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# Effect of cover upon the stress–strain properties of concrete confined in steel binders

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*SYNOPSIS*

*Cover to the stirrups in reinforced concrete structures is one of the important parameters in the theory of confined concrete. On the basis of experimental results, a new term 'effective confinement index' is introduced. The expressions for increase in maximum stress and the corresponding strain in terms of effective confinement index are given. Theory is developed to obtain the stress–strain curve of total concrete with confined core and unconfined cover. The experimental and analytical stress–strain curves are compared. Also the maximum stresses and the corresponding strains of shell concrete are compared with the results obtained by the theory of Sargin et al.*

## Notation

| | |
|---|---|
| $A$ | = constant in stress–strain curve, $A = 5481{\cdot}2\sqrt{f_c'}$ |
| $A_s$ | = area of shell (= unconfined) concrete |
| $A_c$ | = area of core (= confined) concrete |
| $A_t$ | = area of total concrete |
| $B$ | = constant in stress–strain curve |
| $b$ | = breadth of the section |
| $C$ | = constant in stress–strain curve |
| $C_i$ | = confinement index = $(p_b - \bar{p}_b)\dfrac{f_y}{f_c'}\sqrt{b/s}$ |
| $C_i\left(\dfrac{A_c}{A_t}\right)$ | = effective confinement index |
| $E_c$ | = initial tangential modulus of concrete |
| $f$ | = stress |
| $f_c$ | = stress in core concrete |
| $f_c'$ | = maximum strength of plain concrete cylinder or prism |
| $f_c$ | = maximum strength of confined concrete prism |
| $f_s$ | = stress in shell concrete |
| $f_t$ | = stress in total concrete |
| $f_t$ | = maximum stress in total concrete |
| $f_y$ | = yield stress of binder steel |
| $f_v$ | = stress in binder steel |
| $p_b$ | = volumetric ratio, i.e. ratio of the volume of the binder to the volume of the confined concrete, based on the mean lateral dimensions of the binder |
| $\bar{p}_b$ | = same as $p_b$, when the pitch of the binder is equal to 1½ times the least lateral dimensions of the specimen |
| $s$ | = spacing (pitch) of the stirrups |
| $\epsilon$ | = strain |
| $\epsilon_c$ | = strain in concrete |
| $\epsilon_c$ | = strain at maximum stress of plain concrete prism |
| $\epsilon_s$ | = strain in shell concrete |
| $\epsilon_v$ | = strain in binder steel |

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## Introduction

The effect of confinement upon concrete depends upon many factors such as (1) spacing, diameter, strength and shape of binders, (2) strength of concrete, (3) size of concrete core confined and (4) thickness of concrete cover. Though the effect of the first three parameters upon confined concrete has been studied extensively, there has been very little work on the effectiveness of cover, and there are no definite quantitative conclusions. Cover is invariably provided in reinforced concrete structures to protect the reinforcement from corrosion, weathering and fire. Cover is also required to provide the necessary bond between the reinforcement and the concrete in reinforced concrete structures.

Laboratory tests were conducted on 144 prisms to study the effect of cover upon the increase in strength and deformation of confined concrete. Prisms of two sizes, 150 × 150 × 300 and 100 × 100 × 200 mm, were tested to study the effect of size. On the basis of the experimental results, expressions are proposed using an effective confinement index $C_i(A_c/A_t)$ to predict the maximum stresses and the corresponding strains for concrete confined in rectangular ties. The maximum stresses and the corresponding strains of the shell concrete were calculated by using the confined concrete theory. These shell concrete stresses and strains are compared with those obtained by the trial-and-error procedure of Sargin et al.[1] and are found to be within ±10% for the stresses and ±20% for the strains.

## Review of the literature

Many investigators have studied the effect of confinement, the main variables being (1) spacing, diameter, strength and shape of binders, (2) strength of concrete and (3) size and shape of concrete core confined. These studies are due to King[2], Chan[3], Szylcznyski and Sozen[4], Roy and Sozen[5], Bertero and Fellippa[6], Stöckl[7], Nagi Reddy[8], Shah and Vijayarangan[9], Somes[10], Burdette and Hilsdorf[11], Kent and Park[12] and S. R. Reddy[13]. The mechanics of tie confinement with the above variables is well understood and there has been agreement among various investigators regarding increase in strength and ductility.

Cover is required to protect the reinforcement from corrosion, weathering and fire, and also to provide the necessary bond between reinforcement and the surrounding concrete. Building codes recommend the thickness of cover, depending upon the size of the aggregate and the diameter of the reinforcing bar used. A few investigators, Soliman and Yu[14], Sargin[15] and Sargin et al.[1], have studied the effect of cover upon confined concrete along with other variables. Sargin et al.[1] have given a trial-and-error approach to separate the shell concrete stresses from the core concrete stresses making use of the experimental stress–strain curve of the composite concrete and the companion plain concrete specimens. It is a tedious procedure to obtain cover and core stresses and the corresponding strains from a single equilibrium equation

$$f_c A_c + f_s A_s = f_t A_t \quad \ldots\ldots\ldots\ldots(1)$$

where $f_c$, $f_s$ and $f_t$ are the stresses in core concrete, shell concrete and total concrete and $A_c$, $A_s$ and $A_t$ are the corresponding cross-sectional areas. They have also assumed that the stresses in core and shell concrete are the same when the shell reaches the maximum strain, which is usually between 0·10 and 0·15%.

## Experimental programme

Laboratory tests have been conducted on 144 prisms in eight series (A to H) and each series consists of 18 prisms (see Table 1). Series A to D are the bigger prisms (15; × 150 × 300 mm) and E to H are the smaller prisms (100 × 100 × 200 mm). The thickness of the cover concrete is the only variable from series to series. In a series, there are six groups of specimens and each group consists of three identical prisms. From group to group, the only variable is the spacing of the stirrups. One of the six groups was of plain concrete prisms; the other five groups were of confined concrete prisms with five stirrups spacings.

## Materials used for the preparation of specimens

The concrete mix was prepared with ordinary Portland cement conforming to IS 269. River sand with a fineness modulus of 2·60 obtained from a local source was used as fine aggregate. Crushed granite stone of 12 mm maximum size with a fineness modulus of 5·85 obtained from a local crusher was used as coarse aggregate. Throughout the investigation, 1:2:4 concrete mix by weight, with a 0·6 water/cement ratio was used.

For stirrups, 7 mm mild-steel wire was used in

TABLE 1: **Details of prisms cast and tested.**

| Series | Size (mm) | Spacing of stirrups (mm) | Clear cover (mm) | Diameter of stirrups (mm) | $f_y$ (N/mm²) | $f_c$ (N/mm²) |
|---|---|---|---|---|---|---|
| A | 150 × 150 × 300 | 40, 70, 100, 150 225 and plain | 10 | 7 mm mild-steel wire | 368 | 16·9 |
| B | | | 15 | | | 16·0 |
| C | | | 20 | | | 17·9 |
| D | | | 25 | | | 12·0 |
| E | 100 × 100 × 200 | 40, 60, 80, 100 150 and plain | 5 | | | 20·4 |
| F | | | 10 | 4 mm galvanized wire | 417 | 19·1 |
| G | | | 15 | | | 17·1 |
| H | | | 20 | | | 16·3 |

Each series consisted of six groups of three identical specimens. The only factor varied between the six groups was the spacing of the stirrups (as in the third column above).

series A to E and 4 mm galvanized iron wire was used in series F to H. The binders were prepared by a skilled bar bender. Since there was no longitudinal reinforcement in the prism, the ties were placed on two horizontal bars on which the spacing was marked, and the corners of the ties were tied together, to form a cage, by stiff binding wire. The cage was removed from the horizontal bars and placed in the moulds.

To measure strains, a compressometer as shown in Figure 1 was used. The screws of the compressometer may become loosened when the cover concrete spalls off. This was prevented by fixing the screws of the compressometer firmly on 20 × 20 × 3 mm mild-steel plates, which in turn were held in position by welding the plates to 3 mm diameter galvanized iron wire as shown in Figure 1. The cross struts were tied in the cage at a gauge length of 150 mm for the bigger specimens and 100 mm for the smaller prisms, symmetrically about mid-height. The struts were used to fix the compressometer firmly to the specimen, and to keep them in position even after the cover concrete spalled from the specimen.

All the prisms were cast horizontally. To maintain the specified cover, precast mortar blocks of size 15 × 15 mm with the required thickness were placed between the stirrups and the mould. The cast specimens were removed from the moulds after about 48 h and placed in a curing tank for 28 days. Afterwards they were air-dried in the laboratory till the time of testing.

Specimens were tested on a 100 tonne hydraulically operated universal testing machine. The compressometer was attached to the specimen so that the screws of the compressometer had a firm grip on the mild-steel plates of the cross struts. The specimen was gradually loaded at a strain rate of 0·001 per minute by adjusting the inlet valve of the machine.

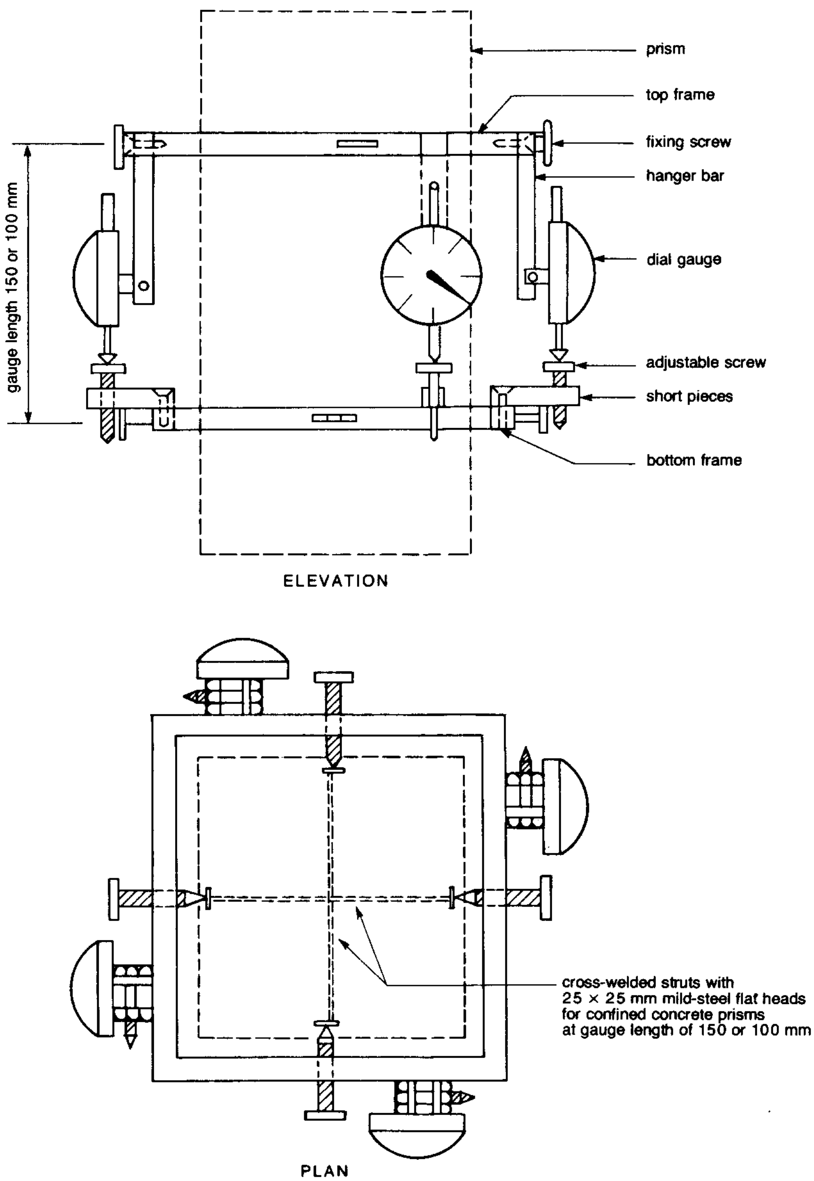



*Figure 1: Details of compressometer.*

The readings of the four dial gauges of the compressometer and the corresponding load were recorded at intervals of half a minute.

## Behaviour of test specimens under load

The load increased rapidly in the initial stages up to about 80% of the ultimate load; beyond 80%, the rate of increase of load was found to be slow. Tests were continued until the load had dropped by 20 to 25% of the ultimate load. Spalling of concrete was noticed before the ultimate load was reached, but it was severe beyond the ultimate stage.

For specimens with thin covers and with binders at small spacings, the shell concrete cover spalls before the ultimate load is reached. This is due to bulging of the binders which push the shell concrete outwards. In specimens with thick cover and with the binders at large spacings, the shell concrete behaved compositely with the core concrete. In these specimens the core concrete in between the binders fails before the leg of the binder steel bulges out. In general, with increase in spacing in a given series, the ultimate load decreases and this indicates a relationship between ultimate load and stirrup spacing. A similar observation is made with reference to strains also.

## Experimental stress–strain curve

From the measurements recorded for each specimen, the axial stresses and strains were calculated by using over-all dimensions of the specimen. Using these values, points of stress–strain curves for the three companion specimens of a set were plotted with the same origin; the mean curve was taken to represent the set as shown in Figure 2. Such mean curves were drawn for each set. From these curves, the maximum stress $\bar{f}_t$ of the total concrete and the maximum stress $f'_c$ of the companion plain concrete were obtained and the ratios $\bar{f}_t/f'_c$ were calculated. Similarly, the ratios of strains $\bar{\epsilon}_t/\epsilon'_c$ were calculated and the results are given in Table 2.

TABLE 2: **Effective confinement index, increase in maximum stress and the corresponding strain.**

| Series | Group | $C_i$ | Core area ratio* | $C_i\left(\frac{A_c}{A_t}\right)$ | $\bar{f}_t$ (N/mm²) | $f'_c$ (N/mm²) | $\frac{\bar{f}_t}{f'_c}$ | $\bar{\epsilon}_t$ (%) | $\epsilon'_c$ (%) | $\frac{\bar{\epsilon}_t}{\epsilon'_c}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| A | A4 | 0·835 | 0·751 | 0·627 | 22·7 | 16·9 | 1·34 | 0·74 | 0·15 | 4·93 |
| | A7 | 0·279 | | 0·209 | 18·5 | | 1·09 | 0·32 | | 2·13 |
| | A10 | 0·119 | | 0·089 | 17·4 | | 1·03 | 0·22 | | 1·47 |
| | A15 | 0·025 | | 0·018 | 16·9 | | 1·00 | 0·17 | | 1·13 |
| B | B4 | 0·892 | 0·640 | 0·571 | 19·3 | 16·0 | 1·21 | 0·52 | 0·16 | 3·25 |
| | B7 | 0·382 | | 0·244 | 17·6 | | 1·10 | 0·45 | | 2·81 |
| | B10 | 0·119 | | 0·076 | 16·3 | | 1·02 | 0·26 | | 1·63 |
| | B15 | 0·040 | | 0·025 | 16·0 | | 1·00 | 0·20 | | 1·25 |
| C | C4 | 0·814 | 0·538 | 0·438 | 20·6 | 17·9 | 1·15 | 0·46 | 0·14 | 3·29 |
| | C7 | 0·321 | | 0·173 | 19·3 | | 1·08 | 0·32 | | 2·29 |
| | C10 | 0·096 | | 0·052 | 17·9 | | 1·00 | 0·18 | | 1·29 |
| | C15 | 0·004 | | 0·002 | 17·6 | | 0·98 | 0·16 | | 1·14 |
| D | D4 | 0·234 | 0·444 | 0·548 | 14·5 | 12·0 | 1·21 | 0·48 | 0·12 | 4·00 |
| | D7 | 0·366 | | 0·163 | 13·0 | | 1·08 | 0·28 | | 2·33 |
| | D10 | 0·121 | | 0·054 | 12·4 | | 1·03 | 0·20 | | 1·67 |
| E | E4 | 0·987 | 0·810 | 0·799 | 29·4 | 20·4 | 1·44 | 1·14 | 0·20 | 5·70 |
| | E6 | 0·300 | | 0·243 | 22·7 | | 1·11 | 0·48 | | 2·40 |
| | E8 | 0·134 | | 0·109 | 21·1 | | 1·03 | 0·30 | | 1·50 |
| | E10 | 0·052 | | 0·042 | 20·5 | | 1·00 | 0·22 | | 1·10 |
| F | F4 | 0·251 | 0·640 | 0·161 | 20·1 | 19·1 | 1·05 | 0·32 | 0·16 | 2·00 |
| | F6 | 0·099 | | 0·063 | 19·7 | | 1·03 | 0·20 | | 1·25 |
| | F8 | 0·040 | | 0·026 | 19·2 | | 1·01 | 0·18 | | 1·13 |
| | F10 | 0·012 | | 0·008 | 18·9 | | 0·99 | 0·16 | | 1·00 |
| G | G4 | 0·279 | 0·490 | 0·137 | 18·0 | 17·1 | 1·05 | 0·26 | 0·16 | 1·63 |
| | G6 | 0·099 | | 0·049 | 18·0 | | 1·05 | 0·18 | | 1·13 |
| | G8 | 0·032 | | 0·016 | 17·0 | | 0·99 | 0·18 | | 1·13 |
| H | H4 | 0·281 | 0·360 | 0·101 | 16·7 | 16·3 | 1·02 | 0·22 | 0·14 | 1·57 |
| | H6 | 0·084 | | 0·030 | 16·6 | | 1·02 | 0·16 | | 1·14 |
| | H8 | 0·010 | | 0·004 | 16·1 | | 0·99 | 0·14 | | 1·00 |

*Core area ratio = area of core concrete/area of total concrete.

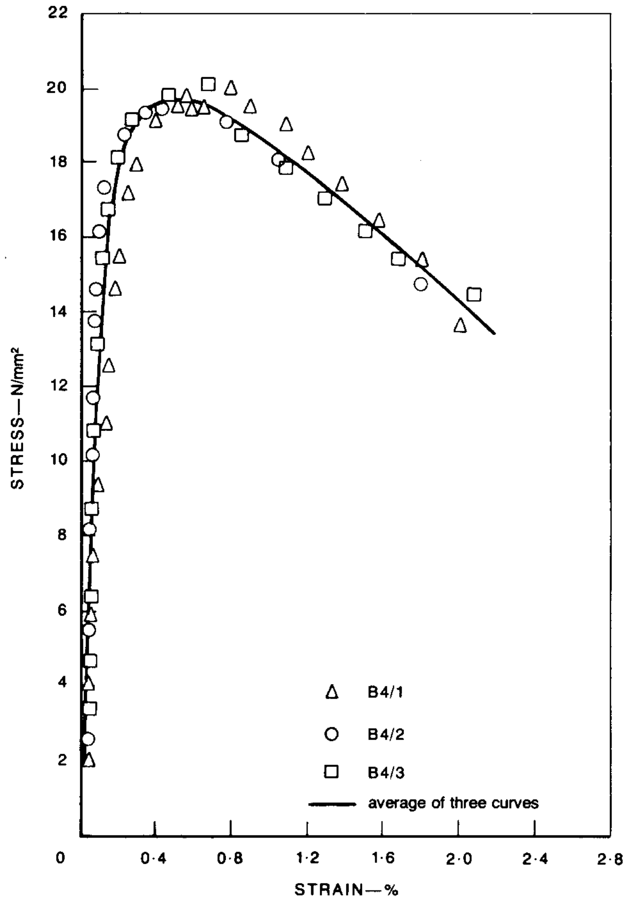



*Figure 2: The average stress–strain curve for three companion specimens of a set.*

## Confinement index and effective confinement index

The confinement is influenced not only by the effective volumetric ratio but also by many factors such as the strength of the binder steel and the strength of the concrete. Nagi Reddy[8] proposed a confinement index as $(p_b - \bar{p}_b)f_y/f_c'$. He obviously assumed that the binder steel had reached its yield strength when the confined concrete had reached its ultimate strength. Later the experiments conducted by S. R. Reddy[13] indicated that the binder steel had not yielded when the spacing of the binders was large and the yield strength of the binder was high. In such a case, the actual stress $f_v$ that develops in the binder steel at the ultimate load has to replace $f_y$ in the confinement index. Also, it is observed from the test results that the confinement index will be more representative of the behaviour of the confined concrete if the breadth of the specimen and spacing of the binders are directly represented on it. S. R. Reddy[13] proposed the following equation for the confinement index,

$$C_i = (p_b - \bar{p}_b)f_v/f_c'\sqrt{b/s} \quad \ldots\ldots\ldots(2)$$

Also, the actual stress in the steel binder is obtained by ascertaining the strain using the equation

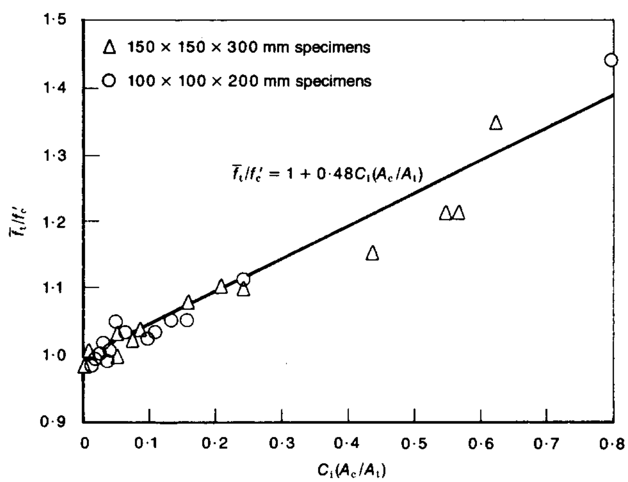
$$\epsilon_v = 0{\cdot}0015 + 0{\cdot}00076(p_b - \bar{p}_b)\sqrt{b/s} \quad \ldots(3)$$

If $\epsilon_c > \epsilon_y$, $f_y$ is to be used in place of $f_v$.

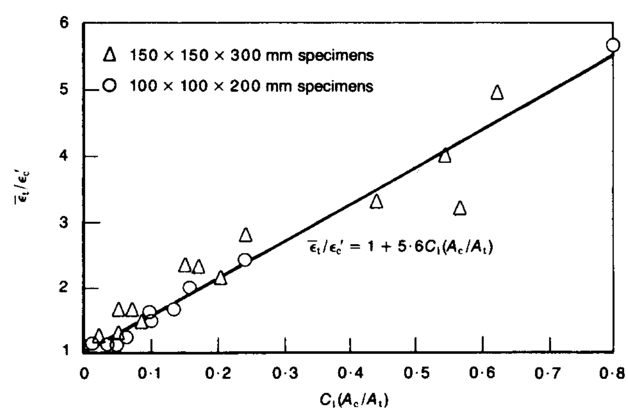
Since the total concrete consists of unconfined cover concrete and confined core concrete, the confinement index of the total concrete will be smaller than that of core concrete. So it is evident that a reduction factor is to be applied to the confinement index of core concrete to get the confinement index of the total concrete. On the basis of the experimental results, this reduction factor is found to be $(A_c/A_t)$, where $A_c$ is the area of the core concrete section based on outer dimensions of stirrups and $A_t$ is the area of the total concrete section and the term $C_i(A_c/A_t)$ is described as the 'effective confinement index'.

## Relationship between increase in maximum stress and the corresponding strains and effective confinement index

From the results given in Table 2 and the plots shown in Figures 3a and b, and by regression




*(a) Relationship between increase in stress and the index*




*(b) Relationship between increase in strain and the index*

*Figure 3: Experimental results and the effective confinement index.*

analysis, it has been found that the best fit is a straight line and the equations are given below

$$\bar{f}_t/f'_c = 1 + 0{\cdot}48C_i\left(\frac{A_c}{A_t}\right) \quad \ldots\ldots\ldots(4)$$

$$\bar{\epsilon}_t/\epsilon'_c = 1 + 5{\cdot}60C_i\left(\frac{A_c}{A_t}\right) \quad \ldots\ldots\ldots(5)$$

The coefficients of correlation are 0·085 and 0·0895 respectively. In the above equations, the term $(A_c/A_t)$ is equal to unity when there is no cover over the binders. Equations 4 and 5 are in reasonably close agreement with those given by S. R. Reddy(13) where the coefficients of $C_i$ are 0·55 and 5·20 respectively for confined concrete without cover.

## Stress–strain curve for confined concrete with cover

The following is the general equation for the stress–strain curve of plain concrete

$$f = \frac{A}{1 + B\epsilon + C\epsilon^2} \quad \ldots\ldots\ldots\ldots(6)$$

where $A$, $B$ and $C$ are constants and are to be evaluated from boundary conditions of the stress–strain curve. In the present investigation, these constants for composite concrete with confined core and unconfined cover are

$$A = E_c = 5481{\cdot}2\sqrt{f'_c}$$

$$B = \frac{E_c}{\bar{f}_t} - \frac{2}{\bar{\epsilon}_t}$$

$$C = \frac{1}{\bar{\epsilon}_t^{\,2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7)$$

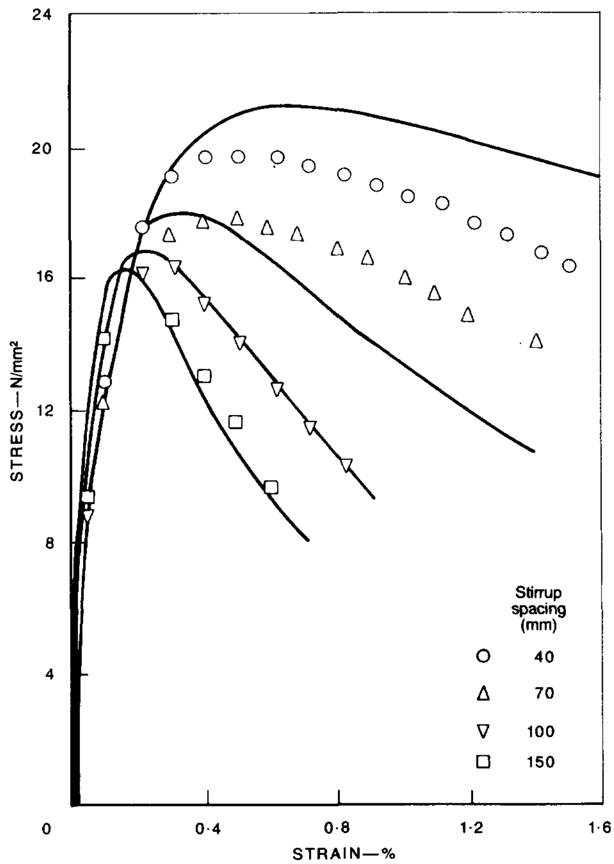



*Figure 4: Comparison of analytical and experimental stress–strain curves.*

## Comparison of experimental and analytical stress–strain curves for composite concrete

The proposed analytical stress–strain curve for total concrete using constants $A$, $B$ and $C$ in terms of $\bar{f}_t$ and $\bar{\epsilon}_t$ have been compared with the experimental curves; a typical one is shown in Figure 4. The maximum values of stresses and strains from experimental and analytical curves are compared in Table 3 and are found to be in agreement.

TABLE 3: **Comparison of experimental and analytical stresses and strains.**

| Series | Group | Experimental | | Theoretical | | | |
|---|---|---|---|---|---|---|---|
| | | Maximum stress $\bar{f}_t$ (N/mm²) | Strain $\bar{\epsilon}_t$ (%) | Maximum stress $\bar{f}_t$ (N/mm²) | Strain $\bar{\epsilon}_t$ (%) | $\frac{\bar{f}_{t\,\mathrm{exptl}}}{\bar{f}_{t\,\mathrm{theor}}}$ | $\frac{\bar{\epsilon}_{t\,\mathrm{exptl}}}{\bar{\epsilon}_{t\,\mathrm{theor}}}$ |
| A | A4 | 22·7 | 0·74 | 22·5 | 0·70 | 1·01 | 1·06 |
| | A7 | 18·5 | 0·32 | 18·8 | 0·40 | 0·98 | 0·80 |
| | A10 | 17·4 | 0·22 | 17·7 | 0·25 | 0·98 | 0·88 |
| | A15 | 16·9 | 0·17 | 16·9 | 0·20 | 1·00 | 0·85 |
| B | B4 | 19·3 | 0·52 | 20·9 | 0·70 | 0·92 | 0·74 |
| | B7 | 17·6 | 0·45 | 17·6 | 0·30 | 1·00 | 1·50 |
| | B10 | 16·3 | 0·26 | 16·6 | 0·25 | 0·98 | 1·04 |
| | B15 | 15·9 | 0·20 | 16·1 | 0·15 | 0·98 | 1·34 |
| C | C4 | 20·6 | 0·46 | 21·9 | 0·60 | 0·94 | 0·77 |
| | C7 | 19·3 | 0·32 | 19·2 | 0·30 | 1·01 | 1·07 |
| | C10 | 17·9 | 0·18 | 18·4 | 0·20 | 0·97 | 0·90 |
| | C15 | 17·6 | 0·16 | 17·9 | 0·15 | 0·98 | 1·07 |

TABLE 3 *continued*

| Series | Group | Experimental: Maximum stress $\bar{f}_t$ (N/mm²) | Experimental: Strain $\bar{\epsilon}_t$ (%) | Theoretical: Maximum stress $\bar{f}_t$ (N/mm²) | Theoretical: Strain $\bar{\epsilon}_t$ (%) | $\frac{\bar{f}_{t\,\mathrm{exptl}}}{\bar{f}_{t\,\mathrm{theor}}}$ | $\frac{\bar{\epsilon}_{t\,\mathrm{exptl}}}{\bar{\epsilon}_{t\,\mathrm{theor}}}$ |
|---|---|---|---|---|---|---|---|
| D | D4 | 14·5 | 0·48 | 15·5 | 0·70 | 0·94 | 0·69 |
| | D7 | 12·9 | 0·28 | 13·1 | 0·30 | 0·98 | 0·93 |
| | D10 | 12·4 | 0·20 | 12·4 | 0·20 | 1·00 | 1·00 |
| E | E4 | 29·4 | 1·14 | 26·8 | 0·70 | 1·09 | 1·63 |
| | E6 | 22·7 | 0·48 | 23·2 | 0·40 | 0·98 | 1·20 |
| | E8 | 21·1 | 0·30 | 21·7 | 0·25 | 0·97 | 1·20 |
| | E10 | 19·6 | 0·22 | 21·0 | 0·20 | 0·93 | 1·10 |
| F | F4 | 20·1 | 0·32 | 21·0 | 0·30 | 0·96 | 1·07 |
| | F6 | 19·7 | 0·20 | 19·8 | 0·20 | 0·99 | 1·00 |
| | F8 | 19·2 | 0·18 | 19·4 | 0·20 | 0·99 | 0·90 |
| | F10 | 18·9 | 0·16 | 19·2 | 0·15 | 0·98 | 1·07 |
| G | G4 | 17·9 | 0·26 | 18·6 | 0·30 | 0·96 | 0·87 |
| | G6 | 17·9 | 0·18 | 17·6 | 0·20 | 1·02 | 0·90 |
| | G8 | 16·9 | 0·18 | 17·1 | 0·15 | 0·99 | 1·20 |
| H | H4 | 16·7 | 0·22 | 17·3 | 0·25 | 0·96 | 0·88 |
| | H6 | 16·6 | 0·16 | 16·6 | 0·20 | 1·00 | 0·80 |
| | H8 | 16·1 | 0·14 | 16·3 | 0·15 | 0·99 | 0·93 |

TABLE 4: **Comparison of maximum stresses and strains obtained experimentally in the shell concrete with the theory of Sargin et al.[1]**

| Series | Group | Confined concrete theory: $f_s'$ (N/mm²) | Confined concrete theory: $\epsilon_s'$ (%) | Sargin et al. theory: $f_s''$ (N/mm²) | Sargin et al. theory: $\epsilon_s''$ (%) | $\frac{f_s}{f_s''}$ | $\frac{\epsilon_s'}{\epsilon_s''}$ |
|---|---|---|---|---|---|---|---|
| A | A4 | 16·6 | 0·16 | 16·4 | 0·146 | 1·01 | 1·10 |
| | A7 | 16·5 | 0·18 | 14·4 | 0·128 | 1·14 | 1·41 |
| | A10 | 16·8 | 0·17 | 15·5 | 0·138 | 1·08 | 1·23 |
| | A15 | 16·6 | 0·16 | 16·7 | 0·149 | 0·99 | 1·07 |
| B | B4 | 15·9 | 0·18 | 15·9 | 0·160 | 1·00 | 1·13 |
| | B7 | 15·2 | 0·15 | 14·7 | 0·148 | 1·03 | 1·01 |
| | B10 | 14·8 | 0·16 | 14·3 | 0·143 | 1·03 | 1·12 |
| | B15 | 15·5 | 0·16 | 15·3 | 0·153 | 1·01 | 1·05 |
| C | C4 | 16·8 | 0·17 | 13·1 | 0·103 | 1·28* | 1·65 |
| | C7 | 17·5 | 0·18 | 13·2 | 0·104 | 1·32* | 1·73 |
| | C10 | 16·8 | 0·16 | 16·6 | 0·130 | 1·01 | 1·24 |
| | C15 | 17·2 | 0·14 | 17·1 | 0·134 | 1·00 | 1·04 |
| D | D4 | 11·8 | 0·15 | 11·9 | 0·120 | 0·99 | 1·25 |
| | D7 | 12·1 | 0·17 | 10·7 | 0·107 | 1·13 | 1·59 |
| | D10 | 12·0 | 0·16 | 11·4 | 0·114 | 1·05 | 1·40 |
| E | E4 | 19·2 | 0·16 | 20·1 | 0·196 | 0·95 | 0·81 |
| | E6 | 19·9 | 0·20 | 18·3 | 0·180 | 1·08 | 1·11 |
| | E8 | 19·1 | 0·18 | 18·1 | 0·178 | 1·05 | 1·01 |
| | E10 | 19·9 | 0·17 | 20·4 | 0·200 | 0·97 | 0·85 |
| F | F4 | 18·1 | 0·18 | 15·2 | 0·127 | 1·19 | 1·42 |
| | F6 | 19·1 | 0·17 | 19·1 | 0·160 | 1·00 | 1·06 |
| | F8 | 18·7 | 0·16 | 19·1 | 0·160 | 0·98 | 1·00 |
| | F10 | 18·5 | 0·13 | 18·7 | 0·157 | 0·99 | 0·83 |
| G | G4 | 16·8 | 0·18 | 16·1 | 0·155 | 1·04 | 1·19 |
| | G6 | 17·1 | 0·18 | 16·9 | 0·158 | 1·01 | 1·14 |
| | G8 | 16·6 | 0·16 | 16·3 | 0·153 | 1·02 | 1·04 |
| H | H4 | 16·2 | 0·17 | 16·1 | 0·139 | 1·01 | 1·22 |
| | H6 | 16·4 | 0·16 | 16·3 | 0·140 | 1·01 | 1·14 |
| | H8 | 15·8 | 0·12 | 16·0 | 0·138 | 0·98 | 0·87 |

*Deleted when the average was calculated.

## Stress–strain curve for the shell concrete

The equilibrium equation (equation 1) is used to separate the shell-concrete stress–strain curve from the core-concrete stress–strain curve, the latter being obtained by evaluating the constants $A$, $B$, $C$ in equation 6 with the values of $f_c$ and $\epsilon_c$ given by S. R. Reddy[13]. This core-concrete stress–strain curve is superposed on the experimental stress–strain curve of composite concrete to obtain the shell-concrete stress–strain curve as shown in Figure 5.

The maximum shall-concrete stress $f_s$ and the corresponding strain $\epsilon_s$ are compared with those obtained based on the Sargin et al.[1] theory, and the results are given in Table 4 and plots are given in Figure 6a and b. It will be seen that most of the points are found to be lying within ±10% for stresses and ±20% for strains. In general, the maximum shell-concrete stress attained is slightly less than or equal to the plain-concrete stress.

## Conclusions

From this investigation, the following conclusions can be drawn regarding the stress–strain characteristics of concrete with confined core and unconfined cover.

(1) The effect of cover upon the ultimate stress and strain of a confined concrete core is a function of an effective confinement index.

(2) The increases in ultimate stress and strain of concrete with confined core and shell are found to vary linearly with respect to effective confinement index and are given by equations 4 and 5.

(3) It is possible to compute the complete stress–strain curve for the shell concrete from the stress–strain curve for the total concrete (experimental) and the confined core concrete (analytical).

(4) At closer spacing, the shell concrete spalls when it reaches the maximum stress value and after that only the core concrete contributes to the strength.

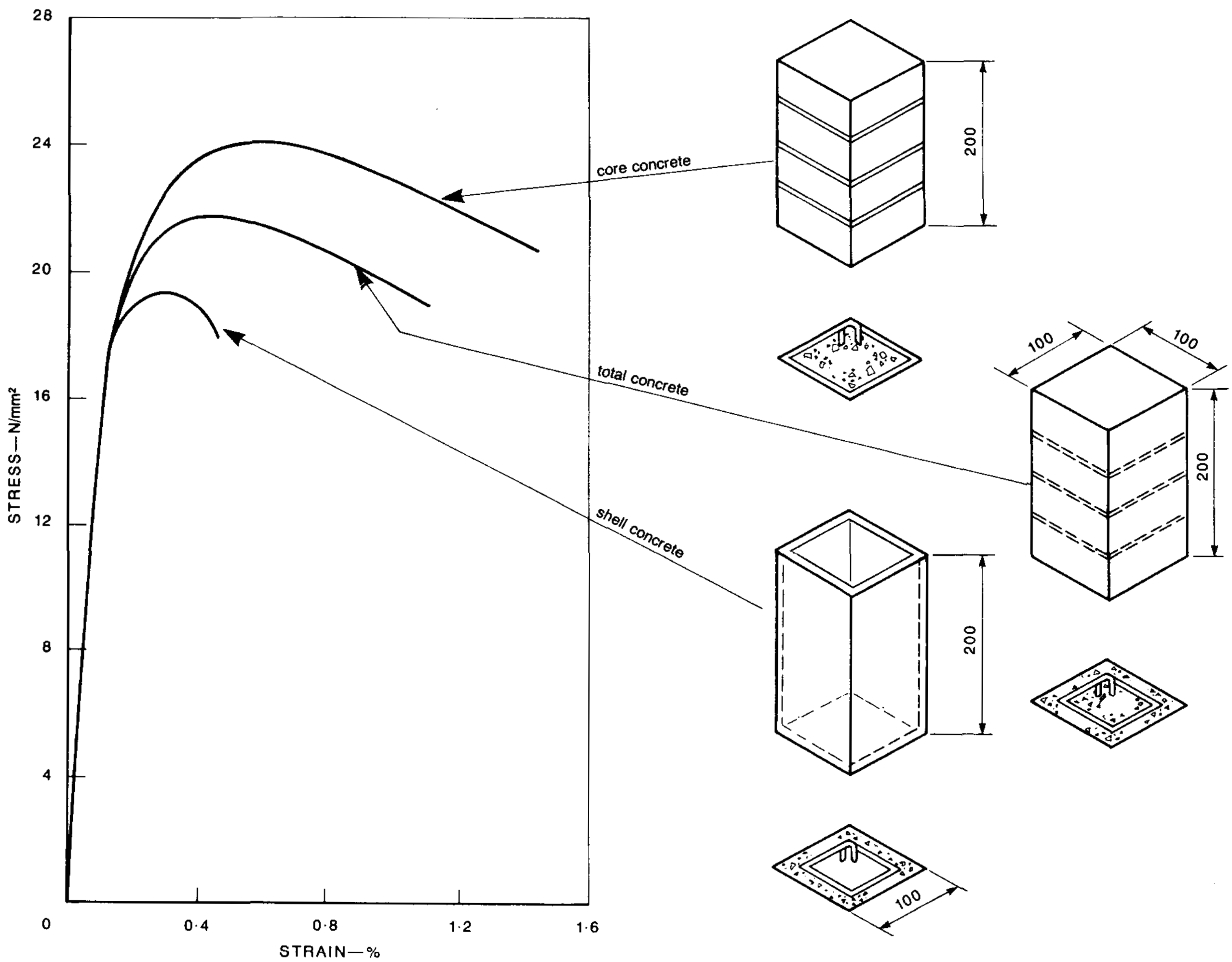
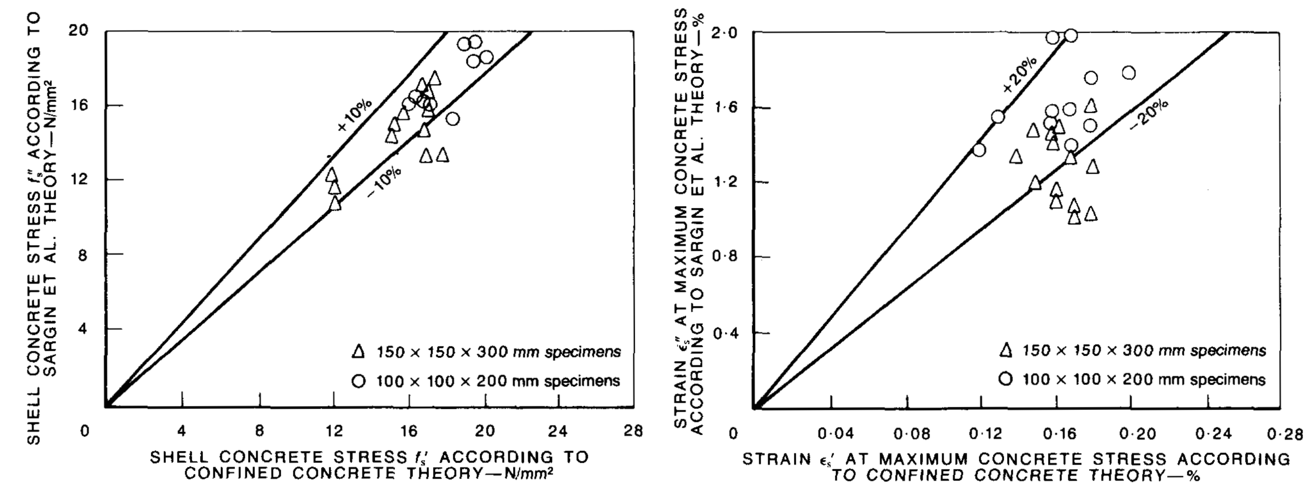



*Figure 5: Stress–strain curves for the shell concrete.*




*(a) Stresses* *(b) Strains*

*Figure 6: Comparison of peak values obtained experimentally in the shell concrete with the theory of Sargin et al.*[1]

(5) In general, the maximum value of stress in the shell concrete is less than or equal to that of plain concrete.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