PUNCHING SHEAR TESTS ON CIRCULAR HIGH STRENGTH CONCRETE SLABS WITHOUT SHEAR REINFORCEMENT

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ABSTRACT

Punching shear tests reported in the literature up to now have been performed for concrete cube strengths of 20 to 45 MPa and in a few cases up to 55 MPa. Existing design methods for verification of the punching shear resistance are based on these test results. As the use of high strength concrete becomes more common, it is important to study if the design methods are valid also for high strength concrete slabs.

Two circular slabs without shear reinforcement, supported on a centric column, have been tested. The concrete strength measured on 150 mm cubes was about 110 MPa. Comparisons are made with results from earlier tests on identical slabs with cube strengths of about 30 MPa. Observed ultimate loads are also compared with the punching loads calculated by different design methods and expressions in codes.

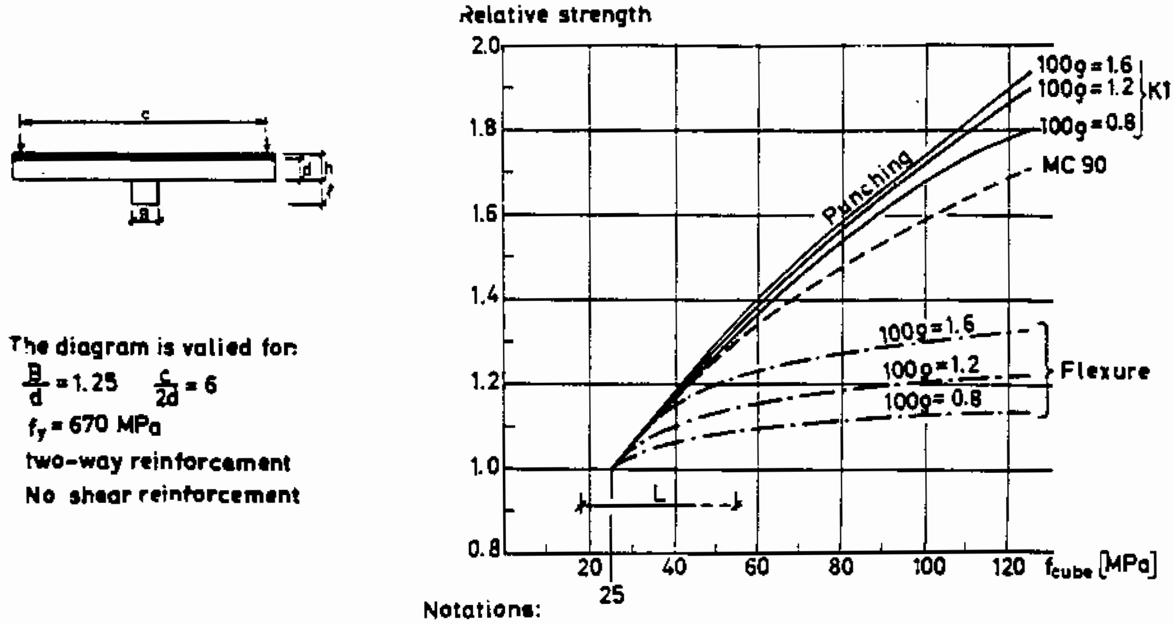
Key words: Punching shear, High strength concrete, Reinforced concrete, Slabs, Flat plates, Tests

1. INTRODUCTION

Punching shear strength of column supported slabs has been the subject of many studies in the last three decades. The cube strength of the test slabs has varied from 20 to 45 MPa and in some few cases up to 55 MPa. The existing design models and empirical expressions for verification of the punching shear resistance are based on results from these tests. Production of concretes with a cube strength up to 120 MPa is now feasible and the use of high strength concrete is a subject of increasing interest due to its beneficial properties.

The column supported slab is a common type of structure in buildings and bridges. It has become highly interesting to check the validity of the punching design rules when applied to high strength concrete. If necessary, the design rules and methods have to be modified, or even changed.

FIG 1 gives for different concrete cube strengths the calculated load capacity for symmetrically loaded, column supported, circular slabs in relation to that for cube strength 25 MPa. The flexural failure load was calculated according to the yield-line theory. The punching failure load was calculated according to the theory developed by Kinnunen and Nylander /1/.



Notations:

——— Relative punching strength. Punching failure load calculated according to the method in K1-1964

- - - - - Relative punching strength. Punching resistance calculated from eq [6.5-16] in CEB Bulletin No 196

-·—·— Relative flexural failure load

ρ Ratio of flexural reinforcement

L Range of f_{cube} in tests reported in literature

FIG.1 Effect of concrete strength and ratio of reinforcement on load carrying capacity of circular slab on central column.

In this theory the physical behaviour of the slab is described by a mechanical model which takes into account the deformations of the slab. The method is also given in /2/ and described in /3/. In FIG 1 curves corresponding to this theory are denoted K1 and are given for three different ratios, ρ, of flexural reinforcement. The punching resistance was also calculated according to the method proposed in the CEB-FIP Model Code 1990 /4/. In FIG 1 the corresponding curve is denoted MC 90. FIG 1 indicated that there is a considerable potential to increase the punching capacity by use of high strength concrete.

2. TEST SPECIMENS AND TESTING PROCEDURE

2.1 Specimen details

Two circular high strength concrete slabs without shear reinforcement have been tested. The object was to study the effect of the concrete strength on the structural behaviour of the slab, including the deformation capacity and the load bearing capacity. For comparison the slabs had the same size and flexural reinforcement as some of the normal strength concrete slabs which were tested earlier at the Department of Structural Mechanics and Engineering by P. Tolf /5/. The total diameter of the slabs was 2540 mm. The diameter, c, of the circle along which the load was uniformly distributed, the total thickness, h, the effective depth, d, and the column diameter, B, are given in TABLE 1. All slabs had two-way flexural reinforcement consisting of ϕ16 mm Swedish deformed bars of Grade Ks60S. The ratio of reinforcement was 0.008. The nominal dimensions of the slabs and the arrangement of the reinforcement are shown in FIG 2.

TABLE 1. Slab details

Slab No.	c mm	h mm	d mm	B mm	f_{cube} MPa	Flexural reinforcement ϕ mm	ρ %	f_{sy} MPa	Shear reinforcement n	ϕ mm	Ultimate load P_u kN
HSC-S2.1	2400	240	200	250	112.9	16	0.80	643	-	-	965
HSC-S2.2	2400	245	200	250	106.9	16	0.80	627	-	-	1021
NSC-S2.1 /5/	2400	240	200	250	30.3	16	0.80	657	-	-	603
NSC-S2.2 /5/	2400	240	199	250	28.6	16	0.80	670	-	-	600
NSC-S2.2s /5/	2400	240	195	250	31.1	16	0.80	669	32	10	894
NSC-S2.1s /5/	2400	240	195	250	29.5	16	0.80	673	32	10	851

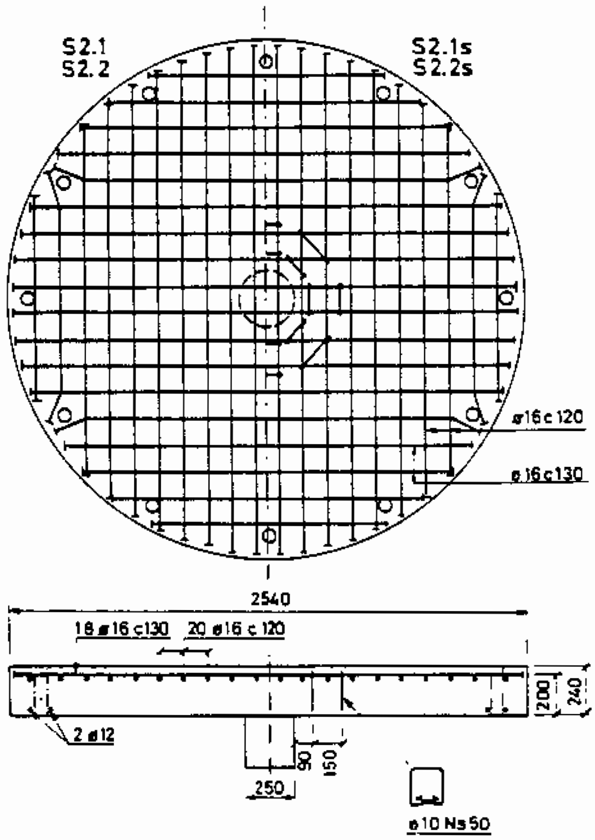


FIG.2 Reinforcement and nominal dimensions of the test slabs. The left part of the figure shows the slabs without shear reinforcement and the right part shows the slabs with shear reinforcement, tested by Tolf /5/.

2.2 Materials

The yield strength of the flexural reinforcement, f_{sy}, and the concrete cube strength, f_{cube}, are given in TABLE 1. The 150 mm concrete cubes were cured, at a temperature of about 20° C, for one day covered with wet burlap sacks, for the next four days in water and after that, until testing, in a humid room. The normal density high strength concrete was supplied as ready mix from Upplandsbetong AB. The concrete mix details are given in TABLE

2. The Portland cement used in these mixes was Swedish "Anläggningscement", a sulphate resistant type with low alkali and a moderate heat of hydration. Melamine (Cementa V33) was used as superplasticizer and lignosulphonate (Rescon P) as retarder.

TABLE 2. Concrete mix proportions for the high strength concrete slabs.

Mix	HSC-S2.1 kg/m^3	HSC-S2.2 kg/m^3
Portland cement	550	400
Silica fume	50	40
Crushed aggregate 8 - 18 mm	860	1000
Aggregate 0 - 8 mm	740	870
Water	151	119
Melamine	30	18
Lignosulphonate	5,5	4
w/(c+s)	0.25	0.27

2.3 Testing

The test slabs were supported on a centric circular column and subjected to load uniformly distributed along the circumference by twelve tie rods and spreader beams. The testing arrangement is shown in FIG. 3. The load was increased in steps of 80 kN. Between each loading step the crack pattern and crack widths were observed and recorded. Concrete strains in radial and tangential directions on the bottom surface of the slabs were measured continuously during the test and recorded by an automatic data acquisition system. Deflections of the slabs and strains in the reinforcement were measured continuously in the same way.

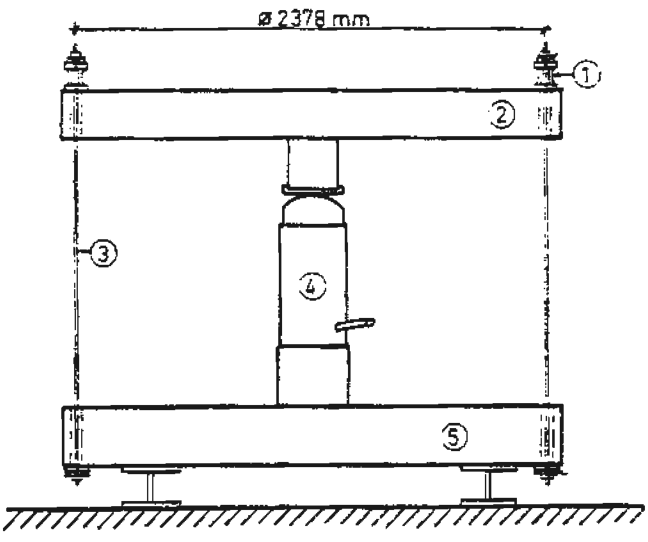


FIG. 3 Testing arrangement:
1 Spreader beam
2 Test slab
3 Tie rod
4 Hydraulic jack
5 Counter slab

3. TEST RESULTS AND COMPARISON WITH NSC SLABS

3.1 Ultimate loads

All slabs failed by punching. The ultimate loads, P_u, including the weight of the slabs, 30 kN, are given i TABLE 1. The high strength concrete slabs HSC-S2.1 and HSC-S2.2, with concrete cube strengths of about 110 MPa, failed at loads 60 % and 70 % higher than the ultimate loads of the corresponding normal concrete strength slabs NSC-S2.1 and NSC-S2.2, with concrete cube strengths of about 30 MPa, tested by Tolf /5/. Even in comparison with normal strength concrete slabs having the same dimensions and the same ratio of flexural reinforcement but provided with shear reinforcement, the high strength concrete slabs showed higher load capacities. The slab HSC-S2.1 failed at a load 8 % and 13 % higher than NSC-S2.1s and NSC-S2.2s, respectively. The slab HSC-S2.2 failed at a load 14 % and 20 % higher than NSC-S2.1s and NSC-S2.2s, respectively. The details of the slabs NSC-S2.1s and NSC-S2.2s, tested by Tolf /5/, are given in TABLE 1 and FIG 2.

3.2 Strains and deflection

Concrete strains in tangential and radial directions measured on the bottom surface of the slabs at a distance of 60 mm from the column face are shown in FIG 4. Both tangential and radial strains appear to have increased more slowly for the high strength concrete slabs than for the normal strength concrete slabs. The radial strain decreased in the high strength concrete slabs as well as in the normal strength concrete slabs when the load was approaching its ultimate value, and in some cases the strain became even tensile.

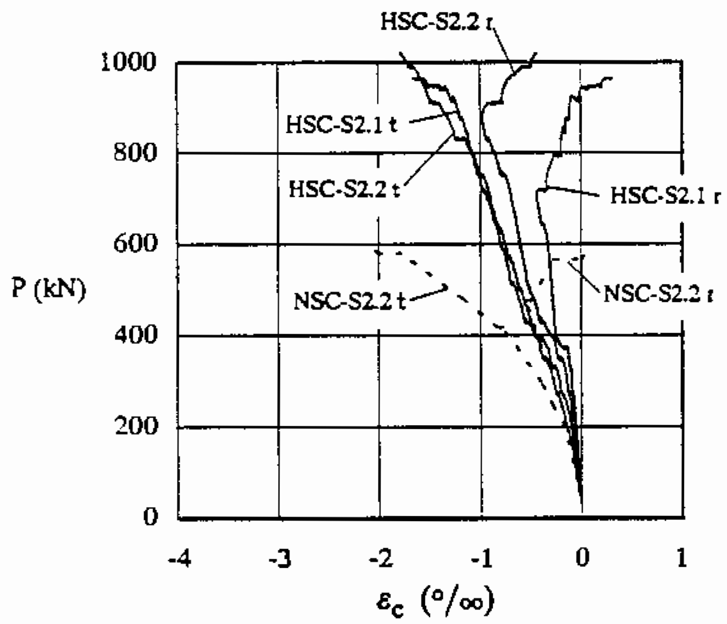


FIG. 4. Concrete strains, ε_c, at the bottom of the slab 60 mm from the column face in relation to the load, P.
t : Strain in tangential direction
r : Strain in radial direction

The tangential compressive strain in the high strength concrete slabs reached values of about 1.7 °/oo . This is somewhat less than in the slab NSC-S2.2, where the tangential compressive strain exceeded 2.0 °/oo . The observed lower values of the tangential compressive strain in the slabs HSC-S2.1 and HSC-S2.2 seem to indicate that in high strength concrete slabs failure occurs at lower values of the ultimate concrete strain, ε_{cu} .

The measured strains in the flexural reinforcement above the column at a distance of 65 mm from the centre of the slab are shown in FIG 5. At failure the tensile strains in the reinforcement of the two normal strength concrete slabs, NSC-S2.1 and NSC-S2.2, reached values of about 2 °/oo. In the high strength concrete slab HSC-S2.1, however, the reinforcement reached a tensile strain almost twice as much. The reinforcement in the slab HSC-S2.2, having a somewhat lower yield strength than the reinforcement in HSC-S2.1, started to yield before failure.

At a load of about 350 kN the strain in the reinforcement of both high strength concrete slabs began to increase faster, which is due to the appearance of an inclined shear crack, propagating through the slab. The appearance of a shear crack at this load level was also indicated by strain measurements in two vertical gauge strips at a distance from the circumference of the column equal to half the effective depth. This has also been observed in earlier tests reported in /1/. In the high strength concrete slabs, however, this change of increase of the tensile strain in the reinforcement seems to bee more sudden than in the normal strength concrete slabs, indicating a more brittle behaviour of the high strength concrete.

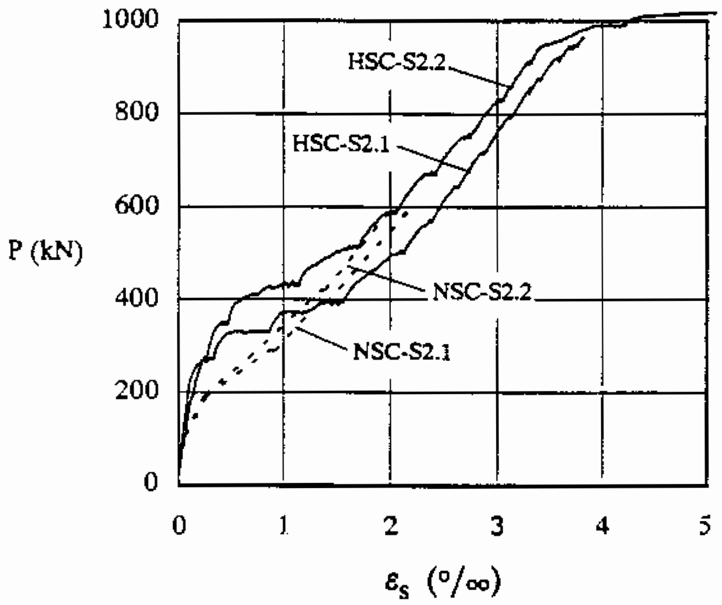


FIG. 5. Observed tensile strains in the flexural reinforcement, ε_s, above the column in relation to the load, P. The strain was measured at a distance of 65 mm from the centre of the slab.

The angles of rotation of the slab portions outside the shear cracks, calculated from the measured deflections, are shown in FIG 6. It appears that the high strength concrete slabs, HSC-S2.1 and HSC-S2.2, allowed a higher angle of rotation than the normal strength concrete slabs NSC-S2.1 and NSC-S2.2. Compared with normal strength concrete slabs provided with shear reinforcement, NSC-S2.1s and NSC-S2.2s, the high strength concrete slabs showed approximately the same angle of rotation.

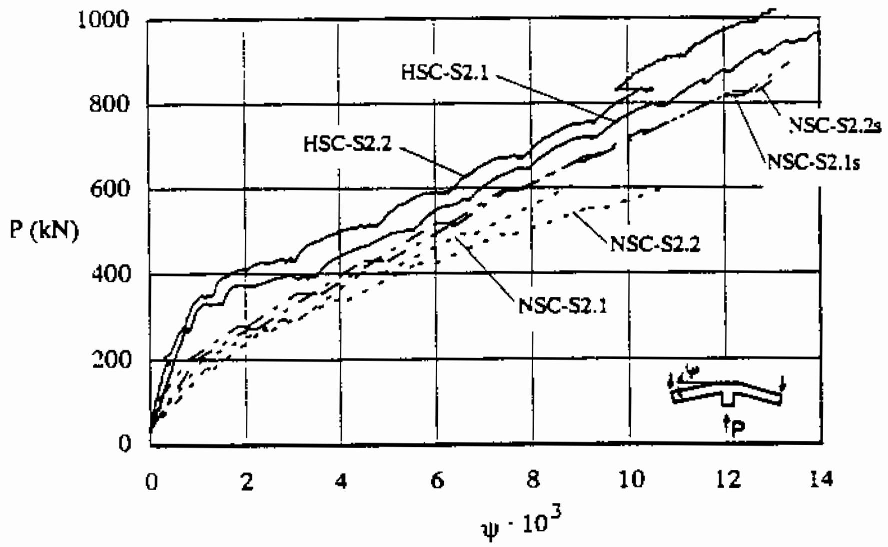


FIG. 6. Angle of rotation, ψ, of the slab portion outside the shear crack in relation to the load, P.

Thus, the high strength concrete slabs show higher deformation capacities than the corresponding normal strength concrete slabs without shear reinforcement, and about the same deformation capacities as the normal strength concrete slabs provided with shear reinforcement. Just as in FIG 5 the curves for the high strength concrete slabs in FIG 6 show a sudden faster increase of deformation at a load level of about 350 kN, indicating shear cracking.

4. COMPARISON WITH EXISTING DESIGN METHODS AND CODES

In TABLE 3 the observed ultimate loads, P_u, of the tested high strength concrete slabs and of normal strength concrete slabs without shear reinforcement are compared with the calculated punching loads, P_c, obtained according to existing design methods by Kinnunen and Nylander in K1 /2/ and by Broms /7/. Furthermore comparison is made with the simplified design method given by Nylander and Kinnunen in the Swedish Handbook for Concrete Structures /6/, and the expressions given in the Swedish Regulations for Concrete Structures BBK 79 /8/ and the CEB-FIP Model Code 1990 /4/. For details, of these methods, the reader is referred to /2/, /4/, /6/, /7/ and /8/.

TABLE 3. Comparison of observed ultimate loads and calculated punching loads

		K1 /2/ with size effect		Handbook /6/		Broms /7/		BBK 79 /8/		MC 90 /4/	
Slab No.	P_u kN	P_c kN	P_u/P_c	P_c kN	P_u/P_c	P_c kN	P_u/P_c	P_c kN	P_u/P_c	P_c kN	P_u/P_c
HSC-S2.1	965	1051	0.92	978	0.99	1129 (1025)	0.85 (0.94)	623	1.55	890	1.08
HSC-S2.2	1021	1025	1.00	956	1.07	1099 (1000)	0.93 (1.02)	623	1.64	874	1.17
NSC-S2.1 /5/	603	637	0.95	521	1.16	610	0.99	354	1.70	574	1.05
NSC-S2.2 /5/	600	625	0.96	506	1.19	583	1.03	338	1.78	563	1.07

P_u : observed ultimate load

P_c : calculated punching load ( ) : values calculated with a reduced column diameter, B_{red}

4.1 Design method by Kinnunen and Nylander

The design method developed by Kinnunen and Nylander, given in /1/ and K1 /2/, is based on a mechanical model, which takes into account the observed behaviour of the slab and satisfies the equations of equilibrium of the forces acting on a segmental slab part outside the inclined shear crack. The model also takes into account the deformation of the slab. Failure is assumed to occur when the tangential concrete strain at the bottom surface of the slab, at a point located vertically under the root of the shear crack, reaches a characteristic value.

In TABLE 3 the calculated punching loads have been modified with respect to a size effect according to /6/. The method overestimates the punching load for the normal strength concrete slabs and the high strength concrete slab HSC-S2.1, but gives an almost perfect fit for the slab HSC-S2.2.

4.2 Design method in the Swedish Handbook for Concrete Structures

The design method given in the Swedish Handbook for Concrete Structures /6/ is based on the method given in /2/, but is somewhat simplified. In the comparison here, the reduction factor in eq 32a is removed when calculating the punching load in TABLE 3. The concrete strength, f_c , used in the method, equivalent to cylinder strength, has been calculated by:

$$f_c = 0.83\ f_{cube} \quad \text{(MPa)} \qquad (1)$$

The method underestimates the punching loads for the normal strength concrete slabs and for the slab HSC-S2.2, while it overestimates the punching load for the slab HSC-2.1 slightly.

4.3 Design method by Broms

The method developed by Broms /7/ is similar to the method in /2/. The ultimate tangential concrete strain has been modified and is given as a function of the cylinder strength. A new criterion is introduced in the method, limiting the radial compression stress in the concrete. As recommended in the paper by Broms /7/, the cube strength has been transformed to cylinder strength by:

$$f_c = 0.7\ f_{cube} + 0.5 \quad \text{(MPa)} \qquad (2)$$

The method gives a good fit to the normal strength concrete slabs, but overestimates the punching loads for the high strength concrete slabs. For the slabs HSC-S2.1 and HSC-S2.2 the point, at which the condition of moment equilibrium is computed, lies within the circumference of the column. If this is considered with a reduced column diameter, B_{red}, the results by this method are also in agreement the observed punching loads of the high strength concrete slabs.

4.4 BBK 79

The Swedish Regulations for Concrete Structures, BBK 79, give a formal punching shear strength expressed as:

$$f_{v1} = \xi\ (1+50\rho)\ 0.45\ f_{ct} \quad \text{(MPa)} \qquad (3)$$

where ξ is a size effect factor
f_{ct} is the concrete tensile strength, given in a table in relation to f_{cube}

For f_{cube} exceeding approximately 65 MPa f_{ct} is kept constant. The punching resistance is calculated from the punching shear strength acting on a control perimeter at a distance of 0.5 d from the circumference of the column.

The expression (3) underestimates the punching shear strength of all the tested slabs given in TABLE 3, but is less conservative for the high strength concrete slabs.

4.5 MC 90

In the CEB-FIP Model Code 1990, MC 90, /4/ a formal punching shear strength is given as:

$$v_{Rd} = 0.12\ \xi\ (100\rho \cdot f_c)^{1/3} \quad \text{(MPa)} \qquad (4)$$

The punching resistance is calculated from the punching shear strength acting on a control perimeter at a distance of 2.0 d from the circumference of the column.

In the comparison given in TABLE 3 the safety in the expression was removed by changing the factor 0.12 into 0.16. The cylinder strength was calculated from eq. (1).

The expression (4) underestimates the punching shear strength of all the tested slabs given in TABLE 3.

5. CONCLUSIONS

The tests indicate the following:

- The tested high strength concrete slabs had 60 to 70 % higher load oapacity than the corresponding normal strength concrete slabs without shear reinforcement, and 8 to 20 % higher load capacity than the corresponding normal strength concrete slabs with shear reinforcement.

- The tested high strength concrete slabs had about 45 % higher angular rotation capacity than the corresponding normal strength concrete slabs without shear reinforcement, and about the same deformation capacity as the corresponding normal strength concrete slabs with shear reinforcement. Higher rotation capacity of the high strength concrete slabs resulted in more efficient use of the flexural reinforcement. The deformation curves in FIG 6, however, indicated a more brittle behaviour of the high strength concrete.

- Comparing the design methods, the method given in K1 and in the Swedish Handbook for Concrete Structures gave the best fit to the tested high strength concrete slabs. The method developed by Broms gave the best fit to the normal strength concrete slabs. Calculated with a reduced column diameter Broms method also gave a good fit to the high strength concrete slabs.

- BBK 79 gave the largest deviations between observed and calculated loads for both high strength and normal strength concrete slabs, underestimating the punching loads in all tests. BBK 79 is less conservative for the high strength concrete slabs than for the normal strength concrete slabs.

- CEB-FIP MC 90 also underestimates the punching loads in all tests.

As the tests until now include slabs of only one size and one ratio of reinforcement, and the comparison includes only cube strengths of 110 MPa and 30 MPa, the conclusions are only preliminary and valid for slabs with the same characteristic.

6. NEEDS FOR FURTHER RESEARCH

Tests on slabs with concrete cube strengths between 30 and 110 MPa should be performed. Tests on high strength concrete slabs varying in size and reinforcement, with and without shear reinforcement, would also be of interest. An interesting question in this connection is the relation between the size effect and the type of concrete.

The new test results can then be compared with results obtained by different design methods. The methods can be modified or even changed, if necessary.

7. ACKNOWLEDGMENTS

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