Time Deformation Studies on Two Expanded Shale Concretes

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This report describes and presents results of tests on concretes produced with lightweight aggregates that are available in quantity in Oregon. The principal aggregates discussed are expanded shales produced by two local firms. The studies consisted of the development of lightweight concrete mixes having suitable workability and strength for use in prestressed members and the measurement of creep and shrinkage in concrete prisms cast from selected mixes. The time deformation studies were conducted on prisms 5 in. by 6 in. by 107 in., with gage points 100 in. center to center. A stress of 2,000 psi, applied at an age of 28 days, was maintained at a relatively constant value by periodic retensioning of stressing bars.

Previous studies on creep and shrinkage characteristics of common concretes and other lightweight concretes are reviewed for purpose of comparison. Values are presented for estimating the stress loss in prestressing steel used in conjunction with the expanded shale aggregates in prestressed members. By comparison with other test results, it appears that the time deformations of the expanded shale prisms are not appreciably higher than might be found in sand and gravel concrete of equal strength; however, the measured deformations greatly exceed those provided for in typical recommendations for estimating stress losses in prestressing steel.

• SINCE THE USE of prestressed concrete in highway structures has gained general acceptance in this country, an increasing interest has been directed to the practicality of using lightweight concrete in the construction of prestressed bridge members. Within a limited range of sizes, the use of lightweight concrete permits members to be cast on the ground and erected with mobile equipment where the additional weight of conventional concrete members prohibits their erection in this manner. The saving in construction costs made possible by the elimination of falsework required for cast-in-place members is sometimes appreciable. The reduction in weight, which might be as much as 30 percent, can also be expected to yield savings in that larger members can be cast in a central yard before being hauled to the site. Furthermore, any appreciable reduction in superstructure weight is reflected favorably in the cost of piers and footings. Because of inadequate knowledge of the physical properties, particularly with respect to creep and shrinkage, the use of lightweight concrete in prestressed members has not received the acceptance its weight reduction would justify.

A research project was initiated in 1955 by the Oregon State Highway Department to gather information on the physical characteristics of concrete made from locally available lightweight aggregates. The two principal aspects of this project were (a) development of lightweight concrete mixes having suitable workability and strength for use in prestressed concrete members, and (b) the study of creep and shrinkage in concrete prisms cast from selected mixes developed under the first phase. The latter phase has not been concluded, but the measurements presently available provide sufficient information to warrant reporting the results.

AGGREGATES

Lightweight aggregates available in quantity in Oregon consist of volcanic cinders, pumice, and expanded shales produced by two local firms. To distinguish between them, the expanded shales will be designated as Aggregate A and Aggregate B.

Both of the expanded shales are the product of crushing and burning Keasey shales, mined in Oregon. In each case, the burning is accomplished in a rotary horizontal kiln at temperatures in the vicinity of 2,000 F. Before the burning process, Aggregate A is crushed to a maximum particle size of 2 in. and the material under $\frac{1}{8}$ in. is removed by screening. The resulting 2-in. to $\frac{1}{8}$-in. material is burned and then crushed and screened into the desired sizes; usually $\frac{3}{8}$ in. to $\frac{3}{16}$ in. and $\frac{3}{16}$ in. to 0, although a third size, $\frac{3}{4}$ in. to $\frac{3}{8}$ in., is sometimes used. Aggregate B is handled in a similar manner except that the maximum size before burning is 3 in. and the fine material is not removed. A sieve analysis for the $\frac{3}{8}$-in. to $\frac{3}{16}$-in. and $\frac{3}{16}$-in. to 0 sizes of the two expanded shale aggregates used in time deformation studies is shown in Table 1.

TABLE 1

SIEVE ANALYSIS OF EXPANDED SHALE AGGREGATES

	Percent by Weight Retained on Sieve			
	Aggregate A		Aggregate B	
Sieve No.	Fine	Coarse	Fine	Coarse
$\frac{3}{8}$"		0		0
$\frac{1}{4}$"		69		15
4	0	95	0	52
8	9	96	32	89
16	25	97	60	91
30	40	97	69	91
50	53	97	74	91
100	63	100	79	92
Fineness modulus	1.89	5.81	3.14	5.06
Unit wt (lb per cu ft)	50.4	37.8	56.0	43.8

MIX DEVELOPMENTS

Concrete mixes made with each of the expanded shales, with cinders, and with a combination of cinders and pumice were cast into standard cylinders and tested at 28 and 60 days to determine the ultimate compressive strengths of the concrete. In addition to ultimate strength, the various mixes were judged for workability, and the slump and wet weights were measured. Variations were made in proportioning the aggregates as to gradation, and the cement content was varied between values of 6 and 9 sacks per cu yd for the different mixes. Six cylinders were cast from each of the trial mixes. The cylinders were cured in a moist room until three of each were tested at 28 and 60 days.

In designing the mixes, a minimum 28-day strength of 4,500 psi was sought because, although there is no general agreement, this seems to be near a minimum value for the economical construction of prestressed members. All concrete for the project was mixed in a Lancaster laboratory mixer in 1- to $1\frac{1}{4}$-cu ft batches.

Several trial mixes were produced from volcanic cinders taken from natural deposits in central Oregon. This material is a very sharp, granular aggregate, and the resulting concrete was considered too harsh for use in prestressed members. The weight of concrete produced from cinders was higher than was desirable—about 122 pcf wet. Because of these characteristics, mixes with a full range of cement contents were not made. The 28-day strength of cylinders from mixes having a cement factor near 7 sacks per cu yd was about 3,000 psi.

To reduce the weight and improve the workability, pumice was combined with cinders to form an aggregate of 51.5 percent pumice and 48.5 percent cinders by weight. In mixes using this combination, the cinders were used as a coarse aggregate with particle sizes ranging from $\frac{1}{2}$ in. to $\frac{3}{16}$ in. and pumice comprised the fine aggregate with particle sizes from $\frac{3}{16}$ in. to 0. The wet weight of concrete produced from this aggregate combination was reduced from 122 pcf for cinders alone to 101 pcf. Although some improvement was realized in the workability, the concrete produced was considered too harsh for use in bridge members. All of the mixes using the combined aggregates fell far short of meeting the minimum strength requirement of 4,500 psi at 28 days. Since the workability and strength characteristics of these trial mixes were not satisfactory, no time deformation studies were conducted on them.

The initial trial mixes with Aggregate A were made with two sizes of aggregate: ¾ in. to ⅜ in. and ⅜ in. to 0. Other mixes were made with only the ⅜-in. to 0 aggregate. For later mixes, the ⅜-in. to 0 aggregate was furnished by the producer in two sizes; ⅜ in. to $\frac{3}{16}$ in. and $\frac{3}{16}$ in. to 0. The concrete resulting from all of these combinations seemed slightly harsh. To improve the workability, 2 cu ft natural sand per cubic yard of concrete were combined with the ⅜-in. to $\frac{3}{16}$-in. and the $\frac{3}{16}$-in. to 0 aggregates. Using these components, a 9-sack mix having excellent workability and adequate ultimate strength was selected for a study of time deformation characteristics. Details of this mix for two slightly different cement factors and water-cement ratios are shown in Table 2.

The most extensive experimentation with the various aggregates was with Aggregate B for which 16 trial mixes were made. Several mixes developed by an independent testing laboratory were obtained and used as a basis for the earlier mixes developed in connection with this project. The aggregate was furnished in three sizes: ¾ in. to ⅜ in., ⅜ in. to $\frac{3}{16}$ in., and $\frac{3}{16}$ in. to 0. In most of the trial mixes all three sizes, in varying proportions, were used. For four of the mixes only the ⅜-in. to $\frac{3}{16}$-in. and the $\frac{3}{16}$-in. to 0 sizes were used because the producers of the expanded shales have discontinued production of the ¾ in. to ⅜ in. except for special orders. The unit weights of concrete made with and without ¾-in. to ⅜-in. aggregate were approximately the same and the omission of this size generally produced a stronger and more workable concrete. A reduction in the proportion of the $\frac{3}{16}$-in. to 0 size produced a lighter, stronger concrete for a given cement factor, but the mixes were considered too harsh to be practical for prestressed members. Several mixes in the 8 sacks per cu yd range yielded concrete meeting the selected minimum value of 4,500 psi for 28-day ultimate strength, but to be sure of sufficient strength, the 9-sack mix shown for Aggregate B (Table 2) was chosen for use in the time deformation studies.

Because the absorption of lightweight aggregates fluctuates widely, depending on the porosity and the moisture content of the particles, it is not satisfactory to design a mix with a rigid water-cement ratio. A more satisfactory design results from a specified slump. For comparable workability, the lightweight concretes will have appreciably less slump than corresponding sand and gravel concretes. It was found that expanded shale concretes having a slump of 1½ in. were easily placed and tended to flow readily under the action of a vibrator. Although no direct comparisons were made, it was estimated that the consistency of this

TABLE 2

MIX DESIGNS CHOSEN FOR TIME DEFORMATION STUDIES

Property	Aggregate A		Aggregate B
Prism numbers	1, 2	3, 4	5, 6
Coarse aggregate, ⅜"-$\frac{3}{16}$" lb per cu yd	311	283	366
Coarse aggregate, cu ft per cu yd	8.2	8.5	8.3
Fine aggregate, $\frac{3}{16}$"-0 lb per cu yd	960	986	1,213
Fine aggregate, cu ft per cu yd	19.0	19.6	21.7
Natural sand, lb per cu yd	186	159	0
Natural sand, cu ft per cu yd	2.0	2.0	0
Cement factor, sacks per cu yd	8.6	8.8	9.0
Dispersing agent, lb per sack	0.25	0.25	0.25
Water/cement ratio, gal per sack	7.3	7.1	6.2
Wet weight, lb per cu ft	102	103	107
Slump, inches, avg	2½	1⅜	1½
28-day compressive strength, psi	4,450	5,078	5,690
Modulus of elasticity, psi	1,722,000	2,014,000	2,069,000

concrete was similar to sand and gravel concrete having a slump of 3 to 4 in.

Because of the porosity of the lightweight aggregates, it was found that a better mix resulted when 1/3 to 1/2 of the mixing water was added to the aggregates before the addition of cement and dispersing agent. This procedure provides assurance that water required for proper cementatious action will not be drawn into the pores of the aggregate. It also eliminates a rapid drying out and corresponding reduction in workability immediately after mixing.

MODULUS OF ELASTICITY

Static modulus of elasticity was determined for most of the cylinders cast from expanded shales. Axial strains were measured by means of a strainometer secured to the concrete specimen. The secant modulus of elasticity, taken at 2,000 psi, was determined from these measurements. Values of modulus of elasticity of Aggregate A are shown in Figure 1. The equation of the straight line which most nearly represents the modulus of these cylinders was determined by the method of least squares to be

$$E = 405f_c' - 30{,}000 \qquad (1)$$

Within practical limits, this can be represented more simply as

$$E = 400f_c' \qquad (2)$$

All of the values (Fig. 1) were obtained from cylinders having a maximum aggregate size of 3/8 in. An earlier study of the properties of concrete made from this aggregate, in which particles with a maximum size of 3/4 in. were used, provided the equation $E = 750{,}000 + 250f_c'$.

The difference between these equations could be because of a difference in aggregate size, mixing techniques, water-cement ratios, inherent variation in the aggregate, or a combination of these factors. Although the equations are appreciably different at low and high strengths, they intersect at a compressive strength of 5,000 psi and vary less than 10 percent between 4,000 and 6,000 psi.

Figure 2 shows the modulus of elasticity versus ultimate compressive strength for concrete in which Aggregate B was used. The straight line equation that best represents the modulus of elasticity of these mixes is $E = 580{,}000 + 270f_c'$. The cylinders represented in this figure were primarily from concrete having a maximum particle size of 3/8-in., although some cylinders contained 3/4-in. aggregate. The omission of the cylinders containing 3/4-in. aggregate did not significantly change the equation for modulus of elasticity; therefore, they were included in the figure and used in calculating the equation shown.

PROCEDURE FOR TIME DEFORMATION STUDIES

To determine the time deformation in the lightweight concrete mixes that were chosen for these studies, six rectangular prisms 5 in. by 6 in. by 107 in. were cast. A flexible metal tube was cast longitudinally at the center of the cross-section to allow introduction of reinforcement for stressing. Near each end of the prisms, steel bars 9 in. long were cast perpendicular to the longitudinal axis at points 1 in. from the top and bottom faces of the beams. These bars protrude 2 in. from each side of the prism and are placed 3½ in. from each end to provide gage points 100 in. center-to-center. Four pairs of gage points, at the top and bottom of each side, are thus provided for each prism. The bars had lathe center holes drilled ½ in. from each end of each bar to receive the pointed ends of a length gage. The length gages consisted of a ½-in. diameter Invar bar with a dial indicater graduated to 0.001 in. mounted at one end and a pointed stud at the other end. The bracket holding the dial indicator to the bar was so constructed that the stem of the indicator was on the extended axis of the Invar bar. A 60-deg contact point was used on the gage stem. A bracket on the fixed end held the pointed stud in firm contact with the gage point. Two of these gages were assembled as a precaution against loss of data due to possible damage to a gage.

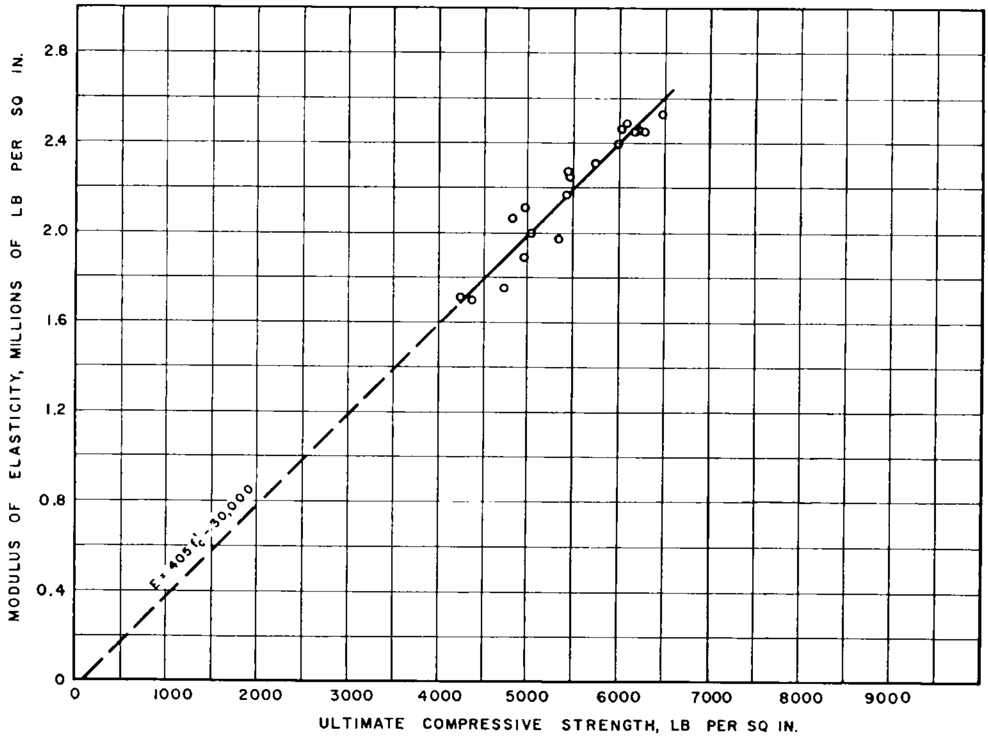

Figure 1. Expanded shale Aggregate A.

During the measurements, the gage was supported at the ends and at four intermediate points so that the deflection of the Invar bar was practically eliminated.

To minimize the floor space occupied by this project, the prisms were cast one above another in a rack. To prevent excessive resistance to shrinkage and creep the bottom form for each prism was covered with sheet aluminum which was coated with a parting compound before the prisms were cast. During the studies, the prisms remained fully supported by this bottom form (Fig. 3). To distribute the compressive force created by stressing the axial reinforcement, 1¼-in. steel plates having the same areal dimensions as the prism cross-section were placed at each end of the prisms. These plates were used to comprise the ends of the forms. After the concrete had hardened, only the side forms were removed, and the end plates became an integral part of each prism.

Initially, two prisms were cast from the Aggregate A mix shown in column 1 of Table 2. The concrete for these prisms was mixed in 1-cu ft batches in the Lancaster mixer. A slump test was performed on each batch and this, rather than a fixed water-cement ratio, was used in obtaining concrete of the desired consistency. For the first prisms, a slump of 2 to 3 in. was selected and the average measured slump for the various batches was 2½ in. The concrete was consolidated in the forms by means of hand rodding and tamping with wooden blocks. In addition to pouring the two prisms, 12 standard cylinders were cast from the same mix. The cylinders were tested for modulus of elasticity and ultimate strength — three each at 28 days, 60 days, 180 days, and 1 year. All cylinders were stored continuously in a moist curing room from the time the cylinder molds were removed until the tests were made. After the concrete had set, a wet

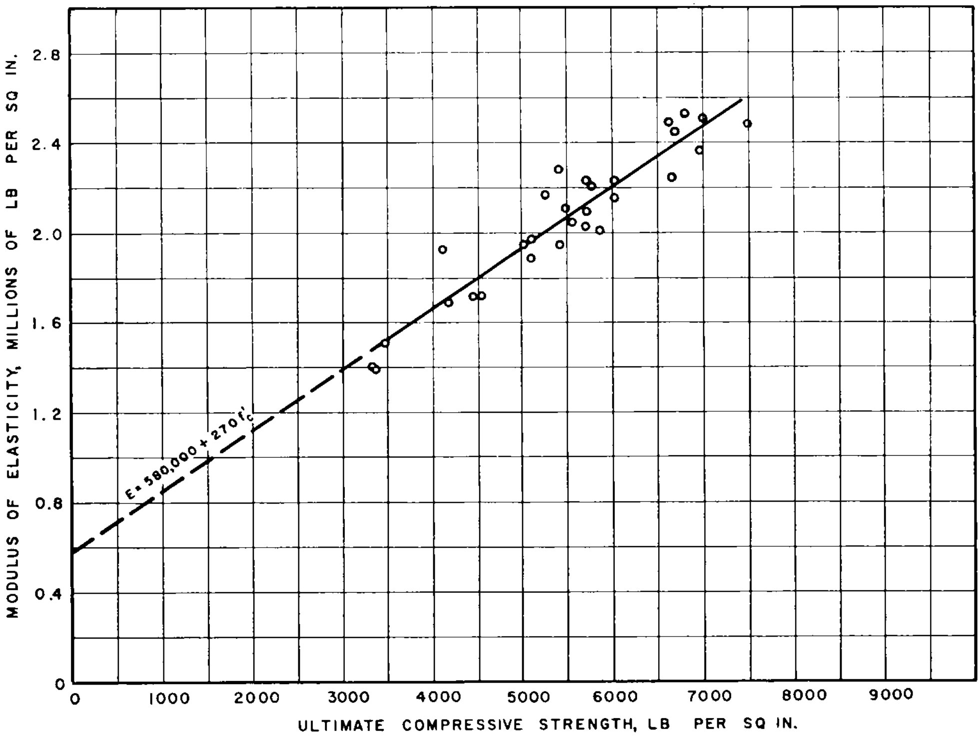

Figure 2. Expanded shale Aggregate B.

burlap covering was maintained over the prisms for about two weeks.

Approximately 24 hours after the concrete was poured the side forms were removed and initial measurements of the prism lengths were made. Measurements were always made at each of the four gage positions on each prism so that any differential shrinkage or creep could be detected. Periodic length measurements were made during a 28-day curing period to determine the shrinkage accompanying this curing. No measurable shrinkage occurred during the 2-week period that the prisms were kept moist. During the following two weeks some drying out occurred and measurable shrinkage took place. Since the prisms rested on aluminum sheets, the upper portion of each prism dried more rapidly than the lower portion. Twenty-eight days after casting the prisms, the shrinkage indicated by the average of the upper measurements was 0.00011 in. per in. and that for the lower measurements was 0.00004 in. per in. The value of these measurements might be questionable because it is not known to what degree drying out occurred. However, during the period from 14 to 28 days, free air circulation was permitted on the sides and top surface. Since the prisms had a small cross-section in comparison with typical beam dimensions, it seems likely that the 28-day shrinkage in an air cured beam would not exceed the value of 0.011 percent measured near the top of the prisms and would probably be closer to the lower value measured near the bottom. No control of temperature and relative humidity was maintained on this project; however, temperatures were recorded at the time measurements were made and relative humidities were checked occasionally and correlated approximately with average monthly relative humidities reported by the U. S. Weather Bureau. From this, the average relative humidity of the storage

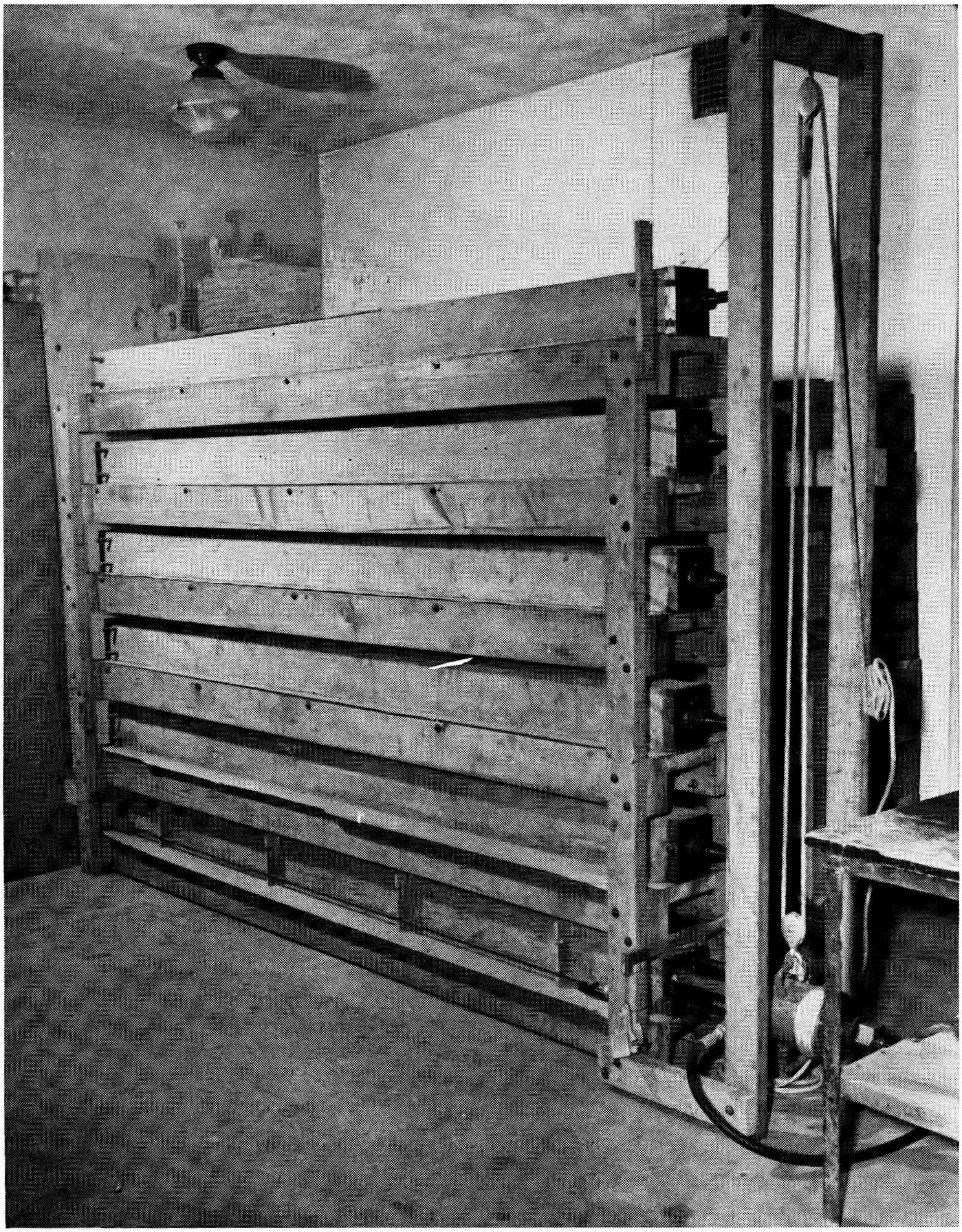

Figure 3. Lightweight prisms and supporting rack.

room was estimated to be approximately 60 percent. The temperature of the room averaged about 70 F.

To study creep in the prisms, a stress of 2,000 psi was applied and, within narrow limits, maintained. To apply the stress, a 1-in. diameter Stressteel bar was introduced at the longitudinal axis of each prism through the metal conduit cast into the prism. Stress adjustments on these bars were accomplished by adjusting the anchorage nuts on the threaded bar ends rather than by the insertion of split washers as would be required if the bar were stressed to its design capacity. The bars were tensioned by means of a calibrated, center hole ram and pump. The tensioning apparatus is shown in use in Figure 4. The bars were retensioned at intervals so that the stress in the concrete was maintained between the values of 1,900 and 2,100 psi. In practice, the stress seldom varied by more than 75 psi; from a low of 1,950 psi to a high of 2,025 psi. To determine the stress loss between times of retensioning, the anchorage nut rested against a horseshoe shaped washer which was lifted upward by a cord and weight system at the instant the total tension in the bar was taken by the jack. The reading of the hydraulic pressure gage was noted at the time of this occurrence. Additional tension was then applied to the bar to restore the stress of 2,000 psi on the concrete. At this stress, the horseshoe washer was reinserted and the anchorage nut was adjusted until firm contact with the washer was obtained. Afterwards, the pressure in the ram was released, completing the tensioning operation.

Because of the urgency of other work, the second pair of prisms was not cast until about six months after the first pair.

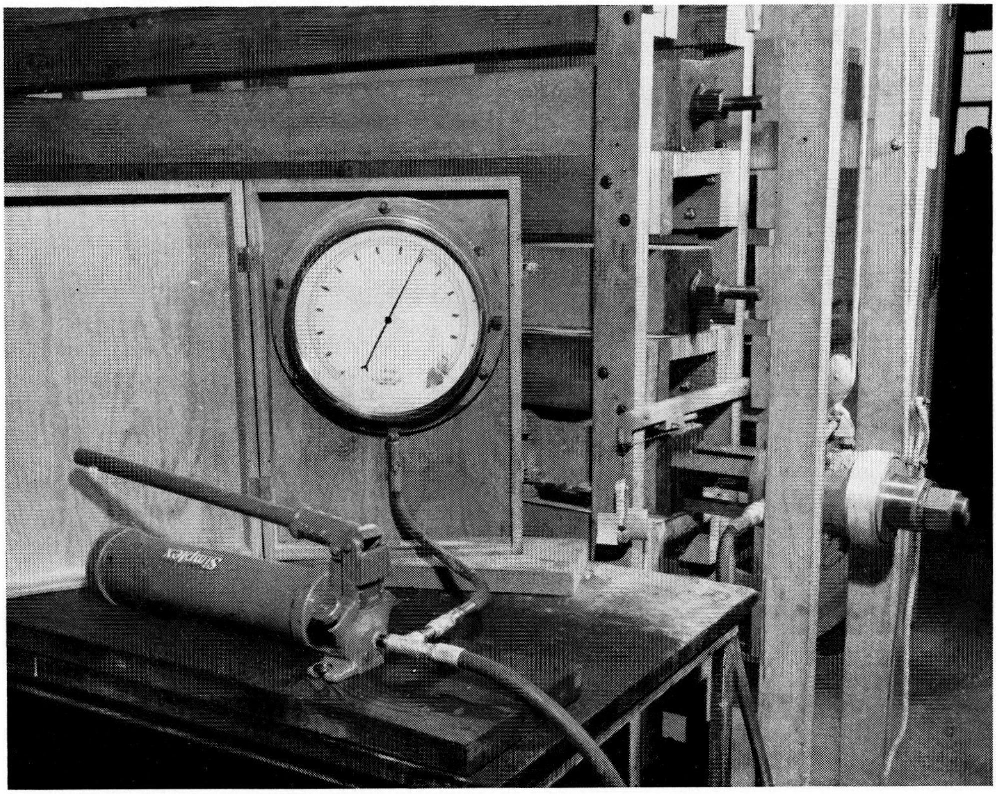

Figure 4. Tensioning apparatus in use.

During this period, an appreciable amount of deformation was measured in the original prisms. This fact was influential in the decision to cast two prisms of the same aggregate having a lesser water content and to use a mechanical, internal vibrator to consolidate the concrete in the forms. The mix shown in column 2 of Table 2 was used for these prisms. The mixing procedure for this concrete was the same as that described earlier, except that the slump of the various batches was maintained between 1 and 2 in. Twelve cylinders were cast, cured, and tested as in the previous case. A laboratory model, electrically operated vibrator was used in consolidating the concrete. The procedures pertaining to curing, measuring, and stressing were the same for all prisms. During the 28-day curing period the average shrinkage for prisms 3 and 4 at the upper gage points was 0.00009 in. per in., and for the lower gage points it was 0.00003 in. per in. These values are slightly lower than those measured for prisms 1 and 2, but the difference is not considered significant since a slight difference in moisture content would account for the variance.

A period of about two months elapsed between casting the second and third pairs of prisms. During this time, the initial deformation of the second pair of prisms, which were consolidated with a vibrator, was somewhat higher than for the first pair, which were consolidated by hand rodding and tamping. Although the cause of this characteristic is not known, it was feared that over vibration occurred due to the inconvenience in readily shifting the vibrator in the small forms. Since the time deformation did appear higher in the vibrated prisms, the method of hand rodding and tamping was again used to consolidate the concrete in the third pair of prisms. The mix shown in column 3 of Table 2 was used for these prisms. For the various batches, a slump between 1 and 2 in. was maintained. The average shrinkage of these prisms, at the end of 28 days, was 0.00016 in. per in. at the upper gage points and 0.00006 in. per in. at the lower gage points. This shrinkage is somewhat higher than that measured for the prisms cast from concrete using Aggregate A, perhaps because of a more complete drying out during the ages of 14 and 28 days.

The difference in the 28-day shrinkage between the top and bottom gages of the various prisms is attributed to a logical differential in the rate of drying; however, a segregation of the concrete mix could result in a similar effect. It is thought that the concrete mixes used in these prisms were sufficiently stiff to make segregation unlikely, and observation did not indicate the occurrence of any segregation.

Since the purpose of this project was to determine the over-all suitability of lightweight concrete in prestressed members, no provision was made to determine the change in length due to shrinkage alone after the prisms were stressed. Only the combination of creep and shrinkage was measured after the initial 28-day curing period. In this paper, the total deformation attributable to the combination of creep and shrinkage will be termed "time deformation," the change occurring independent of stress "shrinkage," and the change due to the application of stress "creep."

RESULTS OF TIME DEFORMATION STUDIES

The unit change in length for the various prisms is shown plotted against time in Figure 5. The origin of the ordinate on these curves is based on the length measured immediately after the initial stressing, and the abscissa originates from the day of this stressing. For purpose of comparison, the instantaneous deformation occurring during the stressing operation is shown as a bar on the left side of each curve. The plotted points represent the average of the four measurements made on each prism taken immediately after restressing the prism to 2,000 psi. The plotted points were not corrected for temperature fluctuations, but the smooth curves make these corrections. In general, the deviations of the points from the smooth curve can be correlated with room temperatures recorded at the time of measurement. An exact correlation was not accomplished because the room temperature was subject to greater fluctuations than those that apparently occurred in the prisms.

In the curves for prisms 1 and 2, the extreme deviation between the plotted points and the curve during the period of 164 days and 221 days is attributed to an unintentional addition of moisture at the beginning of this period. Some of the water used in curing prisms 3 and 4, which were cast above the prisms under discussion, contacted these prisms by dripping and splashing, causing a swelling tendency in the concrete. The deviation of the low point plotted at 282 days is attributed to unusually warm weather prevailing at the time rather than a remaining effect of this moisture.

In all of the prisms, the measurements show a greater time deformation at the upper gage points than at the lower points. At the end of one year, this difference ranged from 1 percent in prism 1 to 14 percent in prism 5. The difference in prism 5 reached a maximum of 21 percent about two weeks after stressing, and it has gradually diminished since that time. Each of the other prisms displayed the same characteristic of reaching a maximum percentage differential between top and bottom deformations within a period ranging from two weeks to two months after stressing and a subsequent gradual reduction. This reduction indicates that one cause of the difference is the rate of drying of the upper part of the prism as compared to the lower part. Other possible causes are eccentricity of the stressing bar, dead load effect of the prism on its timber support, and segregation of the concrete during placing.

At the end of one year the average unit deformation for each pair of prisms was 0.00171 in. per in. for prisms 1 and 2, 0.00193 in. per in. for prisms 3 and 4, and 0.00183 in. per in. for prisms 5 and 6. As indicated previously, these values include shrinkage deformation and are for a uniform stress of 2,000 psi. The difference between the higher and lower values is

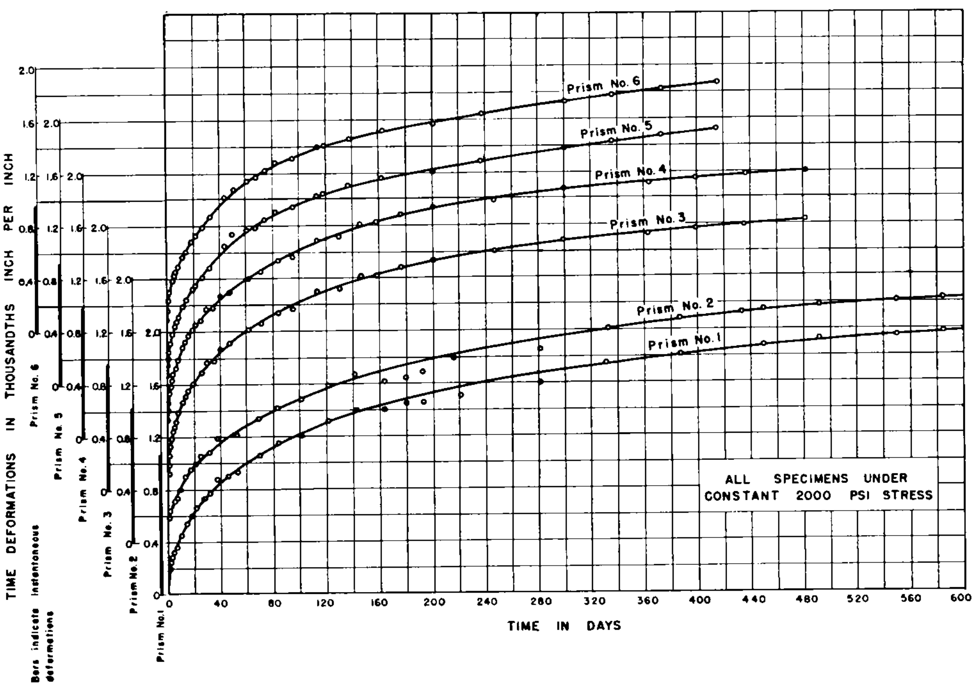

Figure 5. Time deformation curves.

about 12 percent. From the slope of the curves, it seems probable that this difference will be reduced as the prisms become older. Figure 6 shows the average time deformation for each pair of prisms. At an early age, the rate of deformation is appreciably different for the different mixes, but the curves presently show some convergence; indicating the ultimate creep might be quite similar for the different pairs. The present difference between the curves is small enough that no indication of significant superiority is detectable for either of the two mixes using Aggregate A, or the mix using Aggregate B. Creep is often expressed in terms of unit strain per unit applied load. If no allowance is made for shrinkage, the average of the above values reduces to a deformation of 0.00000091 in. per in. per pound per square inch. Since some portion of the deformation is from shrinkage not related to stress, the creep coefficient for an age of one year would be somewhat smaller than this value.

REVIEW OF DEFORMATION STUDIES ON CONVENTIONAL CONCRETE MIXES

Reports on previous studies to determine the creep and shrinkage in concretes made from normal aggregates and from other lightweight aggregates were reviewed for the purpose of comparing the time yield of the expanded shale concretes used in this study with the common heavier concretes and other lightweight concretes. Also, literature on prestressed concrete design was reviewed to obtain recommended allowances for the stress loss in prestressing steel due to creep and shrinkage of concrete made with normal

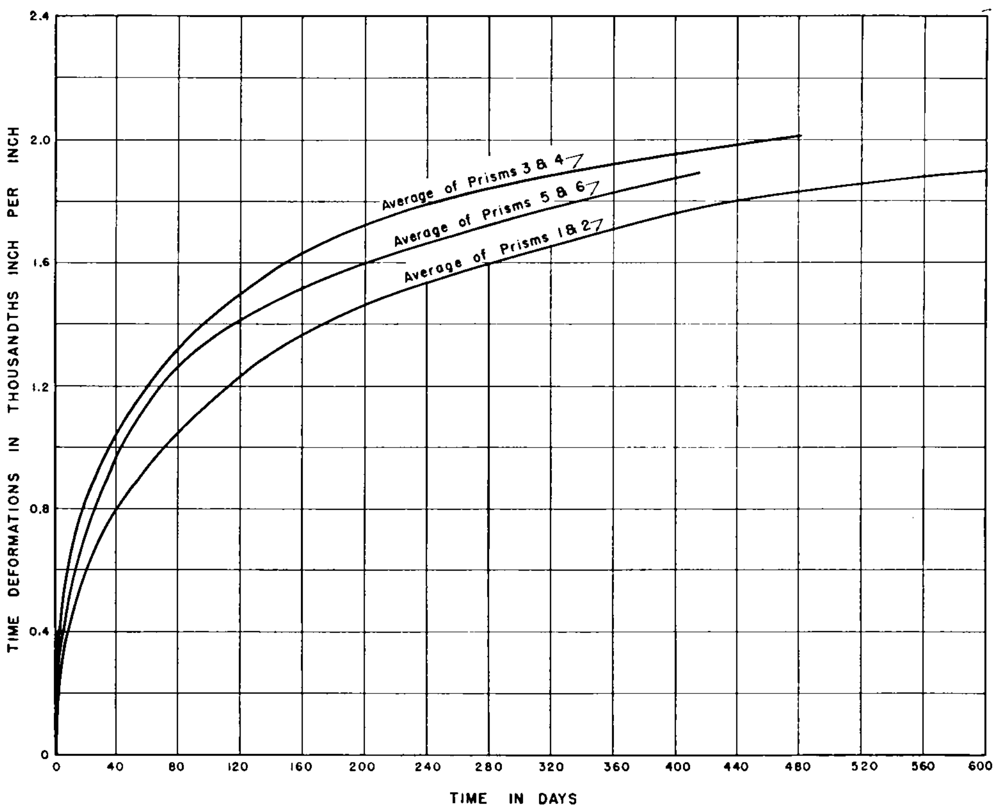

Figure 6. Time deformation curves for different mixes.

aggregates. A wide range of values was obtained for both creep and shrinkage in the various studies that were reviewed.

In a report by Davis and Davis (*1*), the influence of relative humidity, richness of mix, age at time of loading, and type of aggregate is shown. The aggregates used in this study did not include any lightweight materials. In concretes which were identical except for type of aggregate, it was found that limestone had the least creep, followed by quartz, granite, gravel, basalt, and sandstone. At the end of two years, the creep measured in limestone concrete was about 55 percent as great as gravel and about 40 percent of the value for sandstone. A gravel mix having a cement factor of 7.7 sacks per cu yd, a water-cement ratio of about 5.3 gal per sack, and a slump of 3 to 4 in. was loaded at 28 days and stored at 50 percent relative humidity. This was a rich mix that might be typical for prestressed work. At the end of one year, the creep in these specimens averaged 0.00000062 in. per in. per psi. Other studies to determine the effect of relative humidity on creep indicate that this value might be only 0.7 as great at 70 percent relative humidity and only 0.3 as great at 100 percent relative humidity. In these studies, corrections have been made for shrinkage; the values are for deformations contributable to stress alone. In a mix of about 6 sacks per cu yd and a water-cement ratio of 6.5 gal per sack, unstressed columns stored at 50 percent relative humidity showed a shrinkage of 0.00062 in. per in. at an age of one year.

In a paper by Staley and Peabody (*2*), a shrinkage of unstressed specimens of 0.00087 in. per in. is reported for bars stored at 70 F and 50 percent relative humidity for 400 days. The concrete used in these experiments was made from sand and gravel aggregate with a maximum particle size of ¾-in. The cement factor was 6.4 sacks per cu yd and the water-cement ratio was 5.6 gal per sack. The ultimate 28-day strength of test cylinders was 4,500 psi. At an age of 380 days, the creep coefficient for this concrete averaged about 0.00000070 in. per in. per psi under storage conditions of 70 F and 50 percent relative humidity. This value is in good agreement with the previous coefficient considering the difference in the richness of the concrete mixes.

Hogan (*3*) stated that for average concrete mixes made with the usual aggregates, the total contraction due to shrinkage is of the order of 0.0005 in. per in. from the completely saturated condition to the dry condition. Hogan states the ultimate magnitude of creep can be taken as 0.000001 in. per in. per psi. The comment is made that, for average mixes, the amount of shrinkage is nearly proportional to the quantity of water used and the creep is practically proportional to the percentage of a given paste in the concrete. For a given consistency, the higher paste content of the rich mix is more than offset by the lower water-cement ratio so that rich mixes creep less than lean ones.

Values of creep and shrinkage occurring in a high quality concrete are reported by Magnel (*4*). The concrete used in these studies was a crushed stone mix having a cement factor of 7¼ sacks per cu yd and a water-cement ratio of 4.23 gal per sack. The 28-day strengths of this concrete averaged about 8,500 psi. The shrinkage of these specimens between the time of stressing at 28 days and the conclusion of the tests at 800 days averaged only 0.000174 in. per in. The creep in the specimens averaged 0.00000021 in. per in. per psi during this period. Almost 95 percent of this value occurred during the first year, making the creep at the end of one year 0.00000020 in. per in. per psi. These values of creep and shrinkage are generally less than one-third of corresponding values in the studies previously discussed. These low values are probably due primarily to the low water-cement ratio, but the type of aggregate and the relative humidity of the storage area might also have been significant.

An extensive study on lightweight aggregate concrete by Shideler (*5*) presents values of creep and shrinkage for eight different lightweight concretes and includes values for a common sand and gravel concrete as a comparison. The creep coefficients for the sand and gravel aggregate mixes ranged from 0.00000099 in. per in. per psi for a 3,000-lb concrete loaded at 28 days to 0.00000032 in. per in. per psi for a 10,000-lb concrete loaded at 28 days. The creep coefficients for lightweight concretes ranged from 19 to 57 percent higher than normal-weight concrete of similar strength. For a 4,500-lb sand and gravel mix, loaded at 28 days, a creep coefficient of 0.00000074 in. per in. per psi is listed. The shrinkage measured gave values of 0.000713 in. per in. for a 3,000-lb sand and gravel concrete to 0.000620 in. per in. for a 10,000-lb sand and gravel concrete. The corresponding values for the various lightweight aggregate mixes ranged from 6 to 38 percent higher than those for the normal concrete. These tests were conducted at 50 percent relative humidity and 73 F.

Guyon (*6*) states that shrinkage may amount to 0.0003 in. per in. and that creep will be about 0.00000036 in. per in. per psi for normal-weight concrete of a quality commonly used in prestressed construction.

Lin (*7*) mentions British recommended shrinkage allowances of 0.0003 in. per in. for pretensioning and 0.0002 in. per in. for post-tensioning. For creep, a value of 0.0000004 in. per in. per psi for pretensioning and 0.0000003 in. per in. per psi for post-tensioning are recommended. He also cites German specifications which make allowance for humidity conditions. For an ordinary atmosphere, these specifications allow for a strain due to creep of about 0.0000005 in. per in. per psi, and a strain due to shrinkage of 0.0002 in. per in. For very moist air, smaller strains are anticipated, and for very dry air, greater allowances are made. An allowance for creep strain of about 0.0000004 in. per in. per psi is usually considered safe, and an average value of shrinkage strain would be about 0.0002 to 0.0004 in. per in.

The shrinkage allowances recommended by the Bureau of Public Roads (*8*) are 0.0002 in. per in. for pretensioned members and 0.0001 in. per in. for post-tensioned members. The allowance recommended for creep is 0.00000039 in. per in. per psi stress in the concrete at the centroid of the prestressing steel. It seems rather typical to consider a concrete having a 28-day ultimate strength of 5,000 psi as being satisfactory for prestressed construction. However, the studies reviewed indicate that a 5,000 psi sand and gravel concrete will have far more creep and shrinkage than the recommended allowances provide for. From these studies on normal aggregate concrete, it appears a 28-day strength of 7,000 psi to 8,000 psi would be required to maintain the time deformations within the allowances at 50 percent relative humidity.

In comparing the time yield of the lightweight concrete prisms with the time yields found for natural aggregates in the studies by Davis and Davis, Staley and Peabody, Shideler, and the values discussed by Hogan, it appears that the deformation of the expanded shale concrete is not appreciably different than might be expected for a good quality concrete made from natural aggregates. However, the values measured on this project exceed by a wide margin the values found by Magnel and the allowances recommended by Guyon, Lin, and the Bureau of Public Roads. Other things being equal, the amount of creep and shrinkage is markedly affected by the amount of water used in the mix. It appears that for sand and gravel aggregate, the water-cement ratio should not exceed a value of perhaps 4½ to 4¾ gal per sack to maintain the ultimate creep and shrinkage within the recommended allowances. Even with the high cement content of the expanded shale concretes used in this study, it seems doubtful that lightweight concrete with a water-cement ratio as low as 5 gal per sack could be properly placed in forms by normal field techniques. The water contents of the mixes used in this project were considered to be near a practical minimum for the aggregates. Drier mixes can be placed by employing external vibration, but this is usually restricted to a plant-type operation.

If an allowance of 0.0003 in. per in. is made for shrinkage of the expanded shale prisms subsequent to stressing, and it is assumed that three-fourths of the ultimate creep has occurred at the end of the first year, the ultimate creep of the concrete would be about 0.000001 in. per in. per psi. This is equal to the value stated by Hogan, and it is similar to the values reported by Davis and Davis, and Staley and Peabody if it is assumed the 1-year coefficients reported are three-fourths of the ultimate. The value is approximately midway in the range of Shideler's studies on other lightweight aggregates and about 35 percent higher than sand and gravel concrete of similar ultimate strength. However, the value is between 2 and 2½ times as large as the recommended design allowances cited. This fact, of course, does not prevent the use of expanded shale aggregates in prestressed concrete members, but it does dictate the advisability of greater allowances for creep and shrinkage than commonly recommended.

SUMMARY

These tests have shown that good, workable concretes having adequate ultimate strengths can be produced from the two expanded-shale aggregates processed in Oregon. Trial mixes in which the natural lightweight aggregates that are locally available were used did not meet what is normally considered a practical minimum ultimate strength for prestressed concrete. Because of insufficient strength, heavier unit weight, and harshness of the trial mixes, time deformation studies were not conducted on mixes containing cinders or pumice.

For expanded shale concretes used in these studies, the modulus of elasticity was found to be about 2,000,000 psi for cylinders having an ultimate strength of 5,000 psi. In considering the pretensioning method of prestressing, this low modulus of elasticity is an unfavorable characteristic since the elastic shortening occurring at the release of prestress would cause an appreciable stress loss in the steel. For equal concrete stresses, the loss in steel stress due to elastic deformation of the member would be about twice as great in expanded shale concrete as in normal sand and gravel concrete. This loss, of course, is not involved when a post-tensioning process is applied. In a post-tensioned member having numerous cables, the loss in steel stress in the first stressed cables caused by elastic shortening of the member as subsequent cables are tensioned would sometimes be excessive. Retensioning of some of the earlier stressed cables after all the cables were stressed would effectively eliminate the stress loss from this source.

During the 28-day period between casting and stressing the prisms, the average shrinkage was about 0.00012 in. per in. at the upper gage points and about 0.00004 in. per in. at the lower gage points. This differential between top and bottom is attributed to a slower rate of drying of the bottom portion of the prisms. With the additional mass of a typical prestressed bridge girder, it seems likely that the shrinkage occurring during the curing period would correspond more nearly to the lower figure, leaving more of the shrinkage to be allowed for in estimating the steel stress losses in a post-tensioned member. In this project only the combined effect of shrinkage and creep was measured after the prisms were stressed. However, it appears from other studies that a reasonable allowance for shrinkage after stressing might be 0.0003 in. per in. For a very dry atmosphere, this value might be low; in a very moist atmosphere, a smaller allowance might be adequate.

If it is assumed that 0.0003 in. per in. of the time yield is attributable to shrinkage and that 75 percent of the ultimate creep has occurred during the first year, a creep coefficient of about 0.000001 in. per in. per psi of concrete stress is obtained. To apply this value in estimating the loss of prestress in the steel, the concrete stress should be an average between initial and final values, and an average of the dead load stress from end to end of the beam at the centroid of prestressing steel. In design practice, the refinement of using an average between initial and final concrete stresses is not normally followed because of the uncertainty of the precise creep coefficient. This reasoning would apply equally to lightweight concrete design. The average relative humidity under which these prisms were stored was lower than would normally be encountered in an open-air application in Oregon. Since the concrete creep in dry air is greater than in moist air, this creep coefficient is considered a conservative design value for concrete of similar quality to that used in the test prisms.

RECOMMENDATIONS

In considering the use of expanded shale concrete in the construction of prestressed bridge members, the following characteristics warrant particular attention:

1. The unit weight of the concrete used in these tests was about 105 pcf, a reduction of 30 percent over the commonly assumed weight of 150 pcf for conventional concrete. The weight of concretes made from other expanded shales may vary appreciably from the value of 105 pcf, but in any case, a significant reduction from the weight of conventional concrete would result. Frequently, appreciable savings in construction costs could be anticipated for projects employing lightweight concrete.
2. A high cement factor and a low water-cement ratio are required in expanded shale mixes to obtain the high-strength concrete that is desirable in prestressed applications. The 9-sack mixes used in this study had good workability and adequate ultimate strength. Since the elastic deformation and creep is generally considered to be inversely proportional to the ultimate strength, a high ultimate strength takes on added importance. A reduction in water-cement ratio from that used on this project is desirable if the resulting concrete can be properly placed. Unless a significant reduction in the water-cement ratio can be achieved, a cement factor of 9 sacks per cu yd is recommended.
3. The modulus of elasticity of expanded shale concrete is much lower than that of conventional concrete, being about 2,000,000 psi for concrete having an ultimate strength of 5,000 psi. Affected by the low modulus of elasticity are the deflection under dead and live loads, the camber caused by eccentric prestress forces, and in pretensioned members, the loss in steel stress caused by elastic shortening. These items all require consideration in any prestressed concrete design; the magnitudes are merely increased in lightweight concrete.
4. Except during the first 28 days, shrinkage was not measured separately on this project. However, from a review of other studies, it seems a reasonable allowance for ultimate shrinkage would be 0.0004 in. per in. for pretensioned members and 0.0003 in. per in. for post-tensioned members.
5. The coefficient representing the ultimate creep of the expanded shale concretes used in this study is estimated at 0.000001 in. per in. per psi. This coefficient is considered a conservative design value for open-air applications in moist regions. In very arid regions a larger allowance might be warranted.
6. Since the allowances recommended above for estimating the loss of prestress in the steel are larger than the values used with conventional concrete, the importance of using a very high-strength steel becomes more pronounced. It is estimated that the steel stress losses from all sources will be about 25 percent for post-tensioned members and about 30 percent for pretensioned members if steel having an ultimate strength of 240,000 psi is stressed initially to 0.8 of its ultimate. Since the major source of prestress loss is the creep of the concrete, slightly larger beam cross-sections with correspondingly lower concrete stresses would serve to reduce the stress loss in the steel.

For convenience, the allowances recommended for estimating the decrease in prestress in steel due to shrinkage, creep in steel and concrete, and elastic shortening of the concrete in prestressed members can be represented by the following equations:

Pretensioned concrete

$$\Delta f_s = 11{,}200 + 42 f_c + 0.04 f_s \qquad (3)$$

Post-tensioned concrete

$$\Delta f_s = 8{,}400 + 28 f_c + 0.04 f_s \qquad (4)$$

In which Δf_s is the stress loss in pounds per square inch, f_c is the average end-to-end concrete stress under dead load and prestress at the centroid of the steel, and f_s is the initial steel stress in pounds per square inch. In Eq. 4, no allowance is made for slip in the anchorage, nor for loss in first-stressed cables caused by elastic shortening of the concrete as subsequent cables are tensioned. These factors will sometimes be significant and should be considered. The last term of these equations, representing a 4 percent allowance for relaxation in the steel, is listed on the basis of a review of literature on prestressed concrete design. Measurements made to determine the creep in the steel bars used in these tests were somewhat inconsistent. Being of questionable value, these data have been omitted. The equations shown resulted from tests on two particular expanded shale aggregates and are not necessarily applicable to lightweight aggregates from other regions. The magnitude of the creep in the test prisms fell about midway in the range of values reported for various lightweight aggregates by Shideler (*5*); being about 35 percent higher than the creep of sand and gravel concrete of equal strength.

Future studies might show that the factors for estimating prestress loss in the steel can be reduced, particularly if concrete having a lower water-cement ratio can be properly placed, but in the meantime, the recommendations herein do not prohibit the economic use of expanded shale in prestressed concrete members.

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