Slender CRC Columns

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Published in:
Nordic Concrete Research

Publication date:
2005

Document Version
Publisher's PDF, also known as Version of record

Link to publication from Aalborg University

Citation for published version (APA):
Aarup, B., Jensen, L. R., & Ellegaard, P. (2005). Slender CRC Columns. In Nordic Concrete Research (pp. 80-97). Nordic Concrete Federation. (Publication; No. 34).

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ABSTRACT

CRC is a high-performance steel fibre-reinforced concrete with a typical average compressive strength in the range of 120-160 MPa. Design methods for a number of structural elements have been developed since CRC was invented in 1986, but the current project set out to further investigate the range of columns for which current design guides can be used. The columns tested had a slenderness varying from 1.11 to 12.76, and a reinforcement ratio (area of reinforcement to area of concrete) ranging from 0 to 8.8%. A total of 77 tests were carried out – 61 columns were tested in ambient conditions and 16 columns were tested in standard fire conditions. The tests showed good correlation between test results and results calculated according to established design guides. The fire tests demonstrate that load capacity of slender columns can be reduced very quickly due to thermal stresses and a reduction of stiffness – also in cases where temperature at the rebar is still relatively low. However, guidelines for achieving acceptable fire resistance can be determined based on the test results.

Key words: columns, CRC, fire resistance, design, tests

1. INTRODUCTION

CRC – short for Compact Reinforced Composite - is a high-performance steel fibre-reinforced concrete developed in 1986 [1]. The fibre content is typically 2-6% by volume and the average compressive strength is in the range 120-160 MPa. CRC has a very low porosity which means that durability and resistance to corrosion are very good, so that a very small cover to the reinforcement can be used. This is very important because CRC is often used for slender structures and because a combination of passive reinforcing bars and fibre reinforcement is used in CRC.

Over the last 6-7 years, CRC has been used increasingly for a number of small structural applications such as staircases and balcony slabs in Denmark [2,3], and there is a growing interest for elements such as beams and columns. CRC has been investigated extensively and part of the development of CRC has been carried out in a number of European Research projects. Based on the input from these projects design guides have been developed [4]. However, the experimental background is relatively limited for columns. Hi-Con, the worlds largest producer of CRC elements, who have been producing CRC since 2001, wanted to establish a broader base for design of CRC columns. This was done in the current project, sponsored by Mål 2 – A European Union Regional programme. The project was headed by Hi-Con, with support from CRC Technology and Carl Bro as. Testing was carried out at Aalborg University (AAU) and the Technical University of Denmark (DTU) in Copenhagen. The project was initiated in September 2002 and concluded in September 2004.

Figure 1 - Cantilevered Hi-Con CRC balcony slabs used in apartments in Aalborg, Denmark.

2. COLUMNS TESTED UNDER AMBIENT CONDITIONS

2.1 Test programme

The programme focused on centrally loaded columns in ambient conditions – where a total of 57 columns were tested. The columns ranged from 80x80 mm cross-section with a height of 4.2

metres to 200x200 mm cross-section with a height of 2.7 metres. Other parameters in addition to size and slenderness were shape, reinforcement ratio, size of reinforcement and steel fibre content. The programme also included 4 columns tested with eccentric load with an eccentricity of 25 mm.

26 columns were tested at DTU – mostly those with a height differing from 2725 mm, while 51 columns were tested at AAU, including the 16 columns tested in fire conditions and the 4 columns tested with eccentric load. The setups are shown in figure 2.

At AAU the testing was done in a newly built 2000 kN press with hinges at the top and the bottom. The centre of rotation was placed so that the physical length of the columns was equal to the theoretical length shown in table 2. The hinges allow for deflections in all directions. Load was introduced in increments and at each load level, 10 measurements of displacements were taken. In each test series, at least one column was loaded to failure, while for others, the test was stopped after a load reasonably above the predicted failure load had been achieved.

The testing at DTU was carried out in a 5000 kN press. The columns were simply supported at each end, i.e. such that the ends of the column were free to rotate in one plane and rotationally restricted perpendicular to this plane. The theoretical column length, which is given in table 2, was slightly larger than the physical length of the columns as the distance from the surface of the supports to the centre of rotation was added. The tests at DTU were carried out in displacement control at a constant rate of travel of the crosshead of the testing machine.

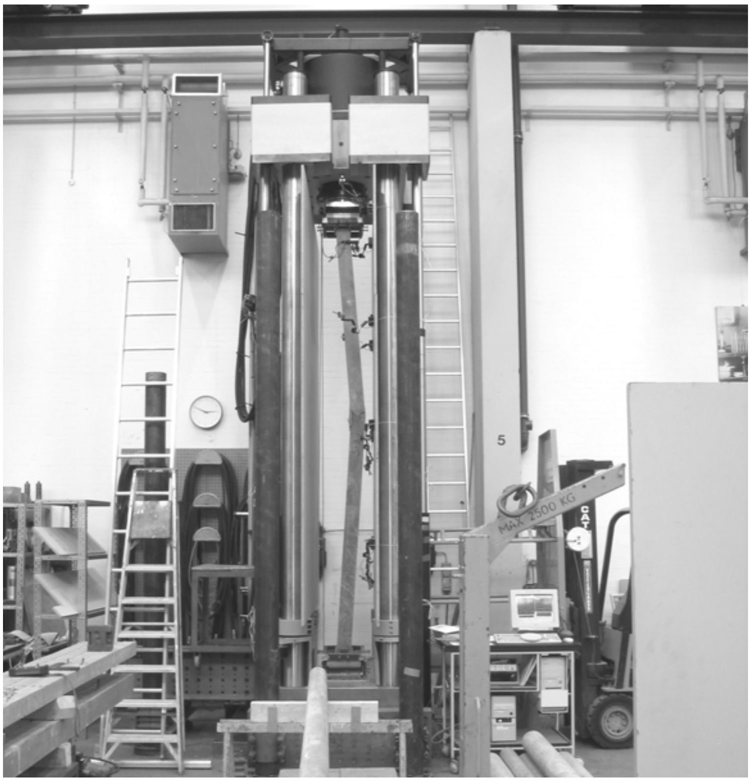

Figure 2 - Testing setup at DTU (on the left) and AAU.

All columns were produced at Hi-Con as part of their normal production – with the precision which is normal for the industry regarding placing of reinforcement, preparation of ends and initial curvature. Square columns were cast on the side while round columns were cast standing up. Composition for 1 m^3 was:

CRC binder	940 kg
Sand 0-2 mm	664 kg
Sand 2-4 mm	661 kg
Water	154 kg

CRC binder is a mix consisting of cement, micro silica and dry super plasticizer. The steel fibre content was 160, 320 or 480 kg depending on whether a 2, 4 or 6% mix was used. The steel fibres were straight, smooth and had a length of 12.5 mm and a diameter of 0.4 mm. Generally, cover to the reinforcement was 15 mm except in the case of the columns with cross-sections of 200x200 mm, which had a nominal cover of 25 mm.

2.2. Results for central loads

The properties used for calculations are shown in table 1. The table shows 4 sets of values, all based on results for 100x200 mm cylinder tests, a sample size which is standard for CRC:

- "Expected" – mean values (conservative estimate) based on other tests with CRC [4]
- "Characteristic" – the 5% fractile value of "expected" values
- "Design" – design value for E modulus is the same as the characteristic value, while the design value for compressive strength is obtained by dividing by a material factor of 1.65
- "Test" – results found in testing at AAU for this specific project on production batches

The test values for the mix with 4% of fibres were expected to fall between the values achieved with 2 and 6% of fibres, but the values are relatively low. This could perhaps be attributed to differences in exact water content and compaction. Mixes with 4 and 6% of fibres were produced in smaller batches than the mixes with 2% of fibres, as the 2% mixes are part of the normal production at Hi-Con. Fibres are added manually. For the 4% mixes it was observed, that there was little variation in the properties measured for test specimens from one batch, while there was a relatively large difference from one batch to another. The standard deviation was generally larger than what is observed in the normal quality control at Hi-Con.

Table 1 - Properties used for calculations.

Fibre content Vol.%	E expected GPa	E charact. GPa	E design GPa	E Test, mean GPa	E test stan.dev. GPa	f_{CRC} expected MPa	f_{CRC} charact. MPa	f_{CRC} design MPa	f_{CRC} test, mean MPa	f_{CRC} test, stan. dev. MPa
0	39.0	38.05	38.05	-		120	105	63.6	-	
2	41.0	38.05	38.05	41.00	1.65	120	105	63.6	145	4.7
4	42.5	39.40	39.40	40.35	2.50	130	115	69.7	137	14.5
6	45.0	41.50	41.50	44.24	3.60	145	120	72.7	154	11.4

Some of the main results of the column tests are shown in table 2. There was considerable variation in the test loads that were carried, but in general the carrying capacity was larger than expected. The estimated capacity shown in table 2 was calculated using the properties measured in the project and marked "Test", while the design capacity was calculated based on design properties. Also shown in table 2 are two ratios. Ratio 1 is the maximum test load divided by the estimated capacity, while ratio 2 is the maximum test load divided by the design capacity. In a number of cases the columns were not loaded to failure as testing was stopped after the estimated capacity had been achieved. This is indicated with * and in these cases the maximum test load carried corresponds to the minimum carrying capacity for the column.

The formulas used for calculating slenderness index α, and capacity $N_{CRC,CR}$ are shown below. They have been derived from tests carried out in the EUREKA project Compresit [5] and the Brite/EuRam project HITECO [6], where short columns were tested and the Brite/EuRam project MINISTRUCT [7], where also slender columns were tested. The formulas differ only slightly from the conventional calculation methods, but they predict a slightly higher load capacity than conventional methods. As the increase in strength for CRC compared to conventional concrete is much higher than the increase in Young's modulus the slenderness index for CRC will often be relatively high. With other types of aggregate the ratio between stiffness and strength would be different, i.e. with calcined bauxite as aggregate compressive strength would typically be 200 MPa while Young's modulus would be 75 GPa.

$N_{CRC,CR}$ is the lower value of:

$$\min\begin{cases} \beta \cdot \dfrac{\sigma_{cr}}{f_{CRC}}\left(1+\dfrac{E_s \cdot A_s}{273 f_{CRC} \cdot A_{CRC}}\right) \cdot f_{CRC} \cdot A \\ \beta \cdot \left(\dfrac{\sigma_{CRC}}{f_{CRC}} + \dfrac{f_s \cdot A_s}{f_{CRC} \cdot A_{CRC}}\right) f_{CRC} \cdot A \end{cases} \qquad (1)$$

where:

$$\frac{\sigma_{cr}}{f_{CRC}} = \frac{1}{\sqrt{1+\alpha^2}} \qquad (2)$$

$$\alpha = \frac{\lambda^2 \cdot f_{CRC}}{\pi^2 \cdot E_{CRC0}} \qquad (3)$$

$$\lambda = \frac{l_c}{r_g} \qquad (4)$$

l_c	: free column length
r_g	: radius of gyration
f_{CRC}	: uni-axial compressive strength of CRC matrix
f_s	: strength of reinforcement
σ_{CRC}	: compressive stress in CRC matrix
A	: cross-sectional area
A_s	: cross-sectional area of reinforcement
A_{CRC}	: cross-sectional area of CRC matrix
E_{CRC}	: modulus of elasticity of CRC matrix
E_s	: modulus of elasticity of reinforcement

$\beta = (0.95 - \frac{A_s}{A})$ if $\alpha < 1.5$

$\beta = 0.95$ if $\alpha >= 1.5$

*Table 2 - Results at ambient conditions with central loading. * shows that the column was not tested to failure.*

Cross-section mm	Length mm	Slenderness index	Reinforcement	Fibre content Vol.%	Estimated capacity kN	Design capacity kN	Maximum test load kN	Ratio1	Ratio2
80x80	2725	4.79	4 ø10	4	218	142	339 339* 297	1.56 1.56* 1.36	2.39 2.39* 2.09
80x80	4358	12.76	4 ø12+ 4 ø6	2	103	70	120 140	1.17 1.36	1.71 2.00
120X120	2725	2.22	none	2	815	440	894 821* 821*	1.10 1.01* 1.01*	2.03 1.87* 1.87*
120x120	2725	2.22	1 ø25	2	964	571	1087* 1481 1484*	1.13* 1.54 1.54*	1.90* 2.59 2.57*
120x120	2725	2.13	1 ø25	4	951	588	1537 1378 1272*	1.62 1.45 1.34*	2.61 2.34 2.16*
120x120	2725	2.13	4 ø12	4	938	577	1597 1510 1510*	1.70 1.61 1.61*	2.77 2.62 2.62*
120x120	2725	2.22	4 ø20	2	1219	796	1898 1696* 1770*	1.56 1.39* 1.45*	2.38 2.13* 2.22*
120x120	3898	3.95	4 ø20	0	644	416	1040 430	1.57 0.65	2.50 1.03
120x120	3898	4.54	4 ø12	2	499	304	510 580 490	1.02 1.16 0.98	1.68 1.91 1.61
120x120	3898	4.36	4 ø12	4	494	310	600 570	1.22 1.15	1.94 1.84
120x120	3898	4.54	4 ø16	2	558	356	570 600 1430	1.02 1.08 2.56	1.60 1.69 4.02
120x120	4358	5.67	4 ø20	2	515	348	570 890 1590*	1.11 1.73 1.41*	1.64 2.58 2.28*
120x130	2725	2.22	4 ø16	2	1132	696	1484* 1166	1.31* 1.03	2.13* 1.68
120x130	2725	2.13	4 ø16	4	1120	713	1643 1298* 1272	1.37 1.16* 1.14	2.30 2.08* 1.78
120x130	2725	2.18	4 ø16	6	1202	740	1272 954* 1298*	1.06 0.79* 1.08*	1.72 1.29* 1.75*
Ø100	3898	8.71	4 ø10	2	143	89	100 230	0.70 1.61	1.12 2.58
Ø150	3898	3.87	4 ø20	2	773	530	1110 990	1.44 1.28	2.09 1.87
Ø180	4358	3.23	4 ø12	4	1057	642	1540 1210	1.46 1.15	2.40 1.89
180x180	2898	1.11	4 ø25+ 4 ø16	2	3916	2274	3390 4250	0.87 1.09	1.49 1.87
200x200	3898	1.63	4 ø20	2	3360	1870	3350 3090	1.00 0.92	1.79 1.65

2.3 Results for eccentric load

The formulas used for calculating load capacity and displacements under eccentric loads are equivalent to the methods used in the Danish standard DS411 and are given in the following:

The modulus of elasticity is determined as [4]:

$$\frac{E_c}{E_{c,0}} = \sqrt{1 - k(\frac{\sigma_{c,\max}}{f_c})^2 - (1-k)(\frac{\sigma_{c,\min}}{f_c})^2} \tag{5}$$

k is set to 0.14 from limit values.

The ultimate capacity for the column is determined the traditional way – as shown in DS411 - and includes the second order moments from the deformations. The sectional forces are given by the axial force N_s and the moment $M = M_0 + (e_1 + e_2)N_s$, where M_0 is the moment from transverse loading, e_1 is the eccentricity for the axial force and e_2 is the deformation at the middle of the column.

e_2 is determined by the curvature of the column $u_m = \kappa_m \frac{l_s^2}{10}$.

$\kappa_m = \frac{\sigma_{c,\max} - \sigma_{c,\min}}{E_c \cdot \Delta h}$ where $\sigma_{c,\max}$ and $\sigma_{c,\min}$ are respectively the largest and smallest concrete compressive stress in the cross section and Δh is the distance between the points in the cross section with stresses $\sigma_{c,\max}$ and $\sigma_{c,\min}$. The stresses are given by:

$$\sigma_{c,\max} = \frac{N_s}{A} + \frac{M}{W} \text{ and } \sigma_{c,\min} = \frac{N_s}{A} - \frac{M}{W} \tag{6}$$

where A is cross-section area and W is the rotational section modulus. The ultimate bearing capacity of an eccentric loaded column is determined as the load N_{cr} where the cross-section fails due to a combination of N_{cr} and M.

The results are shown in tables 3 and 4. The tables show loads and displacements in ultimate limit state as well as the expected service loads and displacements at that level. Ultimate capacity is calculated based on "test"-properties, while design capacity is calculated based on "design"-properties. The service loads were determined from the design capacity by assuming that 60% of the load on the column is dead load, while 40% is live load with a safety factor of 1.3. The columns were not actually loaded to failure as this could have caused damage to the displacement transducers, but testing was stopped shortly after the loads had exceeded the ultimate load capacity. The initial eccentricity e_1 in the tests was 25 mm.

Table 3 - Results from column testing at ambient conditions with eccentric load – comparisons between calculated capacity and test loads.

Cross-section mm	Length mm	Slenderness index	Reinforcement	Fibre content Vol.%	Test load kN	Ultimate capacity kN	Design capacity kN	Service load kN
120x120	2725	2.13	4 ø12	4	410	403	284	257
120x120	2725	2.13	4 ø12	4	488	403	284	257
120x120	2725	2.22	4 ø20	2	604	573	359	326
120x120	2725	2.22	4 ø20	2	604	573	359	326

Table 4 - Results at eccentric load – comparisons between calculated displacements and results in tests.

Cross-section mm	Reinforce-ment	Test load kN	Meas. deform. mm	Ultimate load kN	Exp. deform. mm	Charact. load kN	Exp. deform. mm	Meas. deform. mm
120x120	4 ø12	410	43	403	70	257	9.6	8
120x120	4 ø12	488	35	403	70	257	9.6	9
120x120	4 ø20	604	44	573	61	326	9.2	8.5
120x120	4 ø20	604	42	573	61	326	9.2	8.5

2.4. Discussion

As shown in table 2, the test loads are always higher than the design capacity, and in most cases test loads are also higher than the ultimate capacity calculated with properties obtained in the material testing. This is in part due to the steel fibres, which provide the matrix with a tensile strength higher than 7 MPa [4]. The real variations in the results are lower than they appear – at least for the tests carried out at AAU – as only some of the columns were actually loaded to failure, as described earlier. In some cases the columns were slightly curved prior to testing, which led to eccentric loading, early deformations and thus a lower carrying capacity in the test. The difference for 2 similar columns is shown in figures 3 and 4, a case which was probably the most extreme. The graphs show loading of the columns along with displacements in the centre and at the quarter points. The column shown in figure 3 had a slight curvature prior to testing and, as indicated on the graph, the column started to deflect at a relatively low load and actually failed in bending, while the column shown in figure 4 showed only small deflections. In figures 5 and 6 the 2 columns are shown after the test. The column shown in figure 5 had a displacement of 30 mm at maximum load, and the failure was very ductile, while the column shown in figure 6 had a brittle type of failure where displacement at maximum load was only 2 mm.

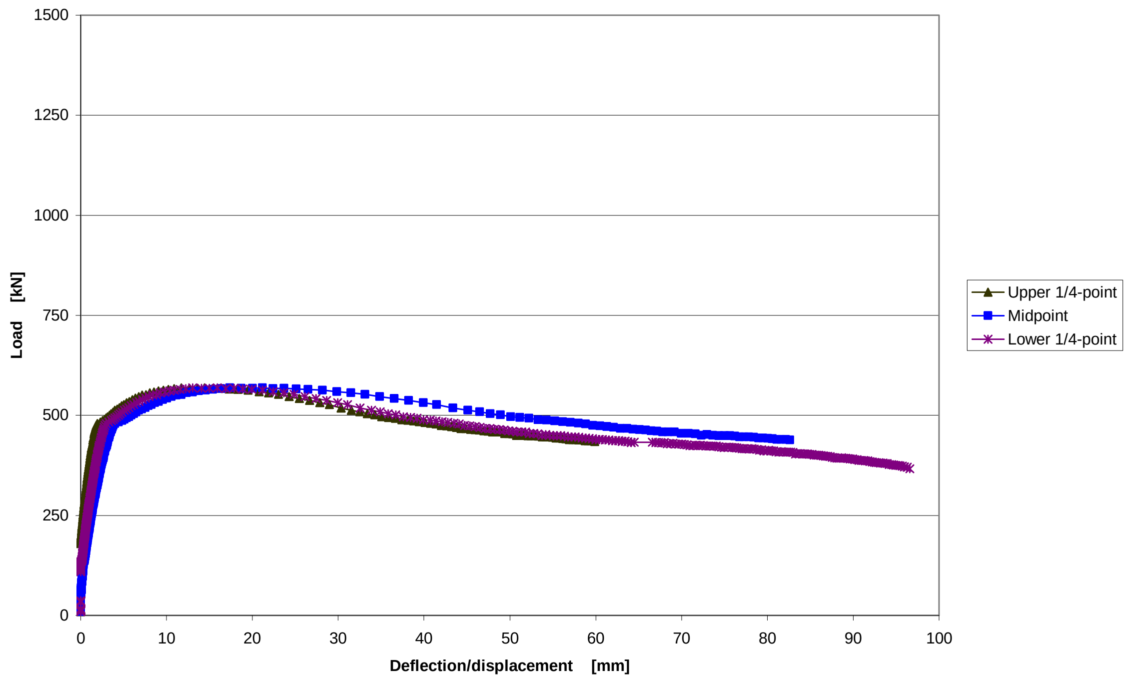

Figure 3 - Load-displacement curve for DTU test on column with 120x120 mm cross-section, 2% fibres, length 3898 mm, reinforcement 4Y16, maximum test load 570 kN.

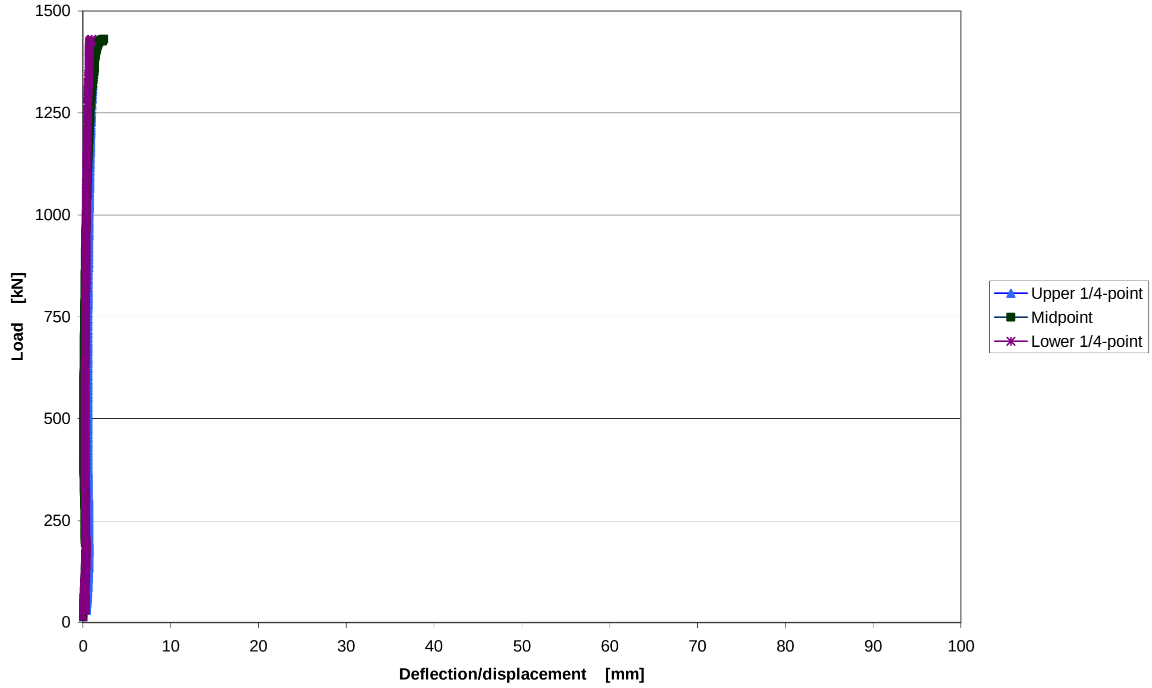

Figure 4 - Load-displacement curve for DTU test on column with 120x120 mm cross-section, 2% fibres, length 3898 mm, reinforcement 4Y16, maximum test load 1430 kN.

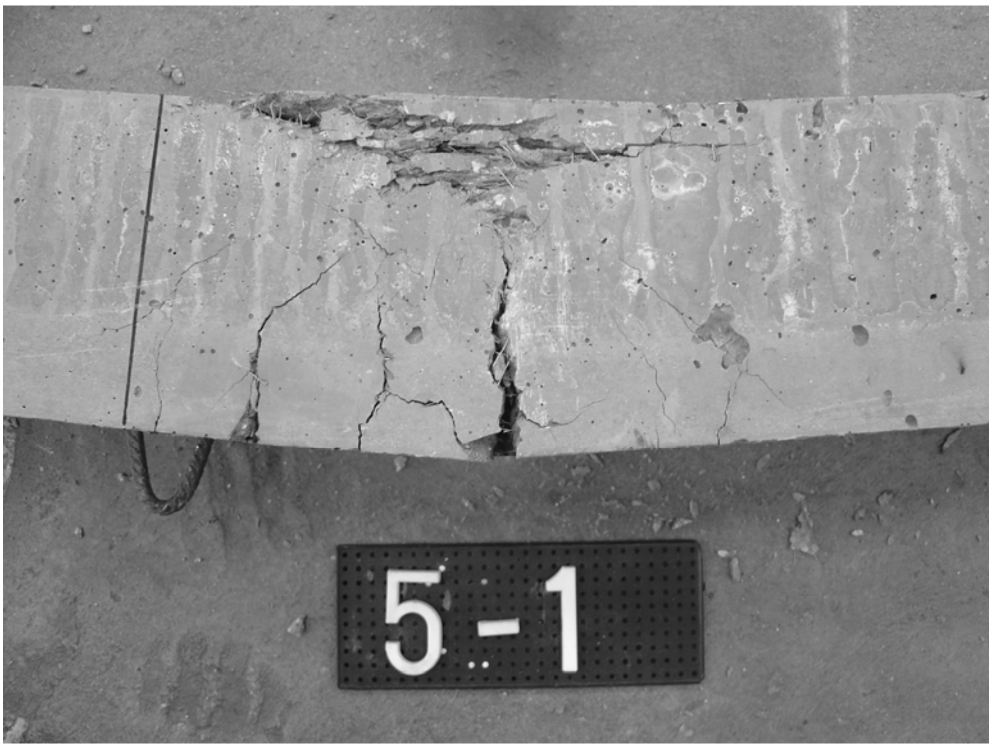

Figure 5 - Column tested at DTU with 120x120 mm cross-section, 2% fibres, length 3898 mm, reinforcement 4Y16, maximum test load 570 kN.

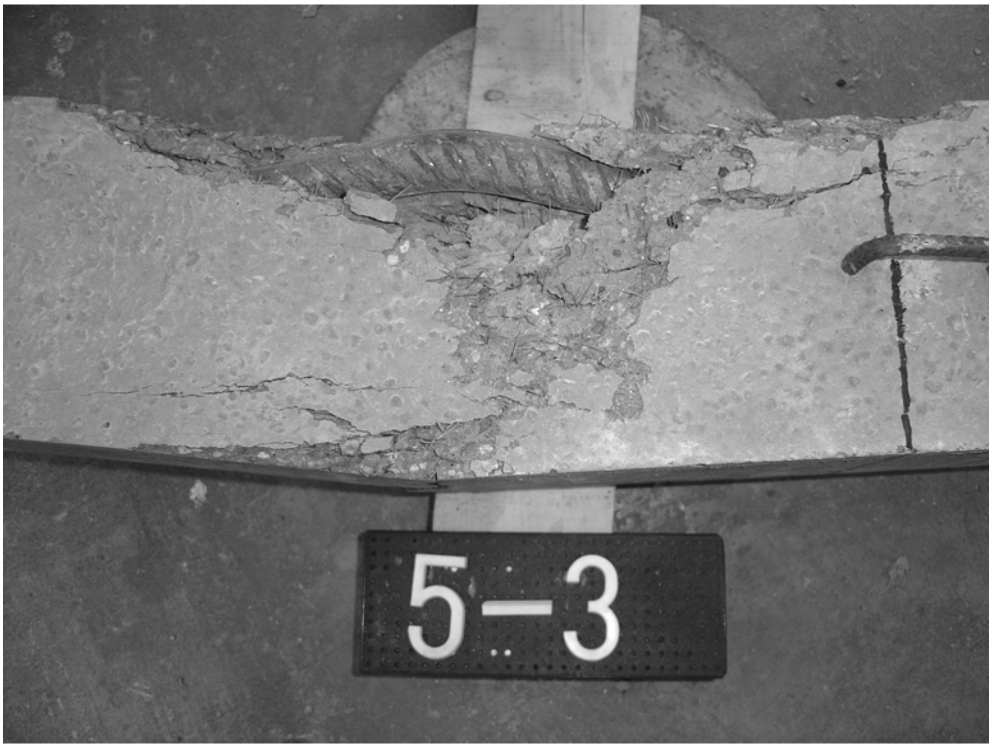

Figure 6 - Column tested at DTU with 120x120 mm cross-section, 2% fibres, length 3898 mm, reinforcement 4Y16, maximum test load 1430 kN.

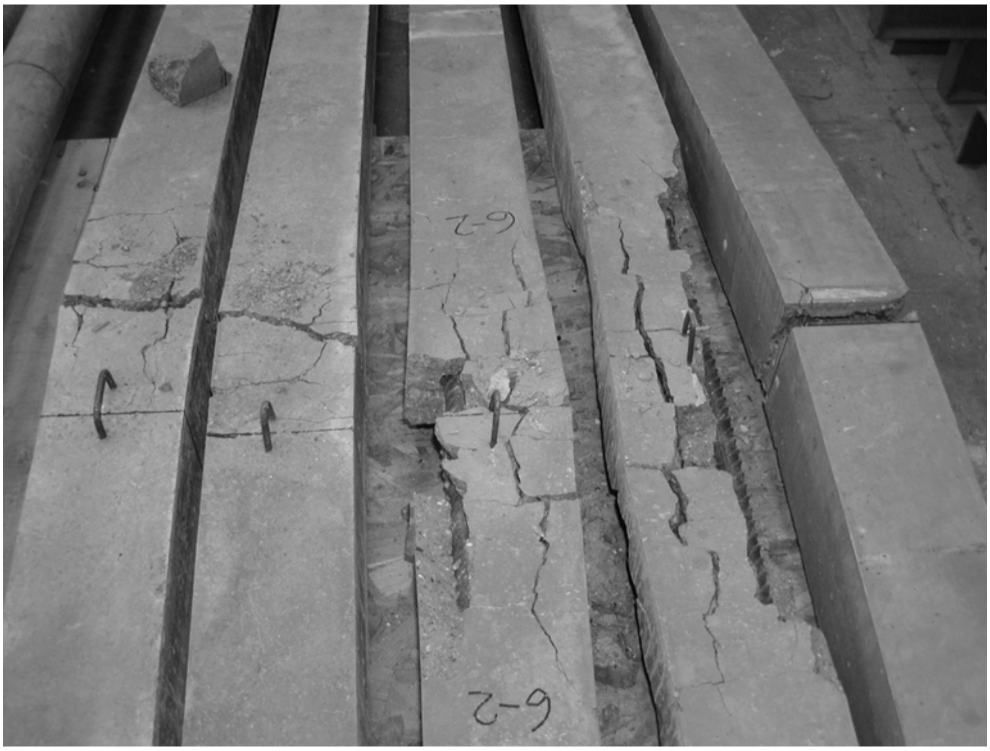

Figure 7 - Other columns tested at DTU. The one on the far right shows failure at a stirrup, while the centre column shows a cover to the reinforcement considerably larger than the nominal cover of 15 mm.

Figure 7 shows a few of the other columns tested at DTU. The picture indicates some of the other problems encountered in testing – and in interpretation of the results – as the column on the far right failed where a stirrup had been placed at mid-height of the column with a very small cover. Stirrups were used to keep the reinforcement in place, so they were either placed at a distance of 600 mm or in the case of the shorter columns stirrups were only used at the ends of the columns.

The column in the middle – with a cross-section of 120x120 mm and without steel fibres – definitely had a cover larger than 15 mm leading to a lower strength than expected. Spacers were used to maintain the cover, but investigations carried out after testing showed that while the minimum cover was adhered to with good precision, the cover could in some cases be up to 10 mm larger than expected. On the slender columns this kind of variation is very significant for predicting the ultimate capacity and will also cause eccentric loading and thus reduced capacity.

Based on the results shown in table 2, it is concluded that the formulas suggested here are applicable to a wide range of CRC columns. The formula was equally good at predicting results whether the columns had a low or a high slenderness ratio or whether they were round or rectangular. There was a slight indication that the safety factor in using the formula is reduced for columns with a large cross-section and a low slenderness index, as ratio1 in table 2 was very close to 1 for columns with cross-section 180x180 and 200x200 mm. Columns with no fibres were very brittle with a huge variation in test results. Based on the tests it was not possible to establish a difference in behaviour between fibre contents of 2, 4 and 6% - especially as only some of the columns were loaded to failure – but the general impression was that an increase in fibre content resulted in increased ductility. The formulas were originally developed in the Brite/EuRam project MINISTRUCT based on a very limited number of column tests.

For columns with eccentric load it is observed that test loads are always higher than predicted by calculation and the displacements at service capacity are smaller than expected – but very close to the calculated result.

3. COLUMNS LOADED UNDER FIRE CONDITIONS

3.1 Test programme

A number of projects have investigated the behaviour of CRC under fire conditions or at high temperatures. The most extensive project has been the HITECO project, which included testing of compressive and tensile strength as well as Young's modulus at high temperatures. These tests showed that while compressive and tensile strength was reduced at high temperatures, the loss in stiffness occurred earlier and was much more drastic – as is also observed for conventional concrete [8]. Earlier investigations have shown that even though CRC has a specific heat capacity and a thermal conductivity that differs from conventional concrete, the difference is sufficiently small that conventional approaches can be used also for CRC [9].

A total of 16 columns – all equipped with thermocouples - were tested in the current project at a fire load corresponding to the standard fire curve. The testing was done with hinges at the top and the bottom. The centre of rotation was placed so that the theoretical length of the columns was equal to the physical length plus 260 mm. The physical lengths are shown in table 7. The hinges allow for deflections in all directions. The load was kept constant during the test. Variations included shape, dimensions, length, fibre content and size and amount of rebar. Also, one series of columns was tested with just a central reinforcing bar. 15 mm of cover to the rebar was used in general, except for the largest columns which had a 25 mm cover. In addition, to investigate heat transfer properties, 2 large slabs were exposed to a standard fire, one of CRC and the other of a high performance concrete similar to the type of concrete used in the Great Belt project in Denmark. This concrete has a strength above 60 MPa and includes fly ash and micro silica. Thermocouples were placed at different depths in the 2 slabs as shown in figure 3. Thermocouples 1-3 were placed at a depth of 10 mm, 4-5 at 20 mm, 7-9 at 30 mm and thermocouples 10-12 were placed at a depth of 40 mm. The slabs were exposed from one side, while the columns were exposed to the fire from 4 sides.

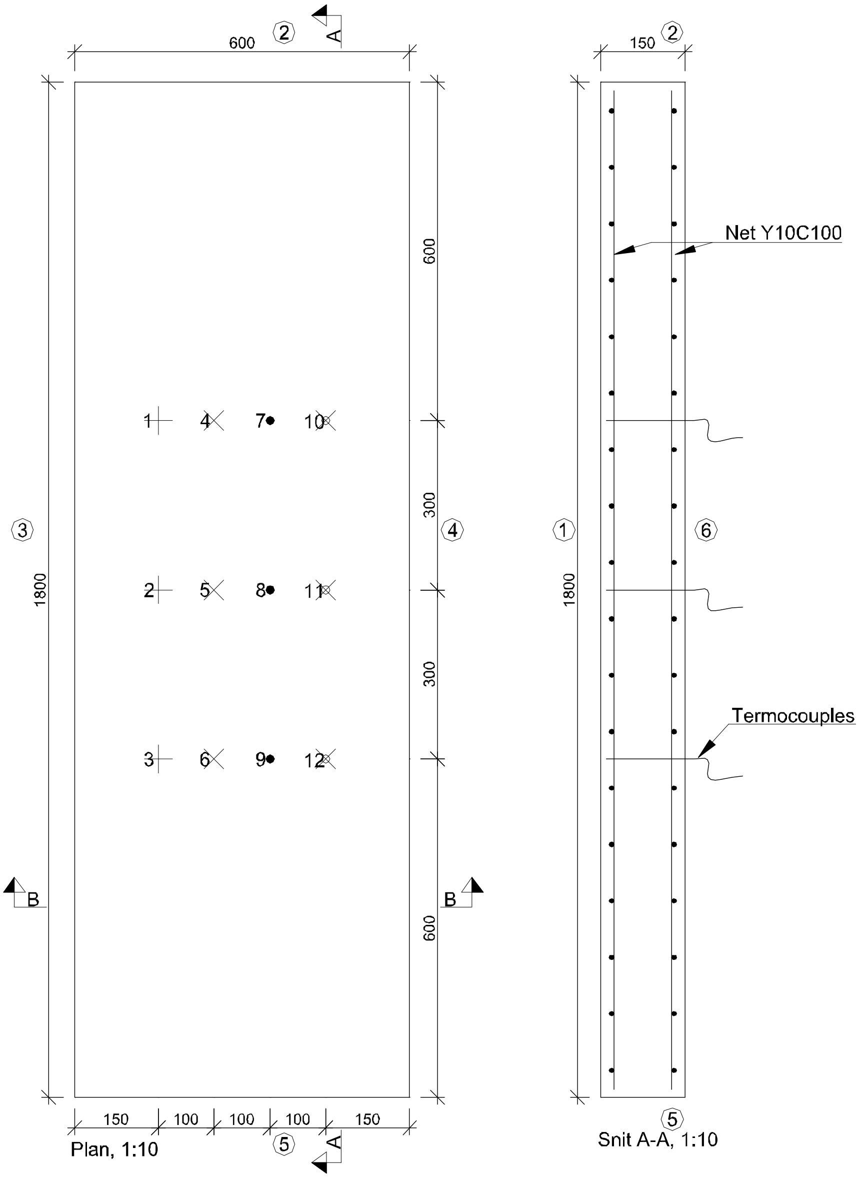

Figure 3 - Plan and cross-section of panel tested in fire exposure.

3.2 Results for slabs

Figure 4 shows a picture from the test shortly after it has started. Water is visible on the back of the slabs as it is driven out along the cords of the thermocouples. After 20 minutes it is clear that much more water is driven from the CRC slab and foam is also coming out. After an additional 15 minutes, hardly any water is coming from the conventional concrete, while there is still a lot of water coming from the CRC.

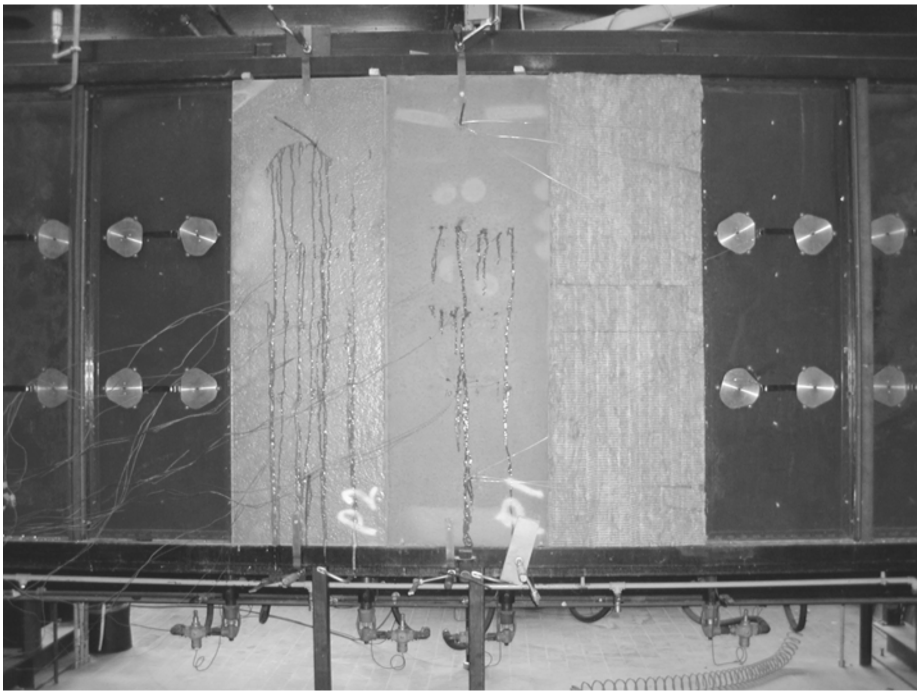

Figure 4 - Back of slabs during fire exposure. The CRC slab is the one on the left and insulation mats are shown on the right.

In tables 5 and 6 results are shown from measurements of temperatures in the 2 slabs after 30 minutes and 60 minutes. The mean value of the temperature is also shown for each depth. The measurements are compared with an estimated temperature and different ratios are shown in the table. The calculated temperature is found by using the basic calculation method from the Danish standard DS411 with the default properties for the concrete as shown below.

$$\theta_1(x,t) = 312 \cdot \log_{10}(8 \cdot t + 1) e^{-1,9k(t)\cdot x} \cdot \sin(\frac{\pi}{2} - k(t) \cdot x) \tag{7}$$

where:

$$k(t) = \sqrt{\frac{\pi \cdot \rho \cdot c_p}{750 \cdot \lambda \cdot t}} \tag{8}$$

x	:	distance from the surface in metres
t	:	time in minutes
λ	:	thermal conductivity in W/m°C – for normal concrete 0.75 W/m°C . The value is to be verified by tests.
ρ	:	density in kg/m^3.
c_p	:	Specific heat capacity – for normal concrete 1000 J/kg °C

Table 5 – Measured temperatures – in ºC – and mean values after 30 minutes of testing.

	10 mm cover		20 mm cover		30 mm cover		40 mm cover	
CRC	464 439 464	456	310 336 346	331	221 242 252	238	162 178 185	175
Concrete	291 336 333	320	256 280 278	271	187 205 196	196	159 141 156	152
DS411 value		491		310		186		104
CRC/concrete		1.42		1.22		1.22		1.15
CRC/DS411		0.93		1.07		1.28		1.68
Concr./DS411		0.65		0.88		1.05		1.46

Table 6 – Measured temperatures – in ºC – and mean values after 60 minutes of testing.

	10 mm cover		20 mm cover		30 mm cover		40 mm cover	
CRC	670 636 651	652	512 535 544	530	401 411 433	415	304 331 335	323
Concrete	475 515 515	502	431 453 453	446	349 366 350	355	281 296 306	294
DS411 value		627		460		329		229
CRC/concrete		1.30		1.19		1.17		1.10
CRC/DS411		1.04		1.15		1.26		1.41
Concr./DS411		0.80		0.97		1.08		1.29

3.3 Results for columns

The results for the columns subjected to fire are shown in table 7.

*Table 7 - Results from the fire tests. * indicates that the column was not tested to failure.*

Cross-section mm	Length mm	Reinforcing bars	Fibre content %	Test load kN	Cover mm	Time min	Temp. at re. bar ºC		Temp. at centre ºC
120x120	3420	4 ø12	2	200	15	31	587	479	261
120x120	3420	4 ø12	2	200	15	27	551	412	201
120x120	3420	4 ø12	2	200	15	29	540	456	230
120x120	3420	1 ø25	2	180	15	22	158	130	
120x120	3420	1 ø25	2	180	15	32	241	234	
120x120	3420	1 ø25	2	100	15	36	221	308	
120x120	2725	4 ø20	2	160	15	58	690	672	505
120x120	2725	4 ø20	2	130	15	62	784	732	547
120x120	2725	4 ø20	2	160	15	55	748	690	533
Ø 150	3420	4 ø20	4	180	15	58	781	659	531
Ø 150	3420	4 ø20	4	180	15	52	655	630	563
Ø 150	3420	4 ø20	4	160	15	62	728	669	514
200x200	3420	4 ø20	2	400	25	79*	721	762	649
200x200	3420	4 ø20	2	1000	25	60*	551	436	201
200x200	3420	4 ø20	2	1000	25	89	700	615	318

The temperatures in the columns were typically measured with 10 thermocouples, but not all of the results have been included in the table. The two values given at the reinforcing bar are from thermocouples actually placed on the bars. The first value is an average from the thermocouples placed on two different bars as close to the surface as possible, while the second value is an average from the thermocouples placed on the same two bars, but at the “back” of the bar. In some cases the difference between two values that should ideally have been the same – and which are just given in the table as an average – was up to 100 ^{o}C. In the case of the column with one central reinforcing bar only two thermocouples are placed on the bar and they are spaced 90º apart. In the case of the central bar, there is no measurement from the centre of the column. The values measured on the reinforcing bars are generally higher than the values measured with other thermocouples, which are placed at an equivalent depth in the concrete.

The temperatures shown are measured at the end of the test and they will thus vary from one column to another as exposure time was different for each column. The table also shows the test load, which was kept constant for the duration of the test, and the test time. Typically the test was continued until the column had very large deformations. The columns did not actually break, but sustained a very ductile failure as can be seen in figure 9. In a few cases – indicated with * in table 7 - the test was stopped while the column was still able to carry the load with no problems.

Figure 9 - Some of the columns after fire exposure.

3.4 Discussion

In general the columns had a lower fire resistance than expected. As shown in table 7, most of the columns have a fire resistance time shorter than 1 hour. This was also the case for the columns that have a central reinforcing bar, which is a bit surprising as the temperature at the rebar is only a few hundred degrees. However, with the slender columns the heat capacity is limited and the increase of temperature overall is high. This has led to a decrease in stiffness, which – along with an effect of thermal stresses - from the appearance of the columns after testing has had more influence on the fire resistance time than reduced strength of the reinforcement. As mentioned earlier at high temperatures the reduction in Young's modulus will occur faster than reduction in strength for CRC – as for conventional concrete – which is a problem for slender columns.

In order to be able to test the columns at an early age, the columns had been dried out in a humidity chamber and in a number of cases this had led to some curvature even prior to testing. This was possibly due to reinforcement that was not properly placed or it could be caused by the columns only being supported at the ends during drying. This initial curvature and possible internal eccentricity from the placing of the reinforcing bars causes eccentric loading and reduces the fire resistance time observed in the tests.

Based on evaluation of the results – and of results from earlier tests in the HITECO project – it is difficult to establish design rules, but some general guidelines – in the form of a list of examples - can be established. As an example a column with ø150 mm cross section, 2% fibre content, 4Y20 bars with a minimum cover of 15 mm and a length of no more than 3420 mm is considered BS60 – capable of sustaining a standard fire for 60 minutes – provided the central load is below 160 kN.

4. CONCLUSIONS

A number of columns have been tested in the project, including tests with central load and eccentric load under ambient conditions and with central load under fire conditions. The columns were designed to cover a range of variations in parameters such as slenderness, shape, size, reinforcement, fibre content etc.

The tests carried out in ambient conditions – for central and eccentric load - showed good correlation between test results and expected bearing capacities calculated according to design guides established based on earlier CRC investigations. In general the CRC columns tested in ambient conditions can be shown to have a capacity equivalent to that of a corresponding steel tube column of similar cross-section and length.

The fire resistance tests demonstrated that the slender columns were very sensitive to thermal stresses and changes in stiffness due to high temperatures – as well as to imperfections prior to loading. Thus very slender columns failed early in the tests even though temperatures at the reinforcement were low. Failure was always ductile and there was no spalling. Not all columns were tested to failure and it was demonstrated that fire resistance above 1 hour can be achieved.

The results of the project have already been utilised in a number of projects where CRC columns have been used. Generally, slenderness is kept below 3.5 to avoid premature failure in case of fires.

Figure 10 CRC Balcony slabs and columns produced by Hi-Con at Askehaven, Vejle in Denmark.

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