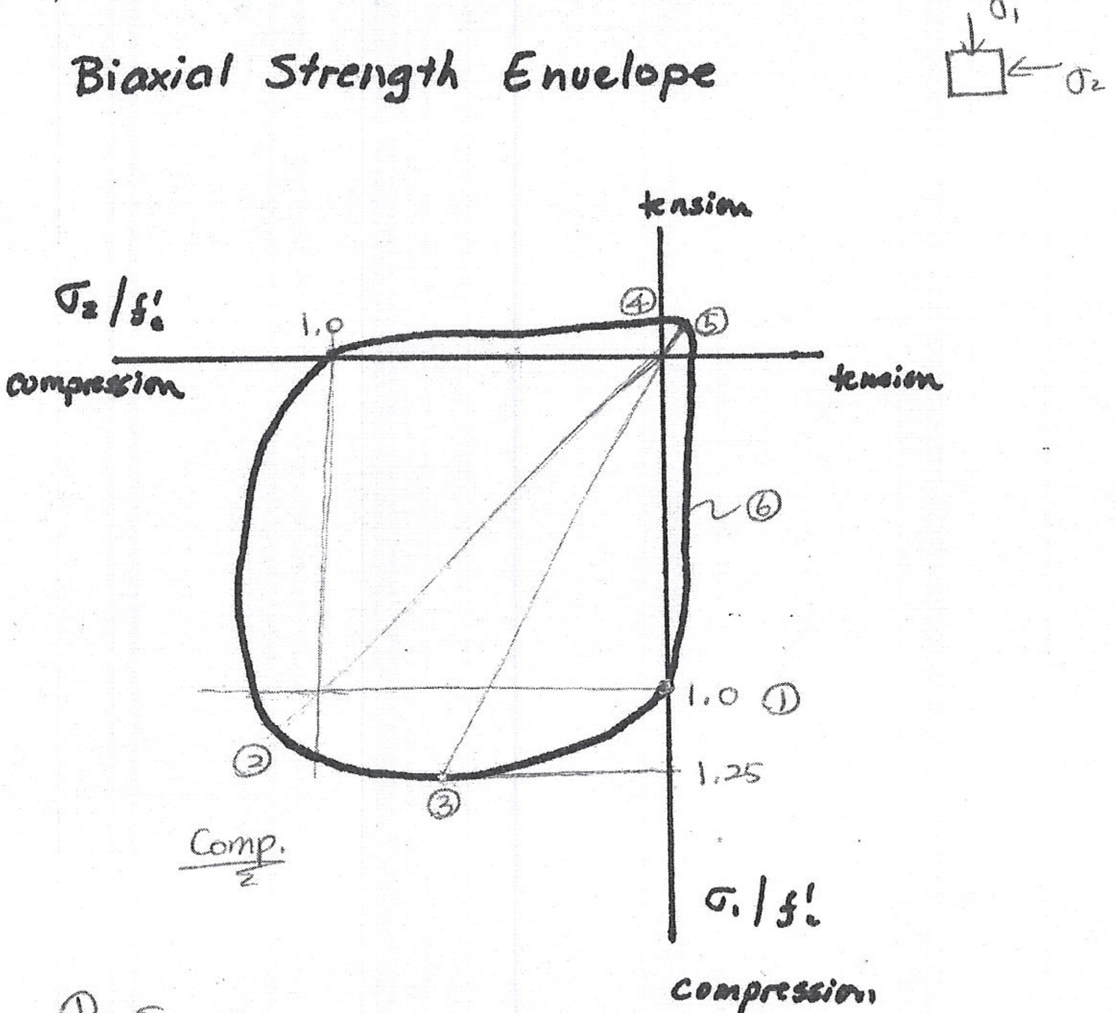
A-11-A

Biaxial Strength Envelope



① $\sigma_2 = 0$

② $\sigma_1 = \sigma_2$

③ $\sigma_2 = \frac{1}{2}\sigma_1$

④ $\sigma_2 = 0$, tension σ_1

⑤ $\sigma_1 = \sigma_2$, tension

⑥ Sensitive to a little tension .

A-11-B

Tension Development - Monotonic Load

ACI Comm 408 : $\ell_{db} = \dfrac{5500\ A_b}{\phi\ K \sqrt{f'_c}}$; Grade 60

$\phi = 0.8$

$K = C_{min} + K_{tr}$

min of side cover, 1/2 spacing, and bottom cover, all measured to center of bar.

K_{tr} = corresponding $\dfrac{A_{tr} f_y}{1500 s} < d_b$

and $K \leq 3 d_b$

top bars : factor by 1.3

LWC : 1.25

Excess steel $A_{s req} / A_{s prov}$

$f_y \neq 60$ $f_y / 60000$

ACI 318 adds factors for epoxy coated bars

A-11-C

Tension Lap Splice - Monotonic Load

ACI Comm 408 - same as l_d

ACI 318 - $l_{sp} = \alpha\, l_d$

$\alpha = 1.0$ if $\leq 1/2$ bars spliced

and $f_s < \frac{1}{2} f_y$

otherwise $\alpha = 1.3$

Hook Anchorage - Monotonic Load

ACI Comm 408 - $l_{hb} = 1200\, d_b / \sqrt{f'_c}$ Grade 60

× 0.7 for ≤ #11, 180° hook, side cover ≥ 2½

× 0.7 for ≤ #11, 90° hook, side cover ≥ 2½"

and tail cover ≥ 2"

× 0.8 for ≤ #11 with ties @ $3\, d_b$

× 1.3 for LWC

A-11-C'

Hooked Anchorage - Seismic

ACI 318 - $l_{dh} = f_y d_b / 65 \sqrt{f'_c}$ #3-11

$\geq 8 d_b$, 6 in.

Straight Bars - Seismic

ACI 318 - $l_d = 2.5\ l_{dh}$ bottom

$\geq 3.5\ l_{dh}$ top

× 1.6 for any length not in confined core

Lap Splices - Seismic

Suvikumar, Gergely, White ; Conc. Int'l, Feb 83

a) $l_s \geq \frac{1860}{\sqrt{f'_c}} d_b \geq 20\ d_b$

× 1.4 top cast

b) $s \leq \frac{A_{tr}\ l_s}{d_b^{\ 2}} \leq 6$ in. for #3

× 3/8 / d_b stirrup for other than #3

CE 244A

JPM

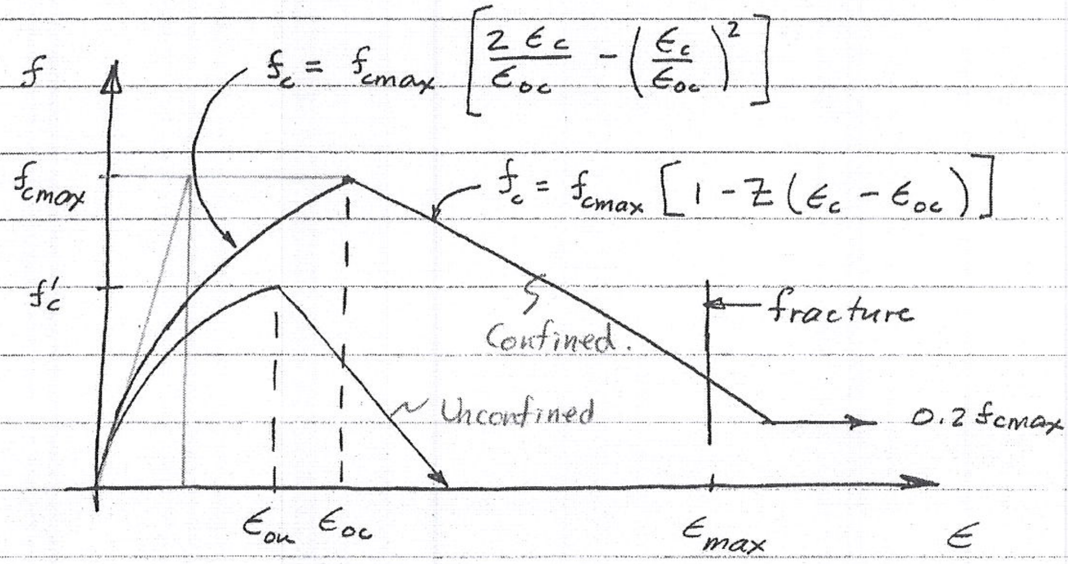
A-11-D

3 Sept 92

9-9-92 (Wed)

Lecture

Confined Concrete Stress-Strain Relation (cont'd)



$$Z = \frac{0.5}{\frac{3 + 0.002 f'_c}{f'_c - 1000} + \frac{3}{4}\rho''\sqrt{\frac{h'}{s}} - \epsilon_{co}}$$

0.003667

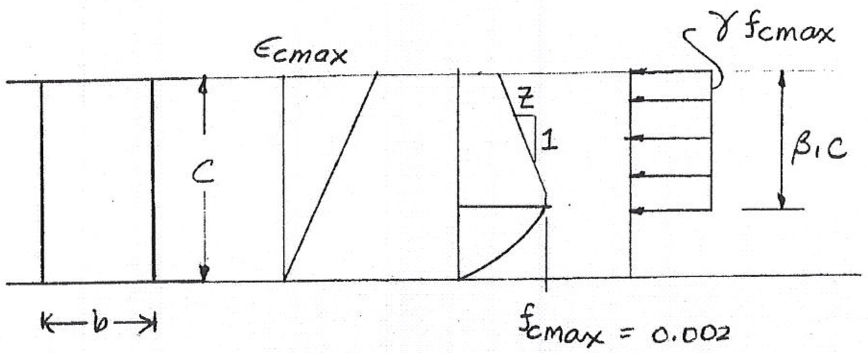
h = cross-section dimension

s = longitudinal spacing

ρ'' = transverse steel ratio

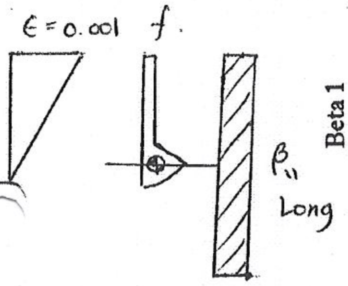
f'_c in psi

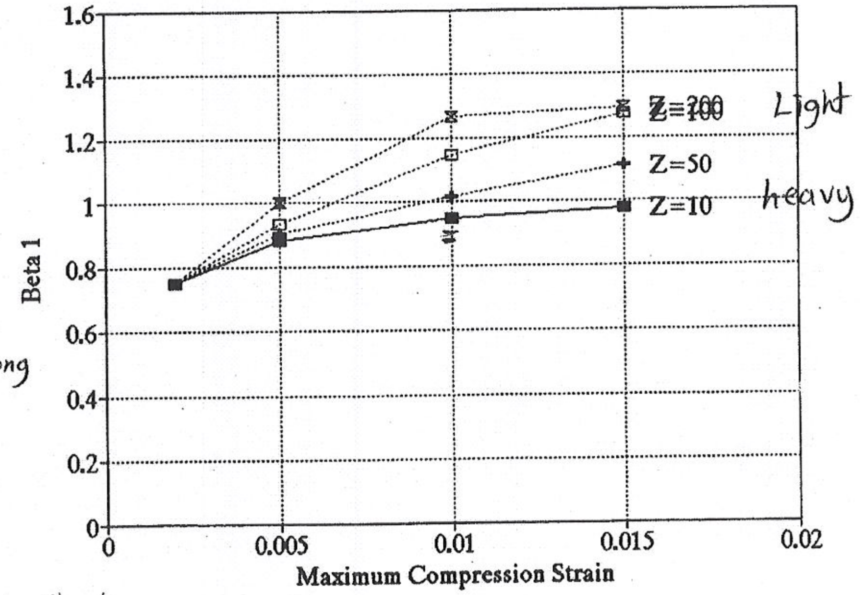
A-11-E

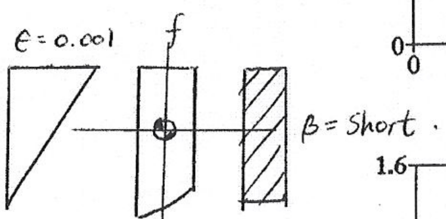


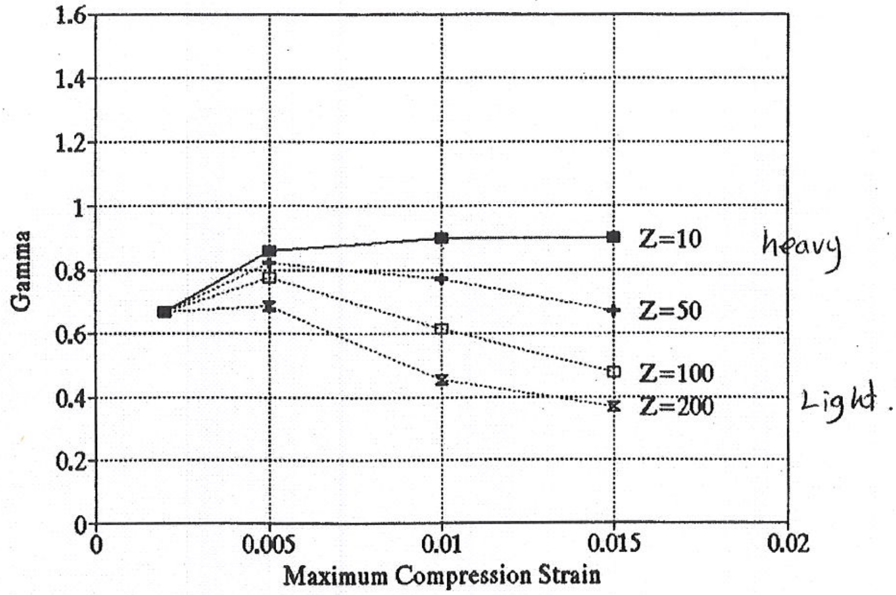
Heavely confined section = shallow slope (Z is small).

Slope = $\frac{Z}{1}$.









A-11-F

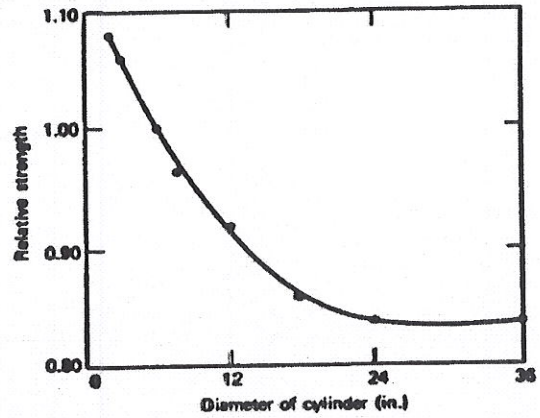


FIGURE 3.5. **Compressive strength of cylinders of different sizes.**[24]

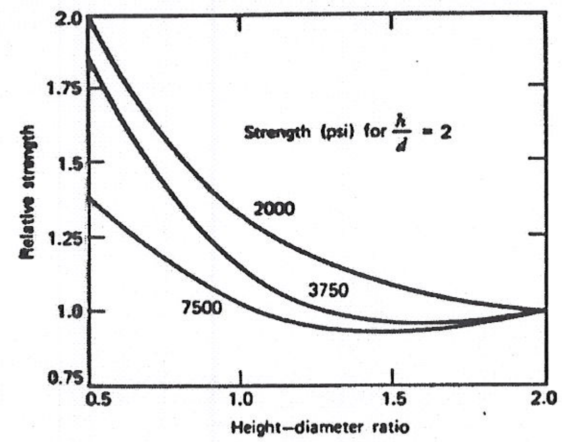


FIGURE 3.4. **Influence of the height–diameter ratio on the measured strength of a cylinder for different strength levels.**[20] **(Reprinted by permission of the American Society for Testing and Materials, Copyright ASTM.)**

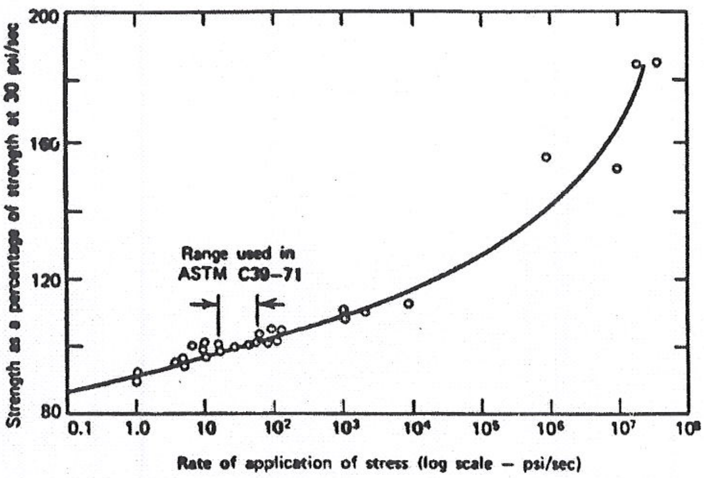
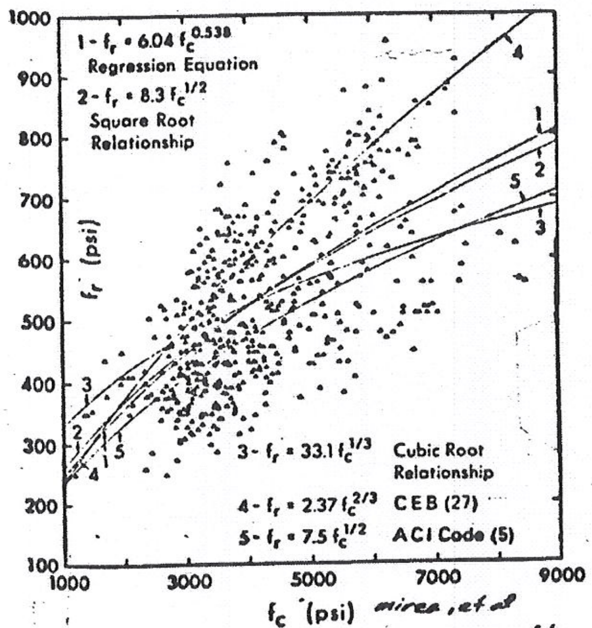


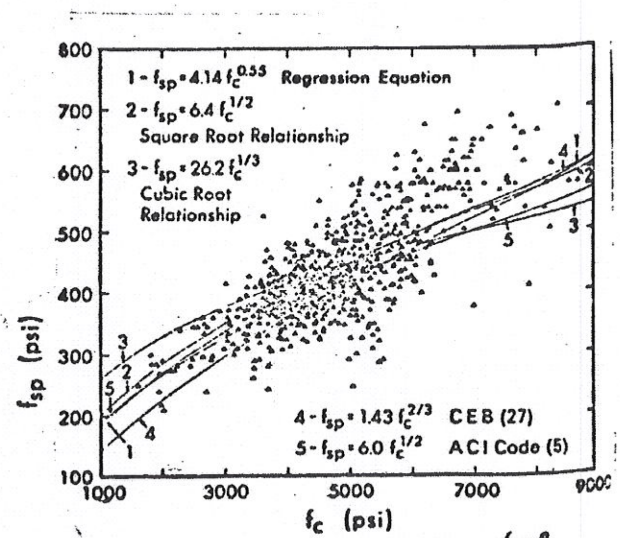
FIGURE 3.7. **Influence of the rate of application of load on the compressive strength of concrete.**[28] **(Reprinted by permission of the American Society for Testing and Materials, Copyright ASTM.)**

Tensile Strength of Concrete

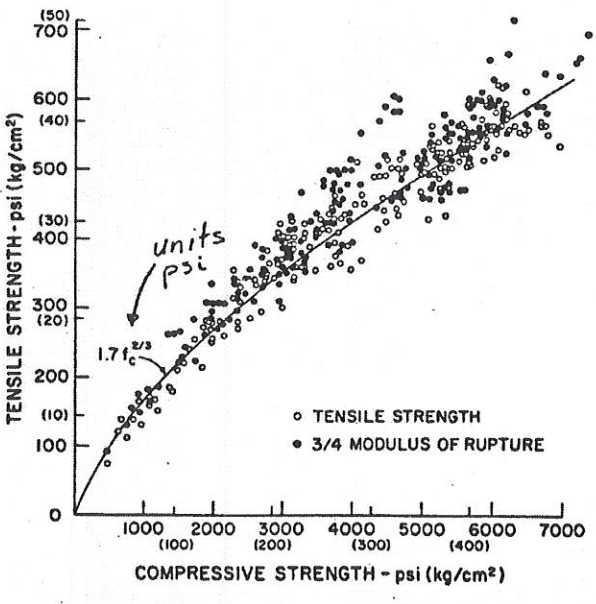
A-11-G



Relation Between f_r and f_c'
(All units psi)



Relation Between f_{sp} and f_c'
(All units psi)



Raphael, J. ACI March-April 1984

Fig. 6—Tensile strength versus compressive strength by two types of test

CE 244A - Biaxial Behavior of Concrete

A-11-H

Reference:

Kupfer, Rüsch and Hilsdorf. J. ACI Aug. 1969

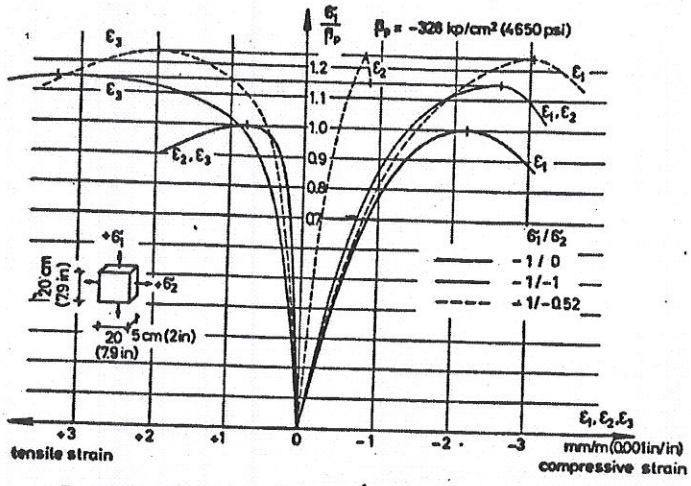


Fig. 9—Stress-strain relationships of concrete under biaxial compression

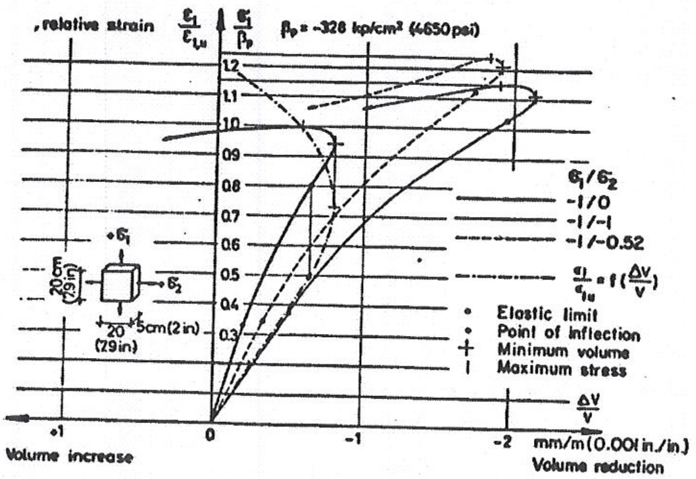


Fig. 14—Volumetric strain of concrete under biaxial compression

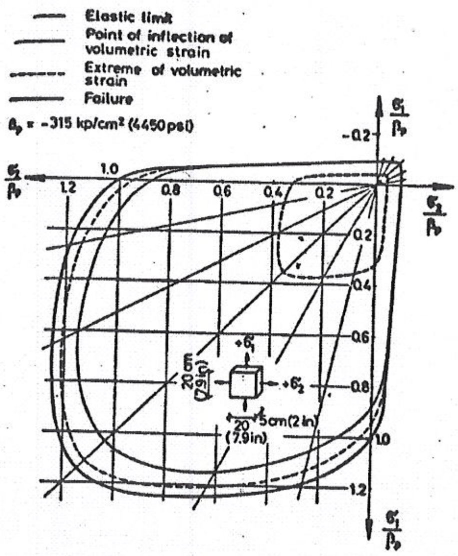
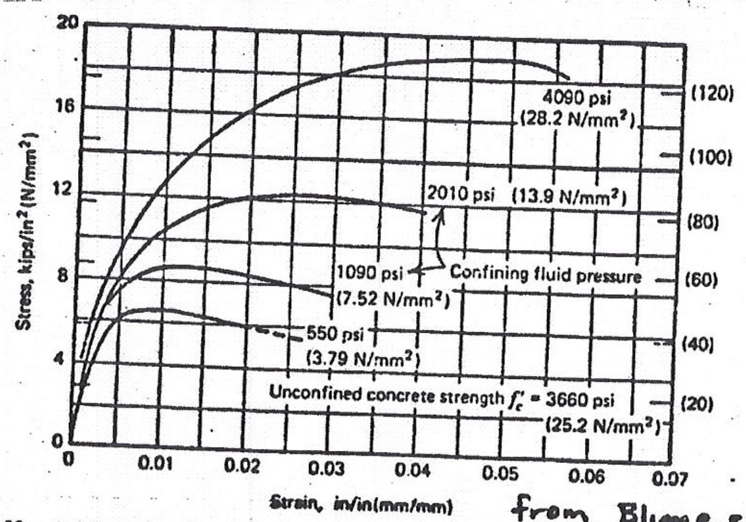


Fig. 15—Stresses at the elastic limit, minimum volume and failure of concrete subjected to biaxial stress states

Strength of Concrete Under Triaxial Stress

A-11-I



from Blume, et al

Fig. 2.11. Axial stress-strain curves from triaxial compression tests on concrete cylinders.[2.13]

$\sigma_1 / f'_c = f_{c\,max} / f'_c$

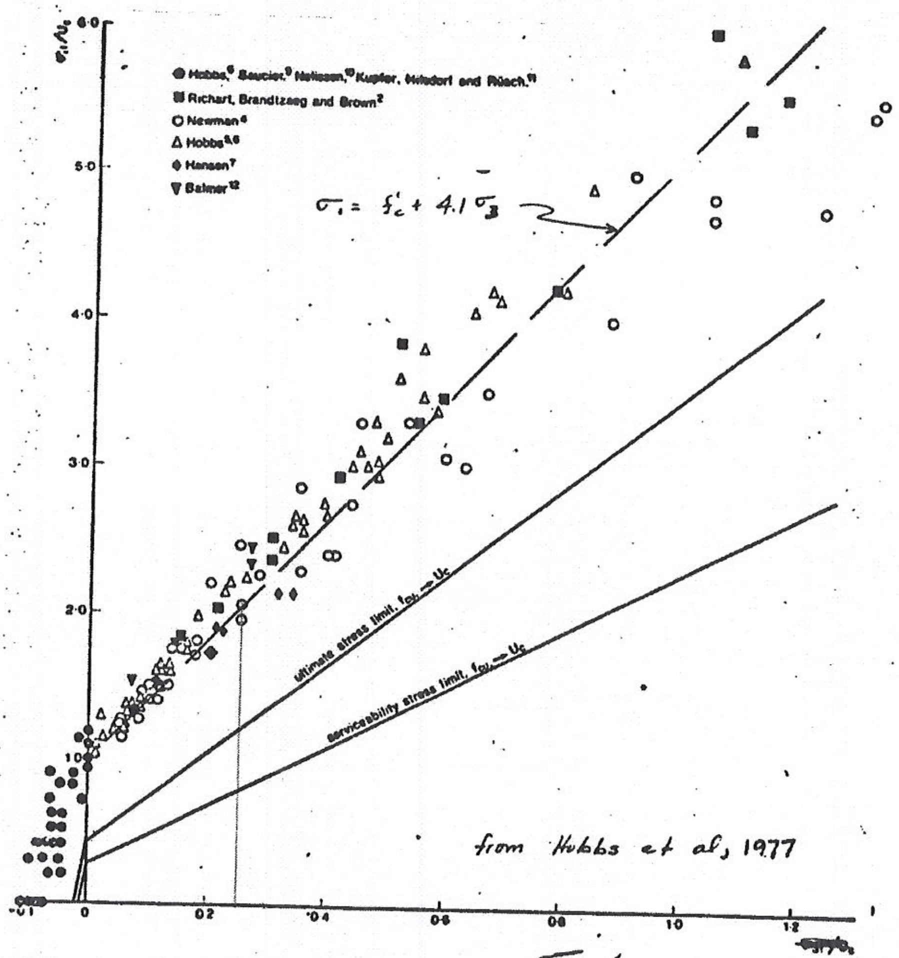


Fig A1. Strength of concrete under multiaxial stress. Wet or air dried concrete

$\sigma_3 / f'_c = \sigma_{min} / f'_c$

Ref: Scott, et al, J. ACI, Jan. 1982. A-11-J

COMPARISON OF TEST RESULTS WITH THE MODIFIED KENT AND PARK STRESS-STRAIN CURVE FOR CONFINED CONCRETE

Low strain rate

The stress-strain relation for concrete confined by rectangular hoops proposed by Kent and Park,[12] and more recently modified by Park, Priestley, and Gill[8], was derived from tests with low strain rates. In the modified Kent and Park relation, the maximum stress attained, Kf_c', is assumed to be reached at a strain of $0.002K$, and the stress-strain relation is

For $\varepsilon_c \leq 0.002K$

$$f_c = Kf_c'\left[\frac{2\varepsilon_c}{0.002K} - \left(\frac{\varepsilon_c}{0.002K}\right)^2\right] \quad (1)$$

For $\varepsilon_c > 0.002K$

$$f_c = Kf_c'\left[1 - Z_m(\varepsilon_c - 0.002K)\right] \quad (2)$$

but not less than $0.2Kf_c'$

where

$$K = 1 + \frac{\rho_s f_{yh}}{f_c'} \quad (3)$$

and

$$Z_m = \frac{0.5}{\dfrac{3 + 0.29f_c'}{145f_c' - 1000} + \dfrac{3}{4}\rho_s\sqrt{\dfrac{h''}{s_h}} - 0.002K} \quad (4)$$

where ε_c = longitudinal strain in concrete, f_c = longitudinal stress in concrete (MPa), f_c' = concrete compressive cylinder strength (MPa), f_{yh} = yield strength of hoop reinforcement (MPa), ρ_s = ratio of volume of hoop reinforcement to volume of concrete core measured to outside of the hoops, h'' = width of concrete core measured to outside of the peripheral hoop (mm), and s_h = center-to-center spacing of hoop sets (mm), where 1 MPa = 145 psi and 1 mm = 0.0394 in.

The modified Kent and Park relation is shown compared with the measured core concrete stress-strain curves for concentrically loaded units in Fig. 9 to 12, and 16 and 17. On the whole, the agreement with measured curves for the low strain rate is good. For the high strain rate, as expected, the relation is conservative.

High strain rate

On the basis of the observed stress-strain behavior in these tests, the modified Kent and Park stress-strain relation may be adapted for the high strain rate by applying a multiplying factor of 1.25 to the peak stress, the strain at the peak stress, and the slope of the falling branch.

Notes: ε_o assumed to be 0.002. A more general form in terms of ε_o is (in MPa)

$$f_c = Kf_c'\left[\frac{2\varepsilon_c}{\varepsilon_o K} - \left(\frac{\varepsilon_c}{\varepsilon_o K}\right)^2\right], \quad \varepsilon_c \leq \varepsilon_o K \quad (1a)$$

$$f_c = Kf_c'\left[1 - Z_m(\varepsilon_c - \varepsilon_o K)\right], \quad \varepsilon > \varepsilon_o K \quad (2a)$$

but not less than $0.2Kf_c'$

$$\text{where } K = 1 + \frac{\rho f_y}{f_c'} \quad (3a)$$

$$Z_m = \frac{0.5}{\varepsilon_{50u} + \varepsilon_{50h} - \varepsilon_o K} \quad (4a)$$

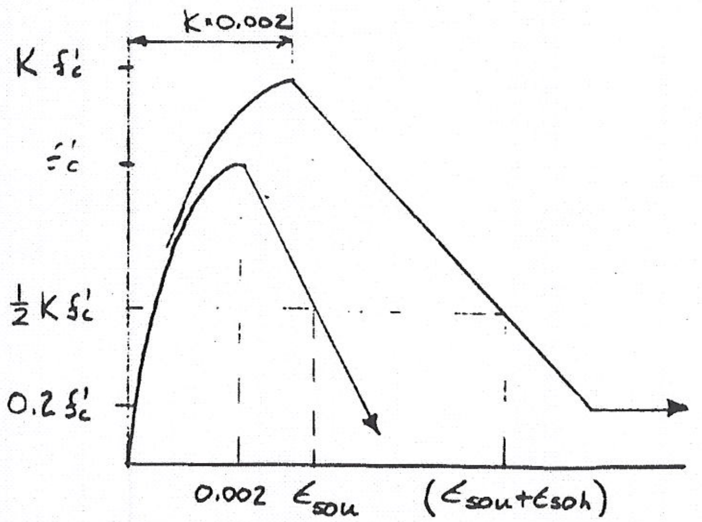
$$\varepsilon_{50u} = \frac{3 + 145\,\varepsilon_o f_c'}{145 f_c' - 1000} \quad (5a)$$

$$\varepsilon_{50h} = \frac{3}{4}\rho\sqrt{\frac{h}{s}} \quad (6a)$$

<u>In psi units</u>

Change 5a to read

$$\varepsilon_{50u} = \frac{3 + \varepsilon_o f_c'}{f_c' - 1000} \quad (5b)$$



A-11-K

Ref: Sheikh and Uzumeri, J. Structural Division, ASCE, Dec. 1982.

Effect of Confining Pressure.—A number of researchers (7,16) have reported that the effect of rectilinear reinforcement is proportional to the square root of volumetric ratio of steel ($\sqrt{\rho_s}$). Richart, et al. (21,22) and Balmer (1) also found that the increase in the concrete strength confined by fluid pressure or spiral reinforcement was proportionately higher for lower confining pressures. Therefore, it appears that the effect of confining pressure in enhancing the strength of the confined concrete is proportional to $(\rho_s \cdot f'_s)^\gamma$, in which f'_s = the stress in the lateral steel at the time of maximum resistance of confined concrete, and γ is a constant (the value of which is less than one). The gain in the strength of confined concrete can then be expressed as equal to $\beta(\rho_s f'_s)^\gamma$ in which β is a constant.

⋮

The final equation then can be written as follows

$$K_s = 1.0 + \frac{2.73\,B^2}{P_{occ}}\left[\left(1-\frac{nC^2}{5.5\,B^2}\right)\left(1-\frac{s}{2B}\right)^2\right]\sqrt{\rho_s f'_s} \qquad (9)$$

in which f'_s is in kips per square inch and P_{occ} in kips;

$$K_s = 1.0 + \frac{B^2}{140\,P_{occ}}\left[\left(1-\frac{nC^2}{5.5B^2}\right)\left(1-\frac{s}{2B}\right)^2\right]\sqrt{\rho_s f'_s} \qquad (10)$$

in which f'_s is in megapascals and P_{occ} in kilonewtons.

K_s values determined from Eq. 9 are listed in Table 2 for all 24 columns, and are compared with the experimental values. Fig. 6 shows the comparison of experimental and the analytical K_s values for all the present tests as well as tests by Roy and Sozen (23), Soliman and Yu (28), and Vallenas, et al. (30).

The effect of distribution of longitudinal and lateral steel on the effectiveness of confinement as represented by λ^* (Eq. 4), is shown in Fig. 7 for square column cores. It is apparent that when spacing of ties is twice the core size, the confinement is not effective at all in enhancing the strength of concrete.

Determination of Parameter ϵ_{s1}.—The ϵ_{s1} is the minimum strain corresponding to the maximum concrete stress. The equation suggested by Soliman and Yu (28) is modified to calculate this strain value. The equation is as follows

$$\epsilon_{s1} = 0.55\,K_s f'_c \times 10^{-6} \qquad (11)$$

for f'_c in pounds per square inch and

$$\epsilon_{s1} = 80\,K_s f'_c \times 10^{-6} \qquad (12)$$

for f'_c in megapascals.

Values of ϵ_{s1}, using the analytical K_s values, are also given in Table 2.

Determination of Parameter ϵ_{s2}.—This is the maximum strain corresponding to the maximum stress in the concrete. The value of ϵ_{s2} is the indication of ductility in concrete achieved by the rectilinear lateral reinforcement. Sargin (24) and Vallenas, et al. (30) proposed similar equations for the calculation of strain corresponding to the maximum stress. However, these equations do not take the steel distribution into account. From the application of these equations to the writer's 24 tests (27), it was observed that the spacing of ties had more pronounced effect than given by the equations. With these points in view, an equation for ϵ_{s2} is proposed on the pattern of the Sargin's equation, which takes into account the steel configuration in addition to tie spacing, amount of lateral reinforcement, and the material properties. The predictions from this equation of the tests conducted by Sargin (24) and Vallenas, et al. (30) are not significantly different from the predictions given by their own equations. The proposed equation is as follows:

$$\frac{\epsilon_{s2}}{\epsilon_{oo}} = 1 + \frac{0.81}{C}\left(1-5.0\left(\frac{s}{B}\right)^2\right)\frac{\rho_s f'_s}{\sqrt{f'_c}} \qquad (13)$$

in which ϵ_{oo} = strain corresponding to the maximum stress in plain concrete (0.0022 in the case of present tests); C is in inches and stresses are in pounds per square inch. In the SI units, the equation would be as follows:

$$\frac{\epsilon_{s2}}{\epsilon_{oo}} = 1 + \frac{248}{C}\left[1-5.0\left(\frac{s}{B}\right)^2\right]\frac{\rho_s f'_s}{\sqrt{f'_c}} \qquad (14)$$

in which C is in millimeters, and stresses are in megapascals.

The values, calculated by using Eq. 13 are listed in Table 2, along with the maximum strain values corresponding to the maximum concrete force obtained in the tests. A comparison of these numbers by themselves would not give a good indication of the accuracy of the predicted curves. The analytical curves are compared with the experimental curves in Fig. 8.

Determination of Parameter ϵ_{s85}.—The slope Z of the unloading part of the curve is calculated by using the equation proposed by Kent and Park (16) with some modification. The suggested equation is

$$Z = \frac{0.5}{\frac{3}{4}\rho_s\sqrt{\frac{B}{s}}} \qquad (15)$$

The strain values corresponding to 0.85 times the maximum concrete stress are calculated, as follows:

$$\epsilon_{s2} = \frac{0.15}{Z} + \epsilon_{s2} \quad \text{or} \quad \epsilon_{s85} = 0.225\,\rho_s\sqrt{\frac{B}{s}} + \epsilon_{s2} \qquad (16)$$

Calculation of ϵ_{s85} was necessary because a number of the experimental stress-strain curves obtained during this study do not extend beyond this point. For some of the columns, even ϵ_{s85} was not available, either due to sudden failure of the column or due to a large strain in the concrete that could not be measured with the instruments used. The experimental and calculated values of ϵ_{s85} are listed in Table 2.

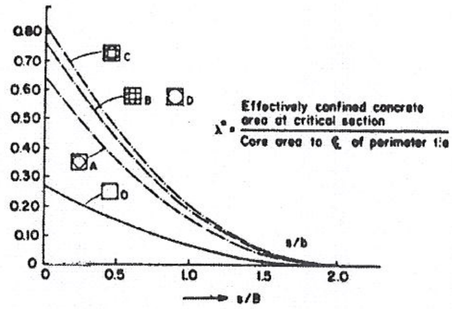
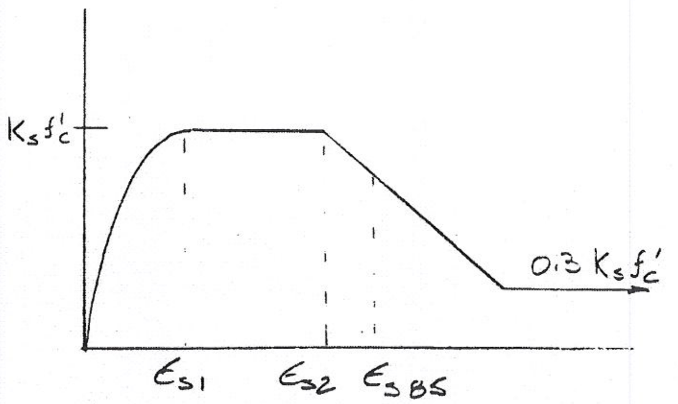


FIG. 7.—Effectively Confined Concrete Area as a Function of Tie Spacing and Core Size for Various Square Steel Configurations



stress-strain curve.

A-11-L

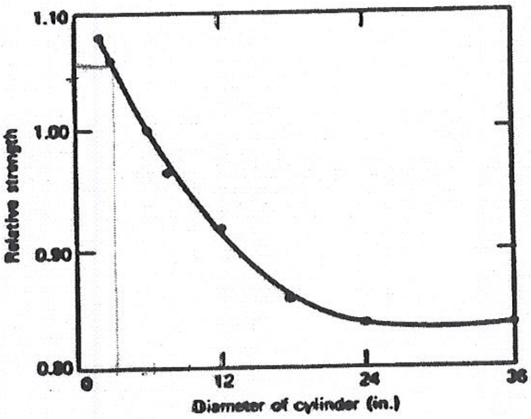


FIGURE 3.5. Compressive strength of cylinders of different sizes.[24]

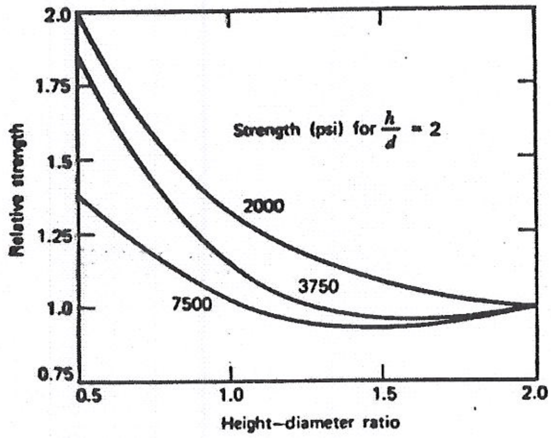


FIGURE 3.4. Influence of the height–diameter ratio on the measured strength of a cylinder for different strength levels.[20] (Reprinted by permission of the American Society for Testing and Materials, Copyright ASTM.)

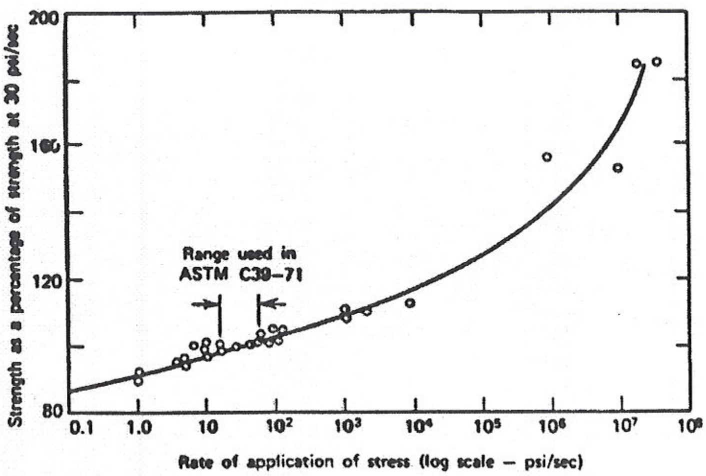


FIGURE 3.7. Influence of the rate of application of load on the compressive strength of concrete.[28] (Reprinted by permission of the American Society for Testing and Materials, Copyright ASTM.)