RESULTS OF THE NEW REINFORCING TECHNOLOGY PROJECT

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ABSTRACT

This paper presents results of the TEKES-project "A New reinforcing technology", which studied the geometrical model in punching shear of reinforced concrete slabs, need for secondary reinforcement in one-way reinforced concrete slabs, plastic deformation (rotation) capacity of reinforced concrete slabs with cold-formed reinforcing bars, and anchorage of reinforcement.

Keywords: Punching shear capacity, critical section, the geometry of the punching cone, concrete slabs, secondary reinforcement, ductility of reinforcement, deformation capacity, anchorage capacity.

1. GEOMETRICAL MODEL IN PUNCHING SHEAR OF REINFORCED CONCRETE SLABS

1.1 AIM OF THE RESEARCH AND TEST ARRANGEMENTS

The main reason for the research was a large difference between the calculation results according to different code formulas of the punching shear capacity of reinforced column-slabs without shear reinforcement, e.g. between the Building Code of Finland and Eurocode 2. In most concrete codes the critical section of the punching formula is set to different distances from the column, varying between 0.5d – 2.0d. For instance, in Eurocode 2 this critical section distance value is 1.5d , in CEB-FIP model code (MC 90) 2d and in the Building Code of Finland 0.5d. The aim of this study was to solve the problem of the distance of the suitable critical section in a calculation model. Ten square concrete slabs (L1-L10) were tested. The depth of the slabs was 200 mm. The column sizes were 200 mm, 400 mm and 900 mm. The compressive strengths were 32.7 MPa and 24.1 MPa (mean value) for slabs L1-L6 and L7-L10, respectively. The amounts of flexural reinforcement in the slabs are given in Table 1 (steel grade A500HW). Testing arrangements are shown in Figure 1.

The effects of the flexural reinforcement and the effect of the column diameter on a slab without shear reinforcement were studied.

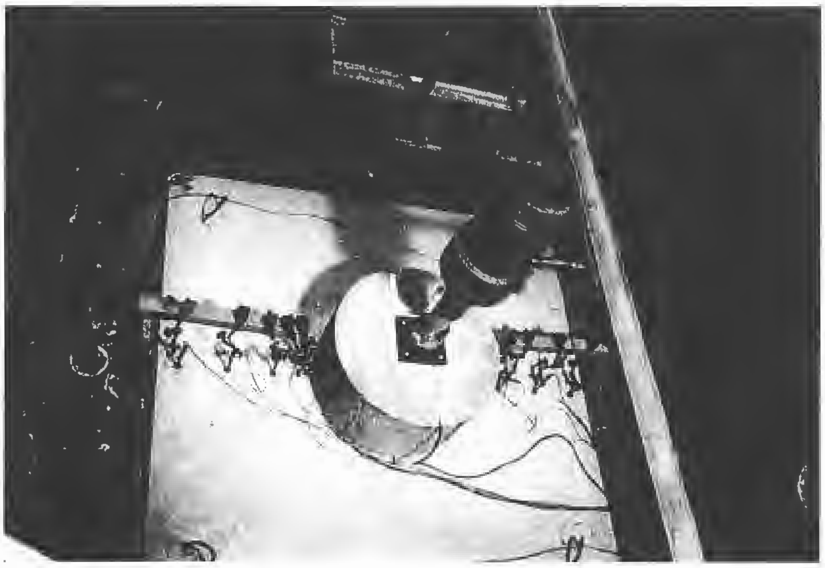

Figure 1. Testing slab L10 (column diameter D = 900 mm).

1.2 RESULTS

The safety factors for punching shear failure $P_{u,test}/V_c$ varied between 1.39 – 1.82 calculated according to the Building Code of Finland (Table 1 and Figure 2). With smaller column sizes the punching shear capacity was much higher than calculated with the Building Code of Finland. The safety factor decreases perceptibly when the column size increases. The Building Code of Finland is too conservative as to the smaller column sizes. On the basis of the test results, the limitation of the circular column diameter $D \leq 3.5d$ seems to be unnecessary. The limitation $\rho \leq 8$ ‰ (used for instance in the Building Code of Finland) for maximal flexural reinforcement area used in the calculation formula seems to be quite good according the test results. The effect of tensile strength of concrete in lower strength classes underestimates test results, whereas at higher strengths the tensile strength of concrete overestimates the punching shear capacity. If the critical section is situated at the distance of 0.5d from the column, the safety level calculated according to the Building Code of Finland is too high when the column diameter is small.

Table 1. Failure load of test slabs.

Slab	$P_{u,test}$ [kN]	V_c [kN]	$P_{u,test}/V_c$ [-]	D [mm]	ρ [‰]	f_y [MPa]
L1	503	290	1.74	202	4.6	621
L2	537	297	1.81	202	4.5	621
L3	530	291	1.82	201	4.5	621
L4	686	476	1.44	402	6.7	612
L5	696	478	1.46	399	6.6	612
L6	799	492	1.62	406	6.5	612
L7	478	263	1.82	201	6.4	586
L8	1111	781	1.42	899	11.6	576
L9	1107	770	1.44	897	11.7	576
L10	1079	778	1.39	901	11.6	576

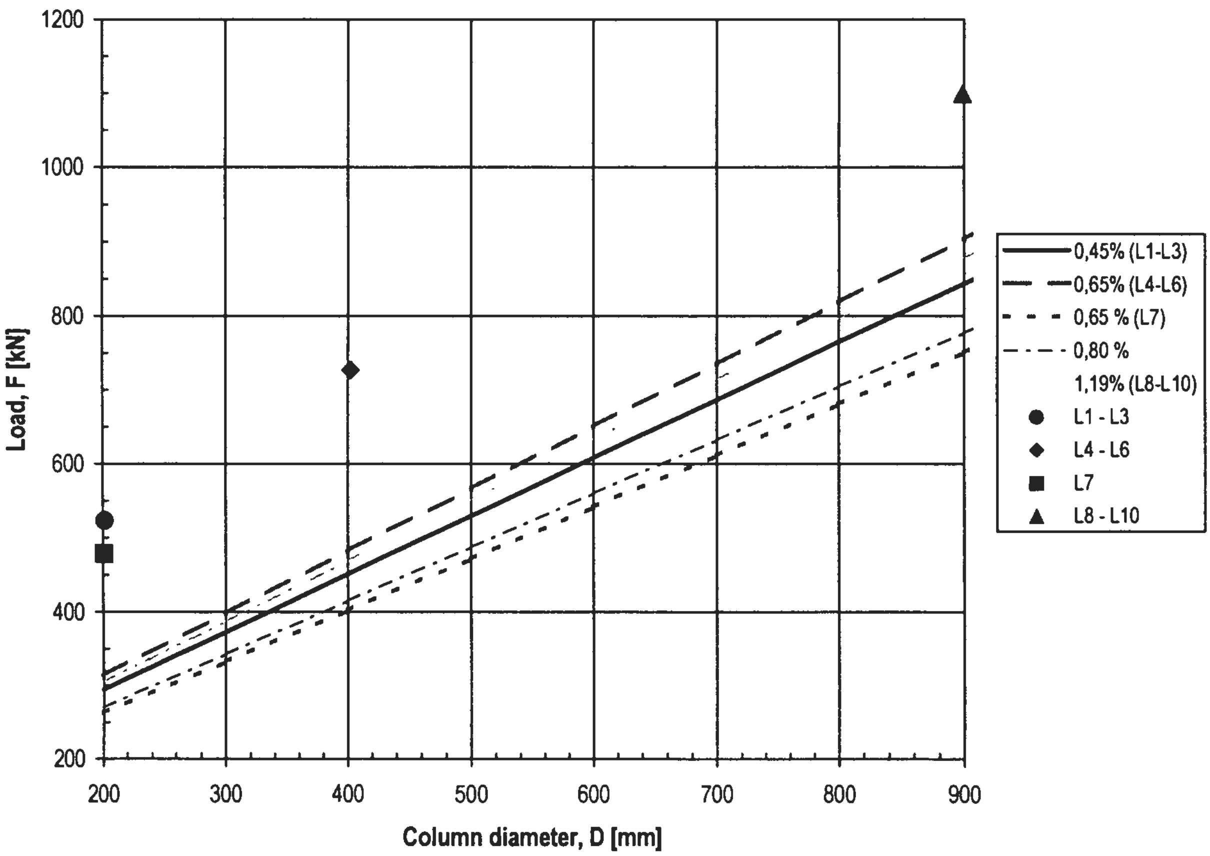

Figure 2. Dependence between the column diameter and failure load as a function of the flexural reinforcement ρ, calculated according to the Building Code of Finland. The test results are also shown.

The test results were compared with the theoretical results based on the different building codes and calculation models (Table 2 and Figure 3). Hallgren's test results are shown in Table 3 (see also Figure 3). The CEB-FIP Model Code (MC 90) and Eurocode 2 seems to give good results.

The control perimeter in MC 90 is located outside the real punching cone. The suitable critical section depends on the type of calculation model. In the test slabs the mean inclination of the punching cone varied between 32° - 38°. At least the compressive strength of the concrete, the effective depth of the slab and the flexural reinforcement affect the inclination.

Table 2. Failure load of the test slabs compared with theoretical results.

Calculation model	Slab			
V_c [kN]	L1-L3	L4-L6	L7	L8-L10
RakMk B4 (Finnish code)	293	482	263	776
EC 2	521	697	462	1003
MC 90	419	575	437	918
ACI 318	353	537	311	863
BS 8110	410	587	433	1006
DIN 1045	290	527	280	996
BBK 94	267	439	241	758
NS 3473	484	700	461	1213
$P_{u,test}$	523	727	478	1099

Table 3. Test results of Hallgren's thesis (1996).

Slab	f_{cc} [MPa]	B [mm]	h [mm]	D [mm]	ρ [‰]	f_y [MPa]	P_u [kN]
HSC0	90.3	250	240	200	8.0	643	965
HSC1	91.3	250	240	200	8.0	627	1021
HSC2	85.7	250	240	194	8.2	620	889
HSC4	91.6	250	240	200	11.9	596	1041
HSC6	108.8	250	240	201	6.0	633	960
N/HSC8	94.9	250	240	198	8.0	631	944

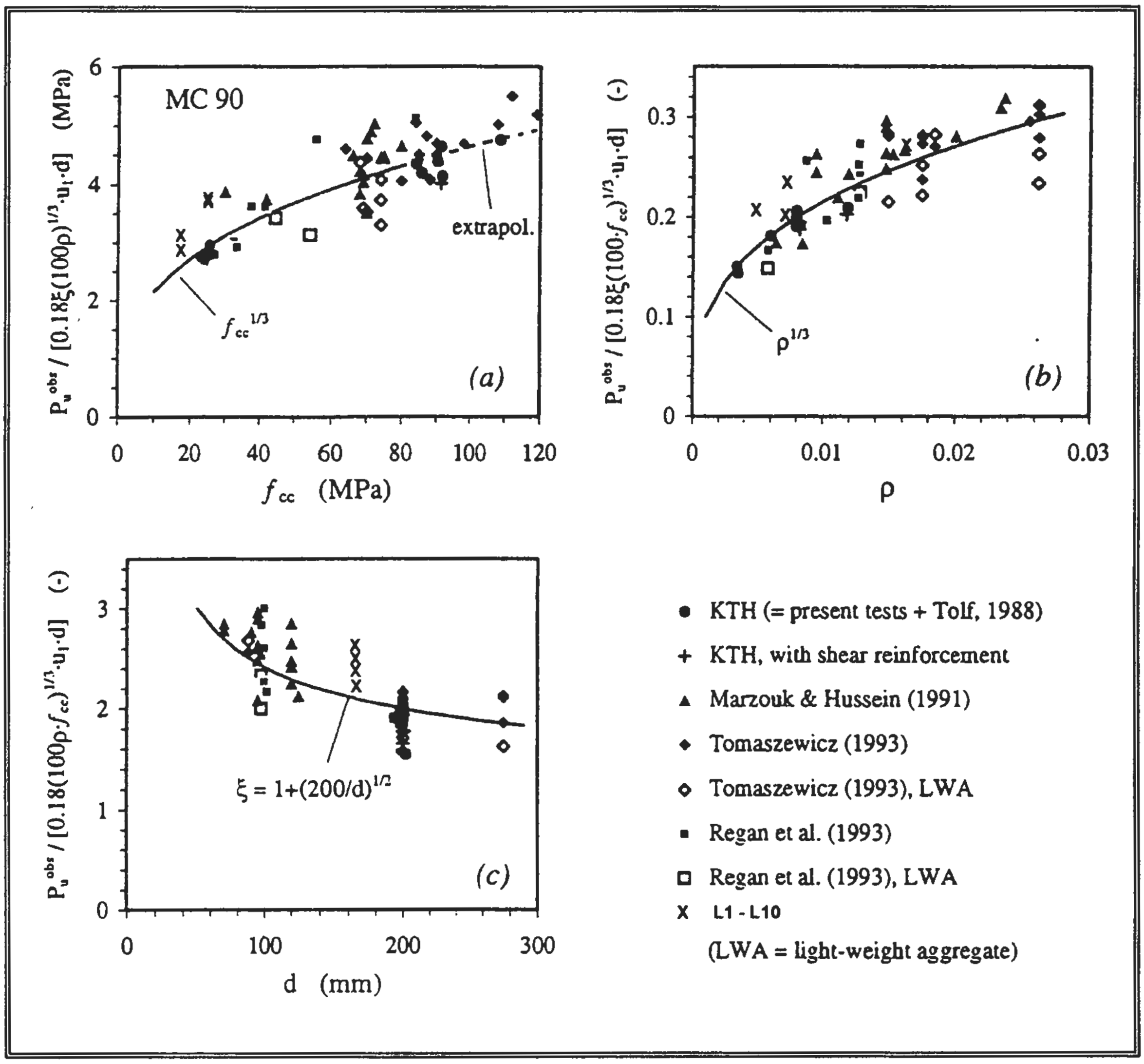

Figure 3. Test results (L1-L10 and other results) compared with CEB-FIP Model Code (MC 90).

A certain section, in which the surface of the cylinder and the punching cone are the same, can be measured as a function of an inclination θ. The distance from the column to the critical section n_{cr} has been calculated for test slabs in Figure 4. The section n_{cr} can be formulated as

$$n_{cr} = \frac{1}{2}\left[\frac{D}{d}\left(\frac{1}{\sin\theta}-1\right)+\frac{\cos\theta}{\sin^2\theta}\right], \quad (1)$$

where D = column diameter [mm],
d = effective depth [mm],
θ = the mean inclination of the punching cone [°]
n_{cr} = the distance from the column [×d].

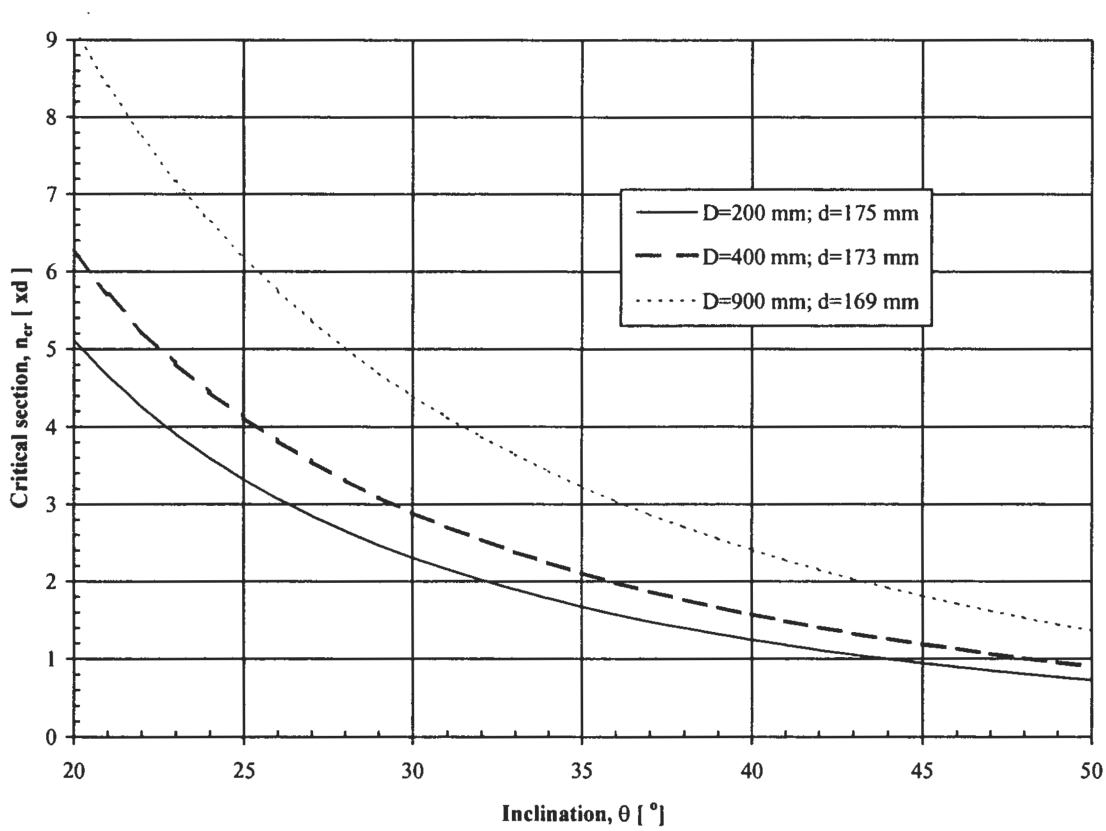

Figure 4. Correspondence between the critical section and mean inclination of the punching cone.

The internal inclination (mean value) was 32° for slabs L1-L3, 36° for slabs L4-L6, 38° for slab L7 and 33° for slabs L8-L10. The critical section is about 2d for slabs L1-L6, 1.5d for slab L7 and 3.5d for slabs L8-L10. Thus the critical section varied in all slabs.

As a result of the study, two failure models for the punching shear capacity are presented: one is based on the surface area of the punching cone (Figure 5) and the other on the summation of the the concrete and the flexural reinforcement;

The failure model based on the surface area of the punching cone

$$A_1 = \frac{\pi \cdot x}{\sin \alpha_1}\left[D + \frac{x}{\tan \alpha_1}\right], \quad (1)$$

$$A_2 = \frac{\pi(d-x)}{\sin \alpha_2}\left[D + \frac{2x}{\tan \alpha_1} + \frac{(d-x)}{\tan \alpha_2}\right], \quad (2)$$

$$A_3 = \frac{\pi(c+\phi)}{\sin \alpha_3}\left[D + \frac{2x}{\tan \alpha_1} + \frac{2(d-x)}{\tan \alpha_2} + \frac{(c+\phi)}{\tan \alpha_3}\right], \quad (3)$$

$$A_{sum} = A_1 + A_2 + A_3. \quad (4)$$

$$x = 0{,}8 \cdot d\sqrt{\frac{E_s}{E_c} \cdot \rho\left(\frac{f_y}{500}\right) \cdot \left(\frac{35}{f_{cc}}\right)}. \quad (4)$$

$$\alpha_i = f(d; \rho; f_c; L; D). \quad (6)$$

A good approximation is: $\alpha_1 = 45°$, $\alpha_2 = 35°$ and $\alpha_3 = 20°$

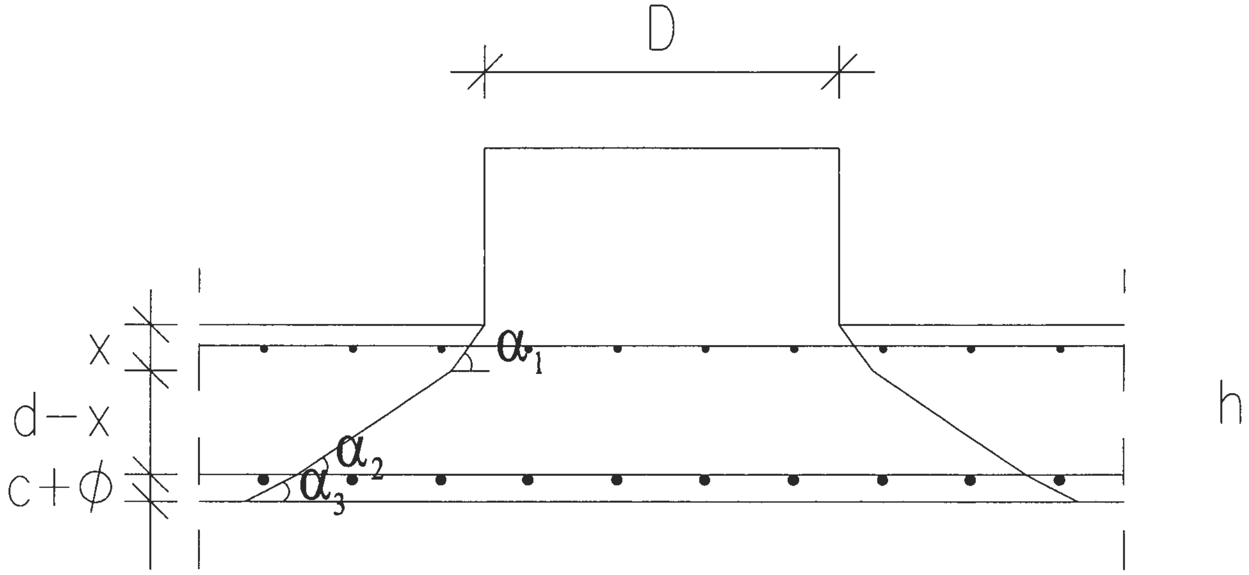

Figure 5. A principle of a calculation model.

$$V_u = \tau_s \cdot A_{sum}. \quad (7)$$

$$\xi = \sqrt{\frac{200}{d}}. \quad (8)$$

$$\tau_s = 0{,}35 \cdot \xi \cdot (100\rho \cdot f_c)^{1/4}. \quad (9)$$

The failure model based on the summation of the the concrete and the flexural reinforcement

The concrete:

$$u_1 = \pi(D + 1{,}4d). \quad (10)$$

The flexural reinforcement:

$$u_2 = \pi(D + 3d). \quad (11)$$

$$V_c = \beta \cdot \xi \cdot f_{ct}^{2/3} \cdot u_1 \cdot h. \quad (12)$$

$$\xi = \sqrt{\frac{200}{d}}. \quad (13)$$

β is an eccentricity factor, $\beta = 0{,}40$ for centrally situated load.

$$f_{vr} = 0{,}75\left(\frac{f_y}{500} \cdot 100\rho\right)^{1/3}. \quad (14)$$

$$V_r = f_{vr} \cdot u_2 \cdot d. \quad (15)$$

$$V_{pun} = V_c + V_r. \quad (16)$$

1.3 CONCLUSION

The calculation formula for the punching shear capacity should be modified so that the punching capacity for larger columns can also be calculated with it. The correct critical section ought to be set to distance d from a column, using also a flexural reinforcement factor ($1.2+40\rho$) and the current reinforcement limitation $\rho \leq 8$ ‰ in the Building Code of Finland. The critical section at a distance of 0.5d from the column makes the value of the punching shear capacity too high: the safety is higher with smaller column sizes when comparing with the larger columns. The safety level of the Building Code of Finland is higher than calculated with Eurocode 2. The length of the control perimeter and shear stress formula of concrete have an effect on this value. The best calculation method to solve the problem of the punching formula is to choose the right failure mechanism, in other words, the real inclination and the situation of the punching cone, and to calculate the shear capacity in that way. A large test series would be needed, with different slab depths, percentages of reinforcement and column diameters, to define the right failure mechanisms.

2. NEED FOR SECONDARY REINFORCEMENT IN REINFORCED CONCRETE SLABS

Secondary reinforcement is used in one-way concrete slabs to counteract tensile stresses perpendicular to the load-bearing direction of the slabs. The reasons for the use of secondary reinforcement are the splitting forces of the reinforcement bars, concentrated loads, restrained shrinkage of concrete, temperature changes and gradients, the so-called Poisson's phenomenon of materials in tension (tensile stresses also in the direction perpendicular to the span) and flexible support line (flexible beam, Figure 6).

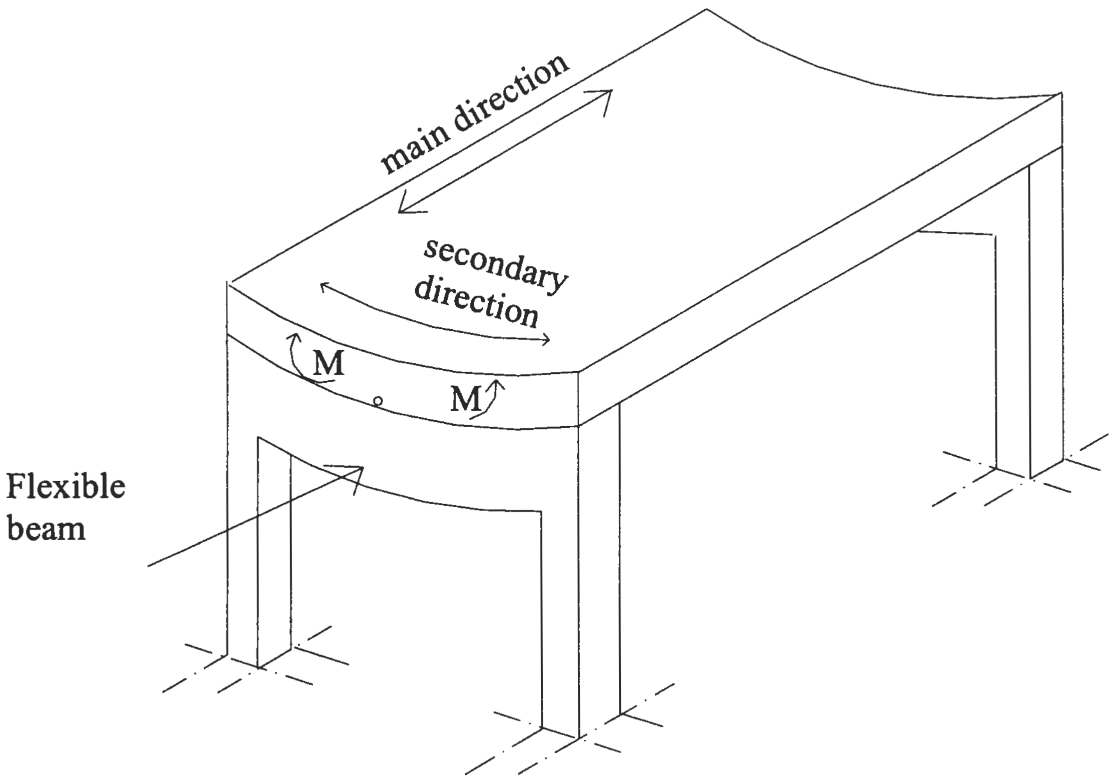

Figure 6. Flexible support line.

There are different rules in the national codes about how the secondary reinforcement amount should be calculated. Generally, in the codes a calculation formula is presented, in which the minimum reinforcement area that is needed in the secondary direction is based on the size of the main reinforcement area. In some codes, the secondary reinforcement area is calculated according to the concrete area of the cross section.

As an example in Table 4. the minimum secondary reinforcement areas are presented for slab thickness of 180 mm according to different codes when the span length in the main direction varies from 3 metres to 6 metres.

Table 4. The minimum secondary reinforcement areas for slab thickness of 180 mm according to different codes, when the span length in the main direction varies from 3 metres to 6 metres. Steel grade A500HW, uniform loading 5.0 kN/m^2, reinforcing areas in [mm^2 / m].

Slab span [m]	ACI 318/ 318R-89	BS 8110	DIN 1045	EC 2	RakMk B4 (Finnish code)	NS 3473
3.0	224	375	59	59	59	230
4.0	224	375	108	108	108	230
5.0	224	375	176	176	176	230
6.0	224	375	271	271	271	230

In the following, the reasons which affect the need for secondary reinforcement in one-way concrete slabs are discussed.

Uniformly distributed loading in a situation in which the width of the loaded area is about one quarter the width of the slab and the loading is situated on the side of the slab can produce a secondary moment which is about 23% of the moment in the main direction. A concentrated load can also produce a secondary moment which is about 50% of the moment in the main direction. The secondary moments are comparatively high and secondary reinforcement is needed.

The deflection of the supporting line, like a beam, also produces a need for secondary reinforcement in the slab. According to the research results, the secondary reinforcement area should be designed specially in the case of flexible beams.

In the case of temperature gradients, there is a need to install secondary reinforcement in concrete slabs. For instance, if the temperature difference between the upper and lower surfaces of the slab is 40°C and the slab thickness is 180 mm, the need for secondary reinforcement is 25% of the reinforcement area in the main direction. The secondary reinforcement should be designed on the basis of the temperature difference.

Concrete shrinkage also is a reason to install secondary reinforcement in concrete slabs. For instance, if the upper surface is in wet conditions and the opposite surface in dry conditions, the need for secondary reinforcement is 22.5% of the reinforcement area in the main direction, when the slab thickness is 180 mm.

It can be concluded that secondary reinforcement is generally needed in one-way slabs. There are many factors which affect the need for secondary reinforcement in one-way supported concrete slabs. The secondary reinforcement area ought to be calculated in every case taking into account all loading effects. The amount 20% of the main reinforcement area, which is mentioned in many codes, is generally either too small or too large for the real need. Concrete shrinkage and temperature gradients are the main reasons why secondary reinforcement is always needed.

3. ROTATION CAPACITY OF REINFORCED CONCRETE SLABS

The main aim of this study was to clarify the relation between the rotation capacity of reinforced concrete slabs and strain properties of reinforcing bars, when the reinforcing areas are relatively small and the diameter of the reinforcing bars is small. The slab reinforcement consisted of single cold-formed rebars, which had a high yield strength R_e, a low relation R_m/R_e between the tensile strength R_m and yield strength R_e and a very low total elongation of steel A_{gt}.

The yield and tensile strengths of the cold-formed 6 mm reinforcing bars were between 557 - 636 MPa and 628 - 690 MPa, and the total elongations A_{gt} between 1.75 - 3.47 %. In 8 mm bars the values were 597 - 612 MPa, 650 - 686 MPa and 2.51 - 2.55 %. The strength relations R_m/R_e varied for both bar sizes between 1.086 - 1.140. The yield strength exceeded the nominal strength value by 11.4 - 27.2 %.

The greatest difference in the mechanical properties was the fact, that in 6 mm bars R_m decreased when R_m/R_e increased and in 8 mm bars R_m increased when R_m/R_e increased. This fact ought to be taken into account when the rotation capacities are compared and the effect of different parameters is studied. When comparing the mechanical properties of A500HW (hot-rolled) and B500K (cold-formed), it can be concluded that the ultimate strengths are similar but the A_{gt}-value of A500HW was about four times greater than that of B500K (6 mm diameter).

The deformation (rotation) capacity of reinforced concrete structures has a great influence on safety, economy and reliability of the structure. All beams and slabs ought to be designed in such a way that they have enough rotation capacity in the ultimate state, so that cracks or larger deflections can be observed before the structure is in the ultimate state. One way to do this is to give a minimum deflection requirement in the ultimate state. It can be, for instance, L/50 (L is the span length). The deflection / span width ratio should be a function of the boundary conditions.

The maximum deflections of the test beams in the ultimate state were between 8.46 - 4.99 mm (calculated with span length L/331 - L/80). So they were in all cases much smaller than the recommended value L/50. Because the reinforcement amounts of the test beams had been designed according to the minimum reinforcement rules of CEB-FIP model code 1990 (1993), ENV 1992-1-1:Eurocode 2 (1994) and RakMk B4 (Finnish code), it can be concluded that the present minimum reinforcement rules of the codes do not guarantee sufficient rotation capacity and reliability of reinforced concrete structures.

4. ANCHORAGE NEED OF REINFORCEMENT

The main aim of the study was to clarify, how much of the bending reinforcement area in the span must be taken for the supports in structures where there is no shear reinforcement. The amounts were found to be quite variable in codes. In beams, the amounts varied between 25 - 33 % and in slabs between 25 - 50 % of the calculated value according to the maximum moment. For the force to be anchored, the different standards and codes also gave quite variable calculation results.

With theoretical calculations it was observed that the force that must be anchored near the support varies, as well as the anchorage length. The force that must be anchored can vary within ± 50% between the different codes. Also, the shear angle was studied, because it affects the tensile force of the bending reinforcement. The result was that the k_a – factor in the Building Code of Finland, with which the shear force must be multiplied to achieve the increase of the tensile force in bending reinforcement, varies.

It was concluded that in structures where there is no shear reinforcement, the most representative value is about $k_a = 2.0$. In many codes the value today is $k_a = 1.5$ or even smaller. In structures with shear reinforcement, the value $k_a = 1.0$ is quite good.

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