TEST OF RETANGULAR CONFINED CONCRETE COLUMNS FOR STRENGTH AND DUCTILITY


Eythor Thorhallsson **Pall V. Bjarnason**

Abstract

This research test the ductility and strength of reinforced concrete columns cast by using Icelandic cement and aggregate. Icelandic aggregate has a high porosity. Pour water in Icelandic aggregate is about 3–8% while pore water in an aggregate in North-Europe is about 0,5%.

The question of the ductility is specially interesting in Earthquake Engineering design. For Iceland this information's is valuable as part of the country has high earthquake risk. The test programme consisted of reinforced concrete columns with different longitudinal reinforcement and hops spacing. The large scale columns were tested under compressive concentric loading after 28 days of curing at the Structural Engineering and Composites Laboratory at Reykjavík University (SEL). The test specimens were loaded on a hydraulic press with load controlled capabilities.

The test results showed that ductility capacity of Icelandic concrete is quite lower than compared tests from Europe and USA. This result indicates that it is necessary to establish specially formulas for Icelandic confined concrete.

Keywords: Confined concrete, Rectangular concrete columns, Confinement.

1 Test program

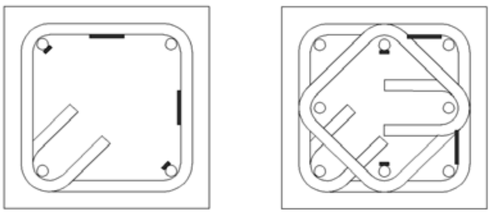

Fig 1. Type A Type B

The test program consisted of reinforced concrete columns, which were cast as large scale test specimens 180 x 180 x 1400 mm. The specimens are identified with a letter, A or B, corresponding to the longitudinal reinforcement ratio and a number which corresponds to the hoop spacing. Columns in type A are light confined and columns in type B are highly confined.

The columns were cast with specified 28-day strength of 25 MPa with 20 mm maximum size aggregate. The concrete was mixed at the concrete plant BM-Vallá under high supervision. Cylinders were also cast to determine the strength of plain concrete and to measure modulus of elasticity.

Table 1. Material Properties

	Concrete			Longditudinal reinforcement			Lateral reinforcement			
Column	f_{ck} mesured (MPa)	E_c mesured (GPa)	Concrete cover (mm)	Bars	ρ_l (%)	f_y	d_h (mm)	s (mm)	ρ_h (%)	f_{yh}
A1	29	23	15	4 K12	1,40	500	8	45	0,0315	500
A2	29	23	15	4 K12	1,40	500	8	90	0,0157	500
A3	29	23	15	4 K12	1,40	500	8	135	0,0105	500
A4	29	23	15	4 K12	1,40	500	8	180	0,0079	500
B1	29	23	15	8 K10	1,94	500	8	45	0,0537	500
B2	29	23	15	8 K10	1,94	500	8	90	0,0269	500
B3	29	23	15	8 K10	1,94	500	8	135	0,0179	500
B4	29	23	15	8 K10	1,94	500	8	180	0,0134	500

The columns were reinforced with ribbed steel bars both longitudinally and laterally. The bars have the yield strength of about 500 MPa and steel grade of B500C. All longitudinal bars are confined with lateral hoop reinforcement with different spacing for each test specimen. The hoop spacing for each column type can be seen in Table 1 and on Figure 1.
For each steel bar diameter, three bars were tested in tension to measure the accurate yield strength of the reinforcing steel. The average yield strength was measured 628 MPa. The reinforcement deformations were measured by electrical resistance strain gages, glued to the steel bars. Two longitudinal steel bars at the opposite corner in each column of type A and opposite side of each column of type B were instrumented at their middle lengths. A set of ties located at the centre of each column specimen was also instrumented with strain gages placed on two adjacent sides of each tie of a chosen set. The axial displacement of each specimen was recorded during testing. The positions of the strain gages can be seen on Figure 1 for reinforcement type A and B. Strain gauge is marked with black line marks on the steel bars on Figure 1.

2 Test results

The results can be shown in Table 2. The maximum axial load N_{max} is shown and the calculated axial value N_0 from Eurocode 2. During the ascending part of loading, confinement has little or no effect, and the concrete cover is visually free of cracks up to the first peak. That corresponds to the load N_{C1} when the concrete cover suddenly separates, vertical cracks are visual. At this point, the stress in the transverse reinforcement is below 50 % of its yield stress. After the first peak is reached the concrete axial strength loses 5-10 % of its maximum value due to the sudden spalling of cover. At this stage, lateral strain increases significantly and the passive confinement becomes very significant. The concrete core gains significant strength, while the concrete cover falls off. In highly confined test specimens the lateral strain reached the steel maximum yield strength at the second peak. The second peak corresponds to the N_{C2} and is the maximum axial load. In poorly confined test specimens the steel stresses at the second peak were recorded down to 50 % of tie yield strength and in highly confined specimens some ties ruptured. The value of N_{C2} may be lower or higher than N_{C1}. It depends on efficiency of confinement at each specimen. Specimens with poor confinement did not show a well defined second peak.

At the end of testing longitudinal bars had been buckled. Some ties in well confined specimens had ruptured and inclined shear sliding surfaces separated the concrete. This shear sliding causes the axial strength to drop very rapidly. The inclination of the shear sliding plane with the vertical axis varied from about 25° for poor confined specimens up to about 45° for highly confined specimens.

Table 2. Experimental Results

	Axial loads							Axial strains			
Column	N_{max} (kN)	N_0 (kN)	N_{max}/N_0	N_{C1} (kN)	N_{C1}/N_{0C}	N_{C2} (kN)	N_{C2}/N_{0CC}	ε_{C1}	ε_{C2}	ε_{CU85N0}	ε_{25Nmax}
A1	1162,70	1013,70	1,147	1098,20	1,402	1162,70	2,339	0,0033	0,0039	0,0104	0,0287
A2	1149,90	1013,70	1,134	831,60	1,062	1149,90	2,313	0,0027	0,0037	0,0062	0,0142
A3	1046,50	1013,70	1,032	831,60	1,062	1046,20	2,105	0,0024	0,0031	0,0057	0,0125
A4	953,90	1013,70	0,941	549,02	0,701	953,60	1,919	0,0019	0,0031	0,0045	0,0105
B1	1275,60	1097,30	1,162	1098,24	1,402	1275,35	2,566	0,0048	0,0095	0,0348	0,0599
B2	1124,30	1097,30	1,025	913,30	1,166	1124,02	2,261	0,0037	0,0082	0,0125	0,0223
B3	1007,00	1097,30	0,918	886,23	1,132	1006,74	2,025	0,0032	0,0060	0,0087	0,0124
B4	993,80	1097,30	0,906	831,63	1,062	993,51	1,999	0,0026	0,0034	0,0041	0,0097

Table 2 shows also the strain at each peak. Strain, ε_{C1} corresponding to N_{C1} ranges from 0,00190 – 0,00475, with a mean value of 0,00307. These values are equal or greater than the value ε_{C1} according to Eurocode, 0,00200. The strain, ε_{C2} corresponding to N_{C2} when the confined concrete reaches its maximum strength, ranges from 0,00306 – 0,00945, with a mean value of 0,00510. Maximum usable strain according to Eurocode, ε_{CU} is 0,0035, according to these results maximum usable strain for sections cast with Icelandic concrete should not exceed 0,0030. The strain ε_{CU85No} is the ultimate strain according to Eurocode and corresponds to 85% of the maximum unconfined strength, N_0 on the descending branch of the stress-strain curve. The ultimate strain ranges from 0,00406 for poor confined test specimens to 0,03484 for well confined test specimens. Compared to Cusson and Paultre, 1994 when the concrete reaches its maximum strength, strain ranges from 0,0033 – 0,0321.

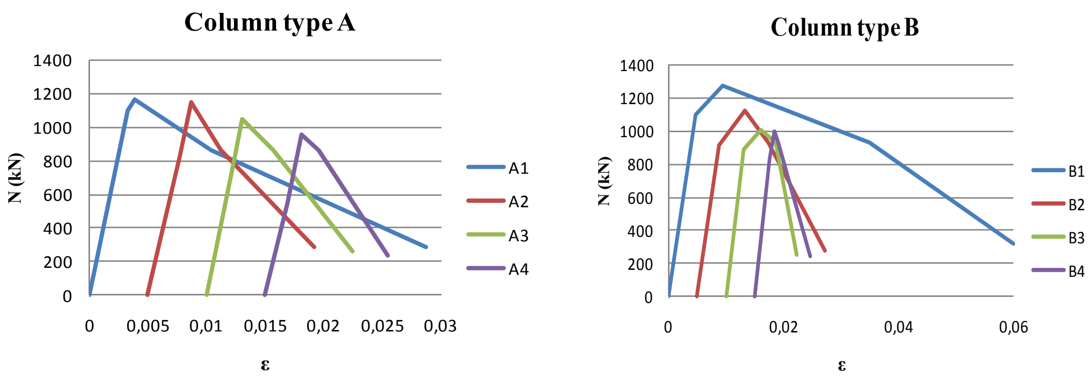

Figure 2. Total load versus axial strain curves for test specimens A and B

3 Discussions

This research program has shown the effectiveness of confined concrete compared to unconfined specimens. The confinement provided by the lateral hoop figuration in a highly confined specimen improves both axial strength and ductility of the column. The most improvement in ductility is in a specimen B1 and axial strength improve in the specimen B1 and B2. Axial strength in the specimen B3 and B4 is quite lower than expected.

The gain in axial strain in a highly confined specimen is great from peak 1 to peak 2. The axial strain gain in light confined specimens from peak 1 to peak 2 is little as was expected. The maximum value of $\varepsilon_{C2}/\ \varepsilon_{C1}$ for specimen B is 2,2 while the maximum value for specimen A is 1,61. Deformability of about 3,5 % as results for section B1 showed is a lot for structural concrete and counts as high ductile and well confined section.

From these test results, we can see that a good ductility was reach for only one specimen, section B1 with hoop spacing of 45 mm. All specimens with hoop spacing more than 90 mm show a little gain in axial strain and lack of ductility behaviour. The configuration of lateral reinforcement in type B specimen has a good effect in improving the strength of the column.

What is also interesting is the maximum usable strain should not exceed 0,0030 in concrete sections cast with Icelandic concrete. The drop in axial strength was between 5–10 % for a concrete cover of 15 mm, which shows that this drop will increase with increased concrete cover. For well confined specimens tie yield strength has increased effects. Measured tie yield strength in this research was 628 MPa and in columns A1 and B1 some ties ruptured.

There are several things from these results that could have effects in the future on structural design in Iceland. This research came up with results that need to be further investigated in the nearest future. It is clear from this research results that confined concrete cast with Icelandic concrete acts differently than confined concrete does from foreign researches.

References

[1] Cusson, D., and Paultre, P. (1994). *High-Strength Concrete Columns Confined by Rectangular Ties.* J. Struct. Eng., ASCE, 120(3), 783-804.

[2] Eurocode 2. (2004). Design of concrete structure – Part 1: General rules and rules for buildings. EN1992-1. European Committee for Standardization, Brussel.

[3] Guðni Jónsson. (2006), Deformations in concrete – Modulus of elasticity and creep. Innovation Center Iceland.

[4] NS 3473: (1992), Norwegian Concrete code, Norwegian Concrete Association.

[5] Cusson, D., Larrard, F., Boulay, C. and Paultre, P. (1996). *Strain Localization in Confined High-Strength Concrete Columns.* J. Struct. Eng, 1055.

[6] Cusson, D., and Paultre, P. (1995). *Stress Strain Model For Confined High-Strength Concrete.* J. Struct. Eng., ASCE, 121(3), 7257

[7] Kappos, AJ. and Penelis, G. (1996). *Earthquake Resistant Concrete Structures.* Taylor & Francis Group.

[8] Park R,Priestley MJN, Gill WD (1982). *Ductility of Square Confined Concrete Columns.* J. Struct. Eng. ASCE 108(4) 929-950.

[9] Sheikh SA and Uzumeri SM. (1980). *Strength and Ductility of Tied Concrete Columns.*J. Struct. Eng. ASCE 106(5) 1079-1102.

[10] Paultre, P and Légeron F. (2008). *Confinement Reinforcement Design for Reinforced Concrete Columns.*J. Struct. Eng. 134(5) 738-749.

[11] Mander, J. B., Priestley, M. J. N., and Park, R. (1988). *Observed stress-strain behavior of confined concrete.* J. Struct. Engrg., ASCE, 114(8), 1827-1849.

[12] Cusson, D., and Paultre, P. (1992). *Behavior of high-strength concrete columns confined by rectangular ties under concentric loading. Report No. SMS-9202,* Dept. of Civ. Engrg., Univ. of Sherbrooke, Sherbrooke, Canada.

Associate Prof. Eythor Thorhallsson.
✉ Reykjavik University
School of Science and Engineering
Menntavegur 1
101 Reykjavik, Iceland
☎ +354 5996200
🗐 +354 5996301
☺ eythor@ru.is
URL http://www.reykjavikuniversity.is/

Pall V. Bjarnason, C.Eng.
✉ Reykjavik University
School of Science and Engineering
Menntavegur 1
101 Reykjavik, Iceland
☎ +354 5996200
🗐 +354 5996301
☺ pallb06@ru.is
URL http://www.reykjavikuniversity.is/