Architectural Panel Design and Production Using Post-Tensioning

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Presents an economical design method, including practical manufacturing techniques, for producing architectural precast concrete panels for both crack and deflection control using low-friction unbonded tendons. Design examples illustrate the proposed method and a cost comparison of post-tensioned versus mild steel reinforced panel production is given.

The procedure in designing architectural precast concrete panels can vary considerably among precasters. Nevertheless, the primary objective is to provide a structurally sound and architecturally pleasing member (see Fig. 1) that is reasonably crack-free during its intended life span.

Over the years, the method of pre-tensioning panels has provided excellent quality, but in most cases has not been practical because of the long line casting beds required.

Today, however, new developments in plastic-coated strand (7-wire 270 kips), makes post-tensioning available to any precaster at basically no initial set-up costs. (Renting or purchasing a small portable stressing jack is the only cost.)

Post-tensioning with this mono-strand system provides a method for
prestressing most architectural panels without any appreciable change from present casting procedures.

### Design Considerations

The design of architectural panels not only includes the usual considerations of service loads, wind, gravity, and restraint to volume changes, but also careful attention to stripping, handling, shipping, and erection loads. In fact, the critical factor affecting the design of architectural precast panels is usually due to erection loads.

Today, it is common for design engineers and architects to proportion panels for service conditions once they are in place. Unfortunately, in many cases, adequate attention is not given to problems of handling and erection.

In practice, many slender panels are detailed to span vertically two or three stories or horizontally from column to column thus eliminating heavy supporting members. Architectural panels usually perform their function on the building quite easily, but because of the architectural finish they must be cast horizontally face down.

Frequently, the high handling stresses require multi-point picking and handling. Therefore, yard equipment must have high clearances to permit this sophisticated method of handling.

Also, transportation becomes a problem and often panels must be shipped flat because of their size and unbalanced trailer loadings. Since the trailers are very flexible, the method of shipping panels supported at more than two points becomes very risky.
One solution to these problems is to prestress the panels adequately to handle them with a simple two-point pick. Unfortunately, pretensioning requires jacking bulkheads or self-stressing forms. This technique usually becomes economical only in long line casting situations.

Since architectural panels usually require several form changes and are in many cases very elaborate, a long line of form would be very costly. Post-tensioning eliminates the jacking bulkheads and can be easily adapted to a multitude of panel shapes and configurations.

A secondary design consideration is the control of deflections. Many architectural panels may not be subjected to high stresses, but may show undesirable deflections. Post-tensioning allows for a variety of strand configuration, and with the appropriate strand eccentricities deflections can be reduced to a minimum.

Deflections, which are usually not considered in architectural panels, can be easily controlled. Thus, post-tensioning is not only useful in controlling stresses and cracking during handling, but provides the precaster the ability to design architectural panels for deflections.

### Design Criteria

The basic design philosophy is to eliminate cracking during stripping, handling, shipping, erection, and service conditions. The following design criteria will help fulfill this goal:

**Limiting tensile stress**

At the start, the designer must decide on the limiting tensile stress allowable at some percentage of the theoretical cracking stress. For hard-rock concrete the theoretical cracking stress is $7.5 f'_{c}$.

Depending on the use of the panel and the quality control requirements, the limiting tensile stress might vary from zero to the actual theoretical cracking stress. For most cases, a limiting stress at 50 to 75 percent of theoretical cracking will provide adequate safety against cracking.

**Load factors**

In addition to the limiting tension stress, the designer must decide on load factors to compensate for overloads due to impact, suction during stripping, incorrect handling, and other unforeseen field problems. Depending on the panel shape, casting procedures, and quality control, the load factor could vary from 1 to 1.5. In general, a factor of 1.15 to 1.25 for handling, shipping, and erection loads is adequate.

Service loads do not require any load factor, since the appropriate load factors would be covered by ultimate strength design. Suction loads during stripping vary greatly with the casting procedures, but for most cases would be covered by a 10 to 20 psf force during stripping.

**Prestressing force**

The basic stress equation is:

$$\frac{P + Pe - \alpha M}{A E Z} - \frac{\beta f'}{Z} \geq \frac{\beta f'_{c}}{Z} \quad (1)$$

where

- $M = \text{applied moment}$
- $P = \text{effective prestressing force}$
- $A = \text{concrete area}$
- $Z = \text{appropriate section modulus}$
- $e = \text{eccentricity of prestressing force}$
- $f'_{c} = \text{concrete compressive strength}$
- $\alpha = \text{appropriate load factor}$
- $\beta = \text{factor determining limiting tensile stress}$

Note that a positive sign indicates compression and a negative sign denotes tension.

In general, the panel would be concentrically prestressed. Therefore, the eccentricity of the prestressing force
would be zero. A load factor of 1.25 will give satisfactory results in most cases, and \( \beta \) in Eq. (1) at 67 percent of theoretical cracking stress would equal 5.

Substituting \( e = 0 \), \( \alpha = 1.25 \), and \( \beta = 5 \) into Eq. (1) and solving for \( P \), the following formula is obtained:

\[
P \approx A \left[ 1.25 \left( \frac{M}{Z} \right) - 5 \sqrt{\frac{f_c}{c}} \right]
\]

(2)

Eq. (2) represents the general formula for calculating the minimum prestressing force required for concentrically prestressed flat panels.

**Prestress losses**

The calculation of prestress losses becomes particularly interesting when the strands are looped. The low friction qualities of the plastic coated strand allows the strand to be looped which produces considerable savings in stressing time and anchorage equipment. (The looping procedure and other pertinent details will be discussed later in this paper.)

An estimate of friction losses is calculated from the formula:

\[
T_x = T_o e^{-\left( \mu \alpha + k \pi \right)}
\]

(3)

where

- \( T_x \) = prestressing force at point \( x \)
- \( T_o \) = prestressing force at jacking end
- \( e \) = base of Naperian logarithm
- \( \mu \) = coefficient of angular friction
- \( \alpha \) = total angular change in radians from jacking end to point \( x \)
- \( k \) = wobble factor
- \( x \) = length of cable from jacking end to point \( x \) measured in feet

The *Post-Tensioning Manual* provides a nomogram and other design aids for solving Eq. (3). In general, for low-friction strand, where \( \mu \) varies from 0.03 to 0.05, \( k \) can be assumed to be negligible.

The other prestress losses to be considered include shrinkage, creep, steel relaxation, and anchorage set.

As an example of a detailed calculation of prestress losses, consider the same panel treated later in the paper (Example 1, Fig. 7). The dimensions of the panel are 8 x 32 ft and 6 in. thick.

The panel is prestressed with six \( \frac{1}{2} \)-in. diameter 270-kip strands, each strand applying a force of 23,134 lbs. Assume the prestress losses are about 20 percent.

The average prestress is:

\[
P = \frac{23,134 \times 6}{8 \times 12} = 241 \text{ psi}
\]

**Anchorage set** is compensated for by slight over-stressing and the strands are jacked from both ends of the bed.

Therefore, let \( \alpha = 0, \pi, \) and \( 2\pi \) for the first, second, and third strands, respectively. Applying Eq. (3), with \( \mu = 0.04 \), we obtain:

\[
\begin{align*}
T_x &= T_o (1) \quad \text{(first strands)} \\
T_x &= T_o (0.882) \quad \text{(second strands)} \\
T_x &= T_o (0.778) \quad \text{(third strands)}
\end{align*}
\]

\[
\text{Avg } T_x = T_o (0.8866) \text{ or } 11.3 \text{ percent prestress losses due to friction losses.}
\]

The prestress losses due to shrinkage, creep, and steel relaxation are found using the design aids furnished in the *PCI Design Handbook* (see especially pp. 5-57 to 5-61) suitably modified by judgment factors based on past experience.

Shrinkage ............... 6,000 psi
Creep = 16 (241) ........ 3,856 psi
Steel relaxation .......... 11,000 psi

\[
20,856 \text{ psi}
\]

(i.e., \( 20,856/189,000 = 0.11 \) or 11%)

Total losses (approx.) = 22 percent which is very close to the original estimate.

In conclusion, it should be mentioned that there are even more accurate methods available for calculating prestress losses. However, from a
practical standpoint a deviation of a few percentage points from the "true" value will not alter significantly the final design or performance of the member.

Finally, a word on friction losses in relation to actual panel production.

In the actual production of panels it has been noted that after the first end had been fully stressed and the jack applied to the second end very little elongation occurred. When the strand went through as much as \( \pi \) radians of curvature, it was observed that no movement of the jacking head occurred until 40 to 50 percent of the required gage reading was recorded.

The theoretical friction loss using a \( \mu \) and \( \alpha \) of 0.04 and 7\( \pi \), respectively, is about 60 percent. Therefore, by visual inspection only, the friction losses as calculated show good correlation with observed values. It should be emphasized, however, that these were only visual observations and that more accurate measurements using load cells would be more conclusive.

### Bearing stresses

When using looped strands some consideration should be given to local bearing stresses at the loops. For the minimum radius of curvature, the bearing stresses should be kept below the compressive strength of the concrete.

This minimum radius of curvature can be found from the basic equation for the tension in a ring subjected to a uniform pressure:

\[
y = \frac{T}{pb}
\]  
\[
y = \frac{29,000}{(3000 \times 0.5)}
\]  
\[
y = 19.33 \text{ in. (minimum)}
\]

Note that at the bend the strands exert a local bearing stress and that at several locations in Fig. 7 there are two or more strands at the same relative position. In calculating the minimum radius of curvature, the bearing stresses directly adjacent to the strands are only considered. Therefore, the crossing of the strands does not affect this calculation.

It should be noted that the effect of looping is not critical from the standpoint of the strands cutting through the concrete like a cheese slicer, but that the strands (if located too close to the surface) actually spall off the concrete on the surface allowing the strands to shift to the outside of the panel.

This type of failure has been effectively controlled by adding an extra layer of welded wire fabric at the looped areas, so that there is always a layer of mesh on each face of the panel. The purpose of the mesh is to spread out the localized spalling force. In addition, this reinforcement provides additional strength at the corners of the panel which are not post-tensioned.

### Deflection and crack control

Another important design criterion is the control of deflections and cracking in beams and panels used as structural members with architectural considerations. In many cases a structural member, which is exposed to view, is restricted to certain dimensions for architectural reasons. The member can be reinforced adequately to handle the design loads, but will exhibit undesirable effects such as large deflections or cracking.

With the use of plastic coated strand, deflections and cracking can be limited to any degree desired. In this respect, Eq. (1) is a helpful guide.
in controlling service load stresses and thus prevents cracking under full or partial service loads.

With post-tensioned design the strands can be placed in almost any desired location within the member. Thus, the eccentricity of the prestress force is almost unlimited. This makes the control of deflections a relatively simple procedure using load balancing techniques.

So in conclusion, although the actual design calculations still involve the usual problems such as prestress losses, creep, and other time-dependent effects, post-tensioning does enable the designer to control the behavior of any particular member within practical limits.

**Ultimate strength**

The final design consideration is the ultimate strength of the member. The ACI Code indicates the methods of calculating the ultimate strength of post-tensioned members with unbonded tendons and minimum bonded steel requirements. The total amount of prestressed and non-prestressed reinforcement must be adequate to develop a design load in flexure at least 1.2 times the cracking load calculated on the basis of the modulus of rupture.

In the case of structural members this requirement is usually supplied with the reinforcement required by strength design. However, for architectural flat panels, which are most conveniently post-tensioned concentrically, the cracking strength requirement is usually not satisfied without adding large amounts of mild reinforcing steel.

Because of the high handling stresses compared to the service load stresses for flat architectural panels, the amount of post-tensioning required for the handling stresses usually results in very favorable stresses during service loads. In most cases, the stresses are very low and the cracking moment requirement could be justifiably waived, since the panel would not crack until several times the design service loads were applied.

**Materials and Equipment**

The mono-strand post-tensioning system described is available from most suppliers of post-tensioning devices. The strand is greased with a low friction rust prohibitive grease and covered with a continuous plastic coating. It is generally supplied in reels similar to standard prestressing strand.

Each supplier has his own particular anchorage device consisting of a cast-iron anchor of which two or three part wedges are used to lock the strand in place. The anchors are usually recessed so that they may be concealed and protected from the elements once they are grouted. (Fig. 2 shows the pocket, strand, and anchor.) Plastic pocket formers are supplied, which conform to the shape of the particular anchor being used.

All that is needed to complete the post-tensioning operation is a stressing jack. A small hydraulic jack, which operates on standard 110-volt current is sufficient. The amount of extension of the ram needs to be less...
Fig. 3. Jacking apparatus.

than a foot, as compared to the jacks used for prestressing beds which require much larger extensions, because of the length of the strands being stressed.

The jack should also have an accurate pressure gage and have a capacity in excess of 30,000 lbs. An accurate gage is a necessity, since stressing the strands by calculating the elongation would be difficult because of the elaborate strand configurations and losses which occur during the jacking process. (Fig. 3 shows the jacking process.)

These jacks can be rented so that initial costs can be relatively inexpensive.

Production Techniques

Production techniques for post-tensioned members can be easily incorporated into methods currently being used for mild steel reinforced members. The only major change from conventional forming methods is that the strands must extend through the appropriate side or end rails.

Usually, a ¼-in. diameter hole is adequate, and the rail must be removable so that the strand can be tensioned before stripping. Plastic pocket formers are available which bolt onto the side rail, and provides the recess for the strand anchor.

Fig. 4 shows the layout of the looped tendons at the production site. In flat panels the strand is placed on light mesh so that the appropriate strand configuration can be maintained by tying the strand to the mesh. The mesh can then be supported on chairs or suspended from the top of the form by wire and cross bars.

These methods, especially chairing the mesh, give very good control over the final location of the strand within the panel. The panel shown in Fig. 7 can have the strands located within ±1 in. in the horizontal direction and ±¼ in. in the vertical direction. (Note that the directions are in relation to casting the panel flat face down.) The critical dimension to hold for the flat panel design is the vertical one.

The panel is cast using any appropriate casting procedure, and after the concrete has reached the minimum jacking strength the actual post-tensioning force is applied. After the rail and the pocket former are removed the plastic is stripped from the strand extending out of the pocket, and the strand cleaned. Then the wedges and strand anchor are placed over the strand and seated properly within the pockets. Now the panel is ready to be post-tensioned.

Throughout the post-tensioning operation it is important to remember that appropriate safety precautions must be taken. As with all prestressing operations, a ½-in. diameter strand, which has been stressed to 70 percent of its ultimate strength, has a tremendous amount of stored energy. If a failure should occur at this stage and this energy suddenly released, the consequences can be catastrophic.

Therefore, after the jack is attached to the strand, all personnel should be cleared from the area. The hydraulic pump and gages, which operate the jack, can be located a safe distance from the member.

The strand can then be jacked
slowly to the correct gage pressure. It is a good idea to check the elongation of the strand to correlate this with an approximate calculated elongation. This is done to ensure that the strand is entirely stressed. It is possible that the strand becomes bonded or kinked or the anchor is not working properly and the strand is not able to be fully stressed. If the measured elongation of the strand differs greatly from the initial estimate, it would be prudent to investigate the jacking operation.

Depending on the strand configuration it might be necessary to jack the strand from both ends to reduce the losses due to friction. It should be noted that both ends do not have to be tensioned at the same time. One end is fully or partially stressed and then the jack is moved to the other end of the strand. Since the strand is unbounded, the strand can easily be detensioned or retensioned if desired.

After the strands have been tensioned the panel can be moved to the storage area. Here the portion of the strand extending beyond the anchor can be cut back to within about 1 in. of the wedges, and the pocket grouted. The panel is now ready for shipping and installation.

Design Examples

To demonstrate the applicability of the foregoing design principles, a couple of numerical examples are worked out.

The first example treats a flat rectangular panel of uniform thickness. The second example covers the design of a rectangular beam supporting a double-tee roof system.

In following the design calculations close reference should be made to the PCI Design Handbook because many intermediate steps are omitted.
EXAMPLE 1

Consider a flat panel, 6 in. thick, 32 ft long and 8 ft wide, as shown in Fig. 5. The design wind load is 20 psf and the effective wind span is 15.5 ft.

Because of the architectural intent of the panel, it must be cast face down and shipped flat, two to a flat bed trailer. The yard handling is provided for by inserts at 6 ft 8 in. from each end. The erection is accomplished with inserts in the top of the panel, while the panel is raised with the handling inserts as shown in Fig. 6.

**Design data**

- \( A = 576 \text{ in.}^2 \);
- \( Z = 576 \text{ in.}^3 \);
- \( e = 0 \text{ in.} \) (concentric prestressing)
- \( f_c' = 3000 \text{ psi at stripping} \)
- Panel weight = 600 lb per ft
Stripping, handling, and erection moments

Moment at handling inserts:

\[ 6(600)(6.67)^2 = 160,160 \text{ in.-lb} \]

Maximum moment between inserts:

\[ 1.5(600)(18.66)^2 - 160,160 = 153,216 \text{ in.-lb}. \]

The minimum prestressing force is computed from Eq. (2) modified with \( \alpha = 1.5 \) and \( \beta = 3.75 \):

\[
P \geq A \left[ \alpha \left( \frac{M}{Z} \right) - \beta \sqrt{J_c} \right]
\geq 576 \left[ 1.5 \left( \frac{160,160}{576} \right) - 3.75 \sqrt{3000} \right]
\geq 121,932 \text{ lb}
\]

Assume that \( \frac{1}{2} \)-in. diameter 270-kip strand will be used with a working force of 23,134 lb allowing for 20 percent prestress losses.

The number of strands required is

\[
121,932 / 23,134 = 5.28
\]

Fig. 6. Handling and transportation of panels.
For design purposes use six strands producing a total prestress force of
$6 \times 23,134 = 138,804$ lb (see Fig. 7).

Check wind loading

Assume working stresses at 20 psf.
The maximum moment at floor support is:

$$M = 1.5(160)(15.5)^2$$
$$= 57,660 \text{ in.-lb}$$

The possible wind stress is found by subtracting the floor support stress from the total prestress:

$$f_w = 138,804/576 - 57,660/576$$
$$= 141 \text{ psi}$$

Fig. 7. Plan and section of flat rectangular panel used in Example 1.
Check ultimate moment

\[ M_u = 1.3(1.5)(20)(8)(15.5)^2 \]
\[ \approx 74,958 \text{ in.-lb} \]

\[ M_{cr} = \left[ 138,804/576 + \ 7.5\sqrt{5000} \right] 576 \]
\[ \approx 444,084 \text{ in.-lb} \]

Since \( M_{cr} \) is much greater than \( M_u \) required and the working stress design indicates the member is never in tension, it is reasonable to waive the 1.2\( M_u \) requirement.

Minimum \( M_u \) supplied:

\[ A_y = 0.918 \text{ in.}^2 \text{ at } d = 3 \text{ in.} \text{ (strand data)} \]

\[ A_y = 0.96 \text{ in.}^2 \text{ at } d \text{ (min.) = 2.5 in. (mild steel data)} \]

\[ \rho_y = 0.918/(3 \times 96) = 0.0031875 \]

\[ f_{se} = 151,200 + 10,000 + \frac{5000}{(100)(0.0031875)} \]
\[ = 176,886 \text{ psi} \]

\[ T_u = 176,886(0.918) + 60,000(0.96) \]
\[ = 219,981 \text{ lb} \]

\[ a/2 = 219,981/[0.85(5000)(96)(2)] \]
\[ = 0.27 \text{ in.} \]

\[ M_u = 0.9\left[ 162,381(3 - 0.27) + 57,200(2.5 - 0.27) \right] \]
\[ = 513,771 \text{ in.-lb} \]

\[ M_u \geq M_u \text{ (required)} \]

Therefore the section is satisfactory.

**EXAMPLE 2**

In this design example consider a rectangular beam as shown in Fig. 8. Assume that the beam has a 31-ft span and supports a 10 + 2\( \times \)T load.

The dimensions of the beam are predetermined to satisfy the architectural requirements. The superimposed loads are 10 psf dead load and 30 psf live load. Because of the architectural intent of the beam, large deflections and cracking are not allowed.

**Data**

Beam weight = 300 lb per ft

Superimposed dead load

\[ = 21(38 + 10) \]
\[ = 1008 \text{ lb per ft} \]

Superimposed live load

\[ = 21(30) \]
\[ = 630 \text{ lb per ft} \]

\[ W_u = 1.4(300 + 1008) + 1.7(630) \]
\[ = 2902 \text{ lb per ft} \]

\[ M_u = 1.5(2902)(31)^2 \]
\[ = 4,183,000 \text{ in.-lb} \]

**Conventionally reinforced design**

\[ f_y = 60,000 \text{ psi}; f'_e = 5000 \text{ psi} \]

\[ A_s = 2-\#11 \text{ bars plus 1-\#10 bar} \]

Total steel area = 4.39 in.\(^2\)

\[ d = 21.33 \text{ in.}; b = 12 \text{ in.} \]

\[ M_u = 0.9(4.39)(60,000)(21.33 - 2.58) \]
\[ = 4,444,900 \text{ in.-lb (OK)} \]

See Fig. 9 (bottom) for the reinforcement details of the section.

**Check deflections due to dead load only**

\[ I_g = 12(24)^3/12 = 13,824 \text{ in.}^4 \]

\[ S_b = 12(24)^2/6 = 1152 \text{ in.}^3 \]

\[ M_{cr} = 7.5\sqrt{5000} (1152) \]
\[ = 610,940 \text{ in.-lb} \]

Moment due to dead loads:

\[ M_{dl} = 1.5(1308)(31)^2 \]
\[ = 1,885,482 \text{ in.-lb} \]

\[ E_{ce} = 4,300,000 \text{ psi}; n = 6.75 \]

\[ I_{cr} = 7312 \text{ in.}^3 \]

Using ACI Code Eq. (9-4):

\[ I_e = 7533 \text{ in.}^3 \]

The dead load deflection is:

\[ \delta = 22.5(1308)(31)^4/[4,300,000(7533)] \]
\[ = 0.84 \text{ in.} \]

The long-term deflection could amount to 0.84 + 2(0.84) = 2.52 in.
This amount of deflection could be objectionable in an architectural panel. However, as we shall see below, the technique of post-tensioning offers a solution to alleviating undesirable deflections and cracking in panels.

**Post-tensioned design**

First, determine the eccentricities of the strand in the panel section.

$e_{\text{end}} = 4$ in. (Note that the top stress due to post-tensioning is zero.)

$e_{\text{midspan}} = 12 - 2.25 = 9.75$ in.

**Determine prestress requirements**

Assume that the strands are parabolically draped. Then:

$$\delta_{\text{pre}} = 4(18)P(31)^2/EI + 5.75(15)P(31)^2/EI$$

$$\delta_{\text{dl}} = 22.5(1308)(31)^4/EI$$

Setting the prestressed camber equal to the dead load deflection and solving for the prestress force we obtain $P = 178,719$ lb.

Assume $\frac{1}{2}$-in. diameter strands with each strand having a 25,000-lb force.

Use seven strands amounting to a
total prestress force of 175,000 lb.

Check initial stresses at midspan

\[ f_t, f_b = \frac{202,419}{288} + \frac{202,419(9.75)}{1152} \pm 1.5(300)(31)^2/1152 \]

\[ f_t = -635 \text{ psi}; \quad f_b = 2040 \text{ psi} \]

To resist the high tensile stress at the top of the panel, use two #6 mild reinforcing steel bars \( A_s = 0.88 \text{ in.}^2 \).

See Fig. 9 (top) for details of the reinforcement.

Check final stresses at midspan

\[ f_z, f_b = \frac{175,000}{288} + \frac{175,000(9.75)}{1152} + \frac{(1938)(31)^2}{1152} \]

\[ f_z = 1552 \text{ psi}; \quad f_b = -336 \text{ psi (OK)} \]

Check ultimate strength

\[ f_{se} = 164,000 + 10,000 + 5000 \frac{(12)(21.75)}{(100)(7)(0.153)} \]

\[ = 186,185 \text{ psi} \]

\[ A_s = 0.004(144) = 0.573 \text{ in.}^2 \]

Use two #6 bars \( A_s = 0.88 \text{ in.}^2 \).

**POST-TENSIONED DETAILS**

![Diagram](image)

**ENDS**

**MIDSPAN**

**CONVENTIONALLY REINFORCED**

Fig. 9. Reinforcement details of beam; (top) post-tensioned design; (bottom) mild steel reinforced design.
\[ T_u = 186,185(7)(0.153) + 60,000 \]
\[ = 252,204 \text{ lb} \]
\[ a/2 = 252,204/ \left[ (0.85)(5000)(12)(2) \right] \]
\[ = 2.47 \text{ in.} \]
\[ M_u = 0.9(252,204)(21.75 - 2.47) \]
\[ = 4,376,000 \text{ in.-lb (OK)} \]

The calculation of the post-tensioning force did not take into consideration time-dependent effects which could affect the actual cambers. Many factors could affect the final camber or deflection including such items as initial losses, concrete strength at post-tensioning, creep, shrinkage, length of time after post-tensioning before the superimposed dead loads are applied and many other factors. However, the proposed design method does ensure a reasonably crack free member with a controlled amount of camber or deflection.

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**Cost Comparison**

As of May 1976, the post-tensioning strand and anchorage prices quoted by a major manufacturer, were $0.20 per ft for the strand (½-in. diameter 270-kip), $0.90 for the plastic pocket former, and $3.30 for the strand anchor and wedges (prices are FOB plant).

These prices were obtained from a manufacturer which greases and coats the strand and not a post-tensioning supplier. Material obtained from a supplier could cost more.

The panel in Example 1 would have reinforcing costs of 256 sq ft of 4 x 4-4/4 wwf @ $0.19 per sq ft = $48.64, 215 ft of strand @ $0.22 per ft (which includes approximate shipping costs) = $47.30, and two pockets with strand anchorage and wedges @ $4.50 per anchor = $9.00. The total cost is approximately $141 or $0.55 per sq ft.

It is difficult to make an actual cost comparison with a similar panel reinforced with mild reinforcing steel only. In Example 1 4 x 4-4/4 wwf was used @ $0.19 per sq ft, but depending on the ultimate strength requirements a much lighter mesh could be used costing under $0.10 per sq ft. In a comparable mild steel reinforced panel the handling and shipping would have to be more sophisticated in order to maintain the same stress levels, and this could contribute to the cost.

There is also the consideration of the cost of one panel cracking and being rejected. It is difficult to assign an actual value to these considerations, but the reinforcing costs of from $0.45 to $0.55 per sq ft for the post-tensioned design are reasonable within the total cost for architectural panels costing from $4.00 to $8.00 per sq ft.

With the second design example it is easier to make a cost comparison. Neglecting the small savings in the amount of shear reinforcement that might be realized in the post-tensioned design, the post-tensioned beam would have reinforcing costs of seven strands @ $0.22 per ft = $1.54 per ft and four #6 bars @ $0.23 per ft = $0.92 per ft for a total cost of $2.46 per ft (which does not include the anchorage).

The mild steel reinforced design would cost two #4 bars @ $0.10 per ft = $0.20 per ft, one #10 bar @ $0.65 per ft = $0.65 per ft, and two #11 bars @ $0.80 per ft = $1.60 per ft for a total cost of $2.45 per ft.

The exact cost of the anchorage detail for the post-tensioned beam could vary, and this cost would be the basic difference in cost between the two designs. The additional cost of the anchorage would be a small price to pay compared to the problems which might result from large deflections or cracking in the mild steel reinforced design.
Limitations

The major disadvantage to unbonded post-tensioning is that if a strand is ever cut for any reason all the stressing is lost. In the panel in Example 1 this is very critical because the strand is looped, and the total prestressing force would be lost. The only physical way the strand could be cut is with a core drill. Standard masonry drills will not cut the strand, and the possibility of the strand breaking prematurely due to natural causes is very remote.

If any opening should be desired after the panel has been cast, then it must be located between the strands or the panel in question would have to be recast. In most panel applications every opening whether existing or set for future purposes can be easily located before the panels are cast. Indeed, it would be unusual to request an opening in the field in a location that was not predetermined.

Because strands are not tensioned until after the member has been cast, almost any strand configuration can be accomplished. In Example 1 the strand pattern can be easily modified to allow for window or door openings. In Example 2 a parabolic strand configuration was used without any particular difficulty.

Conclusion

The post-tensioning system illustrated is easily handled by any precast operation, even those not familiar with prestressing procedures. With a minimum of changes from conventional forming procedures, one can obtain a post-tensioned panel with the assurance of a crack-free panel. Yard handling and trucking can be accomplished with simple two-point picks providing the small precasters the ability to make large panels without sophisticated yard handling equipment.

Post-tensioning also allows for the control of deflections in architectural and structural members. Many beams or architectural panels which do not lend themselves to being pretensioned because of difficulties with long line casting, can be easily post-tensioned.

Thus, post-tensioning provides the precaster with an economical method of designing architectural panels and beams for both stresses and deflections. The design procedures follow standard design methods, and the forming procedures remain basically the same as for conventionally reinforced concrete members.

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Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by November 1, 1977.