Concrete Beton, Volume 119, pp 8-15, September 2008

Self-Compacting Concrete Columns under Concentric Compression

Authors: Chien-Hung Lin[1], Chao-Lung Hwang[2], Shih-Ping Lin[3] and Chih-Hsiuan Liu[4]

[1]Department of Construction Engineering, National Chung Hsing University, Taichung, Taiwan
[2]Department of Construction Engineering, National Taiwan University of Science and Technology, Taipei, Taiwan
[3]Consultant, Taichung, Taiwan
[4]Former Graduate student, National Chung Hsing University, Taichung, Taiwan

ABSTRACT: Two series of column specimens were tested to investigate the behaviour of self-consolidating concrete (SCC) columns under concentric compression. The first series contained 16 columns made with normal concrete (NC) and the second 16 columns were made with SCC. The test variables included the concrete strength, amount of longitudinal reinforcement, volumetric ratio of transverse reinforcement, strength of transverse reinforcement and arrangement of transverse reinforcement. Comparisons were made between the SCC and NC specimens. Behaviour of the SCC used in this study was also compared with that of high-flowability concretes in other studies. The results show that SCC can have better structural performance than NC, as long as the concrete is properly proportioned. The ductility and crack control ability of SCC columns are better than NC columns. Stiffness of SCC is also higher than that of NC. Mechanical behaviour of the SCC in this study was better than other SCC compared, due to larger amounts of coarse aggregate used.

Journal Contact Details:
PO Box 75364
Lynnwood Ridge
Pretoria, 0040
South Africa
+27 12 348 5305

CONCRETE SOCIETY
OF SOUTHERN AFRICA

admin@concretesociety.co.za

Concrete Beton, Volume 119, pp 8-15, September 2008

www.concretesociety.co.za

Self-Consolidating Concrete Columns under concentric compression

Chien-Hung Lin, Chao-Lung Hwang, Shih-Ping Lin, and Chih-Hsiuan Liu

ACI member **Chien-Hung Lin** is a Professor in the Department of Civil Engineering, National Chung Hsing University, Taichung, Taiwan. He is a member of Joint ACIASCE Committee 441, Reinforced Concrete Columns. His research interests include reinforced concrete and pre-stressed concrete.

Chao-Lung Hwang is a Professor in the Department of Construction Engineering, National Taiwan University of Science and Technology, Taipei, Taiwan. His research interests include concrete technology and construction materials.

Shih-Ping Lin is employed at a consulting company in Taichung, Taiwan. He received his MS and PhD from the Department of Civil Engineering, National Chung Hsing University.

Chih-Hsiuan Liu is a former graduate student at National Chung Hsing University where he received his MS in civil engineering.

Two series of column specimens were tested to investigate the behaviour of self-consolidating concrete (SCC) columns under concentric compression. The first series contained 16 columns made with normal concrete (NC), and the second 16 columns were made with SCC. The test variables included the concrete strength, amount of longitudinal reinforcement, volumetric ratio of transverse reinforcement, strength of transverse reinforcement, and arrangement of transverse reinforcement. Comparisons were made between the SCC and NC specimens. Behaviour of the SCC used in this study was also compared with that of high-flowability concretes in other studies. The results show that SCC can have better structural performance than NC, as long as the concrete is properly proportioned. The ductility and crack control ability of SCC columns are better than NC columns. Stiffness of SCC is also higher than that of NC. Mechanical behaviour of the SCC in this study was better than other SCC compared due to the larger amount of coarse aggregate used.

INTRODUCTION

Self-consolidating concrete (SCC) has become more popular in the past decade due to its excellent flowability. SCC consolidates under its own weight without the need of external vibration, and its flowability is even better than that of high workability concrete (HWC).[1,2] HWC contains more coarse aggregate and exhibits lower slump than SCC. It is quite suitable for casting heavily reinforced concrete members such as columns and beam-column joints with SCC. In seismic design, it is usually required that a large amount of transverse reinforcement should be provided to confine the core concrete and longitudinal reinforcement in those members.[3] Apart from easier concrete placement, it has been found that SCC can have better bond with reinforcing bars[4] and better ductility.[5] In some cases, however, SCC may also exhibit lower stiffness and strength[5] and less ductility[6] than NC. To achieve high flowability, it is usually necessary to reduce the amount of coarse aggregate to some extent. For SCC, the amount of coarse aggregate used in the concrete usually ranges from 750 to 850 kg/m^3 (46.7 to 53.0 lb/ft^3) of concrete. The amount of coarse aggregate and water in the fresh concrete, however, has a significant effect on the behaviour of the hardened concrete. The coarse aggregate provides restraint when the cement paste deforms. A larger amount of water used in the concrete also tends to relatively reduce the amount of coarse aggregate, and it could result in lower concrete strength. In addition, less coarse aggregate and a larger amount of water in the fresh concrete would result in higher creep and shrinkage of the hardened concrete, as implied in the ACI 209 report.[7] In this study, the amount of coarse aggregate in SCC was kept approximately the same as in normal concrete (NC) greater than 900 kg/m^3 (56.1 lb/ft^3). The amount of water was kept as low as possible.

RESEARCH SIGNIFICANCE

SCC can be used to ease the casting of heavily reinforced construction elements such as columns and beam-column joints. SCC, however, may have lower stiffness and ductility than NC based on the same strength condition.[5,6] Research on HWC columns has been reported,[1,2] and it suggests that a larger amount of coarse aggregates should be used in the concrete for better structural performance. With a larger amount of coarse aggregate, the column has larger stiffness and the concrete spalls more gradually after the column reaches the peak strength. This study used approximately the same amount of coarse aggregates in the SCC as normally used in NC. Test results of the SCC columns under concentric compression were compared with NC, HWC,[1] and other SCC[6] column specimens. The results show that structural performance of the SCC used in this study was better than NC and other SCC specimens.[6]

Proportioning of SCC

It is rather easy to meet the flowability requirements in proportioning SCC, but it becomes difficult when the mechanical behaviour of hardened concrete is to be considered at the same time. The mixture proportions of SCC in this study followed the "densified mixture design algorithm (DMDA)."[8-11] The design procedures are different from the conventional ACI method.[12] In ACI procedures, it begins with determining the amount of water and cement and ends with calculating the amount of fine aggregates. In the DMDA method, it begins with determining the maximum density of solid materials and ends with calculating the amount of water and cement.

It has been found that the maximum packing density of aggregate is advantageous for making concrete regarding workability, strength, stiffness, creep, shrinkage, permeability, and durability. The DMDA method applies the particle packing concept and develops a particle filling model to proportion material mixture by minimizing voids and hence maximizing the weight of larger particles. Class F fly ash is used as fine particles to fill the void between the aggregates rather than as partial replacement for cement or sand in the traditional method, and it is expected to react with free lime generated from the hydration of cement to chemically form low-density gel. Minimum cement paste acts as glue to bind 426 ACI Structural Journal/July-August 2008 all solid particles together and to fill the rest of the void. In this study, Type I Portland cement and blast-furnace slag cement are used and deemed major binders. Slag partially replaces cement not only to maintain a proper amount of binder for early strength but also to reduce the stickiness of SCC due to the large amount of fine material used. The carboxylic acid-based high-range water-reducing admixture added to the mixture develops a steric hindrance effect on the surface of cement particles and can significantly reduce the internal shear force and greatly reduces the water content.

It significantly increases the flowability of concrete under minimum water content to ensure SCC with no obvious bleeding and segregation. Consequently, the cement content of concrete will be significantly reduced. Also, because the size of coarse aggregate affects the flowability of concrete, a smaller-sized coarse aggregate was used in this study, which was limited to 10 mm (0.4 in.). Details of the DMDA approach can be found elsewhere.[10,11]

To prevent bleeding and segregation, silica fume was added to the SCC in this study. Silica fume can be categorized as a type of viscosity-modifying admixture (VMA).[12] With the use of silica fume, a larger amount of coarse aggregate can be used in the SCC, resulting in high flowability and better structural performance. With the use of silica fume, SCC can contain more than 900 kg/m^3 (56.1 lb/ft^3) of coarse aggregate without difficulty.[12]

The SCC in this study satisfied the Japan Society of Civil Engineers (JSCE) flowability requirements.[13] The flow test, V-funnel test, and U-box test were employed to estimate the flowability of the fresh concrete. The flow test is almost the same as the slump test, except that the flow test measures the spread diameters of the concrete specimens after removing the slump cone. The V-funnel test measures the time that fresh concrete flows out from a funnel.

Table 1–Specimen properties.

Specimen no.	f_c', MPa	ρ_g	f_{yh}, MPa	s, mm	$\rho_s f_{yh}$, MPa	Tie arrangement
N1	31.1	0.0255	*	*	*	A
N2	43.2	0.0255	*	*	*	A
N3	56.1	0.0255	*	*	*	A
N4	31.1	0.0255	447.2	90	5.87	B
N5	44.5	0.0255	447.2	90	5.87	B
N6	55.1	0.0255	447.2	90	5.87	B
N7	44.5	0.0172	447.2	90	5.87	B
N8	40.4	0.0344	447.2	90	5.87	B
N9	41.3	0.0255	560.0	90	6.02	B
N10	44.2	0.0255	339.4	90	4.35	B
N11	43.7	0.0255	447.2	150	3.51	B
N12	43.2	0.0255	447.2	60	8.80	B
N13	43.2	0.0255	447.2	60	13.18	C
N14	41.6	0.0255	339.4	68.4	5.73	B
N15	44.5	0.0255	560.0	112.8	4.80	B
N16	44.4	0.0255	447.2	135	5.87	C
S1	29.0	0.0255	*	*	*	A
S2	40.9	0.0255	*	*	*	A
S3	53.7	0.0255	*	*	*	A
S4	30.2	0.0255	447.2	90	5.87	B
S5	41.9	0.0255	447.2	90	5.87	B
S6	53.2	0.0255	447.2	90	5.87	B
S7	40.6	0.0172	447.2	90	5.87	B
S8	39.2	0.0344	447.2	90	5.87	B
S9	41.2	0.0255	560.0	90	6.02	B
S10	43.1	0.0255	339.4	90	4.35	B
S11	41.8	0.0255	447.2	150	3.51	B
S12	42.5	0.0255	447.2	60	8.80	B
S13	42.2	0.0255	447.2	60	13.18	C
S14	43.7	0.0255	339.4	68.4	5.73	B
S15	42.0	0.0255	560.0	112.8	4.80	B
S16	42.7	0.0255	447.2	135	5.87	C

*Specimens with no ties.
Note: 1 MPa = 0.145 ksi; 1 mm = 0.0394 in.

The concrete with good flowability would take a shorter time to flow out. The U-box apparatus serves as a measurement for the self consolidation of concrete. The fresh concrete is placed in the upper box and it flows through the gate into the lower box.

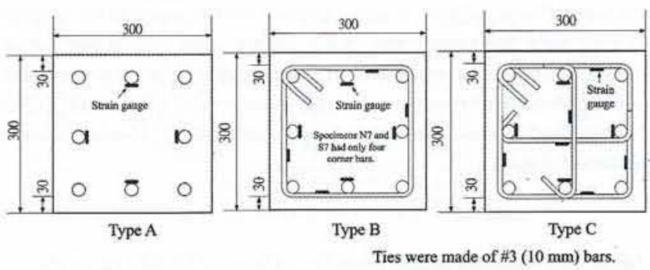
When the flow stops, self-compaction performance is estimated by the height reached in the lower box.

TEST PROGRAMME

This is the second project of a series of study on high flowability concrete columns under concentric compression carried out at the National Chung Hsing University. The first one was on HWC[1] columns. The test programme in this second project is similar to that presented in Reference 1 for easier comparison. Thirty-two column specimens were constructed and tested in this project. Sixteen of the specimens were made with NC (slump less than 200 mm [8 in.]), whereas the others were made with SCC. The column ends were tapered to prevent unexpected local failure at the ends, and the test region was in the middle (600mm [24 in.]) of the specimen.

The cross section of the columns was 300 x 300mm (12 x 12 in.) in size, as shown in Fig. 1. Three concrete strengths were used: 28, 41, and 55MPa (4, 6, and 8 ksi). The yield strength of longitudinal reinforcement was 552MPa (80 ksi). The specimen properties are shown in Table 1. The f_c' and f_{yh} values shown in Table 1 are actual material strengths. Series N represents NC columns and Series S represents SCC columns.

Six specimens (N1, N2, N3, S1, S2, and S3) without ties were prepared as unconfined specimens to establish the in-place strength of concrete in columns to be compared with the standard cylinder test results. The spacing of 10mm (0.4 in.) transverse reinforcement was 60, 68, 90, 113, 135, and 150mm (2.4, 2.7, 3.5, 4.4, 5.3, and 5.9 in.) for other specimens, and the amount of transverse reinforcement used met the requirements of ACI 318-05, Section 21.4.4,[3] for seismic design.



(a) Cross sections and test strain gauge setup.

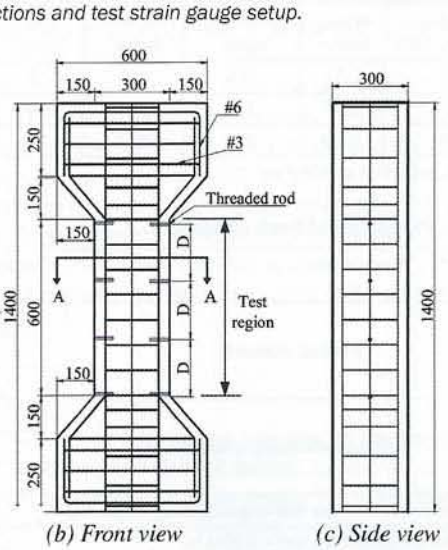


Fig. 1–Specimen details.

The mixture proportions for the concrete are shown in Table 2.

Table 2(a)—Mixture proportions for normal concrete

Concrete strength, MPa	Water/ cement	Cement, kg/m^3	Water, kg/m^3	Coarse aggregates, kg/m^3	Fine aggregates, kg/m^3
28	0.60	359	216	988	762
41	0.54	398	214	988	735
55	0.48	465	222	945	700

The maximum aggregate size of SCC was 10mm (0.4 in.), whereas the maximum aggregate size of NC was 19mm (0.75 in.). Class F fly ash and Type G high-range

water-reducing admixture were used in this study. The concrete was mixed and placed in the laboratory. The capacity of the mixer was approximately 0.3m^3 (10.6 ft^3), and each specimen was cast with one batch of concrete. The NC was consolidated based on suggestions by ACI 309.[14] Twelve ϕ100 × 200mm (ϕ4 × 8 in.) concrete cylinders were made at the time each column specimen was cast to monitor the strength development of the concrete. The specimens and the cylinders were covered with wet burlaps for the first three days and then cured under ambient temperature and humidity.

The properties of fresh concretes and the JSCE requirements for SCC are shown in Table 3.

Six electric dial gauges were installed in the test region of the specimen to measure the axial deformations of the column. Strain gauges were attached to four longitudinal steel bars and every tie leg at the mid-height of the column.

Linear variable differential transformers (LVDTs) were also mounted in the test region to monitor the lateral displacements.

The load was applied by a 6 000kN (1 348 kip) material testing system. The specimens were tested under monotonic loading. During each load step, the crack widths were measured using a portable stand microscope that contained a 25X magnifier and a scale chamber with minimum scale division of 0.05mm (0.002 in.). The applied load was controlled by displacement. The test setup is shown in Fig. 2.

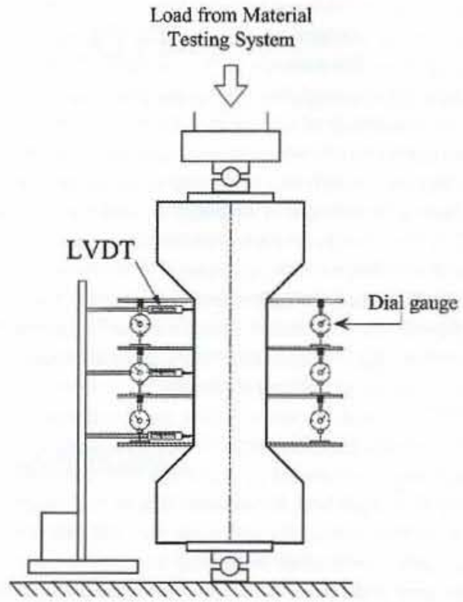


Fig. 2—Test setup.

TEST RESULTS

General behaviour

Figure 3 shows a typical axial load-axial deformation curve of a column specimen.

SCC columns used approximately the same amount of coarse aggregates as the NC columns and they behaved slightly stiffer in the ascending range than NC columns, as illustrated in Fig. 4. SCC columns exhibited smaller crack widths than the NC columns.

The crack widths of SCC columns in this study were even smaller than those of HWC columns[1] due to better flowability and larger amount of supplementary cementitious materials added in

the SCC. The maximum load occurred at an axial strain of approximately 0.00335 for NC columns on average, and 0.00308 for SCC columns on average, as shown in Table 4.

Table 2(b)—Mixture proportions for self-consolidating concrete.

Concrete strength, MPa	Water/ binder	Cement, kg/m^3	Fly ash, kg/m^3	Slag, kg/m^3	Silica fume, kg/m^3	Water, kg/m^3	Coarse aggregates, kg/m^3	Fine aggregates, kg/m^3	High-range water-reducing admixture, kg/m^3
28	0.51	188	120	25	5	168	925	919	3.0
41	0.45	226	110	29	10	167	920	897	3.2
55	0.40	264	110	27	20	165	920	853	4.8

Note: 1 kg = 2.2 lb; 1 m = 39.4 in.

Table 3—Properties of fresh concretes.

	Compressive strength, MPa	Slump, mm	Slump flow, mm	Time to reach 500 mm slump flow after 1 hour, seconds	U-box test, mm	V-funnel test, seconds
Normal concrete	29.3	190	300	—	—	—
	41.8	150	275	—	—	—
	55.2	120	245	—	—	—
Self-consolidating concrete	29.2	290	695	4	305	18
	42.1	275	685	4	310	15
	55.8	265	660	4	312	18
JSCE requirements for self-consolidating concrete	—	—	600 to 700	3 to 15	≥300	7 to 20

Note: 1 MPa = 0.145 ksi; 1 mm = 0.0394 in.

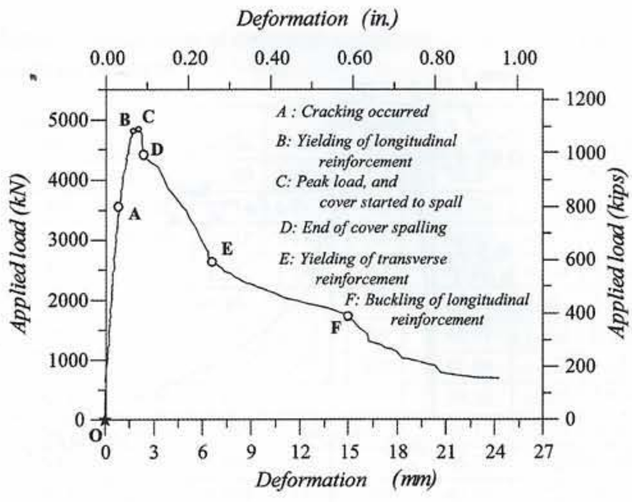


Fig. 3—Typical load-deformation curve.

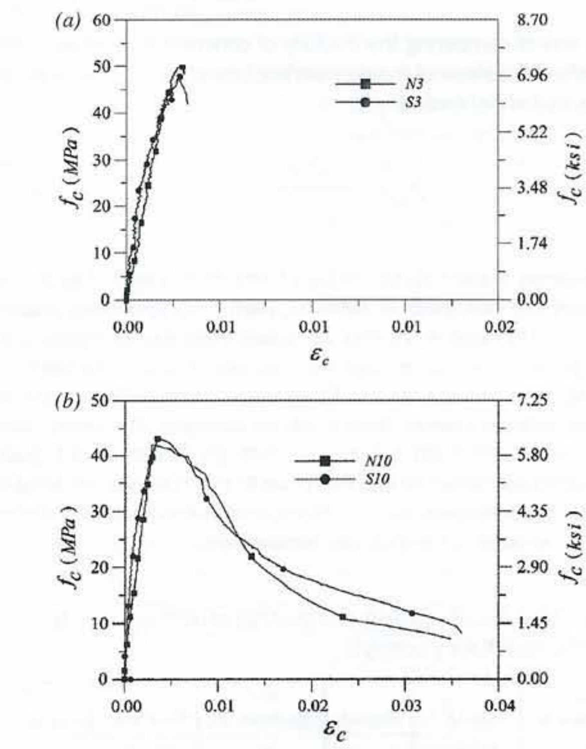


Fig. 4—Comparisons of stress-strain curves of SCC and NC.

The slightly smaller strain in SCC could be attributable to higher stiffness (due to a lower water-cementitious material ratio [w/cm] and higher density) and finer microcracking. In general, SCC columns exhibited better ductility than NC columns in the descending range, and the load dropped more gradually than NC columns after peak load. The better behaviour of SCC over NC could be attributed to better particle gradation, fewer voids, and a denser matrix structure.

Axial strength

Table 4 shows the maximum axial strengths of the specimens. The nominal strengths calculated by the ACI Code method are also listed in the table. All the experimental strengths were slightly greater than the nominal strengths. The average ratio of experimental

Table 4—Axial strains and strengths of specimens.

Specimen no.	Axial strain at maximum load	Maximum load P_{max}, kN	Nominal axial strength P_o, kN	Age at test, days
N1	0.00372	3758	3507	30
N2	0.00325	4724	4408	26
N3	0.00290	5768	5372	29
N4	0.00358	3741	3502	29
N5	0.00351	4938	4506	30
N6	0.00328	5572	5296	28
N7	0.00336	4536	4137	26
N8	0.00314	4795	4571	28
N9	0.00358	4551	4266	30
N10	0.00334	4814	4487	26
N11	0.00294	4798	4448	28
N12	0.00348	4809	4409	28
N13	0.00408	4713	4405	29
N14	0.00312	4569	4285	29
N15	0.00301	4853	4502	26
N16	0.00332	4841	4498	26
Average (SD): 0.00335 (0.0003074)				
S1	0.00317	3548	3349	28
S2	0.00247	4582	4237	30
S3	0.00240	5673	5191	28
S4	0.00342	3782	3437	26
S5	0.00304	4640	4315	30
S6	0.00278	5637	5160	28
S7	0.00311	4216	3842	30
S8	0.00301	4707	4477	26
S9	0.00302	4619	4260	29
S10	0.00290	4824	4398	30
S11	0.00269	4621	4303	28
S12	0.00367	4711	4355	29
S13	0.00387	4707	4336	28
S14	0.00298	4837	4445	30
S15	0.00305	4633	4317	29
S16	0.00366	4785	4368	30
Average (SD): 0.00308 (0.0004134)				

Note: SD = standard deviation; 1 MPa = 0.145 ksi; 1 kN = 0.225 kips.

strength P_{max} to nominal strength P_o was 1.07 for NC columns (with a standard deviation of 0.0131) and 1.08 for SCC columns (with standard deviation of 0.0141). This proved that the consolidation method used for NC was adequate. Both the SCC and NC columns reached their anticipated strengths in this study. Smaller waterbinder (or water-cement for NC) ratios were used in SCC and the strengths of SCC were quite close to those of NC, as shown in Table 1.

The average ratio of unconfined in-place strengths f'_{co} to cylinder strengths f'_c for Specimens N1 to N3 and S1 to S3 was found to be 89% for both NC and SCC columns. This is approximately the same as that in HWC columns. The ACI Code uses 0.85 and is slightly more conservative.

Crack width

Crack widths were observed before yielding of the longitudinal reinforcement. The columns did not exhibit apparent cracks until approximately 80% of the peak loads during the tests. The cracks usually formed in the longitudinal direction, and they coincided with the cover spalling from core. Table 5 lists the maximum crack width at 80% of the maximum load for each column. The values W_{NC} and W_{SCC} are the maximum crack width for NC columns and SCC columns, respectively. Each SCC column, except Specimen S6, had a smaller maximum crack width than its companion NC column. The average ratio of W_{SCC}/W_{NC} was 0.822. It indicates that SCC columns have better crack control ability than NC columns. The ratio was even smaller than that of HWC[1] (0.96).

Table 5—Crack widths of specimens.

Specimen no.	W_{NC}, mm	Specimen no.	W_{SCC}, mm	W_{SCC}/W_{NC}
N1	0.358	S1	0.269	0.751
N2	0.266	S2	0.218	0.820
N3	0.227	S3	0.200	0.881
N4	0.170	S4	0.160	0.941
N5	0.139	S5	0.110	0.791
N6	0.089	S6	0.092	1.034
N7	0.191	S7	0.117	0.613
N8	0.095	S8	0.074	0.779
N9	0.144	S9	0.102	0.708
N10	0.141	S10	0.111	0.787
N11	0.204	S11	0.179	0.877
N12	0.106	S12	0.086	0.811
N13	0.099	S13	0.090	0.909
N14	0.103	S14	0.099	0.961
N15	0.144	S15	0.106	0.736
N16	0.146	S16	0.111	0.760
Average (SD)				0.822 (0.10634)

Note: SD = standard deviation; 1 mm = 0.0394 in.

It is believed that adding supplementary cementitious materials such as fly ash, slag, and silica fume can improve the development of high-flowability concretes to improve the density of the concrete matrix and enhance the bond between the mortar matrix and coarse aggregates. The smaller crack widths of SCC could be because SCC had more supplementary cementious materials than HWC. The transition zone between aggregate and paste became stronger due to pore confinement that occurred after using supplementary cementitious materials.

Stress-strain curves for concrete

The stress-strain curves for different concretes were plotted in this study. The stress in the longitudinal steel was obtained by the measured strain and the stress-strain curve of the longitudinal steel. The force carried by concrete in the specimen was derived by subtracting the longitudinal steel force from the total applied load. The concrete stress was then obtained by dividing the concrete force by the concrete area. Before the peak load (Point C in Fig. 3) of the specimen, the concrete cover was included in the concrete area; and it was totally removed after Point D (Fig. 3). Between Points C and D, the concrete cover spalled gradually, and a linear transition was used. Figure 4 shows some examples of the comparison of the NC and SCC stress-strain curves. Generally, SCC exhibited higher stiffness before peak stress and slower descending rates (better ductility) after peak stress. The stiffness E_{test} is defined as the secant modulus of elasticity of concrete corresponding to $0.45f_c'$, as shown in Fig. 5.

The $E_{test}/\sqrt{f_c'}$ values for NC and SCC are listed in Table 6. The average stiffness of SCC in this study was 1.20 times that of NC.

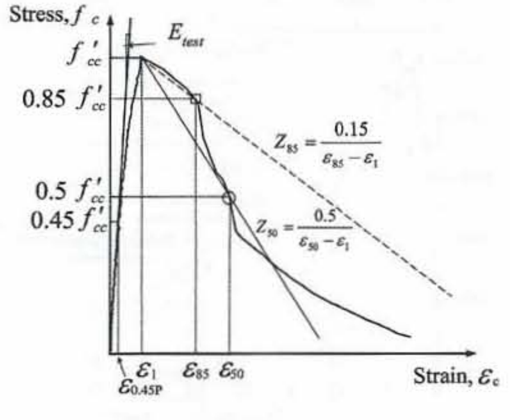


Fig. 5—Comparisons of Z_{85} and Z_{50}.

One way of comparing the ductility of concrete is using an index Z_{50} to reflect the slope of the descending branch of the stress-strain curve, and it is defined as:

$$Z_{50} = \frac{0.50}{\varepsilon_{50} - \varepsilon_1}$$

Definitions of the notation in Eq. (1) are illustrated in Fig. 5. The Z_{50} values are tabulated in Table 6. Each SCC specimen, except Specimen S11, had a smaller Z_{50} value than its companion NC specimen, and it indicates that the ductility of SCC is better than that of NC. The average ratio of $Z_{50,SCC}$ to $Z_{50,NC}$ was 0.782. A smaller Z_{50} value indicates better ductility of the concrete. The unconfined specimens (N1 through N3 and S1 through S3) exhibited much less ductility (as shown in Fig. 4(a)) due to much earlier buckling of longitudinal reinforcement; Z_{50} values were not available, and they were not included in the ductility comparison.

Table 6—Modulus of elasticity and ductility of normal concrete and self-consolidating concrete.

NC specimen no.	$E_{test}/\sqrt{f_c'}$	$Z_{50,NC}$	SCC specimen no.	$E_{test}/\sqrt{f_c'}$	$Z_{50,SCC}$
N1	3127.96	—	S1	3807.16	—
N2	3248.43	—	S2	3842.75	—
N3	3490.96	—	S3	4165.59	—
N4	4105.21	37.913	S4	4357.64	31.965
N5	2528.99	50.186	S5	4430.57	33.661
N6	3115.55	36.930	S6	3747.53	23.111
N7	3722.13	49.761	S7	4237.15	35.293
N8	3240.05	48.648	S8	4340.94	41.799
N9	3483.77	33.447	S9	3740.52	24.341
N10	2217.99	48.814	S10	4024.46	38.992
N11	3599.74	47.165	S11	3941.92	55.748
N12	4378.17	20.813	S12	3317.94	17.655
N13	3275.40	11.145	S13	2102.71	10.239
N14	3054.06	22.080	S14	4695.49	19.923
N15	3476.01	50.505	S15	4199.72	35.135
N16	3673.63	49.130	S16	3419.89	33.775

Note: 1 MPa = 0.145 ksi; — = not available.

Table 7—Comparison of maximum stress and ductility index μ of confined concrete.

Specimen no.	f'_{cc}, MPa	f'_{cc}/f'_{co}	μ_{NC}	Specimen no.	f'_{cc}, MPa	f'_{cc}/f'_{co}	μ_{SCC}
N4	31.21	1.140	5.085	S4	30.36	1.175	6.149
N5	43.81	1.134	4.977	S5	41.74	1.167	6.966
N6	52.59	1.051	7.174	S6	51.90	1.076	9.728
N7	43.72	1.132	4.707	S7	39.85	1.114	5.846
N8	40.28	1.043	5.176	S8	38.77	1.084	5.590
N9	41.69	1.079	6.138	S9	41.71	1.166	8.443
N10	43.13	1.117	5.372	S10	41.86	1.170	6.225
N11	40.80	1.056	4.772	S11	39.36	1.100	5.479
N12	44.78	1.160	8.265	S12	44.12	1.233	9.342
N13	45.94	1.190	7.596	S13	46.73	1.306	15.538
N14	40.72	1.054	8.387	S14	43.38	1.212	10.114
N15	43.52	1.127	4.619	S15	41.80	1.168	6.065
N16	42.94	1.112	4.760	S16	42.01	1.174	7.372

Note: 1 MPa = 0.145 ksi.

Another way of comparing the ductility of the confined concrete is using a ductility index μ. The ductility index μ is defined as A_u/A_p, where A_u is the area under the stress-strain curve before the stress drops to 50% of the maximum stress and A_p is the area under the stress-strain curve up to peak stress. The values of μ are shown in Table 7. Each SCC column had a larger μ value than its companion NC column.

The average of μ_{SCC}/μ_{NC} was 1.324. If A_u were defined as the area under the stress-strain curve before the stress drops to 25% of the maximum stress, the average of μ_{SCC}/μ_{NC} would be 1.208.

The effect of each variable on ductility index μ can be seen in Table 7. An increase of longitudinal reinforcement, increase of transverse reinforcement yield strength, and decrease of transverse spacing would improve the concrete ductility. NC specimens with Type C tie arrangement (Specimen N16) showed less ductility than that with Type B, although Type C had more transverse reinforcement than Type B. This could be attributed to the congestion problem in the NC specimen. On the contrary, the SCC specimen with Type C tie arrangement (Specimen S16) exhibited better ductility than the specimen with Type B (Specimen S5), and this was due to higher flowability of SCC and better concrete quality obtained in the specimen.

Specimens N5, N14, N15, S5, S14, and S15 had the same $\rho_s f_{yh}$ value (5.9 MPa [0.86 ksi]), but different fyh values (447, 339, and 560 MPa [64.8, 49.2, and 81.2 ksi]). It shows that specimens with smaller tie spacing (larger ρ_s)would have better ductility, although the values were the same. It also shows that the SCC specimens had better ductility than NC specimens. As for the effect of concrete strength, the values of μ and Z_{50} did not show a reasonable trend in this study as seen in Tables 6 and 7. Usually concrete with higher strengths exhibit less ductility, but specimens with higher strength had larger μ values (Specimens N6 and S6) and a smaller Z_{50} value (Specimen S6) in this study. The f'_{cc}/f'_{co} value, however, did decrease as the concrete strength increased, as shown in Table 7. The values for the confined specimens in Table 8 were taken from the unconfined column (Specimens N1, N2, N3, S1, S2, and S3) tests. The other test variables had similar effects on the ratio as on the ductility index μ as depicted in Table 7. An increase of longitudinal reinforcement, increase of transverse reinforcement yield strength, and decrease of transverse spacing would increase the f'_{cc}/f'_{co} ratio. The average f'_{cc}/f'_{co} ratio for NC was 1.107 and 1.165 for SCC.

The SCC had approximately the same amount of coarse aggregate as NC, but it contained more supplementary cementitious materials and less water content than NC, and it had a denser matrix structure and exhibited better performance than NC.

Comparisons with other high-flowability concretes

Comparisons between SCC used in this study and HWC,[1] and other SCC[6] were made. The results show that the overall behaviour of HWC was better than the SCC made in this study, whereas the SCC in this study was better than the other SCC.[6]

Comparisons with HWC

The HWC specimens had the same cross-section size and were tested in the same way as in this study. The maximum size of aggregate used in the HWC was 10 mm (0.4 in.), which is the same as that used in SCC in this study. The concrete strength and transverse reinforcement yield strength in HWC were approximately the same as those in SCC. The slump of HWC was 230 ± 20 mm (9.06 ± 0.79 in.), which was less than that of SCC (greater than 270 mm [10.6 in.]). But HWC contained more coarse aggregates (>1000 kg/m^3 [62.3 lb/ft^3]) than SCC, and it exhibited a better mechanical performance than SCC.

The stiffness E_{test} depends on the unit weight and strength of the concrete, as indicated in the ACI 318-05 Code.3 The unit weights of the high-flowability concretes and NC were approximately the same, as shown in Tables 8 and 9, and the $E_{test}/\sqrt{f'_c}$ values listed in Table 8 and 9 were used for comparisons.

Table 8—Modulus of elasticity and ductility of high-workability concrete[1] and self-consolidating concrete[6].

Specimen no.	$\rho_s f_{yh}$, MPa	w_c, kg/m^3	$E_{test}/\sqrt{f'_c}$	Z_{50}
		HWC[1]		
H4	5.93	2355	6271.42	18.137
H5	5.93	2351	5318.30	15.875
H6	5.93	2408	5680.45	15.185
H7	5.93	2351	5355.57	15.695
H8	5.93	2351	4836.83	15.652
H9	6.66	2351	6649.82	15.383
H10	4.91	2351	4462.21	15.311
H11	3.57	2351	5059.48	28.818
H12	8.91	2351	5365.65	9.206
H13	5.93	2351	5454.48	19.109
H14	6.06	2351	6467.23	11.712
H15	5.45	2351	4918.16	19.844
		SCC[6]		
SCC-28-Q-1	11.06	2334	3528.90	31.118
SCC-28-Q-2	11.06	2334	3145.42	35.413
SCC-28-Q-3	11.06	2334	2800.93	26.507
SCC-28-S-1	8.78	2334	3353.87	71.306
SCC-28-S-2	8.78	2334	3537.40	65.531
SCC-28-S-3	8.78	2334	3564.23	70.842
SCC-42-Q-1	11.06	2345	2672.70	34.693
SCC-42-Q-2	11.06	2345	2802.48	40.796
SCC-42-S-1	8.78	2345	2983.03	78.040
SCC-42-S-2	8.78	2345	3664.44	77.736

Note: 1 MPa = 0.145 ksi.

Table 9—Modulus of elasticity and ductility of companion normal concrete for high-workability concrete[1] and self-consolidating concrete[6].

Specimen no.	$\rho_s f_{yh}$, MPa	w_c, kg/m^3	$E_{test}/\sqrt{f_c'}$	Z_{50}
		HWC[1]		
N4	5.93	2268	5784.45	43.771
N5	5.93	2300	4142.64	40.548
N6	5.93	2347	7019.37	20.299
N7	5.93	2300	5181.61	40.644
N8	5.93	2300	2648.98	65.669
N9	6.66	2300	3245.08	32.060
N10	4.91	2300	2576.88	41.353
N11	3.57	2300	7154.99	62.893
N12	8.91	2300	3867.87	28.840
N13	5.93	2300	3161.90	59.673
N14	6.06	2300	3079.58	33.548
N15	5.45	2300	5369.11	73.569
		SCC[6]		
OPC-28-Q-1	11.06	2339	2577.02	28.920
OPC-28-Q-2	11.06	2339	2733.76	32.671
OPC-28-Q-3	11.06	2339	2864.62	30.912
OPC-28-S-1	8.78	2339	1866.32	51.937
OPC-28-S-2	8.78	2339	2134.85	61.516
OPC-28-S-3	8.78	2339	2965.13	69.252
OPC-42-Q-2	11.06	2347	2948.22	41.890
OPC-42-Q-3	11.06	2347	2343.01	31.399
OPC-42-S-1	8.78	2347	2650.14	74.538
OPC-42-S-2	8.78	2347	2619.21	76.593
OPC-42-S-3	8.78	2347	2849.55	67.889

Note: 1 MPa = 0.145 ksi.

The average stiffness of HWC was approximately 1.39 times that of its companion NC (with a standard deviation of 0.4878), whereas the average stiffness of SCC in this study was 1.19 times that of NC (standard deviation of 0.3470). The HWC exhibited higher stiffness than the SCC in this study.

Many studies have shown that transverse reinforcement is quite essential in confining the core concrete. Spacing, configuration, and strength of the transverse reinforcement all affect the confining effect. For simplicity, the product $\rho_s f_{yh}$ was used to evaluate the effect of transverse reinforcement on confinement, where ρ_s is the volumetric ratio of the transverse reinforcement and reflects spacing and configuration of the transverse reinforcement, and f_{yh} represents the yield strength of the transverse reinforcement. The $\rho_s f_{yh}$ values of HWC were close to those of SCC in this study, as shown in Table 8, but HWC had better ductility than SCC. The average ductility indicator Z_{50} of HWC was 0.40 times that of NC (standard deviation of 0.1316), whereas the average Z_{50} of SCC in this study was 0.81 times that of NC (standard deviation of 0.1477). It is apparent that the better ductility of HWC is not due to the transverse reinforcement. It has been pointed out that the column concrete tends to spall more gradually after the peak load (Point C in Fig. 3) if a larger amount of coarse aggregate is added to the concrete.1

The stress-strain curves of HWC and SCC in this study were normalized and plotted in the same figure (Fig. 6). In general, the HWC curves cover the curves of SCC in this study as shown in Fig. 6. The HWC contained less supplementary cementitious materials than the SCC in this study and it had better mechanical performance than SCC. It seems that the effect of adding supplementary cementitious materials in the concrete is not as prominent as that of coarse aggregate on the stress-strain behavior. The reason why HWC exhibited higher stiffness and better ductility could be attributed to its larger amount of coarse aggregate. The amount of coarse aggregate affects both the slope of the ascending branch of the stress-strain curve (stiffness) and the slope of the descending branch (ductility index Z_{50}).

Comparisons with other SCC

Studies on the behavior of confined SCC have also been carried out.[5,6] The SCC in Reference 5 had less stiffness and strength than NC, whereas the SCC in Reference 6 had less ductility than NC. The amount of coarse aggregate contained in the SCC in Reference 5 was approximately 830 kg/m^3 (51.7 lb/ft^3) of concrete for normal-strength concrete ($f_c' \leq 50$ MPa [7.25 ksi]). The amount of coarse aggregate contained in the SCC in Reference 6 was 790 kg/m^3 (49.2 lb/ft^3) of concrete. These were all less than the amount used in this study (920 kg/m^3 [57.3 lb/ft^3] of concrete).

The mixture proportions of SCC in Reference 6 are shown in Table 10.

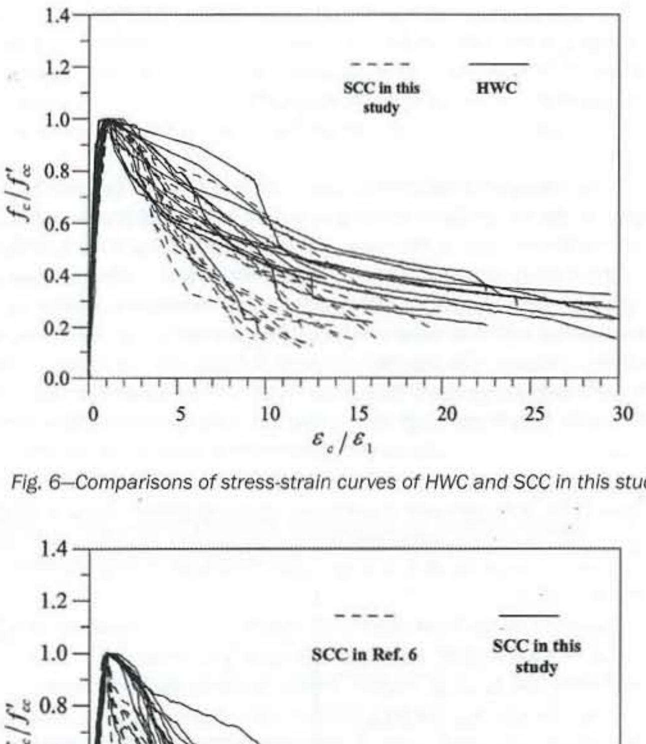


Fig. 6—Comparisons of stress-strain curves of HWC and SCC in this study.

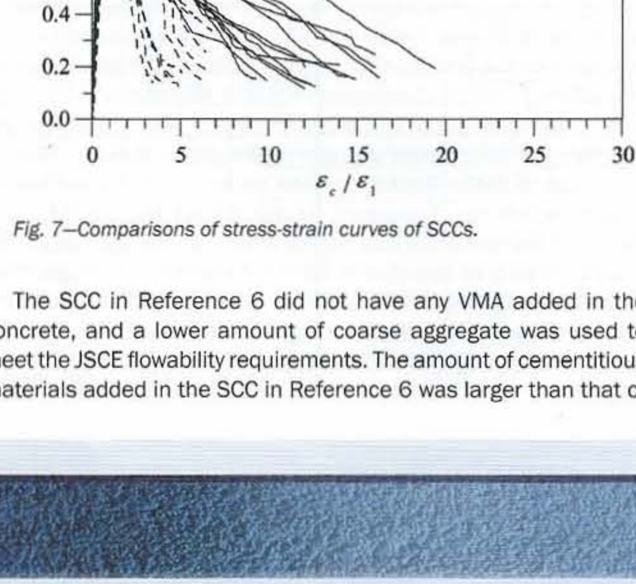


Fig. 7—Comparisons of stress-strain curves of SCCs.

The SCC in Reference 6 did not have any VMA added in the concrete, and a lower amount of coarse aggregate was used to meet the JSCE flowability requirements. The amount of cementitious materials added in the SCC in Reference 6 was larger than that of

Table 10–Mixture proportions of SCC[6].

Design strength, MPa	Cement, kg/m^3	Limestone, kg/m^3	Water, kg/m^3	Coarse aggregate, kg/m^3	Fine aggregate, kg/m^3	HRWRA, kg/m^3
28.0	310	230	170	790	830	4.4
42.0	350	190	170	790	840	4.6

Note: HRWRA = high-range water-reducing admixture; 1 kg = 2.2 lb; 1 m = 39.4 in.

SCC in this study. The $\rho_s f_{yh}$ values of the SCC specimens in Reference 6 were larger than those of the SCC specimens in this study, as shown in Table 8. The average Z_{50} value of SCC in Reference 6 was approximately 1.07 times that of its companion NC (standard deviation of 0.1680). The ductility of SCC used in Reference 6 was significantly less than that of SCC in this study even though it had a higher amount of transverse reinforcement. The normalized stress-strain curves of SCC in Reference 6 were compared with the SCC in this study, as illustrated in Fig. 7. The figure shows that the curves of the SCC in this study cover those of the SCC in Reference 6, and it indicates that the SCC in this study had higher stiffness and better ductility. Again, this could be attributed to the larger amount of coarse aggregate used in the SCC in this study.

CONCLUSIONS

Based on the experimental and analytical results presented herein, the following conclusions can be made:

1. The SCC in this study has higher stiffness than NC (with approximately 15% increase), but less than that of HWC[1] (39% increase). The ductility of confined SCC was found to be better than that of NC (with an increase of 32%) but is less than that of HWC (77% increase). The higher stiffness and better ductility of HWC could be attributed to the higher amount of coarse aggregate contained in the HWC (>1000 kg/m^3 [62.3 lb/ft^3] of concrete);
2. SCC columns showed smaller crack widths than NC columns in this study. The crack widths of SCC columns are approximately 82% of those of NC columns. The crack widths of SCC specimens are even smaller than those of HWC due to better flowability and larger amounts of supplementary cementitious materials added in the SCC;
3. A larger amount of coarse aggregates improves the mechanical behaviour of the hardened concrete. It is suggested that the amount of coarse aggregates in SCC should be kept approximately the same as that in NC (900 kg/m^3 [56.1 lb/ft^3] of concrete, could be a minimum). The SCC used in this study exhibited satisfactory structural performance.

NOTATION

A_g = gross area of column section
A_{st} = total area of longitudinal reinforcement
b_c = core dimension measured center-to-center of perimeter tie
E_{test} = modulus of elasticity of concrete corresponding to $0.45f'_{cc}$, as defined in Fig. 5
f'_c = concrete strength obtained from cylinder test
f'_{cc} = compressive strength of confined concrete in member
f'_{co} = compressive strength of unconfined concrete in member
f_l = average confinement pressure
f_s = stress in transverse reinforcement
f_y = yield strength of longitudinal reinforcement
f_{yh} = yield strength of transverse reinforcement
P_{max} = maximum column axial load
P_o = nominal column axial strength, = 0.85fc'(Ag – Ast) + fy Ast
s = spacing of transverse reinforcement
W = crack width
Z_{50} = slope of descending branch of concrete stress-strain curve calculated based on 0.5fcc'
Z_{85} = slope of descending branch of concrete stress-strain curve calculated based on 0.85fcc'
ε_1 = strain corresponding to peak stress of confined concrete
ε_{50} = strain corresponding to 50% peak stress of confined concrete
ε_c = strain in concrete
ρ_g = ratio of longitudinal reinforcement, Ast /Ag
ρ_s = ratio of volume of transverse reinforcement to volume of con crete core

REFERENCES

1. Lin, C. H.; Lin, S. P.; and Tseng, C. H., "High-Workability Concrete Columns under Concentric Compression," ACI Structural Journal, V. 101, No. 1, Jan.-Feb. 2004, pp. 85-93.

2. Lin, C. H., and Lin, S. P., "Flexural Behavior of High-Workability Concrete Columns under Cyclic Loading," ACI Structural Journal, V. 102, No. 3, May-June 2005, pp. 412-421.

3. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05)," American Concrete Institute, Farmington Hills, MI, 2005, 430 pp.

4. Chan, Y. W.; Chen, Y. S.; and Liu, Y. S., "Development of Bond Strength of Reinforcement Steel in Self-Consolidating Concrete," ACI Structural Journal, V. 100, No. 4, July-Aug. 2003, pp. 490-498.

5. Paultre, P.; Khayat, K. H.; Cusson, D.; and Tremblay, S., "Structural Performance of Self-Consolidating Concrete Used in Confined Concrete Columns," ACI Structural Journal, V. 102, No. 4, July-Aug. 2005, pp. 560-568.

6. Chen, Y. F., "An Investigation on Confinement Behavior of Square Self-Compacting Concrete Columns," MS thesis, Department of Civil Engineering, National Taiwan University, Taiwan, 2003, 245 pp. (in Chinese)

7. ACI Committee 209, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures," Designing for Creep and Shrinkage in Concrete Structures, SP-76, American Concrete Institute, Farmington Hills, MI, 1982, pp. 93-300.

8. Hwang, C. L., and Chen, Y. Y., "The Property of Self-Consolidating Concrete Designed by Densified Mixture Design Algorithm," First North American Conference on the Design and Use of Self-Consolidating Concrete, 2002, pp. 121-126.

9. Hwang, C. L., and Hung, M. F., "Durability Consideration of Self-Consolidating Concrete," First North American Conference on the Design and Use of Self-Consolidating Concrete, 2002, pp. 385-390.

10. Tsai, C. T.; Li, L. S.; and Hwang, C. L., "The Effect of Aggregate Gradation on Engineering Properties of High Performance Concrete," Journal of ASTM International, V. 3, No. 3, Mar. 2006, 12 pp.

11. Hwang, C. L., and Hung, M. F., "Durability Design and Performance of Self-Consolidating Lightweight Concrete," Construction and Building Materials, 1995, pp. 619-626.

12. Mehta, P. K., and Monteiro, P. J. M., Concrete, McGraw-Hill, 2006, 659 pp.

13. Uomoto, T., and Ozawa, K., "Recommendation for Self-Compacting Concrete," Japan Society of Civil Engineering, Aug. 1999, 77 pp.

14. ACI Committee 309, "Guide for Consolidation of Concrete (ACI 309R-96)," American Concrete Institute, Farmington Hills, MI, 1996, 39 pp.