PRESTRESSED CONCRETE FOR ARCHITECTS AND ENGINEERS
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PREFACE

As experience with prestressed concrete has progressed, its use in buildings has spread with great rapidity. This book is designed for the architect and his engineer, to enable them to take the standard prestressed concrete members and tendons now available and assemble them into buildings that are aesthetic, safe, and economical.

Fabrication procedures are described and discussed so that the reader will learn which type of prestressed member is best suited for a particular application and why. Extensive use is made of drawings and photographs to illustrate existing structures and typical details, with special emphasis on joints and connections between precast members. Span-load tables provide data on the size and weight of members required for specific applications.

Criteria governing the structural requirements for joints and connections are set forth in a code on this subject. A study of the structural analysis of prestressed concrete members in bending, shear, etc., requires a book in itself, and texts on this subject are already available. A complete code covering design is included in the Appendix. For design examples based on this code, the reader is referred to existing texts such as "Practical Prestressed Concrete," published by McGraw-Hill in 1960.

Complete coverage of the necessary material would have been
extremely difficult without the illustrations of existing structures from many different sources. Since it proved impossible to get complete data on everyone associated with each project illustrated, it was decided to mention specifically only the person and/or firm instrumental in supplying the illustration and data. The author wishes to express his appreciation to the many persons who went out of their way to assemble the photographs, drawings, and data which help to tell the story.

H. Kent Preston
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MEET PRESTRESSED CONCRETE

1-1 AESTHETIC

Prestressed concrete is a relatively new structural material offering fresh opportunities to the architect who wants to express himself. Most of the energy expended in the development of this new material during the first years of its existence in the United States was necessarily devoted to the more prosaic problems of establishing sound structural design criteria, efficient fabricating procedures, economical high-strength tendons, etc. With these problems settled, the way is open for the development of architectural forms which fully utilize the properties peculiar to this product.

Since the concrete is prestressed (put into compression) during fabrication and remains in compression under all design load conditions, the finished members can be quite shallow. In fact, the governing criterion in many designs is deflection rather than load-carrying capacity. This is not because prestressed concrete members are more limber than economically designed members of other materials (the prestressed concrete members are usually stiffer for a given span-depth ratio), but rather because prestressed concrete is both strong
and elastic. This inherent slenderness is a characteristic of pre-stressed concrete that fosters the development of smooth graceful lines while using the most economical structural cross sections.

One example of what can be accomplished is shown in Figs. 1-1 to 1-4. They illustrate a part of a huge complex of six buildings which house one of the largest science exhibits ever assembled. Combining the economical with the aesthetic, these buildings have pre-stressed concrete for all structural members as well as for the decorative wall panels.

A less striking but more readily applicable combination of the economical with the aesthetic is found in Figs. 1-5 and 1-6. In this structure the architect achieved the desired effect by supporting standardized long-span precast pre-stressed double-T sections on
Fig. 1-2. Various types of precast pretensioned S panels for United States Science Pavilion. See Fig. 1-1.

Fig. 1-3. Handling S panel for United States Science Pavilion. Band-type slings are hung from drum which can be rotated to rotate S panel. See Figs. 1-1 and 1-2.
Fig. 1-4. Fifty-two-ft-high precast pretensioned wall members in one of United States Science Pavilion buildings. S panels carry the roof members which are precast pretensioned T sections. S panel rib is 16 in. deep and averages 6 in. wide. See Fig. 1-1.

an ordinary reinforced concrete frame. Allowing the barrel-roof T sections to cantilever over the sidewalk produced a very pleasing structure.

1-2 AVAILABLE

Precast pretensioned members are available from several hundred plants throughout the United States. Although the dimensions of sections which are standard at one plant may differ slightly from those which are standard at another plant, the design drawings can be so arranged that all fabricators in the area can bid their product. Figure 1-7 illustrates many of the common precast sections and indicates their applications.

For members which are not sufficiently standardized or repetitious to warrant a setup in a pretensioned bed, there are numerous types of post-tensioned tendons now on the market. These tendons are also advantageous when size, weight, or other factors make it desirable to cast the member at the job site or in place in the structure.
Experience in the design and construction of prestressed concrete structures is also available. For precast pretensioned members one of the best sources of information is the local prestressed concrete fabricator. His literature gives full data on the standardized shapes he can supply, including span-load tables and framing details. He usually has a competent engineer on his staff or on retainer who will

Fig. 1-5. Barrel roof at Community Plaza Shopping Center, Boulder, Colo. Prestressed members by Rocky Mountain Prestress, Inc., Englewood, Colo. See Fig. 1-6.

Fig. 1-6. Framing detail for roof (Fig. 1-5). Building includes 35,000 sq ft of double T's on 35- to 50-ft spans. Reinforced concrete arches support double T's to form barrel roof.
<table>
<thead>
<tr>
<th>Section</th>
<th>W</th>
<th>D</th>
<th>Usual maximum span</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Roof</td>
</tr>
<tr>
<td>Double T</td>
<td>48 to 72&quot;</td>
<td>8 to 22&quot;</td>
<td>60'-0&quot;</td>
</tr>
<tr>
<td>Mono-wing</td>
<td>48 to 72&quot;</td>
<td>12 to 16&quot;</td>
<td>50'-0&quot;</td>
</tr>
<tr>
<td>Channel</td>
<td>30 to 48&quot;</td>
<td>6 to 16&quot;</td>
<td>50'-0&quot;</td>
</tr>
<tr>
<td>Single T</td>
<td>72 to 96&quot;</td>
<td>12 to 36&quot;</td>
<td>120'-0&quot;</td>
</tr>
<tr>
<td>Slab</td>
<td>16 to 48&quot;</td>
<td>3 to 8&quot;</td>
<td>35'-0&quot;</td>
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<tr>
<td>Girder</td>
<td>18 to 30&quot;</td>
<td>16 to 48&quot;</td>
<td>×</td>
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<tr>
<td>T joist</td>
<td>6 to 12&quot;</td>
<td>8 to 16&quot;</td>
<td>×</td>
</tr>
<tr>
<td>I joist</td>
<td>6 to 10&quot;</td>
<td>10 to 18&quot;</td>
<td>×</td>
</tr>
</tbody>
</table>

Fig. 1-7. Typical precast prestressed concrete sections. The average casting yard is equipped to produce several typical sections as standard items from forms on hand. When used as a floor, the area-covering members (double T's, mono-wings, single T's, slabs, and channels) usually are covered with a 2- to 4-in. layer of poured-in-place concrete. When used as a roof, a cover of roofing material usually is sufficient. The most common details of these members are shown in the table.

Values indicated by an asterisk are a function of the transverse spacing of the members as well as the span.

T joists usually are used as purlins to support precast roof slabs. They can be cast in the same forms used for double T's or mono-wings.

I joists are used as purlins and also with a poured-in-place concrete slab to form a composite section. (Courtesy John A. Roebling’s Sons Division, The Colorado Fuel and Iron Corp.)
help with special problems. Professional engineers who specialize in prestressed concrete can now be found in all parts of the country. Even more numerous are the consultants in structural engineering who have added a knowledge of this material to their knowledge of steel, reinforced concrete, etc., so that they can work with the one that best suits a given project. There is no excuse for bypassing prestressed concrete because of lack of knowledge on the subject. In addition to the engineers already mentioned, manufacturers of tendons have experts on their staffs who are glad to help or direct the neophyte to the proper source of information. The benefits available from the Prestressed Concrete Institute and the Portland Cement Association will be pointed out in a later chapter.

1-3 ECONOMICAL

The production-line efficiency employed in the fabrication of standardized precast pretensioned members makes them one of the most economical building materials available in many types of structures. At the job site time and labor are saved through the rapid erection of precast columns, beams, and slabs with an absolute minimum of forms and poured-in-place concrete. Then there are the additional benefits which accrue from getting the job under roof in a short time.

From the building owner's point of view the continuing benefits can be at least as important as low first cost. Larger column-free areas due to long span members provide extra space (and income) for any given area covered. Fire-resistant properties (Underwriters Laboratories labels are available) keep insurance premiums down, and maintenance for this crackless material is negligible.

A typical example of an economical prestressed concrete structure is the Mango Avenue Intermediate School, Sunnyvale, California, shown in Fig. 1-8. This school—in which the columns, walls, beams, and roof are precast prestressed concrete—was bid at $30,000 under the allowable budget. Previously 90 per cent of the structures which met the requirements for low-budget schools as determined by the Department of Finance, State of California, were constructed mainly of wood.
1-4 PROVEN

The thousands of existing buildings and bridges throughout the United States and the tens of thousands throughout the world are certainly adequate proof that prestressed concrete is a dependable building material.

Numerous fire tests have been performed by Underwriters Laboratories, Portland Cement Association, and other recognized agencies. Criteria have been established so that a member can be designed to meet any specified fire rating—it is simply a matter of proper dimensions of the cross section. Underwriters Laboratories labels are available for the sections they have tested and approved, and further tests are under way on other sections.

ACI-ASCE Committee 323, which is composed of recognized experts from both societies, has prepared and published Tentative Recommendations for Prestressed Concrete (see Appendix). This is an excellent guide to ensure a structurally sound design. Codes have also been published by PCI, and several local building-code groups.

ACI 318, Building Code Requirements for Reinforced Concrete, was extensively revised in 1963 to include complete coverage of prestressed concrete.

![Diagram of prestressed concrete elements](image)

**Fig. 1-8.** Details of prestressed concrete in Mango Avenue school. Prestressed members by Basalt Rock Co., Napa, Calif.
The safety and stability of this material are established; there is still plenty of opportunity for the exercise of ingenuity in its application.

1-5 DETAILS

If you, the reader, feel that some of the foregoing sounds like an advertisement for prestressed concrete, you could be right. The information given is authentic, but it has been presented in as painless and enticing a manner as possible. At this point it is hoped you will feel there is something in prestressed concrete for you that will make it worth your while to work your way through the unembellished details which make up most of the rest of this book and which are intended to provide the background needed for making the most of this new tool—prestressed concrete.
2

BASIC PRINCIPLES

2-1 PROPERTIES OF REINFORCED CONCRETE

Concrete is the most economical structural material available for carrying compressive loads. However, members of plain concrete cannot be used as beams subjected to bending because the tensile strength of concrete is only 10 to 15 per cent of its compressive strength. Even this tensile strength cannot always be depended upon. Shrinkage cracks can reduce it to zero or a small impact load can cause a sudden tensile failure.

In reinforced concrete, steel bars are cast into the beam in the tensile zones to carry the tensile stresses, and the plain concrete carries the compressive stresses. This has several disadvantages. The concrete in the tensile zone carries no load and is a useless dead weight except that it serves to connect the steel bars with the compressive concrete in the beam. As load is applied and tensile stress builds up in the steel bars, cracks develop in the concrete around the bars. In order to limit the magnitude of these cracks most specifications place a relatively low limit on the allowable stresses in both the bars and the concrete in reinforced concrete beams. As a result
the more economical high-strength bars and concrete which are available cannot be used.

2-2 PROPERTIES OF PRESTRESSED CONCRETE

In the process of fabricating a prestressed concrete beam all the concrete on the tensile side of the neutral axis is put under an initial compressive stress of such magnitude that all design loads applied to the structure in the future can reduce this stress but will not put the concrete in tension. Since all the concrete is carrying load, a member designed for a given span and load will be lighter in weight and of less depth than a corresponding member in reinforced concrete.

The prestress is applied in such a manner that it creates a moment of opposite sign to those that will be produced by applied dead and live loads. In the ideal design, as illustrated in Sec. 2-6, this negative moment carries all the dead load and creates the maximum allowable compressive stress on the tensile side of the member. This feature of eliminating dead-load stresses becomes especially helpful in long-span members.

2-3 STRESSES IN A PLAIN CONCRETE BEAM

Figure 2-1 shows a plain, unreinforced, un prestressed concrete beam. It is a solid rectangular beam 9 in. high by 6 in. wide on a 15-ft 0-in. span. Live load is 184 lb per ft. The following calculations cover section properties, bending moments, unit stresses, and summation of stresses.

Area = \( A_c = 9 \times 6 = 54 \text{ sq in.} \)
Dead weight = \( w_g = 54 \times 15\% = 56 \text{ lb per ft} \)
Section modulus of top fiber = \( Z_t \)
Section modulus of bottom fiber = \( Z_b \)
\( Z_t = Z_b = 6(9^2)/6 = 81 \)
Dead-weight bending moment = \( M_G \)
\( M_G = 56(15^2) \times 1\% = 18,900 \text{ in.-lb} \)
Stress in top fiber due to dead weight = \( f_{ot} \)
Stress in bottom fiber due to dead weight = \( f_{ob} \)
\( f_{ot} = 18,900 / 81 = +233 \)
\( f_{ob} = 18,900 / 81 = -233 \)
(In prestressed concrete calculations a compressive stress in the con-
crete is indicated by a plus sign and a tensile stress by a minus sign.)

Live-load bending moment = \( M_L \)
\[
M_L = 184(15^2) \times 1\% = 62,100 \text{ in.-lb}
\]
Stress in top fiber due to live load = \( f_L^t \)
\[
\begin{align*}
  f_L^t &= 62,100 + 81 = +767 \text{ psi} \\
  f_L^b &= 62,100 + 81 = -767 \text{ psi}
\end{align*}
\]

Summation of stresses:

In top fiber:
\[
f_M^t + f_L^t = +233 + 767 = +1,000 \text{ psi}
\]

In bottom fiber:
\[
f_M^b + f_L^b = -233 - 767 = -1,000 \text{ psi}
\]

Obviously this beam would fail in tension before all the load was applied.

---

![Diagram of Stresses at 6ft Span](image)

Fig. 2-1. Stresses in a plain concrete beam.
2-4 STRESSES IN A BEAM WITH CENTROIDAL PRESTRESS

Figure 2-2 shows the same beam as Fig. 2-1 except that it is prestressed by a force of 54,000 lb acting on the c.g.c. (center of gravity of the concrete section). This force creates a uniform compressive stress of +1,000 psi over the entire cross section of the beam. The stresses due to dead and live load are the same as in Fig. 2-1. The following calculations cover stress due to prestressing force and summation of stresses.

\[ f_P = \text{stress in top fiber due to final prestressing force} \]
\[ f_P = \text{stress in bottom fiber due to final prestressing force} \]
\[ f_P = f_P = F/A_c = 54,000 + 54 = +1,000 \text{ psi} \]

Summation of stresses:

In top fiber:

\[ f_P + f_D + f_L = +1,000 + 233 + 767 = +2,000 \text{ psi} \]

![Diagram of stresses](image)

Fig. 2-2. Stresses in a beam with centroidal prestress.
In bottom fiber:

\[ f_{sd}^b + f_d^b + f_{Le}^b = +1,000 - 233 - 767 = 0 \]

Under the same dead and live loads as the beam in Fig. 2-1, this beam has a net compressive stress of 2,000 psi in the top fiber and zero stress in the bottom fiber. A beam made of good concrete could safely carry these stresses.

2-5 STRESSES IN A BEAM WITH ECCENTRIC PRESTRESS

Figure 2-3 shows the same beam as Fig. 2-1 with the same prestressing force as the beam in Fig. 2-2, except that in Fig. 2-3 the prestressing force is applied 1½ in. below the c.g.c. The distance from the c.g.s. (center of gravity of the steel creating the prestressing force)
to the c.g.c. is called ε, or eccentricity. Thus, in Fig. 2-3, ε = 1½ in. The stresses due to dead load are the same as in Fig. 2-1. The following calculations cover stress due to prestressing force and summation of stresses.

In this example the stress in the concrete due to the prestressing force $F$ is not uniform because the prestressing force is not applied on the c.g.c. Stresses in the concrete due to $F$ can be found by the familiar method of replacing the eccentric force by an equal force on the c.g.c. and a couple. The force on the c.g.c. creates a uniform compressive stress over the entire section equal to the force divided by the area of the section, or $F/A_c$. The couple creates a bending moment equal to the force times the eccentricity, or $Fe$. Stress in the concrete due to the couple is equal to the moment divided by the section modulus of the concrete member, or $Fe/Z$. Thus the stress in the top or bottom fiber of the concrete member due to the eccentric prestressing force can be expressed by the equation

$$f_p = \frac{F}{A_c} \pm \frac{Fe}{Z} \quad (2-1)$$

The sign of $Fe/Z$ is plus (indicating compressive stress) when the prestressing steel is on the same side of the c.g.c. as the fiber for which the stress is being computed and minus when on the opposite side.

Equation (2-1) can be used to find the stresses due to the prestressing force at any point in a simple span member. This is true no matter what path the prestressing steel follows and regardless of its elevation at the ends of the member. The value of $\epsilon$ used in the equation is that measured at the point on the beam where the stresses are being computed. Equation (2-1) does not apply to beams continuous over one or more supports.

Use Eq. (2-1) to compute stresses in top and bottom fibers due to prestressing force.

$$f_{pT} = \frac{F}{A_c} - \frac{Fe}{Z_t} \quad (2-1a)$$

$$f_{pT} = \frac{54,000}{54} - \frac{54,000 \times 1.5}{81}$$

$$f_{pT} = 1,000 - 1,000 = 0$$

$$f_{pB} = \frac{F}{A_c} + \frac{Fe}{Z_b} \quad (2-1b)$$
\[ f_k^b = \frac{54,000}{54} + \frac{54,000 \times 1.5}{81} \]
\[ f_k^b = 1,000 + 1,000 = +2,000 \text{ psi} \]

In Fig. 2-2 the centroidal prestressing force of 54,000 lb created a compressive stress of 1,000 psi in the bottom fiber, and the beam was able to support a total load of 56 + 184 = 240 lb per ft. Having a compressive stress of 2,000 psi, the beam in Fig. 2-3 should be able to support a total load of \( 2 \times 240 = 480 \) lb per ft or a live load of \( 480 - 56 = 424 \) lb per ft. Check stresses using \( w_L = 424 \) lb per ft.

\[ M_L = 424(15^2) \times 1/3 = 143,100 \text{ in.-lb} \]
\[ f_{L}^t = 143,100 + 81 = +1,767 \text{ psi} \]
\[ f_{L}^b = 143,100 + 81 = -1,767 \text{ psi} \]

Summation of stresses:

In top fiber:

\[ f_{k}^t + f_{g}^t + f_{L}^t = 0 + 233 + 1,767 = +2,000 \text{ psi} \]

In bottom fiber:

\[ f_{k}^b + f_{g}^b + f_{L}^b = 2,000 - 233 - 1,767 = 0 \]

The beam in Fig. 2-3 has the same cross section and total prestressing force as the beam in Fig. 2-2, yet it is carrying more than twice as much live load with the same net unit stress. This greater efficiency was achieved by locating the prestressing force in the area of the beam which is subject to tension under applied loads.

### 2-6 STRESSES IN A BEAM WITH DEFLECTED TENDONS

Since the prestressing steel in Fig. 2-3 is at the same elevation for the full length of the beam, the stresses due to \( F \) are the same at all points along the beam. Since there is no bending moment at the supports due to applied loads, the net stresses at the supports are those due to \( F \). The \( \epsilon \) used in Fig. 2-3 was chosen to give maximum prestress in the bottom fiber without creating tension in the top fiber. If the \( \epsilon \) were increased, \( f_{F}^t \) would become a tensile stress. At the center of the span a certain amount of tensile stress in the top fiber from \( F \) is permissible because it is offset by the dead-load moment which is always acting in a prestressed beam. Tensile stress in the top fiber at
the end of the span is not desirable because there is no dead-load moment to relieve it. The benefits of greater eccentricity in the region where dead-load moments exist can be obtained by deflecting the prestressing steel, as shown in Fig. 2-4.

In calculations for Fig. 2-3 we found that $f_{p}^t = 0$ when $e = 1\frac{1}{2}$ in. At the center of the span the eccentricity can be increased until the prestressing force causes a tensile stress in the top fiber equal to the compressive stress from the dead-load bending moment. From previous calculations $f_{o}^t = +233$ psi. The additional $e$ required to create an equivalent tensile stress can be found from the equation

$$f_{o}^t = \frac{F_e}{Z_t}$$

Substituting numerical values,

$$233 = \frac{54,000e}{81}$$

Fig. 2-4. Stresses in a beam with deflected tendons.
from which

\[ \varepsilon = 0.35 \text{ in.} \]

On this basis we can establish the eccentricity at the center of span as \( \varepsilon = 1.50 + 0.35 = 1.85 \text{ in.} \). With this eccentricity the beam should support a live load of \( 424 + 56 = 480 \text{ lb per ft} \) without tensile stress. The following calculations are for the beam in Fig. 2-4.

Substituting in Eqs. (2-1a) and (2-1b),

\[
\begin{align*}
\sigma_P^d &= \frac{54,000}{54} - \frac{54,000 \times 1.85}{81} \\
\sigma_P^b &= 1,000 - 1,233 = -233 \text{ psi} \\
\sigma_L^d &= \frac{54,000}{54} + \frac{54,000 \times 1.85}{81} \\
\sigma_L^b &= 1,000 + 1,233 = +2,233 \text{ psi} \\
M_L &= 480(15^2) \times \frac{1}{12} = 162,000 \text{ in.-lb} \\
f_L^d &= 162,000 \div 81 = +2,000 \text{ psi} \\
f_L^b &= 162,000 \div 81 = -2,000 \text{ psi}
\end{align*}
\]

Summation of stresses at center of span under dead load only:
In top fiber:

\[ \sigma_P^d + f_d^d = -233 + 233 = 0 \]

In bottom fiber:

\[ \sigma_P^b + f_d^b = +2,233 - 233 = +2,000 \text{ psi} \]

Summation of stresses at center of span under all loads:
In top fiber:

\[ \sigma_P^d + f_d^d + f_L^d = -233 + 233 + 2,000 = +2,000 \text{ psi} \]

In bottom fiber:

\[ \sigma_P^b + f_d^b + f_L^b = +2,233 - 233 - 2,000 = 0 \]

To simplify calculations the steel tendons have been located on the c.g.c. (center of gravity of the concrete cross section) over the supports, creating a uniform compressive stress. They can just as readily be located above or below the c.g.c. to suit the details of a particular
structure as long as they do not create stresses in excess of those allowed by the specification.

A comparison of the loads carried by the beams in Figs. 2-3 and 2-4 shows that curving the prestressing steel can increase the load-carrying capacity of a beam by an amount equal to its own dead weight. This advantage of curved prestressing steel over straight steel becomes more significant as the span and the dead weight increase.

2-7 BUCKLING OF PRESTRESSED MEMBERS

The prestressing tendons create a compressive stress, often eccentrically applied, in the prestressed concrete member. When they first study prestressed concrete, many designers become concerned over the possibility of buckling in a member as a result of column action under the prestressing force.

In a prestressed member the tendon is either encased in concrete for its full length or restricted from motion with respect to the concrete at frequent intervals. The tendency for the concrete member to buckle under the compressive loading does exist, but in order to buckle it must take the tendon with it. Since the tendon is under tension and wants to remain in a straight line, it resists the effort to deflect it and no buckling takes place. This principle has been demonstrated by an actual test. An unrestrained concrete bar 4 in. by 4 in. by 100 ft. long was prestressed by a tendon located in a small hole in its center. In spite of the extreme slenderness of the bar there was no buckling. Failure occurred by crushing of the concrete when the unit stress reached the ultimate strength value.

In a properly designed prestressed concrete member there is no buckling due to the prestressing force. Compression loads other than the prestressing force cause buckling which must be resisted by the stiffness of the structural members just as in any other structural material. The tension in the tendons is sufficient only to offset the buckling due to the prestressing force.
3

MATERIALS AND CONCEPTS INTRODUCED BY PRESTRESSED CONCRETE

3-1 PRESTRESSING METHODS

There are two kinds of tensioning elements for prestressed concrete: pretensioned and post-tensioned. The prestressing tendons in a given member will be all one kind or the other or a combination of the two, depending upon conditions.

The term pretensioned means that the tendons are tensioned to their full load before the concrete is placed. They are held under tension by anchors beyond the ends of the prestressed concrete members. After the concrete has been placed and allowed to cure to sufficient strength, the load in the tendons is transferred from the external anchors into the concrete member, thus prestressing it. In the United States the standard tendons for pretensioned work are seven-wire uncoated stress-relieved prestressed concrete strands. The pretensioned method is described fully in Chap. 4.

The term post-tensioned means that the tendons are tensioned after the concrete has been placed and allowed to cure. Frequently the tendon is placed inside a flexible metal hose, the entire assembly
is placed in the form, and concrete is poured around it. After the concrete has cured, the tendon is tensioned and held under load by anchor fittings at its ends. Bond between the tendon and the concrete member is achieved by pumping the metal hose full of grout. Several types of post-tensioned tendons are discussed in Chap. 5.

3-2 STRESS LOSSES

During the fabrication of a prestressed concrete beam the tendons are elongated to a specified tension and anchored at each end under that tension. This is called the initial tension and is designated as $F_I$. Before final equilibrium is reached, several “stress-loss factors” cause a reduction in the magnitude of this tension. The tension remaining after all these losses have occurred is called the final tension and is designated as $F$.

We will consider these stress losses as they occur in the fabrication of a pretensioned member.

*Shrinkage* is a process known to everyone familiar with reinforced concrete. As concrete cures, its volume decreases slightly and it shrinks. When the tension in the tendons is transferred from the external anchors to the concrete members, the compressive force created in the concrete closes all the existing shrinkage cracks and the tendons shorten by an amount equal to the sum of the widths of the cracks. As shrinkage continues to take place, the concrete, being in compression, simply shortens an amount equal to the shrinkage and the tendons shorten with it. In the final analysis the pretensioned tendons shorten an amount equal to the total shrinkage and undergo a corresponding decrease in tension or a stress loss.

*Elastic compression* of the concrete members takes place as the pre-stressing force is applied. As in any elastic member, its magnitude is a function of the intensity of the stress and the modulus of elasticity of the concrete. Here again the pretensioned tendons shorten an amount equal to the total shortening in the member.

*Creep* of concrete, often referred to as plastic flow, is inelastic shortening which takes place over a period of time. It is a function of the intensity of the compressive stress.

*Relaxation* is a loss of stress in the steel tendons which occurs when they are tensioned to a high stress and then held at that length.
On the basis of test data and considerable experience Tentative Recommendations for Prestressed Concrete sets up formulas for the computation of stress losses due to each of the foregoing factors. It also recommends values for total stress loss to be used where some of the factors in the formulas are not known for the specific member being designed. These recommended stress losses are 35,000 psi for pretensioned members and 25,000 psi for post-tensioned members.

3-3 PRESTRESSING TENDONS

High-strength tendons are needed to offset the stress losses enumerated in the previous section. If a steel bar with a yield strength of 50,000 psi were tensioned to its full value of 50,000 psi in a post-tensioned member, it would lose 25,000 psi or one-half of its tension before equilibrium was reached. This would not permit an economical design because the concrete member would be so severely overstressed under the initial tension condition that a failure would occur. If the stresses under initial tension were limited, the prestress under final tension would be inadequate. Tendons popular in the United States are wires or strands having ultimate strengths of 235,000 to 270,000 psi and special bars with ultimate strengths around 145,000 psi.

The prestressing tension creates a greater load in the tendon than will ever again exist unless the structure is loaded well beyond the design load. Taking advantage of this fact, the initial tension is high, usually 70 per cent of the ultimate strength of the tendon, and the final tension is from 55 to 60 per cent of ultimate. Obviously the yield strength of the tendon must be greater than the initial tension. ASTM Designation A416 specifies a yield strength of not less than 85 per cent of ultimate for seven-wire uncoated stress-relieved prestressed concrete strand, and ASTM Designation A421 specifies a yield strength of not less than 80 per cent of ultimate for uncoated stress-relieved prestressed concrete wire. A typical load-elongation curve for a tendon is shown in Fig. 3-1.

Properly engineered structural materials undergo a definite yielding (which permits redistribution of loads and stresses) before failure. The ASTM designations meet this requirement by specifying a minimum elongation at rupture of $3\frac{1}{2}$ per cent in 24 in. for seven-wire
strand and 4.0 per cent in 10 in. for wire. The tendon fabricator produces wire and strand to meet this requirement by stress-relieving them.

Corrosion of the steel tendons is not a problem in a properly designed prestressed concrete member. Tendons, which are under a permanent high stress, are susceptible to corrosion in concrete containing calcium chloride, but prestressed concrete specifications forbid the use of calcium chloride, require adequate cover, etc. The result is a permanent, crackless structure.

3-4 CONCRETE

Prestressing permits the designer to capitalize on the advantages of the high-strength concrete which is now available. Concrete with a minimum cylinder strength of 5,000 psi is most popular, but specification requirements range from 4,000 to 8,000 psi. Since the concrete on the tensile side of the beam or girder is put under an initial compressive stress equal to a certain percentage of its ultimate strength, a high-strength concrete can accept a higher compressive
stress and therefore the beam can carry a greater applied live load. Conversely, for a given live load a smaller concrete cross section is required.

High-strength concrete means a low water-cement ratio and a correspondingly stiff mix. In addition prestressed concrete members are frequently I-shaped or otherwise difficult to pour. These problems have been met by the extensive use of vibrators in placing concrete and admixtures to make it more workable.

Lightweight concrete is often chosen by the designer in his continuing quest for longer spans. It is an excellent material where aggregates of adequate strength are available. Since a few lightweight concretes have a tendency to excessive creep and/or low modulus, data on the properties of the specific type to be used should be ascertained.

3-5 CAMBER

After prestressing, most prestressed concrete beams have an upward camber as a result of the negative moment created by the eccentricity of the prestressing force. A phenomenon difficult for some designers to understand is that this upward camber usually increases with the passage of time.

Control of camber so that adjacent members have equal camber is a problem that increases as the span-depth ratio increases. Methods of camber control are discussed in Chap. 9.

3-6 THE PRESTRESSED CONCRETE INSTITUTE

The Prestressed Concrete Institute (PCI) has its principal office in Chicago. Its purpose is outlined in Article II of the PCI By-Laws as follows:

ARTICLE II

The purposes of this corporation are to stimulate and advance the common interests and general welfare of the Prestressed Concrete Industry and the Precast Concrete Industry;

To collect and disseminate knowledge, statistics, ideas and information relating to design, manufacture and use of prestressed concrete and precast concrete;

To advance prestressed concrete and precast concrete acceptance and use through investigations and research relative to new applica-
tions of prestressed concrete and precast concrete and engineering processes for improvement of the design, manufacture and use of prestressed concrete and precast concrete;

To establish industry-wide standards of design and production of prestressed concrete and precast concrete to improve quality and design of product;

To perform all lawful and desirable activities within the State of Illinois and elsewhere, to promote the efficient, constructive and beneficial operation of the Prestressed Concrete Industry and the Precast Concrete Industry.

PCI was founded in 1954. By 1962 its membership included:

131 active members—firms engaged in the manufacture of prestressed concrete.

58 associate members—firms engaged in business allied to the manufacture of prestressed concrete. These are fabricators of tendons, forms, admixtures, jacking equipment, etc.

376 professional members—registered architects or engineers.

102 affiliate members—persons associated with the industry, but not technically qualified for registration.

73 junior members—architects or engineers in training.

23 student members

5 honorary members

The PCI Journal, published at regular intervals and mailed to all members, carries articles on prestressed concrete structures, plants, research and structural analysis.  

P C Items, also furnished to all members, brings news of current events in the industry.

The annual convention features several days of lectures and panel discussions. Some sessions are separated into two different rooms, with one covering subjects of interest to architects and engineers and the other covering subjects of interest to the fabricators. Booths displaying the products and equipment used in prestressed concrete are located near the meeting rooms. Technical committees meet for a day or two before the beginning of the lectures. The convention generally ends with a visit to a nearby fabricating plant or prestressed concrete structure or both.

* Superscript numbers indicate references listed in the Bibliography at the end of the chapter.
In addition to items of promotional literature which are made available to members from time to time, PCI conducts an advertising campaign in technical magazines which have nationwide coverage. At irregular intervals as the need develops, short courses are sponsored. These run for two or three days and are conducted by individuals who are proficient in the particular subjects to be covered in the course.

Technical committees are constantly at work on research, development, standardization, specifications, etc. The first sections standardized were I beams for bridges, and then box beams, piles, and channel sections. Others will follow.

A continuing fire test program at Underwriters Laboratories, sponsored by PCI, has established fire ratings for numerous building sections, and additional tests are contemplated as necessary.

3-7 THE PORTLAND CEMENT ASSOCIATION

The Portland Cement Association has been very active in developmental work on prestressed concrete.

PCA began its prestressed concrete research and development program in the fall of 1950 with the design, fabrication, and test to destruction of a 23-ft 6-in. span post-tensioned railroad bridge. The Association of American Railroads cooperated in recording the test data. The program has been continuous since then, growing to cover the various factors on which data have been needed. It has included a comprehensive test program and analysis of bond on pretensioned tendons; the design, construction, and performance of composite structures of precast prestressed beams and poured-in-place slabs; studies of various types of continuity and fire tests on numerous shapes of members used in building construction.

Technical reports on test results and conclusions reached are available through local PCA representatives or from the main office in Chicago. Other literature on design procedures, use of materials, etc., is also available.

BIBLIOGRAPHY

4

THE PRETENSIONED METHOD

4-1 BASIC OPERATION

Pretensioning is defined by the ACI-ASCE Committee on Prestressed Concrete as "a method of prestressing reinforced concrete in which the reinforcement is tensioned before the concrete has hardened." Applied to standard practice in the United States, a more specific definition would be "a method of prestressing reinforced concrete in which the reinforcement is tensioned before the concrete is placed."

Basically, one complete cycle on a casting bed has five steps:

1. Tendons are placed on the bed in the specified pattern. They are tensioned to full load and attached to anchors at each end of the bed so that the load is maintained.

2. Forms, reinforcing bars, wire mesh, etc., are assembled around the tendons.

3. Concrete is placed and allowed to cure. In many cases curing is accelerated by the use of steam or other similar methods.

4. When the concrete has reached a strength sufficient to carry the prestressing force, the load in the tendons is released from the anchors.
Since the tendons are now bonded to the concrete, they cannot move independently of the concrete. As they try to shorten, their load is transferred to the concrete by bond. This load is the prestressing force in the concrete member.

5. The tendons are cut at each end of each prestressed concrete member, and the members are moved to a storage area so the bed can be prepared for the next cycle.

4-2 TENDONS

The standard tendon for pretensioned members in the United States is the seven-wire uncoated stress-relieved prestressed concrete strand illustrated in Fig. 4-1 and covered by ASTM Specification A416. These strands have proved superior to single wires because they have much better bonding properties, take up less space for a given prestressing force, and can be placed and tensioned with less labor.

An improved seven-wire strand meeting all the requirements of ASTM Specification A416 and having 15 per cent greater strength was placed on the market in 1962. The physical properties of both the ASTM grade and the high-strength grade are given in Table 4-1. For structural analysis (but not for computing exact elongation in a casting bed) the modulus of elasticity of these strands can be taken as 28 million psi.

In a pretensioned member the load in the tendon is transferred to the concrete by bond. This transfer of load takes place in a short distance, called the transfer length, at each end of the member. For example, test data show that the full initial tension in a ⅛-in.-diameter strand is transferred to the concrete in a distance which varies from 15 to 30 in., depending upon the conditions in the particu-

![Fig. 4-1. Seven-wire uncoated stress-relieved prestressed concrete strand used in pretensioned members. (Courtesy John A. Roebling's Sons Division, The Colorado Fuel and Iron Corp.)](image-url)
<table>
<thead>
<tr>
<th></th>
<th>3/16-in. diameter</th>
<th>5/32-in. diameter</th>
<th>3/16-in. diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ASTM grade</td>
<td>Type 270K</td>
<td>ASTM grade</td>
</tr>
<tr>
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<td>20,000 lb</td>
<td>23,000 lb</td>
<td>27,000 lb</td>
</tr>
<tr>
<td>Area</td>
<td>0.0799</td>
<td>0.0854</td>
<td>0.1089</td>
</tr>
<tr>
<td>Initial tension—70% ultimate</td>
<td>14,000 lb</td>
<td>16,100 lb</td>
<td>18,900 lb</td>
</tr>
<tr>
<td>Initial stress</td>
<td>175,220 psi</td>
<td>188,520 psi</td>
<td>173,550 psi</td>
</tr>
<tr>
<td>Stress loss</td>
<td>35,000 psi</td>
<td>35,000 psi</td>
<td>35,000 psi</td>
</tr>
<tr>
<td>Final stress</td>
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<td>153,520 psi</td>
<td>138,550 psi</td>
</tr>
<tr>
<td>Final tension</td>
<td>11,205 lb</td>
<td>13,110 lb</td>
<td>15,090 lb</td>
</tr>
<tr>
<td>Final tension:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6-270K Strands</td>
<td></td>
<td>78,660 lb</td>
<td></td>
</tr>
<tr>
<td>Final tension:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7-ASTM grade strands</td>
<td>78,435 lb</td>
<td></td>
<td>105,630 lb</td>
</tr>
</tbody>
</table>

lар member. Between the two transfer lengths the only transfer of load from the strand to the concrete is the small amount due to changes in bending moment. If a member is loaded to failure, the bond between strand and concrete must be sufficient to develop the ultimate strength of the strand. Prestressed concrete members fabricated in accordance with standard requirements have the necessary bond properties.

4-3 THE FABRICATING PLANT

Figure 4-2 shows the basic elements of a casting bed on which three pretensioned members are curing.

The anchor posts must carry the full load of the tendons until the concrete has cured to the specified strength. Since the tendons are placed above the surface of the concrete slab, they create a bending moment as well as a horizontal force which must be resisted by the
anchor posts. Details of the anchor posts depend upon soil conditions. Some are pre-stressed concrete piles like those shown in Fig. 4-2, some are gravity blocks similar to those used to anchor suspension-bridge cables, and in some cases the concrete slab is made heavy enough to carry the load in compression without anchor posts. Anchor posts strong enough to hold the tendons of a large girder are often one of the most expensive items in a casting bed. For this reason casting yards with more than one bed usually have just enough beds with heavy anchor posts to meet their requirements for production of heavy members. They have additional beds with smaller anchor posts for the lighter members.

The concrete slab between the anchor posts serves as the pallet on which the members are cast. It is usually equipped with inserts to which the forms can be fastened. When a bed is to be used for casting members with deflected strands, the slab is also equipped with inserts for holding the strands down at deflection points.

Deflecting strands in a pretensioning bed involves additional equipment and quite a bit of extra work, but, as pointed out in Sec. 2-6, it eliminates the stresses due to the dead weight of the beam. This is an important factor in the longer spans, and use of deflected strands is common practice. Figure 4-3 shows a casting bed with some deflected strands and some straight strands in place and tensioned. Figures 4-4 and 4-5 show hold-downs and hold-ups for deflected strands.

4-4 OPERATING THE FABRICATING PLANT

Figure 4-6 shows a three-line casting bed in operation. The reel rack in the lower left-hand corner of the picture contains about 40 reels of strand so that 40 lengths of strand can be pulled into the bed at one time. To the right of the reel rack are the three casting lines in
Fig. 4-5. Hold-up point for deflected strands. This device is in the space between two beams so that it is not encased in concrete but is salvaged for reuse. (*Courtesy John A. Roebling's Sons Division, The Colorado Fuel and Iron Corp.*)

operation. The upper or left-hand line has just been cleared of finished beams and work is starting on the next set. The center line is full of beams which have been covered with tarpaulins and are being steam-cured. The bottom line is complete with tensioned

Fig. 4-6. Pretensioning plant of Material Service Division of General Dynamics Corp. producing pretensioned I beams. See text for full discussion.
strands and reinforcing cages ready for the side forms. As indicated by the members in the storage area, this plant was producing long-span prestressed I beams at the time this picture was taken.

Plant casting in standard steel forms and all the other facilities available for quality control enable the fabricator of pretensioned members to turn out a superior product. Sometimes inspectors or other representatives of the purchaser are inclined to forget the general superiority of the product and carry to extremes their requirements for perfection. The Prestressed Concrete Institute has published a manual entitled "Inspection of Prestressed Concrete," PCI Publication INS-109-60, which covers what should and should not be expected of a pretensioned member. Familiarity with the content of this manual will help the architect properly to represent his client on a prestressed concrete project.

Fig. 4-7. Erecting single-T section 8 ft 0 in. wide by 3 ft 0 in. deep by 110 ft 8 in. long for high school gymnasium roof in Lansdown, Md. This member, which was fabricated by Shockey Bros., Inc., of Winchester, Va., is pretensioned with twenty-four ½-in.-diameter seven-wire strands.
4-5 INSTALLING THE MEMBER

One of the big factors in the economy of pretensioned members is the low cost of erection. The finished member is shipped by truck, or in some cases by rail or barge, to the job site and set in place. Connections between members are simple, usually consisting of two plates welded together or a small amount of poured-in-place concrete around overlapping reinforcing bars. Figure 4-7 shows a 110-ft 8-in.-long 8-ft 0-in.-wide single-T section being set in place for a gymnasium roof. Thus 885 sq ft of roof is being set in one operation.

In the typical roof deck or floor, poor camber control can cause trouble for the erector. Sections like the T in Fig. 4-7 are set flange to flange and attached to each other by welding plates set in the edges of the flanges at intervals along their lengths. If there is an appreciable difference in the camber of two adjacent members, the erector must jack up one or pull down the other until the welded connections have been made. Methods of controlling camber are discussed in Sec. 9-1.

After the members have been placed and attached to their supporting members, the finishing surfaces are applied. Standard roofing materials can be applied directly to the tops of the members. For floors, the precast sections are covered with 2 or 3 in. of poured-in-place concrete for which no forms are needed. The undersurfaces are simply painted or sprayed with acoustical plaster.
5

THE POST-TENSIONED METHOD

5-1 BASIC OPERATION

Post-tensioning is defined as "a method of prestressing reinforced concrete in which the reinforcement is tensioned after the concrete has hardened." Basically, the complete operation has six steps:

1. The tendon is assembled in a flexible metal hose, and anchor fittings are attached to the ends of the tendon.

2. The tendon assembly is placed in the form and tied in place in the same manner as a reinforcing bar. Reinforcing bars, wire mesh, etc., are placed.

3. Concrete is poured and allowed to cure to the strength specified for tensioning.

4. Tendons are elongated by hydraulic jacks, and the anchor fittings are adjusted to hold the load in the tendons.

5. The space around the tendon is pumped full of cement grout under pressure.

6. Anchor fittings are covered with a protective coating.

Although the foregoing procedure is the most common, others are used to suit various conditions. In some cases a hole is cored in the
concrete and the tendon threaded through the hole just before it is to be tensioned. Holes can be cored by casting in a rubber tube of the desired shape and then withdrawing it after the concrete has set. Holes can also be cored by casting in a flexible metal hose. The hose becomes a permanent part of the structure. Since the hose is not stiff enough to maintain its position while the concrete is placed, one or more steel bars are placed inside the hose and are withdrawn after the concrete has set.

In large hollow structures the tendons are threaded through the hollow spaces and tensioned against anchor plates cast in the end block of the structure. Galvanized strands are used in these structures, and grouting is not required.

5-2 COMBINATION OF PRETENSIONED AND POST-TENSIONED METHODS

When the two methods are combined, pretensioned strands are tensioned in a straight line to provide as much of the prestressing force as possible and post-tensioned tendons are used in a deflected path to provide the remaining force.

The pretensioned–post-tensioned combination is used where some deflected tendons are needed and lack of facilities or other reasons prevent the economical use of deflected pretensioned strands. Under most conditions the combination of methods is more economical than an all-post-tensioned structure.

5-3 SYSTEMS

Several different systems or types of post-tensioned tendons are used in the United States. Procedure for fabricating a post-tensioned member is essentially the same with all systems except for the details of the tendons and their anchorages.

Most systems are patented to some degree, but there are seldom any royalty fees. Purchase of materials for a particular system from the patent holders who fabricate the parts includes permission to use the system.

Jacking equipment, grouting equipment, technical advice on use of the system, and any necessary field supervision are available from the suppliers of materials for the various systems.
Details of systems used in the United States are illustrated in later sections of this chapter.

5-4 THE POST-TENSIONED MEMBER

When the cross-sectional dimensions of a beam or girder are being established, they must provide adequate space for the tendons as well as sufficient section modulus to carry the design load.

In comparison to the load they carry, post-tensioned tendons are small in cross section and usually fit easily into the cross section of the concrete member. However, an end block is required at each end of the beam or girder so that the anchor fitting (or bearing plate under the fitting) can distribute its load into the concrete without causing excessive stresses.

Drawings of post-tensioned girders that are being put out for bid usually show details of the concrete and un­pre­stress­ed reinforcing plus the magnitude and location of the prestressing force which must remain after all stress losses have taken place. The bidder is then permitted to offer whichever recognized system he prefers. Of course the designer should make sure that the girder and end blocks he uses provide adequate room for one and preferably two or more systems to encourage competitive bidding and keep his costs down. Data on minimum spacing between anchors, distance from anchorage to edge of beam, etc., are found in the catalogs of most tendon suppliers.

Unless the members are especially large or heavy, post-tensioning one or two girders on a job is seldom economical. Jacking and grouting equipment must be brought to the job site; this represents an appreciable expense unless it can be spread over a number of members.

5-5 PARALLEL-STRAND CABLES

Each of these cables is composed of 6 to 12 seven-wire strands meeting ASTM Specification A416. Both grades of strand shown in Table 4-1 are used, with the ½ in. diameter being the most popular.

Anchor fittings consist of two main parts, an external steel socket with a tapered hole in its center and a grooved tapered plug. The plug has one groove for each strand in the cable. The cable is tensioned by attaching the strands to grips in a hydraulic jacking unit
which stretches them until they reach the specified load. A 100-ft cable will elongate 7.5 to 8 in. When the proper tension is reached, a separate plunger on the jacking unit thrusts the tapered plug into place and the load on the jack is released. Since each strand is wedged between the plug and the socket, it is held in place and its load is transferred to the socket and from the socket into the concrete. The projecting ends of the strands are burned off a short distance from the socket; the cable is pumped full of grout; and the ends and socket are encased in concrete, coated with bitumen or otherwise protected.

In some areas cables can be purchased with the strands cut to length, inserted in flexible metal hose, and coiled for shipment. For jobs of any size, however, it is usually cheaper for the contractor to purchase the component parts and fabricate his own cables at the job site. Anchor fittings and jacking equipment for the Anderson System are available from Concrete Technology Corporation in Tacoma, Washington, and for the Freyssinet System from Freyssinet Company, Inc., in New York, New York. Seven-wire strand is available from numerous manufacturers who supply it to pretensioned fabricators. Flexible metal hose is also available from numerous sources,
<table>
<thead>
<tr>
<th>Maximum socket capacity</th>
<th>Plug tendon designation</th>
<th>No. of strands</th>
<th>Strand diam, in.</th>
<th>Tendon area, sq in.</th>
</tr>
</thead>
<tbody>
<tr>
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<td>12</td>
<td>1/2</td>
<td>1.73</td>
</tr>
<tr>
<td>196 kips</td>
<td>12 - 7/16</td>
<td>12</td>
<td>7/16</td>
<td>1.31</td>
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<tr>
<td></td>
<td>8 - 1/2</td>
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<td>1/2</td>
<td>1.15</td>
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<td>144 kips</td>
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<td>3/8</td>
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<td>0.87</td>
</tr>
<tr>
<td></td>
<td>6 - 1/2</td>
<td>6</td>
<td>1/2</td>
<td>0.86</td>
</tr>
</tbody>
</table>

Fig. 5-2. Basic details of the three sizes of Anderson fittings. Tendon areas are for ASTM grade strands.

and a hose specifically for use in prestressed concrete has been developed and is marketed by Flexico Products, Inc., of Metuchen, New Jersey.

Figures 5-1 to 5-10 illustrate details and use of parallel-strand cables.

5-6 PARALLEL-WIRE CABLES

Each of these cables is composed of a number of single wires meeting ASTM Specification A421. Anchor fittings are of two types, the button-head and the wedge type. In other respects, such as assembling in metal hose, tensioning, anchoring, and grouting, these cables are similar to the parallel-strand cables described in Sec. 5-5.

In the United States the standard wire for button-head cables is 1/4 in. diameter and has a minimum ultimate strength of 240,000 psi. It is specified as ASTM-A421 Type BA. With this system it is quite
Fig. 5-3. Curved beam post-tensioned by Concrete Technology Corp. using Anderson System. Beam is a track section for monorail system used to transport visitors from center of town to Seattle World's Fair Century 21 Exposition.

easy to make fittings for any number of wires, and cable sizes have varied from two wires per cable to over forty wires per cable.

The anchor fitting is a piece of steel with one hole through it for each wire in the cable. As the cable is being fabricated, each wire is threaded through its hole and then the end of the wire is upset or "button-headed." Since the hole in the steel fitting is only large enough to permit the passage of the smooth wire, the button-headed end cannot pull through.

Fig. 5-4. Detail of Freyssinet anchor for twelve ½-in.-diameter strands.
Fig. 5-5. Freyssinet anchorage assembly is held against form by cradle composed of two bolts and connectors. Sleeve from metal hose to anchorage keeps concrete from seeping through to strands during pouring of girder.

Fig. 5-6. Freyssinet jack for tensioning twelve ½-in.-diameter strands. End of jack bears directly on anchor fitting.
Various details are used for tensioning the cable and anchoring the load, but basically the anchor fitting is attached to a jacking unit by a threaded rod or other suitable connection and the cable is tensioned. The load is transferred from the anchor fitting to a steel bearing plate embedded in the end of the concrete beam by an adjustable threaded device or by shims inserted between the fitting and the bearing plate.

Figures 5-11 to 5-13 and Table 5-1 illustrate details of parallel-wire button-head cables of the BBRV system as furnished by Joseph T. Ryerson and Son, Inc.

Wire for cables with wedge-type anchors is specified as ASTM-A421 Type WA. These cables and fittings are similar to the parallel-strand cables described in Sec. 5-5 except that they are made of a number of single wires instead of a number of strands. Anchor fittings and jacking equipment are furnished by Freyssinet Company, Inc. Up to 18 wires 0.196 in. diameter (ultimate strength, 250,000

Fig. 5-8. Minimum spacing and edge distance for Freyssinet anchors of twelve ½-in.-diameter strands.
psi) per cable and up to 12 wires 0.276 in. diameter (ultimate strength, 236,000 psi) per cable are standard sizes.

5-7 HIGH-STRENGTH BARS

The familiar alloy, heat-treated, and other high-strength bars available from many warehouses are not suitable for prestressed concrete. High yield strength, followed by high elongation before failure (discussed in Sec. 3-3), is required in addition to high ultimate strength. Bars made by Rods, Inc., and by Stressteel Corporation are subjected to cold-working and stress-relieving treatments to give them these specific properties. Ultimate strength is about 145,000 psi. At present there is no ASTM specification; however, good specifications are available from the bar manufacturers, and the subject
Max cable force at θ for parabolic-draped cables

Curves — are valid for **one-end** stressing only.
Curves — are valid for **two-end** stressing only.

Cable duct: bright metal sheathing

Max initial cable force at θ, 1,000 lb
Max final cable force at θ, 1,000 lb
(After 25,000 psi loss)

Length l, ft
Fig. 5-10. Prestressing forces obtained with Freyssinet cables of twelve $\frac{1}{2}$-in.-diameter ASTM grade strands, including effect of friction and seating of plugs. Forces obtained with high-strength strand are 15 per cent greater. Flexible metal hose around cable is of 2½ in. outside diameter. Steel area = 1.73 sq in. Ultimate capacity = 432,000 lb.

Note: Graphs indicate attainable forces governed by the following limitations: (1) Jacking force = 324,000 lb = 0.75 $f$'s. (2) Initial stress at any point along cable $\equiv 0.70 f$'s.
Fig. 5-11. Button-head wires are shown in lower right-hand corner of picture. Wires cut with square ends ready for button-heading are shown in upper left-hand corner of picture. (Courtesy of Joseph T. Ryerson & Son, Inc.)

is also covered in Tentative Recommendations for Prestressed Concrete, Secs. 304.5 to 304.5.4.

Bars are available in diameters of $\frac{3}{4}$ to $1\frac{1}{8}$ in. Maximum length is slightly over 80 ft, but longer length can be achieved through the use of couplers. Common procedure is to encase the bar in a flexible metal hose which is pumped full of grout after the bar has been tensioned. It can also be threaded through a cored hole.

A steel bearing plate is provided at each end of the concrete member, and anchor fittings of either wedge type or threaded type transfer the load in the bar to the bearing plate which distributes it to the concrete.

Table 5-2 and Figs. 5-14 to 5-16 illustrate details of Stressteel high-strength bars.

5-8 LARGE-DIAMETER STRANDS

Galvanized strands are used for structures where the prestressing tendons are not encased in concrete. A typical example is a large hollow-box girder in which the tendons pass through the open area
Fig. 5-12. BBRV movable anchor-head assembly for cable composed of button-head wires. (a) Before tensioning. (b) and (c) After tensioning.
of the box and bear only against the diaphragms in the box and against the end blocks of the girder.

Each strand is composed of seven or more galvanized wires. Sizes run from 3/8 in. diameter to over 2 in. diameter. For example, the 1 1/8-in.-diameter galvanized strand has over 60 wires and has an ultimate strength of 352,000 lb.

The strand is cut to required length in the shop, where threaded anchor fittings are attached to each end. When the strand is placed in the girder and tensioned, a nut is turned down on the threaded fitting to transfer the load to a steel bearing plate.

Table 5-3 and Figs. 5-17 and 5-18 illustrate details of large-diameter galvanized strands.
### Table 5-1. Dimensions and Properties of BBRV Cables

*Forces shown are in accordance with maximum allowable stresses specified by Bureau of Public Roads and PCI Building Code Requirements*

<table>
<thead>
<tr>
<th>Type of tendon</th>
<th>14MM or 14MF</th>
<th>28MM or 28MF</th>
<th>40MM or 40MF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of wires, diam</td>
<td>14(3/8&quot;)</td>
<td>28(3/4&quot;)</td>
<td>40(3/4&quot;)</td>
</tr>
<tr>
<td>Base-plate size, in.</td>
<td>6% x 6%</td>
<td>9% x 9%</td>
<td>11 x 11</td>
</tr>
<tr>
<td>Trumpet OD, in.</td>
<td>4</td>
<td>5</td>
<td>5%</td>
</tr>
<tr>
<td>Conduit OD, in.</td>
<td>1%</td>
<td>2%</td>
<td>2 1/2</td>
</tr>
</tbody>
</table>

### Stressing Data for Bonded (Grout-type) Tendons

<table>
<thead>
<tr>
<th>Number of wires, diam</th>
<th>1(3/4&quot;)</th>
<th>14(3/4&quot;)</th>
<th>28(3/4&quot;)</th>
<th>40(3/4&quot;)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section of wires, sq in.</td>
<td>0.04909</td>
<td>0.687</td>
<td>1.3744</td>
<td>1.963</td>
</tr>
<tr>
<td>Final force—after losses, lb</td>
<td>7,070</td>
<td>98,980</td>
<td>197,960</td>
<td>283,000</td>
</tr>
<tr>
<td>Initial force—before losses, lb</td>
<td>8,250</td>
<td>115,500</td>
<td>231,000</td>
<td>330,000</td>
</tr>
<tr>
<td>Overstressing force, lb</td>
<td>9,420</td>
<td>131,880</td>
<td>263,760</td>
<td>377,000</td>
</tr>
<tr>
<td>Ultimate force of tendon, lb</td>
<td>11,780</td>
<td>164,920</td>
<td>329,840</td>
<td>471,260</td>
</tr>
</tbody>
</table>

M—movable or stressing-end anchor; F—fixed-end anchor.

MM—tendon that can be stressed from both ends.

MF—tendon that can be stressed from one end only.

### Table 5-2. Properties of Stressteel Bars

<table>
<thead>
<tr>
<th>Bar size diam, in.*</th>
<th>Weight lb/lin ft</th>
<th>Area, sq in.</th>
<th>Minimum guaranteed ultimate strength</th>
<th>Initial tensioning load, 0.7 f's†</th>
<th>Final design load, 0.6 f's‡</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Regular</td>
<td>Special</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>¼</td>
<td>1.50</td>
<td>0.442</td>
<td>64.1</td>
<td>70.7</td>
<td>44.9</td>
</tr>
<tr>
<td>⅜</td>
<td>2.04</td>
<td>0.601</td>
<td>87.1</td>
<td>96.2</td>
<td>61.0</td>
</tr>
<tr>
<td>⅛</td>
<td>2.67</td>
<td>0.785</td>
<td>113.8</td>
<td>125.6</td>
<td>79.7</td>
</tr>
<tr>
<td>⅜</td>
<td>3.38</td>
<td>0.994</td>
<td>144.1</td>
<td>159.0</td>
<td>100.9</td>
</tr>
<tr>
<td>⅜</td>
<td>4.17</td>
<td>1.227</td>
<td>177.9</td>
<td>196.3</td>
<td>124.5</td>
</tr>
<tr>
<td>⅛</td>
<td>5.05</td>
<td>1.485</td>
<td>215.3</td>
<td>237.6</td>
<td>150.7</td>
</tr>
</tbody>
</table>

* ½-in. and ¾-in.-diam bars are available on special request.
† Losses due to creep, shrinkage, and plastic flow of concrete and steel relaxation should be deducted from this value. Overtension to 0.8f', is permitted to account for friction loss and/or wedge seating loss.
‡ Working stress in the steel should not exceed this value.
### Wedges

<table>
<thead>
<tr>
<th>Bar diam, in.</th>
<th>Part No.</th>
<th>Dimensions, in.</th>
<th>Weight, lb each</th>
<th>Part No.</th>
<th>Dimensions, in.</th>
<th>Weight, lb each</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4</td>
<td>W-6</td>
<td>1/4</td>
<td>0.22</td>
<td>HN6</td>
<td>1/4</td>
<td>0.5</td>
</tr>
<tr>
<td>3/8</td>
<td>W-7</td>
<td>1/2</td>
<td>0.32</td>
<td>HN7</td>
<td>1/4</td>
<td>0.7</td>
</tr>
<tr>
<td>1</td>
<td>W-8</td>
<td>1/2</td>
<td>0.50</td>
<td>HN8</td>
<td>1/4</td>
<td>1.0</td>
</tr>
<tr>
<td>1/2</td>
<td>W-9</td>
<td>1/2</td>
<td>0.70</td>
<td>HN9</td>
<td>1/4</td>
<td>1.5</td>
</tr>
<tr>
<td>5/8</td>
<td>W-10</td>
<td>2</td>
<td>0.80</td>
<td>HN10</td>
<td>2/3</td>
<td>2.0</td>
</tr>
<tr>
<td>3/4</td>
<td>W-11</td>
<td>2 2/3</td>
<td>1.10</td>
<td>HN11</td>
<td>2/3</td>
<td>2.0</td>
</tr>
</tbody>
</table>

### Plates

<table>
<thead>
<tr>
<th>Bar diam, in.</th>
<th>Part No., WP, TP or P*</th>
<th>No. of holes</th>
<th>Dimensions, in.</th>
<th>Weight, lb each</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4</td>
<td>6</td>
<td>1</td>
<td>2 4 2</td>
<td>4.5</td>
</tr>
<tr>
<td>3/8</td>
<td>7</td>
<td>1</td>
<td>2 4 2 2 2</td>
<td>9.5</td>
</tr>
<tr>
<td>1</td>
<td>10</td>
<td>1</td>
<td>3 6 3 3 3</td>
<td>11.7</td>
</tr>
<tr>
<td>1/2</td>
<td>11</td>
<td>1</td>
<td>3 6 3 3 3 7 1/2</td>
<td>17.8</td>
</tr>
<tr>
<td>5/8</td>
<td>6-2</td>
<td>2</td>
<td>2 4 2</td>
<td>20.8</td>
</tr>
<tr>
<td>3/4</td>
<td>7-2</td>
<td>2</td>
<td>2 4 2</td>
<td>29.7</td>
</tr>
<tr>
<td>1</td>
<td>8-2</td>
<td>2</td>
<td>2 4 2</td>
<td>9.1</td>
</tr>
<tr>
<td>1/2</td>
<td>9-2</td>
<td>2</td>
<td>3 6 3 3 3 7</td>
<td>19.0</td>
</tr>
<tr>
<td>5/8</td>
<td>10-2</td>
<td>2</td>
<td>3 6 3 3 7 11/2</td>
<td>23.4</td>
</tr>
<tr>
<td>1</td>
<td>11-2</td>
<td>2</td>
<td>3 6 3 3 7 11/2</td>
<td>34.2</td>
</tr>
<tr>
<td>1/2</td>
<td>12-3</td>
<td>3</td>
<td>2 4 2</td>
<td>43.4</td>
</tr>
<tr>
<td>5/8</td>
<td>13-3</td>
<td>3</td>
<td>2 4 2</td>
<td>57.5</td>
</tr>
<tr>
<td>1</td>
<td>14-3</td>
<td>3</td>
<td>2 4 2</td>
<td>13.6</td>
</tr>
<tr>
<td>1/2</td>
<td>15-3</td>
<td>3</td>
<td>2 4 2</td>
<td>27.6</td>
</tr>
<tr>
<td>5/8</td>
<td>16-3</td>
<td>3</td>
<td>2 4 2</td>
<td>34.0</td>
</tr>
<tr>
<td>1</td>
<td>17-3</td>
<td>3</td>
<td>2 4 2</td>
<td>52.0</td>
</tr>
<tr>
<td>1/2</td>
<td>18-3</td>
<td>3</td>
<td>2 4 2</td>
<td>64.2</td>
</tr>
</tbody>
</table>

*WP = wedge plate; TP = threaded plate; P = plate with drilled hole.

Fig. 5-14. Dimensions of wedges, nuts, and bearing plates for Stressteel bars.
Fig. 5-15. Details of Stressteel bearing plate for wedge anchor.

Fig. 5-16. Stressteel tensioning equipment. Power pump at left includes hydraulic gauge which measures tension in bar. Jack has built-on scale which measures elongation of bar during jacking operation. Hand pump operates plunger which sets wedge after bar is stretched to full load.
Table 5-3. Properties of Galvanized Strands for Post-Tensioning

<table>
<thead>
<tr>
<th>Diam, in.</th>
<th>Weight per foot, lb</th>
<th>Area, sq in.</th>
<th>Minimum guaranteed ultimate strength, lb</th>
<th>Recommended final design load, lb</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.600</td>
<td>0.737</td>
<td>0.215</td>
<td>46,000</td>
<td>26,000</td>
</tr>
<tr>
<td>0.835</td>
<td>1.412</td>
<td>0.409</td>
<td>86,000</td>
<td>49,000</td>
</tr>
<tr>
<td>1</td>
<td>2.00</td>
<td>0.577</td>
<td>122,000</td>
<td>69,000</td>
</tr>
<tr>
<td>1 1/4</td>
<td>2.61</td>
<td>0.751</td>
<td>156,000</td>
<td>90,000</td>
</tr>
<tr>
<td>1 1/8</td>
<td>3.22</td>
<td>0.931</td>
<td>192,000</td>
<td>112,000</td>
</tr>
<tr>
<td>1 1/16</td>
<td>3.89</td>
<td>1.12</td>
<td>232,000</td>
<td>134,000</td>
</tr>
<tr>
<td>1 3/16</td>
<td>4.70</td>
<td>1.36</td>
<td>276,000</td>
<td>163,000</td>
</tr>
<tr>
<td>1 1/8</td>
<td>5.11</td>
<td>1.48</td>
<td>300,000</td>
<td>177,000</td>
</tr>
<tr>
<td>1 1/4</td>
<td>5.52</td>
<td>1.60</td>
<td>324,000</td>
<td>192,000</td>
</tr>
<tr>
<td>1 3/4</td>
<td>5.98</td>
<td>1.73</td>
<td>352,000</td>
<td>208,000</td>
</tr>
</tbody>
</table>

5-9 GROUTING OF TENDONS

In Sec. 5-1 the fifth step in the post-tensioning operation involves pumping the space around the tendon full of cement grout. Once this grout has hardened, the tendon is bonded to the concrete member in much the same way that a pretensioned strand or an unstrained reinforcing bar is bonded to the member. There can be no motion between the tendon and the concrete. As indicated in Tentative Recommendations for Prestressed Concrete, Sec. 209.2.2, the ultimate strength of a prestressed member with bonded tendons is greater than the ultimate strength of an identical member with unbonded tendons.

The use of unbonded tendons is recognized since the above-mentioned section of Tentative Recommendations outlines a procedure for computing the ultimate strength of a member with unbonded tendons. Nevertheless, the author, who has sometimes been called conservative, approaches their use with caution where they furnish all or part of the load-carrying capacity of the member.

If the computed ultimate strength of a member with unbonded tendons is inadequate, it can be increased by properly placed unstressed reinforcing bars. The author’s concern, however, is not with design analysis but with fabrication. If the member is to have throughout its entire life the strength determined by the design calculations, every part of every tendon must be thoroughly protected from corrosion and fatigue. In view of the conditions on typical construction jobs this is asking a great deal. Yet if the unbonded
### Type SDS 34

![Diagram of Type SDS 34 fitting](image)

### Type SDS 35

![Diagram of Type SDS 35 fitting](image)

### Type SS 2

<table>
<thead>
<tr>
<th>Strand diam</th>
<th>Measurements, in.</th>
<th>Total weight, lb</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Type SDS34</td>
</tr>
<tr>
<td>0.600</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.835</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1½</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

All SDS 34 and 35 fittings are proofloaded to a stress in excess of the recommended design stress after being attached to the strand.

### Type SS-2

<table>
<thead>
<tr>
<th>Strand diam</th>
<th>Measurements, in.</th>
<th>Total weight, lb</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>29</td>
</tr>
<tr>
<td>1½</td>
<td></td>
<td>38</td>
</tr>
<tr>
<td>1¾</td>
<td></td>
<td>45</td>
</tr>
<tr>
<td>1¼</td>
<td></td>
<td>54</td>
</tr>
<tr>
<td>1⅛</td>
<td></td>
<td>74</td>
</tr>
<tr>
<td>1⅛</td>
<td></td>
<td>77</td>
</tr>
</tbody>
</table>

Fittings Type SS-2 can also be supplied with permanent studs and no external threads when necessary.

**Fig. 5-17.** Anchor fittings for galvanized strands. *(Courtesy John A. Roebling's Sons Division, The Colorado Fuel and Iron Corp.)*
tendon is weakened by fatigue at the anchor fitting or by corrosion at any point in its length, it will fail and the prestress it provided will be lost for the entire length of the structure.

A bonded tendon has several advantages. Repeated or fatigue loads never reach the anchor fitting, which is the most critical point for fatigue failure. The grout around the tendon is one of the best available protections against corrosion. Even if the tendon becomes weakened at one point by corrosion or other means, its load will be transferred to the member through bond, and the effectiveness of the tendon will be lost at only the one point along the length of the member.
When unbonded tendons are used, the design of the anchor fittings is especially important. The fatigue strength of most tendons, and the ultimate strength of many, is reduced by anchorages which cause nicks in the surface of the tendon. The efficiency of a fitting under both static and fatigue loads should be ascertained before it is used with unbonded tendons.

One exception to the need for grouted tendons occurs with strands that are composed of galvanized wires and that have end anchors of the type shown in Fig. 5-17 in which the individual wires are anchored in a bed of zinc. These tendons can be used in hollow-box type of structures where they are in the open part of the box and subject to periodic inspection.
6

TYPICAL MEMBERS AND THEIR APPLICATIONS

6-1 CHARACTERISTICS OF PRESTRESSED CONCRETE

Combining high-strength concrete with high-strength steel tendons produces strong members that will cover long spans with comparatively shallow sections. Throughout the development period the emphasis has been on standardization and duplication to reduce costs and turn out a product more economical than other building materials. This goal has been achieved for most of the common functional structures such as schools, office buildings, parking garages, and warehouses, which make use of standard single T's, double T's, and other sections described later in this chapter. Used in both floors and roofs, these sections provide large column-free areas at minimum cost.

With standard members well established, it is time to explore the aesthetic possibilities of prestressed concrete. There is practically no limit to the architectural effects which can be created with the strong slender members typical of this material. Of course, architects will be required to do some original thinking and many fabricators will have to be pushed into anything that does not come out of a
Fig. 6-1. Natatorium, Woodrow Wilson High School, Tacoma, Wash. Prestressed I-section girders 44 in. deep with 18-in.-wide flanges are spaced 15 ft 0 in. on centers and span 105 ft 0 in. Prestressed channel-section roof slabs are 48 in. wide by 6 in. deep. Channel slabs are designed for composite action with I sections for live load which is 25 psf. Prestressed members by Concrete Technology Corp., Tacoma, Wash.
standard form, but these things are already beginning to happen—especially in those structures where the owner can stand a little extra cost to provide beauty as well as utility.

Standardized sections do offer maximum economy. They are turned out with production-line efficiency from standard steel forms by experienced personnel. Good quality control is one of the built-in advantages of this method, and the undersurfaces of the members can be left exposed, forming the ceiling of the room. A typical finish for the underside of the members is a spray coat of pastel-shaded vermiculite or similar material applied after erection. Where acoustical properties and/or insulation are not factors, the undersurfaces are usually so smooth from the steel forms that a simple coat of paint

Fig. 6-2. Entrance hall of Administration Building, Pacific Lutheran University, Tacoma, Wash. Prestressed rectangular beams 8 in. wide by 18 in. deep spaced 15 ft 0 in. on centers span 30 ft 0 in. Prestressed channel slabs 48 in. wide by 6 in. deep are designed for composite action with beams to carry live load. Prestressed members by Concrete Technology Corp.
is an adequate finish. Hung ceilings can be added but are usually an unnecessary expense.

Fire resistance is another built-in advantage. Most prestressed members have a fire rating of 2 hr or better. As will be discussed in Sec. 8-6, any desired fire rating can be achieved by adjusting the details of the member when it is designed.

6-2 STANDARDIZED PRECAST PRETENSIONED BUILDING MEMBERS

The term standardized pretensioned members does not mean standardized in the same sense as structural steel shapes which are tabulated in the AISC handbook and which are available from every producer of structural steel. As applied to one specific prestressed concrete fabricator, it means the sections which he can make in the steel forms on hand in his plant and for which he distributes a catalogue illustrating details of the members, span-load tables, etc. Figure 1-7 shows the most common sections readily available from casting yards. Most fabricators can make several different types but seldom all types.

Once an architect has established the basic dimensions of a building, he should obtain catalogues from all prestressed fabricators within economical shipping distance of the job site—frequently 150 miles or more. Even though the various fabricators in a particular area do not make identical sections, their sections will be similar to

---

![Diagram](image)

**Fig. 6-3.** Typical dimensions of a single-T section.
Fig. 6-4. Span-load capacities of 6-ft-wide single T's of regular weight concrete. See text for illustration of use of chart. $D =$ depth of T; $W =$ weight of T, psf; $WC =$ weight of T plus 2-in. poured slab. Capacity about the same with or without 2-in. topping.

each other and the drawings can be prepared so that all can bid, thus giving the building owner the benefit of competitive bidding.

Of all the "standardized" sections the single T is probably the one for which dimensions and properties are most nearly uniform throughout the United States. This section comes in a 6-ft width and in an 8-ft width with depths varying from 12 to 36 in. as a function of the span and load. It is available in regular concrete and, in many localities, in lightweight concrete.

Figure 6-3 shows dimensions for a typical single-T section. Figures 6-4 to 6-7 are charts intended to give some idea of the span-load capacity of the various single-T sections. All final designs should be based on the specifications of the actual members available in the subject locality.

Maximum capacity of T's made of lightweight concrete is often determined by excessive camber or deflection rather than by structural strength or load-carrying capacity. The curves for these members (Figs. 6-6 and 6-7) have been stopped at approximately the spans
Fig. 6-5. Span-load capacities of 8-ft-wide single T's of regular weight concrete. \( D = \) depth of T; \( W = \) weight of T, psf; \( WC = \) weight of T plus 2-in. poured slab. Capacity about the same with or without 2-in. topping.

Fig. 6-6. Span-load capacities of 6-ft-wide single T's of lightweight concrete. \( D = \) depth of T; \( W = \) weight of T, psf. Capacity about the same with or without 2-in. topping.
Fig. 6-7. Span-load capacities of 8-ft-wide single T’s of lightweight concrete. \( D = \) depth of T; \( W = \) weight of T, psf. Capacity about the same with or without 2-in. topping.

where camber and deflection might be a problem and should be investigated.

As an example of the use of the charts in Figs. 6-4 to 6-7, select a member to carry an applied load of 85 psf on a 55-ft span. Enter chart in Fig. 6-4 at 85 psf and move to the right to an intersection with a 55-ft span. From this point move horizontally to the right or vertically upward to the next line representing a member. This is a 24-in.-deep section weighing 63 psf without a poured-in-place concrete topping. Figure 6-5 for 8-ft-wide T’s of regular-weight concrete gives a 32-in.-deep section weighing 69 psf, Fig. 6-6 for 6-ft-wide T’s of lightweight concrete gives a 24-in.-deep section weighing 46 psf, and Fig. 6-7 for 8-ft-wide lightweight concrete gives a 28-in.-deep section weighing 48 psf. From these data and a knowledge of the sections available in his area, the architect can select the section most suitable for his structure.

When single T’s, double T’s, etc., are used as roof members, they are usually covered with a waterproof roofing material and may or may not have an insulating material as well. In such designs the
precast member carries all the dead and live load; there is no composite action. When these members are used for floors, they are usually covered with a poured-in-place slab of concrete about 2 in. thick. Here the precast member must support its own dead weight plus the weight of the poured-in-place slab. After the slab has cured, it acts with the precast member as a composite section to carry live load. As a result, the live-load capacity of a given precast single T is about the same by itself or with a poured-in-place slab. Figures 6-6 and 6-7 do not list the dead weight of precast T plus poured-in-place slab because the slab could be either lightweight or regular-weight concrete with a corresponding difference in total weight.

Double-T sections were the first standardized precast prestressed concrete building members produced in the United States. They are now available throughout the country and account for more square feet of floors and roofs than any other prestressed concrete shape. The most common width is 4 ft, but 5-, 6-, and/or 8-ft widths are available from some fabricators. Depths of the precast section
vary from 8 to 22 in. A 2- to 3-in. poured-in-place slab making a composite section with the precast member is common practice when they are used as a floor. Figures 6-8 and 6-9 show typical double-T members.

While single T’s are the obvious choice for the longest spans and double T’s for the shortest spans, there is no clear-cut line at which the one stops and the other begins. In the range where either section could be used, the factors of availability, economy, and architectural suitability will influence the decision.

The author attempted to prepare charts for double T’s similar to those already presented for single T’s but without success. Variations in width of member, thickness of stems, etc., are so numerous that truly representative data could not be given. Catalogues from local producers are readily available to architects and should be used in any design except the most preliminary type. Tables 6-1 and 6-2 are presented merely as a rough guide. They cover only members of regular-weight concrete, but lightweight double T’s are also available in many localities.

These tables give the depth of section needed for a given span-load requirement. As an example, we will find the depth of member required for a floor of 45-ft span with a live load of 60 psf. Since the

| Table 6-1. Span Load for Double-T Roof Members without Concrete Topping |
|-------------------------------|---|---|---|---|---|---|---|
| Applied load, psf            |   |   |   |   |   |   |   |
|                              | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
| 150                          |    |    |    |    |    |    |    |    |
| 140                          |    |    |    |    |    |    |    |    |
| 130                          | 14 |    |    |    |    |    |    |    |
| 120                          |    | 14 |    |    |    |    |    |    |
| 110                          | 14 | 16 |    |    |    |    |    |    |
| 100                          |    | 12 | 14 |    |    |    |    |    |
| 90                           | 12 | 14 | 20 |    |    |    |    |    |
| 80                           | 12 | 16 | 20 |    |    |    |    |    |
| 70                           |    | 12 | 16 | 20 |    |    |    |    |
| 60                           | 14 | 16 | 20 |    |    |    |    |    |
| 50                           | 14 | 16 | 20 |    |    |    |    |    |
| 40                           | 12 | 14 | 20 |    |    |    |    |    |
| 30                           | 12 | 14 | 20 |    |    |    |    |    |

Dead weight, psf: 8 in. = 40; 12 in. = 48; 14 in. = 52; 16 in. = 55; 20 in. = 62.
Table 6-2. Span Load for Double-T Floor or Roof Members with 2-in. Concrete Topping

(See text for details of use)

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Dead weight, psf: 8 in. = 65; 12 in. = 73; 14 in. = 77; 16 in. = 80; 20 in. = 87.

Block corresponding to these two values is blank in Table 6-2, we move vertically upward from it and come to the number 16. Thus a 16-in.-deep member is required, and the total floor depth will be 16 in. plus 2 in. of poured-in-place concrete. From the same table, the dead weight of 16-in. double T plus 2-in. slab is given as 80 psf. Remember that these tables are only approximate, and the local fabricator’s catalogue should govern any final design.

Figure 6-10 illustrates a mono-wing section. The load-carrying capacity is the same as that of double T’s of equal depth. The joint between mono-wing sections is not visible from below, whereas the joint between double T’s is visible unless sprayed with vermiculite or similar material. Mono-wings are available from a number of plants but are not as common as double and single T’s.

Precast channel slabs are of two types. Those illustrated in Figs. 6-1 and 6-2 are comparatively wide and shallow for use as roof members. Not too many plants have forms for these members.

Where spans and/or applied loads are large in relation to the depth of member permitted, heavy channel slabs are used. These are cast in standard double-T or mono-wing forms by blocking off the projecting parts of the slab to produce the type of section shown
in Fig. 6-11. When placed adjacent to each other, their live-load capacity per square foot is roughly 1.65 times that of double T’s of equal depth.

Prestressed concrete cored flat slabs, like double T’s, come in so many different details that the only way to prepare a good design is to obtain catalogues from local fabricators. Cored holes are round, oval, or rectangular and are formed by mandrels or deflectable rubber tubes which are withdrawn after the concrete takes initial set or by corrugated cardboard tubes or boxes which are left in the member. The cored holes are excellent places for utilities and are also used as ducts for air conditioning and heating. Undersurfaces create a flat ceiling. For given span-load combinations slabs are generally shallower and heavier than double T’s.

Depths of precast slabs vary from 4 to 20 in. with 8-in. members being among the most popular. Standard widths include 16, 24, 48, and 72 in. A typical 8-in. slab in regular concrete weighs 55 psf as cast and 80 psf with a 2-in. topping. In lightweight concrete it weighs
37 psf as cast and 62 psf with a topping of 2-in. regular concrete. With 2-in. topping the lightweight concrete will carry a 65 psf floor load on a 31-ft span, and the regular concrete will carry it on a 28-ft span. Without topping the regular concrete will carry a 30 psf roof load on a 35-ft span, and the lightweight reaches 40 ft.

All the precast members discussed thus far are of the area-covering type that are placed adjacent to each other to make floors and roofs. In single-bay structures these members span from load-bearing wall to load-bearing wall. In multibay structures beams or girders are needed to support them at their interior ends. It is a characteristic of prestressed concrete structures with rectangular bays that the most economical design frequently has girders spanning the short direction and double T’s or similar members spanning the long direction.

One of the popular sections for supporting slabs or T’s is the ledger beam illustrated in Fig. 6-12. Dimension $A$ is made equal to the depth of the slabs or T’s to be supported so that their tops are flush with the top of the beam. Dimensions $D$ and $W$ vary with the span and loading to give the beam the necessary structural strength. Most fabricators can furnish precast prestressed ledger beams with an $A$ dimension to accommodate the other members they make.

As with other precast members, properties of available beams and girders should be obtained from local fabricators. The charts in Figs. 6-13 and 6-14 are intended only as a guide for preliminary design, and members selected from these charts will be the minimum size capable of handling the span-load values given. In many cases

Fig. 6-12. Precast prestressed ledger beam used for supporting slabs or T sections.
Fig. 6-14. Span-load capacities of simple-span prestressed I beams. (By permission of Concrete Technology Corp.)
slightly larger members may be required if they are made of 5,000-psi concrete.

The chart in Fig. 6-13 covers simple-span beams of rectangular cross section. Each curve is labeled with numbers which indicate its dimensions. $RB$ stands for rectangular beam. The two numbers represent width and depth. Thus $RB$ 12/36 indicates a rectangular beam 12 in. wide and 36 in. deep.

The chart in Fig. 6-14 covers simple-span I beams. Thus $IB$ 24/60 indicates an I beam with 24-in.-wide flanges and a depth of 60 in.

Working in a joint committee the American Association of State Highway Officials and the Prestressed Concrete Institute have established standardized members for prestressed concrete bridges. Such members are available throughout the country from plants that are involved in bridge work. Both the I beams and the piles find a place in building construction.

AASHTO-PCI standard I beams are illustrated in Fig. 6-15. They are designed with a large bottom flange and a small top flange for use with a poured-in-place top slab which will have composite action with the beam for carrying live load. Beams of this type are economical where a poured-in-place top flange or slab is added at the job site, creating a composite member and making full use of the large bottom flange.

Prestressed concrete piles, available in practically all parts of the United States, are one of the most durable types of pile made and can be economically competitive in first cost with all but wood piles. Prestressed to a minimum of 700 psi, they withstand shipping and extremely hard driving without cracking or spalling. Once they are in place, the 700-psi compressive stress keeps the piles crackless so that they are impervious to the freezing and thawing cycles so damaging to ordinary reinforced concrete piles.

Drawings with complete details, specifications, and design load criteria for the standard AASHTO-PCI piles are available from PCI for a nominal charge. These standards cover square and octagonal piles beginning with the 10-in. size and increasing in increments of two to 24 in. For values of $L/D$ from 0 to 12 a design load capacity of 1,000 psi is specified. For values of $L/D$ from 12 to 25 the design load capacity is given as $1,240 - 20L/D$.

Prestressed concrete sheet piles have become increasingly popular. They vary in size from 4 by 16 in. to 16 by 48 in. with the
Fig. 6-15. Dimensions of AASHO-PCI standard I beams as published in 1957 by joint committee of American Association of State Highway Officials and the Prestressed Concrete Institute. The "Recommended spans" listed in the first line refer to bridges but will give an idea of how they could apply in building work also.

usual tongue-and-groove detail. The larger sizes frequently have holes cored the full length, as do the larger square and octagonal bearing piles. One project with especially heavy loads used 24-in.-square tongue-and-groove piles as combination bearing and sheet piles. Their durability and freedom from maintenance make them particularly desirable in sea walls and marinas where exposure is constant and severe.

Two types of prestressed joists are shown in Fig. 6-16. Some fabri-
cators have separate forms for such members, and others cast them in standard double-T forms. Their chief use is as roof purlins to support slabs of Insulrock, Tectum, or similar materials. A space is left between the slabs over the center of the joist and is filled with dry pack or grout, thus keying all the members together.

A good floor system is provided by using precast prestressed I joists and a poured-in-place slab to create a composite system, as illustrated in Fig. 10-2. At the present time this system is not as popular in the continental United States as the double- and single-T systems, but it is available in some areas.

6-3 MEMBERS CAST AT JOB SITE

The chief reasons for casting members at the job site are because they are too long or heavy to ship from a prestressing plant or because precast members could not be satisfactorily framed into the structure. These members are post-tensioned because the cost of setting up pretensioning facilities at the site is more than one building could absorb.

The design of a member to be cast at the job site should include a study of the procedure for casting and erecting it. It can be cast on the ground, cured, post-tensioned, and lifted into place; or it can be cast in place, cured, and post-tensioned before the supporting formwork is removed.

Casting on the ground keeps formwork and temporary supporting structure to a minimum and simplifies the labor of the concrete casting operation. On a properly scheduled job the prestressed members (which often have to cure 3 to 4 weeks before tensioning) can be cast so that they are cured, tensioned, and grouted by the time the other

Fig. 6-16. Typical precast joists.
parts of the structure are ready to receive them. The finished members are lifted into place by cranes or, in the case of especially heavy members, cast directly under their final position and jacked up their supporting columns with hydraulic jacking units. Casting on the ground is a good method if the details of the structure can be adapted to supporting and anchoring the heavy precast girder in its proper place.

Although casting the member in place is not as economical as some other procedures, it is frequently the only way to get a satisfactory structure—and do not forget that a reinforced concrete member would have to be cast in place too and would contain considerably more concrete. Of course the 3- to 4-week curing period before post-tensioning and removal of temporary shoring may be a nuisance on some jobs. The long curing period is due to specification requirements of 4,500- or 5,000-psi concrete before post-tensioning and the lack of steam-cured facilities for site-cast members. A big advantage of cast-in-place members is that the tendons can pass through adjacent walls, columns, etc., so that when the tendons are tensioned they tie the entire structure solidly together.

When the member to be tensioned is attached to other members in the structure at the time of tensioning, careful consideration should be given to their interaction during the tensioning operation. As a simple example, consider a rigid frame composed of two vertical columns fixed at their foundations with a post-tensioned girder

![Post-tensioned I-section girder cast at plant of Eastern Prestressed Concrete Corp., Line Lexington, Pa., and trucked 30 miles to be installed in a post office annex in Philadelphia. This was one of fourteen such girders varying in length from 30 to 56 ft. For further details see text and Fig. 6-18.](image-url)
framed to their tops and spanning between them. When the cables are tensioned, the girder will shorten as the shrinkage cracks close and as elastic compression takes place. As the girder shortens, the top of the column to which it is attached will move with it, thus bending about its base. However, it takes a force to make the top of this column move horizontally, and this force comes from the prestressing cables. As a result a portion of the prestressing force intended for the girder is absorbed by the column and the girder is under- prestressed. This particular problem has been solved by designing the column with a temporary hinge at the bottom which remains until the prestressing operation is completed. Then the base is made rigid by welding steel components together, adding concrete, or both.

6-4 GIRIERS$^{1-3}$

There is no definite line of demarcation between prestressed beams and prestressed girders as there is between rolled structural steel beams and fabricated plate girders. One approach is to define a beam as a member which can be fabricated with pretensioned strands furnishing all the prestressing force and to define a girder as a member in which some or all of the tendons are post-tensioned because the prestressing force required exceeds the capacity of the average casting bed. Many of the members cast at the job site as discussed in the preceding section would fall into this classification for girders.

Figures 6-17 and 6-18 illustrate a post-tensioned I-section girder which was precast and trucked to the job site where it was erected over busy railroad tracks with a minimum of interference with traffic. Statistics on this particular girder are: length, 53 ft 5½ in.; depth, 8 ft 6 in.; width at end, 4 ft 0 in.; top flange width, 4 ft 0 in.; bottom flange width, 2 ft 0 in.; web thickness, 7 in.; weight, 60 tons; prestressing force, 1,750,000 lb provided by 14 ft 1¾-in.-diameter Stressteel bars; applied load, 5,450 lb per ft plus a concentrated column at center of span of 408,000 lb; concrete strength, 6,000 psi. Additional details on this structure are covered in an article in the April, 1962, issue of Civil Engineering.

Prestressed girders are usually custom-designed for a specific project and cast in wooden forms. Sometimes the standard steel

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* Superscript numbers indicate references listed in the Bibliography at the end of the chapter.
forms for the I-beam sections in Fig. 6-15 can be adapted economically to produce a desired girder cross section. One simple expedient is to spread the forms farther apart, thus increasing the thickness of the web and both flanges. Another is to add side plates, thus increasing the depth of the top flange and the overall depth of the member.

The post-tensioned connection detail shown in Fig. 6-18 provides continuity between girder and column to create a rigid frame, but a rigid frame can also be obtained with reinforcing bars projecting from each member into a cavity which is filled with poured-in-place concrete after the bars are welded together.

6-5 FOLDED PLATES

Prestressed folded plates offer an economical solution to the problem of providing long column-free areas. They can be either poured-in-place post-tensioned or precast pretensioned, to suit the conditions of a particular structure.

Figures 6-19 and 6-20 illustrate a 123-ft-long cast-in-place folded plate roof over a 50-lane bowling alley. Each 22-ft corrugation is post-tensioned with 14 Freyssinet cables composed of 12 wires, 0.276
in. diameter per cable. Eight cables are straight in the bottom flange and three follow parabolic curves in each web. Two 22-ft corrugations (Fig. 6-20) were cast at one time, cured to 5,000 psi in 5 days, and then post-tensioned. During the curing period forms, reinforcing, etc., for the next two corrugations were placed and concrete poured. After the first two corrugations were tensioned, their forms were stripped, moved to the space adjacent to those still curing, and set up for the next two corrugations. The 8-in. open joint (Fig. 6-20) between adjacent pours was filled with concrete after the corrugations on each side of it had been tensioned. Until the joint was poured and cured, temporary tie rods were used to hold in place the inclined web and its section of top flange. A complete, illustrated article on this structure is printed in Journal of the Prestressed Concrete Institute for March, 1960.

Figure 6-21 is an end view of a school roof composed of precast pretensioned folded plate sections fabricated by Perma-Stress, Inc. They have used these sections on spans up to 72 ft with 8-ft cantilevers at each end for a design live load of 40 psf, and tested them to twice live load by plugging the ends and filling them with water. The section is 8 ft wide by 28 in. high and 3½ in. thick. The precast section included a raised lip along the edge of the top flange for the full length of the section. When the members had been placed side by side in the roof, a flash cap was placed over adjacent lips, completely sealing the joint between the two members.

![Fig. 6-19. Aerial view of Cloverleaf Lanes Bowling Alley in Dade County, Fla. Architect: Alfred Browning Parker of Miami. Prestressing by R. H. Wright, Inc., of Ft. Lauderdale. Folded plates cover column-free area 120 by 286 ft. Flat roof section in foreground is precast double T's over restaurant, cocktail lounge, etc. See Fig. 6-20 and text.](image-url)
Fig. 6-20. Structural details of folded-plate roof over Cloverleaf Lanes Bowling Alley. See Fig. 6-19 and text. (Reproduced from J. Prestressed Concrete Inst., March, 1960.)
WALL PANELS\textsuperscript{12–14}

The field for practical application of precast concrete wall panels has been greatly increased by prestressing. Development of cracks during shipment and erection, a major problem limiting the size of ordinary reinforced panels, is virtually eliminated by prestressing. Even if cracks are caused by handling, they are tightly closed by the constant prestress compression once the panel is in place and free of temporary handling stresses. The only remaining limitations on size of precast panels are those posed by shipping clearances.

An infinite variety of finishes is available, from plane surfaces through exposed aggregates of almost any color to intricate patterns such as that shown in Fig. 6-22, which was achieved with the use of a plastic form material.

Framing is simple and inexpensive, as illustrated by details in Figs. 6-24, 6-26, and 6-27. The need for lintels is eliminated, and panels can be supported on clip angles spaced at intervals instead of on continuous spandrel beams. It is apparent that panels of shaped sections such as single and double T’s can be used as load-bearing members, but relatively thin solid slabs also make good load-bearing panels. For example, Basalt Rock Company, Inc., of Napa, California, has a column-free building three sides of which are composed of prestressed panels 4 in. thick by 8 ft 0 in. wide by 18 ft 0 in. high. The roof is composed of 50-ft clear-span 14-in. double T’s which cantilever 14 ft one end and 4 ft the other end.

\textbf{Fig. 6-21.} Precast pretensioned folded-plate roof members on Florida school. Fabricated by Perma-Stress, Inc., of Holly Hill, Fla.
Fig. 6-22. Prestressing made it feasible to handle these 7½-ton precast wall panels which are 7 ft 6 in. wide by 41 ft 0 in. high and only 4 in. thick. Cast by San Diego Prestressed Concrete Co., they are part of the May Co. Department Store.

Fig. 6-23. Precast prestressed single T's combine architectural and load-carrying function as wall panels of Charlottetown Office Building, Charlotte, N.C. See Fig. 6-24.
Fig. 6-24. Crossarms precast as part of single-T wall panels carry double-T floor members. All T's plus prestressed piles supporting building fabricated by Concrete Materials, Inc., Charlotte, N.C. See Fig. 6-23.

Sheets or slabs of insulation are sometimes cast into the wall panel instead of attached to the surface. These "sandwich panels" have a relatively thin layer of concrete on each side of the insulation and are solid along the edges, which are prestressed to provide the necessary structural strength. In wide slabs there may also be a solid section down the middle. A variation of the insulated panel is a double T with a 5½- or 6-in.-thick flange, most of which is made up of the slabs of insulation.

Fig. 6-25. Plant and office of Thompson Ramo Wooldridge, Inc., Worthington, Ohio. Vertical pretensioned double-T wall panels fabricated by Permacrete Products Corp. of Columbus create interesting appearance. See Fig. 6-26. Architect: Brooks and Coddington.
6-7 LIFT SLABS 15–22

The basic principle of lift-slab construction is that of casting all floor slabs and roof at ground level and then raising the finished slabs to final elevation after they have been cured and post-tensioned. This method should be investigated on multistory buildings where flat-slab construction is desirable.

Advantages of lift-slab construction include practically no form cost since each slab is cast directly on the one below it until all floors and roof are cast, elimination of scaffolding, no pouring of concrete high in the air, and minimum slab thickness because of the prestressed feature. These advantages are at least partially offset by the cost of raising the slabs from the ground to their final elevation. They are raised by jacking units mounted on each column and coordinated from a central “console.” The jacking units and their controls are expensive and their operation requires specialized experience, so that this operation must be done by a firm specializing in it.

Typical procedure for constructing a lift-slab building is as follows:

1. Set columns in place and anchor them to their footings.
2. Cast bottom slab, usually on grade, in its final position.
3. After bottom slab has set, cover it with bond-preventing material so that next slab can be poured directly on it.
4. Place collar around each column. This collar will connect slab to column when raised to final position.
5. Place reinforcing steel, prestressing tendons, electrical conduits, etc.
6. Pour slab. When it has cured sufficiently, cover with bond-preventing material, prepare and pour next slab, etc.
7. When a given slab has reached the required strength, it can be post-tensioned and grouted. A slab cannot be lifted until it has been tensioned.
8. Lift top slab to final elevation and make permanent attachment of collars to columns.
9. Lift next slab to its final location, etc. Temporary guys are used to keep the structure plumb until the slabs are in final position and permanent shear walls or other details are in place.

BIBLIOGRAPHY

5. Gym Roof of Concrete Has Record Span, *Modern Concrete*, October, 1961, pp. 57, 61.
7

TYPICAL DETAILS OF MEMBERS

7-1 COMPOSITE STRUCTURES$^1-3$ *

Maximum economy is achieved in buildings composed of precast pretensioned members and poured-in-place concrete designed and erected so that the composite sections form a continuous or rigid-frame structure. Numerous load tests conducted throughout the country have proved that full composite action and continuity are obtained. One good example is a report by Jack R. Janney and John F. Wiss entitled Load-deflection and Vibration Characteristics of a Multi-story Precast Concrete Building, which appeared in *Journal of the American Concrete Institute*, April, 1961. Under Conclusions, this report states “the load tests demonstrated that an assembly of precast units properly designed to act with cast-in-place topping results in an effective composite floor system which acts as if it were monolithic”; also “the degree of composite action combined with the high-strength concrete used for the precast members resulted in a greater stiffness of structural elements ... than is normally encountered in a cast-in-place reinforced concrete structure with similar dimensions.”

* Superscript numbers indicate references listed in the Bibliography at the end of the chapter.
Fig. 7-1. Precast pretensioned beam is placed on column formwork. Horseshoe reinforcing bars projecting from top of beam will tie it to poured-in-place slab.

Fig. 7-2. Setting precast pretensioned slabs on precast pretensioned beams. Note temporary shoring halfway between beams to support precast slab until poured-in-place slab has cured to form composite section.
The construction pictures of an addition to Sheraton's Princess Kaiulani Hotel in Honolulu (Figs. 7-1 to 7-4) illustrate some of the details of composite construction. For this structure the designer elected to use precast pretensioned beams and slabs in conjunction with poured-in-place columns and topping.

The reinforcing bars in Fig. 7-3 are simply laid in place over the negative-moment area and are made long enough to extend to the end of the negative-moment region. Their full tensile strength is developed by bond with the poured-in-place slab. In some instances the reinforcing bars which carry the tensile load to a joint are cast with the precast member and project from the end of it. These bars

Fig. 7-3. Column has been poured and precast pretensioned slabs and reinforcing placed. Heavy bars running from lower left-hand corner of picture toward upper right will provide tension steel for negative moment in composite beam. Lighter bars at right angles are negative-moment steel for slab. Poured-in-place slab will make a composite section with bars and precast beams and slabs.
must be connected to similar bars projecting from another precast member on the other side of the joint so that the tensile load is carried across the joint. Where space permits, the simplest means of transferring load from one bar to another is to lap them the proper distance and pour concrete around them. Where space is limited, the bars are connected by welding, as shown in Fig. 7-5. A connecting angle is most efficient because only pure tension is developed. Lapped bars tend to rotate some as tension is applied, and the joint between them should be longer than the minimum required to provide just enough weld to carry the tension. Mild-steel bars can be welded without
problems. Intermediate-grade bars can be welded with proper electrodes and procedure. High-carbon bars should not be used where welding is required.

Obtaining adequate bond between the precast and the poured-in-place elements of the structure to produce full composite action is not difficult. Section 212 of Tentative Recommendations for Prestressed Concrete (see Appendix) spells out the required details. In all the tests of composite sections which have been loaded to failure, the author is not aware of any in which the cause of failure could be attributed to insufficient bond between precast and poured-in-place concrete. Sometimes a question is raised concerning the effect of shrinkage of poured-in-place concrete which is resting on and bonding to precast concrete that has already sustained at least a large part of its shrinkage,\textsuperscript{4,5} but the problem has not proved significant enough to change the design criteria set forth in Tentative Recommendations.

The combination of precast members with poured-in-place concrete offers many economies to the fabricator. Much of the concrete is poured and cured at the casting yard. On the job formwork is eliminated except for small filler strips along beams, girders, and columns because the job-site concrete is placed directly on the precast members. Electrical conduit, etc., can be placed without interference on top of the precast slabs or T's and encased in the concrete topping. The dimensional tolerances of the precast sections can be set so that they are relatively easy for the precaster to meet and yet the members frame together satisfactorily and minor discrepancies are covered by the poured-in-place concrete.

A number of factors influence the magnitude of camber in a given prestressed member, and means of control are discussed in Sec. 9-1. In considering composite sections, we note that camber can change the thickness of the poured-in-place portion of a member. Consider a 40-ft span double-T floor system with 2 in. of poured-in-place topping. How should the topping be placed if the camber of the T sections is such that \(\frac{1}{2}\)-in. camber will remain after the topping has been placed? From the structural designer's point of view the only logical procedure is to put a 2-in. layer of concrete over the entire area. This gives both the properties and the dead weight used in the design anal-
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$ND = $ not determined.
ysis. It also gives a floor surface with ½-in. camber which may or may not suit the architect. A level floor can be obtained by letting the topping thickness vary from 2 in. at center of span to 2½ in. at ends of span.

Under Dimensional Inspection, Item 2, the PCI manual "Inspection of Prestressed Concrete" (see Sec. 8-5) says "dimensional and camber tolerances should be established by the architect or engineer." This is certainly proper procedure, but the architect should ascertain from local fabricators what tolerances, especially with respect to camber, can be held without undue cost. With this established, the design drawings can indicate whether the topping is to have a constant or variable thickness.

One type of composite floor is made up of a ledger beam (Fig. 6-12) supporting double T's on its ledges, with the entire system joined by projecting reinforcing steel and a poured-in-place concrete slab.

Composite floors or roofs are also made up of double T's, a precast pretensioned soffit beam, and poured-in-place concrete, as illustrated in Figs. 7-6 and 7-7. A precast soffit beam is usually a shallow rectangular member pretensioned with compressive stress across its entire cross section. It is set in place in the structure and supported at short intervals by temporary shores so that dead-load bending stresses are eliminated until the job-site concrete has been poured and cured. Double T's with cut-back flanges are set in place, side forms are added, and the space over the soffit beam is filled with concrete. When this has cured, the temporary shores are removed and the composite beam carries the total load. Figure 7-6 shows details which would be used for a roof. For a floor the poured-in-place concrete would be continued to make a topping slab, as shown in Fig. 7-7.

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**Fig. 7-5.** Tensile tests on three types of welded connections using intermediate-grade reinforcing bars. These data are part of a report by Arthur R. Anderson presented before the Structural Division Session on Composite Design in Building Construction at an ASCE Convention in Washington, D.C., and later reprinted in the September, 1960, issue of *Progressive Architecture*, pp. 172–179.

All bars are intermediate-grade billet steel, ASTM A15, rolled to ASTM A305.

All welding electric arc with low-hydrogen electrodes AWS class E7015 or E7016. On multiple passes, surface is thoroughly cleaned prior to welding next pass.

On Type I joints, cross-section area of structural angle bar should be at least 1.5 times the reinforcing-bar cross section.
Fig. 7-6. Composite structure of precast pretensioned soffit beam, double T's, and poured-in-place concrete. See text for complete details.

The bottoms of the double T's are shown coped at the ends so they are flush bottom with the soffit beam. This adds to their cost, and if it is not necessary for clearance or appearance, they can be left straight so that the bottom of the T is level with the top of the soffit beam. When the span and/or load of the soffit beam is large, this approach is desirable in any case to give a deeper composite beam.

7-2 CONNECTIONS BETWEEN PRECAST MEMBERS\textsuperscript{11–14}

The technique of designing, fabricating, and erecting structures of precast prestressed concrete has changed considerably since the floor slabs for the Lumberville Bridge—a five-span suspended footbridge over the Delaware River near New Hope, Pennsylvania—were precast in 1947. Each development broadened the application of precast prestressed members a certain amount, which would be limited, in its turn, by some new factor that required analysis, test programs, etc., before it could be passed safely. Several years ago it became increasingly apparent that a standard approach to the design and
detail of connections between precast members was a necessity, and two committees were formed to study the subject.

In March, 1963, ACI-ASCE Committee 512's Recommendations for Design of Joints and Connections in Precast Structural Concrete were approved by the American Concrete Institute at its convention in Atlanta, Georgia. These Recommendations, which are reprinted in Chap. 8, deal with methods by which joints and connections may be designed, and should be studied by anyone who intends to design a structure composed of precast concrete members.

At the time of this writing the PCI Committee on Connection Details for Precast-Prestressed Concrete Buildings has completed a tentative report which will become available after approval by the Prestressed Concrete Institute. This report includes sketches illustrating approximately fifty types of connections, each accompanied by comments that will help to produce a safe and economical design for that type of connection.

Since connections are of such importance in the design of a prestressed concrete building, an assortment of details covering the most common situations is presented in Figs. 7-8 to 7-21. Some are taken from the literature of prestressed concrete fabricators, some are simi-

![Fig. 7-7. Soffit beam and double-T floor system ready for poured-in-place concrete.](image)
The bottoms of the double T's are shown coped at the ends so they are flush bottom with the soffit beam. This adds to their cost, and if it is not necessary for clearance or appearance, they can be left straight so that the bottom of the T is level with the top of the soffit beam. When the span and/or load of the soffit beam is large, this approach is desirable in any case to give a deeper composite beam.

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Since connections are of such importance in the design of a prestressed concrete building, an assortment of details covering the most common situations is presented in Figs. 7-8 to 7-21. Some are taken from the literature of prestressed concrete fabricators, some are simi-
lar to those developed by the PCI committee, and some are based on
the author's experience. Details of wall-panel connections will be
found in Figs. 6-24, 6-26, and 6-27. It would be impossible to illus-
trate all the types of joints that have been used, and innovations in
the arrangement of members will necessitate the development of new
types. Lack of precedent should not hinder a designer in the de-
velopment of a new type of joint as long as it adheres to the criteria set
forth in the Recommendations of ACI-ASCE Committee 512. (See
Chap. 10 for details of existing structures.)

In Fig. 7-8 main vertical reinforcing bars in the column project a
short distance through the base plate and are welded to it both above
and below before concrete for the column is poured. Threaded
anchor bolts project from the footing at each corner of the base plate.
A leveling nut is placed on each anchor bolt and the column is
erected. After the column is plumbed by adjusting the leveling nuts,
anchor nuts are turned down tight against the top of the plate, and the space between bottom of plate and top of footing is dry-packed. With a bearing plate of adequate thickness this detail will transfer some moment from column to footing but is not recommended where moments are large. A variation of this detail can be used when the column forms are continuous so that there is no room for a bearing plate larger than the column. When the column is cast, the concrete is held back 6 to 12 in. from the end of the vertical bars. After removal from the forms, the base plate is welded to the bars, and the space between precast concrete and base plate is dry-packed.

In Fig. 7-9 a wet mix of nonshrinking grout is placed in grout holes; the column is set in position, plumbed with leveling nuts against temporary angles; and space between column and footing is dry-packed. When grout has cured, temporary angles and bolts are removed. This detail will develop a large moment between column and footing if the vertical bars can be grouted into a sufficiently deep hole.

In Fig. 7-10 the weld between steel plates and vertical bars should be sufficient to transfer the entire shear from the beams. Angles should be thick enough to withstand bending induced in horizontal leg by vertical load. Thinner angles can be used by turning them with vertical leg down and putting in stiffeners to eliminate bending. If fire-resistant construction is required, cast beam with bottom coped out or with recess for angle so that bottom of angle is above bottom of beam. Apply concrete cover to underside of angle for fire protection.

The foregoing and other beam to column details can also be used where a beam connects to one side of a column only. Of course, the eccentricity created must be considered.

![Fig. 7-10. Beam to column detail. See text.](image-url)
When prestressed beams, T's, or other members are rigidly attached to columns or girders as in Fig. 7-11, the effect of future creep, shrinkage, and temperature changes should be considered. Any shortening tendency due to these factors will create a tensile stress in the member. The problem can be approached in several ways. The rate of both creep and shrinkage is rapid when the member is new but drops off with time, as shown in Fig. 7-12. Effects can be minimized by allowing as much time to elapse as possible before
attaching both ends of the member to the supporting structure. It can be erected with one end rigidly attached and the welding or grouting of the other end delayed for a time. In any detail, the bars which are attached to the steel bearing plates or angles and their bond to the concrete should be designed to resist tensile forces. Of course, if one end of a member can be permitted to move with relation to the supporting structure, the problem is eliminated. In fact this problem seldom develops in single-bay structures because the supporting walls or columns bend the small distance necessary to relieve

![Diagram](image)

**Fig. 7-14.** Connection of simple-span beam to column. Structural inserts should be welded to reinforcing bars or have sufficient bearing surface in concrete to transfer shear.
the tension. It is in multibay structures that steps must be taken to minimize the effects of shortening. (See Sec. 301.3.5 of ACI-ASCE Committee 512 Recommendations in Chap. 8.)

What must be considered when one precast member rests on and transfers its load to another precast member? Bearing pressure. Rotation of one member with respect to the other. Lateral motion of one member with respect to the other.
Section 301.3.3 of the ACI-ASCE Recommendations in Chap. 8 gives a formula for allowable bearing stress. This does not apply where the concrete of one section rests upon the concrete of the other. It would apply where a steel plate cast into one member rests on a steel plate cast into the other or where an elastic pad is used between the two members. It would also apply to a dry-packed or grouted area.

The connection should deliberately be designed to permit rotation or else to resist it. Elastomer pads, Neoprene, etc., and for light loads felt or asbestos sheets can be proportioned to permit some rotation without the concentration of pressure at the edges which would occur between inflexible materials. If rotation is to be prevented, the joint should be considered rigid and a connection made that will carry the moment developed in such a joint.

Lateral motion should be anticipated and provided for if possible.

Fig. 7-18. Detail of cantilever double T on bearing wall as shown by Nebraska Prestressed Concrete Co. A similar detail can be used when roof or floor sections end at the wall.
Lateral forces built up in a long structure which has no provision for shortening and/or lengthening of members usually become a problem. Suppliers of elastic pads such as Neoprene have literature which includes formulas for determining the details of the pad for specific loads and motions. The allowable working stress in an elastic pad seldom exceeds 1,000 psi and is frequently in the vicinity of 800 psi or less if any appreciable motion must be accommodated along with the transfer of bearing pressure.

7-3 CONTINUOUS DESIGN\textsuperscript{6-8}

Details already discussed under composite structures (Sec. 7-1) and connections (Sec. 7-2) have illustrated the ease with which precast prestressed members can be erected and joined into a rigid frame.
Fig. 7-21. Some of the many methods of supporting a hung ceiling from precast prestressed members. (Courtesy Nebraska Prestressed Concrete Co.)

With the exception of the detail in Fig. 6-18, which is post-tensioned, these joints are all formed of unstressed reinforcing bars connected by field welds or poured-in-place concrete. Portland Cement Association has conducted an extensive series of tests on structures made continuous with unprestressed reinforcing at the joints which prove that "adequate continuity and negative moment are obtained."
For some structures the designer may decide that a rigid frame created by post-tensioning the joints between members is most economical. An example is the garage illustrated in Fig. 7-22. 10 Five-story-high precast columns support 75-ft-long precast single-T sections. A poured-in-place slab spans between the T's which are 20 ft on centers. Cables from T's were coupled through columns with rods and post-tensioned after slabs and grouted joints between T's and columns had cured. Seismic forces were resisted in one direction by rigid frames of T's and columns and in the other direction by shear wall poured in place between interior row of columns.

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   Bull. D35—Horizontal Shear Connections
   Bull. D43—Further Tests of Continuous Girders
   Bull. D45—Shear Tests of Continuous Girders
   Bull. D46—Creep and Shrinkage Studies
   Bull. D51—Test of Half-scale Highway Bridge Continuous over Two Spans


8

CODES AND SPECIFICATIONS

8-1 DEVELOPMENT

Expansion in the application of a new building material is largely influenced by the existence of precedents and of codes and specifications. In the United States prestressed concrete was fortunate on both counts. Hundreds of structures were already in service in Europe before the first ones were considered here, and although the details developed varied from the European, the basic principles were the same.

Realizing that some regulation was needed long before reports from official code-writing bodies would be available, the Bureau of Public Roads prepared a set of tentative rules and submitted them to men in the prestressed concrete field both here and abroad for criticism. After making revisions based on these criticisms, in 1954 BPR issued Criteria for Prestressed Concrete Bridges with the following comment in the preface:

The Criteria for prestressed concrete bridges presented in this pamphlet have been developed in the hope that they may be useful until such time as more complete specifications, covering the subject in far greater detail, may be presented to the civil engineering profession by American specification and code writing bodies.
In November, 1954, Prestressed Concrete Institute issued Specifications for Pretensioned Prestressed Concrete (Tentative). This was revised in 1957 but still marked "tentative." PCI's Specifications for Post-tensioned Prestressed Concrete (Tentative) were published in February, 1958.

A Tentative PCI Standard Building Code for Prestressed Concrete was published in November, 1959. An edition, revised on the basis of suggestions from many persons closely associated with prestressed concrete, was published in 1961 with a statement in the preface that it was "intended to serve as an interim code until the new edition of the ACI Building Code Requirements is published."

The value of PCI's contributions to the development of adequate criteria cannot be overemphasized. Its tentative and interim specifications served as a temporary basis for production and furnished a guide for the official code-writing bodies to follow. In fact the prestressed concrete portion of the ACI Code (ACI 318) approved in March, 1963, is to a large degree a copy of the interim PCI Code published in 1961.

8-2 CODES IN USE

Tentative Recommendations for Prestressed Concrete is the most comprehensive instrument setting forth criteria for designing prestressed concrete that is available. Prepared by ACI-ASCE Committee 323 (which included a number of prestressed concrete experts from all fields), it was published in 1958. Every architect and engineer who plans to work with prestressed concrete structures should become thoroughly familiar with this code, which is reproduced in its entirety in the Appendix.

ACI 318, Building Code Requirements for Reinforced Concrete, as revised in 1963, includes full coverage of prestressed concrete and is an excellent guide for the design of prestressed concrete structures. There may be some minor discrepancies between it, when finally published, and the Tentative Recommendations printed in the Appendix. If so, the new ACI 318 would take precedence.

Prestressed concrete was accepted by the International Conference of Building Officials in 1960, and their requirements for it are covered in the 1961 edition of Uniform Building Code, pages 250 to 254 and 274 to 279.
8-3 JOINTS AND CONNECTIONS

Recommendations for Design of Joints and Connections in Precast Structural Concrete, prepared by ACI-ASCE Committee 512, was approved by the American Concrete Institute in 1963. It deals with the various factors that must be considered in assembling precast members and with criteria for providing adequate structural strength. Since the contents of this report are so closely associated with the function of this book, the entire report is reproduced herewith:

PROPOSED RECOMMENDATIONS FOR DESIGN OF JOINTS AND CONNECTIONS IN PRECAST STRUCTURAL CONCRETE
by ACI-ASCE Committee 512

CHAPTER 1. INTRODUCTION

101 Objective

The use of joints and connections for the transmission of shears, axial loads, moments and torsions from member to member and from member to sub-structure is inherent in precast concrete construction. Because joints and connections directly affect the integrity of the structure in which the precast concrete members are used, their design and fabrication must be adequate for the function intended. The object of this report is to suggest methods by which joints and connections for use in precast concrete construction may be designed.

102 Scope

The intent of these recommendations is to help provide that all joints and connections perform their function at all stages of loading without overstress and with proper factors of safety against failure due to overload.

These recommendations are intended to apply to joints and connections between precast members and precast members; between precast mem-

*As we go to press, this report is being circulated to committee members for final review. There may be a few differences between the report printed here and the one finally published in the Journal of the American Concrete Institute, but they should be minor since all suggested revisions have already been incorporated.
bers and cast-in-place concrete members; and between precast members and structural steel members.

The connections considered in this report are those connecting columns to footings, column to column, beams to columns, beams to girders, and wall, floor or roof slabs to beams. Connections outside the scope of this report are those used to fasten appurtenances such as piping or other mechanical equipment to concrete.

Joints and connections may be made by welding steel reinforcement or structural steel inserts; by bolting; by use of pins; by transfer of tensile stress or compressive stresses by bond or anchorage; by use of clips and other devices which prevent separation of precast members from independently braced supporting members; by use of key-type devices; by use of bonding mediums which affect the adherence of one member to another; by use of friction between members induced by gravity forces; by prestressing; by combinations of the preceding methods; or by any other method which accomplishes the intent, making use of recognized means of design, fabrication, and erection, or methods determined by testing.

103 General Considerations

It is recommended that joints and connections occur at logical locations in the structure, and when practical, at points which may be most readily analyzed and easily reinforced. Precautions should be made to avoid connection and joint details which would result in stress concentrations and the resulting spalling or splitting of members at contact surfaces. Liberal chamfers, steel edged corners, adequate reinforcement, and cushioning materials are a few of the means by which such stress concentrations may be avoided or provided for.

The strength of a partially completed or completed structure should be governed by the strength of the structural members rather than by the strength of the connections; the connection shall not be the weak link in the structure.

CHAPTER 2. DESIGN CONSIDERATIONS

201 Loading Conditions

Loading conditions to be considered in the design of joints and connections are service loads including wind and earthquake forces, volume changes due to shrinkage, creep, and temperature change, erection loads, and loading encountered in stripping from forms, shoring and removal of shores, storage, and transportation of members. Proper attention should be given to loads and the resulting stresses peculiar to the sequence of erec-
tion. Typical examples of construction in which the sequence and manner of erection affect the loading and stresses in the member are possible eccentric loading due to the erection of members on one side only of a member, installation of composite concrete toppings on shored or unshored slabs or beams, and continuity moment connections over supports. All significant combinations of loading should be considered, and the joints and connections should be designed for loadings consistent with these possible combinations of loading. For loadings other than those peculiar to precast concrete construction (decentering, handling, storage, and erection loads), loadings and load distributions as outlined in ACI 318 should be the minimum considered.

If it is not practical to provide for all possible temporary loading conditions which could occur during erection special erection procedures may be warranted. If so, complete erection instructions should be included in the plans and specifications which become part of the erection contract documents. Loading sequences, connection sequences and if necessary shoring or guying schedules should be clearly outlined. The disposition and strength of shoring should be stated and approved prior to construction.

202 Load Factors

202.1 Design Stress

Design stresses, except as noted hereafter, should not exceed those provided in ACI (318) "Building Code Requirements for Reinforced Concrete," ACI-ASCE "Tentative Recommendations for Prestressed Concrete," AISC "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings" or AWS "Standard Code for Arc and Gas Welding in Building Construction," whichever is applicable. Prestressed joints and connections should be treated as "segmental elements" in making use of design stress recommendations in the ACI-ASCE Committee 423 (323) report. Also, where applicable, the "Tentative Recommendations for Design of Composite Beams and Girders for Building" should be considered. *

202.2 Ultimate Load Factors

The ultimate strength capacity of joints and connections should be at least 10 per cent in excess of that required of the members connected. This recommendation may be satisfied by proportioning the joint or connection to provide strength in the connection or joint 1.1 times the ultimate strength capacity required by ACI 318-56, Section A-604.

202.3 Tests

If any reasonable doubt exists with respect to the determination of the load-carrying capacity of a connection this value should be established by test. Such tests should be designed by a qualified engineer and performed under his supervision. All rotations, movements, and forces, including those anticipated as a consequence of creep, shrinkage and temperature should be considered in the design of the tests.

Properly devised and performed tests may serve in lieu of the design procedures of these recommendations provided that the behavior of the test connection is satisfactory at design load and that all sources of stress (shear, axial load, torsion and moment) to which the connection is to be subjected be combined in performing the test. Furthermore, the ultimate load-carrying capacity of the connection in shear, moment, axial load and torsion should exceed the maximum anticipated for each by a factor of 1.1 times the appropriate “U” for the structure as determined from ACI 318-56, Section A-604.

CHAPTER 3. DESIGN

301 Transfer of Shear

301.1 General

The transfer of shear may be accomplished by means of reinforcing steel extended as dowels coupled with cast-in-place concrete placed between roughened concrete interfaces, mechanical devices such as embedded plates or shapes, brackets, prestressing force applied across the connecting surfaces, or any other ways which meet all accepted unit stress requirements for the materials involved and meet the ultimate strength requirements of Section 202.2 or which have been thoroughly tested and meet the requirements of Section 202.3. The entire shear should be considered as transferred through one type of device mentioned above, even though a combination of devices may be available at the joints or supports being considered. The device should be designed to resist the maximum shear in the section at the connecting surfaces.

301.2 Reinforcing Dowels

The extension of reinforcing bars in a flexural member or the placement of dowels anchored in each connecting member either by mechanical anchorage devices or with minimum embedment required to develop the full yield strength of the bar through bond may be used to transfer shear.

The allowable shear, based on extended bars or dowels, should not exceed:
\[ V = \sqrt{(A_s f_s \cos \theta)^2 + (1.5D^2 f_c^2 \sin \theta)^2} \]  

where \( V \) = total vertical shear in connections  
\( A_s \) = total cross-sectional area of bars or dowels  
\( f_s \) = allowable stress in bars or dowels  
\( D \) = sum of the diameter of bars or dowels  
\( f_c \) = concrete design strength  
\( \theta \) = included angle between direction of shear force and extended bar or dowel.

The second term in expression (1) should not be considered if the concrete cover on the dowel is less than three inches. Furthermore, cast-in-place concrete or grout or other positive means should be employed to fill gaps between precast members and the element to which they are connected with dowels.

301.3 Brackets

Any concrete ledge on which a precast member is to rest in a structure should be considered as a bracket. Thus a bracket may be the protrusion cast on to the side of a column or wall to serve as a beam seat, the top of a column, a ledge on a column or beam or a flange of a precast or cast-in-place beam.

For purposes of design the reaction should be considered as applied at a distance of one third of the bearing length of the supported member from the leading edge of the bracket or bearing pad unless other positive means are employed to apply the load to the bracket at an exact location (i.e., bearing bars on bracket plates or on member bearing plates).

From the standpoint of the structural behavior of the precast members a bracket serves as a shear transfer device, but the bracket itself must be designed for flexure, shear, bearing and the splitting forces accompanying bearing.

301.3.1 Flexure

The flexural design of a bracket should follow the procedures of flexural design for ordinary reinforced concrete beams (ACI 318) with a maximum reinforcement index "q" equal to .10 where \( q = (f_p/f_c)(A_s/\beta d) \). However, there is frequently insufficient room to develop the tensile reinforcement through bond. Adequate anchorage provisions should be made. Additional reinforcement to resist horizontal forces which may develop from volume changes due to shrinkage, creep and temperature change should be considered in the design. The horizontal friction force may be estimated or taken at 0.5 the vertical force.

301.3.2 Diagonal Tension

Diagonal tension reinforcement should be computed in accordance
with the provisions of ACI 318 with a minimum stirrup requirement equal to .005 bd. This reinforcement should be placed perpendicular to the direction of the reaction. The reinforcement should be distributed in pairs as close to the side faces of the bracket as cover regulations permit. Furthermore, they should be distributed equally over the depth of the bracket with at least three pairs employed. These bars should either be bent in a "U" shape or welded to cross pieces at the outer face of the bracket as close as cover regulations will permit. Anchorage at the other end (into supporting member) should be adequately provided for.

301.3.3 Bearing

It is desirable to ensure bearing be located away from the edge of a bracket. Bearing stresses under service loads should not exceed

$$f_b = 0.3f'_c 3^{A_e/A_b}$$

(2)

where $A_b =$ bearing area

$A_e =$ maximum area of portion of supporting member or supported member that is geometrically similar to and concentric with the bearing area.

To avoid local spalling the edge of the bracket should either be amored with steel anchored substantially, provided with liberal chamfer, or other means provided to relieve pressure against the edge.

Direct concrete to concrete bearing is not recommended between units unless the bearing is accomplished by cast-in-place concrete or grout.

301.3.4 Splitting

Reinforcement for confinement should be provided to resist the tensile stresses which accompany bearing if the bearing stresses exceed 0.1 $f'_c$. This reinforcement should be placed parallel to the bearing surface in both directions and should be considered for both the supporting and supported member. The reinforcement lateral to the direction of the longitudinal axis of the supported member may be eliminated in the bracket if the width of the bracket is greater than four times that of the supported member. A maximum concrete cover of one inch is recommended. The steel quantity required in each direction for both the supporting member and the supported member may be computed as follows:

$$A_s = \frac{V}{100,000}$$

(3)

where $V =$ total design reactions in pounds supported by the bracket. This reinforcement should be supplied in addition to that required for flexure or tensile stresses from any other source.
301.3.5 Other Tensile Stresses

Positive connections at the bearing of both ends of simply supported members by welding, bolts or any other means which prevent movements arising from creep, shrinkage or temperature change are not recommended. One end should be free to accommodate such movement. However, if such connections are made at both ends, reinforcement should be provided to resist the tensile forces which may develop in both the supporting and supported member from horizontal forces, and the bearing plates or other such devices should be designed to resist these forces.

301.4 Steel Brackets

Brackets made by fastening steel clip angles or other steel shapes to plates which are cast flush with the vertical faces of concrete members should be designed to resist all shears and moments developed by the connection. No allowance should be made for the bearing capacity of the edge of the vertical flush plate. This plate should be provided with positive anchorage into the concrete member of which it is a part of sufficient capacity to develop the vertical shear as well as any anticipated tensile forces normal to the surface of the plate consistent with the factors of safety outlined in Section 202.2. Provision should be made in the design of these brackets to accommodate movements arising from creep, temperature and shrinkage.

301.5 Embedded Structural Steel Shapes or Plates

Structural steel shapes or plates used for the purpose of transferring shear, should be designed for the following considerations:
1. Bearing stress in embedded portion
2. Shear in the steel between faces of connected concrete members
3. Flexural stress in the steel between faces of connected concrete members
4. Confinement of concrete embedding shapes or plates

301.6 Metal Studs

Metal studs welded to bracket plates or similar devices may be used for shear transfer and anchorages. The allowable shear may be determined by the following expression:

\[ V = 110 \ d^2 \ \sqrt{f_c} \]  

(5)

where \( h/d \) is equal to or greater than 4.2.

\[ V = 30 \ kd \ \sqrt{f_c} \]  

(6)

when \( h/d \) is less than 4.2,
where $V =$ shear transfer capacity in pounds
$h =$ height of stud in inches
$d =$ diameter of stud in inches

301.7 Prestressing Force

Shear may be transferred by means of a prestressing force applied across the contact area of elements being connected. Such application should conform to the provisions of the ACI 323 Report with adherence to the stress limitations suggested for segmental elements. The bearing area of one element on the other should provide positive means for proper distribution of the prestressing force. The prestressed connection should be investigated for shear, moment and axial loads just as for any other prestressed concrete structural member.

301.8 By Other Methods

Shear may be transferred by any other method provided that the method satisfies the principles of statics and the unit stress requirements of codes for the materials involved. It is strongly recommended that untried methods be tested to determine ultimate capacity in accordance with the provisions of Section 202.3.

302 Transfer of Moment

302.1 General

The transfer of moment through connections between precast members or a precast member and structural steel member or a precast member and cast-in-place member may be accomplished by: reinforcing steel extended as dowels; composite construction, embedded plates or structural steel shapes; or prestressing force applied to the joint and properly developed by the connecting members. The entire moment should be considered as transferred through one of the types of device mentioned above, even though a combination of devices may be available at the connection.

Reinforcing bars which are lapped or welded or steel plates or shapes which are welded to reinforcing bars, or to other steel plates or shapes, should be detailed so that there is a minimum eccentricity of force which is transferred through the connection. If small eccentricities cannot be avoided and more than one reinforcing bar or other element is to be connected, the disposition of the laps or welds should be symmetrical with respect to the center of gravity of the transferred force if such is practical.

The design of the connection and the adjacent members should consider all stresses due to eccentric loading conditions, plastic shortening, shrinkage, and temperature changes. Provisions of Article 301 should be followed to ensure proper capacity for shear transfer at the connection.
The members at the connection shall be reinforced properly to withstand unavoidable stress concentrations. Care should be taken that any portions of existing concrete that are to be in contact with cast-in-place connection concrete are clean before the concrete at the connection is placed.

302.2 Concrete Joint

Connections making use of cast-in-place structural concrete should be given special attention with respect to compaction, curing and the transfer of stress to the precast element through the concrete so placed.

302.3 Reinforcing Bars

302.3.1 Extension of Reinforcing Bars

The transfer of tension through the protruding reinforcing bars may be accomplished by sufficient lap, by welding which may or may not employ auxiliary hardware, or by other mechanical devices. These direct tension connections should be designed and made to develop the tensile force in accordance with the requirements of Section 204.

302.3.2 Additional Reinforcing Bars

If the precast members are designed with a cast-in-place structural composite concrete topping, continuity may be obtained by placing reinforcing bars in the cast-in-place concrete at the connection. Composite action should be assured by designing in accordance with the provisions of "Tentative Recommendations . . . 333 Report ACI, December 1960."

302.4 Structural Steel Shapes or Plates

Moment may be transferred at a connection by extending embedded structural steel shapes or plates from the members of either or both sides of the connection.

Special care should be taken to ensure that the anchorage is sufficient to develop the bending moment in accordance with Section 202.

In direct tension connections involving structural shapes, provision should be made to develop the full tensile force in accordance with Section 202.

To assure proper structural behavior for all load conditions provision should be made to prevent slippage at the anchorage or point of connection.

Sufficient cover and adequate reinforcement should be made to ensure against bursting of the concrete.

All welding required shall be done in accordance with the American Welding Society code to develop the design moments. The structural steel shall be designed in accordance with the AISC "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings."
302.5 Prestressing Force

The cast-in-place concrete used at the connection should be of a design strength equal to that of the connected members. When the cast-in-place concrete has reached the required strength, the prestressing force may be applied to the connection. The design should provide that no tensile stresses be permitted at any point across the connection for any stage of loading, if there is to be any subject to weather or corrosive environment, otherwise tensile stresses up to $3 \sqrt{f_e}$ may be permitted.

Ultimate strength design of connections shall be by the applicable provisions of ACI 318. Connections including post-tensioned systems subject to both flexural and prestress compression in the same region shall be designed by load factors increased 20 per cent. In connections involving pretensioning members with reinforcement index "q" below .25 no increase of load factor need be made. For "q" values above .25 the load factor shall be increased 20 per cent.

302.6 Stresses Due to Shrinkage, Creep, and Temperature Change

In a moment connection, the shortening of a precast member after erection due to shrinkage, creep and temperature change may cause high stresses in the members and the connection. The stresses produced by this shortening should be provided for in design.

A suggested method of reducing the potential shortening is to specify a minimum age of a precast member at the time of erection.

In a prestressed member, it may be necessary to extend reinforcing bars from the compression zone of the member into the cast-in-place concrete of the connection, to prevent separation of the member from the connection. Such bars should be properly anchored in both the member and the connection to develop the full strength of the bars.

303 Transfer of Torsion

303.1 General

The design of connections should always include an investigation of the possibility of torsional stresses due to unequal loading, wind and seismic forces, differential settlement or temporary erection loading conditions. In general members should be detailed so that torsion is held to a minimum at the connection.

303.2 Design Stresses

Torsional resistance at connections may be supplied by any of the methods outlined in Sections 301 and 302. The maximum stresses from combined transverse and torsional shear should not exceed the permissible values for the materials involved for transverse shear alone.
303.3 Tests
The torsional strength of many connections made on precast structural elements is often difficult to determine analytically. Tests incorporating all possible combinations of loading, bending, axial load, shear and torsion are strongly recommended. These tests should produce results consistent with the recommendations of Section 202.3.

304 Transfer of Axial Tension

304.1 General
Axial tension forces carried through a connection should not produce stress in any part of the connection which combined with bending stresses or torsional stresses exceed permissible stresses for the materials involved. Tensile stresses in concrete should never be relied upon to transfer axial tension. In designing connections which are to transfer tensile forces, special attention should be given to possible eccentricities.

304.2 Reinforcing Steel
The provisions recommended in Section 302.3 should be followed.

304.3 Embedded Structural Steel
Structural steel shapes may be used to transfer axial tension in connections provided that the total stress in each precast element is transferred by means of bond producing devices affixed to the structural steel shape so embedded. The steel preferably should be symmetrically placed within the concrete sections and detailed in accordance with the AISC Specifications. All welding shall be done in accordance with the Standard Code for Arc and Gas Welding in Building Construction of the American Welding Society. All welding shall be done by approved welding operators using the shielded arc method. Extreme care shall be taken in welding adjacent to concrete to prevent damage to the concrete. It may be necessary to provide for expansion from welding.

304.4 Prestressing Force
Axial tension may be transferred by means of a prestressing force applied to the contact area of elements being connected. Such applications should conform to the recommendations of Sections 301.7 and 302.4 covering prestressed concrete design with provisions made to assure proper transfer of the stresses between elements without local failure.

304.5 Concrete
The provisions of Section 302.2 should be followed when cast-in-place concrete is used at a connection.
305 Transfer of Axial Compression

305.1 General

Compression should be transferred across a connection without eccentricity whenever possible. The stresses resulting from axial compression combined with any other stresses on the section should be kept within the allowable stresses for the materials involved.

305.2 Unreinforced Concrete

The connection should be detailed to ensure compression over the entire section. The maximum allowable stress in concrete should not exceed 0.375f’c. The joint should be maintained in alignment by a pin or other positive means, the pin size determined by the amount of shear being transmitted across the joint.

305.3 Reinforced Concrete

Reinforcing bars arranged in a symmetrical pattern should extend completely through the joint lapping distances in accordance with ACI Chapter on columns. The joint should be completed with concrete giving special attention to compactions, curing and the transfer of stress from the dowels to the precast element through the concrete so placed. Due consideration should be given to the effects of shrinkage.

305.4 Embedded Structural Steel Shapes

Structural steel shapes may be used to transfer axial compression from precast elements provided that the total stress in each precast element is transferred by means of bond producing devices affixed to the structural steel shape so embedded. The steel preferably should be symmetrically placed within the concrete sections and detailed in accordance with the Standard Code for Arc and Gas Welding in Building Construction of the American Welding Society. All welding should be done by approved welding operators using the shielded arc method. Extreme care is required when welding to prevent damage to the concrete. Concreting of the joint should be completed as in Section 302.2.

305.5 Low Yield Strength Metals

Axial compression may be transferred from one precast element to another by means of low yield strength metals provided that the yield point of the metal or alloy selected is greater than the average stress on the section. Adequate provisions must be made to prevent the flow of such metal, restricting it three dimensionally. Bearing plates and a positive shear key should be provided.

Note: The ACI-ASCE Committee on Precast Structural Concrete was requested by the ASCE Structural Division, Executive Committee and ACI to write a Recommendation for Design of Joints and Connections in Precast Structural Concrete. The committee was handicapped by the scarcity of research and test data concerning
precast concrete connections. The recommendations preceding this section are as specific as committee members could make them considering this research scarcity. New research will be constantly evaluated by the committee for possible use updating this recommendation.

The sketches following are for the purpose of making the recommendations more clear. The appropriate section number is listed in the sketch title.
Beams

Columns

Fig. 301.5.

Fig. 301.6.
Grout or other suitable element to ensure distribution of prestressing force

Fig. 301.7.

Fig. 302.1.
8-4 SPECIFICATIONS FOR MATERIALS

Prestressed concrete uses the same cements, aggregates, and reinforcing bars that are used in reinforced concrete and the same specifications apply to these materials.

At the time of this writing there are two ASTM specifications dealing with prestressing tendons. ASTM-A416-59T covers 7-Wire Uncoated Stress-relieved Strand for Prestressed Concrete and ASTM-A421-59T covers Uncoated Stress-relieved Wire for Prestressed Concrete.

When a specification is required for a high-strength bar or large-diameter strand, it is suggested that the supplier of the item be asked to furnish one and that it be compared to the general requirements outlined in Sec. 304 of Tentative Recommendations for Prestressed Concrete.

8-5 INSPECTION

Precast prestressed concrete members can be made with smooth finish and other high-quality features at a reasonable cost—*if the requirements set are reasonable.* What are reasonable requirements? They are pretty well defined in the 35-page manual "Inspection of Prestressed Concrete," which is available from Prestressed Concrete Institute for a nominal charge. It covers the entire process of making a prestressed member, from checking the tension in the tendons, to what is permissible in the repair of a fault in the concrete to make it structurally safe and architecturally satisfactory.

In prestressed concrete the finished member itself is in some degree a check on proper fabrication. If the tendons are not in the right place at about the right tension, the camber will not be correct. If the concrete is not strong enough, the camber will be excessive. A visual inspection at any time will show the texture of the finished surfaces and whether or not any extensive patching has been done.

8-6 FIRE RESISTANCE

By their very nature all prestressed concrete members are fire-resistant because the prestressing tendons are protected from the heat of the fire by a layer of concrete of some thickness. The fire rating of
a member is a function of the thickness of this layer of concrete and some other factors.

To the best of the author's knowledge the only official instrument by any of the code-writing bodies which includes specific data on fire resistance is Tentative Recommendations for Prestressed Concrete, in which Sec. 215 says:

215—FIRE RESISTANCE

215.1—General

The fire resistance of both prestressed concrete and reinforced concrete is subject to the same general limitations. One is the rate of heat transmission through the concrete from the surface exposed to fire to the unexposed surface. The other is the reduction of steel strength at the temperatures induced in the steel during the test. Either limitation may govern.

215.2—Heat Transmission

Since the rate of heat transmission through prestressed concrete is similar to that of reinforced concrete of the same composition, the critical dimensions to control temperature rise at the unexposed surface will be the same in prestressed or reinforced concrete members.

215.3—Load-carrying Capacity

The ability of the structure to carry required loads during fire test depends largely on thickness of cover over prestressing steel. The following minimum thicknesses of concrete on prestressing steel and end anchorages are recommended for various fire ratings:

<table>
<thead>
<tr>
<th>Hour rating</th>
<th>1 hr</th>
<th>2 hr</th>
<th>3 hr</th>
<th>4 hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum concrete cover</td>
<td>1(\frac{1}{2}) in.</td>
<td>2(\frac{1}{2}) in.</td>
<td>3 in.</td>
<td>4 in.</td>
</tr>
</tbody>
</table>

Data now available are insufficient to make recommendations for such factors as shape of cross section, type and arrangement of prestressing steel. The cover thicknesses recommended are believed to be conservative.

A complete résumé of all fire tests on prestressed concrete in this country, including drawings, loading, and test results on 47 specific members, is presented on pages 14 to 43 of *Journal of the Prestressed Concrete Institute* for October, 1962. This article, An Interpretation of Results of Fire Tests of Prestressed Concrete Building Components by A. H. Gustaferro and C. C. Carlson, both of Portland Cement
Association, summarizes the information gained from the tests. It says

. . . the most important factors affecting the fire resistance of prestressed concrete flexural members are:

1. Thickness of concrete cover between the prestressing steel and the surface exposed to fire (cover).
2. Degree of end restraint.
3. Size of cross-section of the member.
4. Shape of the member.
5. Aggregate type.
6. Moisture content of the concrete.

The article points out that the fire rating of most prestressed members which have some degree of end restraint is determined by "heat transmission" rather than structural failure. Under the requirements of ASTM-E119 for standard fire tests a member is considered to have failed at the time it collapses or at the time the average temperature of its unexposed surface has risen 250°F. Figure 8-1, taken from the article, shows ratings as a function of heat transmission. Table 8-1, from the same article, shows recommended ratings based on type of cross section, total area, and concrete cover, and states that "on the basis of tests performed to date, the recommendations appear to be reasonably conservative."

![Graph showing the effect of slab thickness on fire resistance of concrete floors when failure is due to heat transfer.](image)

**Fig. 8-1.** Effect of slab thickness on fire resistance of concrete floors when failure is due to heat transfer.
Table 8-1. Cover Required for Various Fire Ratings in Prestressed Concrete Members

<table>
<thead>
<tr>
<th>Type of unit</th>
<th>Cross-sectional area* sq. in.</th>
<th>Recommended rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1 hr</td>
</tr>
<tr>
<td>Girders, beams, and joists</td>
<td>40-150</td>
<td>2&quot;</td>
</tr>
<tr>
<td></td>
<td>150-300</td>
<td>1 1/4&quot;</td>
</tr>
<tr>
<td></td>
<td>Over 300</td>
<td>1 1/8&quot;</td>
</tr>
<tr>
<td>Slabs, solid or cored</td>
<td>. . . . .</td>
<td>1&quot;</td>
</tr>
</tbody>
</table>

* In computing the cross-sectional area for joists, the area of the flange shall be added to the area of the stem, and the total width of the flange, as used, shall not exceed three times the average width of the stem.
† Adequate provisions against spalling shall be provided by means of wire mesh.

Two complete reports of fire tests by Underwriters Laboratories, Inc., are available from PCI for nominal charges. They are R-104-58 dealing with tests on a double-T section and R-113-62 dealing with tests on four single-T sections.
9

DESIGN CONSIDERATIONS

9-1 CAMBER AND DEFLECTION

As compared with reinforced concrete, prestressed concrete has certain characteristics which make it stiffer or less flexible. It is made of a much higher strength concrete which gives it a higher modulus of elasticity. All the concrete in the cross section contributes to the moment of inertia because it is crackless. In a reinforced concrete member only the compression concrete in the top and the tensile reinforcing steel in the bottom contribute to moment of inertia. The cracked concrete from the neutral axis down does not add to the stiffness of the member. On the other hand prestressed members usually have a large span-depth ratio which decreases their comparative stiffness.

Computations for camber and deflection of prestressed members are no more difficult than for steel structures of similar shape.\textsuperscript{1,2}\textsuperscript{*} The accuracy of these calculations is dependent upon the designer’s knowledge of the properties of the concrete going into the member.

As pointed out in Chap. 2, one purpose of prestressing concrete is

\textsuperscript{*} Superscript numbers indicate references listed in the Bibliography at the end of the chapter.
to create as much negative moment as possible in the member. What type of deflection is produced by a negative moment? It is an upward deflection or camber. As a result, practically all prestressed concrete beams have a camber whose magnitude is a function of moment of inertia of concrete section, strength of concrete, magnitude of prestress, eccentricity of prestress, and climatic conditions.

A characteristic of prestressed concrete members difficult for some designers to understand is that camber in such a member will increase with the passage of time. Concrete under constant compression undergoes a permanent inelastic shortening called creep. In a fully prestressed member not carrying its live load the bottom fibers are under a constant high compressive stress while the top fibers are under little or no stress. With the passage of time creep causes a shortening of the bottom surface of the member which, since there is no corresponding change in length of the top surface, results in additional camber.

As part of the design of any prestressed member, camber should be computed and camber growth estimated so that its influence on other details of the structure can be considered. Sections 203.2 and 203.3 of Tentative Recommendations for Prestressed Concrete (see Appendix) present data that are useful in the computation of camber and deflection. Members with a small span-depth ratio—girders, for instance—very seldom have too much camber. As the span-depth ratio increases, as with single and double T's, we find more instances where camber is large—sometimes so large that it cannot be economically accommodated by other details of the structure.

There are several approaches to the control of excessive camber. Partial prestress is the only one that does not increase cost, but a cost analysis will show that the additional cost of any of the suggested methods is not great. Any one or a combination of two or more may be the best solution to the problem for a specific structure.

1. Larger member. A relatively small increase in depth with a correspondingly small increase in volume of concrete can show an appreciable increase in moment of inertia and decrease in camber.

2. Larger prestressing force. In unsymmetrical members, such as single and double T's where the neutral axis is near the top of the member, increasing the prestressing force and decreasing its eccentricity will reduce the camber without creating excessive stresses.

3. Stronger concrete. The modulus of elasticity of concrete increases
as the strength increases. Thus curing the concrete to a higher strength before applying the prestressing force will decrease camber.

4. *Prompt loading.* Camber increases with time. This growth is greater for the member under its own dead weight than for the loaded member. Therefore camber growth is minimized by erecting the member and applying poured-in-place topping or other dead loads as soon as feasible after prestressing. This approach is sometimes impractical because of other scheduling requirements.

5. *Partial prestressing.* See discussion in Sec. 9-2.

Deflections are computed in the same manner as camber, for which references have already been given in this section. The shallow depth of some members with respect to the span engenders comparatively large deflections under live load. If the code being followed does not limit maximum deflections, some logical criteria should be established for the structure and the shallower members checked against it. One important item is proper roof drainage. If sufficient water collects on a flat long-span shallow roof to cause noticeable deflection, the deflection permits a greater depth of water which causes more deflection, etc.

**9-2 PARTIAL PRESTRESS**

The term “partially prestressed” refers to a prestressed concrete member which has flexural tension in the precompressed tensile zone under the design load condition. Partial prestress is often used to reduce excessive camber, and in some cases it permits a reduction in the amount of steel required. Requirements for ultimate strength of a partially prestressed member are the same as for a fully prestressed member.

Since there is no established specification for partially prestressed members, their design can be based on Tentative Recommendations for Prestressed Concrete with a few variations. In fact the allowable tensile stress of $6 \sqrt{f'_c}$ in Sec. 207.3.2.b.1.b is already a big step into partial prestress. It represents the highest tensile stress that can be used if the designer wishes to maintain a relatively crack-free structure. Section 207.3.3 says the ultimate tensile strength of concrete is approximately equal to $7.5 \sqrt{f'_c}$.

Just how will the use of partial prestress affect a typical prestressed concrete member? A tensile failure in the concrete will occur at a
stress of about $7.5\sqrt{5,000} = 7.5 \times 70 = 525$ psi. If this were a reinforced concrete member in which $E_a/E_c = 7$, the tensile stress in the steel bar when the crack occurred would be $525 \times 7 = 3,675$ psi. But reinforcing bars are used at stresses up to 20,000 psi. Therefore much higher tensile stresses can be used before cracks occur which are equal to those that are standard in reinforced concrete.

Consider a prestressed member designed by the usual criteria and then found to have an excessive amount of camber. How should it be redesigned as a partially prestressed member with adequate structural properties?

1. Establish an amount of camber that is permissible, compute the prestressing force that will create this camber, and select the prestressing tendons required for this force.

2. Check ultimate strength of member using tendons selected in step 1. If ultimate is not sufficient, add enough tendons to bring it up to specification. Tension all tendons to the same stress such that the total prestressing force will be that determined in step 1. This means that the tendons will not be tensioned to their full allowable stress, but they will have enough ultimate strength to give the member its specified ultimate strength. Another approach is to use enough tendons to give the member the prestressing force determined in step 1 and add high-yield-strength reinforcing bars to bring the ultimate strength of the member up to specification.

3. In designing the web reinforcing for shear, remember the comments in Sec. 210.2.2 of Tentative Recommendations for Prestressed Concrete. The formula given in that section is not always conservative for members with less than a normal prestressing force. The amount of web steel needed will be somewhere between the amount given by that formula and the larger amount required for an ordinary reinforced concrete member. Section 2610 of Building Code Requirements for Reinforced Concrete (ACI 318) as revised in 1963 includes criteria for computing the amount of web reinforcing required in members with less than full prestress.

4. Step 1 established the prestressing force and step 2 the amount of steel needed for ultimate. Some criterion should be established that will limit tensile stress under full design load. Since there is no formal specification covering this item at present, it is suggested that it be based on allowable stresses in reinforced concrete design. These computations can be carried out in two steps. First, apply load to the
prestressed member until the stress in the bottom fiber is reduced to zero. Second, apply the remaining live load and compute the resulting stresses in the member as if it were ordinary reinforced concrete. If the stress developed in the steel by this second loading does not exceed the 18,000 or 20,000 psi allowed in reinforcing bars, the design is satisfactory.

5. Maximum deflection should be checked. Here again the computation can be carried out in two steps. First, compute deflection in the prestressed member for the load which brings the bottom-fiber stress to zero. Second, considering the member as reinforced concrete, compute deflection caused by application of the remaining live load.

Partial prestress is not covered in any of the formal specifications familiar to the author. However, if reinforced concrete members are permitted in a specific structure, then partially prestressed members designed as suggested herein should also be acceptable.

9-3 TEMPERATURE EFFECTS

Prestressed concrete members, like those of other structural materials, undergo changes of length when their temperature changes. The properties of concrete vary from one aggregate and mix to another, but the coefficient of expansion of a prestressed member is roughly the same as that of a piece of structural steel, namely, 0.00000060 to 0.00000065 in. per in. per °F. In structures of any magnitude, therefore, provision must be made to accommodate this tendency to change in length. Either expansion joints must be provided or connections must be designed to transmit the loads created by temperature changes, and members must be designed for the additional stresses. Since the large area of the concrete cross sections would cause large loads if restrained, expansion joints are the common solution. Figures 9-1 and 9-2 illustrate expansion joints in two buildings under construction.

Any structural member will undergo a change in camber or deflection if the temperature of the top surface changes more or less than the temperature of the bottom surface. The magnitude of this vertical motion is larger and more noticeable in some prestressed members because of their long span. If the temperature of the top surface of a member rises more than the temperature of the bottom surface, there
will be an increase in camber. If the temperature of the top surface drops more than the temperature of the bottom surface, there will be a decrease in camber. If the probability of such temperature changes in a member exists, these motions should be allowed for when interior details are worked out. Consider a one-story office building with a double-T roof spanning 50 ft from wall to wall and partitions separating offices and hallway inside. The partitions should not be framed rigidly to the double T's; some provision should be made to permit the slight rise and fall of the roof members without damage to the partitions.

9-4 OTHER CONSIDERATIONS

Prestressed concrete is still comparatively new, and some contractors are not as familiar with it as with other structural materials. One important factor should be kept in mind in the erection of a structure composed of precast members. The structure probably is not stable until field welds have been made and poured-in-place concrete has been poured and cured. Therefore, as installation of

Fig. 9-1. Expansion joint created by two columns side by side and a ledger beam on each column. These joints are about 100 ft apart in building for American Cyanamid Co. in Pensacola, Fla. Prestressed members by Southern Prestressed Concrete, Inc., of Pensacola.
precast members proceeds, there should be a definite schedule involving use of temporary bracing, making of field welds, and pouring concrete so that the partially erected structure is at all times stable and never subject to excessive stresses or danger of collapse. It is almost impossible to avoid temporary eccentric loading during the erection process, but the condition should be recognized, the schedule planned to minimize such conditions, and adequate temporary bracing placed where needed.

In some buildings the possibility of differential settlement must be faced and provided for. Where this possibility exists, members and joints should be designed to survive the motions involved without losing their ability to carry their loads. For simple spans joints must be truly hinged so that rotation can take place without developing end moments in the beams, or else the joint, the beam, and the beam support must be made capable of taking the moment developed—which means it is no longer a true simple span. The best approach for designing rigid frames subject to possible differential settlement is to make sure that the overstressed sections will yield, without failure, and redistribute some of the excess load that has been placed on them.

9-5 DESIGNING AN ECONOMICAL STRUCTURE

Each new structure presents its own set of different problems, or there would be little need for architects and engineers, but a generally applicable procedure for developing an economical design is suggested herewith.
1. Establish basic dimensions of individual areas in building and overall shape. In doing this, keep in mind the span-load tables and other data presented in earlier chapters of this book. This general layout should be made on the basis of the characteristics of prestressed concrete and not of some other building material which might give a different layout. Consider the advantages of precast columns, composite sections, continuity through joints, etc. Do not forget that hung ceilings can be eliminated by spraying the underside of precast prestressed members with an acoustical coating that can be colored almost any desired shade. Is lightweight concrete desirable in this structure?

2. Select type of members to be used and determine approximate depth and other details from data offered in earlier chapters or other data on hand. Do not go beyond preliminary work at this point.

3. Contact local fabricators and ask for their literature covering the type of members involved. If you do not have a list of the local fabricators, Prestressed Concrete Institute can be of help, and so can the fabricators of prestressing tendons.

4. Using the data on locally available sections, proceed with a final design. It is a good idea to show the proposed design to one or two of the fabricators who will be bidding on it and get their comments on details and costs. The shallowest member shown for a given span-load combination may not necessarily be the least expensive. A slight increase in depth and volume of concrete may reduce the amount of strands to yield a net saving by using the deeper member. Information of this type can come only from the local fabricator. Even though the standard sections available vary from one fabricator to another, it is usually possible to prepare the drawings so that all can bid, thus assuring the customer of the best possible price.

5. On your first job or two it is wise, once your basic details are established, to have the design reviewed by someone with considerable experience in the field. There are now consulting engineers all over the country with adequate experience. In addition some of the tendon suppliers have experts on their staffs who will make such a review for architects and/or engineers who are just getting into the prestressed concrete field.

6. ACI Committee 315 is revising its "Manual of Standard Practice for Detailing Reinforced Concrete Structures" to coincide with the recently approved revised code of Committee 318 and to include
standards for prestressed concrete. A copy of the new manual should be obtained and used as a guide so that all prestressed concrete drawings will be as nearly standardized as possible.

BIBLIOGRAPHY


10

DETAILS OF ACTUAL STRUCTURES

10-1 MULTISTORY BUILDINGS

The Marine Plaza Parking Garage, illustrated in Figs. 10-3 to 10-7, is a good example of using the characteristics of prestressed concrete to provide an economical maintenance-free structure. The garage is 125 ft by 360 ft and has 11 levels offset vertically by one-half the story height. It is divided longitudinally by a center line of columns so that the girders (ledger beams) span 60 ft. The beams and double T's are precast prestressed and the columns precast reinforced concrete. Each column was precast full height in one piece. Ledger beams are 29 ft 7½ in. center to center, and three bays are made continuous with an expansion joint at each end of the three-bay group. Split columns and split ledger beams (Fig. 10-5) form the expansion joint. Tendons in the poured-in-place topping of the roof slab are post-tensioned in the direction perpendicular to the double-T members to create a crackless, leakproof roof.

Connections between precast columns and precast beams of the Marine Plaza Parking Garage are post-tensioned to give rigid frame

*Superscript numbers indicate references listed in the Bibliography at the end of the chapter.
Fig. 10-1. Twelve-story Diamond Head Apartments on Island of Oahu. Precast prestressed concrete piles driven into the coral support this glamorous yet economical structure. Prestressed piles by C. W. Winsteadt Construction Co. Prestressed I joists by Concrete Engineering Co. Ltd.

action. The erection sequence following the erection of the columns was as follows:

1. Erect girders.
2. Insert all rods through column and girder, place anchor plates, wedges, and nuts.
Fig. 10-2. Floor system for apartment in Fig. 10-1. Note that precast pretensioned joists are spaced to take standard plywood form sheets for placing of poured-in-place slab.

3. Fill all anchorage access pockets in girder with 5,000-psi concrete.
4. Erect slabs.
5. Pack joint and shear key between column and girder with grout.
6. Tension and grout short rods.

Fig. 10-3. Marine Plaza Parking Garage, Milwaukee, Wis. Precast and prestressed members by Concrete Research, Inc., Waukesha, Wis. See Figs. 10-4 to 10-7 and text.
Fig. 10-4. Marine Plaza Parking Garage. Cross section through typical ledger beam and floor slab. Precast floor slab members are 12-in.-deep double-T sections. (Figures 10-4 to 10-7 courtesy Ross H. Bryan, Consulting Engineer.)

Fig. 10-5. Marine Plaza Parking Garage. Cross section through ledger beams at expansion joint. See Fig. 10-6.
7. Tension and grout long rods.
8. Pour topping slab.

A unique feature of the office building shown in Figs. 10-8 and 10-9 is the floor framing. Precast prestressed I joists of the type shown in Fig. 10-2 were set in place on temporary shoring, and girder and slab were poured making one composite unit. Tensile stresses in girder are carried by standard un prestressed reinforcing bars. Originally designed for structural steel framing, this building was...
Fig. 10-8. Office building in downtown Honolulu. Contractors: Hawaiian Dredging and Construction Co. Ltd. See Fig. 10-9 for floor framing.

redesigned and built with prestressed concrete at the suggestion of the contractor. It was completed on schedule in 10 months at a cost $70,000 lower than the original bid.

Precast reinforced frames provide support for the double-T and poured-in-place slab floor system of the apartment building illustrated in Figs. 10-10 and 10-11. Columns are 23 ft 4 in. center to center, and overall length of precast frames including cantilever arms
Fig. 10-9. Connection details of I joists to beams for floor of building shown in Fig. 10-8.

Fig. 10-10. Continental Luxury Apartment, Memphis, Tenn., uses 140 precast frames and 815 precast pretensioned double-T floor slabs each 6 ft wide by 12 in. deep. Prestressed members by John A. Denic's Sons Co. of Memphis. See Fig. 10-11.
is 34 ft 1 in. Each frame (except the top one) has reinforcing bars projecting from the top of its columns. A cavity in the bottom of the next frame fits over these bars and is filled with grout to join them together. The cross member of the frame is a soffit beam similar to Figs. 7-6 and 7-7 except that it is reinforced instead of prestressed. It becomes composite with the concrete which is field-poured on it to the level of the slab over the T’s.

The 15-story Capp Towers Hotel (Figs. 10-12 and 10-13) in Minneapolis, Minnesota, is made up of 8-in. precast flat beams and prestressed flat slabs supported on structural steel columns. Beams are cast with structural collars that fit around and are welded to the columns. Ends of slabs frame to sides of beams with a shiplap detail that requires no additional height. Bottom surfaces of precast members are chemically treated to provide a roughened surface for direct application of plaster. This structure, with a floor area 86 by 222 ft, was erected at a rate faster than one floor per week.

10-2 SCHOOLS

The characteristics of pretensioned prestressed concrete make it especially adaptable to schools, and hundreds of them have been built throughout the country. Figure 10-14, taken from a brochure on School Framing Systems and Details distributed by a prestressed concrete fabricator, gives an idea of the diversity of choice available to the architect in framing a school structure. The sketches on the left illustrate the plan view, the sections at the top indicate the type of
roof member, and the numbers in the squares tell which page of the brochure has full details on that particular system. As an example sheet S-19 of the brochure is reproduced in Fig. 10-15.

Iselin, New Jersey, Junior High School (Fig. 10-17) utilizes prestressed members in several ways to cover its complex of buildings. Flat roof areas over classrooms are pretensioned double T's supported

Fig. 10-12. Capp Towers Hotel, Minneapolis, Minn. Precast and prestressed members by Prestressed Concrete, Inc., Saint Paul, Minn. See Fig. 10-13 and text.
Fig. 10-13. Erecting prestressed floor slab in Capp Towers Hotel. See Fig. 10-12 and text.

on pretensioned ledger beams (Fig. 10-18), all sprayed with acoustical plaster. Long-span I-section girders support double T’s over the auditorium, and precast arches support double T’s over the gymnasium (Figs. 10-19 and 10-20).

10-3 MOTELS

Precast prestressed concrete members and the modern motel seem to “go together.” Simple design details, fast construction, economical finish of exposed undersurfaces, and fire resistance are some of the characteristics that turn the motel builders toward prestressed concrete. Double T’s are excellent for floors and roofs (Figs. 10-21 and 10-22), while joists and insulating slabs provide an economical roof (Fig. 10-23).
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Fig. 10-14. Chart taken from brochure of Southern Prestressed Concrete, Inc., Pensacola, Fla. See Fig. 10-15 and text.
Fig. 10-15. Page from brochure of Southern Prestressed Concrete, Inc., illustrating single-T framing for schools. See Figs. 10-14, 10-16, and text.
Fig. 10-16. 56-ft 8-in. span single-T roof over assembly hall of Pine Meadow Elementary School in Pensacola, Fla. Underside is finished with sprayed-on acoustical plaster. Blockouts were provided in flanges for electrical boxes, and 2-in. topping was poured on roof members. Prestressed members by Southern Prestressed Concrete, Inc.

Fig. 10-17. Iselin, New Jersey, Junior High School. Prestressed members by Atlantic Prestressed Concrete Co., subsidiary of Warner Co., Trenton, N.J. See Figs. 10-18 to 10-20.
Fig. 10-18. Iselin school. Ledger beam supporting T beams—all sprayed with acoustical plaster. See text.

Fig. 10-19. Construction picture of Iselin school. Three hinged precast reinforced concrete arches support double-T roof over gymnasium. See Fig. 10-20 and text.
10-4 SHELLS

With the application of a little original thinking there is hardly any limit to the forms that can be achieved with prestressed concrete. The precast pretensioned shell-roof system illustrated in Figs. 10-24 to 10-27 was developed by Concrete Technology Corporation of Tacoma, Washington.

Two shell widths are available from this plant: 20-ft bays for spans to 85 ft, and 30-ft bays for spans to over 100 ft. A 30-ft bay consists of three curved shell elements and a V-shaped valley beam. During
Fig. 10-22. Interior of Gulf Winds Motel. Exposed undersurface of double T's provides pleasant ceiling pattern. See Fig. 10-21.

Fig. 10-23. Interior of Howard Johnson Motor Lodge, Pensacola, Fla. Ceiling composed of pretensioned joists and precast fibrous-type slabs. Prestressed members by Southern Prestressed Concrete, Inc.
Fig. 10-24. Details of precast pretensioned shell roof fabricated by Concrete Technology Corp., Tacoma, Wash. (a) Cross section of one bay. (b) Valley beam. (c) Shell element. (d) Valley beam connected to column by welding plate embedded in beam to angles embedded in column. (e) Shell element connected to valley beam or adjacent shell. Rebars are lap-welded and concrete cast in space between elements.

erection, the shell elements are stiffened by steel strongback trusses. Connections are made by welding the transverse reinforcing in adjacent elements and filling the space between with concrete. When the field-placed concrete has cured for 12 to 16 hr, the strongbacks can be removed for use in another bay. When completed, a 30-ft-
Fig. 10-25. Erecting shell roof over prestressing plant of Concrete Technology Corp. Note valley beam in lower left corner and shell element with steel strongback in lower right corner. Crane beams are I sections of precast pretensioned concrete.

Fig. 10-26. Shell element, supported by strongback, nearing final position in roof.
Fig. 10-27. Completed shell roof. Note, in lower left corner, precast pretensioned hollow cylindrical piles—one of the products of this plant.

wide shell has a moment of inertia of 900,000 in.\(^4\), which gives some idea of its strength and capacity. Flat steel plates cast in near the ends of the shell elements are welded to the steel supporting structure. Good light is provided inside the structure by painting undersides of the shells white to reflect the light admitted by the windows at each end of each bay.

**BIBLIOGRAPHY**

Appendix

TENTATIVE RECOMMENDATIONS FOR PRESTRESSED CONCRETE*
By ACI-ASCE Joint Committee 323

*Tentative Recommendations for Prestressed Concrete has been included by permission of the American Concrete Institute and the American Society of Civil Engineers. It first appeared in Journal of the American Concrete Institute, January, 1958, and Journal of the Structural Division, American Society of Civil Engineers. Paper 1519, January, 1958. This report is subject to revision whenever the studies of the committee responsible indicate that developments in prestressed concrete design and construction warrant change. Inquiries concerning revision should be made periodically to the American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Michigan, or to the American Society of Civil Engineers, New York, New York.
SYNOPSIS

A guide to design and construction of safe, serviceable, linear structural members prestressed with high strength steel. Emphasis is on flexural members—beams, girders, and slabs. Most of the recommendations are applicable to both buildings and bridges. Design chapter treats: loading; allowable stress; prestress loss; flexure and shear; bond and anchorage; composite construction; continuity; end blocks; fire resistance; and cover and spacing of prestressing steel. Concrete, grout, prestressing steel, anchorages, and splices are covered in the section on materials. Construction section includes: transportation, placing, and curing of concrete; forms, shoring, and falsework; placement of prestressing steel and application of the prestressing force; grouting; and handling and erection.

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Chapter 3—Materials
Section 301—Introduction; 302—Concrete; 303—Grout; 304—Prestressing steel; 305—Anchorages and splices.

Chapter 4—Construction
Section 401—Introduction; 402—Transporting, placing, and curing concrete; 403—Forms, shoring and falsework; 404—Placement of prestressing steel and application of prestressing force; 405—Grouting; 406—Handling and erection.

CHAPTER 1—INTRODUCTION

101—OBJECTIVE

The objective of this report is to recommend those practices in design and construction which will result in prestressed concrete structures that are comparable both in safety and in serviceability to constructions in other materials now commonly used.

This report constitutes a recommended practice, not a building code or specification. Since it was not written as a code, its use or interpretation as one will not serve the best interests of either the public or the engineering profession. Recommendations contained in the report are presented solely for the guidance and information of professional engineers. Safety and economy of structures in prestressed concrete will depend as much on the intelligence and integrity of engineers preparing the design and supervising or carrying out the construction as on the degree to which these recommendations are followed.
102—SCOPE

102.1—Linear prestressing

This report is confined in scope to linear structural members involving prestressing with high strength steel; circularly prestressed members such as tanks or pipes are not covered. These types of construction have been excluded for two reasons. They have been designed and constructed in this country for a great number of years and procedures have been developed on the basis of research and experience which have proved successful in practice. Design and construction of tanks and pipes in prestressed concrete are confined to a relatively small group of specialists and are not likely to be attempted by persons outside that group. For these reasons there seems to be no immediate need for recommendations regarding circularly prestressed structures.

102.2—Flexural members

For the most part, recommendations in this report relate to flexural members—beams, girders, and slabs. Other structural forms, such as columns, ties, arches, shells, trusses, pavements, etc., are treated only briefly or not at all. In some of these cases, such as columns or ties, the principles involved in design are essentially simple and no need was felt to include them in this report. In other cases, insufficient information was available either from research or experience to permit recommendations to be made at this time. This lack of information is due in some instances to the complexity of the type of structure involved and in others to the infrequency of its use in this country.

102.3—Buildings and bridges

These recommendations are intended to apply to both buildings and bridges. The form and nature of this report are such that almost all recommendations made apply without differentiation to both types of structures. Where this is not the case, separate recommendations are made for buildings and bridges.

103—ACCEPTANCE TESTS

It is recognized by the committee that unusual types of construction, design, or materials may be used in such a manner that these recommendations are not applicable or may not have been complied with. Such structures may be adequate for the purpose intended. In these cases it is recommended that tests be made to verify design.

104—NOTATION

104.1—General

Symbols are assembled into sections pertaining to groups of associated terms. The list comprises only the symbols in this report. No attempt is made to present a complete notation for design of prestressed concrete.
104.2—Dimensions and cross-sectional constants

- $A_b$ = bearing area of anchor plate of post-tensioning steel
- $A_e$ = maximum area of the portion of the anchorage surface that is geometrically similar to and concentric with the area of the bearing plate of post-tensioning steel
- $A_s$ = area of main prestressing tensile steel
- $A_o$ = area of conventional tensile steel
- $A_{ov}$ = steel area required to develop the ultimate compressive strength of the overhanging portions of the flange
- $A_{ovr}$ = steel area required to develop the ultimate compressive strength of the web of a flanged section
- $A_v$ = area of web reinforcement placed perpendicular to the axis of the member
- $b$ = width of flange of a flanged member or width of a rectangular member
- $b'$ = width of web of a flanged member
- $d$ = distance from extreme compressive fiber to centroid of the prestressing force
- $I$ = moment of inertia about the centroid of the cross section
- $j$ = ratio of distance between centroid of compression and centroid of tension to the depth
- $p = A_r/ bd$; ratio of prestressing steel
- $p'$ = ratio of conventional reinforcement
- $psi'$ = percentage index
- $s$ = longitudinal spacing of web reinforcement
- $t$ = average thickness of the flange of a flanged member
- $Q$ = statical moment of cross section area, above or below the level being investigated for shear, about the centroid
- $W$ = effect of wind load, or earthquake load, or traction forces
- $V_c$ = shear carried by concrete

104.3—Loads

- $D$ = effect of dead load
- $L$ = effect of design live load including impact, where applicable

104.4—Stresses and strains

- $E_s$ = flexural modulus of elasticity of concrete
- $E_r$ = modulus of elasticity of prestressing steel
- $f'c$ = compressive strength of concrete at 28 days
- $f'c_{ov}$ = compressive strength of concrete at time of initial prestress
- $f_{cp}$ = permissible compressive concrete stress on bearing area under anchor plate of post-tensioning steel
- $f'p$ = ultimate strength of prestressing steel
- $f_{sp}$ = effective steel stress after losses
- $f_{in}$ = initial stress in prestressing steel after seating of the anchorage
- $f_{su}$ = stress in prestressing steel at ultimate load
- $f_{yu}$ = nominal yield point stress of prestressing steel
- $f'c$ = flexural tensile strength of concrete; modulus of rupture
- $f_{y}'$ = yield point stress of conventional reinforcing steel
- $k_3$ = ratio of distance between extreme compression fiber and center of compression to depth to neutral axis
- $k_1k_2$ = ratio of average compressive concrete stress to cylinder strength, $f'c$
- $E_s/E_r$ = ratio of $E_s/E_r$
- $u_d$ = strain in concrete due to creep
- $u_s$ = strain in concrete due to elastic shortening
- $u_k$ = strain in concrete due to shrinkage
- $v$ = shearing stress
- $\delta_1$ = ratio of loss in steel stress due to relaxation of prestressing steel
- $\delta_2$ = ratio of loss in steel stress due to friction during prestressing
Chapter 2—Design

201—General Considerations

201.1—Purpose

The purpose of design is to define a structure that can be constructed economically, that will perform satisfactorily under service conditions, and will have an adequate ultimate load capacity.

201.2—Mode of failure

Ultimate strength should be governed preferably by elongation of the prestressing steel rather than by shear, bond, or concrete compression.

201.3—Design theory

The elastic theory should be used at design loads with internal stresses limited to recommended values. The ultimate strength theory also should be applied to insure that ultimate capacity provides the recommended load factors.

202—Special Considerations

202.1—Loading conditions

Consideration should be given to all critical loading conditions in design including those that occur during fabrication, handling, transportation, erection, and construction.

202.2—Deflections

Camber and deflection may be design limitations and should be investigated for both short and long time effects.

202.3—Length changes

Length changes of concrete due to prestress and other causes should be investigated for both short and long time effects.

202.4—Reversal of loading effects

Where reversal of moment or shear may occur it should be considered in the design.

202.5—Buckling

General buckling due to prestressing can occur only over the length between points of contact of the prestressing steel with the concrete.
General buckling of an entire member or local buckling of thin webs and flanges under external loads may occur in prestressed concrete as in members made of other materials and should be provided for in design.

203—ASSUMPTIONS

203.1—Basic assumptions

The following assumptions may be made for design purposes:

a. Strains vary linearly over the depth of the member throughout the entire load range.

b. Before cracking, stress is linearly proportional to strain.

c. After cracking, tension in the concrete is neglected.

203.2—Modulus of elasticity

When accurate values for modulus of elasticity are not available, the following values may be used as a guide:

a. Flexural modulus of elasticity of concrete $E_c$, in psi, may be assumed to be 1,800,000 plus 500 times the cylinder strength at the age considered. Actual values may vary as much as 25 percent from those given by the foregoing expression. This expression is not applicable to lightweight concrete, for which $E_c$ should be determined by test.

b. Modulus of elasticity of steel, in psi, may be assumed to be 29,000,000 for cold drawn wire, 27,000,000 for 7-wire strand, 25,000,000 for strand with more than 7 wires, and 27,000,000 for alloy steel bars.

203.3—Deflections

Deflection or camber under short time loading may be computed using values of $E_c$ obtained as described in Section 203.2.a.

Deflection associated with dead load, prestress, and live loads sustained for a long time may be computed on the assumption that the corresponding concrete strains are increased as a result of creep. The increase in strain may vary from 100 percent of the elastic strain in very humid atmosphere to 300 percent of the elastic strain in very dry atmosphere. These values may not pertain to concrete made with lightweight aggregates.

204—LOADING STAGES

204.1—Loading

Loading stages listed in the following sections should be investigated. No attempt is made to list all significant loading stages that may occur. Stages listed are those that normally affect the design.

204.2—Initial prestress

Prestressing forces are applied in prearranged sequence and sometimes in stages. If prestressing forces are not counteracted by the effect of the dead load of the member, or if the stressing operation is accompanied by temporary eccentricities, concrete stresses should be investigated.
204.3—Initial prestress plus dead load of member
For determination of concrete stresses at this stage, losses in prestress are those which occur during and immediately after transfer of prestress.

204.4—Transportation and erection
Support conditions for precast members during transportation and erection may differ from those during service loads. Handling stresses should be included together with prestress and dead load. Losses in initial prestress up to time of handling should be considered.

204.5—Design load
This stage includes stress due to effective prestress after losses, dead loads, and maximum specified live load.

204.6—Cracking load
Complete freedom from cracking may or may not be necessary at any particular loading stage. Type and function of the structure and type, frequency, and magnitude of live loads should be considered.

204.7—Temporary overload
This stage refers to any large live load in excess of design load, which is of short-time duration and expected to occur infrequently during life of structure. For such a load, stresses may exceed those recommended for design load but elastic recovery must be assured.

204.8—Ultimate load
Ultimate load is that load which applied statically in a single application causes failure. Such a large load would never intentionally be placed on the structure, but it is used as a measure of safety. In statically determinate structures, failure will occur at a single cross section. In statically indeterminate structures, the load which causes moment in one section to reach its ultimate value may not be sufficient to cause failure of the structure because of moment redistribution. Since it is not always possible to predict that full redistribution will take place in accordance with limit design, it is suggested for the time being that moments be determined by elastic analysis.

205—LOAD FACTORS
205.1—General
A load factor is a multiple of the design loads used to insure safety of the structure.

205.2—Cracking load factors
If cracking of concrete is undesirable, load factors for cracking load should be chosen to reflect the greatest load that can be expected during life of structure. Formation of a crack under temporary overload may not be objectionable. If reopening such a crack under subsequent design load is objectionable it may be avoided by proper choice of concrete stress permitted for cracking load.
205.3—Ultimate load factors

The ultimate load capacity should be computed since stresses are not linearly proportional to external forces and moments throughout the entire load range. For the present, it is recommended that moments, shears, and thrusts produced by external loads and prestressing forces be investigated by elastic analysis.

The load factors recommended are believed to be consistent with current viewpoints. It may be desirable to modify or expand the load factor formulas to fit special conditions that may occur in unusual structures, extremely long spans, or for unique loadings. Deviations from the recommended values should be substantiated by suitable investigations.

205.3.1—Buildings

For the present, to correlate prestressed concrete with reinforced concrete practice in current use, the committee recommends that ultimate load capacity be investigated to insure meeting the following requirements:

\[
\begin{align*}
1.2 D + 2.4 L \\
or 1.8 (D + L) \\
or 1.2 D + 2.4 L + 0.6 W \\
or 1.2 D + 0.6 L + 2.4 W \\
or 1.8 (D + L + \frac{3}{4} W) \\
or 1.8 (D + \frac{3}{4} L + W)
\end{align*}
\]

whichever is greater.

205.3.2—Highway bridges

The following load factors for highway bridges are recommended by a subgroup appointed by American Association of State Highway Officials, Committee on Bridges and Structures.*

\[
1.5 D + 2.5 L
\]

The committee is not prepared at this time to make recommendations for load factors involving the effect of lateral loads on bridges.

205.3.3—Railway bridges

Ultimate load factors for railway bridges are currently being studied by the American Railway Engineering Association. The committee is not prepared to recommend such factors at this time.

206—REPETITIVE LOADS

206.1—General

Ultimate strength of concrete or steel subjected to repetitive loading may be less than static strength because of the phenomenon of fatigue. Full importance of fatigue in prestressed concrete members has not yet been determined. Fatigue failure may occur in concrete, steel, anchorages, splices, or bond.

*These load factors are considered adequate for spans of moderate length, simply supported. For exceptionally long spans and for continuous members, special investigation to consider a possible increase in load factors is recommended.
206.2—Concrete

Fatigue strength of concrete in both tension and compression depends on magnitude of stress, range of stress variation, and number of loading cycles. Since high stresses and stress ranges are common, fatigue should be considered when repetition of loading cycles may occur.

Fatigue failure is unlikely if the allowable stresses of Section 207.3.2 are not exceeded and there is no reversal of stress. If a large number of overloads are anticipated a reduction in the safety factor may occur.

206.3—Prestressing steel

Fatigue strength of prestressing steel depends on magnitude and range of stress, and number of cycles of loading. Minimum stress is the effective prestress. Maximum stress and range of stress depend on magnitude of live loads or overloads that may be repeated. Range of stress under service loads will usually be small unless concrete is cracked. Cracking may occur if tension is permitted in concrete. Fatigue failure of steel should be considered in such cases, especially when a high percentage of ultimate strength is used for prestress.

Devices for splicing steel may contain strain concentrations that lower fatigue strength. Consideration should be given to fatigue whenever splices are used.

206.4—Anchorages

If steel is fully bonded, no difficulty should be expected in the anchorage or end bearing as the result of repetitive loads. With unbonded steel, fluctuations in stress due to repeated service loads or overloads are transmitted directly to anchorages and fatigue strength of the anchorage will require special consideration.

206.5—Bond

Failure of bond under repetitive loading is unlikely unless the member is cracked under design loads or a significant number of repetitions of overload. High bond stresses adjacent to cracks may be a source of progressive failure under repeated loads.

206.6—Shear and diagonal tension

Since inclined cracks may form under repetitive loading at appreciably smaller stresses than under static loading, web reinforcement should always be provided in members subjected to repetitive loading.

206.7—Design recommendations

Fatigue should not result in a reduction of strength if the following recommendations are observed. When the recommendations cannot be followed, fatigue strength of all elements comprising the prestressed member should be considered.

a. Flexural compressive concrete stress should not exceed \(0.4f'c\) under either design load or an overload that may be repeated many times.
b. Tension should not be permitted in concrete at the critical cross section under either design load or overloads that may be repeated a large number of times.

c. Reversal of stress should not occur under repeated loads.

d. Prestressing steel should be bonded.

e. Web reinforcement should be provided.

207—ALLOWABLE STEEL AND CONCRETE STRESSES

207.1—Prestressing steel

207.1.1—Temporary stresses

Under normal design loads stress in prestressing steel will almost always be less than stress at initial prestress. Stress at the anchorage immediately after seating has been effected should not exceed 0.70\(f'_t\) for material having stress-strain properties defined in Chapter 3. Overstressing for a short period of time to 0.80\(f'_t\) may be permitted provided the stress, after seating of anchorage occurs, does not exceed 0.70\(f'_t\).

207.1.2—Stress at design loads

Effective steel stress after losses described in Section 208 should not exceed:

\[0.60f'_t\text{ or } 0.80f'_{ew}\]

whichever is smaller.

207.2—Non-prestressed reinforcement

Non-prestressed reinforcement provided to resist tension in conformance with requirements of Section 207.3.1.b.2 may be assumed stressed to 20,000 psi.

207.3—Concrete

207.3.1—Temporary stresses

Concrete stress in psi before losses due to creep and shrinkage should not exceed the following:

a. Compression
   - For pretensioned members: \[0.60f'_{et}\]
   - For post-tensioned members: \[0.55f'_{et}\]

b. Tension
   1. For members without non-prestressed reinforcement:
      - Single element: \[3\sqrt{f'_{et}}\]
      - Segmental element: zero
   2. For members with non-prestressed reinforcement provided to resist the tensile force in the concrete, computed on the basis of an uncracked section:
      - Single element: \[6\sqrt{f'_{et}}\]
      - Segmental element: \[3\sqrt{f'_{et}}\]
207.3.2—Stresses at design loads

After full prestress losses, stresses in psi should not exceed the following:

a. Compression
   1. Single element
      a. Bridge members...........................................0.40f'c
      b. Building members.......................................0.45f'c
   2. Segmental elements
      a. Bridge members...........................................0.40f'c
      b. Building members.......................................0.45f'c

b. Flexural tension in the precompressed tensile zone
   1. Single element
      a. Bridge members...........................................zero
      b. Pretensioned building elements not exposed to weather or
         corrosive atmosphere...................................6√f'c
      c. Post-tensioned bonded elements not exposed to weather
         or corrosive atmosphere................................3√f'c
   2. Segmental elements
      a. Bridge members...........................................zero
      b. Building members.......................................zero

Allowable flexural tension of 6√f'c in Section 207.3.2.b.1.b may be exceeded provided it is shown by tests that the structure will behave properly under service conditions and meet any necessary requirement for cracking load or temporary overload.

207.3.3—Stress at cracking load

Flexural tensile strength (modulus of rupture) should preferably be determined by test. When test data are not available the ultimate flexural tensile stress in psi may be assumed as:

\[ f_t' = 7.5\sqrt{f'c} \]

For lightweight concrete, \( f_t' \) should be determined by tests.

207.3.4—Anchorage bearing stresses

The maximum allowable stress at post-tensioning anchorage in end blocks adequately reinforced in conformance with Section 214.4 may be assumed as:

\[ f_{p} = 0.6f'c\sqrt[3]{A_e/A_s} \]

where \( A_s \) = bearing area of the anchor plate.
\( A_e \) = maximum area of portion of the member that is geometrically similar to and concentric with the area of bearing plate.

The allowable value of \( f_{p} \) should not exceed \( f'c_t \).
208—LOSS OF PRESTRESS

208.1—Introduction

Initial prestress is that stress in steel which exists immediately after seating of anchorage. Stress diminishes with time and finally reaches a stable condition of effective prestress assumed to be permanent.

208.2—Sources of prestress loss

208.2.1—Friction loss in post-tensioned steel

If post-tensioned steel is draped, or irregularities exist in alignment of ducts, steel stress will be less within the member than at the jack because of friction between prestressing steel and duct. Magnitude of this friction should be estimated for design and verified during stressing operation.

Friction loss may be estimated from an analysis of forces exerted by prestressing steel on duct. One method for determination of friction loss at any point is given below.

\[ T_e = T_s e^{(KL + \mu \alpha)} \]

where \( T_e \) = steel stress at jacking end

\( T_s \) = steel stress at point x

\( e \) = base of Naperian logarithms

\( K \) = friction wobble coefficient per ft of prestressing steel

\( L \) = length of prestressing steel element from jacking end to point x in ft

\( \mu \) = friction curvature coefficient

\( \alpha \) = total angular change of prestressing steel element in radians from jack to point x

For small values of \( KL \) and \( \mu \alpha \) the following formula may be used:

\[ T_e = T_s (1 + KL + \mu \alpha) \]

The following values of \( K \) and \( \mu \) are typical and may be used as a guide. They may vary appreciably with duct material and method of construction. Values of \( K \) and \( \mu \) used in design should be indicated on the plans for guidance in selection of materials and methods that will produce results approaching the assumed values.

<table>
<thead>
<tr>
<th>Type of steel</th>
<th>Type of duct or sheathing</th>
<th>Usual range of observed values</th>
<th>Suggested design values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( K )</td>
<td>( \mu )</td>
</tr>
<tr>
<td>Wire cables</td>
<td>Bright metal sheathing</td>
<td>0.0005-0.0030</td>
<td>0.15-0.35</td>
</tr>
<tr>
<td></td>
<td>Galvanized metal sheathing</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Greased or asphalt-coated and wrapped</td>
<td>0.0030</td>
<td>0.25-0.35</td>
</tr>
<tr>
<td>High strength bars</td>
<td>Bright metal sheathing</td>
<td>0.0001-0.0005</td>
<td>0.08-0.30</td>
</tr>
<tr>
<td></td>
<td>Galvanized metal sheathing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Galvanized strand</td>
<td>Bright metal sheathing</td>
<td>0.0005-0.0020</td>
<td>0.15-0.30</td>
</tr>
<tr>
<td></td>
<td>Galvanized metal sheathing</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Workmanship in placing, supporting, tying, and fabricating prestressing elements and ducts influences the magnitude of wobble factor, $K$. The larger the sheath or duct in relation to the size of prestressing steel element, the smaller $K$ will be. With normal placing tolerances wobble effect may be neglected if sheath is 1 in. greater in diameter than prestressing steel element.

Effect of overestimating friction loss should be considered since excessive prestress may cause undesirable permanent stress conditions. Underestimating friction loss may result in an error in computing cracking load and deflection.

208.2.2—Elastic shortening of concrete

Loss of prestress caused by elastic shortening of the concrete occurs in prestressed concrete members. This loss equals $n(\Delta f_c)$. For pretensioned concrete, $\Delta f_c$ is the concrete stress at the center of gravity of the prestressing steel for which the losses are being computed. For post-tensioned concrete where the steel elements may not be tensioned simultaneously, $\Delta f_c$ is the average concrete stress along one prestressing element from end to end of the beam caused by subsequent post-tensioning of adjacent elements.

208.2.3—Shrinkage of concrete

Shrinkage depends on many variables. Unit shrinkage strain may vary from near 0 to 0.0005. A value between 0.0002 and 0.0003 is commonly used for calculation of prestress loss. Shrinkage loss may be greater in pretensioned members where the prestress is transferred to the concrete at an earlier age than is usual for post-tensioned members. Shrinkage of lightweight concrete may be greater than the values obtained with the above factors.

208.2.4—Creep of concrete

Creep is the time-dependent strain of concrete caused by stress. For pretensioned and post-tensioned bonded members, concrete stress is taken at center of gravity of prestressing steel under effect of prestress and permanent loads (normal conditions of unloaded structure).

In post-tensioned unbonded members, stress is the average concrete stress along the profile of center of gravity of prestressing steel under the effect of prestress and permanent loads. Additional strain due to creep may be assumed to vary from 100 percent of elastic strain for concrete in very humid atmosphere to 300 percent of elastic strain in very dry atmosphere.

Creep of some lightweight concretes may be greater than indicated above.

208.2.5—Relaxation of steel stress

Loss of stress due to relaxation of prestressing steel should be provided for in design in accordance with test data furnished by the steel manufacturer. Loss due to relaxation depends primarily on properties of the steel and initial prestress. This loss is generally assumed in the range of 2 to 8 percent of initial steel stress.
208.3—Alternate procedures for estimating prestress losses

Two methods are suggested for estimating prestress losses. Method 1 should be used when individual losses may be predicted with reasonable accuracy. Method 2 applies when specific loss data are lacking.

The ultimate strength is not significantly affected by the magnitude of steel stress loss. An error in choosing the loss is reflected in the cracking load and amount of camber.

208.3.1—Method 1

The total stress loss in prestressing steel:

$$\Delta f_s = (u_s + u_u + u_d) E_s + \delta_1 f_{si} + \delta_2 f_{si}$$

208.3.2—Method 2

Loss in steel stress not including friction loss may be assumed as follows:

- Pretensioning: 35,000 psi
- Post-tensioning: 25,000 psi

For camber calculations these values may be excessive.

208.4—Lightweight concrete

Losses due to concrete shrinkage, elastic shortening, and creep should be based on results of tests made with the lightweight aggregate to be used.

209—FLEXURE

209.1—Stresses due to dead, live, and impact loads

Prestressed concrete members may be assumed to function as uncracked members subjected to combined axial and bending forces provided stresses do not exceed those given in Section 207.

In calculations of section properties prior to grouting, areas of the open ducts should be deducted unless relatively small. The transformed area of bonded reinforcement may be included in pretensioned members and post-tensioned members after grouting.

For calculation of stress due to prestress in T-beams no definite recommendations are made at this time, but attention should be given to the possibility that the entire available flange width may be included in calculation of section properties.

209.2—Ultimate flexural strength

209.2.1—General method

(a) Rectangular sections—For rectangular sections or flanged sections in which the neutral axis lies within the flange, ultimate flexural strength may be expressed as:

$$M_* = A_d f_{*s} d \left(1 - \frac{k_s f_{*s}}{k_1 k_2 f'_{*s}}\right)$$

\(M_*\) denotes the moment capacity of the member.
where \( f_{ru} \) = average stress in prestressing reinforcement at ultimate load
\( d \) = depth to centroid of force
\( k_2 \) = ratio of distance between extreme compressive fiber and center of compression to the depth to neutral axis
\( k_3 \) = ratio of average compressive concrete stress to the cylinder strength, \( f' \)

The results of numerous tests have shown that the factor \( k_2/k_3k_4 \) may be taken equal to 0.6 for members and materials considered in this report. Determination of the value of \( f_{ru} \) requires knowledge of the stress-strain characteristics of the prestressing steel, effective prestress and crushing strain of the concrete. Assumptions must be made regarding the relation between steel and concrete strains. These assumptions will be different for bonded and unbonded construction.

The ultimate moment may be computed from Eq. (a) whenever sufficient information is available for the determination of \( f_{ru} \). The approximate method of Section 209.2.2 may be used if the required conditions are satisfied.

(b) *Flanged sections*—If a flange thickness is less than \( 1.4d/pf_{ru}/f' \), the neutral axis will usually fall outside the flange and the following approximate expression for ultimate moment should be used:

\[
M_u = \frac{A_{ru}f_{ru}}{d} \left( 1 - 0.5 \frac{A_{ru}f_{ru}}{b'df'c} \right) + 0.85f'(b - b')t(d - 0.5d) \quad \ldots \ldots \quad (b)
\]

where
\[ A_{ru} = A_s - A_{uf} \] = the steel area required to develop the ultimate compressive strength of the web of a flanged section
\[ A_{uf} = 0.85f'(b - b')t/f_{ru} \] = steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.
\[ t \] = average thickness of flange

The expressions for \( f_{ru} \), given in Section 209.2.2 may be used if the required conditions are satisfied.

209.2.2—*Approximate method*

The following approximate expressions for \( f_{ru} \) may be used in Eq. (a) and (b) of Section 209.2.1 provided the following conditions are satisfied:

1. The stress-strain properties of the prestressing steel are reasonably similar to those described in Section 304.
2. The effective prestress after losses is not less than \( 0.5f' \).

(a) *Bonded members*

\[
f_{ru} = f' \left( 1 - 0.5 \frac{pf'c}{f'c} \right)
\]
(b) Unbonded members—Ultimate flexural strength in unbonded members generally occurs at lower values of steel stress than in bonded members. Wide variations between stress levels reported by different investigators reflect the fact that several factors influence the stress developed by unbonded steel at ultimate moment. These factors include: magnitude of effective pre-stress, profile of the prestressing steel, shape of the bending moment diagram, length/depth ratio of the member, magnitude of the friction coefficient between the prestressing steel and duct, and amount of bonded non-prestressed supplementary steel.

Unless the proper value of $f_{su}$ is known from tests of members closely approximating proposed construction with respect to the several factors listed in the preceding paragraph, it is recommended that:

$$f_{su} = f_{s} + 15,000$$

209.2.3—Maximum steel percentage

To avoid approaching the condition of over-reinforced beams for which the ultimate flexural strength becomes dependent on the concrete strength, the ratio of prestressing steel preferably should be such that $p f_{su}/f'_{s}$ for rectangular sections, and $A_p f_{su}/b'df'_{s}$ for flanged sections are not more than 0.30.

If a steel ratio in excess of this amount is used, the ultimate flexural moment shall be taken as not greater than the following values when either the general or approximate method of calculation is used.

(a) Rectangular sections

$$M_u = 0.25 f'_{c} bd^3$$

(b) Flanged sections—If the flange thickness is less than $1.4 \frac{dp f_{su}}{f'_{s}}$ the neutral axis will usually fall outside the flange and the following formula is recommended.

$$M_u = 0.255 df'_{s} + 0.85 f'_{s} (b - b') (d - 0.54)$$

209.2.4—Non-prestressed reinforcement in conjunction with prestressing steel

209.2.4.1—Conventional reinforcement—Non-prestressed conventional reinforcement may be considered to contribute to the tensile force in the beam at ultimate moment an amount equal to its area times its yield point provided that

$$\frac{pf_{su}}{f'_{s}} + \frac{p'f'_{s}}{f'_{s}} \text{ does not exceed } 0.3$$

where $f'_{s}$ = yield point of conventional reinforcement

$p'$ = ratio of conventional reinforcement

209.2.4.2—High tensile strength reinforcement—If untensioned prestressing steel or other high tensile strength reinforcement is used in conjunction with prestressed reinforcement, the ultimate moment should be calculated by means of the general method of Section 209.2.1.
210—SHEAR

210.1—General

210.1.1—Ultimate strength

It is essential that shear failure should not occur before ultimate flexural strength required in Section 209.2 is developed. If this condition is satisfied, it is unnecessary to investigate shear or principal tensile stresses at design loads.

210.1.2—Inclined cracking

Formation of inclined cracks precedes failure in shear. They are caused by inclined principal tensile stresses that are the resultant of shearing stresses and normal bending stresses. Compressive prestress reduces the principal tensile stress thereby increasing the load necessary to cause inclined cracks. The use of thin webs will increase inclined stresses.

210.1.3—Conditions for shear failure

The resistance to formation of inclined cracks is greater with larger pre-stress and increasing web thickness. The significance of inclined cracks is less with low ultimate flexural strength caused by low ratio of reinforcement. Their significance is also less with low shear/moment ratios. If inclined cracks occur in an unreinforced web, sudden failure by shear is almost certain. If the web is adequately reinforced, ultimate flexural strength can be developed.

210.2—Web reinforcement

210.2.1—Critical percentage of tensile steel

Experimental data, although limited, indicate that inclined tension cracks will not form and web reinforcement will not be required if the following condition is satisfied:

$$\frac{pf'}{f'_c} \geq 0.3 \frac{f'_{cc} b'}{f'_c b}$$

where $b'$ = thickness of web; $b$ = width of flange corresponding to that used in computing $p$

This expression may be conservative for members having span/depth ratios greater than about 15 or for uniformly loaded members. In such cases, web reinforcement may not be required even though the percentage index, $pf'/f'_c$, exceeds that given in the above expression. The omission of web reinforcement in such members may be allowed when justified by tests.

210.2.2—Design of web reinforcement

The amount of web reinforcement necessary to develop required ultimate flexural capacity is a function of the difference between inclined cracking load and ultimate load in flexure. This difference varies rather widely as a function of prestress force, web thickness, amount of tensile reinforcement, and shear/moment ratio but is usually smaller for prestressed concrete than for conventional reinforced concrete. Current design procedures for web reinforcement in reinforced concrete are conservative for prestressed concrete.

Available test data indicate that the following expression for area of web reinforcement, with its factor of $\frac{1}{2}$, will give reasonably conservative results
for prestressed members of usual dimensions and properties. Since the formula does not involve the prestress force it may not be conservative for very low prestress or where only a portion of the reinforcement is stressed. For such cases it may be necessary to increase the factor of \( \frac{1}{2} \) as the member approaches the condition of conventionally reinforced concrete.

\[
A_\ast = \frac{\frac{1}{2} (V_u - V_a) s}{f'_y j d}
\]

where

- \( A_\ast \) = area of web reinforcement at spacing \( s \), placed perpendicular to the axis of the member
- \( V_u \) = shear due to specified ultimate load and effect of prestressing
- \( V_a \) = 0.06\( f'_y b' j d \) but not more than 180 \( b' j d \)
- \( s \) = longitudinal spacing of web reinforcement
- \( f'_y \) = yield strength of web reinforcement

210.2.3—Minimum quantity of web reinforcement

Because of the nature and limited knowledge of shear failures, it is suggested that some web reinforcement be provided even though the criterion of Section 210.2.1 is satisfied.

Where the web reinforcement is designed by Section 210.2.2, the minimum amount of web reinforcement should be \( A_\ast = 0.0025 b'/s \). This requirement may be excessive for members with unusually thick webs and the amount of web reinforcement may be reduced if tests demonstrate that the member can develop its required flexural capacity.

Heavily loaded members with thin webs and relatively small span/depth ratios, such as highway bridge girders and crane girders should have web reinforcement (see Section 206.6).

210.2.4—Spacing of web reinforcement

The spacing of web reinforcement should not exceed three-quarters the depth of the member. In members with relatively thin webs, spacing should preferably not exceed the clear height of the web.

210.2.5—Critical sections for shear

Because formation of inclined cracks reduces flexural capacity the critical sections for shear will usually not be near the ends of the span where the shear is a maximum but at some point away from the ends in a region of high moment.

For the design of web reinforcement in simply supported members carrying moving loads, it is recommended that shear be investigated only within the middle half of the span length. The web reinforcement required at the quarter-points should then be used throughout the outer quarters of the span.

For simply supported members carrying only uniformly distributed load, the maximum web reinforcement may be taken as that required at a distance from the support equal to the depth of member. This amount of web reinforcement should be provided from this point to the end of member. In the middle third of the span length, the amount of web reinforcement provided should not be less than that required at third-points of the span.
211—BOND AND ANCHORAGE

211.1—Pretensioning

211.1.1—Prestress transfer bond

Bond between the pretensioned steel and concrete is necessary to establish a prestress in the concrete. The transfer of force from the steel to the concrete takes place in a finite length in the end region of a member and the function of the resulting bond, termed “prestress transfer bond,” is anchorage of prestressing steel. Prestressing force varies from near zero at the end to a maximum value some distance from the end.

Transfer length will generally be of minor significance in long members, but it should be considered for short members or those in which the loading conditions may cause cracking in or near the region of prestress transfer.

211.1.2—Flexural bond

Flexural bond is the bond stress developed as a consequence of flexure. Bond stress at design loads in uncracked members is usually not critical since the increase in steel stress resulting from flexure is usually not significant. If cracking is anticipated under design loads, bond stress should be given special consideration.

211.1.3—Significance of bond stress at ultimate load

Bond failure should not occur prior to the development of the required ultimate flexural capacity.

For span lengths usually associated with prestressed concrete, bond failure is not a significant design factor. Bond adequacy in extremely short members should be investigated by test.

The factors affecting bond are concrete strength, perimeter shape, area and surface condition of prestressing steel, stress in the steel at ultimate strength, length of transfer zone, and superimposed load pattern.

212—COMPOSITE CONSTRUCTION

212.1—Introduction

Prestressed concrete structures of composite construction are comprised of prestressed concrete elements and plain or conventionally reinforced concrete elements interconnected in such a manner that the two components function as an integral unit. The prestressed elements may be pretensioned or post-tensioned and may be precast or cast in place. The plain or reinforced concrete elements are usually cast in place.

212.2—Interaction

212.2.1—Shear connection

To insure integral action of a composite structure at all loads, a connection should be provided between the component elements of the structure capable of performing two functions:
(1) To transfer shear without slip along the contact surfaces, and
(2) To prevent separation of the elements in a direction perpendicular to the
contact surfaces.

212.2.2—Transfer of shear
Slip may be prevented and shear transferred along the contact surfaces
either by bond or by shear keys. It should be assumed that the entire shear
is transferred either by bond or by shear keys.

212.2.3—Anchorage against separation
Mechanical anchorage in the form of vertical ties should be provided to
prevent separation of the component elements in the direction perpendicular
to the contact surfaces. Web reinforcement or steel dowels adequately
embedded on each side of the contact surface will provide satisfactory mechanical
anchorage.

212.3—Design of shear connection

212.3.1—Loading stage
The shear connection should be designed for ultimate load.

212.3.2—Magnitude and transfer of ultimate shear
The shear at any point along the contact surface may be computed by the
usual method as $v = (V_sQ)/I$. If the bond capacity is less than the computed
shear, full width shear keys should be provided throughout the length of the
member. Keys should be proportioned according to concrete strength of each
component of the composite member.

212.3.3—Capacity of bond
The following values are suggested for ultimate bond resistance of the contact
surfaces.

- When minimum steel tie requirements of Section 212.3.4 are fol-
  lowed .................................................. 75 psi
- When minimum steel tie requirements of Section 212.3.4 are fol-
  lowed and the contact surface on the precast element is artificially
  roughened ............................................. 150 psi
- When additional steel ties in excess of the requirements of Section
  212.3.4 are used and the contact surface of the precast element
  is artificially roughened .......................... 225 psi

212.3.4—Vertical ties
In the absence of experimental information on the capacity of vertical ties
it is recommended that all web reinforcement be extended into the cast-in-
place concrete.

Spacing of vertical ties should not exceed four times the minimum thickness
of the composite elements, or 24 in. whichever is less. The total area of vertical
ties should not be less than that provided by two #3 bars spaced at 12 in.

* Lack of experimental data makes the committee hesitate to recommend a shear stress at the root of a key.
Indications are that for keys on bridge girders in current use shear stress at the root of a key as high as 0.3fy would
sometimes be required to transmit ultimate shear forces.
For light pretensioned members such as those used for building floors not subjected to repetitive loads the above minimum requirements may be too severe. The committee is not prepared to recommend an amount or spacing of steel for this type member.

212.4—Design of composite structures

212.4.1—Design of composite section.
Physical properties of the composite section should be computed on the assumption of complete interaction between component elements. For structures composed of concretes of different qualities, the area of one of the component elements should be transformed in accordance with the ratio of the two moduli of elasticity.

212.4.2—Beam and slab construction
If the structure is composed of beams with a cast-in-place slab placed on top of the beams, effective slab width should be computed in the same manner as for integral T-beams.

212.4.3—Allowable stress with different concrete strengths
In structures composed of elements with different concrete strengths, the allowable stresses should be governed by strength of the portion under consideration.

212.4.4—Superposition of stress
Stresses may be superposed in design calculations that involve elastic stresses. Superposition of stresses should not be used in computing ultimate strength since inelastic action of the material is involved.

212.4.5—Stress after structure becomes integral
The properties of the composite cross section should be used in computing stresses due to loads applied after the structure becomes integral.

212.4.6—Shrinkage stresses
In structures with a cast-in-place slab supported by precast beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of precast beams. Stresses due to differential shrinkage are important only insofar as they affect cracking load. When cracking load is significant, such stresses should be added to the effects of loads.

212.4.7—Ultimate strength
Ultimate strength of a composite section should be computed in the same manner as ultimate strength of an integral member of the same shape.

213—CONTINUITY

213.1—Determination of moments, shears, and thrusts
Moments, shears, and thrusts produced by external loads and prestressing force should be determined by elastic analysis. Effects of axial deformation should be considered. Determination of effects produced by the prestressing
forces should take into account the restraint of attached structural elements and supports.

213.2—Stresses
   Allowable stresses are those recommended in Section 207.

213.2.1—Prestress
   When prestressing is to be applied in more than one stage, the internal stresses should be investigated at each stage.

213.3—Frictional losses
   Frictional losses in continuous post-tensioned steel may be more significant than in simply supported members.

213.4—Ultimate strength
   The ultimate strength of a continuous member should be evaluated not only at points of maximum moment, but also at intermediate points. In applying ultimate load factors where dead load causes effects opposite to those of live load, consideration should be given to load factor combinations in which dead load factor may equal one. It is recommended that moment redistribution not be considered in design at the present time.

214—END BLOCKS

214.1—Purpose
   An enlarged end section, called an end block, may be required to transmit concentrated prestressing forces in a shaped member from the anchorage area to the basic cross section.

   End blocks may be required to provide sufficient area for bearing of anchorages in post-tensioned design. They may be needed to transmit vertical and lateral forces to supports and to facilitate end detailing.

214.2—Requirements
   In pretensioned members with large concentrated eccentric prestressing elements, end blocks should be used. For lightly pretensioned members, or members of approximately rectangular shape, end blocks may be omitted. However, reinforcement should always be provided in the anchorage zone.

   In post-tensioned, shaped members, end blocks should be provided.

214.3—Proportioning
   End blocks are usually proportioned by experience. Depending on the degree of concentration and eccentricity of the prestressing force at the end surface, the length of the end block should be from one-half the depth of the member to the full depth. In general, shallow members should have an end block length equal to the depth, and deep beams should have an end block length equal to three-quarters of the depth. Length of an end block can be considered as the distance from beginning of anchorage area to the point where the end block intersects the narrowest width of member.
214.4—Reinforcement
Reinforcing is necessary to resist tensile bursting and spalling forces induced by the concentrated loads of the prestressing steel. A reinforcing grid with both vertical and horizontal steel in the plane of the cross section should be provided directly beneath anchorages to resist spalling forces. Closely spaced reinforcement should be placed both vertically and horizontally throughout the length of the end block to resist tensile forces.

215—FIRE RESISTANCE

215.1—General
The fire resistance of both prestressed concrete and reinforced concrete is subject to the same general limitations. One is the rate of heat transmission through the concrete from the surface exposed to fire to the unexposed surface. The other is the reduction of steel strength at the temperatures induced in the steel during the test. Either limitation may govern.

215.2—Heat transmission
Since the rate of heat transmission through prestressed concrete is similar to that of reinforced concrete of the same composition, the critical dimensions to control temperature rise at the unexposed surface will be the same in prestressed or reinforced concrete members.

215.3—Load-carrying capacity
The ability of the structure to carry required loads during fire test depends largely on thickness of cover over prestressing steel. The following minimum thicknesses of concrete cover on prestressing steel and end anchorages are recommended for various fire ratings:

<table>
<thead>
<tr>
<th>Hour rating</th>
<th>1 hr</th>
<th>2 hr</th>
<th>3 hr</th>
<th>4 hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum concrete cover</td>
<td>1½ in.</td>
<td>2½ in.</td>
<td>3 in.</td>
<td>4 in.</td>
</tr>
</tbody>
</table>

Data now available are insufficient to make recommendations for such factors as shape of cross section, type and arrangement of prestressing steel. The cover thicknesses recommended are believed to be conservative.

216—COVER AND SPACING OF PRESTRESSING STEEL

216.1—Cover
The following minimum clear concrete covers are recommended for prestressing steel, ducts, and non-prestressed steel.

<table>
<thead>
<tr>
<th>Minimum concrete cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete surfaces exposed to weather</td>
</tr>
<tr>
<td>Concrete surfaces in contact with ground</td>
</tr>
<tr>
<td>Beams and girders not exposed to weather</td>
</tr>
<tr>
<td>Prestressing steel, and main reinforcing steel</td>
</tr>
<tr>
<td>Stirrups and ties</td>
</tr>
<tr>
<td>Slabs and joists not exposed to weather</td>
</tr>
</tbody>
</table>
216.2—Spacing at ends

216.2.1—Spacing of pretensioning steel
Minimum horizontal or vertical clear spacing between pretensioning steel elements at ends of members should be three times the diameter of the steel or $1\frac{1}{2}$ times the maximum size of coarse aggregate, whichever is greater.

216.2.2—Spacing of post-tensioning ducts
The clear space between conduits at the ends should be a minimum of $1\frac{1}{2}$ in. or $1\frac{1}{2}$ times the maximum size of coarse aggregate, whichever is greater.

216.2.3—Dimensions of post-tensioning ducts
When steel is placed inside conduits which are to be filled with cement grout, such conduits should have a minimum inside diameter $\frac{3}{4}$ in. larger than the diameter of the prestressing steel.

216.3—Draped prestressing steel
When prestressing steel is placed in a curved or deflected position, steel or conduits may be bundled together in the middle third of the span length provided the minimum spacing recommended in Section 216.2.1 and 216.2.2 is maintained for a minimum distance of 3 ft at each end of member. The committee is not prepared to suggest limits for the number of conduits or prestressing steel elements that may be bundled horizontally and vertically. Excessive bundling may lead to insufficient bond capacity in pretensioned members, resulting in bond slip.

CHAPTER 3—MATERIALS

301—INTRODUCTION
The nature and economics of prestressed concrete construction require the use of high strength materials. Ability to sustain high stresses with a minimum of time-dependent change in stress or strain is essential.

These requirements are more severe than those for conventionally reinforced concrete. Highest standards of manufacture and construction should be observed. Prior to adoption of new materials, sufficient test data should be obtained to verify properties assumed in design.

302—CONCRETE

302.1—Scope
Particular attention should be given to properties of individual materials used in prestressed concrete and their effect on compressive strength, modulus of elasticity, drying shrinkage, creep, bond strength, and uniformity of concrete in place.

When new materials and methods are employed, trial mix investigations should include tests for drying shrinkage, creep, and modulus of elasticity.
302.2 Materials

302.2.1 Portland cement

Portland cement should conform to one of the following:
Specifications for Portland Cement (ASTM C 150)
Specifications for Air-Entraining Portland Cement (ASTM C 175)
Specifications for Portland Blast Furnace Slag Cement (ASTM C 205)
Specifications for Portland-Pozzolan Cement (ASTM C 340)

302.2.2—Concrete aggregates

Concrete aggregates should conform to one of the following:
Specifications for Concrete Aggregates (ASTM C 33)
Specifications for Lightweight Aggregates for Structural Concrete (ASTM C 330)

Mineral composition and soundness of aggregates may have a marked influence on compressive strength, modulus of elasticity, drying shrinkage, and creep.

Concretes made with some lightweight aggregates may exhibit a lower modulus of elasticity, greater creep and drying shrinkage than do concretes of the same strength made with aggregates of normal weight.

The range of properties possible in the same concrete mix with different lightweight aggregates may be large. Therefore, it is recommended that test data should be obtained for compressive strength, modulus of elasticity, drying shrinkage, creep, modulus of rupture, and bond.

302.2.3—Water

Water for mixing concrete should be clean and free of injurious quantities of substances harmful to concrete or to prestressing steel. Sea water should not be used for making prestressed concrete.

302.2.4—Admixtures

Certain admixtures may be beneficial to fresh or hardened concrete. However, admixtures should not be used until shown by test to have no harmful effect on the steel or concrete.

The use of calcium chloride or an admixture containing calcium chloride is not recommended where it may come in contact with prestressing steel.

302.3—Proportioning, batching and mixing

The proportioning of materials, batching, and mixing of concrete for prestressing should be done in accordance with the ACI Manual of Concrete Inspection, the U. S. Bureau of Reclamation Concrete Manual, or other comparable regulations including ACI Standards "Recommended Practice for Winter Concreting (ACI 604-56)," "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)," "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)," and "Standard Specifications for Ready-Mix Concrete" (ASTM C 94).

Available materials should be proportioned to produce concrete meeting specification requirements with a minimum water content. Slump of fresh
concrete should be as low as feasible. Cement, sand, and narrow-size ranges of coarse aggregate should be separately batched by weight. Water and some liquid admixtures may be batched by volume with accurate measuring equipment. Close control of all materials and operations is essential.

302.4—Strength

The strength required at given ages should be specified by the designer. Controlled concrete should be used and tested in accordance with Section 304 as modified by Section A602(f) of "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

303—GROUT

303.1—General

When required by job specifications, post-tensioned steel should be grouted to completely fill the void surrounding the prestressing steel with a portland cement grout to insure high flexural bond strength and provide permanent protection for the steel.

303.2—Materials

Grout should be made of either (a) cement and water or (b) cement, fine sand, and water. Mix (a) should be used where the cavity is very small. Either Mix (a) or Mix (b) may be used where the cavity is relatively large. Admixtures should conform to recommendations of Section 303.2.4.

303.2.1—Portland cement

Same as Section 302.2.1.

303.2.2—Sand

Sand should preferably be a natural quartz sand meeting "Tentative Specification for Aggregate for Masonry Mortar (ASTM C 144)," except for gradation requirements. The sand should pass a No. 30 sieve, about 50 percent should pass a No. 50 sieve, and about 20 percent should pass a No. 100 sieve.

303.2.3—Water

Same as Section 302.2.3.

303.2.4—Admixtures

Certain admixtures may be beneficial to fresh or hardened grout. However, no admixture should be used until shown by test to have no harmful effect on the steel or grout.

Calcium chloride or an admixture containing calcium chloride is not recommended for use in grouting post-tensioned members.

303.3—Proportioning

Proportions of grouting materials should be based on results of tests made on fresh and hardened grout prior to beginning work. Grout should have the consistency of thick cream or heavy paint. When permitted to stand until setting takes place, grout should neither bleed nor segregate.
304—PRESTRESSING STEEL

304.1—General

High tensile strength steel is required in prestressed concrete to provide necessary internal concrete stresses after losses have occurred. The following four types are in common use:

(a) High tensile strength single wire, applied in the form of assemblies made up of two or more substantially parallel wires. They may be used for either pretensioning or post-tensioning purposes.

(b) Small diameter, high strength strand, shop fabricated, is usually made up of six wires spiraled around a center wire. Small diameter strand is normally, though not exclusively, used for pretensioning purposes.

(c) Large diameter high strength strand is usually shop fabricated with factory attached end fittings for post-tensioned construction. It has 7, 19, 37, or more individual wires.

(d) High strength alloy steel bars are produced by a cold stretching or drawing process. They are currently available in diameters ranging from $\frac{3}{16}$ to $1\frac{3}{16}$ in. Alloy steel bars are used principally for post-tensioned construction.

Each type of prestressing steel should be made to distinctly separate specifications, of which the following sections give a general description.*

304.2—High tensile strength single wire

High tensile strength single wire is generally made from high carbon steel hot rolled into rods. It is then heat treated by a process termed "patenting" and cold drawn to produce the required final tensile strength. In its most commonly used form the wire is then stress relieved by a controlled time-temperature treatment that improves elastic properties within the tensile range usually employed in prestressing concrete. It also produces a straighter, more easily handled wire.

High tensile strength wire produced by the oil tempering process is not recommended for use in prestressed concrete.

304.2.1—Ultimate tensile strength

High tensile strength wire for prestressed concrete is made to minimum tensile strengths as high as 250,000 psi for a diameter of 0.190 in. Higher tensile strengths are available at smaller diameters and lower tensile strengths at larger diameters.

304.2.2—Shape of stress-strain curve

Stress relieved wire for prestressing should display a high yield strength and a reasonable elongation before rupture. Minimum yield strength at 1 percent elongation under test load should be equal to 85 percent of specified ultimate tensile strength. Minimum elongation after rupture should be 4 percent in 10 in. Elongation tests should conform to "Specification for Mechanical Testing of Steel Products" (ASTM A370-54T).

*The American Society for Testing Materials is currently formulating specifications for prestressing steels.
304.2.3—Ductility
Wire for prestressing should be capable of a reasonable amount of cold deformation without failure. It should have a minimum reduction in cross-sectional area of 30 percent at rupture.

304.2.4—Creep and relaxation
Data concerning typical creep and stress relaxation properties of the material should be obtained from the manufacturer. Special acceptance tests for individual lots are usually expensive and unnecessary.
Creep tests and short-term relaxation tests do not necessarily represent long-time stress relaxation characteristics.

304.3—Small diameter high strength wire strand
Small diameter high strength strand is normally made of seven wires. A straight center wire is enclosed tightly by six spirally wound outer wires. Because of its small diameter, strand can be given a final stress-relieving treatment similar to that for single wires. This treatment improves elasticity and handling characteristics. Acceptance tests, when required, should be made on the strand rather than single wires.
Physical properties should be based on the total metallic area of all the individual wires. Ultimate tensile strength, shape of stress-strain curve, ductility, creep and relaxation should be the same as described in Section 304.2 (high tensile strength single wire) except as follows:
(a) Minimum elongation at rupture, 3.5 percent in 24 in.
(b) Minimum yield strength at 1 percent elongation under test load equal to 85 percent of specified ultimate tensile strength.

304.4—Large diameter high strength wire strand
Large strand may be made of 7, 19, 37, or more galvanized or uncoated hard-drawn wires, spirally wound. Galvanized strand is most commonly used.
Because large diameter strand cannot be given a final stress-relieving treatment, some of its physical properties differ from those of wire or small strand. Acceptance tests, when required, should be based on properties of the strand rather than individual wires.

304.5—Cold stretched high strength alloy steel bars
These bars are usually made from alloy steel designated AISI 5160 or AISI 9260. After hot rolling, the bars are either heat treated or cold worked. Each bar is then cold stretched to a minimum of 90 percent of the specified ultimate strength.

304.5.1—Ultimate tensile strength
High strength alloy steel bars are produced with a minimum tensile strength of 145,000 psi for all diameters.

304.5.2—Shape of stress-strain curve
High strength bars for prestressing should have a minimum yield strength at 0.2 percent permanent strain equal to 90 percent of the specified ultimate
tensile strength. Minimum elongation after rupture should be 4 percent in a length of 20 diameters.

304.5.3—Ductility

Bars for prestressing should be capable of a reasonable amount of cold deformation without failure. The bar should have a reduction of area of not less than 15 percent at rupture.

304.5.4—Creep and relaxation

Data concerning typical creep and stress relaxation properties of the material should be obtained from the manufacturer. Special acceptance tests for individual lots are usually expensive and unnecessary.

Creep tests and short-term relaxation tests do not necessarily represent long-time stress relaxation characteristics.

304.6—Corrosion

Since prestressing steels are susceptible to corrosion, they should be protected during storage, transit, and construction.

The term stress corrosion is applied to the embrittlement of steel that occurs under the combined effects of high stress and some corrosive environments. It may take place without apparent surface impairment.

Normally, steel cast in concrete or properly grouted will not be subject to such corrosion. When post-tensioned steel is not grouted, special precautions should be taken to protect the steel (see Section 404.3.2).

305—ANCHORAGES AND SPLICES

305.1—General

Anchorages for post-tensioning elements now in general use consist of:

Threaded ends and wedge anchors for bars; factory attached end fittings for large diameter strand; button-head, sandwich plate, and conical wedges for parallel lay wire systems; and conical wedges for small diameter strand.

Splines are used primarily for bars and consist of threaded couplings.

305.2—Ultimate strength

Anchorages and splines should be capable of developing the ultimate strength of attached steel elements without excessive deformation.

305.3—Anchorage set

Movement of prestressing steel in anchorage during seating should be stated by the manufacturer and substantiated by test data.

CHAPTER 4—CONSTRUCTION

401—INTRODUCTION

This chapter outlines construction procedures that should result in sound and durable structures.

Prestressed concrete members are composed of high strength concrete and steel. Design stresses are closely controlled, but behavior in service depends
upon the specified concrete being properly placed in forms of the correct dimensions around accurately positioned prestressing steel or ductwork for steel. Construction requires accuracy and care. Deviation from careful workmanship may result in an unsafe structure and should not be condoned.

402—TRANSPORTING, PLACING, AND CURING OF CONCRETE

402.1—General

Quality of the finished concrete members depends on care used in transporting, placing, and curing. Recommended practice is outlined in “Building Code Requirements for Reinforced Concrete (ACI 318-56),” Sections 403-406, and “Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42).”

402.2—Placing

Low slump, high cement content mixes should be placed in the shortest possible time after mixing is completed to prevent loss of workability.

Concrete should be deposited close to its final position. The method of placement should be such that segregation will not occur.

402.3—Vibration

Internal or external vibration or both are usually necessary to produce dense, well-compacted concrete.

Vibrators should not be used to move concrete horizontally in the form. Overvibration should also be avoided.

When internal vibration is used, vibrator heads should be smaller than the minimum distance between ducts or prestressing steel. Care must be exercised to avoid damage to or misalignment of ducts for post-tensioning steel.

Vibration is not a substitute for workability. Judgment should be used in specifying slump, and approved methods of vibration used to achieve maximum compaction.

402.4—Construction joints

In long cast-in-place members the use of construction joints is recommended (1) to reduce cracking near columns caused by settlement or movement of shoring and falsework, and (2) to allow for shrinkage. In general, joints should be placed near falsework supports.

Construction joints preferably should be perpendicular to prestressing steel. Joints should not be made parallel to prestressing steel unless the provisions of Section 212 (composite construction) are followed.

402.5—Curing

Curing should start soon after finishing. If high temperature curing is used, an initial setting time prior to application of heat should be required. Curing should continue until the required strength for application of the prestress force is reached. Fresh concrete should be protected from rain or the rapid
loss of moisture prior to the curing period. Rapid drying should be prevented until the final design strength is obtained.

When high temperature curing is used, the rate of heating and cooling should be controlled to reduce thermal shock to the concrete.

Where identical precast members are required, curing conditions should be uniform to maintain proper quality control.

402.6—Protection from freezing

During periods of freezing temperatures, ungrouted ducts should be blown clear of water or protected against freezing.

403—FORMS, SHORING, AND FALSEWORK

403.1—General

Quality of concrete members depends on the care used in constructing forms and falsework. Correct practices outlined in “Building Code Requirements for Reinforced Concrete (ACI 318-56)” Sections 501 and 502 are recommended.

403.2—Special requirements

Forms for pretensioned members should be constructed to permit movement of the member without damage during release of the prestressing force.

Forms for post-tensioned members should be constructed to minimize restraint to elastic shortening during prestressing and shrinkage. Deflection of members due to the prestressing force and deformation of falsework should be considered in design. Form supports may be removed when sufficient prestressing has been applied to carry dead load, formwork carried by the member, and anticipated construction loads.

404—PLACEMENT OF PRESTRESSING STEEL AND APPLICATION OF PRESTRESSING FORCE

404.1—General

The location of the center of gravity of the prestressing steel, initial and final prestressing force, and the assumed losses due to creep, shrinkage, elastic shortening, and friction shown on the plans are based on the use of specified materials. Other materials not specified but capable of producing the same results may be used with approval of the engineer.

Unless tolerances for location of the prestressing steel are shown, a variation of ± 1/8 in. to ± 1/4 in. depending on size of the member, is suggested as maximum permissible.

404.2—Pretensioning steel

404.2.1—General

Steel should be kept clean and dry. Foreign matter, grease, oil, paint, and loose rust should be removed prior to casting concrete. A light coat of
rust is permissible and sometimes preferable provided loose rust has been removed and the surface of the steel is not pitted.

404.2.2—Measurement of prestressing force

Pretensioning force should be determined by measuring elongation and checking jack pressure on a calibrated gage. Measurement of elongation will usually give more consistent results. When there is a difference of over 5 percent between the steel stress determined from elongation and from the gage reading, the cause of the discrepancy should be ascertained and corrected.

If several wires or strands are stretched simultaneously, provision must be made to induce the same initial stress in each.

404.2.3—Transfer of prestressing force

The force in the prestressing steel should be transferred to the concrete smoothly and gradually. If the force in the wires or strands is transferred individually, a sequence of release should be established by the engineer to avoid subjecting the member to unanticipated stresses. Any variation in this sequence should be submitted to the engineer for approval.

404.2.4—Protection

Ends of pretensioning steel exposed to weather or corrosive atmosphere should be protected by a coating of asphaltic material. They should preferably be recessed in the member, coated with asphaltic material and covered with mortar.

404.3—Post-tensioning steel

404.3.1—General

The steel should be kept clean and dry. For bonded construction, foreign matter, grease, oil, paint, and loose rust should be removed prior to placing steel in ducts. A light coat of rust is permissible provided loose rust has been removed and the surface of the steel is not pitted.

404.3.2—Protection

For general use in unbonded construction, galvanizing may be considered to protect the steel from corrosion when coated with grease or asphalt-impregnated material and enclosed in a sheath. Uncoated galvanized steel may be used when it is accessible for inspection and points of bearing are equipped with special shoes to prevent damage to the galvanizing.

If wrappings and coatings are used on nongalvanized steel, the coating should protect the steel from corrosion during shipment, storage, construction, and after the steel is in place. It should permit movement of steel during stressing with minimum friction. The method of protection should be specified or approved by the engineer.

Anchorage and end fittings should be given protective treatment consistent with that given the prestressing steel. They should preferably be recessed in the member and covered with mortar.
404.3.3—Placement of steel and enclosures
Ducts or enclosures for prestressing steel are formed in the concrete using tubing, metallic casings, or other materials. They should be positioned and secured to maintain the prestressing steel within the allowable placement tolerances.

For bonded construction, ducts or duct-forming devices should be free from grease, paint, or other foreign matter. Ducts should be protected against entrance of foreign matter prior to grouting.

Anchorage hardware to be cast in the member should be firmly fastened to forms in the proper location.

404.3.4—Measurement of the prestressing force
Values of total elongation, corrected for assumed friction loss and anchorage set, and corresponding jack pressures at various increments of prestress should be supplied by the engineer. When a difference of over 5 percent exists between steel stress determined from the corrected elongation and from corresponding gage reading, stressing operation should cease. If the cause of the discrepancy is neither faulty measurement nor equipment, the engineer should be consulted.

404.3.4.1—Factors influencing friction—As prestressing force is applied, friction between prestressing steel and curved enclosure reduces steel stress at points away from the jack. The amount of friction loss is a function of degree of curvature, type and length of prestressing steel, duct material, presence of friction reducing agents, accuracy of placing the duct, and degree of disturbance during concrete placement.

It is the responsibility of the contractor to be aware of these factors. He should use materials specified and insure that the quality of workmanship results in accurate duct positioning with minimum displacement during construction.

404.3.5—Prestressing in stages
When the prestressing force is to be applied in more than one stage, excessive concrete stresses should be avoided during intermediate stages. The engineer will designate location and magnitude of the forces to be used for each stage and allowable external loads that may be placed on the member. The contractor should be aware of the significance of overloading the member.

404.3.6—Anchorage set
For friction type anchorages the manufacturer or supplier should state the amount of slip normally expected in seating the anchorage device.

404.3.7—Effect of temperature
Changes in temperature should have little effect on prestressing reinforcement unless there is a significant temperature differential between concrete and steel.
405—GROUTING

405.1—General
When grouting is specified for post-tensioned members it should completely fill all enclosure voids.

405.2—Mixing
Grout should be mixed in a mechanical mixer. Immediately after mixing, it should be passed through a strainer into pumping equipment which provides for recirculation. Grout should be pumped into the duct as soon as possible after mixing but may be pumped as long as it retains the proper consistency.

405.3—Arrangement of grout pipes
Ducts must be provided with entrance and discharge ports, each of which can be closed. Extension pipes may be used when necessary.
For long members, grout may be introduced at one end until it discharges from an intermediate point. The point of application may then be moved successively forward. Grout may be introduced at an intermediate point if discharge ports are provided at duct ends. The sequence of grouting should be planned to insure complete filling. Devices for bleeding air may be required at high points of the duct profile.

405.4—Test for passage of grout
Free passage of grout from entrance to discharge port must be assured. Tests may be made by pumping water, air, or other fluids through the duct.

405.5—Application of grout
Grout should be applied continuously until it flows steadily from the discharge port indicating removal of trapped air and water. The discharge port should then be closed and grouting pressure maintained for the length of time necessary to insure complete filling of the void. The entrance should then be closed and the pumping nozzle removed.

405.6—Protection against freezing
Adequate precautions must be taken to prevent freezing fresh grout.

406—HANDLING AND ERECTION
Where precast members are specified, methods of handling and/or the sequence of erection should be indicated. When these are not indicated on the plans, the contractor should submit for approval the location of pick-up points, minimum concrete strength when handled, method of transporting, and sequence of erection.
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