Time Dependent Deformations of Two Lightweight Aggregate Concretes

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A research project was conducted by the Oregon State Highway Department in cooperation with the U.S. Bureau of Public Roads to determine the creep and shrinkage characteristics of 2 expanded shale aggregate concretes. For the earlier phase of the project, measurements were made for about 8 years on specimens subjected to a constant compressive stress. Other prisms were subjected to a compressive stress which was permitted to diminish as creep and shrinkage of the concrete occurred. Deformations of the latter prisms were measured over a 5-year period.

To provide a basis for comparison, the relaxing stress phase of the project included identical specimens cast from normal-weight, sand and gravel aggregate concrete. The magnitudes of creep, shrinkage, and loss of prestress measured on the lightweight prisms are compared with the values measured on the normal-weight prisms.

The various measurements for both the constant stress and the relaxing stress specimens are presented and the significance of the values as applied to design is discussed.

For the specimens subjected to a relaxing prestress force, the loss of prestress was 50 percent greater in the lightweight prisms than in the normal-weight prisms.

•THE PURPOSE of this project was to evaluate the suitability of lightweight concretes, using aggregates readily available in Oregon, for use in prestressed concrete bridge members. The project consisted of 3 phases: first, a rather limited investigation of mix design; second, a long-term investigation of time deformations of concrete prisms subjected to a constant stress; and third, determining the deformations of prisms under a relaxing prestress force. In addition to the studies of lightweight aggregates, the latter phase included identical specimens cast from sand and gravel aggregates to obtain comparable values of deformations between lightweight and normal-weight concretes.

Many engineers concerned with the design and construction of bridges and other structures have recognized potential advantages that would result from the use of lightweight aggregate concrete in prestressed members. However, lightweight aggregates have not been used extensively in prestressed work because of the limited knowledge of the physical properties of the lightweight concretes, particularly with regard to creep and shrinkage. During recent years, a number of studies have been undertaken to broaden the engineer's knowledge of the physical properties of structural quality lightweight concretes.

During 1955 the Oregon State Highway Department initiated a research project to study the characteristics of concretes manufactured from lightweight aggregates that are readily available within the state. The objectives included the design of concrete

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mixtures having suitable strength and workability for use in prestressed members and the measurement of time dependent deformations in specimens cast from selected concrete mixtures. One group of test prisms was maintained under constant stress for approximately 8 years. During this time periodic measurements of the change in length of the specimens were made. Another group of prisms was subjected to a gradually reducing prestress force for a period of over 5 years. Progress reports on both groups of prisms have been published (1, 2). The rate of deformation has now reduced to the point that future deformations would appear to be insignificant, and the project is therefore being terminated.

AGGREGATES

The aggregates investigated in connection with the project were volcanic cinders, pumice, and 2 expanded shale aggregates produced commercially in Oregon and marketed under trade names. For this project, the expanded shales have been designated as aggregate A and aggregate B. The normal-weight aggregate used in the control specimens was Santiam River sand and gravel.

The expanded shale aggregates are the products of crushing and burning Keasey shales. Before burning, aggregate A is crushed to a 2-in. maximum size and aggregate B to a 3-in. maximum size. Both materials are expanded by heating them to about 2,000 F in rotary horizontal kilns. After burning, the aggregates are crushed to the desired sizes for marketing. The resulting material is primarily uncoated; that is, most of the larger particles and many of the small ones have porous faces. Initial studies were made using 3¾-in. maximum particle sizes in the expanded shales, but the mixes chosen for time deformation studies used ⅜-in. to 3/16-in. and 3/16-in. to 0. The grain size analyses of the various aggregates used in the time deformation studies are given in the progress reports (1, 2).

The maximum particle size used in the sand and gravel concrete was ¾ in. These normal-weight materials are sound and durable and have a good service record as concrete aggregates.

The pumice and volcanic cinders were obtained from natural deposits in central Oregon and subsequently crushed to desired gradings.

MIX DEVELOPMENT

In the beginning stages of the project, concrete mixes made with the 2 expanded shales, with cinders, and with a combination of cinders and pumice were cast in standard cylinders and tested at ages of 28 and 60 days to determine the ultimate compressive strengths of the concrete. In addition, the various mixes were judged for workability and the slump and wet weights were measured. In designing the mixtures, it was decided the concrete should have a minimum 28-day ultimate strength of 4,500 psi to be economically feasible for the intended purpose. Cement contents were varied between 6 and 9 sacks per cu yd and the coarse and fine aggregate proportions were varied to evaluate the effect on workability.

Several trial mixes were made using volcanic cinders. This material is very sharp and concrete produced with it was considered too harsh for use in prestressed members. The wet weight of the concrete was about 122 pcf, somewhat higher than was desirable. Because of the harshness and the weight of the concrete, mixes with a full range of cement content were not made. A 7-sack mix had a 28-day ultimate strength of about 3,000 psi.

To reduce the weight and improve the workability, pumice were used as the fine aggregate and cinders as the coarse aggregate. This combination produced concrete that was much lighter (101 pcf wet) and the workability was somewhat improved, but the concrete still seemed somewhat harsh and the ultimate strength of these mixes fell far short of meeting the desired 4,500-psi 28-day strength. Because of the unsatisfactory strength and workability, no time deformation studies were made on these concretes.

The initial trial mixes with expanded shale aggregate A used ¾-in. to ⅜-in. material for the coarse fraction and ⅜-in. to 0 for the fine fraction. Other mixes were made with only the ⅜-in. to 0 material. For later mixes the manufacturer supplied ⅜-in. to

$^3/_{16}$-in. and $^3/_{16}$-in. to 0 material and these were used as the coarse and fine fractions. Because of a slight harshness of these mixes and influenced by previous experience of the manufacturer, 2 cu ft of natural sand per cubic yard of concrete were combined with the $^3/_8$-in. to $^3/_{16}$-in. and the $^3/_{16}$-in. to 0 aggregates. Using these components, a 9-sack mix having excellent workability and adequate ultimate strength were selected for a study of time deformation characteristics.

The most extensive experimentation with the various aggregates was with aggregate B, for which 16 trial mixes were made. Several mix designs developed by an independent testing laboratory were obtained and used as a basis for the earlier mixes developed in connection with this project. Initially the aggregate was furnished in 3 sizes—$^3/_4$- to $^3/_8$-in., $^3/_8$- to $^3/_{16}$-in., and $^3/_{16}$-in. to 0. In many of the mixes all 3 sizes were used in varying proportions but a stronger, more workable concrete resulted from using $^3/_8$ in. as the maximum particle size. To provide a margin of safety on the desired ultimate strength, a 9-sack mix was selected for use in the time deformation studies. Details of the concrete mixes used in the time deformation studies are given in Table 1. Aggregates for prisms numbered 7 through 12 were obtained at a different time than those that were used in prisms 1 through 6 and although the volume ratios of coarse aggregate to fine aggregate were maintained the same for a given aggregate, variations in gradation and unit weight caused significant differences in the resulting concrete mixes.

To obtain high-strength concrete with the expanded shale aggregates used in this study, it was found that a rather high cement factor was required. An objective of the project was to develop mixes having high ultimate strength and suitable workability to make the placing of the concrete feasible using conventional field techniques. By using lower water-cement ratios or higher proportions of coarse aggregate in relation to the amount of fine aggregate, concretes having satisfactory strengths can be produced with lower cement factors; however, these mixtures did not seem suitable for field placement. For the construction of members in a casting yard, where rigid controls and adequate vibration can be assured, the drier, harsher mixes can be used to economical advantage.

Because the absorption of water by lightweight aggregates fluctuates widely, depending on the porosity and the moisture content of the particles, it was found impractical to design a mixture with a rigid water-cement ratio. To control consistency to a specified slump is more satisfactory. For comparable workability, the lightweight concretes will have appreciably less slump than corresponding sand and gravel concretes. In this study, it was found that expanded shale concretes having a slump of $1^1/_2$ in. were easily placed and tended to flow readily under the action of a vibrator. It was estimated

TABLE 1

CONCRETE MIXTURES USED IN TIME DEFORMATION STUDIES

Property	Mixture					
Aggregate type	A	A	B	A	B	Sand & Grav.
Prism numbers	1, 2	3, 4	5, 6	7, 8	9, 10	11, 12
Coarse aggregate, $^3/_8$ in. -$^3/_{16}$ in., lb per cu yd	311	283	366	307	431	—
Coarse aggregate, cu ft per cu yd	8.2	8.5	8.3	9.2	7.9	—
Fine aggregate, $^3/_{16}$ in. -0, lb per cu yd	960	986	1,213	1,056	1,183	—
Fing aggregate, cu ft per cu yd	19.0	19.6	21.7	21.2	20.7	—
Gravel, $^3/_4$ in. -$^3/_{16}$ in., lb per cu yd	—	—	—	—	—	2,220
Gravel, cu ft per cu yd	—	—	—	—	—	20.0
Natural sand, lb per cu yd	186	159	0	173	0	907
Natural sand, cu ft per cu yd	2.0	2.0	0	2.0	0	9.0
Cement factor, sacks per cu yd	8.6	8.8	9.0	9.5	8.6	7.5
Admixture, lb per sack	0.25	0.25	0.25	0.25	0.25	0.25
Water/cement ratio, total water, gal per sack	7.3	7.1	6.2	6.4	7.5	4.7
Wet weight, pcf	102	103	107	108	104	153
Slump, in., average	$2^1/_2$	$1^3/_8$	$1^1/_2$	$1^3/_4$	$1^1/_2$	$2^1/_4$
28-day compressive strength, psi	4,450	5,078	5,690	6,260	5,880	6,420
Secant modulus of elasticity at 28 days, psi	1,722,000	2,014,000	2,069,000	2,244,000	2,233,000	5,010,000

that the consistency of this concrete was similar to sand and gravel concrete having a slump of about 3 in.

Of concern to designers of prestressed concrete members is the modulus of elasticity of the concrete. The secant modulus of elasticity, calculated at 2,000-psi stress level, was determined for most of the trial mixes as well as the mixes used in the time deformation studies. Values of modulus of elasticity vs ultimate strength are plotted for aggregate A in Figure 1 and for aggregate B in Figure 2. The straight-line equations that best represent the modulus of elasticity of these concretes were determined by the method of least squares to be:

1. for aggregate A, $E = 420,000 + 320 f'_c$, and
2. for aggregate B, $E = 560,000 + 275 f'_c$.

Since the cylinder strengths used in obtaining these equations varied only from about 4,000 to 7,000 psi, the equations can only be assumed to be applicable within this range.

The modulus of elasticity values of concretes from both of the expanded shale aggregates were consistently lower than the values calculated from the equation $E = w^{1.5}\sqrt{f'_c}$, contained in the 1963 ACI Building Code revision (3), where w is the weight of the concrete in pounds per cubic foot and E and f'_c are in pounds per square inch. The measured values are approximately 80 percent as great as values calculated from this equation.

For the limited data available from the concretes tested on this project, an equation of the type proposed by the ACI Code committee that fits the values for the lightweight

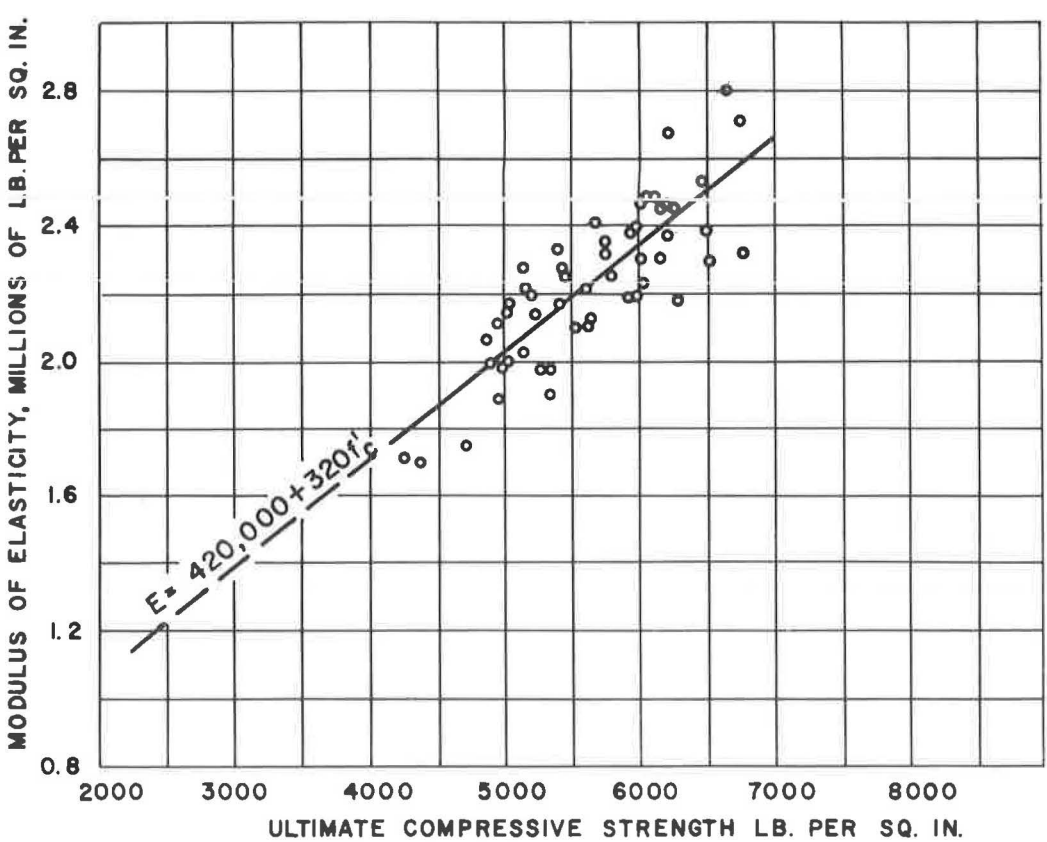

Figure 1. Modulus of elasticity vs ultimate strength for concrete using aggregate A.

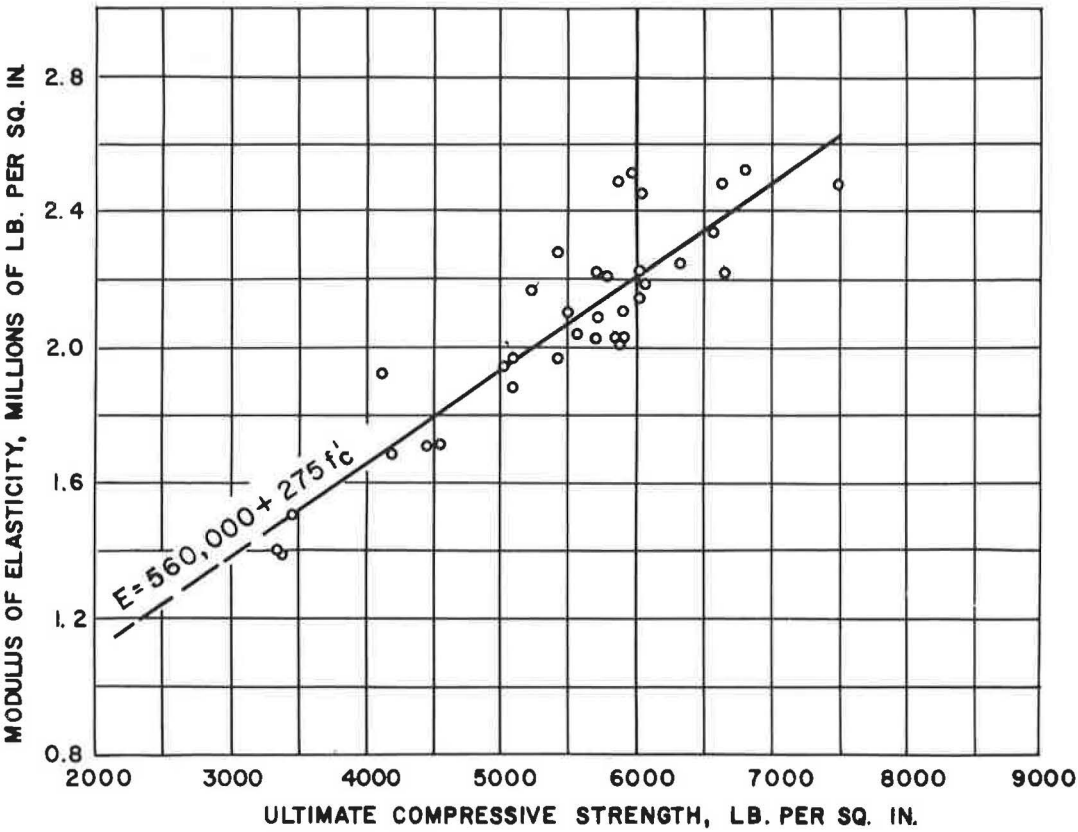

Figure 2. Modulus of elasticity vs ultimate strength for concrete using aggregate B.

concretes with less error and also fits the sand and gravel concrete is $E = 1.1\ w^{2.18} \sqrt{f'_c}$ This equation is presented for purposes of comparison only. Although it provides reasonably good values for the concretes used in this study, its general applicability is unknown.

PROCEDURE FOR CREEP AND SHRINKAGE STUDIES

To determine the total time deformation in the concrete mixtures that were chosen for these studies, rectangular prisms 5 in. by 6 in. by 107 in. were cast. A flexible metal conduit was cast into the prisms along the longitudinal axis to permit the introduction of reinforcement for stressing. Four steel bars, 2 near each end, were cast in the prisms perpendicular to the axis of the prisms. These bars protruded from each side of the prisms and provided 4 pairs of gage points 100 in. center-to-center for each specimen. The gage points were 1 in. from the top and bottom surfaces of the prisms. The gage used for length measurements consisted of an Invar rod with a dial indicator graduated to 0.001 in. mounted at one end and a pointed stud and clamp at the opposite end. During the measurements, the gage was supported at the ends and at 4 intermediate points so that deflection of the Invar rod was practically eliminated.

The prisms were cast and stored in timber racks. To reduce resistance to creep and shrinkage, the prisms were supported by aluminum covered members on which a parting compound was used to prevent bond between the aluminum and the concrete. The side forms were removed 24 hours after casting the prisms. The ends of the prisms were provided with load distributing plates 1¼ in. thick having the same areal dimensions as the prism cross section. These plates served as the end forms in casting the members.

In the portion of the study in which specimens were subjected to a constant stress, 2 prisms were cast from aggregate B concrete and 4 prisms from aggregate A concrete. These prisms were numbered 1 through 6; the mix details are given in Table 1.

To determine the time deformation of concretes subjected to a relaxing prestress force, 6 additional 107-in. prisms were cast—2 each from aggregate A, aggregate B, and sand and gravel concretes. In this latter study, 2 shorter prisms were cast from each of the 3 mixes to provide shrinkage specimens. The shrinkage prisms had the same cross-sectional dimensions as the longer prisms and, to maintain the same area-perimeter ratio, they also had a flexible conduit at the longitudinal axis. The shrinkage specimens had a 24-in. gage length but otherwise the arrangement of the gage points was the same as those on the longer prisms. To obtain adequate sensitivity, dial indicators graduated to 0.0001 in. were used on gages for measuring shrinkage.

Twelve standard cylinders were cast from each of the concrete mixes at the time the prisms were poured to provide information on strength and modulus of elasticity. All concrete for the project was mixed in a Lancaster laboratory mixer in batches of about 1.3 cu ft. Consistency was controlled by a slump test on each batch. Average slumps are given in Table 1. Except for prisms 3 and 4, the concrete was consolidated in the forms by means of hand rodding and tamping. An electrically operated laboratory vibrator was used for prisms 3 and 4 but because of some difficulty in using the vibrator in the restricted forms and higher initial time deformations in the prisms on which it was used, its use was discontinued. The cylinders were cured in a moist room until tested and the prisms were cured by maintaining a wet burlap covering over them for a 2-week period. Approximately 24 hours after casting the various prisms, the side forms were removed and initial measurements of length were made. To detect any differential shrinkage or creep, measurements were always made at each of the 4 gage positions on each prism. Periodic measurements were made on all of the prisms during a 28-day curing period preceding the stressing of the longer prisms.

PRESTRESSING DETAILS

The first 6 prisms were stressed to 2,000 psi and, within narrow limits, this stress has been retained. These prisms are referred to as being subjected to a constant stress although the actual stress varied between values of 1,900 and 2,100 psi. The variation seldom exceeded 75 psi, from a low of 1,950 to a high of 2,025 psi. To apply the stress, a 1-in. diameter Stressteel bar was introduced at the longitudinal axis through the metal conduit cast into each prism. The bars were tensioned by means of a calibrated, center-hole ram and pump and stress adjustments were made through the anchorage nuts on the threaded bar ends. The bars were retensioned at intervals so that a relatively constant stress was maintained.

To study creep under conditions similar to those that exist in typical prestressed members where the stress intensity is gradually diminished as creep and shrinkage of the concrete and relaxation of the steel occur, an initial stress of approximately 1,820 psi was applied to a series of 6 prisms and natural relaxation was permitted to occur. To provide this stress a standard Prestressing Inc. unit having six 0.25-in. diameter wires was inserted through the conduit in each prism. The wires in these units have double buttons at the ends—an outer button for stressing and an inner one for anchoring. The standard anchorage and stressing hardware was used. To provide precise adjustment at the anchorage, a threaded bearing collar was used instead of shims. Because some seating deformation of the buttons was anticipated, the cables were stressed, anchored, restressed, and, after adjusting the threaded collar, anchored again.

In an effort to measure the difference in stress of various wires in a given wire group, type A-12, SR-4 resistance strain gages were installed on each wire. These gages have a 1-in. gage length and a trim width of ⅛ in. Some difficulty was encountered in installing the gages on the small-diameter wires. The gages were sensitive to strain and in most instances reasonable readings were obtained from them; however, the variation in readings was large enough that the difference in stress in individual wires of a group was more or less obscured by the experimental errors in measurement. These data are not considered reliable. The initial total force calculated from

the strains measured in the individual wires varied from 91.5 percent to 94.9 percent of the design load with the average being 93.4 percent of design load. The reduction in strain measured on the individual wires as creep and shrinkage of the concrete occurred was not as great as the reduction in total force on the prisms would indicate. To summarize, the initial SR-4 gage strain readings were low, on the average, and the indicated change in strain as losses occured was also low, but not by a consistent ratio.

To measure the total loss of stress in the wires as creep and shrinkage of the concrete and relaxation of the steel occurred, pressure cells were installed between the end anchorage and the bearing plate on each prism. These cells were fabricated from 12-in. lengths of AISI 4130 Shelby tubing 2½ in. in diameter with 11-gage walls. On opposite sides of each prism dial indicators graduated to 0.0001 in. were installed to measure strain over a 10-in. gage length. One division on the dial indicators represented a change in load of about 280 lb. This sensitivity is equal to approximately 0.5 percent of the initial applied load. Prior to use, the pressure cells were preloaded briefly to reduce the relaxation of the steel in the cells and were calibrated in a hydraulic testing machine.

Immediately before and after stressing a given prism, length measurements were made and readings were taken on the pressure cells and SR-4 gages. Since the design load was applied by a calibrated ram, these various measurements provided several methods of evaluating the initial force. Agreement within 1 percent was obtained between the force indicated by hydraulic ram and pressure cell on all but one prism. On the one prism, the pressure cell indicated an initial force about 8 percent higher than the hydraulic indication. It appears an erroneous zero reading was obtained for this cell, perhaps because of some slight damage during stressing.

RESULTS OF SHRINKAGE MEASUREMENTS

On the group of 6 prisms that were subjected to a constant stress, no provision was made to measure shrinkage separately. Shrinkage was measured during the 28-day curing period preceding stressing, but subsequently only the combination of creep and shrinkage was obtained. During the 2-week period that moist burlap was kept over them, a slight growth was measured in most of the prisms. After 28 days, all prisms had some shrinkage. The average values for the individual pairs were as follows: prisms 1 and 2, 0.75 ten-thousandths in. per in.; prisms 3 and 4, 0.60 ten-thousandths in. per in.; and prisms 5 and 6, 1.10 ten-thousandths in. per in. These pairs of prisms were cast at different times and it is likely that slight variations existed in the amount of water available to the concrete from the moist burlap so the differences in shrinkage existing at the time of stressing were not considered to be significant. In each of the prisms, higher shrinkage was measured at the upper gage points than at the lower points. This difference is attributed to a more rapid drying near the top since the prisms rested on aluminum sheets, preventing loss of moisture at the bottom.

Shrinkage specimens were cast to accompany the prisms subjected to a relaxing prestress force. During the 28-day curing period, periodic length measurements were made on the shrinkage specimens and the longer prisms that were prepared for stressing. During the 2-week period that the prisms were kept moist, some swelling occurred in those cast from expanded shales, whereas those cast from sand and gravel showed some shrinkage. The longer prisms showed less change in length during the curing period than the shorter shrinkage specimens. The longer prisms of sand and gravel concrete had a very slight shrinkage during the initial 2-week period and the longer prisms of lightweight concretes show less growth than the shorter prisms of the same concrete. All prisms received identical treatment, but slight variations in available moisture are likely and the frictional resistance of the longer prisms to changes in length would be greater than for the shorter prisms. Figure 3 shows the average shrinkage for each of the concretes as measured on the shrinkage specimens. The initial measurements were made immediately after the side forms were removed and the study on these specimens was continued for over 5 years. The prisms were numbered to correspond with the same concrete in the longer prisms. Prisms 7 and 8 are from aggregate A concrete, prisms 9 and 10 from aggregate B, and prisms 11 and 12

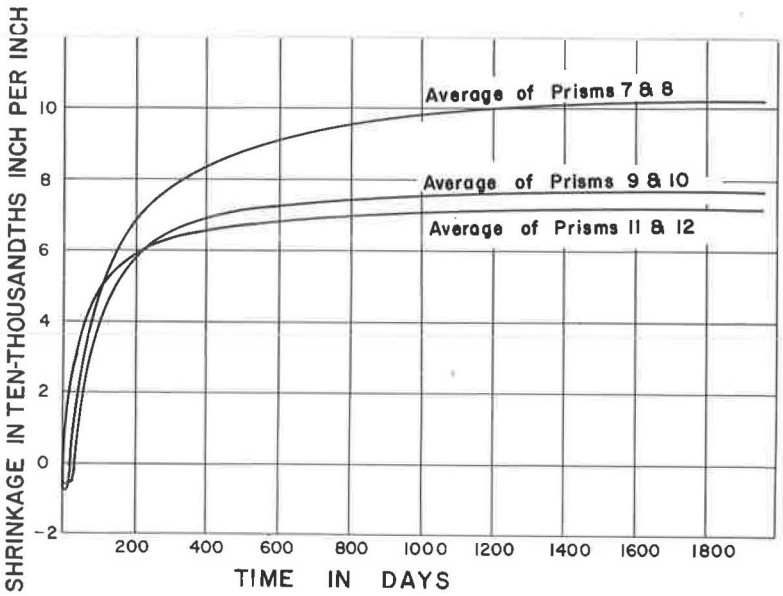

Figure 3. Average shrinkage of prisms.

from sand and gravel. Slight variations in shrinkage existed between the individual prisms cast from a given concrete, but these differences were considered insignificant, and only the average values for each pair are shown. The shrinkage of the aggregate B concrete exceeds that of the normal-weight concrete by only 4 percent but the shrinkage of the aggregate A concrete is 41 percent greater than that of the normal-weight mixture. The reason for the significant difference in shrinkage between the 2 lightweight concretes cannot be established from the limited scope of this study. However, reports of other studies suggest several items that probably contributed to the greater shrinkage of the aggregate A concrete. Jones and Hirsch (4) found that mixtures having a higher cement content, with aggregate type and gradation and consistency of the concrete being the same, had higher shrinkage. Also, it has been found that high percentages of fine particles in the aggregate result in higher shrinkage of the concrete. The aggregate A concrete had a higher cement content and a higher percentage of material passing the No. 100 sieve than the aggregate B mix. Aside from the greater shrinkage of prisms 7 and 8, the difference in shape of the normal-weight concrete curve from those of the lightweight concretes is significant. In considering the loss of prestress due to shrinkage, it is the shrinkage that occurs after the tendons are stressed that is of importance. In a post-tensioned application, where the tendons may not be stressed for 3 or 4 weeks after the concrete is cast, the measurements on the prisms indicate appreciably less subsequent shrinkage for the normal-weight concrete than for either of the lightweight concretes.

The values of ultimate shrinkage measured on this project exceed by a wide margin the usual allowance contained in recommendations for the design of prestressed members. For normal-weight concrete, the recommendations of different authorities usually vary between 0.0002 and 0.0004 in. per in. for shrinkage allowance. No recommended allowances are presented for lightweight concrete.

A number of studies conducted on structural quality concretes during recent years report shrinkage strains comparable to those measured on this project. A paper by Raymond E. Davis and Harmer E. Davis (5) reports that unstressed specimens stored at 50 percent relative humidity showed a shrinkage of 0.00061 in. per in. at an age of one year. Shideler (6) reports ultimate shrinkage values for normal-weight concrete ranging from 0.00073 in. per in. for a 4,500-psi mixture to 0.00053 in. per in. for a

7,000-psi mixture. For corresponding lightweight concretes, Shideler found shrinkage values ranged from 6 to 38 percent higher than for normal-weight mixes. These tests were conducted at 50 percent relative humidity. In a paper by Staley and Peabody (7) shrinkage of 0.00087 in. per in. is reported for normal-weight concrete bars stored at 70 F and 50 percent relative humidity. Shrinkage values for various lightweight concretes ranging from a high of about 0.0009 in. per in. to a low of about 0.0004 in. per in. are reported by Jones and Hirsch (4) for specimens stored at an average relative humidity of 60 percent. The higher values were for the richer concrete mixes. A paper by Corley, Sozen, and Siess (8) shows shrinkage values of about 0.0006 to 0.0007 in. per in. for laboratory specimens stored at 70 F and 50 percent relative humidity.

On this project, no control of temperature and relative humidity was maintained. Temperatures were recorded each time measurements were made and these values indicate the average temperature was about 72 F. From occasional measurements of relative humidity, it is estimated this value averaged around 50 percent. The average relative humidity of the storage area would be somewhat lower and much more uniform than that which would be expected in a typical outdoor installation in Oregon. This lower relative humidity would accelerate the time rate of shrinkage and probably increase the total amount. The paper by Jones and Hirsch compares the shrinkage of laboratory-stored specimens with identical specimens that were stored outdoors where they were exposed to rain, fog, and light freezing. Even though the average relative humidity and the average temperature was almost the same for both exposures, the field specimens show less than one-third as much shrinkage as the laboratory specimens. This extreme variation makes it difficult to estimate how much shrinkage should be anticipated for concrete in a bridge member. The deck would provide shelter from direct precipitation, but fog and condensation would probably reduce the shrinkage far below the laboratory measured values for bridges constructed in other than a very arid climate. Further research is needed in this area.

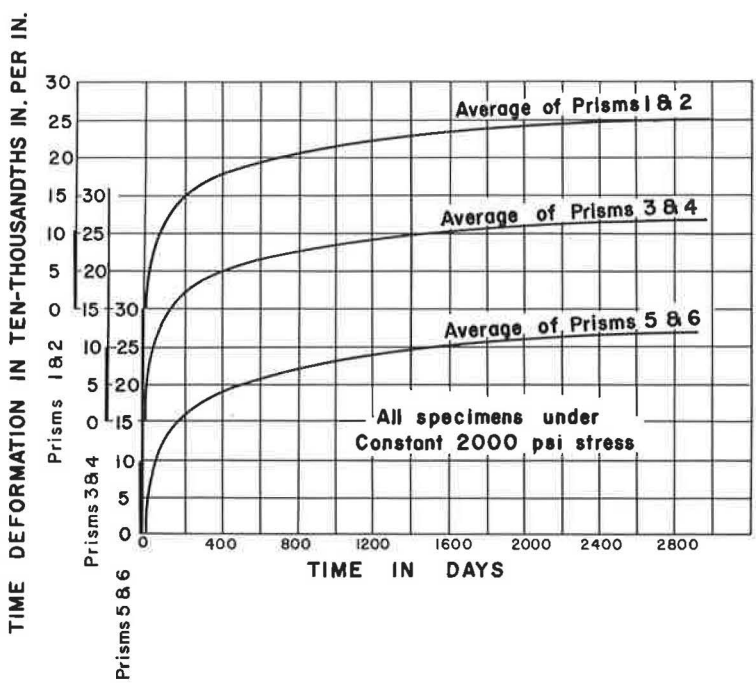

Figure 4. Time deformation curves.

TIME DEFORMATION OF SPECIMENS

The unit change in length for the prisms subjected to constant stress is shown in Figure 4. This deformation includes both creep and shrinkage and will be referred to as "time deformation." 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 purposes of comparison, the instanteous, or elastic, deformation is shown as a bar on the left side of each curve. Prisms 1, 2, 3, and 4 were cast from aggregate A concrete, 2 each from 2 slightly different mixes. Prisms 5 and 6 were from aggregate B concrete. Deformations of the individual prisms of the various pairs were very similar. The greatest difference was between prisms 1 and 2 where the variation was about 5 percent. The average deformation of the different pairs is also quite similar. At an age of 8 years, the averages were 0.00252 in. per in. for prisms 1 and 2; 0.00268 in. per in. for prisms 3 and 4; and 0.00270 in. per in. for prisms 5 and 6. Although the difference is hardly significant, it is interesting to note that prisms 1 and 2, having the lowest time deformation, were from concrete having lower ultimate strength, higher slump, higher water-cement ratio, but also, a lower cement content.

The values of time deformation at the termination of the measurements can be considered ultimate values with very little error. It is frequently estimated that approximately 75 percent of the ultimate creep will occur during the first year. These prisms had one-year values of creep plus shrinkage that ranged from 68 percent to 73 percent of the ultimate values—similar to the 75 percent estimate.

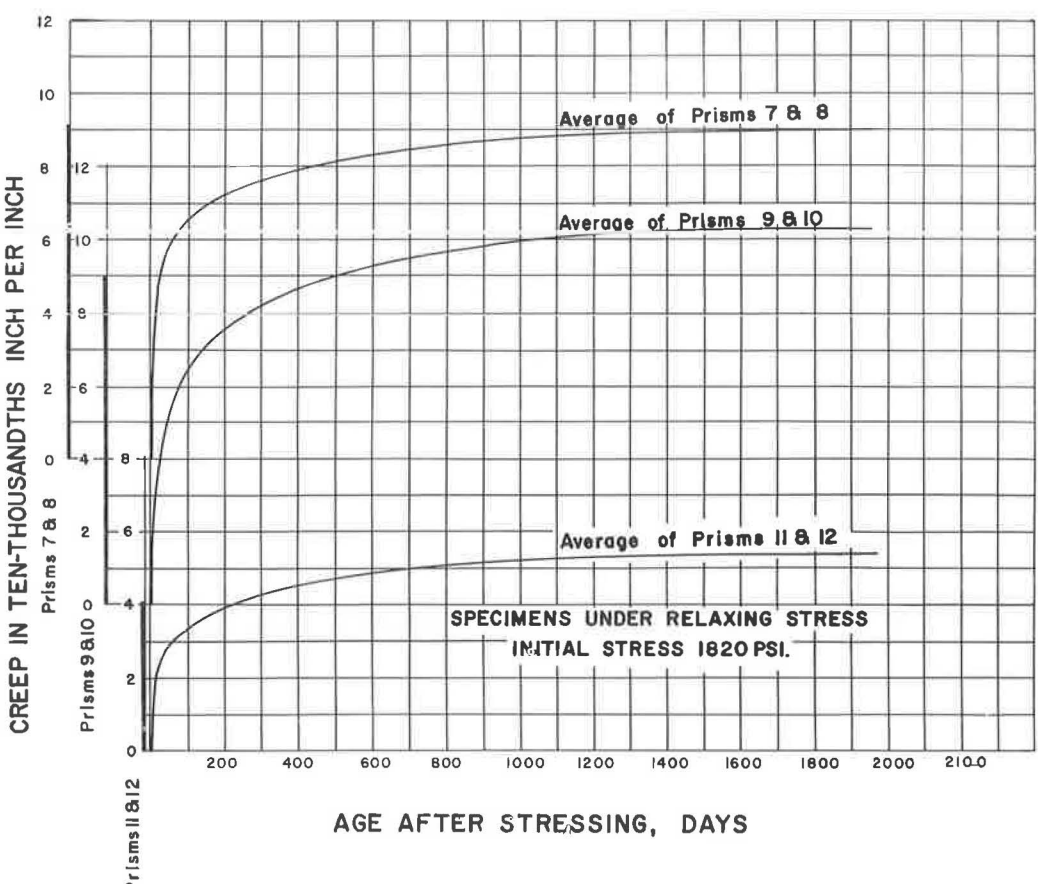

Figure 5. Creep deformation of prisms.

For the prisms subjected to a relaxing prestress force, companion specimens remained unstressed to measure the shrinkage of the concrete. The creep was determined by subtracting the shrinkage from the measured total deformation. The average unit creep for these pairs of prisms is shown in Figure 5. In this group, prisms 7 and 8 were made with aggregate A, prisms 9 and 10 with aggregate B, and prisms 11 and 12 with sand and gravel concrete. The measurements extend over a period of about 5.3 years and from the present slope of the curves it would seem that any future creep would be insignificant. As would be expected, the fact that these prisms were under a relaxing force caused them to attain a condition of equilibrium at an earlier age than those prisms that were under a uniform stress. The deformations of the individual prisms from a given concrete were all quite similar. A maximum difference of 7 percent occurred between prism 9 and prism 10, which were cast using aggregate B. The average elastic deformation of each pair of prisms is shown as a bar on the ordinate. Comparing the different concretes, the average unit creep at the conclusion of measurements was 0.00091 in. per in. for prisms 7 and 8, 0.00103 in. per in. for prisms 9 and 10, and 0.00054 in. per in. for prisms 11 and 12. On these prisms, the measured creep for the aggregate A concrete is 69 percent greater than that for the sand and gravel concrete and the aggregate B concrete is 90 percent greater than the value for the sand and gravel concrete. Comparing the total deformations (creep plus shrinkage), the differences among the 3 concretes are not as great. On this basis, the total deformations are 52 percent greater for aggregate A and 41 percent greater for aggregate B than the value measured for the sand and gravel concrete.

LOSS OF PRESTRESS

The designer of prestressed concrete members must estimate the magnitude of the initial prestressing force required to assure the retention of some minimum design value after the ultimate creep and shrinkage of the concrete and the ultimate relaxation of the steel have occurred. The decrease in prestress can be estimated with reasonable accuracy if sufficient information about the physical characteristics of the concrete and steel is available. For the prisms subjected to a relaxing prestress force, pressure cells were installed to measure the loss of prestress directly. Figure 6 shows the change in prestress expressed in terms of the initial prestress remaining. The measurements cover over 5 years and it appears that the ultimate values have been attained. The average values for the different aggregates indicate a 30.1 percent loss for aggregate A prisms, a 29.9 percent loss for aggregate B, and a 19.8 percent loss for the normal-weight concrete. Calculations of loss of prestress based on the measured time deformations of the concrete indicate average losses of 26.0 percent for aggregate A, 24.2 percent for aggregate B, and 17.0 percent for the normal-weight concrete prisms. Comparing the values of measured and calculated losses, the difference indicates average losses from relaxation of the steel of 4.1 percent for the pair of prisms from aggregate A, 5.7 percent for aggregate B, and 2.8 percent for the sand and gravel concrete. These are not unreasonable values for relaxation losses; however, since all of the steel was from the same lot and stressed to the same level, the relaxation losses should be very nearly equal for all prisms. The variations are probably the result of experimental errors in measurements.

The value of loss of prestress measured on the test prisms would require some modification to permit their application in the design of prestressed concrete members, even though the same stress intensity were applied. In the test specimens, the concrete portions of the prisms were 105 in. long whereas the length of the stressed steel was approximately 125 in., the difference being the combined length of pressure cell, bearing plates and miscellaneous anchorage hardware. In a typical beam of commercial dimensions, the difference in length between the concrete and the stressed steel is an insignificant percentage of the total length, whereas in the test prisms the steel was 19 percent longer than the concrete. The stress losses caused by shortening of the concrete would be increased by this amount if steel and concrete were the same length, all other factors being equal.

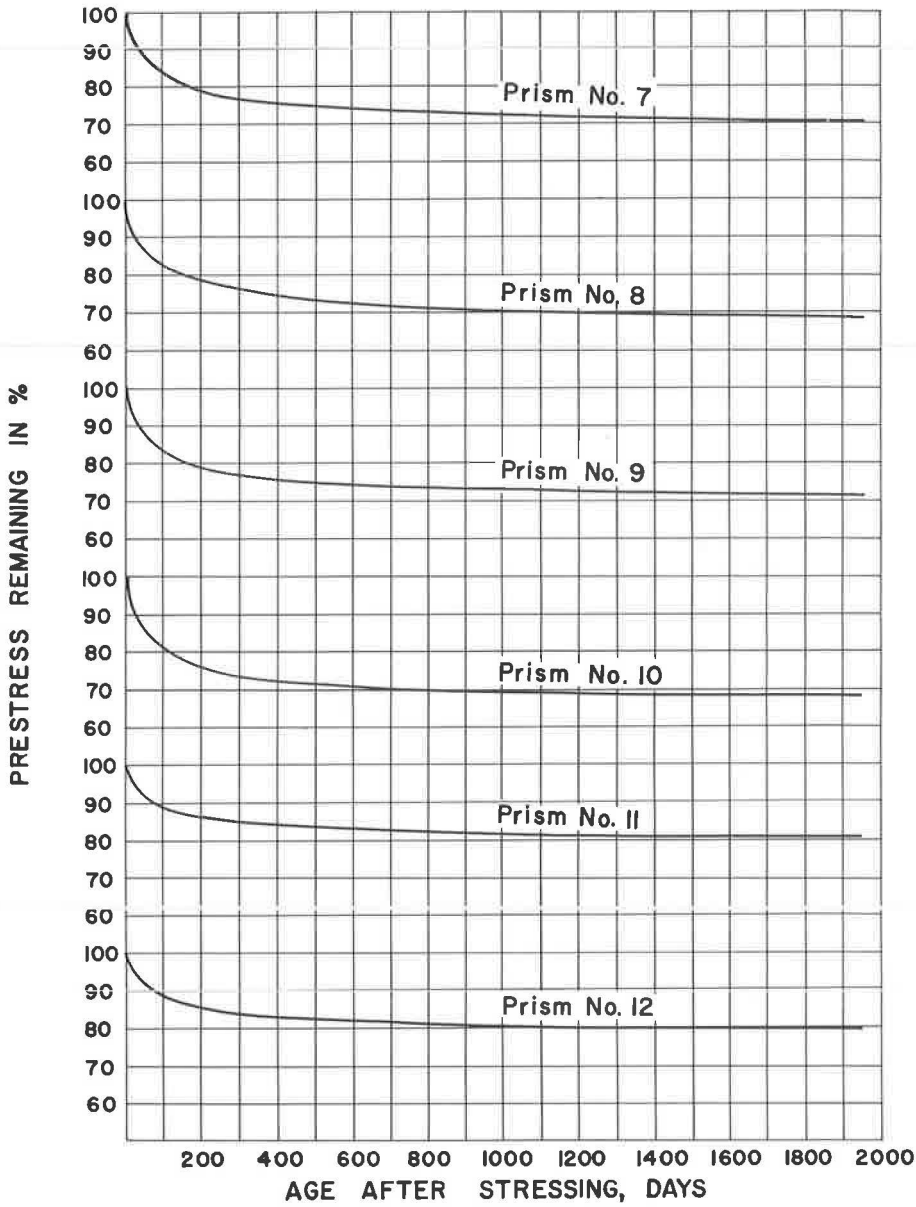

Figure 6. Loss of prestress curves.

COMPARISON OF CONSTANT STRESS AND RELAXING STRESS

To provide a basis for comparison of the prisms subjected to constant stress with those under a relaxing stress, it is necessary to assume values of shrinkage for the constant-stress prisms. The best estimate is probably provided by assuming that the shrinkage of the constant-stress prisms was the same as that of the concretes used in the relaxing-stress phase of the project. The differences between the concretes used in the studies leave some doubt about the accuracy of such an assumption; however, the magnitude of error should not be great.

After correction for shrinkage, the constant-stress prisms yield an average creep coefficient of 0.79 microinches per in. per psi for aggregate A concrete and 0.97 microinches per in. per psi for aggregate B concrete. The constant-stress studies did not include normal-weight concrete prisms so no comparisons are available between the normal-weight and lightweight materials.

To obtain a visualization of the effect of the diminishing stress intensity as compared with specimens under constant stress, the coefficients for the relaxing-stress study can be computed on the basis of the initial stress level. On this basis, the average creep coefficients for the different prisms were 0.50 microinches per in. per psi for aggregate A, 0.57 microinches per in. per psi for aggregate B, and 0.30 microinches per in. per psi for the normal-weight concrete. Numerous investigators have determined a linear or near linear relationship between stress and creep. This fact permits the use of such coefficients to predict losses that may occur under stress intensities different from those used in the tests. Since these latter coefficients were evaluated under a relaxing stress condition that would resemble the stress changes occurring in a prestressed member, they could be used to estimate the ultimate creep of prestressed members constructed from similar concretes by considering only the initial stress conditions. Some variation would result for different amounts of shrinkage and relaxation since these factors would influence the magnitude of the effective stress existing over a period of time. However, the effect of such differences would be minor.

A comparison between the creep coefficients obtained for the constant stress prisms and the latter coefficients computed on the basis of the initial stress reveals the latter are about 60 percent of the former. This comparison demonstrates the reduction in creep deformation that occurred in the lightweight prisms subjected to a relaxing prestress force. A smaller difference would exist with the normal-weight prisms since the stress reduction was less for the material.

In utilizing prestressed members, different increments of dead load may be applied at significantly different times, an illustration being the time elapsing between prestressing of beams and casting of deck on bridges. In some instances the total load is applied in a short time and the assumption that it was all applied at the same time would be satisfactory for calculating losses. In other cases, where more time elapses between the application of increments of load, it is necessary to consider the effect of the individual loads. For this condition, the shape of the creep curve is involved and an equation to express the shape is convenient. For the prisms subjected to constant stress, the deformation curves had the shape:

$$\Delta = 0.150 \log_e (t + 3) - 0.19$$

where Δ = fraction of the total deformation from creep plus shrinkage, and t = time after stressing in days.

This equation is equal to 1.00 at 2,800 days which can be assumed to cover the total deformation. Actually, a semilog plot of the data is still linear at the termination of the measurements. Thus no finite terminal value is indicated; however, the rate of increase is negligible. Creep and shrinkage were not separated on this portion of the project so the equation is for the total deformation in the lightweight concrete prisms. There is no significant difference between the shape of curves for the different lightweight aggregates.

SIGNIFICANCE OF MEASUREMENTS AS APPLIED TO DESIGN

The designer of prestressed members must estimate the magnitude of creep, shrinkage, and relaxation losses that may occur in the materials he intends to use. Typical allowances are available for normal-weight concrete, but the information about the properties of lightweight concrete is limited. Present AASHO specifications permit the use of an assumed total loss in the prestressing steel of 25,000 psi for posttensioned members and 35,000 psi for pretensioned members constructed from typical normal-weight concrete. Values for lightweight concretes must be estimated from tests on

mixtures from the particular aggregates. The AASHO values for loss of stress are similar in magnitude to those calculated from recommended allowances in other literature on prestressed concrete design.

In most of the design literature, recommended allowances for shrinkage range from 0.0002 to 0.0004 in. per in. On this project the shrinkage of the normal-weight concrete was 0.00072 in. per in. which results in a stress loss from shrinkage much greater than that anticipated by the design recommendations. Although the measured shrinkage seems high, many investigators have measured similar values on good quality concretes. Studies of concretes in an outdoor exposure where free moisture in the form of rain or fog is periodically available have shown lower values of shrinkage (4) than laboratory stored specimens even though the average relative humidity was the same in each case. Except for very sheltered locations or use in arid climates, shrinkage allowances somewhat lower than the laboratory-measured values should be satisfactory. Values of about 0.0004 in. per in. were measured on specimens studied in connection with the AASHO Road Test. An allowance of this magnitude would seem reasonable for normal-weight concrete in typical outdoor exposures.

The total shrinkage of the concrete using aggregate B was very similar to that of the normal-weight concrete, but initially, the lightweight concretes increased in length while the normal-weight concrete decreased. For posttensioned members, this initial swelling leaves more of the shrinkage to occur after stressing, resulting in a proportionally greater loss of prestress.

Under like storage conditions, the prisms cast from aggregate A had a total shrinkage about 35 percent greater than aggregate B prisms and 40 percent greater than the normal-weight prisms. For concrete in an outdoor exposure, this percentage difference would probably remain similar so that if an allowance of 0.0004 in. per in. is satisfactory for normal-weight concrete, about 0.0006 in. per in. should be anticipated in aggregate A concrete.

The measured values of shrinkage occurring after the prisms were stressed would cause stress losses of about 14,000 psi for the sand and gravel concrete, 21,000 psi for the aggregate B concrete, and 28,000 psi for the aggregate A concrete, assuming a modulus of elasticity of 28×10^6 psi for the steel.

The average end-to-end stress at the centroid of the prestressing steel will undoubtedly vary appreciably between beam designs, but a value of about 1,000 psi was determined for several beams that were reviewed. If an average stress of 1,000 psi is assumed and the creep coefficients based on the initial stress are applied, the loss of stress from creep would be about 8,400 psi for normal-weight concrete, 16,000 psi for aggregate B concrete, and 14,000 psi for aggregate A concrete.

If a 4 percent relaxation loss is assumed for the steel and the steel is stressed to 170,000 psi, the relaxation loss would be 6,800 psi for all beams. Summing the values from shrinkage, creep, and relaxation gives a total indicated loss of 29,200 psi for normal-weight concrete, 43,800 psi for aggregate B concrete, and 48,000 psi for aggregate A concrete. These values are for a posttensioned application in which the shrinkage characteristics resembled those of the laboratory specimens. For beams in an outdoor exposure, the shrinkage would probably be reduced appreciably which would, in turn, reduce the loss of prestress.

If concretes having the same properties were used in pretensioned members which were subsequently stored in the laboratory atmosphere, the loss of prestress would be about 41,400 psi for the sand and gravel concrete, 57,600 psi for aggregate B concrete, and 63,500 psi for aggregate A concrete. These values are based on the measured deformations, an assumed initial stress of 1,000 psi in the concrete, initial steel stress of 170,000 psi, 4 percent relaxation loss, and sheltered indoor storage.

For posttensioned members, a comparison of values in the preceding paragraphs shows that the loss of prestress of aggregate A concrete is about 67 percent greater than the normal-weight concrete and the loss for aggregate B concrete is about 50 percent greater than the normal-weight concrete. For pretensioned applications, a comparison shows the losses are 53 percent greater for aggregate A concrete and 39 percent greater for aggregate B concrete than the value estimated for the sand and gravel concrete. The high shrinkage of the aggregate A concrete is responsible for the higher losses in this concrete.

SUMMARY

These tests have shown that concretes produced with expanded shale aggregates can provide adequate strength and workability for use in prestressed bridge members. The cement requirement is rather high for the aggregates and gradations used in this study. A nominal cement factor of 9 sacks per cu yd was used. With carefully controlled construction, drier, harsher mixes could probably be used to advantage. With a smaller proportion of fine aggregate, suitable strengths can be obtained with less cement. A reduction in the cement paste would be expected to reduce the shrinkage and probably improve the creep characteristics of the concrete.

For concretes having similar strengths and workability, the use of expanded shale aggregates permits a reduction in weight of approximately 30 percent. The wet weights of the different expanded shale concretes ranged from 102 pcf to 108 pcf while the wet weight of the sand and gravel concrete was 153 pcf. The reduced weight of expanded shale members would make their transportation and erection more economical and in some instances, the reduced weight may permit design alternatives not possible with normal-weight concrete.

The modulus of elasticity of concretes produced from expanded shales is less than one-half as great as for normal-weight concrete of similar strength. The values for the lightweight concretes in this study were around 2×10^6 psi. The low modulus of elasticity is an important consideration in several aspects of design. In pretensioned members, the elastic deformation contributes directly to loss of prestress. Also, the magnitude of camber and deflection would be increased in members having a low modulus of elasticity.

The shrinkage of the concretes used in this study was higher than would ordinarily be anticipated for structural quality concrete in an outdoor application, although other investigators have found similar values in laboratory studies. The shrinkage of the aggregate A concrete was about 41 percent greater than that of the sand and gravel mixture whereas the aggregate B concrete exceeded the sand and gravel concrete by only 4 percent. The amount of shrinkage occurring in the test prisms represents a source of appreciable loss of prestress. An outdoor exposure would probably reduce the shrinkage. Also, concretes having lower mixing water contents would probably exhibit less shrinkage.

By subtracting the measured shrinkage from the total deformation and dividing the remaining creep deformation by the initial stress, creep coefficients were obtained for the prisms subjected to a relaxing prestress force. From these values, the creep of the concrete from aggregate A was 67 percent higher than the normal-weight concrete and the creep of the aggregate B concrete was 90 percent higher than the value for the sand and gravel concrete.

The ultimate loss of prestress indicated by the pressure cells averaged 30 percent for each of the lightweight aggregates, whereas the value for the normal-weight aggregate was 20 percent. Thus, the measured losses are 50 percent greater in each of the lightweight concretes than in the normal-weight concrete. These values include losses due to shrinkage, creep, and relaxation and, although some variation might be expected for different stress levels and different exposures, the percentage difference is likely to remain close to this value. The calculated values discussed earlier indicated total deformations of the lightweight concretes exceeding the normal-weight by 50 percent for aggregate B and 67 percent for aggregate A. Although the values for aggregate A concrete differ, the general range is indicated.

The greater deformations occurring in the lightweight concretes and the resultant increases in loss of prestress would obviously be significant in design; however, with knowledge of the approximate magnitude of the deformations, the expanded shale aggregate concretes can be used satisfactorily in prestressed members. In some applications, the reduced weight would offer important benefits.

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