Critical Mechanical Properties of Structural Lightweight Concrete and Their Effects on Pavement Design

WILLIAM B. LEDBETTER, Assistant Professor of Civil Engineering, Texas A and M University;
ERVIN S. PERRY, Assistant Professor of Civil Engineering,
JAMES T. HOUSTON, Teaching Assistant in Civil Engineering, and
J. NEILS THOMPSON, Professor of Civil Engineering, University of Texas

This study examines the critical mechanical properties of structural lightweight concrete which might affect the performance of concrete pavements. The properties resulting from unrestrained and restrained volume change are presented, with particular attention given to compressive, direct tensile, and indirect tensile (split cylinder) strengths at various ages of the concrete.

The critical properties determined in this study indicate that concrete pavements can be designed with lightweight concrete, and that expected performance from the effects of warping stresses and pavements deflection will be better than that of pavements made with regular weight concrete. However, the effects of restrained volume change of lightweight concrete on pavement performance can be detrimental if improper curing, or curing for too short a time, occurs. The need for further research on the effects of curing on lightweight concrete pavement performance is emphasized.

•THIS STUDY had as its objectives: (a) to explore the properties of volume changes of a structural lightweight concrete due to temperature changes and moisture changes during curing (1, 2); (b) to explore trends of relationships between compressive, split cylinder, and flexural strengths; (c) to report further information on the direct tensile properties (3); (d) to present relationships between direct tensile, compressive, and split cylinder strengths, and to show how these are affected by different curing conditions; (e) to explore properties of the static modulus of elasticity in both tension and compression; and (f) to determine the effects of all lightweight structural concrete properties (4) investigated in this study on the design and performance of concrete pavement structures constructed with structural lightweight concrete.

VARIABLES

Material Variables

The foregoing objectives were accomplished for concrete made with one structural lightweight coarse aggregate and for concrete made with one regular-weight coarse aggregate. Only regular-weight fine aggregate was used. A structural, lightweight, semicoated, expanded shale (5) with a nominal maximum size of ¾ in. was used as a coarse aggregate in all tests of lightweight concrete. Cement factors of 4 sk/cu yd and 5 sk/cu yd were used with air contents of 2 percent (no air entrainment) and 6 percent (using an air entrainment additive).

Paper sponsored by Committee on Rigid Pavement Design.

Curing Variables

The three curing conditions employed were termed bag cured, oven cured, and air cured. Bag curing consisted of moist curing the specimens in sealed polyethylene plastic bags at 75 F, with a relative humidity of approximately 100 percent. The oven-cured specimens were cured at approximately 110 F and low humidity. The temperature in the oven probably varied ±5 F. The air-cured specimens were cured at approximately 50 percent relative humidity and 75 F generally prevailing in the laboratory. Concrete properties were determined at ages of ½, 2, 7, and 28 days in conjunction with the foregoing variables.

CONSTANTS

To isolate the relationships between test parameters, the following variables were held constant throughout this study: (a) mixing time and sequence, (b) cement type, (c) batch size, (d) air-entrainment type, (e) consistency, (f) test procedure (specimen size, rate of loading, etc.), and (g) fine aggregate type (regular weight only).

Materials, mixing techniques, and testing procedures have previously been discussed (1, 2).

DIRECT TENSILE TEST

General

Concrete tensile strength has long been of significance to design engineers. This property has been very difficult, if not impossible, to determine accurately and reliably. Many methods have been developed and tried, but none has experimentally evaluated this important property with any high degree of certainty (6). The main reasons for this difficulty lie in the nature of the material. Concrete, being relatively weak in tension, is significantly influenced by small eccentricities in applied tensile loads, stress concentrations, variations in paste-aggregate ratios throughout the specimen, etc.

In 1955 a method was reported by Todd (7) which offered a technique to determine tensile strength, tensile stress-strain relationship, and the effects of restrained volume changes on tensile strength and stress-strain characteristics of structural concrete. This idea has been amplified and developed at the University of Texas. Several studies have been made on the use of this method and the applications of the information obtained to structural problems(8, 9, 10).The method has been further modified and used extensively in this study.

Test Specimen

The test specimen consists of a thick-walled steel tube on which electrical SR-4 strain gages are mounted; these gages are protected by a brass sleeve around the tube, and the tube is encased in a specimen of concrete. Figure 1 shows an overall view of the steel tube and the brass sleeve. The tube surface has deformations which aid in bonding the concrete to the steel. The brass sleeve, in addition to moisture-proofing the gages, serves to reduce the cross-sectional area of the concrete, and thereby to insure concrete failure at the point where the gages are mounted.

An overall view of the completed tube assembly is shown in Figure 2. Petrosene wax is sloped around the ends of the sleeves in a gradual taper to reduce stress concentrations in the concrete surrounding the steel specimen. Figure 3 shows the complete specimen encased in the concrete and loaded to concrete failure, with the failure crack painted to indicate its position. A schematic drawing of the entire assembly is shown in Figure 4. The "O" rings in the sleeve prevent moisture in the concrete from entering the cavity around the gages.

Two etched-foil type 90-deg rosette SR-4 strain gages are mounted on the steel tube, with one grid of each parallel to the longitudinal axis of the tube and the other grid perpendicular to the axis. A close-up view of the strain gages is shown in Figure 5. These two gages (a total of four grids) are wired together to form a full wheatstone

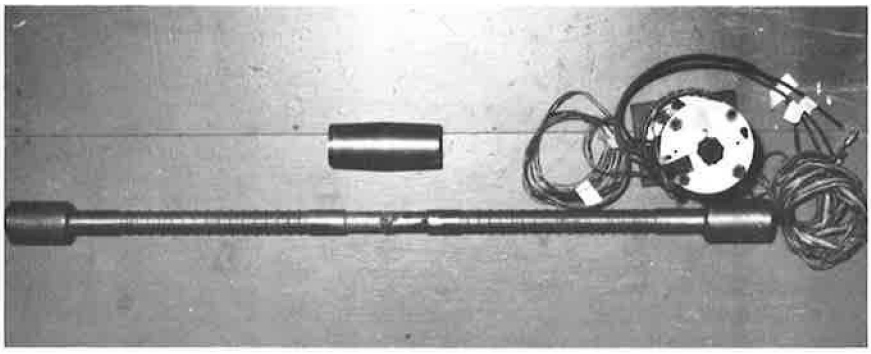

Figure 1. Disassembled steel tube and brass sleeve.

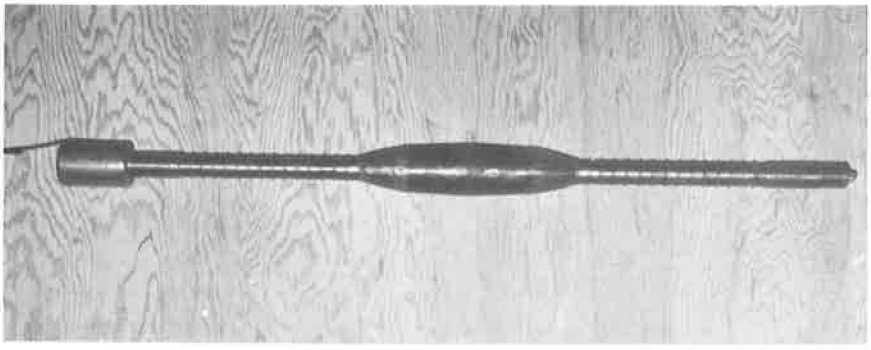

Figure 2. Assembled steel tube and brass sleeve.

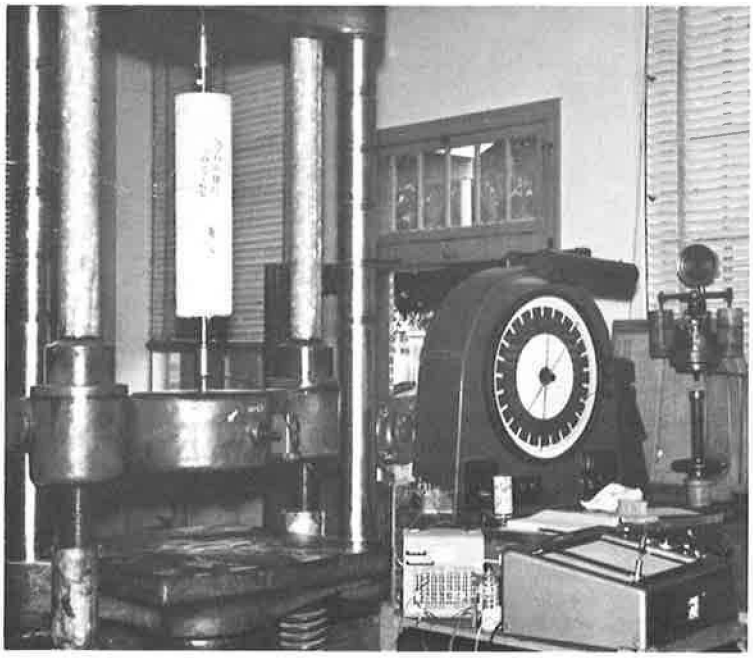

Figure 3. Overall view of direct tensile test concrete specimen loaded to failure.

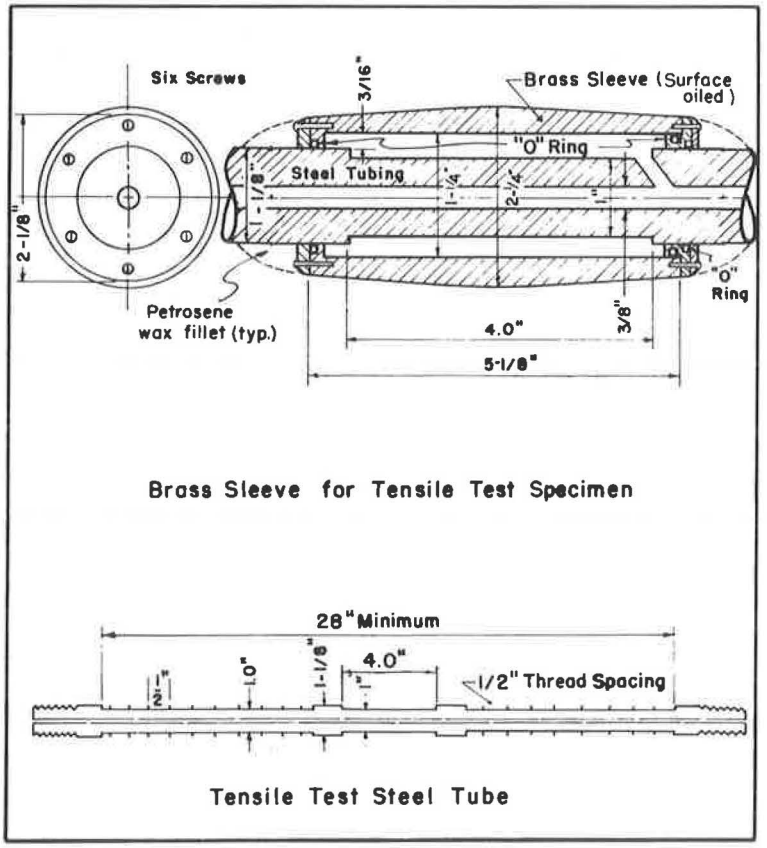

Figure 4. Concrete direct tensile test specimen.

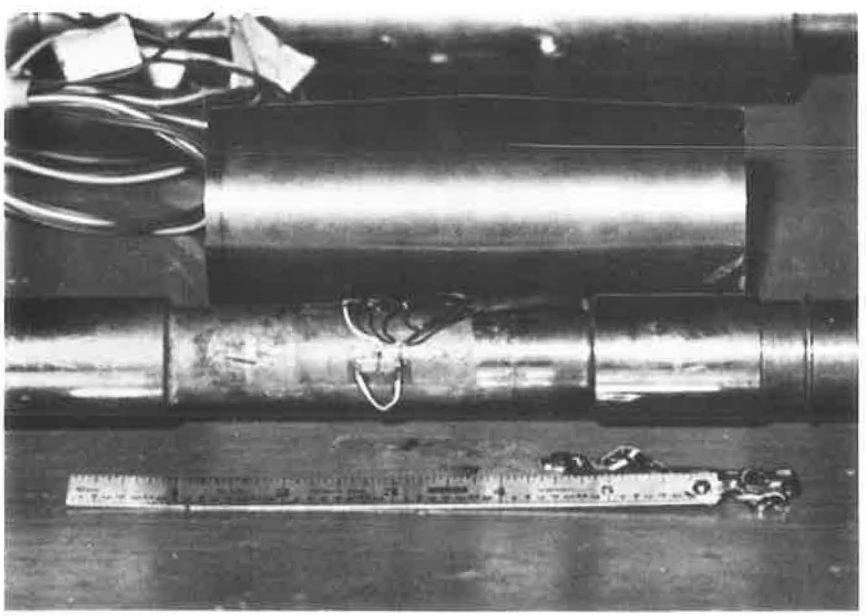

Figure 5. Closeup of direct tension steel tube showing 90-deg rosette etched foil SR-4 strain gage.

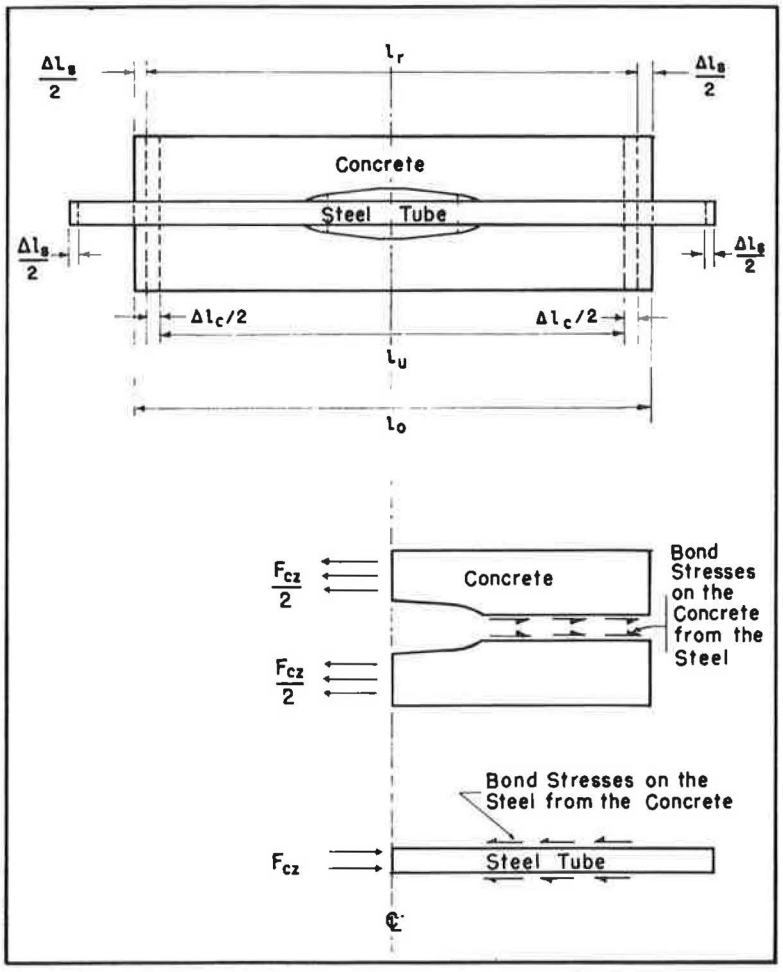

Figure 6. Direct tensile test specimen showing simplification of forces resulting from restrained hydration shrinkage in the concrete.

bridge circuit. The lead wires are carried from the gages along the inside of the tube to the end of the specimen and then to a switch unit.

Restrained Shrinkage Measurement

With the strain-gage arrangement on the steel specimen, the strains occurring on the tube surface can be accurately measured from the time the concrete is molded around the steel specimen. As the concrete hardens from a viscous pseudofluid into a pseudocrystalline solid, extremely complicated chemical reactions and changes of state occur, which cause volume changes to occur in the concrete mass. The steel, being relatively stable dimensionally, partially restrains this volume change in the concrete, thereby causing strains to occur in the steel and the concrete.

A simplified schematic of the effects of a restrained concrete shrinkage is shown in Figure 6. Due to the occurrence of shrinkage phenomenon in the concrete, the concrete of original length ℓ_o would, if unrestrained, shrink to an unrestrained length ℓ_u. But, because of the presence of the steel, only partial shrinkage can take place to a restrained length ℓ_r somewhere between ℓ_o and ℓ_u. Ideally, this results in a net tensile strain in the concrete, ϵ_{cz}, due to restrained concrete volume changes of

$$\epsilon_{cz} = \frac{\ell_r - \ell_u}{\ell_o} = \frac{\Delta\ell_c}{\ell_o}$$

and, ideally, a net compressive strain in the steel ϵ_{sz} of:

$$\epsilon_{sz} = \frac{\ell_o - \ell_r}{\ell_o} = \frac{\Delta\ell_s}{\ell_o}$$

if it is assumed that no slippage occurs between the steel and the concrete. During this volume change period, there are no external forces on the specimen; therefore, since equilibrium exists at the centerline of the specimen, the total tensile force in the concrete must equal the total compressive force in the steel from the restrained shrinkage. Or,

$$F_{cz} = -F_{sz} \tag{1}$$

where

F_{cz} = total force in concrete due to restrained volume changes, lb; and
F_{sz} = force in steel due to restrained volume changes, lb.

It follows, therefore, if the true steel strain the steel ϵ_{sz} is measured, that the force in the concrete, and hence the stress in the concrete, can be computed by:

$$\sigma_{cz} = \frac{F_{cz}}{A_c} = \frac{-F_{sz}}{A_c} = \frac{-\epsilon_{sz} E_s A_s}{A_c} \tag{2}$$

where

σ_{cz} = concrete stress due to restrained volume changes, psi; and
A_c = concrete cross-sectional area, sq in.

Therefore, by measuring the strain in the steel at the centerline of the specimen at given time intervals, or continuously from the time the concrete is poured, a concrete stress vs concrete age can be accurately determined throughout the hydration period.

An important point in the foregoing analysis is that no assumption was made concerning the relationship between the true steel strain due to restrained volume changes ϵ_{sz} and the concrete strain due to restrained volume changes ϵ_{cz} at the centerline during this restrained shrinkage process. Obviously, since the forces in the steel and the concrete are equal as given in Eq. 1,

$$\epsilon_{sz} \neq \epsilon_{cz}$$

at the centerline unless the relationship between A_s / A_c and E_s / E_c is such that the strains could be equal without violating Eq. 1. This presents no difficulties since the steel and concrete are unbonded at the centerline of the specimen and slip at the interface can take place easily.

Direct Tensile Load Test

During the tensile load test, a uniaxial tensile load is applied to the ends of the steel tube. The composite specimen of steel and concrete combines to carry the applied external load. Using a slide-wire potentiometer bridge circuit attached to the load dial on the testing machine, the change in resistance corresponding to the change in the applied load was fed into the Y-axis of a Mosely Autograph X-Y plotter. The signal from

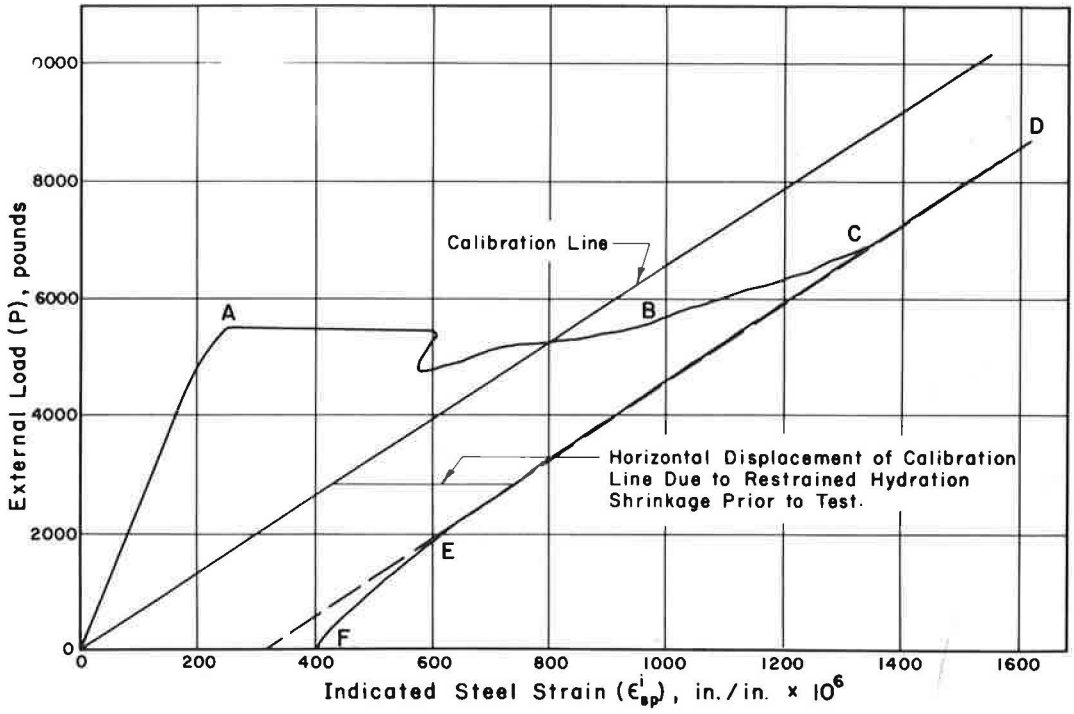

Figure 7. External load-indicated steel strain test trace for direct tensile test of concrete.

the full external bridge strain-gage circuit in the specimen was fed into the X-axis of this same plotter. After suitable load and strain calibration, external applied load vs indicated steel strain was accurately traced on a sheet of graph paper by this plotter. A reproduction of a test trace of external load vs indicated steel strain is shown in Figure 7.

The load is carried by the composite action of concrete and steel from point 0 to point A. At point A the concrete has reached its full tensile capacity and on application of the additional load, the concrete fails in tension. As this occurs, the load being carried by the concrete is suddenly shifted to the steel with a corresponding sudden increase in indicated steel strain.

To determine the net load being carried by the concrete, it is necessary to plot on the graph the results of the load vs indicated steel-strain calibration line of the bare specimen (without the concrete) which is determined by running a separate load test with the bare specimen. Then the external load carried by the concrete P_c is simply the vertical distance between the 0A curve and the steel calibration line.

$$P_c = P - P_s$$

and

$$\sigma_{cp} = \frac{P_c}{A_c} \tag{3}$$

where

P = total external load, lb;
P_s = external load in steel, lb; and
σ_{cp} = concrete stress due to external load, psi.

Often the initial concrete crack does not extend throughout the cross-section of the concrete and, after initial cracking at point A, the reduced concrete section may carry some small additional load before completely failing as is shown between points B and C in the figure. The drop in the applied load between point A and B is due to the fact that the machine is not straining fast enough to counteract fully the rapid changes in strain at concrete failure, thus causing the applied load to drop off momentarily. There is also a slight time lag in plotting time due to the relatively slow dynamic response capability of the X-Y plotter. From points C to D the steel at the centerline of the specimen is carrying the entire external load, and the line is parallel to the steel calibration line. The test trace from C to D is horizontally separated from the steel calibration line by an amount equal to the initial compressive strain, ϵ_{sz}, in the steel. As unloading occurs, the test trace unloads to a point E, parallel to the steel calibration line, then deviates from this parallel line as the concrete crack does not close completely because of broken particles of concrete that become displaced on the fracture face.

During an early portion of the curve up to point A, the external load is carried by both the steel and the concrete at the centerline of the specimen. Assuming there is sufficient bond at both ends of the concrete specimen to insure that no slippage is occurring under load, for compatibility to exist at this centerline,

$$\epsilon_{sp} = \epsilon_{cp} \tag{4}$$

where

ϵ_{sp} = steel strain due to an external load; and
ϵ_{cp} = concrete strain due to an external load.

This strain condition is quite different from that existing from strain volume changes before testing, and thus

$$P_c \neq P_s \tag{5}$$

which is reversed from the conditions existing before loading.

Eq. 4 was verified experimentally by placing two concrete embedment gages inside the concrete adjacent to the steel specimen gages on opposite sides of the cylinder to measure ϵ_{cp} directly. An extremely close agreement was obtained.

This, the true concrete strain, ϵ_{cp}, from the external applied load can be determined from the test trace.

Using Eqs. 3 and 4, the concrete stress-strain curve from applied external loads can be calculated and plotted. From this curve the modulus of elasticity of the concrete in tension E_{ct} can be calculated. Referring to Eq. 2 and assuming the E_{ct} during testing is the same as the E_{ct} just before testing, ϵ_{cz} can be determined by

$$\epsilon_{cz} = \frac{\sigma_{cz}}{E_{ct}} \tag{6}$$

Therefore, combining Eqs. 2 and 3, the total corrected tensile stress σ_{ct} in the concrete can be calculated from

$$\sigma_{ct} = \sigma_{cz} + \sigma_{cp} \tag{7}$$

And, combining Eqs. 4 and 6, the total corrected tensile strain in the concrete ϵ_{ct} can be calculated from

$$\epsilon_{ct} = \epsilon_{cz} + \epsilon_{cp} \tag{8}$$

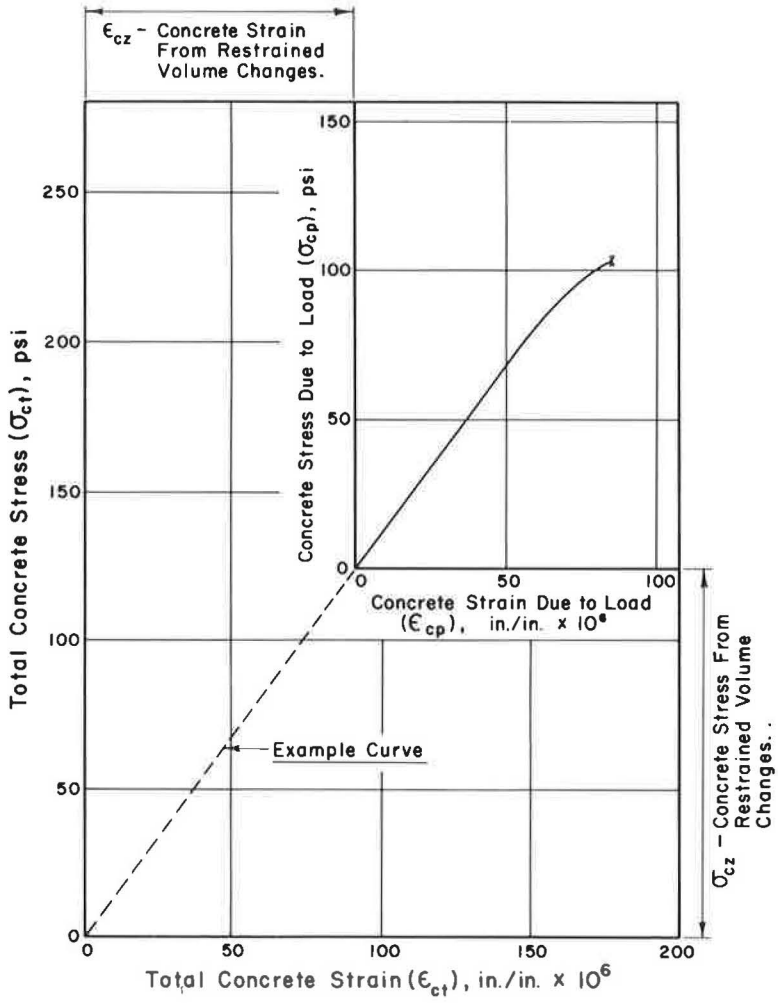

Figure 8. Structural lightweight concrete direct tensile stress-strain curve.

Figure 8 shows a sample concrete direct tensile stress-strain curve and the coordinate axes for the total stress-strain and the stress-strain from the external applied load. This figure is a graphical representation of Eqs. 7 and 8 and summarizes the information which may be obtained from the direct tensile test method.

TENSILE STRENGTH PROPERTIES

General

Tensile stress-strain properties, the effects of curing conditions on tensile strengths, and the various relationships between direct tensile, compressive, and split-cylinder strengths are presented in this section. Data used in plotting some of the curves are given in Table 1.

Effects of Environment on Tensile Stress-Strain Properties

Examples of the tensile stress-strain curves obtained in this investigation are shown in Figures 9 through 11. Each curve represents a single test. The dashed portion of each of the curves, whose ordinate is labeled σ_{cz}, represents the amount of restrained concrete volume-change stress present in the concrete before testing. The solid portion

TABLE 1

STRUCTURAL LIGHTWEIGHT CONCRETE DIRECT TENSILE TEST DATA TABULATION (AVERAGE VALUES)

Curing	Cement Factor (sk/cu yd)	Air (%)	Age (days)	f'_c (psi)	f_{sp} (psi)	f_t (psi)	σ_{cz} (psi)	f_{tu} (psi)	$\epsilon_{ct} \times 10^6$	$\epsilon_{cz} \times 10^6$
Bag	5.0	6.0	7	3,160	398	289	-21	310	177	-10
Air				2,860	326	183	20	163	93	11
Oven				2,860	336	249	89	160	148	40
Bag	5.0	6.0	28	4,920	505	375[a]	-24	399	—[a]	- 6
Air				3,500	468	331	61	270	184	25
Oven				3,260	326	237	143	94	146	74
Bag	4.0	6.0	7	2,170	296	180	-22	202	99	- 9
Air				1,979	249	130	53	77	87	28
Oven				1,600	214	119	87	32	88	54
Bag	4.0	6.0	28	2,620	280	178	- 3	182	84	- 1
Air				2,345	255	156	96	60	95	45
Oven				1,860	216	225	123	102	177	91
Bag	5.0[b]	1.0	7	4,780	456	221	7	214	84	2
Bag	5.0[b]	1.0	28	4,750	496	372	26	346	92	5
Bag	5.0	2.0	7	3,827	420	222	-11	233	102	- 4
Oven				3,093	330	245	136	109	129	63
Bag	5.0	2.0	28	5,475	455	223	-77	300	142	-24
Bag	4.0	2.0	7	2,601	329	190	0	190	113	0
Air				1,944	285	166	30	136	105	12
Oven				1,807	195	158	94	64	95	48
Air	4.0	2.0	28	2,210	368	207	102	105	78	20
Oven				1,740	210	142	136	6	54	41

[a]Bond, rather than tensile, failure occurred in this specimen.
[b]This mix is regular-weight concrete (SG).

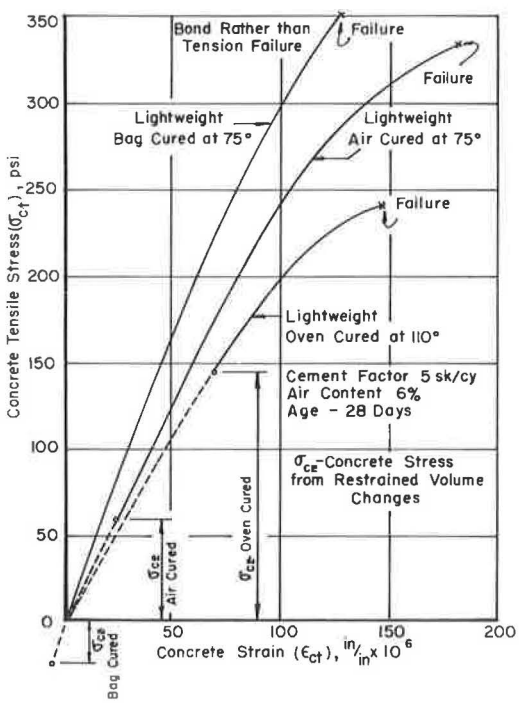

Figure 9. Stress-strain curves for three curing conditions for cement factor = 5 sk/cu yd, 6% air, and age of 28 days.

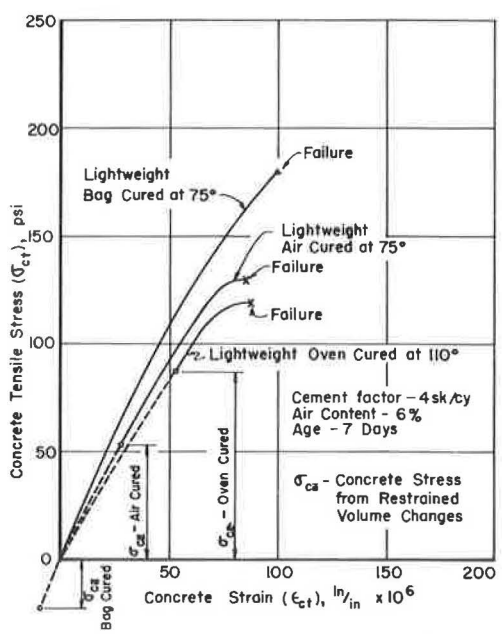

Figure 10. Stress-strain curves for three curing conditions for cement factor = 4 sk/cu yd, 6% air, and age of 7 days.

represents the stress-strain characteristics obtained from the direct tension test. The dashed portion of the curves does not represent the actual stress and strain behavior during the period before testing, but does represent the residual stress existing in the concrete at the beginning of the tension test. The corresponding residual strain was determined by the following procedure.

The strain at the beginning of the solid portion of the stress-strain curves was determined by projecting a straight line with a slope of E, determined from the loading phase of the tension test, back to zero stress. This represents the strain that would relieve the residual stress, using the modulus of elasticity existing at the time of test. The actual residual strain, which was affected by creep and changes in the modulus of elasticity with age, was obviously larger.

Since each curve represents only a single test, values of tensile strength, tensile strain, and modulus of elasticity in these curves may not be the generally expected values.

Observation of these stress-strain curves reveals several important factors which affect the tensile strength of concrete. In general, the bag-cured specimens exhibited the highest tensile strength and the highest modulus of elasticity. Air-cured specimens had the next highest tensile strength and oven-cured specimens had the lowest tensile strength.

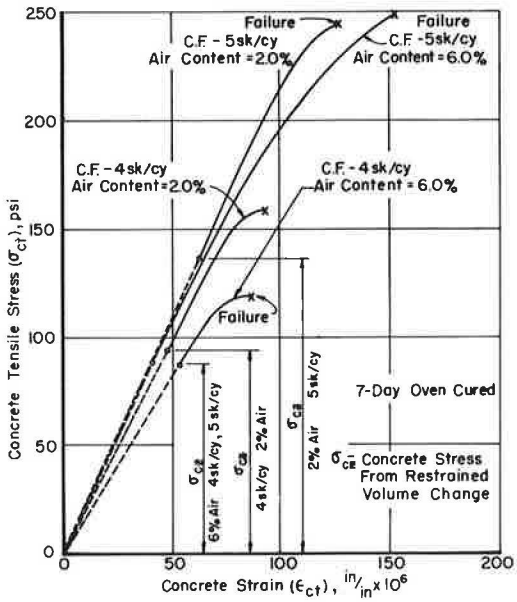

Figure 11. Stress-strain curves for 7-day oven-cured specimens with different cement factors and air contents.

The ultimate tensile strength at 28 days was generally larger than that at 7 days, regardless of environment. The slopes (modulus of elasticity) for the concrete with 5 sk/cu yd were steeper than for the concrete with 4 sk/cu yd.

The results for bag-cured specimens were approximately as expected. However, in regard to air-cured specimens, the tensile strength of the 5-sk, 6 percent air specimen was high, and the 5-sk, 2 percent air specimen was low. The 4-sk tests resulted in lower values than the 5-sk tests, and the higher air contents resulted in lower values of modulus of elasticity. Figures 9 and 10 also show that the residual stresses of the bag-cured specimens tended to be compressive, whereas the residual stresses of the air and oven-cured specimens tended to be tensile.

One other factor, (Fig. 9) was the failure of the bag-cured specimen in bond rather than tension. This probably suggests that bond strength was reduced as a result of an environment such as bag curing which reduces the tendency for shrinkage to occur. This aspect merits further investigation.

The restraint given to the concrete volume changes by the bar in the direct tensile test is quite similar to that of a reinforcing bar in concrete pavements. The percentage of reinforcement is different, however. Deep concrete which has no volume change except near the surface where environmental conditions vary has this same type of restraint. The difference between the ultimate tensile strength (f_t) and the restrained concrete stress (σ_{cz}) represents the usable tensile strength (f_{tu}). Figures 9, 10 and 11 show that the usable tensile strength was greatly affected by oven curing.

Effect of Curing Condition on Split Cylinder Strength

From the results presented thus far, bag curing provided a more favorable environment for hydration, resulting in higher tensile strengths. Figure 12 is a plot of air-cured and oven-cured split-cylinder strengths vs bag-cured split-cylinder strengths. Bag curing is used here as a base and the line of equality represents equal air-cured and oven-cured strengths with companion bag-cured strengths. A least squares fit of the first order to data points was made with a computer for the two curing conditions. The air-cured specimens exhibited only about 91 percent of the split-cylinder strengths

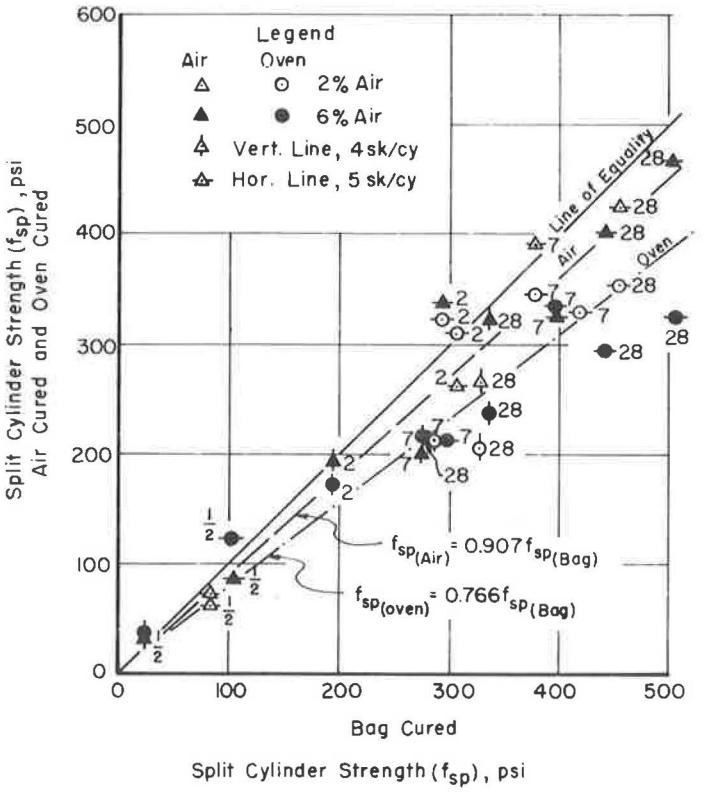

Figure 12. Effect of curing conditions on split cylinder strengths.

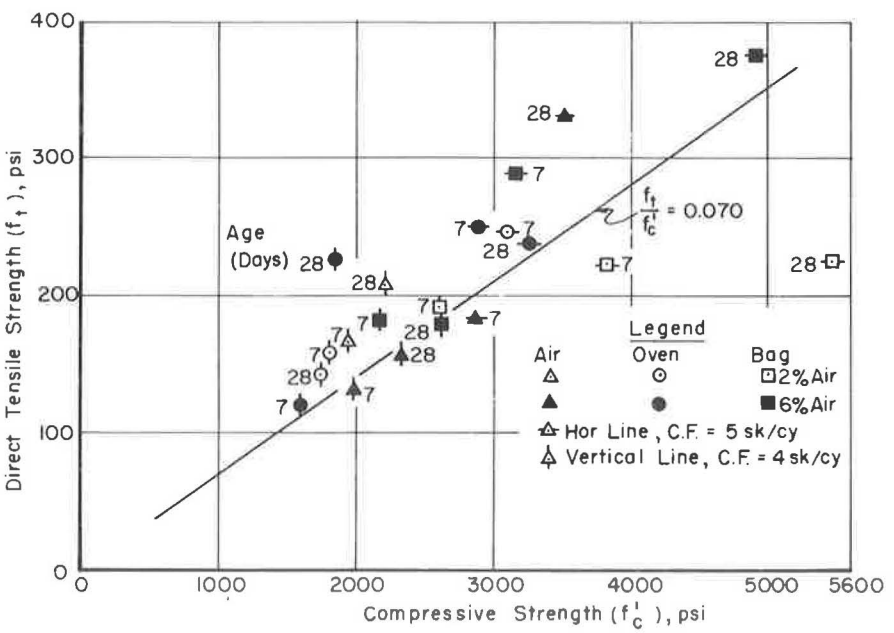

Figure 13. Relationship between direct tensile strength (f_t) and compressive strength (f'_c).

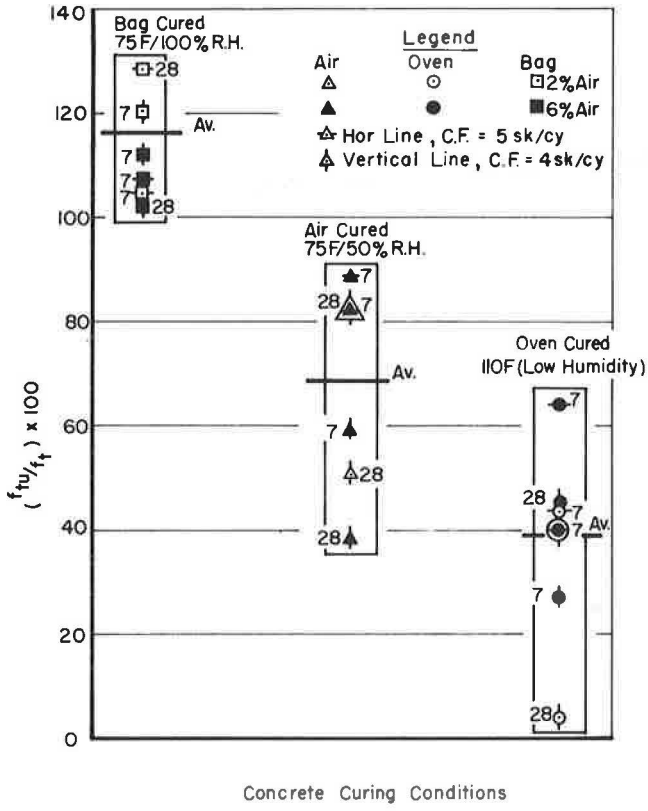

Figure 14. Percent of total available tensile strength for restrained structural lightweight concrete as a function of curing conditions.

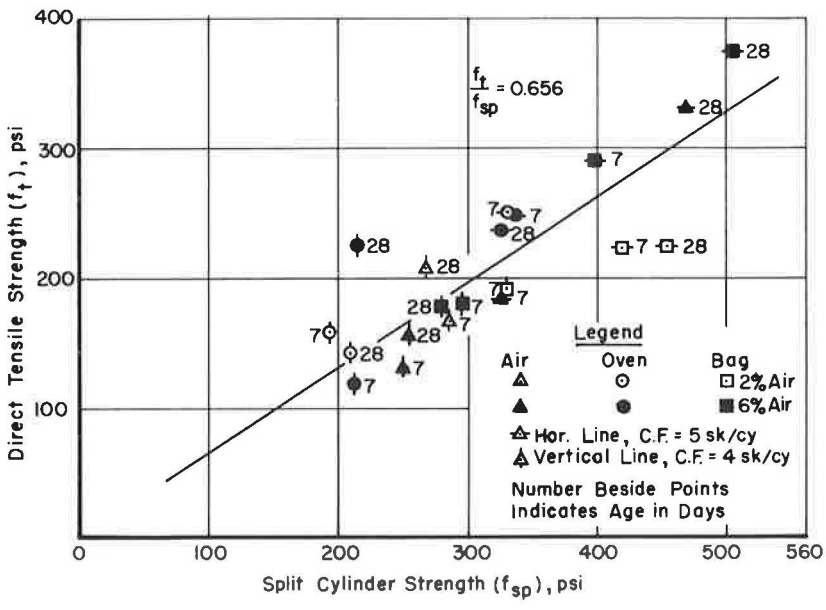

Figure 15. Relationship between direct tensile and split-cylinder strengths.

for the bag-cured specimens. The oven-cured strengths were only about 77 percent of the bag-cured strength.

Relationship Between Direct Tensile and Compressive Strengths

Figure 13 shows a plot of all of the compressive-strength and companion direct tensile-strength data collected in this investigation. Some of the plotted points were average values from two or more tests. Even though some scatter in the data exists, particularly for the bag-cured, 2 percent air-content specimens, a definite straightline relationship was determined between the direct tensile strength and compressive strength. The tensile strength was approximately 7.0 percent of the compressive strength for all curing conditions and strengths investigated. A wide range of strengths is shown in Figure 13, which helps to validate the results.

As discussed earlier, improper curing procedures cause a significant loss in tensile capacity of structural lightweight concrete that is restrained from undergoing volume changes by reinforcement. To demonstrate this, in Figure 14 the percent of usable tensile strength is plotted against type of curing, from the results given in Table 1. The amount of loss in available or usable tensile strength between the bag-cured and either the air- or oven-cured specimens is significant. The following relationship between usable tensile strength (f_{tu}) and compressive strength was determined to exist for data collected:

$$f_{tu} = 0.08\, f'_c - 64 \tag{9}$$

It should be emphasized that this is not a fixed relationship, because of the variable environmental conditions.

Relationship Between Direct Tensile and Split-Cylinder Strengths

The relationship between direct tensile strength and split-cylinder strength is shown in Figure 15. Here again, a very good straightline relationship exists. The split-cylinder should be more than the direct tensile strength, mainly because of the difference in the stress conditions of the two tests. With the aid of Figure 15, the direct tensile strength can be approximated from

$$f_t = 0.07\, f'_c = 0.66\, f_{sp} \tag{10}$$

In cases where the tensile strength of the concrete is used and where restraint to concrete volume change exists, the usable tensile strength should be a more accurate design value than tensile strength, flexural strength, or a percentage of the compressive strength. Figure 16 is a plot of usable tensile strength vs the split cylinder strength for all three environmental conditions. The general trend as determined from a computer analysis for the three environmental conditions indicates that there is no fixed relationship between usable tensile strength and split-cylinder strength. For comparison purposes, the relationship for all data, although not a firm relationship, was computed to be

$$f_{tu} = 1.03\, f_{sp} - 168 \tag{11}$$

Thus, a high value of the usable tensile strength can not be assumed without proper curing and close control. The general trends for compressive strength (Eq. 9) and split cylinder (Eq. 10) are shown in Eq. 12:

$$f_{tu} = 0.08\, f'_c - 64 = 1.03\, f_{sp} - 168 \tag{12}$$

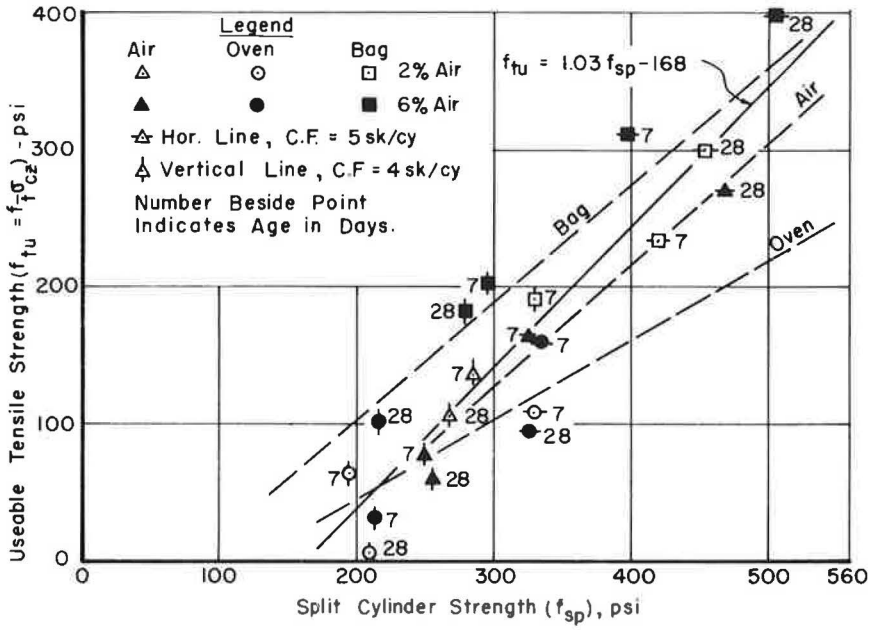

Figure 16. Relationships between usable tensile strength and split-cylinder strengths.

TABLE 2

COMPARISON OF COMPRESSIVE AND TENSILE MODULUS OF ELASTICITY

Curing	Cement Factor (sk/cu yd)	Percent	Age (days)	Modulus of Elasticity		Difference Based on Comp. (%)
				Compressive	Tensile	
Air	4	2	7	2.33	2.42	+ 3.9
Bag			28	2.81	2.74	- 2.5
Oven			7	1.80	2.00	+11.1
Air	4	6	7	2.59	1.86	-28.2
Air			28	2.21	2.12	- 4.1
Bag			7	2.43	2.25	- 7.4
Bag			28	2.60	2.70	+ 3.8
Oven			7	1.55	1.62	+ 4.5
Oven			28	1.83	1.33	-27.3
Bag			7	2.64	2.92	+10.6
Bag	5	2	28	3.09	2.52	-18.4
Oven			7	2.32	2.14	- 7.7
Air	5	6	7	2.62	2.38	- 9.2
Air			28	2.46	2.45	- 0.4
Bag			7	2.81	2.29	-18.5
Bag			28	3.29	3.36	+ 2.1
Oven			7	2.64	2.15	-18.6
Oven			28	2.55	1.93	-24.3
					Avg.	- 7.3

STATIC MODULUS OF ELASTICITY

A summary of all modulus of elasticity data is given in Table 2. Values given are secant modulii at 50 percent of the ultimate strength in compression or 50 percent of the ultimate tensile strength.

From these data it is apparent that the curing environment of structural lightweight concrete has some effect on the modulus of elasticity. The oven-cured specimens had noticeably lower values than those which were air or bag cured. This was especially true for the mixes made with a cement factor of 4 sk/cu yd.

A comparison of the values of modulus of elasticity obtained from tension and compression tests on companion specimens is given in Table 2. In general, the values in tension and compression may be considered as the same since the average difference was less than 10 percent. However, the tensile values of E tend to be slightly less than the compressive values.

According to Pauw (11), the modulus of elasticity of either regular-weight concrete or structural lightweight concrete may be approximated by

$$E = 33.6\, w^{3/2} \sqrt{f'_c} \tag{13}$$

where

w = unit weight of concrete in pcf at time of test, and
f'_c = compressive strength of concrete in psi.

This formula was used to calculate the modulus of elasticity of various specimens for which the unit weights and compressive strengths were recorded in this investigation. The calculated values were compared with the measured values and were approximately 14 percent lower. The data were then plotted similar to Pauw (11) and fitted with a straight line through the origin, using a least-squares technique. This straight-line fit is shown in Figure 17.

From the data obtained in this investigation, the modulus of elasticity of structural lightweight concrete can be better approximated by

$$E = 37.6\, w^{3/2} \sqrt{f'_c} \tag{14}$$

It should be recognized that in determining the slope mathematically, the line was forced to go through zero.

LIGHTWEIGHT CONCRETE DESIGN AND PAVEMENT PERFORMANCE

General

With the properties of structural lightweight concrete determined in this study, it is possible to analyze the design and performance of a concrete pavement structure built with this material. Inasmuch as only one lightweight aggregate was used, the results of this analysis are limited and generalizations involving all lightweight concrete cannot be made. However, test methods and procedures have been developed and analyzed to determine critical properties of structural concrete made with any material and to analyze their effects on pavement structure performance.

The following examines the various properties determined in this study in terms of a comparison of the effects of lightweight concrete properties and regular-weight concrete properties on pavement design and performance.

Concrete Pavement Design Formulations

The present-day design formulations include a design determination of (a) concrete thickness, (b) contraction joint spacing, (c) distributed steel requirements, and (d) continuous reinforcement.

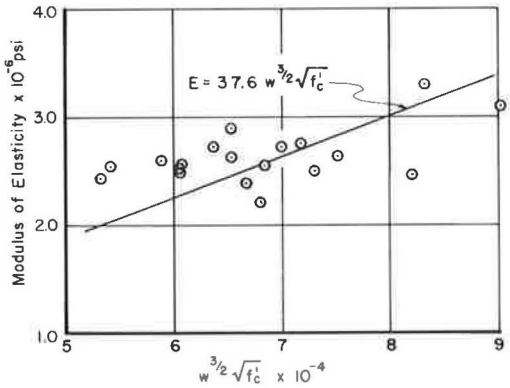

Figure 17. Relationship between secant modulus of elasticity and compressive strength and unit weight of concrete.

Concrete Thickness. — The design formulation of the thickness of concrete pavement was one of the major results of the AASHO Road Test (12). Combining the results of the road test with previous research studies and experience, AASHO published an Interim Guide for the design of rigid pavement structures (13), which was extended by the Texas Highway Department to cover the various types of concrete pavement constructed in Texas (14).

From this development, the following pavement-thickness design equation was developed:

$$\log \Sigma L = -8.8682 - 3.513 \log \left[\frac{J}{S_c D^2} \left(1 - \frac{2.61\, a}{Z^{0.25}\, D^{0.75}} \right) \right] + 0.9155 \frac{G}{\beta} \qquad (15)$$

where

ΣL = number of accumulated equivalent 18-kip single axle loads;
J = a coefficient dependent on load transfer characteristics for slab continuity;
S_c = modulus of rupture (flexural strength) of concrete at 28 days (psi);
D = nominal thickness of concrete pavement (in.);
$Z = \frac{E_c}{k}$;
E_c = modulus of elasticity for concrete (psi);
k = modulus of subgrade reaction (psi/in.);
a = radius of equivalent loaded area = 7.15 in. for road test 18-kip axles;
$G = \frac{4.5 - P_t}{3}$;
P_t = servicability at end of time, t; and
$\beta = 1 + \frac{(1.624)(10^7)}{(D+1)^{8.46}}$.

At first glance, Eq. 15 appears rather formidable and cumbersome to solve for the design pavement thickness (D). However, it was a simple matter to program the equation on the computer and solve for (ΣL) for all combinations of variables, and present the results in the form of a design nomograph (14). It is relatively easy to enter the nomograph with the design parameters and concrete properties involved and arrive at a design pavement thickness.

The comparisons of Eq. 15 for selected example parameters are shown in Figure 18 for continuously-reinforced concrete pavement (CPCR). Similar comparisons for jointed concrete pavement (CPJ) are given in Figure 19. In all cases the required thickness of structural lightweight concrete was less than the required thickness for regular-weight concrete. As the example strengths were the same for the two types of concrete, the difference in design thickness must be due to the lower modulus of elasticity for structural lightweight concrete. This agrees with the theoretical development of slabs on elastic foundations by Westergaard (15). The lower the modulus of elasticity, the lower the tensile stresses in the concrete, and hence the thinner the pavement must be to withstand the applied traffic load. Structural lightweight concrete, therefore, with its lower modulus of elasticity, offers an advantage over regular-weight concrete, in that less material is required to carry the load.

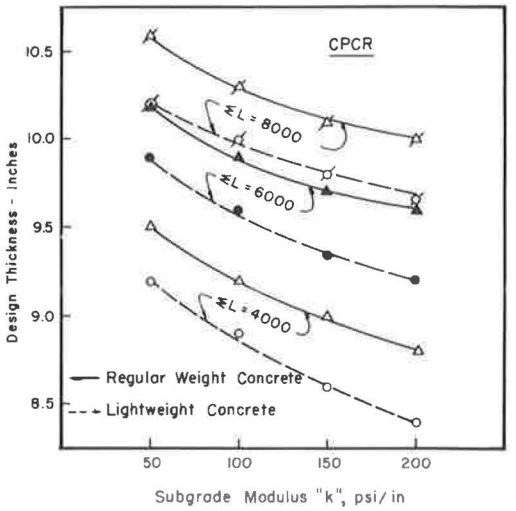

Figure 18. Design thickness for CPCR of regular and lightweight concrete as a function of load application and subgrade modulus.

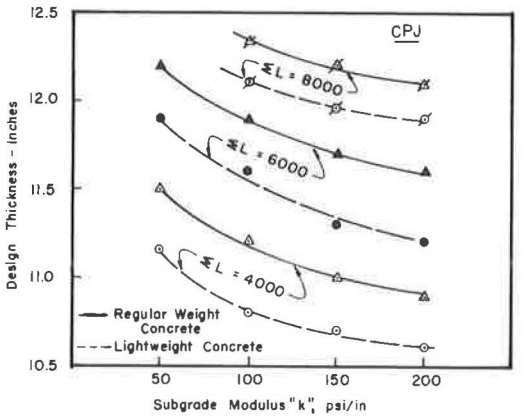

Figure 19. Design thickness for CPJ of regular and lightweight concrete as a function of load application and subgrade modulus.

Contraction Joint Spacing.— Jointed unreinforced concrete pavement requires transverse contraction joints spaced along the pavement length. Unfortunately, there are no design formulations to determine the required joint spacing (16), and, therefore, for the most part, past experience has been relied on to arrive at suitable designs.

The required joint spacing is strongly dependent on the volume changes of the concrete, which is a critical property and is subsequently discussed.

Distributed Steel.— The amount of distributed steel in jointed, reinforced concrete pavement is directly dependent on the weight of the slab, all other factors being equal. Inherent in this concept is the assumption that the concrete is sufficiently strong to support the load. If it is reasonable to assume that the friction factor between the pavement and the subgrade is practically the same for each type of concrete, the regular-weight concrete slab would require 30 percent more distributed steel per foot width of slab than structural lightweight concrete. Therefore, as with the required thickness, less material is required for structural lightweight concrete.

Of course, another important assumption inherent in using this concept for lightweight concrete is that the lightweight concrete volume changes are no greater than regular-weight concrete volume changes.

Continuous Reinforcement.— In the design of continuously-reinforced concrete pavement without transverse joints, enough steel is placed in the slab to force the concrete to develop numerous transverse, hairline cracks. The steel does not prevent cracking; on the contrary, it induces cracking. However, it keeps the cracks tightly closed. This type of pavement has been used extensively in Texas, as well as in many other states, with excellent results. The basic design equation of the steel percentage is given by McCullough and Ledbetter (17). This basic equation has been modified to include a term for the subbase friction factor, F, for inclusion in the AASHO Interim Guide (13).

Using this modified equation to compare the relative amounts of steel required for regular-weight and lightweight concrete pavement, the only difference between the two types of concrete which affects the formula is the modulus of elasticity. Solving the equations using typical example values for the two types of concrete results in slightly more steel being required for lightweight concrete (0.56 percent) than for regular-weight concrete (0.54 percent).

Concrete Pavement Performance

Comparisons have been made between regular-weight and lightweight concrete pavement structures based on existing pavement design formulations. As is the case with

almost all design procedures, the capability of a given product to meet design requirements does not always insure performance in service. Many material properties considered critical to the performance of the concrete pavement structure are not considered directly in the design.

Concrete Warping Stresses. — The effects of concrete volume changes with changes in temperature is an important consideration when evaluating a pavement material which will be subjected to rather severe temperature changes and differentials between the top and bottom of the slab. The theoretical warping stresses developed from temperature differentials was formulated by Bradbury (18).

$$\sigma_t = \frac{C E_c \epsilon T}{2} \tag{16}$$

where

σ_t = maximum stress (psi) in extreme fiber at edge of slab, in direction of slab length;
C = coefficient, directly proportional to slab length;
ϵ = coefficient of thermal expansion (in./in./OF), and
T = difference in temperature between the top and bottom of slab.

The value for the warping stress is directly proportional to the product of coefficient of thermal expansion, ϵ, and the modulus of elasticity E_c. Using typical properties for pavement slabs of the same length and thickness, regular-weight concrete would contain 114 percent more stress due to warping than structural lightweight concrete. This means that the distance between transverse contraction joints, which is reflected in the coefficient C, can be increased on pavements constructed with structural lightweight concrete, and thereby effect a savings in construction costs. This also means that structural lightweight concrete undergoes less volume change from changes in temperature and therefore is more dimensionally stable over long periods of time. This points to an expected increase in the performance capability of structural lightweight concrete in terms of warping stresses.

Volume Changes—Moisture.—Volume changes from changes in moisture content of the concrete constitute another important property seriously affecting pavement structure performance. The restrained concrete volume change stresses can be almost eliminated during the critical early life of the concrete by proper curing. However, before this factor can be fully appraised, additional research is needed on lightweight concretes made with other aggregates and with curing conditions more nearly approximating current field curing practices. The results of this study indicate that restrained lightweight concrete volume changes could be very serious and should be investigated further. Pavement structures constructed with lightweight concrete should be watched closely for any performance effects which may result from restrained volume changes.

CONCLUSIONS

The following conclusions appear valid for the parameters studied in this investigation.

1. The correlation of flexural strengths with either compressive strengths or split-cylinder strengths was poor, which further indicated the difficulty of using flexure as an index of strength and more particularly of tensile strength. A comparison of bag-cured lightweight concrete with the bag-cured regular weight concrete showed that their compressive, direct tensile, and split-cylinder strengths were practically identical; but the flexural strength of the lightweight concrete was often 20 to 25 percent lower than the flexural strength of the regular-weight concrete. The obvious conclusion is that flexural strength test is a poor indicator, often unduly restrictive and incorrect, of lightweight concrete strength and quality.

2. The split-cylinder strengths for the three different environments resulted in the air-environment specimens testing at approximately 91 percent of the bag environment

and the oven-environment specimens testing at approximately 77 percent of the bag environment. This test shows considerable promise as an indicator of concrete strength and quality.

3. This investigation has produced a procedure which not only accurately determines the tensile strength of concrete but also provides a measure of residual tensile stress developed by volume change. This procedure can also be used to establish the influence of different aggregates, mix designs, admixtures, environments, etc., on tensile strength and the development of residual stress from volume changes.

4. When the lightweight aggregate concrete used in this study was restrained by a reinforcing bar, it developed either compressive or tensile residual stresses depending on the curing environment. Residual compressive stresses as high as 80 psi were developed with a bag environment, whereas residual tensile stresses as high as 140 psi were developed in an oven environment. The intermediate environment in air resulted in residual tensile stresses as much as 100 psi.

5. For this lightweight aggregate concrete the (direct) tensile strength was related to the split cylinder strength and compressive strength as $f_t = 0.66 f_s = 0.07 f'_c$.

6. The usable tensile strength of this lightweight aggregate concrete was significantly reduced under unfavorable environmental conditions during the curing period. Further, neither the split-cylinder strength test nor the compression strength test indicated the usable tensile strength of the reinforced concrete.

7. Since the expanded shale used in this investigation is a low absorption aggregate, it is anticipated that aggregates with higher absorptions may result in even lower usable tensile strengths under restrained volume change and unfavorable environmental conditions.

8. For all practical purposes, the modulus of elasticity for both tension and compression for concrete made with this particular expanded shale coarse aggregate was the same except for several values which were affected by experimental inconsistencies in curing.

9. For the tests made in this investigation, the relationship of modulus of elasticity to unit weight and compressive strength was found to be $E = 37.6\, w^{3/2} f'^{1/2}_c$. This relationship is reasonably close to Pauw's (19) value of $E = 33.6\, w^{3/2} f'^{1/2}_c$.

10. Using the values obtained in this study for the properties investigated, the following design comparisons can be made for the various types of concrete pavements: (a) the required pavement thickness for lightweight concrete is around 0.3 in. less than regular-weight concrete of the same strength; (b) for jointed reinforced concrete pavement, regular-weight concrete requires 30 percent more distributed steel than lightweight concrete of the same joint spacing; and (c) for continuously-reinforced concrete pavement, lightweight concrete requires slightly more steel (0.56 percent) than regular-weight concrete (0.54 percent).

11. In evaluating the expected pavement performance the following comparisons can be made: (a) for given slab dimensions, concrete warping stresses due to temperature differentials between the top and bottom of the slab for regular-weight concrete will be 114 percent greater than for lightweight concrete; and (b) volume change of lightweight concrete, if unfavorably cured, can result in sizable residual stresses in the concrete; and if restrained, as they would be in a concrete pavement, these volume changes could be extremely detrimental to the performance of the pavement structure.

RECOMMENDATIONS

1. Since the technique developed in this investigation provides a measure of both the tensile strength and usable tensile strength which might occur under a restrained volume change condition, it is recommended that the effects of the following be investigated: (a) properties of aggregates—particularly absorption, (b) mix design, (c) limiting and practicable curing environment, (d) use of molecular films to reduce evaporation, and (e) percentages of steel.

2. The effects of curing environment on bond strength should be investigated for at least two lightweight aggregate types: one with a relatively high absorption capacity, and one with a relatively low absorption capacity.

3. Dynamic tensile properties should be investigated for a structural lightweight concrete.

4. Test sections of structural lightweight concrete pavement should be constructed and evaluated over a period of time to verify the laboratory conclusions reached in this study.

ACKNOWLEDGMENTS

The authors are grateful to the Texas Highway Department and the U. S. Bureau of Public Roads for their sponsorship of this research.

REFERENCES

1. Ledbetter, W. B., and Thompson, J. Neils. Relationship Between Critical Mechanical Properties and Age for Structural Lightweight Concrete. Univ. of Texas, Center for Highway Res., Feb. 1964.
2. Houston, J. T., and Thompson, J. Neils. Volume Changes in Unrestrained Structural Lightweight Concrete. Univ. of Texas, Center for Highway Res., to be publ.
3. Ledbetter, W. B., Perry, Ervin S., Houston, James T. and Thompson, J. Neils. Critical Mechanical Properties of Structural Lightweight Concrete and The Effects of These Properties on the Design of the Pavement Structure. Univ. of Texas, Center for Highway Res., to be publ.
4. Shideler, J. J. Lightweight Aggregate Concrete for Structural Use. Proc. Jour. of Amer. Concrete Inst., Vol. 54, Oct. 1957.
5. Shideler, J. J. Manufacture and Use of Lightweight Aggregate for Structural Concrete. PCA Dev. Dept. Bull. D40, Jan. 1961.
6. Troxell, G. E., and Davis, H. E. Composition and Properties of Concrete. New York, McGraw-Hill, 1956.
7. Todd, J. D. The Determination of Tensile Stress-Strain Curves for Concrete. Proc. Inst. of Civ. Eng. (England), Vol. 4, No. 2, Pt. 1, Paper No. 6012, Mar. 1955.
8. Hinson, A. J. Testing Mechanism for Concrete in Pure Tension. Univ. of Texas, Unpubl. rept. 1955.
9. Rane, V. S. Shrinkage Effect on Diagonal Tension Strength of Reinforced Concrete Beams. Univ. of Texas, Master's thesis, Jan. 1961.
10. Washa, George W., and Fluck, Paul G. Extent of Sustained Loading on Compressive Strength and Modulus of Elasticity of Concrete. Amer. Concrete Inst., May 1950.
11. Pauw, Adrain. Static Modulus of Elasticity of Concrete as Effected By Density. Proc. Jour. of Amer. Concrete Inst., Vol. 57, No. 6, Dec. 1960.
12. The AASHO Road Test Report 5, Pavement Research. Highway Research Board Spec. Rept. 61E, 1962.
13. AASHO Interim Guide for the Design of Rigid Pavement Structures. AASHO Committee on Design, Apr. 1962.
14. Hudson, W. R., and McCullough, B. F. An Extension of Rigid Pavement Design Methods. Texas Highway Dept. Departmental Research Rept., No. 64-1, 1964.
15. Westergaard, H. M. Stresses in Concrete Pavements Computed by Theoretical Analysis. Public Roads, Vol. 27, No. 2, Apr. 1926.
16. Yoder, E. J. Principles of Pavement Design. New York, John Wiley & Sons, 1959.
17. McCullough, B. F., and Ledbetter, W. B. LTS Design of Continuously Reinforced Concrete Pavement. Transactions of ASCE, Vol. 127, Pt. 4, 1962.
18. Bradbury, Royall D. Reinforced Concrete Pavements. Wire Reinforcement Inst.,Washington, D. C., 1938.