Design of deep concrete members subjected to restraint

S.Liphardt
König und Heunisch, Beratende Ingenieure, Frankfurt, FR Germany
W.Liphardt
Kraftwerk Union AG, Erlangen, FR Germany

1 INTRODUCTION

In the past often heavy reinforcement was provided for deep reinforced concrete members. It was argued in many cases that restraint called for this heavy reinforcement. But it is obvious that this requirement is the result of the unsuitable attempt to consider restraint as a static problem.

Restraint is a geometric problem. In reinforced concrete members compatibility for the imposed volume changes is achieved by formation of cracks. Since cracking reduces the stiffness of the member, restraint usually results in the formation of a very few cracks: well below the number required for a stabilized crack pattern. Thus the design of mainly restrained members has to focus on preventing uncontrolled cracking. The primary function of reinforcement is to maintain sufficient tensile force at existing cracks to assist the existing restraint in forcing development of additional cracks to balance the change in volume. As a result, the design has to provide a minimum reinforcement at least capable of withstanding the forces which develop at the formation of the second and subsequent cracks. In deep members the minimum reinforcement may be less than that one required for withstanding the tensile stresses induced in concrete just before first cracking occurs.

2 EFFECT OF RESTRAINT ON STRENGTH AND SERVICEABILITY

For ideal elastic - ideal plastic material behaviour two major aspects of restraint are important:

1. Restraint does not influence the load carrying capacity but increases the amount of plastic deformations required to form the corresponding kinematic chain. Thus restraint has to be considered for verification of strength only if ductility of the structure seems to be insufficient.
2. Restraint causes redistribution of internal forces which can lead to plastic deformations even under service loads. Thus restraint has to be considered for verification of serviceability.

Reinforced concrete members are distinguished by a peculiarity which may be illustrated for a member under axial tension (Figure 1):

The load deformation behaviour shows two "yield plateaus", of which the first one is due to primary cracking (development of stabilized crack pattern) and the second one due to yielding of reinforcement. Hence it follows that contrary to ideal elastic - ideal plastic material behaviour

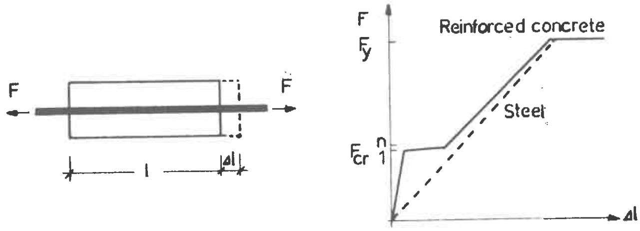

Figure 1. Load deformation behaviour of reinforced concrete member under tension.

redistribution of internal forces and reduction of forces due to restraint already take place under working load as cracking occurs at least in the extremely stressed regions. To take advantage of this behaviour the minimum requirement for design of reinforced concrete members should be formulated as follows:

The "plastic" deformations of the first yield plateau which develop due to opening of cracks should possibly be formed by a large number of small contributions, thereby avoiding uncontrolled large cracks. In the case of deep members, this requirement should be effective for the member surface whereas in the inner part of the member in most cases broader collective cracks would be acceptable.

Based on this minimum requirement design rules can be derived, which allow to adjust the bar diameter in relation to required deformations.

3 DESIGN CONCEPT

For the design of reinforced concrete members it should be distinguished between members mainly subjected to external loading and those mainly subjected to restraint.

Members mainly subjected to external loading are characterized by a stabilized crack pattern over a greater region. In this case the reduction of stiffness due to cracking ensures that additional forces due to restraint remain small. Thus a design based on external loading only is sufficient. In general the crack width also remain under control without increase of reinforcement if suitable bar diameters are provided because steel strains do not grow remarkably due to restraint.

Those members where the external loading results in internal forces less than those required to initiate cracking are known as members mainly subjected to restraint. A design based on external loading only is insufficient because control of cracking requires a minimum reinforcement. This reinforcement has to prevent that change of volume is balanced by only one single crack. It should be able to maintain a tensile force sufficient to force the development of secondary cracks (Figure 2). To achieve this, it is necessary to know the cause of restraint and its re-

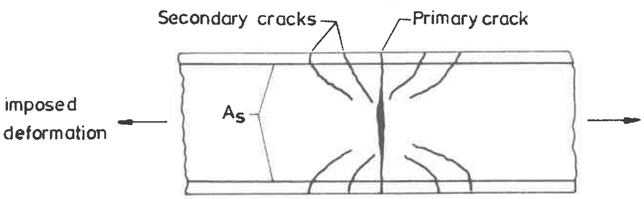

Figure 2. Crack development in deep members.

lation to the increase of stresses in steel.

Based on the relations mentioned before a design concept can be derived comprising the following 3 steps:

1. Definition of design forces in order to determine the minimum reinforcement for members mainly subjected to restraint. In case of members mainly subjected to loads, design is carried out for the loading forces.
2. Determination of the relationship between the cause of restraint and steel stresses for members designed according to step 1.
3. Description of the relationship between steel stresses and the requirements for crack control in form of design rules (e.g. maximum bar diameter for a given steel stress).

The design concept will be explained considering a member subject to bending due to restraint (e.g. linear temperature gradient) in combination with bending (M_L) and axial force (N_L) due to external loads. The concept has to fullfill the following conditions:

Under pure restraint the ultimate design moment Mu has to achieve the desired control of cracks on the member surface. This means that in a shallow member the internal forces corresponding to cracking (e.g. Mcr) have to be resisted with a steel stress aspired with respect to crack control. For deep members Mu may be less than Mcr (see chapter 4).

If external loading is dominant the ultimate design moment shall be based on external loading only neglecting restraint.

For conditions between these two limiting cases Mu will be interpolated linearly (Figure 3).

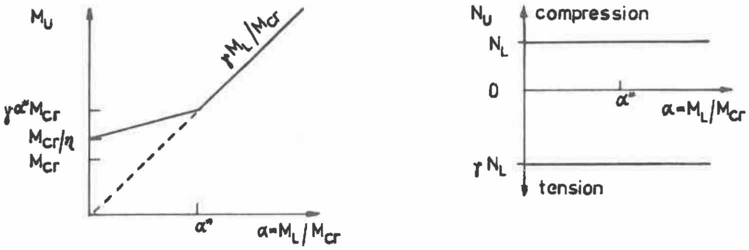

Figure 3. Design forces Mu, Nu as a function of M_L/Mcr.

For ductile members mainly under restraint ($M_L < \alpha^*$Mcr) this results in

(1) $\quad Mu = Mcr/\eta + (\gamma - 1/(\alpha^*\eta))\, M_L$

(2) $\quad Nu = N_L$ (compression), γN_L (tension)

and for members mainly under external loading ($M_L > \alpha^*$Mcr):

(3) $\quad Mu = \gamma\, M_L$

(4) $\quad Nu = N_L$ (compression), γN_L (tension)

where γ = safety factor (e.g. 1.75 acc. to DIN 1045 or $\gamma = \gamma_f \cdot \gamma_m$ acc. to CEB), α^* = minimum ratio chosen to neglect restraint, η = ratio of steel stress σ_s aspired with respect to crack width and the yield strength fy. The cracking moment Mcr is given by

(5) $\quad Mcr = (fct + N_L/Ac)\, Wb$

where fct=modulus of rupture of concrete, Ac=gross area of section and Wb=section modulus.

The concept is checked considering a beam built in at both ends under a uniform restraint moment due to $\Delta\vartheta$ along its length (Figure 4). M_L due to

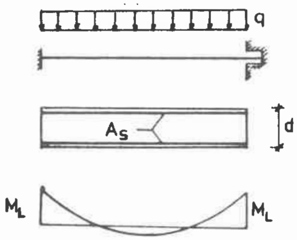

Figure 4. System and loading.

external loading is varied to check the cases of mainly imposed deformation and mainly imposed loading. Maximum steel stresses σ_s due to loading (M_L) and restraint ($\Delta\vartheta$) are calculated and shown in figure 5. Based on these results it can be concluded:

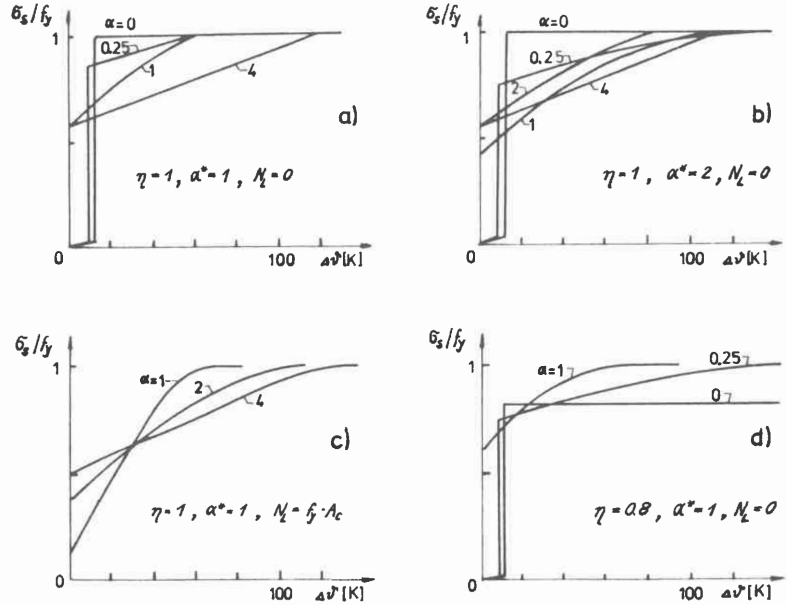

Figure 5. Steel stresses due to restraint for different design conditions

Members mainly under restraint behave unfavorably with respect to the increase in steel stresses with increasing $\Delta\vartheta$ when compared with members mainly under load.

Members mainly under restraint can take a $\Delta\vartheta$ of about 60K by choosing α^*=1 and η =1 before fy is reached (Figure 5a).

A greater $\Delta\vartheta$ will be allowed if α^* is increased, e.g. for α^*=2 $\Delta\vartheta$ goes up to 80K before fy is reached (Figure 5b).

If the steel stress σ_s is restricted with respect to crack width $\Delta\vartheta$ will decrease (Figure 5 d).

Due to a compressive axial force additional steel stresses due to restraint will decrease for a given $\Delta\vartheta$, however maximum $\Delta\vartheta$ will not increase for a given σ_s (Figure 5c).

A similar investigation performed for a beam clamped on one end showed that in this case Mu should be increased by about 25% for M_L<0.25 Mcr to achieve satisfactory results.

Based on a systematic evaluation limit values α^* depending on stress ratio η and amount of restraint ($\Delta\vartheta$) are given in figure 6.

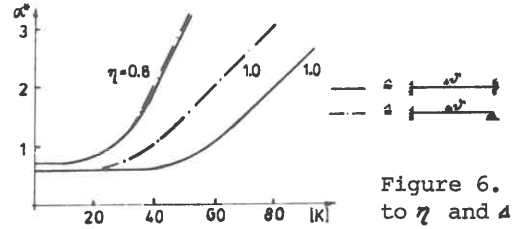

Figure 6. Limit value α^* in relation to η and $\Delta\vartheta$.

The basic idea of the design concept is to begin with the determination of stress ratio σ_s/fy with respect to the desired crack width, than α^*can be determined as a function of restraint (e.g. $\Delta\vartheta$) and design can be performed.

4 MINIMUM REINFORCEMENT

For deep members it seems unnecessary to determine the minimum reinforcement with respect to the cracking forces of the gross section, especially in those cases where only a limited amount of deformability is required (e.g. wall restrained by the base). In these cases it is sufficient to provide a minimum reinforcement related to a reduced concrete area in tension Act,eff, which corresponds to the cracking forces of secondary cracks (Figure 2).

A series of 4 tentative test was performed to study the behaviour of deep members with reduced minimum reinforcement. According to figure 7

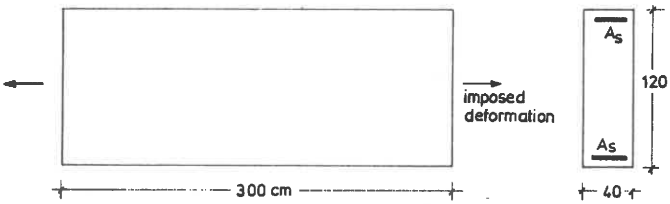

Figure 7. Specimen tested for imposed deformations.

specimen representing e.g. a part of a 1.2m thick wall were tested under imposed axial deformations. Only a surface reinforcement BSt 500/550 was provided on both faces to eliminate disturbances by stirrups. Reinforcement was between 50% and 100% of minimum reinforcement based on gross section, bar diameter were Ø6 and Ø16mm. The specimen tested are given in table 1. Proper curing and control of temperature gradient after

Table 1. Tentative test series.

Specimen	Minimum reinforcement	Bar diameter	Reinforcement ratio
VK1	100%	16	0.34%
VK2	50%	16	0.17%
VK3	70%	6	0.25%
VK4	50%	6	0.18%

casting avoided strength reducing effects due to hydration temperatures. Tests started when concrete tensile strength reached strength according to 100% minimum reinforcement.

The crack pattern of the 4 tests are shown in figure 8. The first specimen (VK1) with 100% minimum reinforcement and bars Ø16mm showed

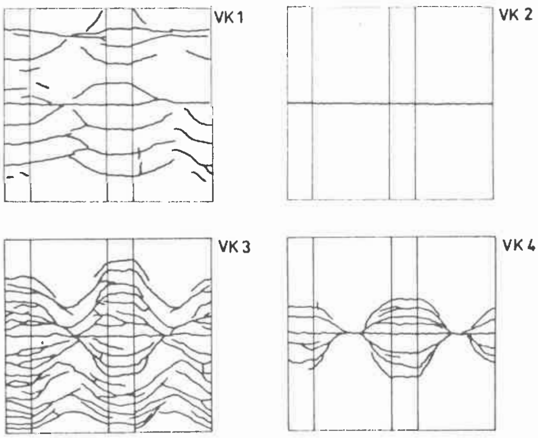

Figure 8. Crack pattern of the test specimen VK1 - VK4.

an acceptable development of cracks whereas with 50% of minimum reinforcement and the same bar diameter due to the long embedment length no secondary cracks developed (VK2). The results for VK3 show that 70% of minimum reinforcement in combination with bar diameter Ø6 caused so many and deep secondary cracks that further primary cracks developed at a lower force level compared to the first crack. A limit was reached providing only 50% of minimum reinforcement in combination with bar diameter Ø6 (VK4). Now only one bunch of secondary cracks developed.

Based on these results it can be stated that the area Act,eff is a function of the development length of reinforcement. Thus bar diameter, concrete area Act,eff and reinforcement ratio are coupled.

The work is still going on with an additional test series at which the thickness of member is increased (up to 2.4m) and additional bar diameters are considered.