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Strength and ductility of confined high strength concrete

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ABSTRACT: The results of an experimental investigation on the structural behavior of high strength concrete, confined by rectilinear steel ties and subjected to generalized compression loads, are presented. Analytic relationships were derived to predict the experimentally observed entire stress-strain curves for confined high strength concrete under compression. A computer program was developed to calculate the moment-curvature relationships of beams and columns up to their ultimate strength. From the theoretical moment-curvature curves it was shown that properly detailed sections made from confined high strength concrete exhibit ductile response.

1 INTRODUCTION

High strength concrete of more than 60 MPa compressive strength can be produced using already available component materials and conventional methods of mixing, placing and curing.

Although it is generally accepted that confined concrete shows increased ductility and strength, there is a difference of opinion on both the enhancement of ductility and strength in confined high strength concrete.

The basic philosophy for the use of confined concrete in seismically resistant designed structures [2], requires that confinement is capable of increasing the capacity of the concrete structures to sustain large deformations in the post elastic range without a substantial strength loss.

The aim of this study was to examine the influence of determined degrees of confinement on the behavior of high strength concrete specimens confined by rectilinear steel ties and subjected to generalized seismic simulating compression loads.

Analytic relationships were derived to represent the experimentally observed entire stress-strain curves for confined high strength concrete core in compression.

A computer program was developed, based on the analytic relationships, for the examinations of the influence of the degree of confinement on the moment curvature relationships of beams and columns.

2 EXPERIMENTAL PROGRAM

2.1 *Test specimens*

Tests were performed on 40 prismatic high strength concrete specimens (150×150×450 mm) with three steel tie spacings (s = 25, 50, 75 mm, series A, B, C), fig.1. The test specimens included 4 plain unreinforced prismatic specimens.

MIXTURE PROPORTIONS

Portland Cement	450 kg/m^3
Aggregates	1830 kg/m^3
Fly Ash	30 kg/m^3
Silica Fume	20 kg/m^3
Water	165 kg/m^3
Superplasticizer	3 %

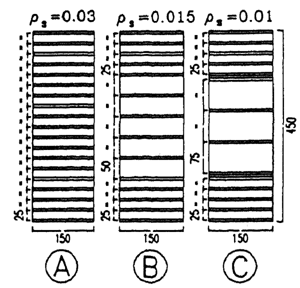


Fig.1 Mixture proportions and details of test specimens

2.2 *Concrete*

The high strength concrete mixture was designed in order to obtain a specific compressive control cylinder strength at 28 days of age ($f_c \cong 65$ MPa). Portland cement, type 525 (comparable to ASTM type III), river sand, gravel, silica fume and fly ash (Class F) were used. The maximum size of the coarse aggregate was 20 mm. Mixture proportions are shown in fig.1.

2.3 *Reinforcement*

The reinforced specimens contained only rectilinear lateral steel ties; no longitudinal reinforcement was used in the specimens.

Deformed reinforcing bars (diameter = 6 mm) with yield strength (f_y) of about 440 MPa (type Feb44k) were used. To prevent premature failure, all steel ties were welded. The outer dimension of the ties was equal to the inside dimensions of the specimen forms, as a result no cover was provided.

2.4 *Testing machine and instrumentation*

Specimens were tested in a closed-loop servo controlled hydraulic testing machine. Displacements were measured by four LVDT over a central 175 mm gage length on each vertical face of the specimens.

Strains in the steel tie were measured using two strain gauges centered at mid height of the specimen and at diametrically opposite sides of the tie.

2.5 *Test procedure*

Three loading histories, involving monotonic, gradually increasing strain cyclic and constant strain cyclic were used during the compressive tests.

3 ANALYSIS OF RESULTS

3.1 *Compressive strength*

Typical compressive stress-strain curves (σ-ε) for the tested specimens are shown in fig.2a,b,c.

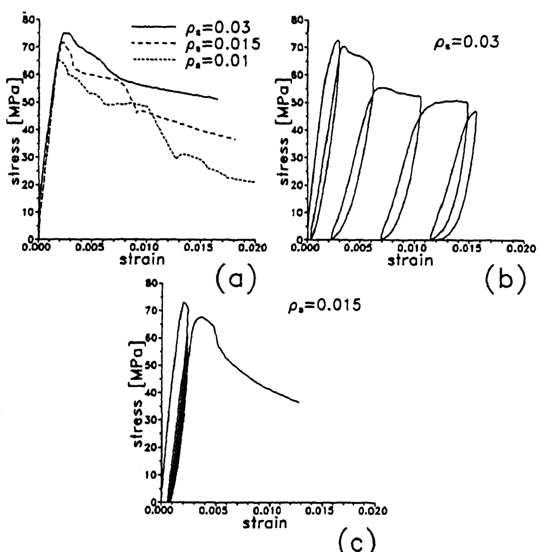


Fig.2 Typical stress-strain curves

The mean compressive strength (f''_c) of the unreinforced specimens was in the order of 60 MPa. The strength ratio of unreinforced specimens to control cylinder strength (f''_c/f_c) was 0.88.

The mean compressive strength (f'_c) for the three series of specimens (A,B,C) was 75.1, 72.5 and 67.7 MPa respectively. These increases in strength are comparable with other published data [4] for high strength confined concrete. Similar values of confinement degree show that the increase in strength of high strength concrete is lower than for lower strength concrete.

The increase in compressive strength of confined concrete was evaluated on the basis of the Nilson et al., Park et al. and Sheikh-Uzumeri equations [4], [5], [6], [7]. The tests' results were used to calibrate the parameters for the above equations.

The criterion for the calibration of the parameters was the minimization of the cumulative absolute error $\Sigma_1^n \left|(f'_{cexp,i} - f'_{cth,i})\right|/f'_{cexp,i}$ for all the specimens. The final equations can therefore be written as follows:

$$f'_c = f''_c + 3.25 f'_2 (1 - s/d_c) \qquad (1)$$

where $f'_2 = 2A_s f_y / d_c s$, A_s = area of the steel tie cross section, d_c = outside diameter of tie;

$$f'_c = f_c(1 + 0.52\rho_s f_y / f_c) \qquad (2)$$

where ρ_s = ratio of volume of steel tie to volume of concrete core;

$$f'_c = f''_c\left[1 + \frac{1}{40P_{occ}}\left(1 - \frac{nC^2}{8B^2}\right)\left(1 - \frac{0.5s}{B}\right)^2 B^2(\rho_s f_y)^{0.13}\right] \qquad (3)$$

where $P_{occ} = f''_c A_c$ in kilonewtons, A_c = area of concrete core, n = number of laterally supported longitudinal bars, B = center to center distance of perimeter tie, C = distance between the laterally supported longitudinal bars. Fig.3 shows the comparison of experimental and the theoretical f'_c values calculated for all the specimens with equations (1), (2) and (3).

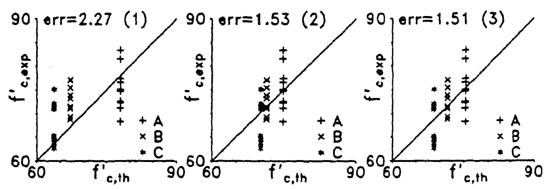


Fig.3 Comparison of experimental and theoretical f'_c values calculated with eq. (1), (2), and (3).

3.2 *Axial strain at maximum stress*

The peak strain (ε_0) showed an increase as the volumetric ratio of the steel ties increased. The following equation [6] is suggested to calculate this strain value

$$\varepsilon_0 = 30 k f_c 10^{-6} \qquad (4)$$

where f_c in MPa, $k = f'_c/f''_c$ from eq. (3).

3.3 *Modulus of elasticity*

It was found that the lateral reinforcement of specimens does not influence the modulus of elasticity (E_c).

The secant moduli of elasticity calculated at $0.45f'_c$ were found to be almost the same as those calculated for unreinforced specimens. The mean value of E_c of all the specimens was 40590 MPa.

3.4 *Plasticity ratio*

The plasticity ratio (β), defined as the ratio of axial strain (ε_{85}), corresponding to $0.85f'_c$ on the descending branch of the σ-ε curve of the reinforced specimen, to the axial strain ε_0, was calculated. The mean values for specimens of series A, B, C were β = 2.42, 2.25, 2.18 respectively. These results indicate that β increases approximately linearly with increase in ρ_s. Similar values of confinement degree show that the rate of increase is less for high strength concrete than for lower strength concrete.

3.5 *Stress in steel ties*

The steel tie stress did not reach yield at maximum axial load in any specimen during testing. In general the steel tie stress increased almost linearly up to a strain of about 0.006 on the descending branch of the compressive σ-ε curve of the reinforced specimens.

4 ANALYTICAL PROGRAM

4.1 *Theoretical model for confined concrete*

On the basis of the experimental results, the optimum parameters for an analytical model for the prediction of the hysteretic compressive σ-ε curve of confined high strength concrete core were calibrated through an identification technique.

The model used in this study was a modification of the Tanigawa model [3]:

$$\sigma = f'_c\frac{\varepsilon}{\varepsilon_0}\left[\frac{D}{D-1+(\varepsilon/\varepsilon_0)^D}\right] \tag{5}$$

where $D = D_1$, D_2 in the ascending and descending branch of the envelope curve.

The equations for the parameters D_1, D_2 and the hysteresis rules can be written as follows:

$$D_1 = 8.5 - 80\rho_s\sqrt{B/s} \tag{6}$$

$$D_2 = 1.13/(\rho_s\sqrt{B/s})^{0.1} \tag{7}$$

$$\varepsilon_r/\varepsilon_0=\varepsilon_u/\varepsilon_0\left[1-e^{((-\varepsilon_u/\varepsilon_0)/4.5)}\right]\left[1-(\varepsilon_u/\varepsilon_0+1)\right]^{(-\varepsilon_u/\varepsilon_0)} \tag{8}$$

where ε_r = residual strain at σ = 0, ε_u = strain on the envelope curve;

$$E_u/E_c = 1.41/(\varepsilon_u/\varepsilon_0)^{0.6} \tag{9}$$

where E_u = tangent modulus of unloading curve at the common point;

$$\sigma_{cp}/\sigma_u = 0.86 = \text{constant}$$

where σ_{cp} = stress at the common point obtained by reducing the corresponding stress σ_u on the envelope curve to a scale of 0.86;

$$\sigma_n/\sigma_u = 1-0.113(\varepsilon_u/\varepsilon_0)\ln(n)$$

where σ_n gives the stress reduction due to constant cyclic loading.

The experimental and best theoretical curves σ-ε are plotted in fig.4a,b,c and demonstrate that the model agrees well with the experimental curves.

A similar trend is observed in fig.4d where areas under experimental curves (monotonic and cyclic) are compared with areas calculated with the proposed model.

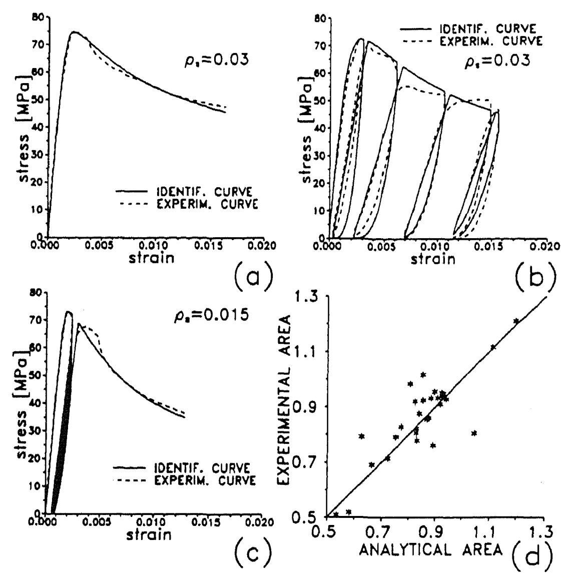


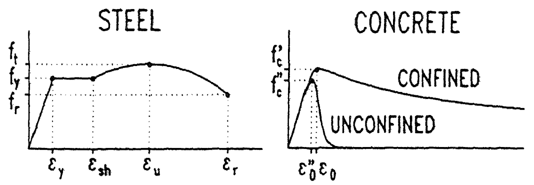
Fig.4 Experimental and best theoretical σ-ε curves (a,b,c). Comparison between areas under exp. σ-ε curves and the corresponding analytical areas (d).

4.2 *Non linear analysis of reinforced concrete sections*

A computer program was written to calculate the moment curvature relationships for reinforced concrete beams and uniaxially eccentrically loaded columns. The program was based on the model proposed for confined concrete and the model for steel [8] as described in fig.5.

The cover concrete was assumed to be effective up to a concrete compressive strain of 0.004 and then to be lost at higher strains. No buckling of longitudinal steel was considered.

Moment curvature relationships were computed for reinforced concrete beams, rectangular in cross section, reinforced with six amounts of longitudinal tensile reinforcement (ρ_l) and two amounts of confining reinforcement (ρ_s).



$\sigma = E_s \varepsilon$ for $0 < \varepsilon < \varepsilon_y$

$\sigma = f_y$ for $\varepsilon_y < \varepsilon < \varepsilon_{sh}$

$Y = \dfrac{AX + BX^2}{1 + CX + DX^2}$ for $\varepsilon > \varepsilon_{sh}$

$X = (\varepsilon - \varepsilon_{sh})/(\varepsilon_u - \varepsilon_{sh})$

$Y = (\sigma - f_y)/(f_t - f_y)$

$f_y = 455\text{MPa}$ $\varepsilon_{sh} = 0.0158$

$f_t = 670\text{MPa}$ $\varepsilon_u = 0.13$

$A=2,\ B=-0.98,\ C=-0.13,\ D=0.07$

$\left.\begin{array}{l} f''_c = 60\text{MPa} \\ \varepsilon''_0 = 0.0022 \end{array}\right\}$ UNCONFINED

$\left.\begin{array}{l} f'_c = 75.1 \div 67.7\text{MPa} \\ \varepsilon_0 = 0.0026 \div 0.0023 \end{array}\right\}$ CONFINED

$D_1 = 2.74 \div 7.40$

$D_2 = 1.47 \div 1.73$

Fig.5 Analytical stress-strain curves for steel and concrete

The ductility factors (ϕ_u/ϕ_y) were calculated at two values of extreme fiber concrete compression strain: at a strain of 0.005 and at a strain of 0.01. It can be seen from fig.6a,b that confined high strength concrete leads to an increase of ductility and that the amount of confinement has a relevant effect on ϕ_u/ϕ_y for low values of ρ_l and high values of extreme fiber compression strain. The same trend can be observed in fig.6c,d where ϕ_u/ϕ_y values were calculated for columns loaded with different values of axial load ratios N/N_u, in which N_u is the ultimate load of the concentrically loaded column.

The load-moment interaction curves were calculated according to the CEB MC [1] assumptions (but without including under strength factors) and for comparison, in accordance with the proposed model for the confined concrete and for a value of extreme fiber concrete compression strain $\varepsilon_{cu} = 0.005$.

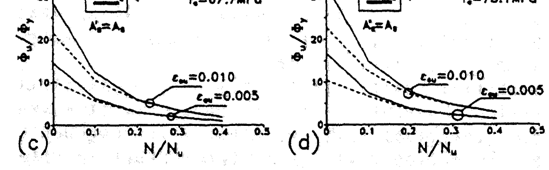


Fig.6 Comparison of ductility factors for beams and columns.

Fig.7a,b show that for axial loads higher than that corresponding to the balanced condition and for a low level of confining reinforcement, the CEB MC can be unconservative.

In addition fig.7c shows the enhancement (M/M_f) in flexural capacity of the column section. M_f is the flexural capacity for N=0.

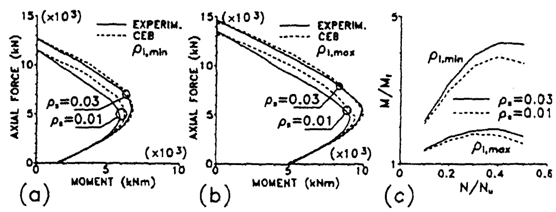


Fig.7 Comparison of load-moment interaction curves and effect of axial load on moment enhancement ratio.

5 CONCLUSIONS

The following conclusions can be drawn from this study:

- The compressive strength, strain at maximum stress and ductility of high strength concrete core specimens confined by rectilinear steel ties, all increased as the amount of confinement degree increased. However, the rate of increase was lower than for normal strength concrete with similar confinement degrees.
- As a result an analytic model to predict the compressive stress-strain curves (monotonic and cyclic) of confined high strength concrete core was proposed. It was found that the analytic model agreed accurately with the experimental σ-ε curves.
- Based on the previous model, the ductility factors, ultimate moments and loads of beams and columns were calculated using varying amounts of longitudinal and confining reinforcement. It was found that properly reinforced sections made of high strength concrete exhibit ductile structural response, and that the amount of confinement has a great effect on large strains or curvatures. In comparing the load-moment interaction curve as provided by the CEB MC and that based on experimentally observed stress-strain curves for confined high strength concrete, it can be seen that for axial loads higher than those corresponding to the balanced condition, the CEB MC can be unconservative.

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