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POST-TENSIONED T- BEAMS; EFFECTIVE WIDTH; TEMPERATURE TENDONS AND UNDESIRABLE EFFECTS[1]

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This Technical Note discusses the background to the somewhat controversial practice of calculating an “effective width” for the structural design of the post-tensioned flanged beams (T-beams) commonly used in parking structures (Fig. 1). It also discusses the use of “temperature tendons” in the design of these beams.

Some engineers who design post-tensioned beam and one-way slab parking structures (Fig. 1) account for the shrinkage and temperature requirements of the slab through the addition of temperature tendons. Temperature tendons are placed in the slab without profile, parallel to beams. The number of tendons is calculated using the area of the slab outside the effective widths of the adjacent beams. Hence, the number of tendons required can be related to the value assumed for the effective width of the beam flanges.

The Note highlights the undesirable effects of the temperature tendons under both service conditions and with respect to the safety of these beams. It concludes with recommendations for treatment of effective width, as well as shrinkage and temperature design of post-tensioned flanged beams.

The concept and application of “effective width” for post-tensioned flanged beams (T-beams) has been a matter of concern among structural engineers for many years [Chacos 2006].

In an attempt to define the effective width of post-tensioned flanged beams, a PTI Journal article by a pioneer of post-tensioning industry, Greg Chacos, recommended the following, based on a parametric study [Chacos 2006].

> “ 24t + bt, because the numbers come out so nicely for many common cases.”,
> where "t" is the slab thickness, and “b” the beam width.

Chacos' recommendation in the same PTI Journal continues by saying:

> “ … most important rule, in four parts:
> a. Pick a flange width!
> b. Go with it!
> c. Stick with it!
> d. Don’t worry about it!. “

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Up until the 2014 version of ACI 318, the American Concrete Institute (ACI) recognized the wide difference between the distribution of precompression and flexural effects in flanged beams, and warned against using the effective width recommendations intended for reinforced concrete (RC) sections in the design of post-tensioned T-beams [3]. Until the 2014 edition of ACI 318, ACI's opinion on the practice of calculating an effective flange width and using temperature tendons seemed to be limited to commenting that "This amount of prestressing ….. has been successfully used on a large number of projects[4]." ACI's attitude was that based on observation of structures designed and constructed using the code-recommended option of temperature tendons, structural performance was acceptable.

ACI 318-2014, in an apparent attempt to simplify navigation through the code for design of similarly loaded members, recommends in one section that the "effective width" of post-tensioned flanged beams be the same as that of RC members[5]. However, ACI also recognizes that post-tensioned members are subject to both bending and precompression, with widely different responses in force distribution. Thus it allows the entire cross-sectional area of the member to be used for distribution of precompression; in other words, the entire section is assumed to be effective [6].

This confusion with respect to treatment of effective width for prestressed flanged beams in ACI 318-14 requires clarification of the underlying concept.

The fact that the concept of effective width does not apply to post-tensioned flanged beams and the undesirable effects of temperature tendons on both the performance and safety of the structure were known to the pioneers of the field have been reported previously [Walter P. Moore, 2002].

In a recent communication with a leading veteran of post-tensioning industry[7] in connection with the question of the adverse effects of temperature tendons, the comment was "… not seen any reference to, or seen any, actual structure in distress because of temperature tendons."

The argument "It was done before. Nothing happened." can be valid, if we do not know any better. However, once it is known that improved performance, greater safety, and increased economy can be achieved through a different design approach, continuing the original approach is questionable, particularly when the practice means our clients pay more while receiving what could be considered an inferior product. It is now known that temperature tendons do not improve either the serviceability or the safety of the structure – in contrary they have the opposite effect.

[3] ACI 318-11, Section 18.1.3
[4] ACI 318-11 R7.12.3
[5] ACI318-14 6.3.2.1 and 6.3.2.2
[6] ACI 318-14 Sections 7.6.4.
[7] Does not wish his name to be disclosed on this sensitive topic

This Technical Note is intended to:

1 – Explain the concept of "effective width" as used for the conventionally reinforced concrete (RC) T-beams;
2 – Describe why the concept of effective width – as used for RC members – does not apply to post-tensioned flanged beams;
3 – Clarify why, when using modern computational methods the “effective width concept” is unnecessary for design of RC T-beams, and finally.
4 – Why “temperature tendons” commonly placed in the slabs of beam-and-slab construction can lead to undesirable effects in the performance (in-service condition) and safety of the structure.

These issues are addressed in an excerpt from the reference [Aalami, 2014].

FIGURE 1 Post-tensioned beam and slab construction with temperature tendons in the slab between beams. The stressing pockets of three temperature tendons at the edge of the slab are visible.(P355)

Excerpt from the book [Aalami, 2014]

4.8.3 Effective Flange Width of T-Beams
Common understanding and treatment of effective width for flanged beams are rooted on its historical development coupled with the application of simple analysis methods. With progress in structural engineering, and advent of new analysis options, the “effective width” concept is either no longer applicable, or has to be looked at differently. This Section begins with its historical background, and concludes with its proper application in today’s design practice. The review covers the “effective width” to serviceability design (SLS) of post-tensioned members, deflections for both conventionally reinforced and prestressed members, and the requirements for the ultimate strength (ULS) of a member.

4.8.3.1 Effective Width Concept. Historically, the effective width concept was introduced to estimate the bending stress in the flange of a T-beam, using the simple beam theory (Exp 4.8.3.1-1).

$f = (M/I)c$ (Exp 4.8.3.1-1)

Where

c = distance of the fiber at which stress is calculated from the neutral axis;
f = bending stress at distance c from the neutral axis;
I = second moment of area; and
M = applied bending moment.

Consider the simply supported flanged beam under uniform loading shown in Fig. 4.8.3.1-1. Using a two-dimensional linear elastic theory, it is observed that the distribution of compression stress in the flange is tapered as shown in part (c) of the figure. The stress is maximum at the stem-flange interface and tapers off with distance from the stem [Girkman, 1963]. The simple bending formula (Exp 4.8.3.1-1) is based on the assumption that the stress at a distance "*c*" from the neutral axis is constant; hence it cannot capture the reduction of force in flange from the tapered distribution of stress. To correct the stresses calculated using the simple beam formula, the tapered distribution of stress must be substituted by a rectangular stress block that provides the same flange force, and has the same maximum stress as the tapered distribution. The width of the rectangular block of equal force is termed "effective width" of the flange. For the geometry and loading shown in the figure, most references in the literature suggest a value between 8 to 12 times the flange thickness on each side of the stem, when dealing with concrete T-beams. The exact value depends on the parameters of the cross-section used.

Effective width varies with the configuration of the applied load and a beam's boundary conditions. Figure 4.8.3.1-2 illustrates several examples [Girkman, 1963]. Observation of variations in the effective width shown in the figure implies that the application of a fixed multiple of flange width for effective width is not realistic – it is an approximation.

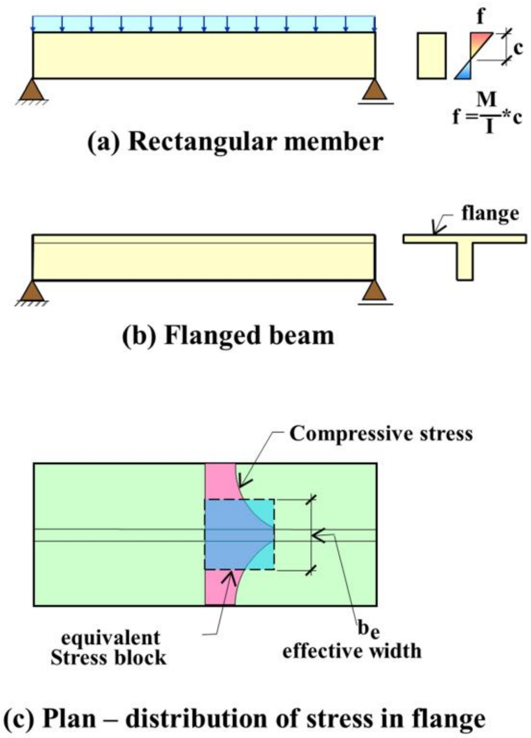


FIGURE 4.8.3.1-1 Illustration of Effective Width for a Simply Supported Flanged Beam under Uniform Loading (P539)

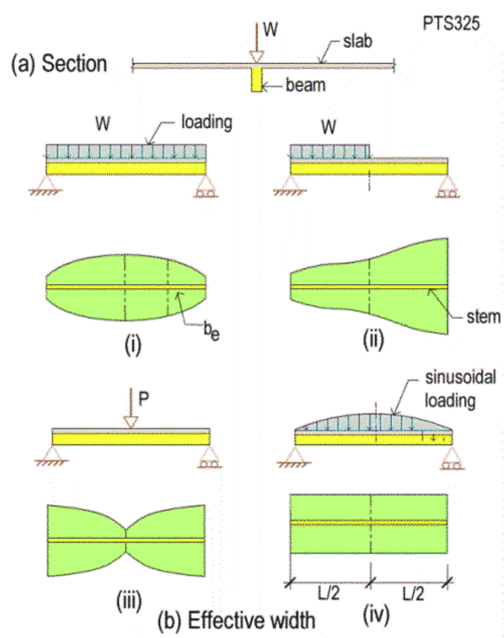


FIGURE 4.8.3.1 -2 Variation of Effective Width (b_e) with Load

A. Effective Width and FEM: A first corollary to the foregoing is that where stresses are calculated using a formulation that captures the non-uniform distribution of stress in the flange of a beam, the effective width concept does not apply, since a correction to the value of the calculated stress is no longer necessary. Properly formulated FEM analysis tools recognize the interaction of a beam stem and its flange, resulting in a tapered distribution of stress in latter. The maximum stress reported where it occurs. Hence, the correction intended for flanged beams by way of an "effective width," is no longer applicable.

B. Allowable Stress Design: In early days, concrete members were designed using the "Allowable Stress" procedure, where the computation of stress was an integral part of design. Coupled with the fact that finite element technology was not available, the "effective width" became a central parameter in design of flanged beams. Its introduction led to the determination of "stresses" necessary in RC design. Today, concrete design is based on factored loads, used to estimate of a member's ultimate strength. The safety of a flanged beam rests on formation of a hinge line across its width, where depending on the configuration, a width as much as the beam's tributary can be mobilized to resist the applied loads.

C. Service Design of Post-Tensioned Members: One consideration in the service design (SLS) of post-tensioned members is the computation of a "hypothetical" stress for crack control. ACI 318[8] explicitly disallows the application of the "effective width," as used for conventionally reinforced sections to be applied to post-tensioned members. For post-tensioned members, the stress at a point is a combination of bending and axial effects. The ratio of bending to axial stress varies from point to point along the same member. Further, axial stresses distribute with a constant value over the entire cross-section, while bending stresses can have a tapered distribution across the flange. Since the ratio of contribution of bending and axial to the total stress at a point is not constant, it is impractical to arrive at a single universally applicable "effective width" for a post-tensioned flanged beam, even where simple beam theory is to be used. Where a designer wishes to use the effective width concept, the bending and axial effects each should enter the analysis with its own effective value. In other words, a single value of "effective width" does not apply to both effects. The following Section explains further.

4.8.3.2 Effective Width for Axial Loads: Post-tensioned tendons apply an axial force on the member they act upon. In addition, they tend to flex the member if the prestressing force is eccentric with respect to the member's centroidal axis. In this Section, we concentrate on the distribution of axial force from post-tensioning. To crystallize the concept, we first consider the example of the familiar column shown in Fig. 4.8.3.2-1. The forces applied at the end of the column are intended to simulate those from tendons anchored at member ends

Let the cross-sectional area of the column be the irregular shape shown in part (a) of the figure. An irregular shape is selected to emphasize that the outcome applies to all cross-sectional shapes. In part (b) of the figure, the column is loaded with an axial force *P* applied at the centroid of the section. The physical property of the centroid is that an axial force applied on it disperses uniformly over the entire section. As illustrated in the figure, the uniform distribution of stress takes place a distance away from the discontinuity at the ends, where the force is applied. It is concluded that the entire section, regardless of the shape of its geometry, is equally stressed to resist the applied load at distances far enough from the point of application of the force.

[8] ACI 318-11, Section 18.1.3; commentary R18.1.3 – Sections 8.12.2, 3 and 4

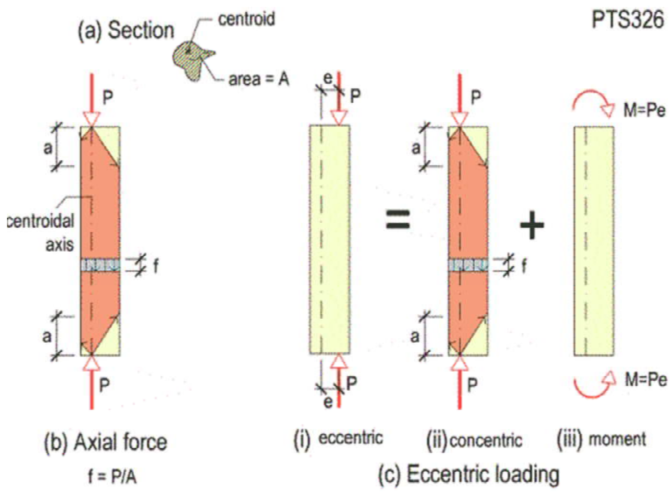


FIGURE 4.8.3.2-1 Dispersion of Force in End-Loaded Member

Consider part (c-i) of the figure, where the axial load is placed with an eccentricity "*e*." This represents the case, where a tendon is anchored eccentric to the centroid of a section. We simply use the common procedure of substituting the applied load by a force at the centroid (part c-ii) and a moment equal to *Pe*. The response of the column to the axial force will again be a uniform distribution of stress at a distance away from the ends. The distribution is identical to part b of the figure. The distribution of stress due to moment, however, can possibly be subject to an "effective width" concept, depending on the formulation we might use to evaluate its effects.

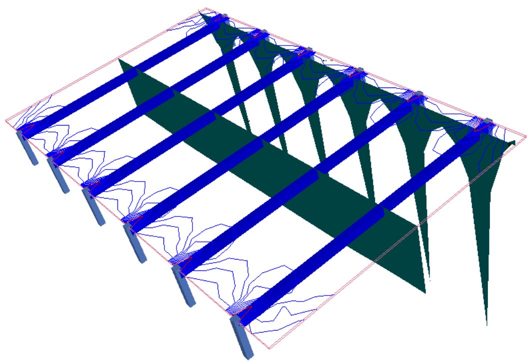
The example illustrates that (i) irrespective of where a tendon is anchored at the ends of a member, and (ii) irrespective of the cross-sectional shape of a member, the axial force from a post-tensioned tendon distributes uniformly over the entire cross-section away from the point of application of the force.

In summary, unlike the case of "bending effects," there is no "effective width" for axial loads, since the entire width of a section becomes mobilized. In post-tensioned members, the local stresses are a combination of bending and axial effects. Since the contribution of each effect is not known a-priori, it is not practical to define a unified "effective width" for the combined stress. As mentioned earlier, each effect has to be considered separately and the outcome combined. This is true, when simple analysis procedures such as SFM or EFM are used. In a properly formulated multi-dimensional FEM analysis, the combined stress is calculated without the necessity to use the "effective width" concept.

A. Precompression in Beam and Slab Construction: Precompression is defined as the average compressive stress over a section. It is the value of the stress at centroid of a section. When dealing with axial load only, precompression will be a uniform constant force over the entire cross-section. The following is a practical application of the conclusion arrived at in the preceding.

Consider a beam and slab construction, such as shown in Fig. 4.8.3.2A-1. In part (a) the slab over the beams is displayed transparent for better visualization. The structure represents the typical geometry common in the post-tensioned parking structures built in the US. Let the tendons be anchored at the end of the beams. Since there is no effective width applicable to axial loads, as illustrated in the foregoing, the distribution of the force in the structure will result in a uniform compressive stress at distances adequately away from the point of application of the axial loads. Part (a) of the figure shows the distribution of precompression at two sections. One section is close to the beam ends, where tendons are anchored, and the second at mid-span.

Near the beam ends where the tendons are anchored, the stress has a peak. At mid-span, the stress is essentially uniform. The same phenomenon is reflected in part (b) of the figure, where the distribution of axial stress in the flange is plotted. The distribution reflects a rapid dispersion of force into the entire cross-sectional area of the structure, leading to uniform distribution across the entire width of the slab.



(a) Distribution of precompression at mid-span and next to support (P320)

(b) Contour of precompression stress (P321)

FIGURE 4.8.3.2A-1 Distribution of Axial Load through a Beam and Slab Construction

Note that the stresses shown in the figures apply to both the slab and the beam. The distribution of axial stress is uniform through the depth and width of the structure. The distribution is uniform, irrespective of the number of spans, shape of the cross-section, or profile of the tendons.

Example 4.8.3.2A-1

Figure 4.8.3.2A-2 illustrates a flanged beam with a concentric straight tendon. The distribution of precompression will be uniform and equal to force (P) divided by the total cross-sectional area (A) as shown in Fig. 4.8.3.2A-2.

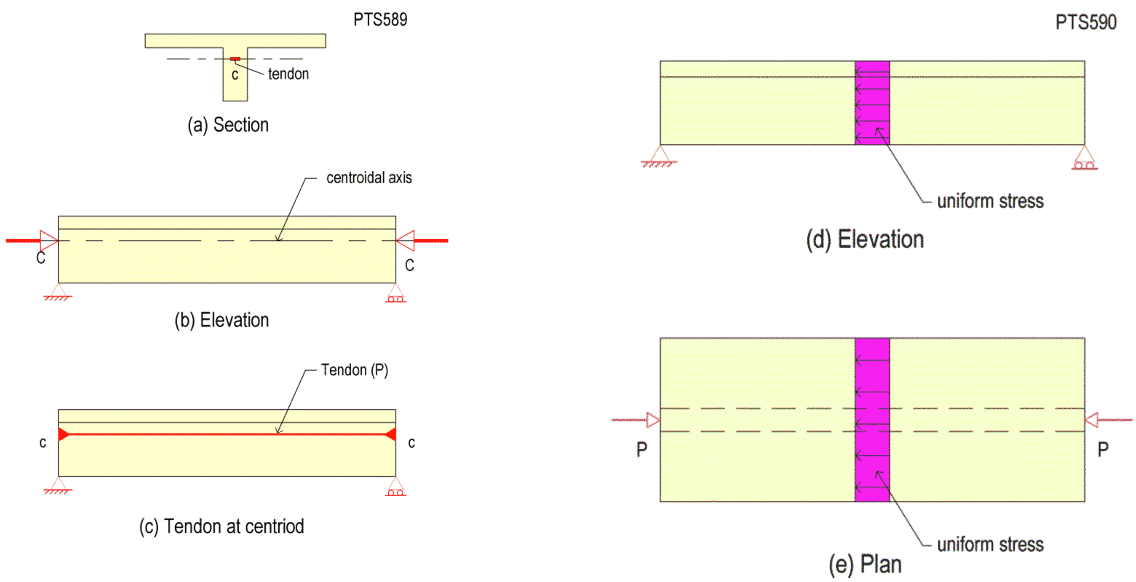


FIGURE 4.8.3.2A-2 Flanged Beam with Eccentric Tendon – Average Precompression

Example 4.8.3.2A-2
Figure 4.8.3.2A-3 shows two beams, featuring eccentric and profiled post-tensioning tendons. One of the beams is multi-span. For each of the beams the distribution of precompression will be uniform and equal to force (P) divided by the total cross-sectional area (A) as shown in Fig. 4.8.3.2A-2 parts (c) and (d).

The above assumes that there is no friction loss along the length of the tendons. Friction loss will result in drop of the compressive stress along a member, but does not invalidate the concept. Also, neither the eccentricity of tendon at the anchorage, nor the profiling of tendon along the member length changes the conclusion regarding uniform distribution of precompression over a member's entire cross-sectional area.

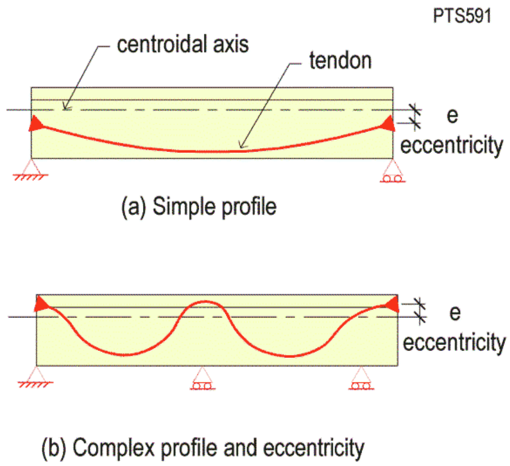


FIGURE 4.8.3.2A-3 Post-Tensioned Members with Arbitrary Tendon Profiles

B. Position of Tendon Anchorage: When designing flanged beams, unless you plan to account for special effects, such as applying moments at member ends to improve the performance of a flanged beam, it is recommended to anchor a tendon at the centroid of the tendon's tributary as shown in part (a) of Fig. 4.8.3.2B-1.

When using "simple analysis procedures," such as SFM, or EFM, the section shown in part (a) of the figure resists the axial force of tendon, while for bending effects the section with reduced width (part b) is used. It is reiterated that when using a properly formulated FEM tool, the selection of effective width and the differentiation between the two effects are no longer applicable.

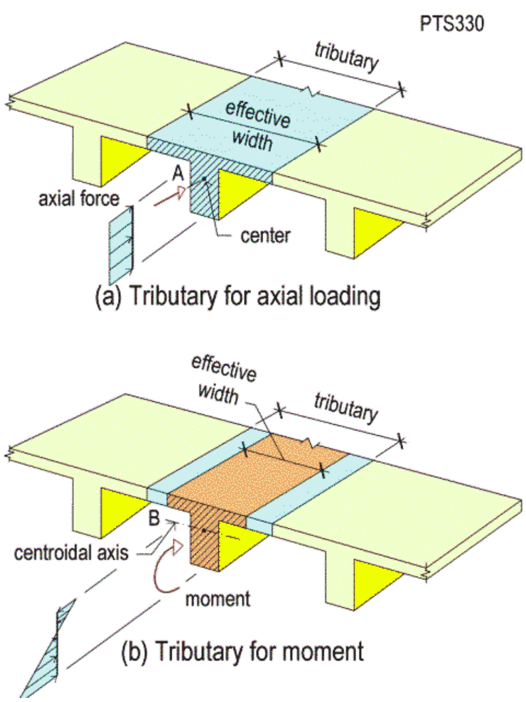


FIGURE 4.8.3.2B-1 Tributaries for Axial and Flexural Actions for Frame Analysis

4.8.3.3 Effective Width for Strength Design: To illustrate the concept, let us review the strength design of a single span simply supported flanged beam using finite elements, where flanged beams are properly modeled as they appear in their physical outline. The stem is offset with respect to the flange and interacts with it through strain compatibility at the interface of the two.

Figure 4.8.3.3-1 shows the geometry of a flanged beam, typical of dimensions used in parking structures in the US, where a parking deck is constructed with parallel beams and one-way slab spanning across the beams.

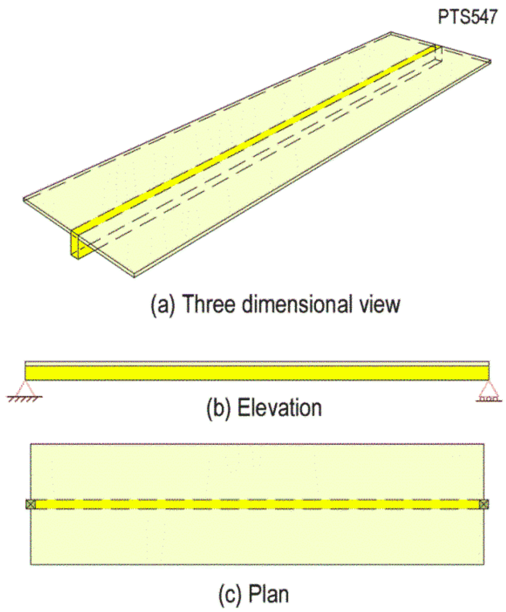


FIGURE 4.8.3.3-1 Simply Supported Flanged Beam under Uniform Loading

The geometry and design parameters of the flanged beam are summarized below. The beam is not post-tensioned for simplicity of demonstration, without compromising the principle being demonstrated.

Geometry
- ❖ Span = 62 ft (18.90 m)
- ❖ Beam = 14 inch wide, 30 inch deep (356x762 mm)
- ❖ Flange = 5 inch (127 mm) thick ; 17 ft (5.182 m) wide

Load
- ❖ Uniform factored load of 1 k/ft (14.6 kN/m) along the centerline of the beam. The load on the actual structure will be somewhat different. The assumed load is to illustrate the concept.

Material
- ❖ Concrete strength f'c = 4000 psi (27 MPa)
- ❖ Reinforcement fy = 60 ksi (413 MPa)

Design Criteria
- ❖ Centroid of reinforcement to beam soffit = 2.30 inch (58 mm)
- ❖ Design code = ACI 318 -11

From the loading and span information, the design values at the beam's midspan are:

- Moment = $(1 \times 62^2 / 8)$ = 480.50 k-ft (6501.46 kNm)
- Shear = 0 k
- Axial load = 0 k

Considering the moment resisted by the entire cross-section of the flanged beam, the reinforcement required for the applied moment 480.50 k-ft is 3.95 in2 (651.46 kNm ; 2,548 mm2).

Next, we calculate the reinforcement in the beam using several different options, each with a different assumed effective width. The objective is to demonstrate that when properly formulated, the outcome of reinforcement design is independent of a designer's choice of the width of the flange that acts with the stem to resist the applied loads – namely, "effective width." In FEM –based designs, the "design strip" typically includes the beam stem and a strip of flange acting with it, when calculating the amount of reinforcement. The objective of the examples is to demonstrate that the width of the strip does not influence the amount of reinforcement needed for safety of the structure.

We select three different scenarios to illustrate the point (Fig.4.8.3.3-2). In part (a) we select a design section that includes the entire flange; part (b) illustrates an option with a design section that considers only one-half of the flange to act with the stem in resisting the applied load. And, finally, in part (c), we assume that the entire load is carried only by the stem of the flanged beam – the design section includes the stem only.

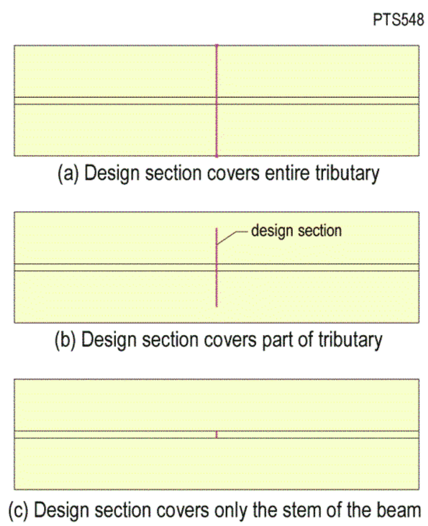


FIGURE.4.8.3.3-2 Plan of Flanged Beam Showing Different Design Section Widths

Table 4.8.3.3-1 reports the design actions generated for each of the design sections, using the proper modeling of the beam as illustrated in Fig.4.6.6-B-1 and a proper finite element formulation. Note that – as expected - the values of bending moment and axial forces reported vary, depending on the amount of the force from the flange that is captured in each design section.

The reinforcement calculated for each of the three design sections is reported in Table 4.8.3.3-2.
The table indicates that the reinforcement reported for the beam stem is essentially the same in all three cases. The variation is about 2.5% for the two extreme cases of full tributary and zero tributary. This is well

within the limits of engineering approximation, when considering the two extreme cases of zero and full effective flange inclusion. The small difference is due to the inaccuracies in the numerical computations, rather than the concept.

The recognition of presence, and allowance for the axial force in the beam stem that in each instance balances the compression in the flange tributary to the stem, make the determination of reinforcement for safety of the structure independent of an "assumed flange width." This conclusion is valid for both conventionally reinforced and prestressed members. It clearly demonstrates that using modern analysis tools, the application of "effective width" is redundant. Where a FEM design tool requires the definition of "effective width" to compute the design moments and the associated reinforcement, it signals clear evidence in the underlying modeling of the beams and the computational formulations.

TABLE 4.8.3.3-1 – Moments, Axial Force, and Shear at Selected Design Sections (T151)

Design section	Moment	Shear	Axial
	k-ft (kNm)	K (kN)	K (kN)
1 – Full section	480.504 (651.467)	-0.235 (1.045)	-0.005 (0.022)
2 – Partial section	448.203 (607.674)	-0.234 (1.041)	70.526 (313.714)
3 – Beam stem only	214.173 (290.376)	-0.230 (1.023)	243.882 (1084.836)

TABLE 4.8.3.3-2 – Required Reinforcement for Strength (T152)

Design section	*As* top	*As* bottom
	in^2 (mm^2)	in^2 (mm^2)
1 – Full section	0.00	3.95 (2548)
2 – Partial section	0.00	3.86 (2490)
3 – Beam stem only	0.00	4.06 (2619)

The conclusions arrived at is general and apply to multiple beams too. Consider the partial view of a beam and slab construction shown in Fig. 4.8.3.3-3. The beams have the same dimensions and loading as in the previous example. In the following three design sections are selected: one is a section across the entire structure (4.8.3.3-4a); the next shows a section that contains one beam stem and include a greater flange than the tributary of the stem; finally the last (part c) is the stem with a smaller flange than its associated tributary. The objective of the example is to illustrate that in all three cases, the total reinforcement necessary for the safety of the structure is the same, recognizing that at ultimate strength, a hinge forms across the center of the entire structure folding it down.

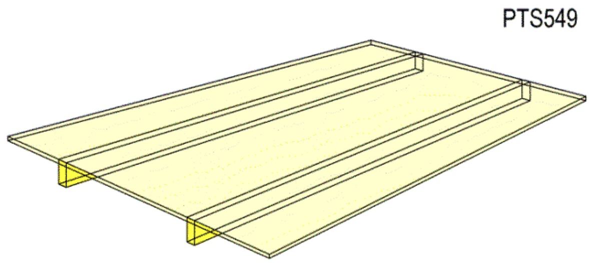


FIGURE 4.8.3.3-3 Beam and One-Way Slab Floor System

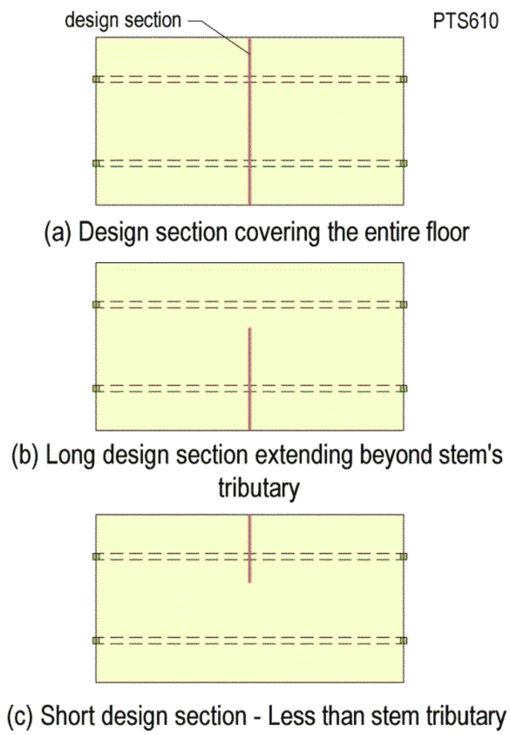


FIGURE 4.8.3.3-4 Design Sections of Different Widths

The bending moments and the associated axial forces calculated for each of the three design sections selected are listed in Table 4.8.3.3-3. There will be no net axial force for the full section, since the structure is supported on rollers. For the other two sections, the tension reported in one (51.192 k) balances the compression reported in the other. If the sections were each covering the natural tributaries of the beams (mid-distance between the stems), the compression developed in the flange of each unit would have been equal to the tension in the stem. No net axial force would have been developed. Also, note that the sum of moments of the two sections across the width of the entire structure adds up to the moment of the design section that covers the entire width of the structure, namely M = 956.055 k-ft.

TABLE 4.8.3.3-3 – Resultant of Actions for Different Design Section Widths (T153)

Design section	Moment	Axial
	k-ft (kNm)	k (kN)
1 – Full section	960.994 (1302.92)	0.019 (0.08)
2 – Partial large	497.345 (674.30)	-51.173 (227.63)
3 – Partial small	458.710 (621.92)	51.192 (227.71)

The reinforcement obtained for each of the three design sections is listed in Table 4.8.3.3-4. The values of the table illustrate that the total reinforcement for the structure (7.58 in^2; 4,890 mm^2) is provided by the sum of the partial designs in the latter two cases. Since a properly formulated FEM analysis determines and accounts for the continuity of the beam stem and the slab, as well as the presence of the axial force in each part of the design section, the reinforcement calculated becomes less sensitive to the width of the design section. More importantly, the total reinforcement reported for the entire width of the structure remains essentially constant, irrespective of the width of the design sections that add up to make the entire width. Again, similar to the moment sum, the total reinforcement across the width of the structure consisting of the two design sections adds up to 7.68 in^2 (4,955 mm^2), which agrees well with the single design section over the entire width of the slab (Table 4.8.3.3-4; As bot = 7.68 in^2; 4,955 mm^2)

TABLE 4.8.3.3-4 REINFORCEMENT REQUIREMENT (T154)

Design section	*As* top	*As* bottom
	in^2	in^2
1 – Full section	0.00	7.58 (4890)
2 – Partial large	0.00	3.94 (2542)
3 – Partial small	0.00	3.74 (2413)

Sum of rebar of partial sections 7.68 in^2, (4954)

The slight difference between the distributions of reinforcement among the two partial designs is due to numerical approximations. The difference is within the limits of engineering computations, and is not of design significance, since the hinge line developed at ultimate state extends across the entire width of the structure, thus mobilizing the reinforcement in each of the two beam stems.

4.8.4 Judicious Placing of Tendons

A non-prestressed reinforcement bar is mobilized on demand. When concrete that contains the bar is stretched or compressed, the bar encased in concrete becomes stressed and resists the applied action. Since non-prestressed rebar develops resistance to an applied force, its presence in concrete is generally beneficial.[9] Engineers refer to non-prestressed bars as passive reinforcement. The bars react and oppose the deformation of the concrete that surrounds them.

[9] Excessive reinforcement reduces ductility, a topic not covered

The same does not apply to prestressing. A prestressed bar or strand applies a force to the member that contains it, irrespective of the member's demand – a prestressed bar is active. If a prestressed bar or strand is not positioned favorably in a member, its prestressing force can counteract the resistance that a member has to develop in response to an externally applied load. Many engineers consider the prestressing force as an applied load similar to dead and live loading, and position the prestressing such that the force it provides benefits the design objective.

In the following we review a simple example to illustrates how the positioning of prestressing at an unfavorable location can harm the design-intended performance of a structure, whereas addition of nonprestressed reinforcement at the same location would improve it. It is intended to highlight the concept that prestressing is not always beneficial. Its application requires engineering evaluation.

Consider Fig.4.8.4-1. It shows a simply supported reinforced concrete member. In one case, the member is provided with two conventional non-prestressed bars (2 #9; 1290 mm^2) at the bottom to resist the design load. In the alternative case, four 0.5" (13 mm) post-tensioning strands at the bottom are used. The objective is to review the impact of added top rebar (case 1) or added top prestressing strands (case 2) on the serviceability and safety of the member.

Figure 4.8.4-2 illustrates the outcome. In the case of non-prestressed conventional reinforcement, the addition of two top bars reduces the downward deflection; slightly reduces the potential of cracking at the bottom of the beam; and slightly increases its design moment capacity. In the same figure, the addition of the four post-tensioning strands at the top (i) reduces the safety of the beam by lowering its design capacity, (ii) makes the member develop cracks at a lower level of externally applied loads, and (iii) increases the downward deflection of the member – in summary addition of tendons in this instance is detrimental in both the in-service performance and the safety reserve of the member.[10]

[10] The leading parameters of the example are: f'_c = 4000 psi (28 MPa). Grouted tendons were assumed. Loss of capacity for unbonded tendons will be more:

Design capacity without prestressing 313.20 k-ft (424.64 kNm)
Design capacity with prestressing 304.54 k-ft (412.90 kNm)

The bottom fiber of the beam with added top prestressing will crack at a stress 357 psi (2.46 MPa) earlier than if it did not have the added PT.

For a beam span of 62 ft (18.90 m) and cross section given in the figure, the added tendons at the top increase the long-term deflection of the beam by 2.19 in. (56 mm)

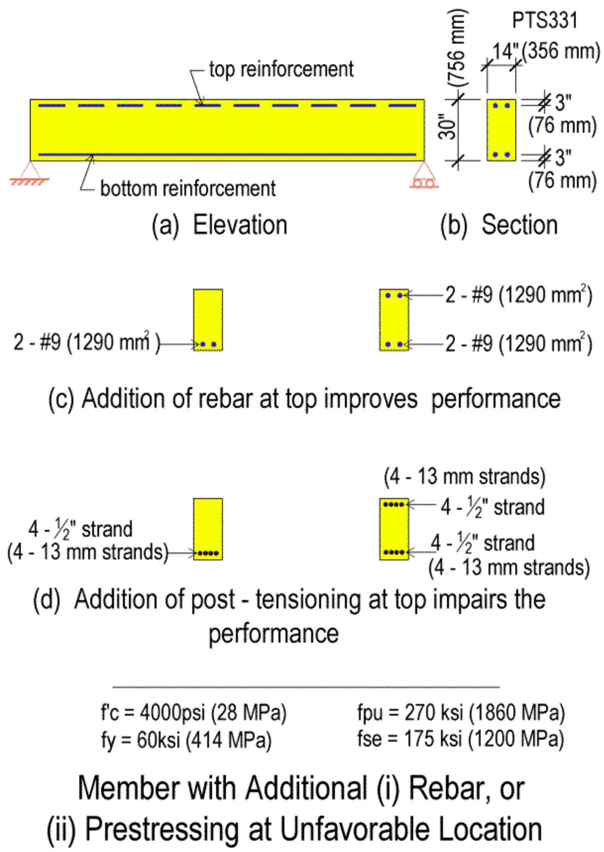


FIGURE 4.8.4-1 Member with Additional (i) Rebar, or (ii) Prestressing at Unfavorable Location

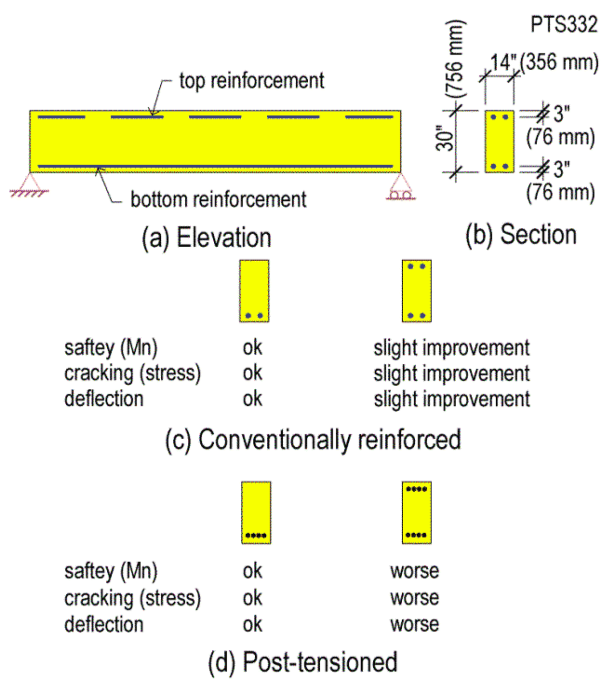


FIGURE 4.8.4-2 Performance of Member with Additional (i) Rebar, or (ii) Prestressing at Unfavorable Location

Consider the flanged beam shown in Fig. 4.8.4-3 – typical of beam and one-way slab construction used in post-tensioned parking structures in the US. Tendons positioned in the flange fall above the neutral axis of the beam, when the structure develops a collapsible mechanism at its ultimate strength[11]. The overall impact of the added strands in the flange is the same as illustrated in the previous example. The flange strands reduce the design strength; increase the deflection and promote cracking at the bottom of the beam stem as illustrated in the preceding example.

At incipient ultimate strength failure, the flange tendons counteract the tensile force developed by the tendons at the bottom of the stem (Fig. 4.8.4-4). This leads to a reduction of the available bottom tension to resist the design moment – hence a reduction in beam's design capacity. More reinforcement has to be added at the bottom of the beam stems to compensate tension from the tendons in the flange, and restore the member's design capacity.

In summary, for flanged beams, such as this example, tendons in the flange and parallel to the beam stem harm the in-service performance of the slab-beam combination and reduce its safety[12].

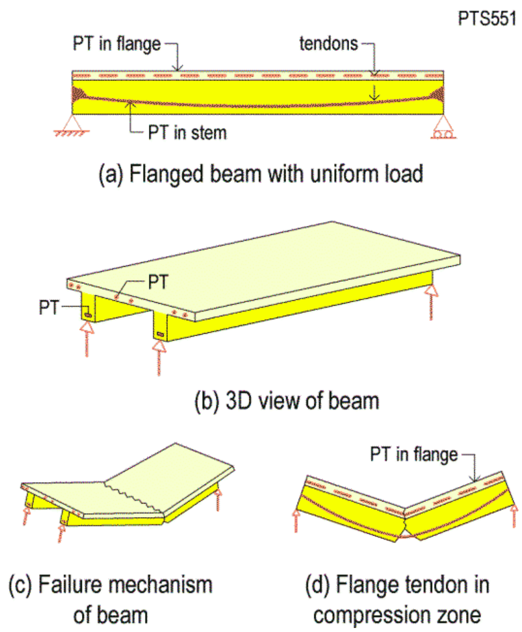


FIGURE 4.8.4-3 Flanged Beam with Tendon in Stem and Flange

Slab tendons between beam stems, as illustrated in Fig. 4.8.4-4 are referred to as "temperature" reinforcement and practiced by a number of designers, with the erroneous perception that the tendons compensate the lack of precompression in a narrow strip of slab midway between the beam stems. Section 4.8.3.2 provides additional background.

[11] BS 5400; part 4; 5.3.1.2 clarifies that at ultimate limit state the entire flange between the stems acts with the beam stem, as shown in the figure.

[12] The practice is not uncommon in the US, due to the strong lobby of the post-tensioning hardware suppliers in the professional and code organizations.

FIGURE 4.8.4-4 Disadvantageous Slab Tendons between Post-Tensioned Beams (P326)

Another scenario, where in common construction positioning of post-tensioning tendons acts unfavorably, is where a slab, or beam acts with its column support to resist lateral forces from wind or earthquake, and the forces from the lateral actions result in reversal of moments at the joint (Fig. 4.8.4-5). If tendons fall in the compression zone, a larger amount of non-prestressed reinforcement is necessary in the tensile zone to compensate the adverse effects of the tendons for strength design. Where, reversal of moments at a post-tensioned joint is probable, a more efficient design will be achieved by placing the tendons closer to the centroid of section – farther away from the anticipated compression zone. This will reduce the tendons' adverse effects at reversal of moments.

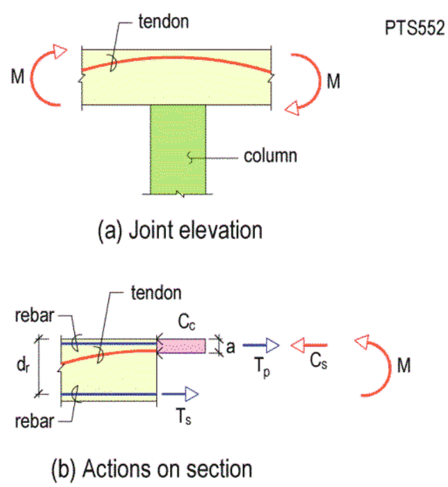


FIGURE 4.8.4-5 Force Distribution in Section with Tendon in Compression Zone

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