2

MATERIALS

2-1 Concrete, Strength Requirements

Stronger concrete is usually required for prestressed than for reinforced work. Present practice in this country calls for 28-day cylinder strength of 4000 to 8000 psi (28 to 55 N/mm^2) for prestressed concrete, while the corresponding value for reinforced concrete is around 3500 psi (24 N/mm^2). The usual cube strength specified for prestressed concrete in Europe is about 450 kg/cm^2, based on 10-, 15-, or 20-cm cubes at 28 days. If cube strength is taken as 1.25 times the cylinder strength, this would correspond to

$$450 \times 14.2/1.25 = 5100 \text{ psi cylinder strength}$$

Although the above are the usual values, strengths differing from these are occasionally specified.

Higher strength is necessary in prestressed concrete for several reasons. First, in order to minimize their cost, commercial anchorages for prestressing steel are always designed on the basis of high-strength concrete. Hence weaker concrete either will require special anchorages or may fail under the application of prestress. Such failures may take place in bearing or in bond between steel and concrete, or in tension near the anchorages. Next, concrete of high compressive strength offers high resistance in tension and shear, as well as in bond and bearing, and is desirable for prestressed-concrete structures whose various portions are under higher stresses than ordinary reinforced concrete. Another factor is that high-strength concrete is less liable to the shrinkage cracks which sometimes occur in low-strength concrete before the application of prestress. It also has a higher modulus of elasticity and smaller creep strain, resulting in smaller loss of prestress in the steel.

Experience has shown that 4000- to 5000-psi (28 to 34 N/mm^2) strength will generally work out to be the most economical mix for prestressed concrete. Although the strength of concrete to be specified for each job must be considered individually, there are some evident reasons why the economical mix usually falls within a certain range. Concrete strength of 4000 to 6000 psi (28 to 41 N/mm^2) can be obtained without excessive labor or cement. The cost of 6000-psi (41 N/mm^2) concrete averages about 15% higher than that of 3000-psi (21 N/mm^2) concrete, while it has 100% higher strength, which can be well utilized and is often seriously needed in prestressed structures. To obtain strength much greater than 6000 psi (41 N/mm^2), on the other hand, not only

will cost more but also will call for careful design and control of the mixing, placing, and curing of concrete which cannot be easily achieved in the field.

Strength of 6000 to 8000 psi (41–55 N/mm^2) will sometimes be specified for precast, prestressed concrete beams. These strengths are commonly attained in plant operations where good quality control can be assured. Higher strengths, although sometimes adopted, are not in common use at this time.

To attain a strength in excess of 5000 psi (34 N/mm^2), it is necessary to use a water-cement ratio of not much more than 0.45 by weight. In order to facilitate placing, a slump of 2 to 4 in. (51 to 102 mm) would be needed, unless more than ordinary vibration is to be applied. To obtain 3-in. (76 mm) slump with water-cement ratio of 0.45 would require about 8 bags of cement per cu yd of concrete. If careful vibration is possible, concrete with $\frac{1}{2}$-in. (13 mm) or zero slump can be employed, and 7 bags of cement per cu yd may be quite sufficient. Since excessive cement tends to increase shrinkage, a lower cement factor is desirable. To this end, good vibration is advised whenever possible, and proper admixtures to increase the workability can sometimes be advantageously employed.

Not only should high-strength concrete be specified for prestressed work, but, when called for, such strength should be more closely attained in the field than for reinforced concrete. Indeed, more parts of prestressed-concrete members are subjected to high stresses than in reinforced concrete. Consider a simple prestressed beam, for example. While the top fibers are highly compressed under heavy external loads, the bottom fibers are under high compression at the transfer of prestress. While the midspan sections resist the heaviest bending moments, the end sections carry and distribute the prestressing force. Hence, in a prestressed member, it is more important to secure uniformity of strength, whereas in reinforced concrete the critical sections are relatively limited. Many engineers believe that, if the concrete is not crushed under the application of prestress, it should be able to stand subsequent loadings, since the strength of concrete increases with age and since excessive overloads are very rare for many structures. Fortunately, a 10% understrength of the concrete will result in very little change in strength of a member, but the engineer should use reasonable precautions to obtain the concrete strength specified.

It is general practice to specify a lower strength of concrete at transfer than its 28-day strength. This is desirable in order to permit early transfer of prestress to the concrete. At transfer, the concrete is not subject to external overloads, and strength is necessary only to guard against anchorage failure and excessive creep, hence a smaller factor of safety is considered sufficient. For example, in pretensioning work, a strength of 3500 psi (24 N/mm^2) at transfer is often sufficient for a specified 28-day strength of 5000 psi (34 N/mm^2).

Direct tensile strength in concrete is a highly variable item, generally ranging from $0.06f'_c$ to $0.10f'_c$, and may be zero if cracks have developed as the result of shrinkage or other reasons. Modulus of rupture in concrete is known to be higher than its direct tensile strength; ACI Code suggests $7.5\sqrt{f_c'}$ as an estimate of modulus of rupture.

Direct shearing strength, not often used in design, ranges from $0.50f_c'$ to $0.70f_c'$. Beam shear produces the principal tensile stress, whose limiting value is commonly gaged on the basis of direct tensile strength in concrete. Beam shear strength is covered in Chapter 7.

2-2 Concrete, Strain Characteristics

In prestressed concrete, it is important to know the strains produced as well as the stresses. This is necessary to estimate the loss of prestress in steel and to provide for other effects of concrete shortening. For the purpose of discussion, such strains can be classified into four types: elastic strains, lateral strains, creep strains, and shrinkage strains.

Elastic Strains. The term elastic strains is perhaps a little ambiguous, since the stress-strain curve for concrete is seldom a straight line even at normal levels of stress, Fig. 2-1. Neither are the strains entirely recoverable. But, eliminating the creep strains from consideration, the lower portion of the instantaneous stress-strain curve, being relatively straight may be conveniently called elastic. It is then possible to obtain values for the modulus of elasticity of concrete. The modulus varies with several factors,[1,2] notably the strength of concrete, the age of concrete, the properties of aggregates and cement, and the definition of modulus of elasticity itself, whether tangent, initial, or secant modulus. Furthermore, the modulus may vary with the speed of load application and with the

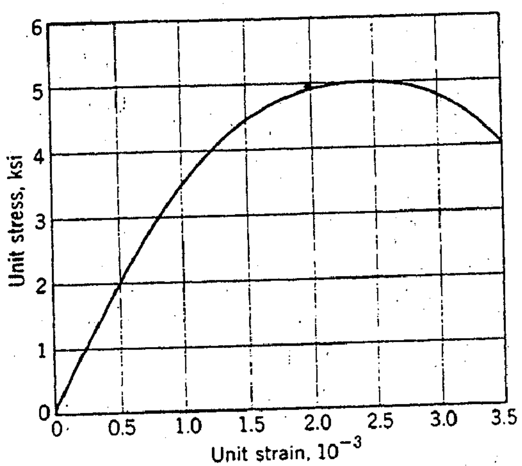

Fig. 2-1. Typical stress-strain curve for 5000 psi (34 N/mm²) concrete.

type of specimen, whether a cylinder or a beam. Hence it is almost impossible to predict with accuracy the value of the modulus for a given concrete.

As an average value for concrete at 28 days old, and for compressive stress up to about $0.40f_c'$, the secant modulus has been approximated by the following empirical formulas.

A. The ACI Code for Reinforced Concrete specifies the following empirical formula:

$$E_c = w^{1.5}33\sqrt{f_c'} \tag{2-1}$$

where the unit weight w varies between 90 and 155 lb per ft³ (1443 and 2485 kg/m³). For normal weight concrete this expression may be simplified as:

$$E_c = 57{,}000\sqrt{f_c'}$$

B. Empirical formula proposed by Jensen:

$$E_c = \frac{6\times 10^6}{1+(2000/f_c')} \tag{2-2}$$

which gives more correct values for f_c' around 5000 psi (34 N/mm²).

C. Empirical formula proposed by Hognestad:

$$E_c = 1{,}800{,}000 + 460f_c' \tag{2-3}$$

which gives results similar to the last one.

Plotting the above proposals in Fig. 2-2, we can see that those of Jensen and Hognestad come quite close to the ACI values. Some equations for E_c give values intended to represent modulus used for computing instantaneous beam deflections while the others based on measured strains from cylinder specimens.

Authorities differ on the relation between the two kinds of moduli. Some tests indicate the agreement of these two values; others tend to show that the modulus for beams is higher than that for cylinders. Not too much work has been done for the modulus of elasticity of concrete in tension, but it is generally assumed that, before cracking, the average modulus over a length of several inches is the same as in compression, although the local modulus in tension is known to vary greatly.

Lateral Strains. Lateral strains are computed by Poisson's ratio.[2] Owing to Poisson's ratio effect, the loss of prestress is slightly decreased in biaxial prestressing. Poisson's ratio varies from 0.15 to 0.22 for concrete, averaging about 0.17.

Creep Strains. Creep of concrete is defined as its time-dependent deformation resulting from the presence of stress. A great deal of work has been done in this country on the creep or plastic flow of concrete.[3,4]

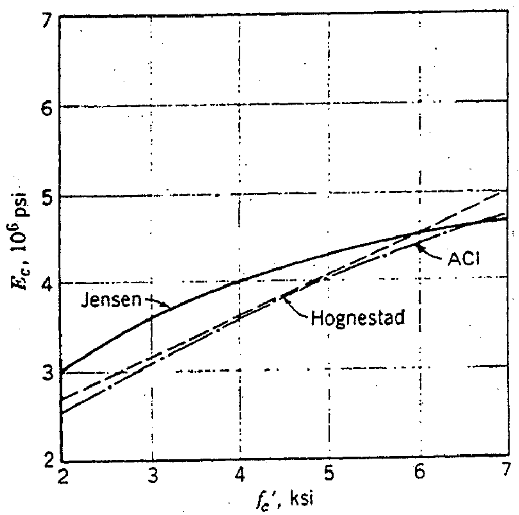

Fig. 2-2. Empirical formulas for E_c for normal weight concrete.

A brief summary of a comprehensive investigation carried out at the University of California extending over a period of 30 years is now presented.[5] For specimens of 4-in. (102 mm) diameter loaded in compression to 800 psi (5.52 N/mm^2) at 28 days and thereafter stored in air at 50% relative humidity and 70°F, the findings are:

1. Creep continued over the entire period, but the rate of change at the later ages was very small. Of the total creep in 20 years, 18–35% occurred in the first two weeks of loading, 40–70% within 3 months, and 60–83% within 1 year. The average values were 25, 55, and 76% respectively. Typical creep-time ratio curves with upper and lower limits are shown in Fig. 2-3 (from reference 5).
2. Creep increased with a higher water-cement ratio and with a lower aggregate-cement ratio, but was not directly proportional to the total water content of the mix.
3. Creep of concrete was appreciably greater for type IV (low-heat) than for the type I (normal) portland cement. For type IV cement the creep was greater for the coarse grind than for the fine, but the reverse was true for the type I cement.
4. Creep of concrete was greatest for crushed sandstone aggregate, followed in descending order by basalt, gravel, granite, quartz, and limestone. The creep for sandstone concrete was more than double that for limestone concrete.

For ages from 28 to 90 days at time of loading, for stresses from 300 to 1200 psi (2.07 to 8.27 N/mm^2), for storage conditions which ranged from air at 50%

relative humidity to immersion in water, and for specimen diameters from 4 to 10 in. (102–524 mm), the following statements apply.

1. The older the specimen at the time of loading, the more complete the hydration of cement, the less the creep. Those loaded at 90 days had less creep than those at 28 days, by roughly 10%.
2. The creep per unit of stress was only slightly greater at the high stresses than at the low stresses.
3. The total amount of creep strain at the end of 20 years ranged from 1 to 5 times the instantaneous deformations (averaging about 3 times) while the combined shrinkage or swelling and creep ranged from 1 to 11 times the instantaneous deformations, the low values occurring for storage in water or fog and for limestone aggregates.
4. The creep in air at 50% relative humidity was about 1.4 times that in air at 70% relative humidity and about 3 times that for storage in water.
5. Creep decreased as the size of specimen increased.

Only a limited amount of data is available concerning the creep of concrete under high stress.[6] Some of these data seem to indicate that when the sustained stress is in excess of about $\frac{1}{3}$ of the ultimate strength of concrete, the rate of increase of strain with stress tends to get higher. It is possible that this increase can become quite pronounced as the stress approaches the ultimate strength of the concrete.

Upon the removal of the sustained stress, part of the creep can be recovered in the course of time.[3] Generally, it takes a longer time to recover the creep than for the creep to take place. For the limited amount of data available, it can be stated that roughly 80 to 90% of the creep will recover during the same length of time that creep has been allowed to take place.

In Europe, the term creep coefficient C_c is employed to indicate the total strain δ_t (instantaneous plus creep strain) after a lengthy period of constant stress to the instantaneous strain δ_i immediately obtained upon the application of stress,[7] thus

$$C_c = \frac{\delta_t}{\delta_i}$$

This coefficient varies widely as reported from different tests, essentially because of the difficulty of separating shrinkage from creep. For purposes of design, it is considered safe to take C_c as around 3.0. For posttensioned members, where the prestress is applied late, the coefficient could be a little less; for pretensioned members, where the prestress is applied at an early age, the coefficient could be a little more.

This same term, creep coefficient, is sometimes used to denote the ratio of the creep strain δ_c (excluding the instantaneous strain) to the instantaneous strain δ_i, thus

$$C_c = \frac{\delta_c}{\delta_i}$$

Hence care should be exercised to find out the exact meaning of "creep coefficient" whenever the term is employed. Using the first definition, the creep coefficient is about 2.5 at the end of one year for the curve in Fig. 2-3; using the second definition, that same coefficient is only 1.5.

Of the total amount of creep strain, it can be roughly estimated that about $\frac{1}{4}$ takes place within the first 2 weeks after application of prestress, another $\frac{1}{4}$ within 2 to 3 months, another $\frac{1}{4}$ within a year, and the last $\frac{1}{4}$ in the course of many years, Fig. 2-3.

There is good reason to believe that, for smaller members, creep as well as shrinkage takes place faster than for larger members. Upon the removal of stress, part of the creep can be recovered in the course of time. Again, owing to the difficulty of separating shrinkage from creep, the amount and speed of such recovery have not been accurately measured.

Shrinkage Strains. As distinguished from creep, shrinkage in concrete is its contraction due to drying and chemical changes dependent on time and on moisture conditions, but not on stresses. At least a portion of the shrinkage resulting from drying of the concrete is recoverable upon the restoration of the lost water. The magnitude of shrinkage strain also varies with many factors, and it may range from 0.0000 to 0.0010 and beyond. At one extreme, if the concrete is stored under water or under very wet conditions, the shrinkage may be zero. There may even be expansion for some types of aggregates and cements. At the

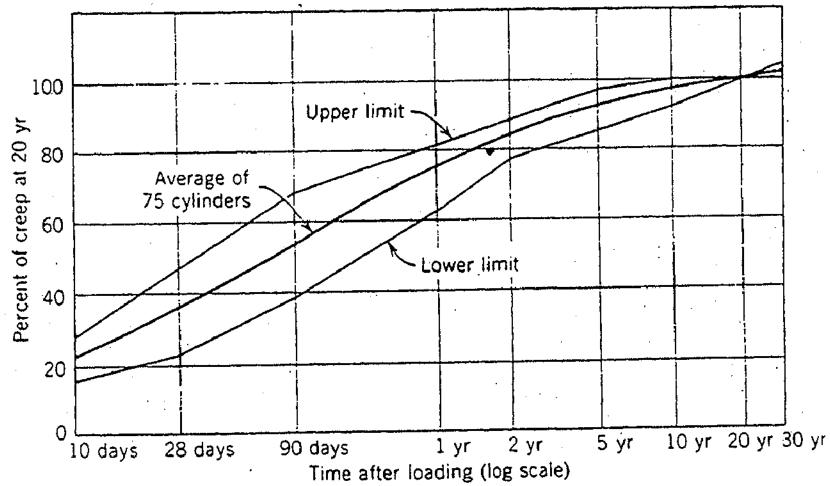

Fig. 2-3. Creep-time ratio curves.

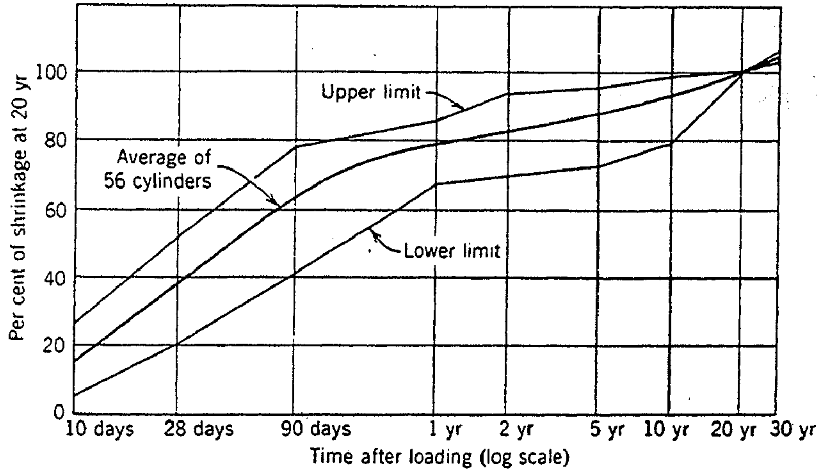

Fig. 2-4. Drying shrinkage-time ratio curves.

other extreme, for a combination of certain cements and aggregates, and with the concrete stored under very dry conditions, as much as 0.0010 can be expected. Reference 5 lists test results showing the magnitude of shrinkage and its rate of occurrence as affected by various factors. Fig. 2-4 shows some typical shrinkage-time ratio curves taken from that reference.

Shrinkage of concrete is somewhat proportional to the amount of water employed in the mix. Hence, if minimum shrinkage is desired, the water-cement ratio and the proportion of cement paste should be kept to a minimum. Thus aggregates of larger size, well graded for minimum void, will need a smaller amount of cement paste, and shrinkage will be smaller.

The quality of the aggregates is also an important consideration. Harder and denser aggregates of low absorption and high modulus of elasticity will exhibit smaller shrinkage. Concrete containing hard limestone is believed to have smaller shrinkage than that containing granite, basalt, and sandstone of equal grade, approximately in that order. The chemical composition of cement also affects the amount of shrinkage. For example, shrinkage is relatively small for cements high in tricalcium silicate and low in the alkalies and the oxides of sodium and potassium.

The amount of shrinkage varies widely, depending on the individual conditions. For the purpose of design, an average value of shrinkage strain would be about 0.0002 to 0.0006 for the usual concrete mixtures employed in prestressed construction. The rate of shrinkage depends chiefly on the weather conditions. Actual structures exposed to weather show measurable seasonal changes in the shrinkage of concrete—swelling during rainy reasons and shrinking during dry ones. If the concrete is left dry, there is reason to believe that most of the shrinkage would take place during the first 2 or 3 months. If it always wet, there may be no shrinkage at all. When stored in air at 50% relative humidity and 70°F, there were indications[5] that the rate of occurrence of shrinkage is comparable to that of creep and that the magnitude of shrinkage is often similar to that of creep produced by a sustained stress of about 600 psi (4.14 N/mm^2).

2-3 Concrete, Special Manufacturing Techniques

Most of the techniques for manufacturing good concrete, whether for plain or reinforced work, can be applied to prestressed concrete. However, they must be investigated for a few factors peculiar to prestressed concrete. First, they must not decrease the high strength required; next, they must not appreciably increase the shrinkage and creep; they must not produce adverse effects, such as inducing corrosion in the high-tensile wires.

Compacting the concrete by vibration is usually desirable and necessary. Either internal or external vibration may be used. In order to produce high-strength concrete without using an excessive amount of mortar, a low water-cement ratio and a low-slump concrete must be chosen. Such concrete cannot be well placed without compaction. There are only a few isolated applications in which concrete of high slump is employed and compaction may be dispensed with. But it will be found preferable to use at least a small amount of compaction for corners and around reinforcements and anchorages.

Good curing of concrete is most important. Too early drying of concrete may result in shrinkage cracks before the application of prestress. Besides, only by careful curing can the specified high strength be attained in concrete. In order to hasten the hardening process, steam curing is often resorted to in the precasting factory; it can also be employed in the field where the amount of work involved justifies the installation. When field work of casting must be carried out in cold weather, steam can profitably be used to raise the temperature of the ingredients and the placed concrete in order that high strength may be attained within a reasonable time.

Early hardening of the concrete is often desirable, either to speed plant production or to hasten field construction. Early high-strength concrete can be produced by any one of a number of techniques or combinations of techniques.[9] High early-strength cement or steam curing is commonly employed. Admixtures to accelerate the strength should be employed with caution. For example, calcium chloride, the most commonly used accelerator, even applied in normal amounts, will increase shrinkage. There is evidence that it will cause corrosion, which could be serious for the prestressing steel. When accelerators are used, care must also be taken not to have the initial set take place too soon.

Wetting admixtures to improve the workability of concrete may be found to be profitable, since they may permit easy placing of high-strength concrete without too high a cement content. Some of these admixtures tend to increase the shrinkage and may offset the advantage of saving cement. Each must be judged on its own merits in conjunction with the nature of the aggregates and cement. Air entrainment of 3 to 5% improves workability and reduces bleeding. When well-recognized, air-entraining agents are employed, there is no evidence of increased shrinkage or creep. Hence proper application of air entrainment is considered beneficial for prestressed concrete.

Precast segmental construction has been recently developed for prestressed bridges.[10] Breaking up a bridge superstructure into transverse segments reduces the individual weight and facilitates casting and handling. These segments can be mass produced in a plant where rigid inspection and control can be effected, or they can be poured-in-place on a traveling carriage. They are used for longer spans than could be handled with a single beam cast in one piece, thus enabling them to compete with structural steel on these larger spans. The joints for precast segments are very thin epoxy-filled space with the end surfaces being match cast for fit between segments. Posttensioning tendons are threaded through to join the segments together, thus forming the completed bridge.

2-4 Lightweight Aggregate Concrete

Since about 1955, lightweight concrete has been gaining in application to prestressed construction, especially in California. The main reason for using lightweight concrete is to reduce the weight of the structure, thus minimizing both the concrete and the steel required for carrying the load. This is especially important when the dead load is the major portion of the load on the structure, or when the weight of the member is a factor to be considered for transportation or erection.

It used to be a task to produce lightweight concrete of sufficient strength for prestressing, but this is no longer true. With experience in control and design of lightweight concrete mixes, 28-day cylinder strength of 5000 psi (34 N/mm^2) can generally be obtained with no difficulty, while 6000–7000 psi (41–48 N/mm^2) or more can be reached if desired. Strength at 1-day transfer of 3500 or 4000 psi is frequently attained by the use of high-early-strength cement and steam curing.

Data giving the physical and mechanical properties of lightweight concrete made with aggregates throughout the country are given in a paper by Shideler.[12] Those related to aggregates in the states of Texas and California are presented in two other papers.[13,14] While these test series did not yield identical values, some general observations can be made from them. When quantitative values are desired for a particular lightweight aggregate used in a given locality, it will be necessary to examine the aggregate and compare it with similar ones in the series, bearing in mind that exact values for either lightweight or regular weight concrete can be obtained only when extensive tests have been conducted for that particular aggregate, and the field conditions are under perfect control. Fortunately, a certain amount of tolerance is permissible so that when properly designed and built, prestressed, lightweight concrete will behave satisfactorily.

One objection against lightweight concrete for prestressing is its low modulus of elasticity, which indicates more elastic shortening under the same unit stress. This means that there is a slightly higher loss of prestress in the steel. It also means that for cast-in-place structures, a greater elastic movement will take place under the application of the prestress. As a rough approximation, it may be said that the E_c for lightweight concrete averages about 55% the E_c for regular weight concrete. For f_c' between 3000 and 6000 psi (21 and 41 N/mm^2), using 60% of Hognestad's formula for E_c (p. 43), we get a fairly good approximation,

$$E_c = 1{,}000{,}000 + 250 f_c'$$

where f_c' = cylinder strength of concrete at the time E_c is measured. However, E_c values may easily vary 20% either way from those given by the above formula, depending on various factors, especially the nature of the lightweight aggregate. The ACI equation (2-1) uses unit weight w directly to obtain E_c for lightweight concrete. Figure 2-2(b) simplifies use of the ACI equation for estimating E_c for lightweight concrete.

Poisson's ratio for lightweight concrete is apparently comparable to that for sand and gravel concrete; values between 0.15 and 0.25 have been reported with an average value of 0.19.[12]

The tensile strength for lightweight concrete varies with different types of manufactured material. The properties of concrete made with a particular lightweight aggregate should be available from the aggregate manufacturer, and actual results from tests should be used when available. The ACI Code uses split cylinder strength as an index for evaluating cracking and shear strength in lightweight concrete. For normal weight concrete the average splitting tensile strength, f_{ct}, is approximately $6.7\sqrt{f_c'}$, but it falls below this value for some lightweight concrete. The ACI Code provides that shear strength be modified using actual $f_{ct}/6.7$ for $\sqrt{f_c'}$ in formulas for shear strength. When the actual f_{ct} is not specified, all values of $\sqrt{f_c'}$ affecting shear strength and cracking moment are multiplied by 0.75 for "all lightweight concrete," and 0.85 for "sand-lightweight concrete." Linear interpolation between these values may be used when partial sand replacement is used. The unit weight of lightweight concrete varies considerably, between 90 and 110 pcf (1443 and 1763 kg/m^3). The addition of fine, natural sand somewhat increases the unit weight and is also known to increase the workability and strength of the mix.

The shrinkage of lightweight concrete is apparently comparable to that of similar sand and gravel concrete.[13] However, some tests showed that it was slightly higher by 6–38%;[12] while other tests indicated that it was much lower.[14] Hence it is concluded that each lightweight aggregate must be studied by itself, but the chances are that they will have no more shrinkage than sand and gravel concrete.

Total creep strain in lightweight concrete is again comparable to that in sand and gravel concrete for specimens under the same sustained stress. Some have higher creep while others have less, probably by a maximum of some 20%, one way or the other. For detailed information, readers are referred to references 12, 13, and 14. It is generally agreed that both shrinkage and creep are related to the cement paste and quite independent of the aggregates.

2-5 Self-stressing Cement

Types of cements that expand chemically after setting and during hardening are known as expansive or self-stressing cements. When these cements are used to make concrete with embedded steel, the steel is elongated by the expansion of the concrete. Thus the steel is prestressed in tension, which in turn produces compressive prestress in the concrete, resulting in what is known as chemical prestressing or self-stressed concrete.

Modern development of expansive cement started in France about 1940.[15] Its use for self-stressing has been investigated intensively in U.S.S.R. since 1953.[16] At the University of California, Berkeley, studies were directed toward the use of calcium sulfoaluminate admixtures for expansive cements in 1956, and their practical chemistry, manufacture, and potentials were analyzed and described in a paper by Klein and Troxell.[17] The physical properties of one such expansive cement were then further investigated, and results were presented in a paper by Klein, Karby, and Polivka,[18] while pilotary effort to study the structural possibilities of such expansive cement, when used for prestressing concrete, was described in another paper by Lin and Klein.[19]

When concrete made with expanding cement is unrestrained, the amount of expansion produced by the chemical reaction between the cement and water could amount to 3–5%, and the concrete would then disintegrate by itself. When restrained either internally or externally with steel or other means, the amount of expansion can be controlled. By applying restraint in one direction, the growth in the other two orthogonal directions can be limited because of the crystalline nature of the hardened paste. The Russian self-stressing cement requires hydrothermal curing resulting in a quick setting, while the component developed in California requires water or fog curing under normal temperatures.

When high-tensile steel is used to produce the prestress, say corresponding to tensile stress at 150,000 psi (1,034 N/mm^2) and an E_s of 27,000,000 psi (186 kN/mm^2), an expansion of

$$\frac{150{,}000}{27{,}000{,}000} = 0.55\%$$

is required. For other stress levels, varying amounts of expansion will be required. For proper development of the bond between steel and concrete, mechanical end-anchorages might be necessary unless the steel has sufficient corrugation to transfer the stress.

Because of the expansion in all three directions, it seems difficult to use the cement for complicated structures cast in place, such as buildings. However, for pressure pipes and pavements, where prestressing in at least two directions is desired, this type of chemical prestressing can be more economical than mechanical prestressing; this is also true for precast slabs, walls, and shells. However, no immediate economy is seen in the making of beams which require eccentric prestressing. Unless the beam soffit is precast by itself, curvature of the beam may result from steel embedded eccentrically in the beam.

Expanding cement has been successfully applied for many interesting projects, especially in France. When a concrete block of expanding cement is cast as the keystone for a concrete arch, it serves as a jack, producing the desired arch compensation to balance rib shrinkage and shortening. When used for underpinning buildings, it tends to lift the structure without jacking. It can be used for pressure grouting or for producing concrete pavements and slabs with no shrinkage joints.

While many problems remain yet unsolved concerning the use of expanding cement for self-stressing, such as the chemical and physical stability and exact control of the stresses and strains, applications for a limited amount of shrinkage compensation are found in the United States, while in the Soviet Union, it is frequently used for stressing precast elements for thin shells which require only concentric prestressing.

2-6 Steels for Prestressing

High-tensile steel is almost the universal material for producing prestress and supplying the tensile force in prestressed concrete. The obvious approach toward

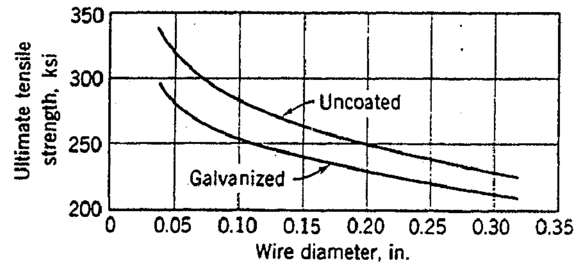

Fig. 2-5. Typical variation of wire strength with diameter.

the production of high-tensile steel is by alloying, which permits the manufacture of such steels under normal operation. Carbon is an extremely economical element for alloying, since it is cheap and easy to handle.[20] Other alloys include manganese and silicon. Other approaches are by controlled cooling of the steels after rolling and by heat treatment such as quenching and tempering. Beneficial results have been obtained by quenching from the rolling heat at a given temperature and also by interrupting the quench at a given temperature.

The most common method for increasing the tensile strength of steel for prestressing is by cold-drawing, high-tensile steel bars through a series of dyes. The process of cold-drawing tends to realign the crystals, and the strength is increased by each drawing so that the smaller the diameter of the wires, the higher their ultimate unit strength. The ductility of wires, however, is somewhat decreased as a result of cold-drawing. A curve giving the typical variation of strength with diameter is shown in Fig. 2-5. The actual strength, of course, will vary with the composition and manufacture of the steel.

High-tensile steel for prestressing usually takes one of three forms: wires, strands, or bars. For posttensioning, wires are widely employed; they are grouped, in parallel, into cables. Strands are fabricated in the factory by twisting wires together, thus decreasing the number of units to be handled in the tensioning operations. Strands, as well as high-tensile rods, are also used for posttensioning.

For pretensioning, 7-wire strands are almost exclusively used in the United States and have replaced much of wire pretensioning in other countries. Although strands cost slightly more than wires of the same tensile strength, its better bonding characteristics make it especially suitable for pretensioning.

While the ultimate strength of high-tensile steel can be easily determined by testing, its elastic limit or its yield point cannot be so simply ascertained, since it has neither a yield point nor a definite proportional limit. Various arbitrary methods have been proposed for defining the yield point of high-tensile steel, such as the 0.1% set, 0.2% set, 0.7% strain, or 1.0% strain. The more commonly accepted methods are probably the 0.2% set and the 1.0% strain.

Table 2-1 Properties of Uncoated Stress-Relieved Wire (ASTM A 421)

Nominal Diameter in. (mm)	Minimum Tensile Strength psi (N/mm^2) Type BA[b]	Type WA	Minimum Stress at 1% Extension psi (N/mm^2) Type BA[b]	Type WA
0.192 (4.88)	[a]	250,000 (1725)	[a]	200,000 (1380)
0.196 (4.98)	240,000 (1655)	250,000 (1725)	192,000 (1325)	200,000 (1380)
0.250 (6.35)	240,000 (1655)	240,000 (1655)	192,000 (1325)	192,000 (1325)
0.276 (7.01)	[a]	235,000 (1622)	[a]	188,000 (1295)

[a] These sizes are not commonly furnished in Type BA wire.
[b] Type BA wire is used for applications in which cold-end deformation is used for anchoring purposes (button anchorage), and type WA is used for applications in which the ends are anchored by wedges and no cold-end deformation of the wire is involved (wedge anchorage). Examples of tendons with button anchorages, more common in United States practice, are shown in Appendix B.

2-7 Steel Wires

Wires for prestressing generally conform to ASTM Specification A-421 for "Uncoated Stress-relieved Wire for Prestressed Concrete." They are made from rods produced by the open hearth or electric-furnace process. After cold-drawn to size, wires are stress-relieved by a continuous heat treatment to produce the prescribed mechanical properties.

The tensile strength and the minimum yield strength (measured by the 1.0% total-elongation method) are shown in Table 2-1 for the common sizes of wires.

A typical stress-strain curve for a stress-relieved $\frac{1}{4}$-in. wire conforming to the ASTM A-421 is shown in Fig. 2-6, with a typical modulus of elasticity of

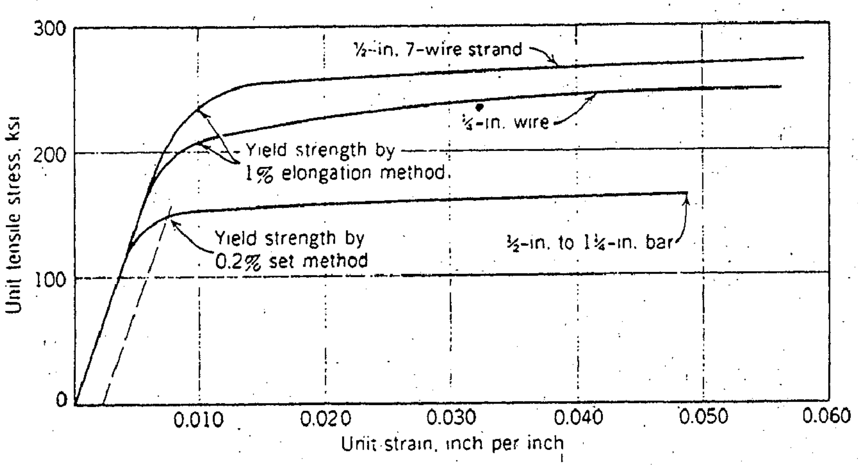

Fig. 2-6. Typical stress-strain curve for prestressing steels.

29,000,000 psi (200 kN/mm^2). The specified minimum elongation in 10 in. (254 mm) is 4.0%, while a typical elongation at rupture is more likely from 5 to 6%.

Curves for a bar and for a 7-wire strand are also shown in Fig. 2-6, and in Appendix B. Typical curves are considered to be sufficiently accurate for the purposes of structural design. For computing exact elongations, it is advised that accurate stress-strain relationships be obtained from the manufacturer or by actual testing of specimens.

Wires are supplied in reels or coils. They are cut to length and assembled either at the plant or in the field. Some steel may need a certain amount of degreasing and cleaning before placement, in order to ensure good bond with concrete. Loose rust or scale should be removed, but a firmly adherent rust film is considered advantageous in improving the bond.

In continental Europe, smooth wires 2 and 3 (sometimes 2.5) mm in diameter and corrugated wires of 4 and 5 mm have been employed in pretensioning work. Small wires possess higher unit strength and furnish better bond, which is helpful. In order to save labor and anchorage costs, larger wires are preferred for prestressing.

In England, wires are based on the British Imperial Gauge, No. 2 which has a diameter of 0.276 in., exactly 7 mm, while No. 6 has a diameter of 0.192 in., which is very close to 5 mm. Hence gage Nos. 2 and 6 are sometimes called for in posttensioning, while the exact equivalents of 7 and 5 mm (0.276 and 0.196 in., respectively) are also frequently employed. In Germany, corrugated wires with oval cross section have been used for posttensioning. These wires have areas of 20, 30, 35, and 40 mm^2, with Oval 40 having a major diameter of 11 mm and a minor diameter of 4.5 mm.

In this country, wires are manufactured according to the U.S. Steel Wire Gage, No. 2 which has a diameter of 0.2625 in. (6.668 mm) and No. 6 has a diameter of 0.1920 in. (4.877 mm). Neither of these is the exact equivalent of the millimeter counterparts. Hence, when the European types of anchorages are adopted, 0.276-in. and 0.196-in. wires are often specified. For posttensioning systems developed in the United States, $\frac{1}{4}$-in. (6.35 mm) wires have been most commonly incorporated.

2-8 Steel Strands

Strands for prestressing generally conform to ASTM Specification A-416 for "Uncoated Seven-wire Stress-relieved for Prestressed Concrete." Two grades are available, 250 ksi and 270 ksi (1,724 N/mm^2 and 1,862 N/mm^2), where the grade indicates minimum guaranteed breaking stress. While these specifications were intended for pretensioned, bonded, prestressed-concrete construction, they are also applicable to posttensioned construction, whether of the bonded or the unbonded type. These seven-wire strands all have a center wire slightly larger than the outer six wires which enclose it tightly in a helix with a uniform pitch between 12 and 16 times the nominal diameter of the strand. After stranding, all strands are subjected to a stress-relieving continuous heat treatment to produce the prescribed mechanical properties.

Seven-wire strands commonly used for prestressing conform to the ASTM A-416 Specifications, having a guaranteed minimum ultimate strength of 250,000 psi or 270,000 psi (1724 N/mm^2 or 1862 N/mm^2). Their properties are listed in Table 2-2. Since 1962, the stronger steel known as the 270K grade has been produced by various companies. For the same nomial size, the 270k grade has more steel area than the ASTM A-416 grade 250 and is about 15% stronger (see Appendix B). The 270K steel is now almost universally used for 7-wire strands in the United States, for both pretensioned and posttensioned structures. Low-relaxation strand is also available in both grades.

A typical stress-strain curve for a stress-relived $\frac{1}{2}$-in. (127 mm) 7-wire strand (ASTM A-416 is shown in Fig. 2-6 (and Appendix B), which is also typical for strands of all sizes. For calculations, a modulus of elasticity of 27,500,000 psi (189,610 N/mm^2) is often used for ASTM A-416 250K grade and 270K grade strand. The specified minimum elongation of the strand is 4% in a gage length of 24 in. (609.6 mm) at initial rupture, although typical values are usually in the range of 6%. When these strands are galvanized, they are about 15% weaker in strength and slightly lower in E_s depending on the amount of zinc coating used. The galvanized strand is not widely used in structural members and information about properties should be obtained from the manufacturer.

Table 2-2 Properties of Uncoated Seven-Wire Stress-Relieved Strand (ASTM A-416)

Nominal Diameter in. (mm)	Breaking Strength lb (kN)	Nominal Area of Strand in.^{2} (mm^2)	Minimum Load at 1% Extension lb (kN)
		Grade 250	
0.250 (6.35)	9000 (40.0)	0.036 (23.22)	7650 (34.0)
0.313 (7.94)	14,500 (64.5)	0.058 (37.42)	12,3000 (54.7)
0.375 (9.53)	20,000 (89.0)	0.080 (51.61)	17,000 (75.6)
0.438 (11.11)	27,000 (120.1)	0.108 (69.68)	23,000 (102.3)
0.500 (12.70)	36,000 (160.1)	0.144 (92.90)	30,600 (136.2)
0.600 (15.24)	54,000 (240.2)	0.216 (139.35)	45,900 (204.2)
		Grade 270	
0.375 (9.53)	23,000 (102.3)	0.085 (54.84)	19,550 (87.0)
0.438 (11.11)	31,000 (137.9)	0.115 (74.19)	26,350 (117.2)
0.500 (12.70)	41,300 (183.7)	0.153 (98.71)	35,100 (156.1)
0.600 (15.24)	58,600 (260.7)	0.217 (140.00)	49,800 (221.5)

As fabricated, 7-wire strands are several thousand feet long. When unwinding strands, care must be taken in laying them along the path to prevent kinking and permanent twisting of the strands.

2-9 Steel Bars

ASTM Specifications A-322 and A-29 are often applied to high-strength alloy steel bars. It is usually required that all such bars be proof-stressed to 90% of the guaranteed ultimate strength. Although the actual ultimate strength often reaches 160,000 psi (1,103 N/mm^2), the specified minimum is generally set at 145,000 psi (1,000 N/mm^2). A typical stress-strain curve for these bars is shown in Fig. 2-b from which it can be noticed that a constant modulus of elasticity exists only for a limited range (up to about 80,000 psi (552 N/mm^2) stress) with a value between 25,000,000 and 28,000,000 psi (172,375 and 193,060 N/mm^2).

The yield strength of high tensile bars is often defined by the 0.2% set method, as indicated in Fig. 2-b, where a line parallel to the initial tangent is drawn from the 0.002 strain, and its intersection with the curve is defined as the yield-strength point. Most specifications would call for a minimum yield strength at 130,000 psi (896 N/mm^2), though actual values are often higher. Minimum elongation at rupture in 20 diameters length is specified at 4%, with minimum reduction of area at rupture at 25%. Common sizes and properties of high-tensile rods for prestressing are listed in Appendix B.

High-tensile bars are available with length up to 80 ft. (24.4 m). Because of difficulty in shipping, the length may have to be further limited. But sleeve couplers are available to splice the bars to any desired length. These couplers have tapered threads in order to develop very nearly the full strength of the bars. They have outside diameters about twice that of the bar and a length about 4 times its diameter.

High-strength, specially deformed bars with ultimate strength of 160 ksi (1,103 N/mm^2) are available in sizes 1 to $1\frac{3}{8}$ in. (25.4 mm–34.9 mm) diameter. Ultimate strength of 230 ksi (1,586 N/mm^2) is available for these bars with $\frac{5}{8}$ in. (15.9 mm) diameter. The deformations on the bars serve as threads to fit couplers and anchorage hardware. Splicing of the bars can occur at any point where the bar may be cut since the coupler threads onto the special rolled-on deformations which are continuous along the length.

Auxiliary reinforcement using nonprestressed steel is commonly employed in prestressed construction. Steel of almost any strength will serve the purpose when properly designed. Generally, reinforcing bars conform to ASTM Specifications A-15, A-16, and A-305, and welded wire mesh conforms to ASTM Specification A-185.

2-10 Fiberglass Tendons

Fiberglass is manufactured by drawing fluid glass into fine filaments. The possible use of fiberglass for prestressing has been under investigation for some years.[21, 22, 23]

Although fiberglass has not yet been commercially applied in prestressed-concrete construction, it does possess certain superior qualities that indicate high promise for prestressing. An ultimate tensile strength of 1,000,000 psi (6,895 N/mm^2) is quite commonly obtained. Values as high as 5,000,000 psi (34,475 N/mm^2) have been reported for individual silica fibers 0.00012 in. (0.003048 mm) in diameter, it being known that the strength varies approximately inversely as the diameter of the fiber.

Fiberglass can be made in three forms: parallel chords, twisted strands, and parallel fibers embedded in plastic. The last form in the shape of fiberglass rods is considered most suitable for prestressing because of its relative simplicity for handling, gripping, and anchoring. At Princeton University, three types of resin have been tried out as bonding agents in the manufacture of fiberglass rods: polyester, epoxy, and polyamide resins. To date, the rods laminated with epoxy resins have appeared to be superior. A short-duration tensile strength has been obtained in excess of 220,000 psi (1,517 N/mm^2), based on the gross area of the rod.

Extensive research was conducted by the U. S. Army Corps of Engineers[23] on both reinforced and prestressed elements utilizing fiberglass rods and fiberglass tendons. For the prestressed test series, the fiberglass reinforced beams were found to be inferior to the prestressed beams with steel reinforcement based on an equal area of prestressing element. They recommended that fiberglass reinforcement be used in conjunction with conventional web reinforcement to avoid diagonal tension failures and that shallow depths be avoided. Other studies[24, 25] have shown that the behavior of fiberglass reinforced beam and slabs is predictable, but the long-term performance of posttensioned elements with fiberglass tendons seems questionable.

An advantage of fiberglass is its low modulus of elasticity, which ranges from 6,000,000 to 10,000,000 psi (41,370 to 68,950 N/mm^2). With its high stress and low modulus, the percentage of loss of prestress would be quite small. Other advantages claimed for this material are high resistance to acids and alkalies and the ability to withstand high temperature. However, some major problems must be solved before it can be applied in practice.

1. The static fatigue limit, that is, the long-time ultimate strength of fiberglass rods as opposed to the short-time ultimate strength. This is an important problem since it is known that the duration of loading has a pronounced effect on the ultimate strength.

2. The dynamic fatigue limit of fiberglass or fiberglass rods, although there is some evidence to indicate that this may not be a serious problem.
3. The chemical stability of fiberglass such as its reaction to the surrounding concrete, especially under wet conditions.
4. The best methods of fabricating cords from fiberglass to obtain an even distribution of stress so as to increase the ratio of the strength of cords to the strength of individual fibers. The minimizing of shearing deformation of the laminating material, since such deformation could result in the breaking of the outer fibers with an inner core of fibers remaining intact.
5. The design of suitable end anchorages, since the brittle material is liable to fail in the grip under the effect of stress concentrations and combined stresses.

If these problems can be solved, there still remains a last hurdle: the economics of the application of the material in competition with high-tensile steel, which is being produced in large quantities and at relatively low cost. On the other hand, it is conceivable that the special properties of fiberglass might make it desirable in special situations, especially in very corrosive industrial environments.

2-11 Auxiliary Materials—Grouting

Among the special auxiliary materials required for prestressed concrete are those for the provision of proper conduits for the tendons. For pretensioning, no such conduits are necessary. For posttensioning, there are two types of conduits, one for bonded, another for unbonded prestressing.

When the tendons are to be bonded, generally by grouting, the conduits (ducts) are made of ferrous metal which may be galvanized. Materials commonly used for these ducts are 22 to 28 gage galvanized or bright spirally wound or longitudinally seamed steel strips with flexible or semirigid seams. Rigid tubing is sometimes provided by stiffening rods placed within the ducts, or rigid tubes. Corrugated plastic ducts have also come into limited use recently.

It is also possible to form the duct by withdrawing steel tubing or rod before the concrete hardens. More frequently, the duct is formed by withdrawing extractable rubber cores buried in the concrete. Several hours after the completion of concreting, these cores can be withdrawn without much effort, because the lateral shrinkage of the rubber under a pull helps to tear the rubber away from the surrounding concrete. In order that the rubber cores may remain straight during concreting, they are stiffened internally by inserting steel pipes or rods into axial holes provided in the rubber. To maintain the cores in position during concreting, transverse steel rods are placed under and over them at 3- to 4-ft (0.91–1.22 m) intervals. Sometimes rubber tubes inflated from their normal diameter can be substituted for the above rubber cores. These tubes can then be deflated and withdrawn.

When the tendons are to be unbonded, plastic or heavy paper sheathing is frequently used, and the tendons are properly greased to facilitate tensioning and to prevent corrosion. Rust inhibitors are usually added to the grease together with additional compounds to ensure its uniform consistency in extremes of temperature. Asbestos fibers are often added to the grease to hold it together during application. Plastic tubes of the split type should be properly overlapped and taped along the seams, so as to seal them against any leakage of mortars, which might bind the tendons to the tubes. When papers are spirally wrapped around the tendons, care should be exercised in wrapping so as to avoid jamming of the papers when the tendons are tensioned.

For bonding the tendons to the concrete after tensioning (in the case of posttensioning), cement grout is injected, which also serves to protect the steel against corrosion. Entry for the grout into the cableway is provided by means of holes in the anchorage heads and cones, or pipes buried in the concrete members. The injection can be applied at one end of the member until it is forced out of the other end. For longer members, it can be applied at both ends until forced out of a center vent.

Either ordinary portland cement or high-early-strength cement may be used for the grout with water and sometimes fine sand.[26] Commercially available additives developed to assure that sound grouting is possible. These materials increase the workability of the grout and sometimes provide for shrinkage compensation. For some situations where the grouting is vertical in walls of considerable height, special additives which prevent the segregation of water from the grout should be added as tests have shown that the water rise to the top results in a portion of the tendon being ungrouted. Such an unprotected tendon may later have severe rusting which can produce failure. To ensure good bond for small conduits, grouting under pressure is desirable; however, care should be taken to ensure that the bursting effect of the pressure on the walls of the cable enclosure can be safely resisted. Machines for mixing and injecting the grouts are commercially available.

Although sand is not used in grouting practice in the United States, it may have advantages in tendons with large void areas. Fly ash and pozzolans are occasionally added as filler material in the United States. Grouting pressure generally ranges from 80 to 100 psi (0.55 to 0.69 N/mm^2) with a maximum pressure specified as 250 psi (1.72 N/mm^2). After the grout has discharged from the far end, that end is plugged and the pressure is again applied at the injecting end to compact the grout. Historically, it has been the practice to flush the ducts with water before grouting is started, the excess water being removed with compressed air. In recent years, grouting experience has indicated that flushing may not be necessary. Grouting should not be done in cold weather because of the possibility that ice may be trapped in the duct, later leaving a void with water which can cause corrosion. The PCI grouting specification has specified concrete temperature of 35°F as the minimum temperature for grouting.

Readers are referred to a paper by Professor Milos Polivka[26] presented at the FIP-RILEM Symposium on Injection Grout for Prestressed Concrete held at Trondheim, Norway, 1961. This paper describes in detail the materials and techniques used for grouting.

The Prestressed Concrete Institute has published[27] its "Tentative Recommended Practices for Grouting Posttensioned Prestressed Concrete" in its journal of November/December 1972. The Post-Tensioning Institute has published a guide specification for grouting.[28]

2-12 Fatigue Strength

The fatigue strength of prestressed concrete can be studied from three approaches: that of concrete itself, that of high-tensile steel, and that of the combination. It may also be studied by utilizing our knowledge on the fatigue strength of reinforced concrete, since so much data has already been accumulated.[29,30,31] There are, however, some differences between prestressed and reinforced concrete. For example, in prestressed concrete, compression in the extreme fibers is frequently near zero under dead load and increases to a maximum under live load, thus varying throughout a wide range. Furthermore, high-strength steel is prestressed to a high level, while its stress range is relatively small.

For prestressed members under the action of design live loads, the stress in steel wires is seldom increased by more than 10,000 psi (69 N/mm^2) from their effective prestress of about 150,000 psi (1034 N/mm^2). It is safe to say that, so long as the concrete has not cracked, there is little possibility of fatigue failure in steel, even though the working load is exceeded. After the cracking of concrete, high stress concentrations exist in the wires at the cracks. These high stresses may result in a partial breakage of bond between steel and concrete near the cracks. Under repeated loading, either the bond may be completely broken or the steel may be ruptured.

Numerous tests have been conducted on prestressed-concrete members, giving considerable data on their fatigue strength. The results of these tests confirm the ability of the combination to stand any number of repeated loads within the working range. Failure started invariably in the wires near the section of maximum moment and often directly over the separators where the wires had a sharp change in direction or at positions where there were preformed cracks in the concrete.

Few tests are available concerning the fatigue bond strength between high-tensile steel and concrete. But, from the results of tests on prestressed-concrete beams, it seems safe to conclude that, if properly grouted, bond between the two materials can stand repeated working loads without failure. This is true because, before the cracking of concrete, bond along the length of the beam is usually low.

A rational method for predicting the fatigue strength of prestressed concrete beams in bending has been developed by Professor Ekberg.[32] It utilizes fatigue-failure envelopes for prestressing steel and concrete, and relates them to the stress-moment diagram for a beam.

A typical failure envelope for prestressing steel is shown in Fig. 2-7(*a*). This envelope indicates how the tensile stress can be increased from a given lower level to a higher level to obtain failure at one million load-cycles. Note that all values are expressed as a percentage of the static tensile strength. Thus the steel may resist a stress range amounting to $0.27f_{pu}$ if the lower stress limit is zero, but only a stress range of $0.18f_{pu}$ if the lower stress limit is increased to $0.40f_{pu}$. At a lower stress limit of $0.90f_{pu}$ or over, it takes only a negligible stress increase to fail the steel at one million cycles. While this fatigue envelope varies for different steels,[33] the curve given here may be considered a typical one.

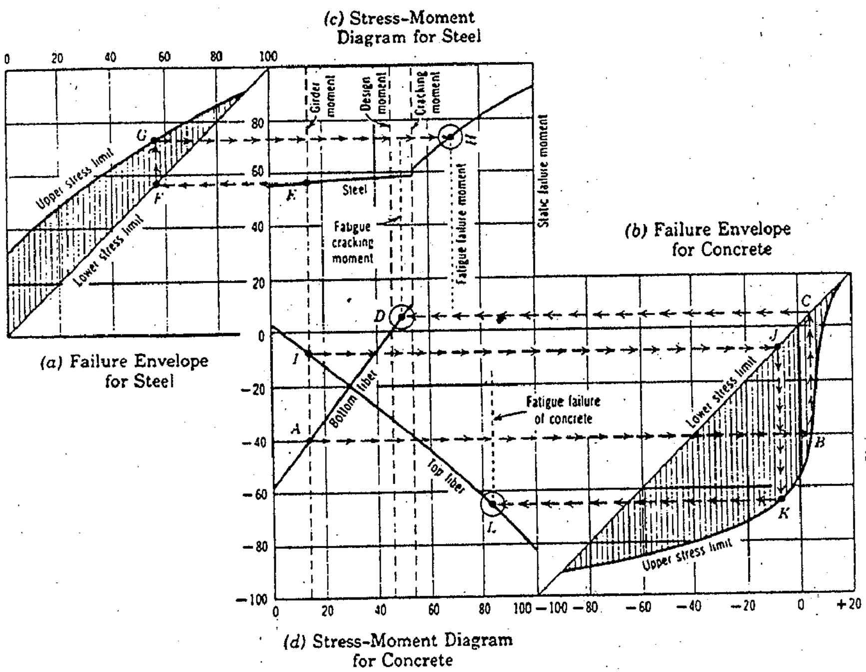

Fig. 2-7. Method for predicting fatigue strength of prestressed concrete beams.

The fatigue-failure envelope for concrete is given in Fig. 2-7(*b*). This is analogous to (*a*) for steel, except it is drawn to cover both tensile and compressive stresses. This diagram indicates that if the lower stress limit is zero, a compressive stress of $0.60f_c'$ may be repeated one million cycles. If the lower stress limit is $0.40f_c'$, the stress range can be $0.40f_c'$. If the compressive stress limit is $0.20f_c'$, the tensile stress limit, to produce cracking, is $0.05f_c'$.

A typical stress-moment diagram for steel is given in Fig. 2-7(*c*), which again expresses nondimensionally both the stress and the moment by relating them to the static strength and the ultimate static moment. For example, when the external moment is 70% of the ultimate static moment, the stress in the steel is shown to be $0.80f_{pu}$. Similarly, Fig. 2-7(*d*) gives the concrete fiber stresses relative to the moment. It is noted that under certain loading conditions, either the top or bottom fibers can be under tension rather than compression.

Combining these four portions (Fig. 2-7) it is possible to determine the fatigue-cracking moment and the fatigue-ultimate moment as limited by steel or concrete. Starting on the stress-moment diagrams at the point of dead-load stress, which represent the lowest possible stress level, we can trace three paths as follows.

Steel: *E-F-G-H*
Concrete top fiber: *I-J-K-L*
Concrete bottom fiber: *A-B-C-D*

The point *H* indicates that for a maximum moment of $0.68M_{ult}$, the steel will fail in tension at one million cycles. The point *L* indicates, for a maximum moment of $0.84M_{ult}$, the top fiber will fail in compression at one million cycles. The point *D* indicates that the fatigue-cracking moment is $0.50M_{ult}$.

Using this analytical approach, Ekberg studied the effect of the level of prestress, the effect of over- and underreinforcing, and the cracking characteristics. The following conclusions were reached:

1. Other conditions being equal, reducing the level of prestress considerably reduces the fatigue-failure moment. This becomes evident when it is realized that cracking would occur sooner for the lower level of prestress and a wider stress range would occur for the steel.
2. Since fatigue failure in concrete is not the controlling criterion, over-reinforcing will generally increase the fatigue strength. The optimum-fatigue moment occurs for a percentage of steel higher than that indicated for a static balanced design.
3. The ratio of dead-load moment to live-load moment has very little effect on the fatigue-cracking moment. Although repetitive loading necessarily reduces the cracking moment, prestressing does delay the occurrence of cracks very substantially.

It is clear that the shape of the member and the location of the steel also have to do with the fatigue strength. When reliable fatigue resistance is to be found, it is desirable that the stress-moment curves and the fatigue-failure envelopes be obtained for the given case and the analytical method outlined above be followed for its determination.

References

1. S. Walker, "Modulus for Elasticity of Concrete," *Proc. Am. Soc. Test. Mat.*, Part II, 1919.
2. R. E. Davis and G. E. Troxell, "Modulus of Elasticity and Poisson's Ratio for Concrete and the Influence of Age and Other Factors upon These Values," *Proc. Am. Soc. Test. Mat.*, 1929. Also see A. D. Ross, "Experiments on the Creep of Concrete under Two-Dimensional Stressing," *Magazine of Concrete Research*, June 1954.
3. R. E. Davis and H. E. Davis, "Flow of Concrete under Action of Sustained Loads," *J. Am. Conc. Inst.*, March 1931 (*proc.*, Vol. 27), pp. 837–901.
4. H. R. Staley and D. Peabody, Jr., "Shrinkage and Plastic Flow of Prestressed Concrete," *J. Am. Conc. Inst.*, January 1946 (*Proc.*, Vol. 42), pp. 229–244.
5. G. E. Troxell, J. M. Raphael, and R. E. Davis, "Long-time Creep and Shrinkage Tests of Plain and Reinforced Concrete," *Proc. Am. Test. Mat.*, 1958.
6. "Creep of Concrete under High Intensity Loading," *Concrete Laboratory Report* No. C-820, Division of Engineering Laboratories, Bureau of Reclamation, U.S. Dept. of the Interior, April 10, 1956.
7. G. Magnel, "Creep of Steel and Concrete in Relation to Prestressed Concrete," *J. Am. Conc. Inst.*, February 1948 (*Proc.*, Vol. 44), pp. 485–500; also *Prestressed Concrete*, McGraw-Hill Book Co., New York, 1954.
8. R. E. Davis and G. E. Troxell, "Properties of Concrete and Their Influence on Prestressed Design," *J. Am. Conc. Inst.*, January 1954 (*Proc.*, Vol. 50), pp. 381–391.
9. P. Klieger, "Early High-strength Concrete for Prestressing," *Proceedings World Conference on Prestressed Concrete*, San Francisco, 1957.
10. N. H. Burns, G. C. Lacey, and J. E. Breen, "State of the Art for Long Span Prestressed Concrete Bridges of Segmental Construction," *J. Prestressed Conc. Inst.*, Vol. 16, No. 5, September–October 1971, pp. 52–77.
11. "Prestressed-Tile Roof and Girders Make All-Tile Building Possible," *Eng. News-Rec.*, April 15, 1954, p. 39.
12. J. J. Shideler, "Lightweight Aggregate Concrete for Structural Use," *J. Am. Conc. Inst.*, October 1957, (*Proc.*, Vol. 54), p. 299–328.
13. T. R. Jones, Jr. and H. K. Stephenson, "Properties of Lightweight Related to Prestressing," *Proceedings World Conference on Prestressed Concrete*, San Francisco, 1957.
14. C. H. Best and M. Polivka, "Creep of Lightweight Concrete," *Magazine of Concrete Research*, November 1959, pp. 129–134.

15. H. Lossier, "L'Autocontrainte des betons par les cimente expansifs," *Memoires Societe des Ingenieurs Civils de France*, March, April 1949, pp. 189–225; also, H. Lossier, "The Self-stressing of Concrete by Expanding Cement," *C.A.C.A.*, London.
16. V. V. Mikhailov, "New Developments in Self-stressed Concrete," *Proceedings World Conference on Prestressed Concrete*, San Francisco, 1957.
17. A. Klein and G. E. Troxell, "Studies of Calcium Sulfoaluminate Admixtures for Expansive Cements," *ASTM* (*Proc.*, Vol. 58), 1958, pp. 986–1008.
18. A. Klein, T. Karby, and M. Polivka, "Properties of an Expansive Cement for Chemical Prestressing," *J. Am. Conc. Inst.*, June 1961.
19. T. Y. Lin and A. Klein, "Chemical Prestressing of Concrete Elements Using Expanding Cements," *J. Am. Conc. Inst.*, September 1963.
20. Alois Legat, "Metallurgical, Metallographical and Economic Problems in the Manufacture of Prestressed Reinforcing Steels," extract of final report of *RILEM Symposium*, Liege, July 1958.
21. I. A. Rubinsky and A. Rubinsky, "A Preliminary Investigation of the Use of Fiberglass for Prestressed Concrete," *Magazine of Concrete Research*, pp. 71–78, September 1954.
22. Frank J. Maguire, III, *Report on Further Investigation Concerning the Feasibility of the Use of Fiberglass Tendons in Prestressed Concrete Construction*, M.S. Thesis, Princeton University, 1960.
23. J. C. Wines and G. C. Hoff, "Laboratory Investigation of Plastic-Glass Fiber Reinforcement for Reinforced and Prestressed Concrete," Reports 1 and 2, Miscellaneous Paper No. 6-779, U. S. Army Corps of Engineers, February 1966.
24. K. M. Gloeckner, "Investigation of Fiberglass Prestressed Concrete," Virginia Highway Research Council, June 1967.
25. E. G. Nawy and G. E. Newerth, "Fiberglass Reinforced Concrete Slabs and Beams," *J. Str. Div.*, Am. Soc. Civil Engineers, Vol. 103, No. ST2, February 1977, pp. 421–440.
26. Milos Polivka, "Grouts for Post-tensioned Prestressed Concrete Members," *J. Prestressed Conc. Inst.*, 1961.
27. "Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete," PCI Committee on Post-Tensioning, *J. Prestressed Conc. Inst.*, Vol. 17 No. 6, November/December 1972.
28. *Post-Tensioning Manual*, Post-Tensioning Institute, Phoenix, Arizona, 1976, pp. 143–148.
29. G. M. Nordby, "Fatigue of Concrete—A Review of Research," *Proc. of the American Concrete Institute*, Vol. 55, 1958–1959.
30. T. Y. Lin, "Strength of Continuous Prestressed Concrete Beams Under Static and Repeated Loads," *J. Am. Conc. Inst.*, June 1955.
31. P. W. Abeles, "Fatigue Tests on Partially Prestressed Concrete Members," *Final Report, Fourth Congress, Int. Assn. Bridge and Structural Eng.*, 1953; also "Fatigue Tests of Prestressed Beams," presented at ACI Convention at Denver, 1954.
32. C. E. Ekberg, Jr., R. E. Walther, and R. G. Slutter, "Fatigue, Resistance of Prestressed Concrete Beams in Bending," *J. Str. Div. Am. Soc. Civil Engineers* (Vol. 83, No. ST4), July 1957.