TABLE OF CONTENTS
MATERIAL PROPERTIES

TABLE OF CONTENTS
MATERIAL PROPERTIES

TABLE OF CONTENTS
MATERIAL PROPERTIES

NOTATION

A = constant

A_s^* = nominal area of prestressing steel

B = constant

$C(t, t_0)$ = creep coefficient at a concrete age of t days

C_u = ultimate creep coefficient

$(E_c)_t$ = modulus of elasticity of concrete at an age of t days

f'_c = specified concrete compressive strength

f'_{ci} = the concrete compressive strength at time of initial prestress

$(f'_c)_t$ = concrete compressive strength at an age of t days

$(f'_c)_{28}$ = concrete compressive strength at an age of 28 days

f_f = fatigue stress range in reinforcement

f_{min} = minimum stress level in reinforcement

f_{ps} = stress in prestressing strand

f_r = modulus of rupture

f'_s = ultimate strength of prestressing steel

H = annual average ambient relative humidity

k_c = product of applicable correction factors = k_{la} x k_h x k_s

k_{cp} = correction factor for curing period

k_h = correction factor for relative humidity

k_{la} = correction factor for loading age

k_s = correction factor for size of member

k_{sh} = product of applicable correction factors = k_{cp} x k_h x k_s

K = constant

r/h = ratio of base radius to height of transverse deformation on reinforcement

S = surface area of concrete exposed to drying

$S(t, t_0)$ = shrinkage strain at a concrete age of t days

S_u = ultimate shrinkage strain

t = age of concrete

t_{la} = loading ages

t_0 = age of concrete at the end of the initial curing period

V = volume of concrete

w_c = unit weight of concrete

ε_{ps} = strain in prestressing strand

λ = concrete weight factor taken as 1.0 for normal weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for all-lightweight concrete

Material Properties

2.1 SCOPE

This chapter contains a description of the properties of all major materials currently used for precast, prestressed concrete bridge structures. It includes a discussion of concrete constituent materials, mix requirements, hardened concrete properties, pretensioning and post-tensioning reinforcement, nonprestressed reinforcement and grouts used between precast members and other components. Recent developments in high performance concrete and nonmetallic reinforcement are also introduced. Discussion of the materials used in fabrication and construction is included in Chapter 3.

2.2 PLANT PRODUCTS

The production of precast concrete components in a plant environment offers several advantages compared to on-site production. Many of these advantages occur because one company is responsible for quality control throughout production. This results in closer monitoring of raw materials, steel placement, concrete production and delivery, concrete curing and shipment. The overall effect is to produce a product with more consistent material properties than can be achieved with site-cast concrete.

2.2.1 Advantages

In many aspects, the material properties of precast components are superior to those of cast-in-place members. Precast concrete components are required to achieve a minimum concrete strength for release and removal from their precasting beds at an early age (12 to 18 hours). This often results in a concrete that has a 28-day compressive strength in excess of the specified 28-day strength. Consequently, the concrete has a higher modulus of elasticity and less creep than would occur if the actual strength were equal to the specified strength. The use of accelerated curing to achieve the release strength also results in less shrinkage and creep. From a durability aspect, precast concrete members have a low permeability and, therefore, are better suited for use in aggressive environments such as coastal areas and areas where deicing salts are used.

2.3 CONCRETE MATERIALS

The five major component materials of concrete produced today are cement, aggregates, chemical admixtures, mineral admixtures and water.

2.3.1 Cement

Cement for use in bridge construction generally conforms to one of the following specifications:

AASHTO M85 Portland Cement

AASHTO M240 Blended Hydraulic Cement

2.3.1.1 AASHTO M85

The AASHTO Specification M85 lists eight types of portland cement as follows:

Type I Normal

Type IA Normal, air-entraining

Type II Moderate sulphate resistant

Type IIA Moderate sulphate resistant, air-entraining

Type III High early strength
Type IIIA High early strength, air-entraining
Type IV Low heat of hydration
Type V High sulphate resistance

Type I portland cement is a general purpose cement suitable for all uses where the special properties of other types of cement are not required. Type II portland cement is used where precaution against moderate sulphate attack is important. Type II cement can also be used to reduce the heat of hydration. Type III portland cement provides high strengths at an early age and is particularly appropriate for obtaining high release strengths. Type IV portland cement is used to reduce the heat of hydration and is particularly beneficial in mass concrete structures. Type V portland cement is used in concrete exposed to severe sulphate attack. Types IA, IIA and IIIA, correspond in composition to Types I, II and III respectively, except that small quantities of air-entraining material are included in the cement.

2.3.1.2 AASHTO M240

The AASHTO Specification M240 lists six classes of blended cement as follows:

Type IS Portland blast-furnace slag cement
Type IP Portland-pozzolan cement
Type P Portland-pozzolan cement
Type S Slag cement
Type I (PM) Pozzolan-modified portland cement
Type I (SM) Slag-modified portland cement

Blended hydraulic cements are produced by intergrinding and/or blending various combinations of portland cement, ground granulated blast-furnace slag, fly ash and other pozzolans. These cements can be used to produce different properties in the hardened concretes. Types IS, IP, I(PM) and I(SM) are used for general concrete construction. Type P is used where high early strengths are not required. Type S is used with portland cement in concrete or with lime in mortar but is not used alone in structural concrete.

2.3.1.3 Restrictions

The *Standard Specifications* generally restrict cement to portland cement Types I, II or III; air-entrained portland cement Types IA, IIA or IIIA; or blended hydraulic cements Types IP or IS. It should also be noted that not all types of cement are readily available and that the use of some types is not permitted by some states.

2.3.2 Aggregates

Aggregates for concrete consist of fine and coarse materials. Fine aggregate for normal weight concrete should conform to the requirements of AASHTO M6. Coarse aggregate for normal weight concrete should conform to the requirements of AASHTO M80. Lightweight aggregate for use in lightweight or sand-lightweight concrete should conform to the requirements of AASHTO M195. The maximum size of aggregate should be selected based on mix-requirements and the minimum clear spacing between reinforcing steel, clear cover to reinforcing steel and thickness of the member in accordance with AASHTO specifications. If aggregates susceptible to alkali-aggregate reactivity are used in prestressed concrete members, special precautions must be observed. These include the use of low alkali cements, blended cements or pozzolans.

2.3.3 Chemical Admixtures

Chemical admixtures are used in precast, prestressed concrete to provide air entrainment, reduce water content, improve workability, retard setting times and accelerate strength development. Chemical admixtures, except air-entraining admixtures, should conform to the requirements of AASHTO M194. This specification lists the following types of admixtures:

Type A Water-reducing
Type B Retarding
Type C Accelerating
Type D Water-reducing and retarding
Type E Water-reducing and accelerating
Type F Water-reducing, high range
Type G Water-reducing, high range and retarding

2.3.3.1 Purpose

Water-reducing admixtures and high range water-reducing admixtures are used to allow for a reduction in the water-cementitious materials ratio while maintaining or improving workability. Accelerating admixtures are used to decrease the setting time and increase the early strength development. They are particularly beneficial in precast concrete construction to facilitate early form removal and release of prestressing. Since admixtures can produce different results with different cements, and at different temperatures, selection of admixtures should be based on the plant materials and conditions that will be utilized in production. Compatibility between admixtures is also important and should be specifically addressed when using combinations of admixtures produced by different companies.

2.3.3.2 Calcium Chloride

Calcium chloride has been used in the past as an accelerator since it is very effective and economical. The use of calcium chloride in concrete promotes corrosion of metals due to the presence of chloride ions. Consequently, calcium chloride should not be permitted in prestressed concrete members. Accelerators without chlorides may be used.

2.3.3.3 Corrosion Inhibitors

Corrosion-inhibiting admixtures are also available for use in concrete to protect reinforcement from corrosion. These admixtures block the passage of chloride ions to the steel reinforcement and, thereby, reduce or eliminate corrosion of the reinforcement. Corrosion-inhibiting admixtures are more likely to be effective in cast-in-place bridge components that are directly exposed to chloride ions than in precast concrete bridge girders that are already highly impermeable.

2.3.3.4 Air–Entraining Admixtures

Air-entraining admixtures are used in concrete primarily to increase the resistance of the concrete to freeze-thaw damage when exposed to water and deicing chemicals. They may also be used to increase workability and facilitate handling and finishing. Air-entraining admixtures should conform to AASHTO M154. The air content of fresh concrete is generally determined using the pressure method (AASHTO T152) or the volumetric method (AASHTO T196). The pressure method should not be used with lightweight concrete. A pocket-size air indicator (AASHTO T199) can be used for quick checks but is not a substitute for the other more accurate methods.

2.3.4 Mineral Admixtures

Mineral admixtures are powdered or pulverized materials added to concrete to improve or change the properties of hardened portland cement concrete. Mineral admixtures are used in concrete to increase early strength development or to reduce the heat of hydration. They may also be used to improve the resistance of concrete to reactive aggregates and to replace cement. They have also been used in high strength concrete to produce higher strengths at later ages. The use of mineral admixtures may affect the workability and finishing characteristics of fresh concrete.

2.3.4.1 Pozzolans

AASHTO M295 lists three classes of mineral admixtures as follows:

Class N Raw or calcined natural pozzolans
Class F Fly ash
Class C Fly ash

High-Reactive Metakaolin (HRM) is a manufactured white powder that meets the requirements of a Class N pozzolan. HRM has a particle size significantly smaller than that of cement particles, but not as fine as silica fume. Fly ash is a finely divided residue that results from the combustion of pulverized coal in power generation plants. Class F fly ash has pozzolanic properties; Class C has some cementitious properties in addition to pozzolanic properties. Some fly ashes meet both Class F and Class C classifications. Selection of these materials will depend on their local availability and their effect on concrete properties.

2.3.4.2 Silica Fume

Silica fume meeting the requirements of AASHTO M307 may also be used as a mineral admixture in concrete. Silica fume is a very fine pozzolanic material produced as a by-product in electric arc furnaces used for the production of elemental silicon or ferro-silicon alloys. Silica fume is also known as condensed silica fume and microsilica. The use of silica fume can improve the early age strength development of concrete and is particularly beneficial in achieving high release strengths in high strength concrete beams. The use of silica fume in concrete generally results in concrete that has low permeability. The use of silica fume increases the water demand in concrete. Consequently, it is generally used in combination with a water-reducing admixture or a high range water-reducing admixture. Concrete containing silica fume has significantly less bleeding and the potential for plastic shrinkage is increased. Therefore, early moisture loss should be prevented under conditions which promote rapid surface drying such as low humidity and high temperatures.

2.3.5 Water

Water used in mixing concrete must be clean and free of oil, salt, acid, alkali, sugar, vegetable or other injurious substances. Water known to be of potable quality may be used without testing. However, if there is doubt, water should meet the requirements of AASHTO T26. Mixing water for concrete should not contain a chloride ion concentration in excess of 1,000 ppm or sulfates as SO_4 in excess of 1,300 ppm.

2.4 SELECTION OF CONCRETE MIX REQUIREMENTS

This section discusses various aspects of concrete mix requirements that need to be considered by the owner or the owner's engineer. Selection of concrete ingredients and proportions to meet the minimum requirements stated in the specifications and contract documents should be the responsibility of the precast concrete producer. Wherever possible, the mix requirements should be stated on the basis of the required performance and not be over-restrictive to the producer. The producer should be allowed to show through trial batches or mix history that a proposed mix design will meet or exceed the specified performance criteria. Consequently, prescriptive requirements such as minimum cement content should be avoided.

2.4.1 Concrete Strength at Release

For prestressed concrete bridge beams, the Engineer generally specifies minimum strengths at time of release of the prestressing strands and at 28 days, although ages other than 28 days may be used. The Engineer may also specify a minimum compressive strength at time of beam erection, or a minimum compressive strength at time of post-tensioning if a combination of pretensioning and post-tensioning is utilized. For most prestressed concrete bridge beams, the specified strength at time of release will control the concrete mix proportions. Based on AASHTO specifications, the release strength is selected so that the temporary concrete stresses in the beam, before losses due to creep and shrinkage, do not exceed 60% of the concrete compressive strength at time of release in pretensioned members and 55% of the concrete compressive strength at time of stressing of post-tensioned members. In addition, the strength is selected so that, in tension areas with no bonded reinforcement, the tensile stress will not exceed 200 psi or $3\sqrt{f'_{ci}}$ where f'_{ci} is the compressive strength of concrete at time of initial prestress in psi. In areas with a specified amount of bonded reinforcement, the maximum tensile stress cannot exceed $7.5\sqrt{f'_{ci}}$.

2.4.2 Concrete Strength at Service Loads

The design of most precast, prestressed concrete members is based on a concrete compressive strength at 28 days of 5,000 to 6,000 psi. However, because the mix proportions are generally dictated by release strengths, concrete strengths at 28 days are frequently in excess of the specified 28-day value and actual strengths of 8,000 psi or more are often achieved. Consequently, mix requirements are generally based on the release strengths and the precaster only has to ensure that the mix will provide concrete with a compressive strength in excess of that specified for 28 days.

2.4.3 High Performance Concrete

Concrete with a compressive strength in excess of 8,000 psi has not been commonly specified for precast, prestressed concrete bridge beams. There is, however, a trend toward the greater utilization of higher strength concretes to achieve more durable and economical structures. Some states are using the higher strength characteristics of high performance concrete to stretch spans or widen beam spacings by using beams with concrete strengths in excess of 10,000 psi. In such cases, strength is typically specified at 56 days because of the strength gain that is possible in higher strength concretes between 28 and 56 days.

The minimum compressive strength, in some cases, may be controlled by the need to meet a minimum requirement for special exposure conditions as discussed in Section 2.4.6.2.

2.4.4 Durability

Durability is a concern when bridges are exposed to aggressive environments. This generally occurs where deicing salts are utilized on highways during winter or in coastal regions where structures are exposed to salt from sea water. The Engineer must be concerned about the deleterious effects of freezing and thawing, chemical attack and corrosion of embedded or exposed metals. The ideal approach is to make the concrete as impermeable as possible. In this respect, precast, prestressed concrete has inherent advantages over cast-in-place concrete since it is produced in a controlled environment that results in high quality concrete. In addition, the mix proportions needed to achieve a relatively high strength concrete often produce a relatively impermeable concrete. As a result, precast, prestressed concrete bridge beams have an excellent record of performance in aggressive environments.

2.4.4.1 Freeze–Thaw Damage

Freeze-thaw damage generally manifests itself by scaling of the concrete surface. This occurs as a result of temperature fluctuations that cause freezing and thawing when the concrete is saturated. Freeze-thaw damage is magnified when deicing chemicals are present. To minimize freeze-thaw damage, a minimum air content is generally specified. The presence of entrained air provides space for ice to expand without developing high pressures that would otherwise damage the concrete. **Table 2.4.4.1-1**, based on ACI 211.1, provides the required air

Table 2.4.4.1-1
Total Air Content for Frost-Resistant Concrete

Nominal Maximum Aggregate Size, in.	Minimum Air Content*, percent	
	Severe Exposure	Moderate Exposure
3/8	7-1/2	6
1/2	7	5-1/2
3/4	6	5
1	6	4-1/2
1-1/2	5-1/2	4-1/2

*The usual tolerance on air content as delivered is ±1.5 percent content for severe and moderate exposure conditions for various maximum aggregate sizes. Severe exposure is defined as a climate where the concrete may be in almost continuous contact with moisture prior to freezing, or where deicing salts come in contact with the concrete. This includes bridge decks. Salt laden air, as found in coastal areas, is also considered a severe exposure. A moderate exposure is one where deicing salts are not used or where concrete will only occasionally be exposed to moisture prior to freezing. This is generally the case for bridge beams. It should be noted that some state highway departments specify air contents that are slightly different from those shown in **Table 2.4.4.1-1**. In addition, many states do not require air entrainment in prestressed concrete beams because beams are sheltered by the deck or other conditions exist such that air entrainment is not required for good performance.

2.4.5 Workability

The ease of mixing, placing, consolidating and finishing freshly mixed concrete is called workability. Concrete should be workable but should not segregate or bleed excessively. Excessive bleeding increases the water-cementitious materials ratio near the top surface and a weak top layer of concrete with poor durability may result. For prestressed concrete bridge beams, particular attention should be paid to ensure that concrete has adequate workability so that it will consolidate around the prestressing strands, particularly at end regions of beams where a high percentage of nonprestressed reinforcement is present. It is also important that concrete can be placed in the webs of beams without segregation. Workability can be enhanced through the use of water-reducing admixtures, high range water-reducing admixtures and air entraining agents. No standard test exists for the measurement of workability. The concrete slump test is the most generally accepted method used to measure consistency of concrete but it should not be used as a means to control workability.

2.4.6 Water-Cementitious Materials Ratio

The water-cementitious materials ratio is the ratio of the amount of water, exclusive of that absorbed by the aggregate, to the amount of cementitious materials in a concrete or mortar mixture. As such, the amount of water includes that within the admixtures and that in the aggregate in excess of the saturated surface-dry condition. The amount of cementitious material includes cement and other cementitious materials, such as fly ash and silica fume. The total cementitious materials content for compressive strengths from 4,000 to 8,000 psi can vary from 600 to 1,000 pcy and will also vary on a regional basis.

2.4.6.1 Based on Strength

When strength, not durability, controls the mix design, the water-cementitious materials ratio and mixture proportions required to achieve specified strength should be determined from field data or the results of trial batch strength tests. The trial batches should be made from actual job materials. When no other data are available, **Table 2.4.6.1-1,** which is based on ACI 211.1, may be used as a starting point for mix design procedures for normal weight concrete.

Table 2.4.6.1-1 Approximate Ratios for Trial Batches

Compressive Strength at 28 days, psi	Water-Cementitious Materials Ratio By Weight	
	Non-Air-Entrained Concrete	Air-Entrained Concrete
6,000	0.41	–
5,000	0.48	0.40
4,000	0.57	0.48

Table 2.4.6.2-1
Maximum Requirements for Various Exposure Conditions

Exposure Condition	Maximum Water-Cementitious Materials Ratio for Normal Weight Concrete
Concrete intended to have low permeability when exposed to water	0.50
Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals	0.45
For corrosion protection for reinforced concrete exposed to chlorides from deicing chemicals, salt, salt water or brackish water, or spray from these sources	0.40

2.4.6.2
Based on Durability

When durability is a major consideration in the concrete mix design, the water-cementitious materials ratios for various exposure conditions should be limited to the values specified in ACI 318 and shown in **Table 2.4.6.2-1**. For precast, prestressed concrete members exposed to deicing salts or spray from sea water, the maximum ratio will generally be 0.40.

2.4.7
Unit Weight

2.4.7.1
Normal Weight Concrete

The unit weight of normal weight concrete is generally in the range of 140 to 150 pcf. For concrete with compressive strengths in excess of 10,000 psi, the unit weight may be as high as 155 pcf. The unit weight will vary depending on the amount and density of the aggregate and the air, water and cement contents. In the design of reinforced or prestressed concrete structures, the combination of normal weight concrete and reinforcement is commonly assumed to weigh 150 pcf but may be assumed as high as 160 pcf.

2.4.7.2
Lightweight Concrete

Lightweight concrete and sand-lightweight concrete (also called semi-lightweight concrete) may also be utilized in precast, prestressed concrete bridge construction with the use of suitable lightweight aggregates. Lightweight aggregate concretes generally have a unit weight of 90 to 105 pcf. Sand-lightweight aggregate concretes have a unit weight of 105 to 130 pcf with a common range of 110 to 115 pcf. When lightweight concrete is used in prestressed concrete members, special consideration must be given to using mix design procedures for lightweight concrete as given in ACI 211.2.

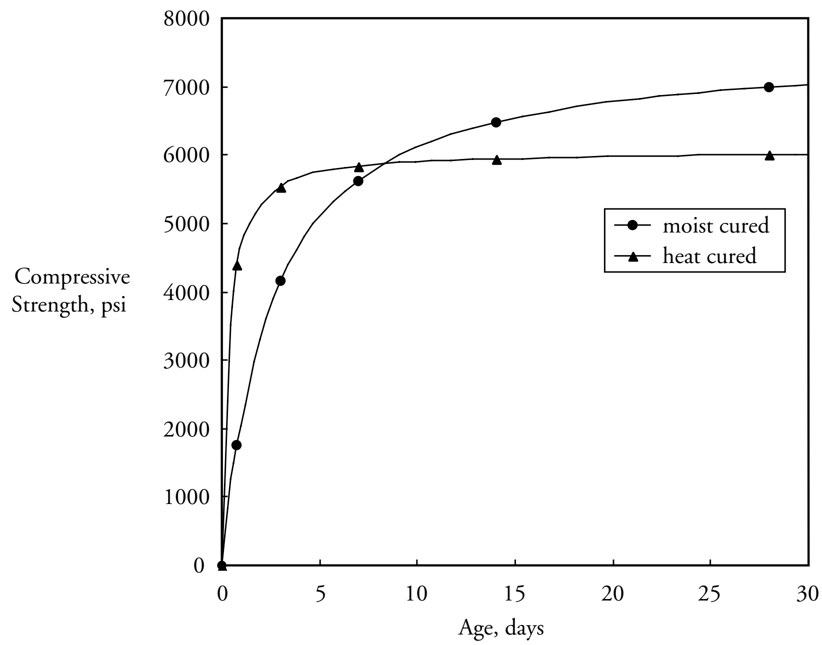
2.4.7.3
Blended Aggregates

Where suitable lightweight aggregates are available, a common practice is to blend lightweight with normal weight aggregates to achieve a desired concrete unit weight. This is done to control beam (or other product) weights to satisfy shipping limitations, jobsite conditions such as crane size or reach limits, or plant or erection equipment capacities.

2.4.8
Effect of Heat Curing

Because of the need for early strength gain, Type III cement is often used in precast concrete so that forms may be reused on a daily basis. This generally requires that the release strength be achieved no later than 18 hours after the concrete is placed and may be achieved at 12 hours or less. To accelerate the strength gain, it is often necessary to raise the temperature of the concrete. In some situations, such as with high strength concrete, the increase in temperature can be provided by the internal heat of hydration. However, in most situations, it is necessary to utilize an external source of heat, such as steam or radiant heat, to reach the necessary release strengths. The use of external heat causes the concrete temperature to be higher at an earlier age than would be achieved from the natural heat of hydration. A consequence of achieving a high release strength is a reduction in the later age strengths compared to strengths that would have been obtained if the concrete had not been heat cured. This is illustrated in **Figure 2.4.8-1.** The effect of heat curing on the concrete compressive strength development must be taken into account in the selection of mix requirements and in the preparation of trial mixes.

Figure 2.4.8-1
Effect of Curing on Concrete Compressive Strength Gain



2.4.9 Sample Mixes

Sample concrete mixes for six different concrete compressive strengths are shown in **Table 2.4.9-1**. These are concrete mixes from different precasting plants. It should not be assumed that these mixture proportions will always produce the same concrete compressive strengths when used with different materials.

2.5 CONCRETE PROPERTIES

2.5.1 Introduction

Concrete properties such as modulus of elasticity, tensile strength, shear strength and bond strength are frequently expressed in terms of the compressive strength. Generally, expressions for these quantities have been empirically established based on data for concretes having compressive strengths up to 6,000 psi. With recent research, these empirical relationships have been reevaluated for concrete compressive strengths up to 10,000 psi. Unless indicated otherwise, the relationships in this section may be assumed applicable for concrete with compressive strengths up to 10,000 psi. Where alternative expressions are available, they are discussed in each section. For concretes with compressive strengths in excess of 10,000 psi, the recommendations given in ACI 363 and Zia et al (1991) should be considered.

2.5.2 Compressive Strength

Compressive strength is generally measured by testing 6x12-in. cylinders in accordance with standard AASHTO or ASTM procedures. The precast concrete industry also uses 4x8-in. cylinders. Some state highway departments permit the use of either 6x12-in. or 4x8-in. cylinders for quality control. For high strength concretes, the use of smaller size cylinders may be necessary because of limitations on testing machine capacities. For precast, prestressed concrete members it is particularly important that the concrete cylinders used to determine release strengths be cured in an identical manner to the bridge members. In general, this is accomplished by curing the concrete cylinders alongside the prestressed concrete member until release of the prestressing strands. A more advanced technique of match curing is also available. In this procedure, the cylinders are enclosed in a container in which the temperature is controlled to match the temperature of the concrete member. The test cylinders then undergo the same time-temperature history as the concrete member.

Table 2.4.9-1 Sample Production Concrete Mixes

Mix	A	B	C	D	E	F
Specified Strength, psi						
Release	3,500	4,000	5,000	6,000	6,000	8,800
28 Days	5,000	6,000	7,500	7,500	10,000	13,100
Quantities per cu yd						
Cement, lb	705	705	850	750	750	671
Fly Ash, lb	0	0	0	140	0	316
Silica Fume, lb	0	0	0	0	95	0
Sand, lb	1,055	1,085	935	1,085	1,030	1,029
Coarse Aggregate, lb	1,790	1,920	1,770	1,980	1,870	1,918
Water, lb	270	285	300	230	230	247
Air Entrainment, fl. oz.	5	0	17	0	3	0
Water-Reducer, fl. oz.	25	53	29	0	10	0
High Range Water-Reducer, fl. oz.	125	0	145	160	85	200
Concrete Properties						
Water-Cementitious Ratio	0.38	0.40	0.36	0.26	0.31	0.25
Slump, in.	3-1/2	4-3/4	4	6	5	9
Unit Weight, pcf	141.5	147.8	140.0	145.0	147.4	UNKN
Air Content, %	6.0	N/A	6.0	N/A	5.0	N/A
Release Strength, psi	3,800	4,350	5,300	6,700	9,070	8,800
28-day Strength, psi	5,700	6,395	8,000	9,400	10,450	13,900
56-day Strength, psi	UNKN	UNKN	UNKN	UNKN	UNKN	15,200

UNKN – Unknown; NA – Not Applicable

2.5.2.1 Variation with Time

The variation of concrete compressive strength with time may be approximated by the following general calculation:

$$(f'_c)_t = \frac{t}{A + Bt}(f'_c)_{28} \qquad \text{(Eq. 2.5.2.1-1)}$$

where:

$(f'_c)_t$ = concrete compressive strength at an age of t days

$(f'_c)_{28}$ = concrete compressive strength at an age of 28 days

A and B = constants

The constants A and B are functions of both the type of cementitious material used and the type of curing employed. The use of normal weight, sand-lightweight or all lightweight aggregate does not appear to affect these constants significantly. Typical values recommended by ACI 209 are given in **Table 2.5.2-1**. The constants for current practice shown in **Table 2.5.2.1-1** are based on the sample mixes shown in **Table 2.4.9-1**. These mixes have release strengths that vary from 63 to 87% of the 28-day strength.

Table 2.5.2.1-1 Values of Constants A and B

Source	Curing	Cement	A	B
ACI 209	Moist	I	4.00	0.85
ACI 209	Moist	III	2.30	0.92
ACI 209	Steam	I	1.00	0.95
ACI 209	Steam	III	0.70	0.98
Current Practice	Heat	III	0.28	0.99

2.5.2.2 Effect of Accelerated Curing

As shown in **Figure 2.4.8-1**, a concrete that is heat cured will have higher initial strengths but lower strength at later ages when compared to the same concrete that is moist cured. It should be emphasized that these are general relationships and variations will occur for different concretes and curing procedures. When fly ash is used as a mineral admixture, it may be appropriate to determine the compressive strength at 56 days to take advantage of the later strength gain. Therefore, it is important that the strength gain relationship be established through trial mixes or previous experience using local producer data. This is particularly important for release strengths which can occur as early as 12 hours. If the relationship is unknown, the values listed in **Table 2.5.2-1** for current practice will give an approximate relationship.

2.5.3 Modulus of Elasticity

The modulus of elasticity is the ratio of uniaxial normal stress to corresponding strain up to the proportional limit for both tensile and compressive stresses. It is the material property that determines the amount of deformation under load. It is used to calculate camber at release, elastic deflections caused by dead and live loads, axial shortening and elongation, prestress losses, buckling and relative distribution of applied forces in composite and non-homogeneous structural members. Modulus of elasticity is determined in accordance with ASTM C 469.

2.5.3.1 Calculations (E_c)

For concrete compressive strengths less than 8,000 psi, the following calculation may be used to predict the modulus of elasticity:

$$(E_c)_t = 33(w_c)^{1.5}\sqrt{(f'_c)_t} \qquad \text{(Eq. 2.5.3.1-1)}$$

where:

$(E_c)_t$ = modulus of elasticity of concrete at an age of t days, psi

w_c = unit weight of concrete, psi

$(f'_c)_t$ = concrete compressive strength at an age of t days, psi

The above equation was based on an analysis for concrete strengths up to about 6,000 psi. According to ACI 363, the above calculation tends to over-estimate the modulus of elasticity for higher strength concretes. Several alternative equations have been proposed for the calculation of modulus of elasticity and the following by Martinez (1982) has received general acceptance:

$$(E_c)_t = \left(40{,}000\sqrt{(f'_c)_t} + 1{,}000{,}000\right)\left(\frac{w_c}{145}\right)^{1.5}$$

2.5.3.2 Variations (E_c)

Deviations from predicted values are highly dependent on the properties and proportions of the coarse aggregate used in the concrete. Consequently, where local producer data are available, they should be utilized in place of the values determined from these standard equations. This is particularly important in computing the camber at release as these modulus of elasticity equations have not been developed specifically for determination of the modulus of heat cured concrete at an early age.

2.5.4 Modulus of Rupture

The modulus of rupture is a measure of the flexural tensile strength of the concrete. It can be determined by testing, but the modulus of rupture for structural design is generally assumed to be a function of the concrete compressive strength as given by:

$$f_r = K\lambda\sqrt{f'_c} \qquad \text{(Eq. 2.5.4-1)}$$

where:

f_r = modulus of rupture, psi

K = a constant, usually taken as 7.5

λ = 1.0 for normal weight concrete

0.85 for sand-lightweight concrete

0.75 for all-lightweight concrete

For high strength concretes, a value of K greater than 7.5 has been proposed. However, for most applications, a conservative value of 7.5 is still used for high strength concretes.

2.5.5 Durability

Durability refers to the ability of concrete to resist deterioration from the environment or service conditions in which it is placed. Properly designed concrete should survive throughout its service life without significant distress. The following test procedures may be used to check the durability of concrete made with a specific mix:

Freeze-thaw resistance	ASTM C 666, C 671 and C 682
Deicer scaling resistance	ASTM C 672
Abrasion resistance	ASTM C 418, C 779 and C 944
Chloride permeability	AASHTO T277 or T259
Alkali-aggregate reactivity	ASTM C 227, C 289, C 342, C 441 and C 586
Sulphate resistance ASTM	C 452 and C 1012

It is not necessary to perform all the above tests to prove that a concrete will be durable. In general, a concrete that has a low permeability will also have a high resistance to freeze-thaw cycles and surface scaling. It should also be noted that a concrete that does not perform very well in the above tests will not necessarily perform poorly in the field. Concrete that performs well in the above tests, will nearly always perform well in an actual structure. This is the case for precast concrete members that are produced under controlled factory conditions.

2.5.6 Heat of Hydration

Heat of hydration is the heat generated when cement and water react. The amount of heat generated is largely dependent on the chemical composition of the cement but an increase in cement content, fineness or curing temperature will increase the heat of hydration. Heat of hydration is particularly important in heat-cured concretes where the heat generated by the chemical reaction of the cement in conjunction with heat curing can be used to accelerate the development of compressive strength. The heat of hydration can be measured using ASTM C 186. When prestressed concrete beams are heat cured, the heat generated by hydration cannot escape from the surface of the member. Consequently, under this condition, the beams may be considered as mass concrete. Procedures for determining the temperature rise in mass concrete are described in ACI 207.1. However, as an approximate calculation, it can be assumed that a temperature rise of 10F will occur for each 100 lb of cement used in the concrete. More precise calculations can be made using the actual concrete mix proportions, specific heat of the concrete and heat generated per unit mass of cement.

2.5.7 Shrinkage

Precast concrete members are subjected to air drying as soon as they are removed from the forms. During this exposure to the atmosphere, the concrete slowly loses some of its original water, causing shrinkage to occur. The amount and rate of shrinkage vary with the relative humidity, size of member and amount of nonprestressed reinforcement.

2.5.7.1 Calculation of Shrinkage

Procedures to calculate the amount of shrinkage and creep have been published in the *LRFD Specifications*, by CEB-FIP (1990) and ACI 209. These procedures are based on the recommendations of ACI 209 which are summarized in this section.

Shrinkage after 1 to 3 days for steam-cured concrete:

$$S(t,t_0) = \frac{(t - t_0)}{55 + (t - t_0)} S_u \qquad \text{(Eq. 2.5.7.1-1)}$$

Shrinkage after 7 days for moist-cured concrete:

$$S(t,t_0) = \frac{(t - 7)}{35 + (t - 7)} S_u \qquad \text{(Eq. 2.5.7.1-2)}$$

where:

$S(t, t_0)$ = shrinkage strain at a concrete age of t days

S_u = ultimate shrinkage strain

t = age of concrete, days

t_0 = age of concrete at the end of the initial curing period, days

Although Eq. 2.5.7.1-1 was developed for steam-cured concretes, it may be applied to radiant heat-cured concretes if more specific information is not available.

In the absence of specific shrinkage data for local aggregates and conditions, the following average value for the ultimate shrinkage strain is suggested:

$$S_u = 545\ k_{sh} \text{ x } 10^{-6} \qquad \text{(Eq. 2.5.7.1-3)}$$

where:

k_{sh} = product of applicable correction factors

$= k_{cp} \text{ x } k_h \text{ x } k_s$ (Eq. 2.5.7.1-3a)

k_{cp} = correction factor for curing period

k_h = correction factor for relative humidity

k_s = correction factor for size of member

Table 2.5.7.1-1
Correction Factor k_{cp} for Initial Curing Period

Moist Curing Period, days	Shrinkage Factor, k_{cp}
1	1.20
3	1.10
7	1.00
14	0.93
28	0.86
60	0.79
90	0.75

For shrinkage of concrete moist-cured for other than 7 days, the curing correction factor, k_{cp} may be taken from **Table 2.5.7.1-1**.

The relative humidity correction factor, k_h, may be taken from **Table 2.5.7.1-2.** A relative humidity map taken from *LRFD Specifications* is shown in **Figure 2.5.7.1-1**.

Table 2.5.7.1-2
Correction Factors k_h for Relative Humidity

Average Ambient Relative Humidity, %	Shrinkage Factor, k_h	Creep Factor, k_h
40	1.43	1.25
50	1.29	1.17
60	1.14	1.08
70	1.00	1.00
80	0.86	0.91
90	0.43	0.83
100	0.00	0.75

The above correction factors are based on the following equations:

Shrinkage: $k_h = 2.00 - 0.0143H$ for $40 \le H \le 80$ (Eq. 2.5.7.1-3b)

$= 4.286 - 0.0429H$ for $80 < H \le 100$ (Eq. 2.5.7.1-3c)

Creep: $k_h = 1.586 - 0.0084H$ for $40 \le H \le 100$ (Eq. 2.5.7.1-3d)

where H = annual average ambient relative humidity in percent

The size correction factor, k_s, depends on the volume to surface area of the member and may be taken from **Table 2.5.7.1-3.** The volume to surface area ratio for long members may be computed as the ratio of cross-sectional area to section perimeter.

Figure 2.5.7.1-1
Average Annual Ambient Relative Humidity

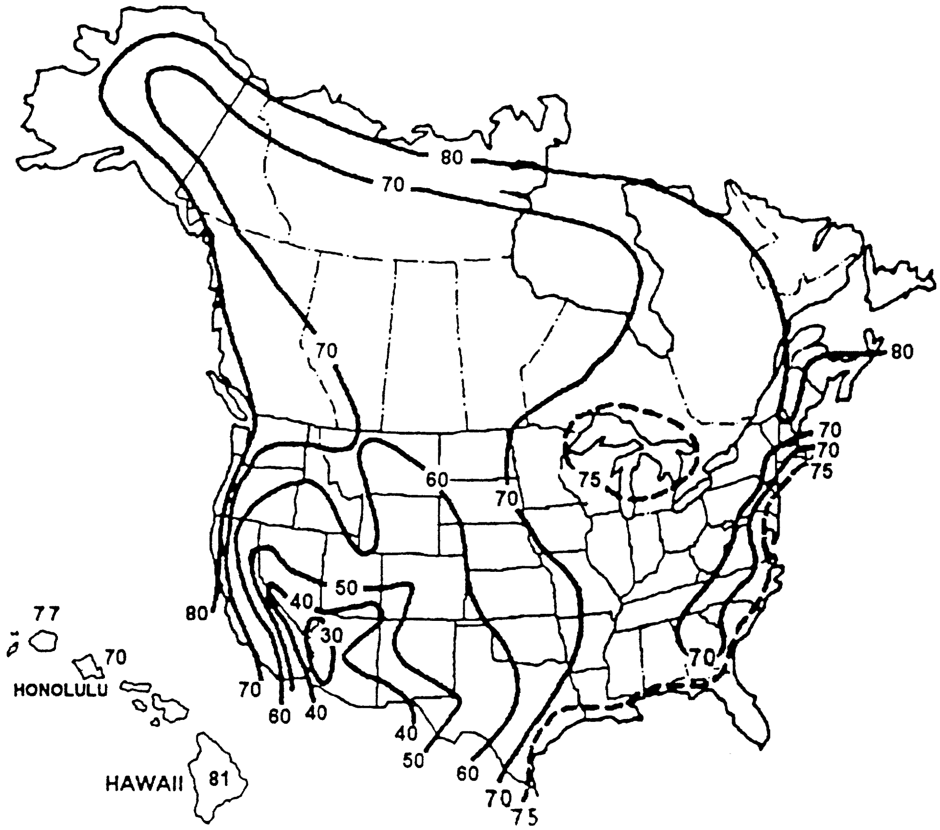


Table 2.5.7.1-3
Correction Factors k_s for Size

Beam Section	Volume/Surface (in.)	Shrinkage Factor, k_s	Creep Factor, k_s
Type I	3.05	0.83	0.81
Type II	3.37	0.80	0.79
Type III	4.06	0.74	0.75
Type IV	4.74	0.68	0.73
Type V	4.44	0.71	0.74
Type VI	4.41	0.71	0.74
BT-54	3.01	0.84	0.82
BT-63	3.01	0.84	0.82
BT-72	3.01	0.84	0.82

The above correction factors are based on the following equations:

Shrinkage: $k_s = 1.2e^{-0.12V/S}$ (Eq. 2.5.7.1-3e)

Creep: $k_s = 2/3\ (1+1.13e^{-0.54V/S})$ (Eq. 2.5.7.1-3f)

where:

V = volume of concrete, in.^{3}

S = surface area of concrete exposed to drying, in.^{2}

2.5.8 Creep

Prestressed concrete beams are subjected to the effects of creep as soon as the prestressing force is released in the plant. Creep of concrete results in time-dependent changes in camber and prestress forces. The amount and rate of creep vary with the concrete age at loading, stress level, relative humidity, size of member and amount of nonprestressed reinforcement. The following calculations are based on ACI 209.

2.5.8.1 Calculation of Creep

Creep strains are determined by multiplying the elastic strains by a creep coefficient, $C(t, t_0)$.

For steam-cured concrete loaded at 1 to 3 days and moist-cured concrete loaded at 7 days:

$$C(t,t_0) = \frac{(t-t_0)^{0.6}}{10+(t-t_0)^{0.6}} C_u \qquad \text{(Eq. 2.5.8.1-1)}$$

where: C_u = ultimate creep coefficient.

Although Eq. 2.5.8.1-1 was developed for steam-cured and moist-cured concretes, it may be applied to radiant heat-cured concretes if more specific information is not available.

In the absence of creep data for local aggregates and materials, the following average value is suggested :

$$C_u = 1.88k_c \qquad \text{(Eq. 2.5.8.1-2)}$$

where:

k_c = product of applicable correction factors

$= k_{la} \times k_h \times k_s$ (Eq. 2.5.8.1-2a)

k_{la} = correction factor for loading age

k_h = correction factor for relative humidity

k_s = correction factor for size of member

For loading ages later than 7 days for moist-cured concrete and 1 to 3 days for steam-cured concrete, the loading age correction factor, k_{la}, may be taken from **Table 2.5.8.1-1.**

Table 2.5.8.1-1 Correction Factors k_{la} for Loading Age

Loading Age, days	Steam Cured, Factor k_{la}	Moist Cured, Factor k_{la}
7	0.94	1.00
10	0.90	0.95
14	0.88	0.92
28	0.83	0.84
60	0.76	0.77
90	0.74	0.74

Correction factors are based on the following equations:

For steam-cured concrete: $k_{la} = 1.13(t_{la})^{-0.094}$ (Eq. 2.5.8.1-2b)

For moist-cured concrete: $k_{la} = 1.25(t_{la})^{-0.118}$ (Eq. 2.5.8.1-2c)

where: t_{la} = loading age, days

The relative humidity correction factor, k_h, may be taken from **Table 2.5.7.1-2**. A relative humidity map taken from the *LRFD Specifications* is shown in **Figure 2.5.7.1-1**.

The size correction factor, k_s, depends on the volume to surface area of the member and may be taken from **Table 2.5.7.1-3**.

2.5.9 Coefficient of Thermal Expansion

The coefficient of thermal expansion of concrete varies with the aggregate type as shown in **Table 2.5.9-1**, which is based on ACI 209. The range for normal weight concrete is generally 5 to 7 x 10^{-6} per °F when made with siliceous aggregates and 3.5 to 5 x 10^{-6} per °F when made with calcareous aggregates. The range for structural lightweight concrete is 3.6 to 6.0 x 10^{-6} per °F depending on the type of aggregate and the amount of natural sand. For design, coefficients of 6 x 10^{-6} per °F for normal weight concrete and 5 x 10^{-6} per °F for sand-lightweight concrete are frequently used. If greater accuracy is needed, tests should be made on the specific concrete. Since the coefficient of thermal expansion for steel is also about 6 x 10^{-6} per °F, the thermal effects on precast, prestressed concrete members are evaluated by treating them as plain concrete and utilizing the coefficient of thermal expansion for concrete.

Table 2.5.9-1 Coefficients of Thermal Expansion of Concrete

Rock Type	millionths/ °F
Chert	6.6
Quartzite	5.7
Quartz	6.2
Sandstone	5.2
Marble	4.6
Siliceous Limestone	4.6
Granite	3.8
Dolerite	3.8
Basalt	3.6
Limestone	3.1

2.6 GROUT MATERIALS

2.6.1 Definitions and Applications

When precast, prestressed concrete members are placed adjacent to each other, load transfer between adjacent members is often achieved through a grouted keyway. The keyway may or may not extend for the full depth of the member. The keyway is grouted with one of several different grouting materials which are described in this section. In some bridges, no additional deck work is performed after grouting. In other bridges, a composite concrete deck may be cast on the members or the top surface of the members may be coated with a waterproofing membrane and overlaid with an asphaltic wearing course.

2.6.2 Types and Characteristics of Grout

ASTM Specification C 1107 covers three grades of packaged dry hydraulic-cement grouts (non-shrink) intended for use under applied load. These grouts are composed of hydraulic cement, fine aggregate and other ingredients and generally only require the addition of mixing water for use. Three grades of grout are classified according to the volume control mechanism exhibited by the grout after being mixed with water:

Grade A – pre-hardening volume-adjusting in which expansion occurs before hardening

Grade B – post-hardening volume-adjusting in which expansion occurs after the grout hardens

Grade C – combination volume-adjusting which utilizes a combination of expansion before and after hardening

2.6.2.1 Performance Requirements

Performance requirements for compressive strengths and maximum and minimum expansion levels are given in ASTM C 1107. Although these grouts are termed non-shrink, the intent is to provide a final length that is not shorter than the original length at placement. This is achieved through an expansion mechanism prior to any shrinkage occurring.

2.6.2.2 Materials

Different cementitious materials may be used to produce grout. These include portland cement, shrinkage-compensating cement, expansive portland cement made with special additives, epoxy-cement resins and magnesium ammonium phosphate cement (Gulyas et al 1995).

2.6.3 ASTM Tests

The properties of grout are determined using the following ASTM test methods:

C 109 Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)

C 138 Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete

C 157 Test Method for Length Change for Hardened Hydraulic-Cement Mortar and Concrete

C 185 Test Method for Air Content of Hydraulic Cement Mortar

C 827 Test Method for Change in Height at Early Ages of Cylindrical Specimens from Cementitious Mixtures

C 1090 Test Method for Measuring Changes in Height of Cylindrical Specimens from Hydraulic-Cement Grout

2.6.4 Grout Bed Materials

The same materials that are used for grouting keyways between precast concrete members may be used for grout beds to support structural and non-structural members. In some cases, the grout will be very stiff and is referred to as dry pack. Dry pack will often have a very high compressive strength because of the low water-cementitious materials ratio. It is often compacted by hand tamping.

2.6.5 Epoxy Resins

Epoxy-resin grouts can be used between precast concrete members where increased bonding and tensile capacity is required. When these are used, consideration should be given to the higher coefficient of thermal expansion and the larger creep properties of epoxy grouts.

2.6.6 Overlays

When concrete overlays are placed on precast concrete members, a 1/16- to 1/8-in. thick layer of grout consisting of one part cement, one part sand and enough water to make a thick, creamy, paint-like consistency is brushed onto the concrete surface. The grout is placed a short distance ahead of the overlay concrete. The grout should not be allowed to dry prior to the overlay placement. Otherwise, the dry grout may act as a poor surface for bonding. It is particularly important that the concrete surface be clean and sound and that the grout be well brushed into the concrete surface.

2.6.7 Post–Tensioned Members

Grouting of post-tensioned members is described in the PTI *Post-Tensioning Manual* (1990).

2.7 PRESTRESSING STRAND

Although prestressed concrete may be produced with strands, wires or bars, prestressed precast concrete bridge members are generally produced using seven-wire strand conforming to ASTM A 416 (AASHTO M203). Seven-wire strand consists of a straight center wire that is wrapped by six wires in a helical pattern. Strand sizes range from 3/8-in. to 0.6-in. diameter, as shown in **Table 2.11.1**. The larger size strands are used in prestressed concrete beams because this results in fewer strands. The use of 0.6-in. diameter strand is essential to take full advantage of high strength concrete.

2.7.1 Strand Types

Two types of strands are covered in ASTM A 416: “low-relaxation” and “stress-relieved” (normal-relaxation). However, in recent years, the use of low-relaxation strand has progressively increased to a point that normal-relaxation strand is seldom used. Two grades of strand are generally used in prestressed concrete construction. These are Grades 250 and 270, which have minimum ultimate strengths of 250,000 and 270,000 psi, respectively. In general, Grade 270 is used in prestressed concrete bridge beams. Grade 250 strand may be used where lower levels of precompression are required. In addition to smooth, uncoated strands, epoxy-coated strands are available.

2.7.1.1 Epoxy-Coated Strand

Epoxy-coated strand is seven-wire prestressing strand with an organic epoxy coating which can vary in thickness from 25 to 45 mils. Two types of coatings are available. A smooth type has low bond characteristics and is intended for use in unbonded, post-tensioned systems, external post-tensioned systems, and stay cables. An epoxy-coated strand with particles of grit embedded in the surface is used in bonded pre-tensioned and post-tensioned systems.

In addition to the strand having an external coating, it can also be manufactured with the interstices between the individual wires filled with epoxy. This prevents the entry of corrosive chemicals, either by capillary action, or other hydrostatic forces. This type of strand should be specified when there is risk of contaminants or moisture entering at the ends of tendons. Epoxy-coated strand should comply with ASTM A 882. This specification requires that all prestressing steel strand to be coated shall meet the requirements of ASTM A 416.

2.7.1.1.1 Effect of Heat

For pretensioned applications with epoxy-coated strands where accelerated curing techniques are employed, the temperature of the concrete surrounding the strand at the time of prestress transfer should be limited to a maximum of 150 F and the concrete temperature should be falling. The epoxy-coating will not be damaged if this recommended temperature is not exceeded during the curing cycle. Concrete temperatures under sustained fire exposure conditions will most likely be considerably higher than the epoxy can withstand. This could result in a complete loss of bond between the strand and the concrete. Although bridge structures may not require a specific fire resistance rating, the likelihood of vehicle fires and subsequent effects of elevated temperatures should be evaluated. More specific information on the use of epoxy-coated strand is given in the report by the PCI Committee on Epoxy-Coated Strand (1993).

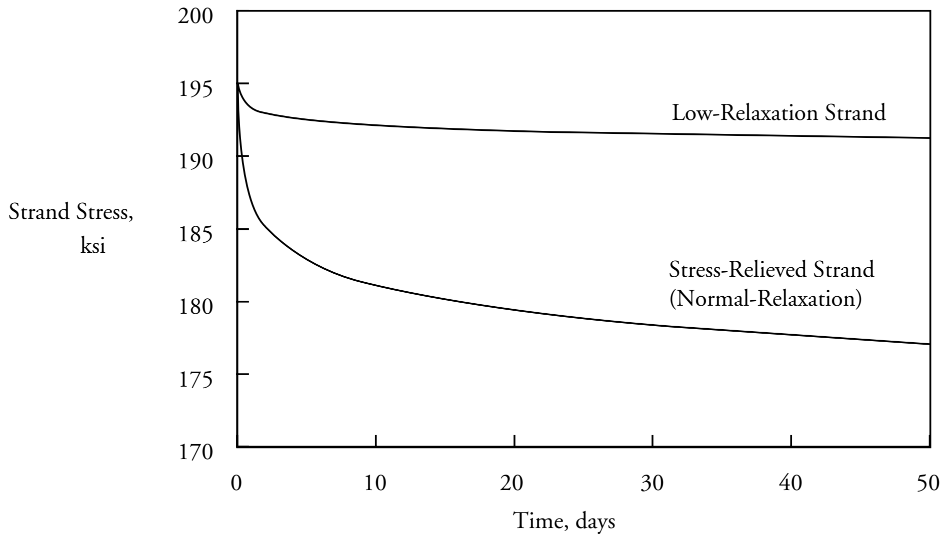
2.7.2 Material Properties

Cross-sectional properties, design strengths and idealized stress-strain curves of Grade 250 and 270 low-relaxation seven-wire strands are given in Section 2.11. Also, see Chapter 8, Section 8.2.2.5.

2.7.3 Relaxation

Relaxation is the time-dependent reduction of stress in a prestressing tendon. When a strand is stressed and held at a constant length, the stress in the strand decreases with time, as illustrated in **Figure 2.7.3-1**. Relaxation losses increase with stress level and temperature. The relaxation losses of low-relaxation strand are considerably less than the losses in normal-relaxation strand. Relaxation of a prestressing strand depends on the stress level in the strand. However, because of other prestress losses, there is a continuous reduction of the strand stress, which causes a reduction in relaxation. Therefore, several complex and empirical relationships have been proposed for the determination of relaxation losses. Several of these methods are based on the loss that would occur if the strand were under constant strain. This loss is then reduced by the effects of elastic shortening, creep and shrinkage. Early research work on relaxation was performed by Magura (1964). Subsequently, many other design recommendations have been made. The most recent recommendation is in the *LRFD Specifications.*

Figure 2.7.3-1
Comparison of Relaxation Losses



2.7.3.1
Epoxy–Coated Strand

Tests of epoxy-coated, low-relaxation strands have shown the relaxation to be significantly higher than that of uncoated strand. The use of relaxation losses equal to double the relaxation loss calculated for uncoated strand have been recommended by manufacturers. Individual manufacturers of epoxy-coated strand should be consulted for suitable relaxation loss values.

2.7.4
Fatigue Strength

If the precompression in a prestressed concrete member is sufficient to ensure an uncracked section at service loads, the stress range in the strands is not likely to be high enough for fatigue of the strand to be a critical design factor. Fatigue considerations have not been a major factor in the specification of prestressing strand for bridges because bridge beams are designed to be uncracked. The actual and allowable fatigue life of prestressing strand depend on the stress range and the minimum stress level. The stress range may be affected by the strand radius of curvature, particularly in harped strand.

2.7.4.1
Stress Range

The following design provisions for fatigue were introduced in the *LRFD Specifications:*

The stress range in prestressing tendons shall not exceed:

- 18,000 psi for radii of curvature in excess of 30 ft and
- 10,000 psi for radii of curvature not exceeding 12 ft

A linear interpolation may be used for radii between 12 and 30 ft

2.7.5
Surface Condition

In a pretensioned member, the prestressing force in a strand is transferred from the strand to the concrete by bond. Strand surface condition has long been recognized as a primary factor affecting bonding of concrete to prestressing strand. An increase in the surface roughness, such as a light surface rust, increases the bond between the concrete and the strand and results in a shorter development length. However, researchers have found it difficult to consistently quantify the effects of surface characteristics (Buckner 1994). This means that the increase in bond strength can possibly provide an extra margin of safety, but is not always consistent and should not be counted on to provide a shorter development length unless tests are conducted with specific strand. Chemicals on the strand surface can result in a reduction in bond between the concrete and strand and longer development lengths. Consequently PCI recommends that “Prestressing strand shall conform to the requirements of ASTM A

416 and shall be certified by its manufacturer to bond to concrete of a normal strength and consistency in conformance with the prediction equations for transfer and development lengths given in both ACI and AASHTO specifications."

2.7.6 Splicing

Lengths of prestressing strand can be connected using specialized strand connectors. Generally, this is not necessary in precast, prestressed concrete bridges. In situations where splicing of strands is necessary, consult the specific manufacturer's literature for details. The use of splice chucks in plant production is described in Chapter 3.

2.8 NONPRESTRESSED REINFORCEMENT

Nonprestressed reinforcement generally consists of deformed bars or welded wire reinforcement (previously refered to as welded wire fabric). Material properties and sizes of nonprestressed reinforcement are given in **Tables 2.11-2** through **2.11-4** and **Figure 2.11-1**.

2.8.1 Deformed Bars

Reinforcing bars should be deformed except plain bars may be used for spirals or for dowels at expansion or contraction joints. Reinforcing bars are generally specified to have yield strengths of 40,000 or 60,000 psi (Grade 40 or Grade 60 respectively). In some situations, a yield strength of 75,000 psi (Grade 70) may be specified, although this would be unusual in bridges.

2.8.1.1 Specifications

Reinforcing bars should conform to one of the following ASTM specifications:

A 615 Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement

A 616 Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement

A 617 Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement

A 706 Specification for Low-Alloy Deformed Bars for Concrete Reinforcement

A 767 Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement

A 775 Specification for Epoxy-Coated Reinforcing Steel Bars

The most widely used type and grade of bars conform to ASTM A 615 Grade 60 and include bars with sizes from No. 3 through No. 11, No. 14 and No. 18. When welding is required or when more bendability and controlled ductility are required, as in seismic-resistant design, low-alloy reinforcing bars conforming to ASTM A 706 should be considered.

2.8.1.2 Corrosion Protection

When coated reinforcing bars are required as a corrosion protection system, the bars may be either zinc-coated or epoxy-coated and conform to ASTM A 767 or ASTM D 3963 (AASHTO M284), respectively. Epoxy-coated reinforcing bars are generally used in bridge decks exposed to a salt environment.

2.8.2 Mechanical Splices

The most common method for splicing reinforcing bars is the lap splice. However, when lap splices are undesirable or impractical, mechanical or welded connections may be used to splice reinforcing bars. In general, a mechanical connection should develop, in tension or compression, at least 125% of the specified yield strength of the bars being connected. This is to ensure that yielding of the bars will occur before failure in the mechanical connection.

2.8.2.1 Types

Mechanical connections can be categorized as compression-only, tension-only and tension-compression. In most compression-only mechanical connections, the compressive stress is transferred by concentric bearing from one bar to the other. The mechanical connection then serves to hold the bars in concentric contact. Various types of mechanical connections are available that will handle both tension and compression forces. These connectors use a variety of couplers that may be cold swaged, cold extruded, hot forged, grout filled, steel filled or threaded. Tension-only mechanical connections generally use a steel coupling sleeve with a wedge. This is only effective when the reinforcing bar is pulled in tension. Most mechanical connection devices are proprietary and further information is available from individual manufacturers. Descriptions of the physical features and installation procedures for selected mechanical splices are described in ACI 439.3R.

2.8.3 Welded Wire Reinforcement

Welded wire reinforcement (WWR) is a prefabricated reinforcement consisting of cold-drawn wires welded together in square or rectangular grids. Each wire intersection is electrically resistance-welded by a continuous automatic welder. Pressure and heat fuse the intersecting wires into a homogeneous section and fix all wires in their proper position. WWR may consist of plain wires, deformed wires or a combination of both. WWR can also be galvanized or epoxy coated. WWR conforms to one of the following ASTM standard specifications:

A 185 Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement

A 497 Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement

A 884 Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement

Wire sizes are specified by a letter, W or D, followed by a number indicating the cross-sectional area of the wire in hundredths of a square inch. Plain wire sizes use the letter W; deformed wire sizes use the letter D. Wire sizes from W1.4 to W45 and D2 to D45 may be specified. Wire spacings generally vary from 2 to 12 in. Common stock styles and wire sizes are listed in **Table 2.11-3**. The Engineer should check on availability of styles before specifying because all sizes may not be locally available.

2.8.4 Fatigue Strength

The *Standard Specifications* limits the allowable stress range in straight reinforcement caused by live load plus impact at service load to:

$$f_f = 21 - 0.33f_{min} + 8(r/h) \quad \text{(Eq. 2.8.4-1)}$$

where:

f_f = stress range between maximum tensile stress and minimum stress

f_{min} = minimum stress level, tensile stress is positive, compressive stress is negative

r/h = ratio of base radius to height of rolled-on transverse deformation, taken as 0.3 when actual values are not known.

2.9 POST–TENSIONING MATERIALS

Post-tensioning systems may be conveniently divided into three categories depending on whether the stressing tendon is wire, strand or bar. For bridge construction, wire systems are generally not used. Further information on post-tensioning systems has been published by the Post-Tensioning Institute (1990). For details of proprietary systems, the manufacturers' literature should be consulted.

2.9.1 Strand Systems

Strand systems utilize the same strand and strand types that are used for pretensioned concrete members. In post-tensioning systems, the strands are generally combined to form a complete tendon and may consist of any quantity from a single strand to 55 strands. Anchorages for strand systems utilize the wedge principle in which the individual strands are anchored with wedges into a single tendon anchorage. In a post-tensioned multi-strand system, all strands are tensionsed at the same time. Strand tendons may be tensionsed in the plant, on the construction site, or in the finished structure.

2.9.2 Bar Systems

Bar systems generally utilize a single bar in a post-tensioning duct. The surface of the bar may be smooth with rolled threads of the required length at both ends, or the thread deformation may be rolled-on over the entire length of the bar during manufacturing. This permits the bar to be cut at any point and threaded fittings added. The bars are anchored using a threaded nut. Different types of anchorages are used at the tensioning and dead end anchorages. Bars for use in post-tensioning systems should conform with ASTM A 722 This specification covers both plain and deformed bars.

2.9.3 Splicing

Various proprietary systems are available for splicing both strand and bar systems. Couplers are required to develop at least 95% of the minimum specified ultimate strength of the tendon without exceeding the specified anchorage set (Post-Tensioning Institute, 1990).

2.9.4 Ducts

Ducts for post-tensioning systems may be either rigid or semi-rigid and made of ferrous metal or polyethylene. They may also be formed in the concrete with removable cores. The use of polyethylene ducts is generally recommended for corrosive environments. Polyethylene ducts should not be used on radii less than 30 ft because of the polyethylene's lack of resistance to abrasion during pulling and tensioning the tendons. The inside diameter of ducts should be at least 1/4 in. larger than the nominal diameter of single bar or strand tendons. For multiple bar or strand tendons, the inside cross-sectional area of the duct should be at least twice the net area of the prestressing steel. Where tendons are to be placed by the pull-through method, the duct area should be at least 2.5 times the net area of the prestressing steel.

2.10 FIBER REINFORCED PLASTIC REINFORCEMENT

2.10.1 Introduction

A newly emerging technology, with potential application in prestressed concrete, consists of prestressing bars and tendons made from fiber reinforced plastic (FRP) composites. This class of material consists of a polymer matrix such as polyester, vinyl ester, epoxy, or phenolic resin which is reinforced with fibers such as aramid, carbon or glass. These composites have tensile strengths similar to conventional strand and bar systems and are particularly suitable for applications where weight, durability, corrosion resistance and resistance to electromagnetic currents are relevant. Details of FRP composites are given in ACI 440.

2.10.2 Mechanical Properties

The mechanical properties of FRP vary significantly from one product to another. Factors such as type and volume of fiber and resin play a major role in establishing the characteristics of the product. The mechanical properties of all composites are affected by loading history, loading duration, temperature and moisture. Furthermore, standardized tests for determination of the mechanical properties have yet to be developed to the same extent that exists for steel products. Despite these limitations, a comparison of properties with conventional steel strands and bars can still be made.

2.10.2.1 Short–Term

The tensile strength, modulus of elasticity, coefficient of thermal expansion and unit weight for several types of FRP composites are given in **Table 2.10.2.1-1**, which is based on ACI 440. Because an FRP bars and tendons are anisotropic, the mechanical properties are those measured in the longitudinal or strong direction. Unlike steel, the tensile strength of FRP bars is a function of bar diameter. Due to shear lag, the fibers located near the center of the bar cross section, are not stressed as much as those near the outer surface of the bar. This results in reduced strength in larger diameter bars. FRP bars and tendons reach their ultimate tensile strength without exhibiting any yielding of the material. Consequently, fiber reinforced plastic composite bars and tendons do not possess the ductility of steel tendons. However, design methods are being developed to ensure that members reinforced with FRP composites will possess adequate ductility.

Table 2.10.2.1-1 Comparison of Properties of Steel Strand and FRP Reinforcement

Property	Steel Strand	Glass Fiber Bar	Glass Fiber Tendon	Carbon Fiber Tendon	Aramid Fiber Tendon
Tensile Strength (ksi)	270	75-175	200-250	240-350	170-300
Modulus of Elasticity (ksi)	29,000	6,000-8,000	7,000-9,000	22,000-24,000	7,000-11,000
Coefficient of Thermal Expansion (millionth/ °F)	6.5	5.5	5.5	0	-0.5
Unit Weight (pcf)	490	94-125	150	94-100	78

2.10.2.2 Long–Term

Fibers such as graphite and glass have excellent resistance to creep whereas the resins exhibit high creep. The orientation and volume of fibers have a significant influence on the creep and performance of the composites. Consequently, relaxation losses may be much higher with FRP composite bars and tendons. FRP bars and tendons exhibit good fatigue resistance.

2.10.3 Applications

Despite the above limitations, composite materials have already been used in a variety of civil engineering applications in the field on a limited basis. Further details of these applications are given in ACI 440.

2.10.4 Products

According to ACI 440, nine companies have marketed FRP composites as concrete reinforcement in North America. At the present time, there is a rapid evolution and considerable research underway on fiber reinforced plastics. Consequently, the reader should verify current products and their availability with individual manufacturers.

2.11
REINFORCEMENT SIZES AND PROPERTIES

Table 2.11-1
Properties and Design Strengths of Prestressing Steel

Seven-Wire Low-Relaxation Strand Grade 270 (f'_s = 270 ksi)

Nominal Diameter (in.)	3/8	7/16	1/2	1/2 Special	9/16	0.6
Nominal Area (A^*_s, in.^{2})	0.085	0.115	0.153	0.167	0.192	0.217
Nominal Weight (plf)	0.29	0.39	0.52	0.53	0.65	0.74
Minimum Tensile Strength (kip)	23.0	31.0	41.3	45.1	51.8	58.6
Minimum Yield Strength (kip)	20.7	27.9	37.2	40.6	46.6	52.7
$0.70f'_sA^*_s$ (kip)	16.1	21.7	28.9	31.6	36.3	41.0
$0.75f'_sA^*_s$ (kip)	17.2	23.3	31.0	33.8	38.9	44.0
$0.80f'_sA^*_s$ (kip)	18.4	24.8	33.0	36.1	41.4	46.9

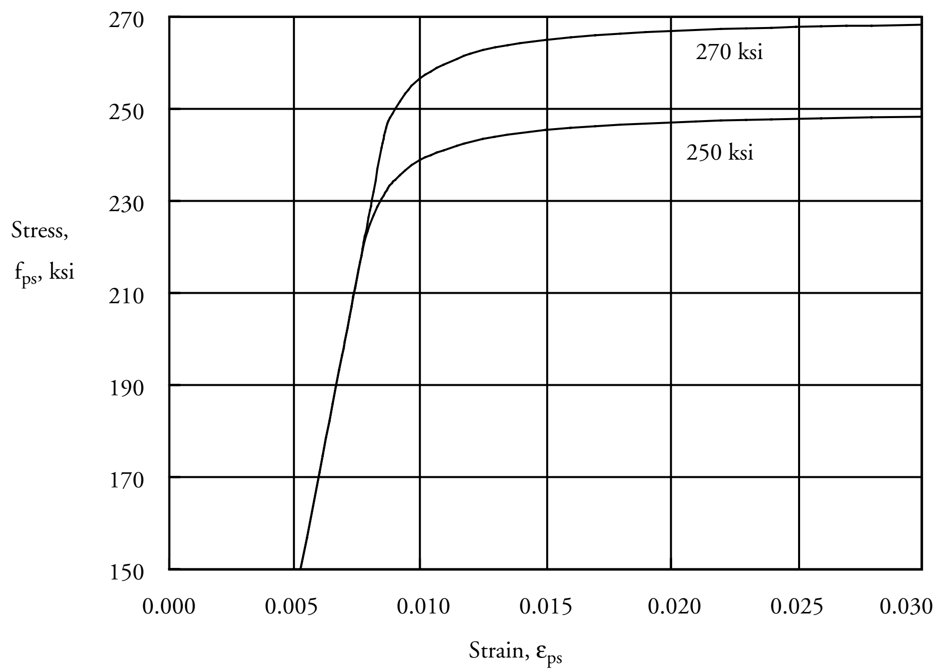
Seven-Wire Low-Relaxation Strand Grade 250 (f'_s = 250 ksi)

Nominal Diameter (in.)	3/8	7/16	1/2	0.6
Nominal Area (A^*_s, in.^{2})	0.080	0.108	0.144	0.216
Nominal Weight (plf)	0.27	0.37	0.49	0.74
Minimum Tensile Strength (kip)	20.0	27.0	36.0	54.0
Minimum Yield Strength (kip)	18.0	24.3	32.4	48.6
$0.70f'_sA^*_s$ (kip)	14.0	18.9	25.2	37.8
$0.75f'_sA^*_s$ (kip)	15.0	20.3	27.0	40.5
$0.80f'_sA^*_s$ (kip)	16.0	21.6	28.8	43.2

Deformed Prestressing Bars Grade 150 (f'_s = 150 ksi)

Nominal Diameter (in.)	5/8	1	1-1/4	1-3/8
Nominal Area (A^*_s, in.^{2})	0.28	0.85	1.25	1.58
Nominal Weight (plf)	0.98	3.01	4.39	5.56
Minimum Tensile Strength (kip)	42.0	127.5	187.5	237.0
Minimum Yield Strength (kip)	33.6	102.0	150.0	189.6
$0.70f'_sA^*_s$ (kip)	29.4	89.3	131.3	165.9
$0.75f'_sA^*_s$ (kip)	31.5	95.6	140.6	177.8
$0.80f'_sA^*_s$ (kip)	33.6	102.0	150.0	189.6

Figure 2.11-1 Idealized Stress-Strain Curve for Seven-Wire Low-Relaxation Prestressing Strand



These curves can be approximated by the following equations:

250 ksi strand

For $\varepsilon_{ps} \leq 0.0076$: $f_{ps} = 28{,}500\ \varepsilon_{ps}$ (ksi)

For $\varepsilon_{ps} > 0.0076$: $f_{ps} = 250 - 0.04/(\varepsilon_{ps} - 0.0064)$(ksi)

270 ksi strand

For $\varepsilon_{ps} \leq 0.0086$: $f_{ps} = 28{,}500\ \varepsilon_{ps}$ (ksi)

For $\varepsilon_{ps} > 0.0086$: $f_{ps} = 270 - 0.04/(\varepsilon_{ps} - 0.007)$(ksi)

Table 2.11-2 Reinforcing Bar Sizes

ASTM Standard Reinforcing Bars				
Bar Size Designation No.	Weight (plf)	Nominal Dimensions		
		Diameter (in.)	Area (in.^{2})	Perimeter (in.)
3	0.376	0.375	0.11	1.178
4	0.668	0.500	0.20	1.571
5	1.043	0.625	0.31	1.963
6	1.502	0.750	0.44	2.356
7	2.044	0.875	0.60	2.749
8	2.670	1.000	0.79	3.142
9	3.400	1.128	1.00	3.544
10	4.303	1.270	1.27	3.990
11	5.313	1.410	1.56	4.430
14	7.650	1.693	2.25	5.320
18	13.600	2.257	4.00	7.090

Table 2.11-3
Common Stock Styles of
Welded Wire Reinforcement

Style Designation		Steel Area (in.^{2}/ft)		Approximate Weight
Former Designation (By Steel Wire Gage)	Current Designation (By W-Number)	Longit.	Trans.	(lb/100 ft^2)
12x6-10x7	12x6-W1.4xW2.5	0.014	0.050	23
12x6-8x4	12x6-W2.0xW4.0	0.020	0.080	35
12x6-10x6	12x6-W1.4xW2.9	0.014	0.058	27
6x6-10x10	6x6-W1.4xW1.4	0.029	0.029	21
4x12-8x12	4x12-W2.1xW0.9	0.062	0.009	25
6x6-8x8	6x6-W2.1xW2.1	0.041	0.041	30
4x4-10x10	4x4-W1.4xW1.4	0.043	0.043	31
4x12-7x11	4x12-W2.5xW1.1	0.074	0.011	31
6x6-6x6	6x6-W2.9xW2.9	0.058	0.058	42
4x4-8x8	4x4-W2.1xW2.1	0.062	0.062	44
6x6-4x4	6x6-W4.0xW4.0	0.080	0.080	58
4x4-6x6	4x4-W2.9xW2.9	0.087	0.087	62
6x6-2x2	6x6-W5.5xW5.5	0.110	0.110	80
4x4-4x4	4x4-W4.0xW4.0	0.120	0.120	85
4x4-3x3	4x4-W4.7xW4.7	0.141	0.141	102
4x4-2x2	4x4-W5.5xW5.5	0.165	0.165	119

Availability of styles should be verified by the local supplier.

Table 2.11-4
Sizes of Wires used in Welded Wire Reinforcement

Wire Size Number		Nominal Diameter*	Nominal Weight*	Area (in.^{2}/ft of width)						
				Center to Center Spacing (in.)						
Plain	Deformed	(in.)	(plf)	2	3	4	6	8	10	12
W45	D45	0.757	1.530	2.700	1.800	1.350	0.900	0.675	0.540	0.450
W31	D31	0.628	1.054	1.860	1.240	0.930	0.620	0.465	0.372	0.310
W30	D30	0.618	1.020	1.800	1.200	0.900	0.600	0.450	0.360	0.300
W28	D28	0.597	0.952	1.680	1.120	0.840	0.560	0.420	0.336	0.280
W26	D26	0.575	0.884	1.560	1.040	0.780	0.520	0.390	0.312	0.260
W24	D24	0.553	0.816	1.440	0.960	0.720	0.480	0.360	0.288	0.240
W22	D22	0.529	0.748	1.320	0.880	0.660	0.440	0.330	0.264	0.220
W20	D20	0.504	0.680	1.200	0.800	0.600	0.400	0.300	0.240	0.200
W18	D18	0.478	0.612	1.080	0.720	0.540	0.360	0.270	0.216	0.180
W16	D16	0.451	0.544	0.960	0.640	0.480	0.320	0.240	0.192	0.160
W14	D14	0.422	0.476	0.840	0.560	0.420	0.280	0.210	0.168	0.140
W12	D12	0.390	0.408	0.720	0.480	0.360	0.240	0.180	0.144	0.120
W11	D11	0.374	0.374	0.660	0.440	0.330	0.220	0.165	0.132	0.110
W10.5		0.366	0.357	0.630	0.420	0.315	0.210	0.158	0.126	0.105
W10	D10	0.356	0.340	0.600	0.400	0.300	0.200	0.150	0.120	0.100
W9.5		0.348	0.323	0.570	0.380	0.285	0.190	0.143	0.114	0.095
W9	D9	0.338	0.306	0.540	0.360	0.270	0.180	0.135	0.108	0.090
W8.5		0.329	0.289	0.510	0.340	0.255	0.170	0.128	0.102	0.085
W8	D8	0.319	0.272	0.480	0.320	0.240	0.160	0.120	0.096	0.080
W7.5		0.309	0.255	0.450	0.300	0.225	0.150	0.113	0.090	0.075
W7	D7	0.299	0.238	0.420	0.280	0.210	0.140	0.105	0.084	0.070
W6.5		0.288	0.221	0.390	0.260	0.195	0.130	0.098	0.078	0.065
W6	D6	0.276	0.204	0.360	0.240	0.180	0.120	0.090	0.072	0.060
W5.5		0.264	0.187	0.330	0.220	0.165	0.110	0.083	0.066	0.055
W5	D5	0.252	0.170	0.300	0.200	0.150	0.100	0.075	0.060	0.050
W4.5		0.240	0.153	0.270	0.180	0.135	0.090	0.068	0.054	0.045
W4	D4	0.225	0.136	0.240	0.160	0.120	0.080	0.060	0.048	0.040
W3.5		0.211	0.119	0.210	0.140	0.105	0.070	0.053	0.042	0.035
W3	D3	0.195	0.102	0.180	0.120	0.090	0.060	0.045	0.036	0.030
W2.9		0.192	0.098	0.174	0.116	0.087	0.058	0.044	0.035	0.029
W2.5		0.178	0.085	0.150	0.100	0.075	0.050	0.038	0.030	0.025
W2.1		0.162	0.070	0.126	0.084	0.063	0.042	0.032	0.025	0.021
W2	D2	0.159	0.068	0.120	0.080	0.060	0.040	0.030	0.024	0.020
W1.5		0.138	0.051	0.090	0.060	0.045	0.030	0.023	0.018	0.015
W1.4		0.134	0.049	0.084	0.056	0.042	0.028	0.021	0.017	0.014

* Based on ASTM A 496

2.12 RELEVANT STANDARDS AND PUBLICATIONS

The following list of standards and manuals is provided for the convenience of the reader because not all documents are referenced in the text of this chapter. The complete serial designation of each document includes a year of adoption. However, since these documents are updated on a frequent basis, the year has been omitted. The reader is referred to the respective organizations for the latest revisions and year of adoption.

2.12.1 AASHTO Standard Specifications

HB *Standard Specifications for Highway Bridges*
LRFD *AASHTO LRFD Bridge Design Specifications*
HM *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*
M6 *Fine Aggregate for Portland Cement Concrete*
M31 *Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*
M32 *Steel Wire, Plain, for Concrete Reinforcement*
M42 *Rail-Steel Deformed and Plain Bars for Concrete Reinforcement*
M43 *Sizes of Aggregate for Road and Bridge Construction*
M53 *Axle-Steel Deformed and Plain Bars for Concrete Reinforcement*
M54 *Fabricated Deformed Steel Bar Mats for Concrete Reinforcement*
M55 *Steel Welded Wire Fabric, Plain, for Concrete Reinforcement*
M80 *Coarse Aggregate for Portland Cement Concrete*
M85 *Portland Cement*
M144 *Calcium Chloride*
M154 *Air-Entraining Admixtures for Concrete*
M194 *Chemical Admixtures for Concrete*
M195 *Lightweight Aggregates for Structural Concrete*
M203 *Steel Strand, Uncoated Seven-Wire for Concrete Reinforcement*
M204 *Uncoated Stress-Relieved Steel Wire for Prestressed Concrete*
M205 *Molds for Forming Concrete Test Cylinders Vertically*
M221 *Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement*
M225 *Steel Wire, Deformed, for Concrete Reinforcement*
M240 *Blended Hydraulic Cement*
M275 *Uncoated High Strength Steel Bar for Prestressing Concrete*
M284 *Epoxy Coated Reinforcing Bars*
M295 *Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete*
M302 *Ground Iron Blast-Furnace Slag for Use in Concrete and Mortars*
M307 *Microsilica for Use in Concrete and Mortar*

2.12.2 AASHTO Standard Methods of Test

T24 *Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*
T26 *Quality of Water To Be Used in Concrete*
T106 *Compressive Strength of Hydraulic Cement Mortar (Using 2 in. or 50 mm Cube Specimens)*
T131 *Time of Setting of Hydraulic Cement by Vicat Needle*

T137 *Air Content of Hydraulic Cement Mortar*

T152 *Air Content of Freshly Mixed Concrete by the Pressure Method*

T160 *Length Change of Hardened Hydraulic Cement Mortar and Concrete*

T161 *Resistance of Concrete to Rapid Freezing and Thawing*

T196 *Air Content of Freshly Mixed Concrete by the Volumetric Method*

T199 *Air Content of Freshly Mixed Concrete by the Chase Indicator*

T259 *Resistance of Concrete to Chloride Ion Penetration*

T277 *Electrical Indication of Concrete's Ability to Resist Chloride*

2.12.3 ACI Publications

207.1 *Mass Concrete*

209R *Predictions of Creep, Shrinkage and Temperature Effects in Concrete Structures*

211.1 *Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete*

211.2 *Standard Practice for Selecting Proportions for Structural Lightweight Concrete*

212.3R *Chemical Admixtures for Concrete*

213R *Guide for Structural Lightweight Aggregate Concrete*

215R *Considerations for Design of Concrete Structures Subject to Fatigue Loading*

221R *Guide for the Use of Normal Weight Aggregates in Concrete*

223 *Standard Practice for the Use of Shrinkage-Compensating Concrete*

226.1R *Ground Granulated Blast-Furnace Slag as a Cementitious Constituent in Concrete*

226.3R *Use of Fly Ash in Concrete*

308 *Standard Practice for Curing Concrete*

315 *Details and Detailing of Concrete Reinforcement*

318 *Building Code Requirements for Structural Concrete*

343R *Analysis and Design of Reinforced Concrete Bridge Structures*

345R *Guide for Concrete Highway Bridge Deck Construction*

363R *State-of-the-Art Report on High Strength Concrete*

423.3R *Recommendations for Concrete Members Prestressed with Unbonded Tendons*

439.3R *Mechanical Connections of Reinforcing Bars*

440 *State-of-the-Art Report on Fiber Reinforced Plastic Reinforcement for Concrete Structures*

2.12.4 ASTM Standard Specifications

A 82 *Specification for Steel Wire, Plain, for Concrete Reinforcement*

A 184 *Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement*

A 185 *Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement*

A 416 *Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete*

A 421 *Specification for Uncoated Stress-Relieved Wire for Prestressed Concrete*

A 496 *Specification for Steel Wire, Deformed, for Concrete Reinforcement*

A 497 *Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement*

A 615 *Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*

A 616 *Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement*

A 617 *Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement*

A 706 *Specification for Low-Alloy Deformed Bars for Concrete Reinforcement*

A 722 *Specification for Uncoated High Strength Steel Bar for Prestressing Concrete*

A 767 *Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement*

A 775 *Specification for Epoxy-Coated Reinforcing Steel Bars*

A 882 *Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand*

A 884 *Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement*

C 33 *Specification for Concrete Aggregates*

C 94 *Specification for Ready-Mixed Concrete*

C 150 *Specification for Portland Cement*

C 260 *Specification for Air-Entraining Admixtures for Concrete*

C 330 *Specification for Lightweight Aggregates for Structural Concrete*

C 470 *Specification for Molds for Forming Concrete Test Cylinders Vertically*

C 494 *Specification for Chemical Admixtures for Concrete*

C 595 *Specification for Blended Hydraulic Cements*

C 618 *Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete*

C 845 *Specification for Expansive Hydraulic Cement Concrete and Mortar*

C 989 *Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars*

C 1107 *Specification for Package Dry, Hydraulic-Cement Grout (Nonshrink)*

C 1240 *Specification for Silica Fume for Use in Hydraulic Cement, Concrete and Mortar*

D 98 *Specification for Calcium Chloride*

D 448 *Specification for Standard Sizes of Coarse Aggregate for Highway Construction*

D 3963 *Specification for Fabrication and Jobsite Handling of Epoxy-Coated Reinforcing Steel Bars*

2.12.5 ASTM Standard Test Methods

C 42 *Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*

C 109 *Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)*

C 138 *Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete*

C 157 *Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*

C 173 *Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method*

C 185 *Test Method for Air Content of Hydraulic Cement Mortar*

C 186 *Test Method for Heat of Hydration of Hydraulic Cement*

C 191 *Test Method for Time of Setting of Hydraulic Cement by Vicat Needle*

C 227 *Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)*

C 231 *Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*

C 289 *Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)*

C 342 *Test Method for Potential Volume Change of Cement-Aggregate Combinations*

C 418 *Test Method for Abrasion Resistance of Concrete by Sandblasting*

C 441 *Test Method for Effectiveness of Mineral Admixtures of Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction*

C 452 *Test Method for Potential Expansion of Portland Cement Mortars Exposed to Sulfate*

C 469 *Test Method for Static Modulus and Poisson's Ratio of Concrete in Compression*

C 512 *Test Method for Creep of Concrete in Compression*

C 586 *Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)*

C 597 *Test Method for Pulse Velocity Through Concrete*

C 666 *Test Method for Resistance of Concrete to Rapid Freezing and Thawing*

C 671 *Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing*

C 672 *Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals*

C 682 *Practice for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures*

C 779 *Test Method for Abrasion Resistance of Horizontal Concrete Surfaces*

C 803 *Test Method for Penetration Resistance of Hardened Concrete*

C 805 *Test Method for Rebound Number of Hardened Concrete*

C 827 *Test Method for Change in Height at Early Ages of Cylindrical Specimens from Cementitious Mixtures*

C 900 *Test Method for Pullout Strength of Hardened Concrete*

C 944 *Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method*

C 1012 *Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution*

C 1090 *Test Method for Measuring Changes in Height of Cylindrical Specimens from Hydraulic-Cement Grout*

C 1202 *Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*

2.12.6 Cross References ASTM-AASHTO

This list of cross references is provided for ease of comparing two similar documents. In many cases, the two documents are not identical and should not be interchanged without review of their content.

ASTM	AASHTO	ASTM	AASHTO	ASTM	AASHTO	ASTM	AASHTO
A 82	M32	A 616	M42	C 185	T137	C 618	M295
A 184	M54	A 617	M53	C 191	T131	C 666	T161
A 185	M55	A 722	M275	C 231	T152	C 989	M302
A 416	M203	C 42	T24	C 260	M154	C 1202	T277
A 421	M204	C 109	T106	C 330	M195	C 1240	M307
A 496	M225	C 150	M85	C 470	M205	D 98	M144
A 497	M221	C 157	T160	C 494	M194	D 448	M43
A 615	M31	C 173	T196	C 595	M240		

2.12.7 Cited References

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