2, yield plateau ending at $\epsilon_p = 1$ % and strain hardening modulus equal to $E_s/500$.

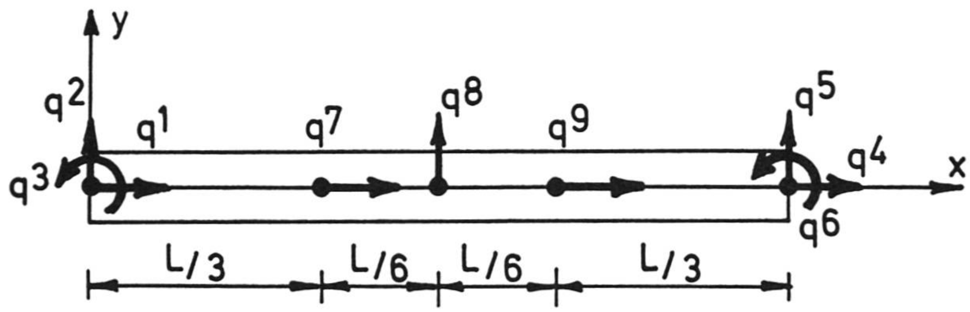


Fig. 10 Beam element with 9 degrees of freedom

The time-dependent effects due to the delayed deformations of concrete are rigorously treated within a viscoelastic analysis based on the numerical evaluation of the heriditary integral of the superposition principle with a trapezoidal rule [10]. The prediction model used in this analysis for the delayed deformations of concrete is in accordance with the CEB-FIP Model Code 1978, with constant environmental conditions at 60 % R.H. and T = 20°C.

Due to symmetry, only one span needs to be considered. The span is modelled by 7 elements in the case of the beams with ρ_{sc} = 0.31 % and by 8 elements in the case of CB-0.66/0.75/0.87.

4.2. Loading history

For each of the continuous beams previously mentioned, two types of analysis were run without considering geometrical nonlinearities :

- An "instantaneous" analysis representing the monotonous loading up to failure and formation of the plastic mechanism. This analysis may be seen as a numerical simulation of the loading tests.
- A "time-dependent" analysis whose scope is to evaluate the effects of sustained loading on the long-term serviceability and on the long-term carrying capacity in a statically indeterminate concrete structure. Its loading scheme is as follows :
  1°) creep and shrinkage of concrete begin at an age of one day ;
  2°) application of the dead load at the age of 7 days ;
  3°) application of the service load P = 45 kN at 28 days ;
  4°) sustained loading during 10000 days ;
  5°) increase of the load P (without prior unloading) up to the plastic and the ultimate stages at the age of 10028 days.

4.3. Numerical results

In fig. 9 the evolution of the deflection at the point loads is indicated for beam CB-1.36/0.53/0.31 and compared with the calculated values. Notheworthy is the close agreement between the results of the instantaneous analysis and the experimental curve. Figure 11 gives the time-dependent increase of the displacement of the point loaded sections during the sustained loading stage.

In figures 6 and 7, the distribution of the bending moments in the critical sections is plotted as a function of the external load P for the two extreme cases of reinforcement distribution, i.e. beams CB-1.36/0.53/0.31 and CB-0.31/0.95/0.31. The results of the time-dependent analysis are also indicated. As expected, the redistribution of the bending moments (both at short- and long-term and even at failure) is negligible in the case CB-0.78/0.78/0.31. It is also very small for beams CB-0.66/0.75/0.31 & 0.87 and from a redistribution point of view, the difference between CB-0.66/0.75/0.31 and CB-0.66/0.75/0.87 is hardly noticeable.

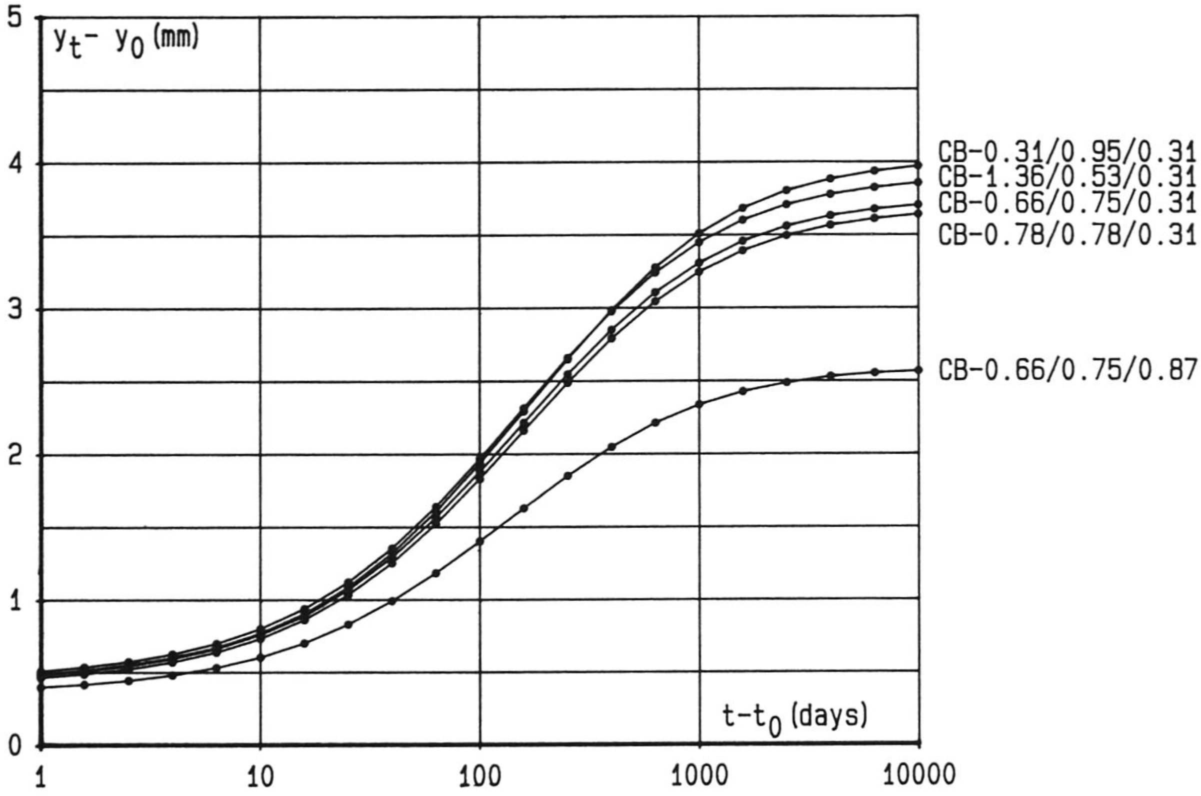


Fig. 11 Calculated evolution of the increase in deflection in function of time

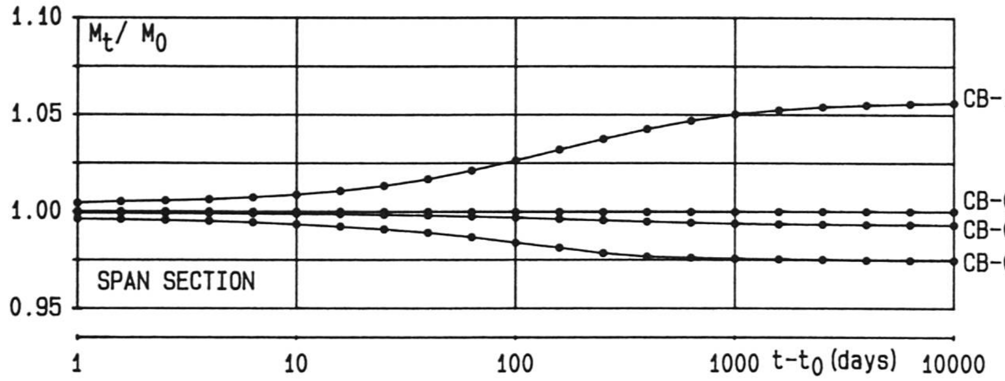
Figure 12 represents the relative increase of the bending moments acting in the critical sections during the sustained loading phase. The cases of the three beams CB-0.78/0.78/0.31 and CB-0.66/0.75/0.31 & 0.87 have just been mentioned. For the two remaining beams, the time-dependent redistribution of the bending moment amounts up to 10 % during the sustained loading stage.

Figure 13 shows the time-dependent evolution of the tensile steel stresses in the critical sections during the sustained loading phase. Obviously, the plastic design leads to a large range of initial stress levels under service load as appears from table 2. In fact, in the case of beam CB-0.31/0.95/0.31, the service load level lies just below the loading level corresponding, in the instantaneous analysis, to the yielding of the reinforcement of the support section. The time-dependent analysis even predicts that yielding of this reinforcement will occur after only a few days of sustained loading.

5. DISCUSSION OF RESULTS

5.1. Serviceability and plastic design

Examination of figures 6 and 7 shows that the "instantaneous" distribution of bending moments is reached again after a few load steps when loading resumes after 10000 days even when a large redistribution of reinforcement (which has resulted in a time-dependent redistribution of bending moments between critical sections) is adopted. The plastic behaviour is hardly influenced by the redis-



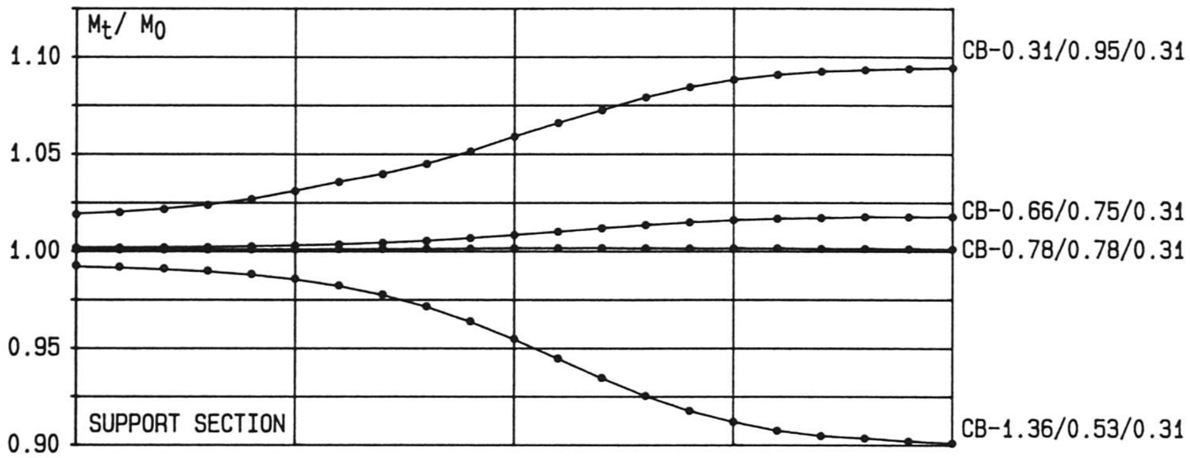


Fig. 12 Calculated relative variation of bending moments

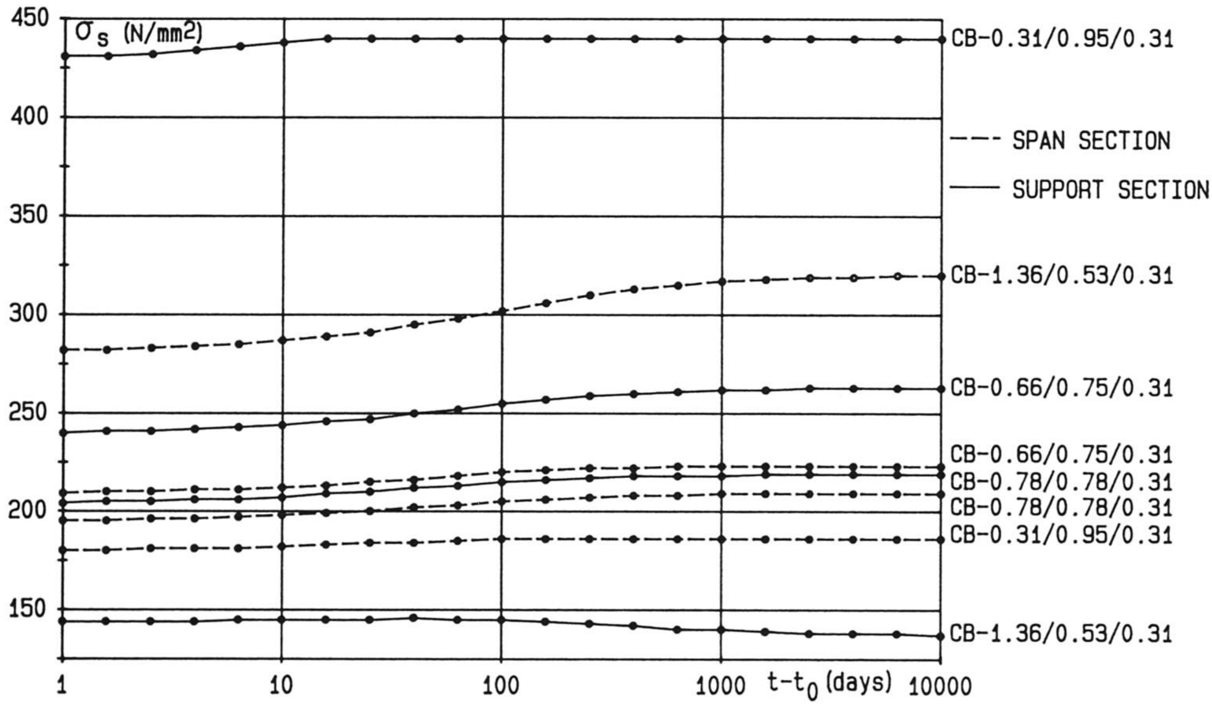


Fig. 13 Calculated evolution of steel stress in function of time

tribution which has occured in the cracked phase under sustained loading. This logically follows from the principle of insensitivity of the plastic load to initial stresses.

Of more fundamental nature is the observation that in an extreme case of design like CB-0.31/0.95/0.31, which results in the formation of a plastic hinge under long-term service load (fig. 13), yielding of the reinforcement does not impair the serviceability requirements in the sense that no abnormal increase may be noticed, neither in the evolution of the redistribution of the bending moment (fig. 12), nor in the evolution of the displacements (fig. 11). This is due to the fact that the rotation in this plastic hinge remains limited because it is also controlled by the deformation of the other critical sections. It may be suggested that if compliance with crack width limitations is ensured at the beginning of the sustained loading, this will still be the case at long-term.

The distribution of the reinforcement as such has little influence on the long-term displacements after 10000 days of sustained loading under service conditions. This confirms a theoretical analysis presented in [11] for a single span beam clamped at both ends. Examination of figure 11 also suggests that the evaluation of the long-term displacements could be obtained from the estimation of the short-term displacements multiplied by a function whose sole parameters are the time-dependent development of the creep coefficient and the compressive reinforcement ratio. This so-called "long-term multiplier" approach is adopted in the ACI 318-83 Building Code for Concrete Structures.

We have previously observed that there is little - if any - difference between the beams CB-0.66/0.75/0.31 and CB-0.66/0.75/0.87 regarding the redistribution aspect. Moreover, it is well known that the compressive steel does not significantly increase the carrying capacity of the structure but mainly its ductility at failure. The compressive steel will also contribute to reduce the long-term displacements (fig. 11) although this aspect should not be overestimated, particularly from an economical point of view.

5.2. Trend of the time-dependent moment and stress redistribution

The non-uniform distribution of reinforcement creates viscoelastic heterogeneities, which cause a time-dependent redistribution of internal forces. It is now commonly accepted, although not widely known, that the trend of this redistribution corresponds to a transfer of internal forces from parts with a high creep rate to parts with a lower creep rate [10].

In the fully cracked state the flexural stiffness of a reinforced concrete sections is given by

$$K = E_s . A_s . d^2 \, (1 - \xi) \, (1 - \xi/3) \qquad (9)$$

where ξ is an increasing function of the product $\alpha\rho$ with $\alpha = E_s/E_c$. In course of time, (effective modulus approach) α and ξ increase whereas K decreases. However, the larger ρ, the faster K will decrease in function of α, which may be considered as an equivalent time variable. It follows that high creep rates correspond to large values of ρ (high stiffness) and low creep rates (low stiffness) to small values of ρ. Consequently, if initially $k < 1$, it will increase due to creep effects and if $k > 1$ at $t = 0$, it will decrease in course of time. From equation (4) and the curve represented in fig. 3, it follows that the same trends will hold for m. For example, considering beam CB-1.36/0.53/0.31, for which initially $k = 2.59$ (see table 1), it follows that k and $m = M_{sup}/M_{span}$ will decrease. Hence the support section is unloaded and the span moments increase in course of time (see fig. 6). The inverse phenomenon occurs for beam CB-0.31/0.95/0.31.

Concluding, it may be stated that the time-dependent redistribution results in a moment transfer from sections with heavy tensile reinforcement to sections with small reinforcement ratio, provided that possible differences in the amounts of compressive reinforcement do not alter significantly the creep rate of the cross sections. Consequently, the time-dependent redistribution partly compensates the instantaneous redistribution which follows from cracking effects and results in a moment transfer from regions with low reinforcement ratio (low stiffness) to regions with high reinforcement ratio (high stiffness).

The aforementioned principle of redistribution of internal forces due to the viscoelastic behaviour of concrete is only valid, strictly speaking, for creep effects. In particular, it cannot serve to explain the redistributions which have been observed in the long-term experiments on reinforced concrete beams mentioned in [1] and [2]. For these cases it is quite easy to show - which has not been done up to now - that the main part of the redistribution is due to shrinkage of concrete and not to creep effects.

The foregoing conclusions cannot be applied without more to the steel stresses σ_s because from the general expression

$$\sigma_s = \frac{M}{zA_s} \qquad (10)$$

where z is the lever arm of the internal stress resultants, it follows that both variations of M and z influence σ_s. Creep effects give rise to a decrease of z in course of time which in its turn results in an increase of σ_s. For sections with increasing moment due to the time-dependent moment redistribution, both effects contribute in the same sense to an increasing steel stress. Even for most unloading sections, the change of z is predominant on the change of M and increasing steel stresses result. Only for the support section of beam CB-1.36/0.53/0.31, a decrease of σ_s at long-term is observed (fig. 13).

6. CONCLUSIONS

- The purpose of this study was to gain some insights in the serviceability of statically indetermined reinforced concrete structures designed by nonlinear methods.

- The analysis concerned a series of two-span beams, reinforced with varying amounts of reinforcement in the spans and on the support, but all characterized by the same ultimate load.

- Although the redistribution with respect to the elastic values was quite large, the observed behaviour of the beams under short-term loading tests showed that the instantaneous serviceability requirements were fulfilled. Noteworthy is the fact that the deflection under service load appears to be almost insensitive to the reinforcement distribution adopted. This experimental finding is in accordance with simple analytical predictions.

- In order to investigate serviceability under long-term loading, a numerical analysis was performed. Due to viscoelastic heterogeneities, a time-dependent redistribution of the bending moments between the critical sections takes place. This redistribution results in a moment transfer from parts with a high creep rate to parts with a lower creep rate. It partly compensates the redistribution of the bending moments which has followed from cracking during the instantaneous loading stage.

- Within the sections, a time-dependent redistribution of stresses between concrete and steel generally results in an increase of tensile steel stresses. This increase never exceeded 15 %. In one extreme case of redistribution of the reinforcement, yielding of the tensile reinforcement occured under sustained loading in one critical section.

- It was observed that also the long-term deflections seem to be almost independent of the adopted reinforcement distribution.

- It results that even large redistributions of the reinforcement generally do not impair the serviceability of continuous beams, neither under short-term nor sustained loading.

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