pti Technical Notes

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Strength Evaluation of Existing Post-Tensioned Beams and Slabs

Analytical Approach

By Bijan O. Aalami[1]

1 - INTRODUCTION

Occasionally, it becomes necessary to determine the strength capacity of an existing design, or structure. Unlike a new design, in the evaluation process, the geometry, the material properties and the reinforcement of the structure are given information. The engineer is expected to determine, either the code-permissible strength capacity of the as-is structure, or the ability of the structure to sustain a specified loading. Such a scenario is not uncommon, when due to changes in the function of a facility, its members become subject to new loading; or, when partial loss in prestressing tendons in an existing building necessitates a strength evaluation - and possibly a remediation program.

A primary step in the strength evaluation is the analytical approach. In the analytical approach the code-permissible capacity of a structure is computed using the as-is parameters of its members. An analytical approach is possible, if all the parameters necessary for its execution are known. These are: geometry, support conditions, material properties and reinforcement.

Should an analytical approach lead to inconclusive results, or not be practical, due to lack of critical information, a load test may be carried out. Although a load test is more cumbersome and time consuming, but it will yield conclusive results.

This Technical Note describes an analytical procedure for the strength evaluation of existing post-tensioned members. It starts with the traditional elastic solutions, and proceeds to take full advantage of the limited post-elastic behavior permitted by code. The permissible post-elastic strength reserve is utilized through the redistribution of elastically calculated moments. This redistribution procedure of moments selected, reveals maximum member strength for specified loading. ACI [ACI, 1992] is used as the code vehicle to describe the procedure, but the method is equally applicable to the Canadian code [CSA, 1984], or the British code [BS, 1985].

An outline of the procedure is first presented. The outline is followed by a numerical example. The focus of the numerical example is on the post-elastic redistribution of demand moments to meet the available capacity.

2 - ANALYTICAL PROCEDURE

As member strength is approached, inelastic behavior at some sections can result in a redistribution of moments in prestressed concrete members. Recognition of this behavior can be advantageous in design under certain circumstances[ACI-318, 1992; section 18.10.4]. The amount of redistribution allowed depends on the ability of the critical sections to deform inelastically by sufficient amounts. This ability is gauged by the amount and property of the existing reinforcement in the section. To this end, ACI and other codes propose simple methods permitting adjustment of elastically calculated moments. The amount of adjustment must be kept within predetermined safe limits stated in the codes.

The limited plastification is envisaged to occur at the face-of-support, and lead to an adjustment in span moment. Fig 2-1 illustrates the face-of-support regions, where limited plastification is permitted to occur. Observe that in Fig. 2-1b, where the member is on simple sup-

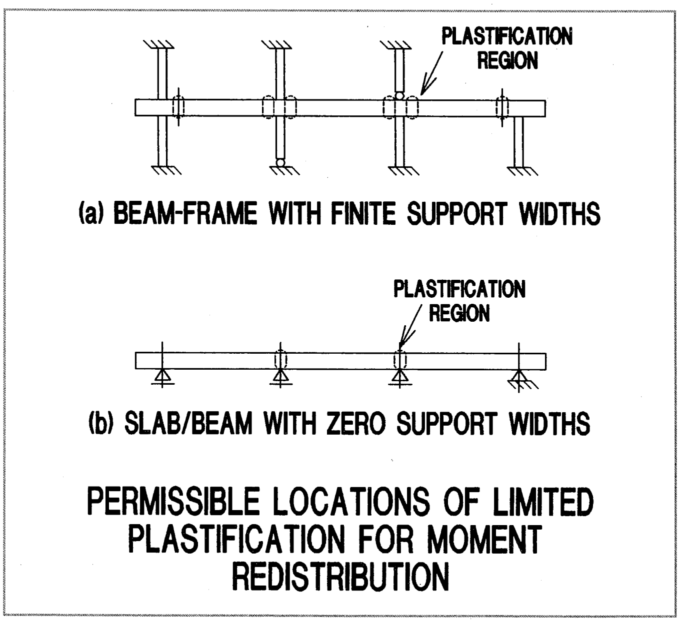

FIGURE 2-1

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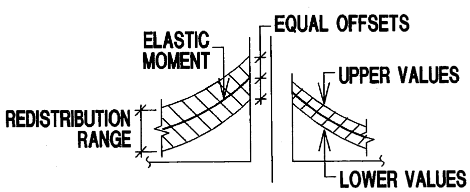
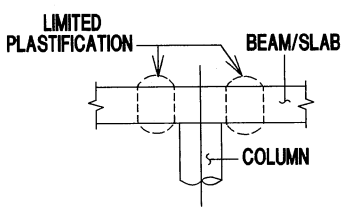
ports, with hypothetically zero support width, only one hinge over each support will be considered. There is no hinge formation at the end supports.

First, the elastic moments are calculated; then, at each potential plastic hinge location (Fig. 2-1), the calculated elastic moment is permitted to be either decreased or increased by a given percentage (Fig. 2-2a). This results in a range within which a support moment can be selected. The range results in a moment envelope. The amount by which a moment will be adjusted within its permissible envelope need not, generally, be the same at both faces of a support. For example, in Fig. 2-3, a support is illustrated at which the moments at the left and the right of the support are selected such as to yield the lowest equal moments on the two faces of the support. It is apparent that, the amount of redistribution applied to the two sides is different. Once an adjustment selection is made, from statics of each span, the remainder of moments in each span must be modified to reflect the changes of moments at supports.



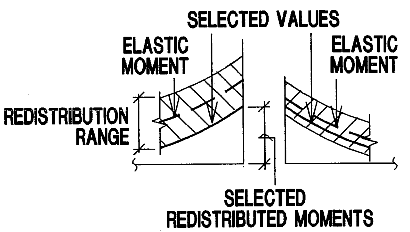
(a) SUPPORT MOMENTS AND ENVELOPE



(b) REGION OF LIMITED PLASTIFICATION AT SUPPORT

NEGATIVE MOMENTS AT SUPPORTS AND THE PERMISSIBLE RANGE OF THEIR DISTRIBUTION

FIGURE 2-2



REDISTRIBUTION SELECTED FOR LOWEST EQUAL MOMENTS AT EACH SIDE OF SUPPORT

FIGURE 2-3

In the evaluation of the available capacity of a member, and its comparison with the demand moment, the redistribution will be aimed at bringing the demand within the available capacity profile. Details of the procedure are described next.

2.1 Determine the factored elastic moments (demand moments)

Using the prescribed values of loading, the existing prestressing and geometry, together with an elastic analysis based on the gross cross-sectional area, determine the moments generated by dead loading, live loading, and hyperstatic (secondary) moments caused by the prestressing. In calculating the prestressing force, use the number of strands available and allow for the immediate (friction and seating of wedges) and long-term stress losses (creep, shrinkage, and relaxation of prestressing). In critical situations, where a member's strength must be evaluated with greater accuracy, use the variable force method to determine the prestressing force at a given section, as opposed to the effective force scheme. The variable force method takes into account the change of prestressing force along the member due to the actual stress losses in the tendons at a given location. In most existing structures, measured concrete strength exceeds the nominal strength at 28 days (f'_c) generally used in the initial design. For the evaluation, use concrete strength associated with the age of the member being analyzed.

Formulate the demand moments using the factored load combinations stipulated in the code. For ACI [ACI, 1992], the demand moment, M_u is given by:

$$M_u = 1.4M_d + 1.7M_\ell + M_{hp} \qquad (2\text{-}1)$$

Where M_d, M_ℓ and M_{hp} are moments due to dead loading, live loading and hyperstatic (secondary) actions of prestressing respectively.

2.2 - Determine the Available Capacity (ϕM_n)

The available strength capacity (ϕM_n) at a section is defined as the nominal strength (M_n) of the section reduced by the strength reduction factor (ϕ=0.90). Using the existing geometry and concrete properties, the prestressing, and the *nonprestressed reinforcement*, determine the available capacity of the member along its length. The computation of the available capacity of a section is an analysis procedure, independent of the applied loading and support conditions. It can be readily computed using the code relationships and long-hand calculation, or through using special purpose software [PULT, 1994].

Where the geometry of a member and its loading are regular and uniform, or nearly uniform, the capacities at only three locations are usually computed. These are: the face of the two supports and at mid-span. The three points are assumed to adequately represent the critical locations along a span. Where conditions deviate from uniform, the critical locations in the span must first be determined.

2.3 Compare Demand Moments (M_u) with the Available Capacity (ϕM_n)

If the available capacities at the critical locations exceed the demand moment computed in Eq. 2.1, the member is adequate. No further computation is needed. The following applies.

$$\phi M_n > M_u \qquad \text{OK} \qquad (2\text{-}2)$$

But if, at one or more locations, the available capacity is less than the computed demand as given below, further computation is necessary to finalize the evaluation.

$$\phi M_n < M_u \qquad \text{NG (No Good)} \qquad (2\text{-}3)$$

2.4 - Compute the Permissible Percentage of Redistribution at the Face-of-support.

Where specified ductility provisions, as required by code and stated herein [ACI, 1992; section 18.10.4] are satisfied, it is permissible to increase, or decrease, the computed elastic demand of a section (M_u) by a percentage, determined from the following relationship (2-4), but not more than 20% .

$$20\{1 - [\omega_p + (d_r/d_p)(\omega - \omega')]/0.36\beta_1\} \text{ percent} \qquad (2\text{-}4)$$

The redistribution of moments at a given section is permitted, only where the minimum code specified bonded reinforcement is available. Further, redistribution of negative moments shall be made only where the applicable reinforcing index does not exceed $0.24\beta_1$.

(i) For prestressed rectangular sections with nonprestressed reinforcement

$$\omega_p + (d_r/d_p)(\omega - \omega') < 0.24\beta_1 \qquad (2\text{-}5)$$

(ii) For prestressed flanged sections:

$$\omega_{pw} + (d_r/d_p)(\omega_w - \omega'_w) < 0.24\beta_1 \qquad (2\text{-}6)$$

The Canadian and the British codes achieve this objective by limiting the depth of the neutral axis.

2.5 - Construct the Adjusted Demand Envelope

Using the permitted percentages of the permitted redistribution of moments computed in the preceding, together with the computed values of the elastic demand moments, construct an adjusted demand envelope. A template, such as illustrated in Fig. 2.5-1 can be useful. On this template, the hatched area is the adjusted demand with allowance for limited plastification.

2.6 - Compare the Available Capacity with the Adjusted Demand Envelope

Enter the computed available capacities (ϕM_n) on the adjusted demand envelope (Fig. 2.6-1).

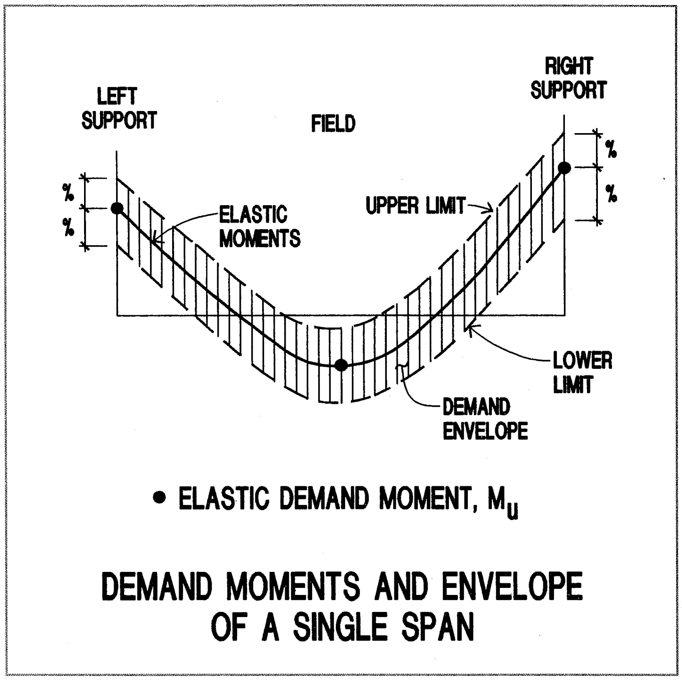


FIGURE 2-5.1

(i) If any of the computed capacity moments falls outside, and short of, the adjusted demand envelope, such as the point at the right support shown in Fig. 2.6-1a, the associated span is deemed inadequate.

(ii) If, on the other hand, all the available capacity values (ϕM_n) fall within the adjusted demand envelope (Fig. 2.6-1b), it is likely that the

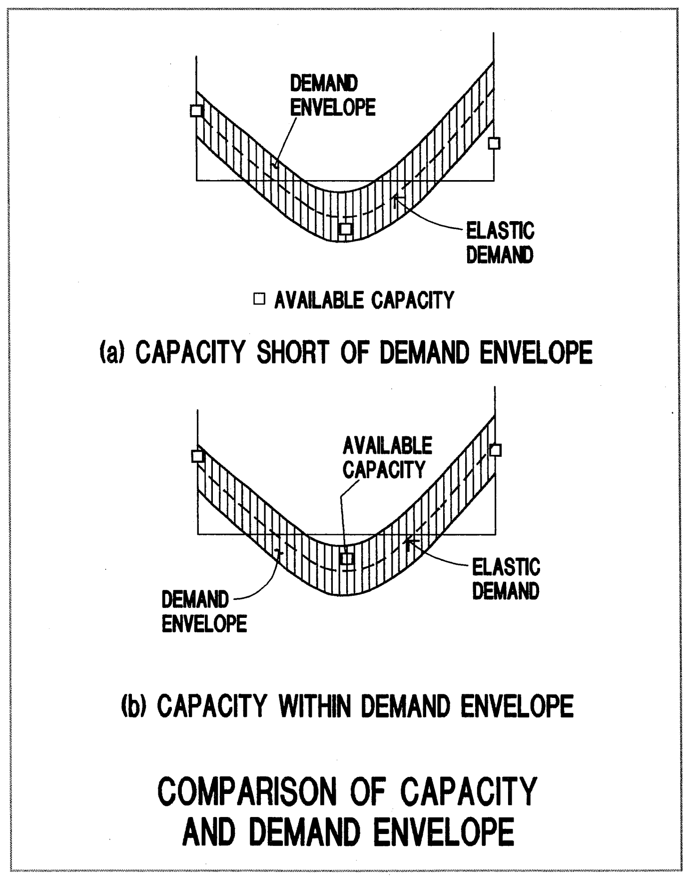


FIGURE 2.6-1

span has the code required adequate strength. Proceed to construct the adjusted capacity profile for conclusion.

2.7 - Construct the Adjusted Demand Profile

Refer to Fig. 2.7-1, where the available capacity values are entered on the demand envelope. For the purpose of the illustration of the procedure, two capacity values, noted as A and B are entered at midspan. In a real condition, only one capacity point will exist at each section. Through comparison of the elastic demand moments and the available capacities at the supports, an adjusted demand profile is constructed in the following manner:

(i) Determine the difference between the capacity (ϕM_n) and demand (M_u) at each support, indicated as a and b for the left and right supports in Fig. 2.7-1. If the available capacity, at a support, is in excess of the upper bound value of the envelope (point D at left support), then reduce the difference (a) to the value DE.

(ii) Determine the adjustment at midspan for the adjusted demand profile. The adjusted demand profile at midspan differs from the calculated elastic moment at that point, by an amount c, necessitated through the adjustments a and b made at the left and the right supports to the elastic moment. The adjustment c, is given by:

Adjustment c = 0.5(sum of adjustments at the supports)

Depending on the sign of adjustments made at the supports, the midspan adjustment will be positive, or negative. The coefficient, 0.5, is applicable to midspan moment. For other points, determine the corresponding coefficient from the statics of the span.

The adjusted demand profile is obtained by joining the capacities at the supports to the adjusted capacity at center (Fig. 2.7-1).

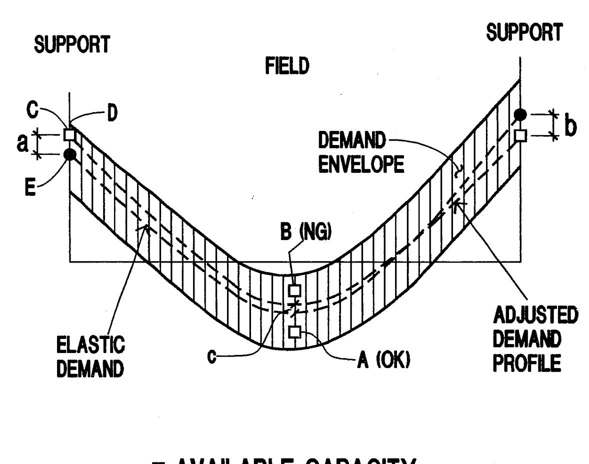


FIGURE 2.7-1

2.8 - Compare the Adjusted Midspan Demand with the Available Capacity

Refer to Fig. 2.7-1

(i) If the computed available capacity (ϕM_n) at midspan is greater than the adjusted demand at that point, such as indicated by point, A, in the figure, the span is adequate.

(ii) If the computed available capacity (ϕM_n) at midspan is less than the adjusted demand profile, such as point, B, shown in the figure, the span is not adequate.

3 - NUMERICAL EXAMPLE

3.1 Purpose

The objective of the numerical example is to determine whether the code required strength of a given post-tensioned frame, with known geometry and reinforcement, is adequate for specified loading.

3.2 Given

The frame shown in Fig. 3.2-1 is a typical interior slab-frame of a multi-level building. The columns in direction perpendicular to the frame are spaced at 28.5 ft on center. The figure illustrates a typical slab-frame level, bounded by the upper and lower columns. The available reinforcement in the slab-frame is shown in Fig. 3.2-1b. Other parameters are as follows:

Material:

Concrete

f'_c = 5,800 psi (at time of analysis)

Nonprestressed steel

f_y = 60 ksi

Prestressing

System : unbonded

f_{pu} = 270 ksi; low relaxation strands

Area of each strand = 0.153 sq in.

f_{se} = 175 ksi

Stress loss parameters

K = 0.0014 per ft; wobble friction

μ = 0.07 per radian; angular friction

Relative humidity 80%

Seating loss 0.25 in.

volume to surface ratio = 3.75 in.

Tendon stressed at day 5

E_{ci} = 2900 ksi at stressing

E_c = 4340 ksi at time of investigation

f_{si} = 0.80 x 270 = 216 ksi

Tendons stressed at both ends

Tendon geometry: see Fig. 3.2-1b

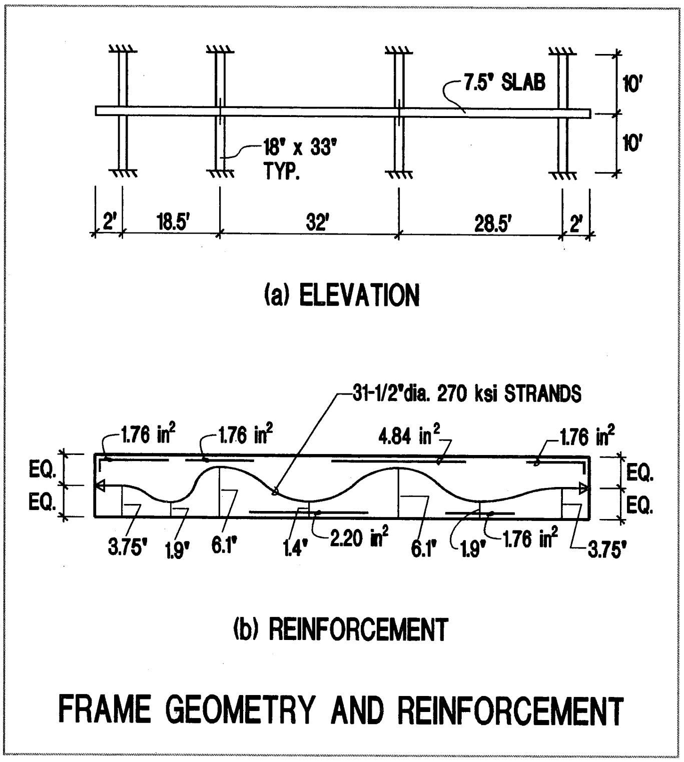


FIGURE 3.2-1

Geometry:
  Frame geometry (see Fig. 3.2-1a)
    Tributary = 28.5 ft
    Columns: 18 in. in direction of frame
      33 in. perpendicular to frame
  Tendon geometry
    Profile per Fig. 3.2-1b
  Nonprestressed steel cover
    Distance of tension steel centroid to concrete surface = 1.63 in.

Reinforcement:
  Prestressing
    31-0.5 in. 270 ksi strands
  Nonprestressed reinforcement
    Disposition as shown in Fig. 3.2-1b

Boundary Conditions:
  Columns are assumed fixed at top and bottom
Loading:
  Dead load : 130 psf, assumed uniform
  Live load : 50 psf (skipped with a skip factor of 1)

3.3 Required

Determine whether the given slab meets the strength requirements of the code [ACI, 1992] for the dead and live loading specified. Allow in the computations for the immediate and long term stress losses in prestressing.

3.4 Solution

Only the 32-ft long span is investigated in this example. Other spans can be treated in a similar manner.

3.4.1 Determine the Factored Elastic Moments (Demand Moments M_u)

For the elastic demand moments, a frame analysis software with the following capabilities is suitable to use; (i) modeling based on the equivalent-frame action of the floor [ACI, 1992]; and (ii) prestressing forces based on a faithful representation of prestressing forces with due allowance to losses in stress along the tendons. Note that in obtaining an elastic solution for the demand moment, the existing nonprestressed reinforcement does not enter the computations.[2] The prestressing is represented by its equivalent load.

Where no special purpose software is employed, a combination of algorithms may be used to achieve the same end. In either case, the demand moments are computed using the existing geometry, prestressing, material and the boundary conditions, subjected to the specified loading.

Herein, the work assumes that the elastic solution to the frame is available, since the emphasis of the work is on the post-elastic treatment. The elastic solution used for the current problem is obtained from a special purpose post-tensioning software [ADAPT, 1994]. The software uses the equivalent frame method, and integrates in its analysis the long-term stress losses in post-tensioning. It is based on the variable force method. Other procedures can be used to arrive at the same solution. Due to the nature of the problem being examined, which requires an *elastic* solution as its entry value, no redistribution of moments are included in the solution obtained. From the solution, the elastic demand moments for the combination of dead, live, and hyperstatic moments, using relationship (2-1) are listed below:

Factored demand moment, M_u
  At left = -458.37 k-ft
  At center = 412.06 k-ft
  At right = - 580.59 k-ft

3.4.2 Determine the Available Capacity (ϕM_n)

From the geometry, material properties, prestressing and the nonprestressed reinforcement in the section, the available capacity each at the left, center, and right of the second span are determined. Only three sections are considered, since the section is prismatic and the loading is uniform. Strictly speaking, the maximum moment may not occur at midspan, but a midspan location is considered accurate enough for the current type of analysis and loading.

(i) Consider the face-of-support at the right support. Its capacity is computed using long-hand calculation. For the remainder of the locations, a computer program is used.

[2] In most computations of the type discussed in this topic, the added stiffnesses of the prestressing and non-prestressed reinforcement to the frame stiffness are disregarded.

Given parameters:

Span length $L = 32 \times 12 = 384$ in.
Thickness $h = 7.5$ in.
Tributary $b = 28.5 \times 12 = 342$ in.
$d_r = 7.5 - 1.63 = 5.87$ in.
$d_p = 6.1$ in.
$f_{se} = 175$ ksi
$f_{pu} = 270$ ksi
$f_y = 60$ ksi
$A_s = 4.84$ sq in.

Stress in prestressing steel at nominal strength:

$A_{ps} = 31 \times 0.153 = 4.743$ sq in.
$\rho_p = A_{ps}/(d_p b) = 4.743/(6.1 \times 342) = 0.002274$
Span-to-depth ratio $= L/h = 384/7.5 = 51.2 > 35$, hence use ACI-318 Eq. 18-5
$f_{ps} = f_{se} + 10{,}000 + (f'_c/300\rho_p)$ (ACI-318 Eq. 18-5)

$f_{ps} = 175 + 10 + [5.8/(300 \times 0.002274] = 193.50$ ksi $<$ $(175+30 = 205$ ksi$)$ OK

Depth of Compression Block:

$a = (A_{ps}f_{ps} + A_s f_{ten})/(0.85f'_c b)$
$= (4.743 \times 193.50 + 4.84 \times 60)/(0.85 \times 5.8 \times 342)$
$= 0.7166$ in.

Moment about centroid of compression block:

$\phi M_n = 0.90[A_{ps}f_{ps}(d_p - 0.5a) + A_s f_y(d_r - 0.5a)]$
$= 0.90[4.743 \times 193.50(6.1 - 0.5 \times 0.7166) + 4.84 \times 60(5.87 - 0.5 \times 0.7166)]/12$
$= 515.26$ k-ft

Using a computer program [ADAPT-PULT, 1994], the available capacities at the face of support and at the midspan are computed:

Available capacity ϕM_n	
At face of left support	: - 443.12 k-ft
At center of span	: 453.26 k-ft
At face of right support	: -515.26 k-ft

3.4.3 Compare the Elastic Demand (M_u) with Available Capacity (ϕM_n)

If at all locations, the available capacity (ϕM_n), computed in 3.4.2, exceeds the demand (M_u), the structure is satisfactory, else additional analytical steps as outlined in the following are necessary.

Comparison	
At face of left support	$\phi M_n = 443.12 < M_u = 458.37$ NG
At center of span	$\phi M_n = 453.26 > M_u = 412.06$ OK
At face of right support	$\phi M_n = 515.26 < M_u = 580.59$ NG

3.4.4 Permissible Percentages of Moment Redistribution

Since the elastic demand exceeds the available capacity at one or more locations, the computation must be continued to redistribution of moments.

Reinforcement ratios

Prestressing
$\omega_p = \rho_p f_{ps} / f'_c$

Strictly speaking, at each location, the associated f_{ps}, must be substituted. This applies when a comprehensive computer program is used. But, for hand calculations, such as the current example, for expediency of computations, it is permissible to use a somewhat conservative approach by assuming the maximum code allowable value for f_{ps}. In this case:

$f_{ps} = f_{se} + 30{,}000$ [ACI-318; section 18.7.2]

$f_{ps} = 175 + 30 = 205$ ksi

$\rho_p = 0.002274$ (from Sec. 3.4.2)

$\omega_p = 0.002274 \times 205/5.80 = 0.08037$
$\omega = \rho f_y / f_c$
where, $\rho = A_s / bd_r$

Since nonprestressed reinforcement varies along the span, ω, will be computed for each location separately.

$\beta_1 = 0.85 - 0.05(5.8-4) = 0.76$ [ACI- 1992; section 10.2.7.3]

Using relationship 2-5, check whether moment redistribution is permissible. The critical location is the right support.

Reinforcing index $= 0.08037 + (5.87/6.1) \times 0.009069 = 0.0891$
$< 0.24\beta_1 = 0.24 \times 0.76 = 0.1824$ OK

(i) At left support

$\rho = 1.76/(342 \times 5.87) = 0.0008767$
$\omega = 0.0008767 \times 60/5.80 = 0.009069$

% redistribution $= 20[1 - (0.08037 + (5.87/6.1) \times 0.009069)/0.36 \times 0.76]$
$= 13.487\%$

(ii) At right support

$\rho = 4.84/(342 \times 5.87) = 0.002411$
$\omega = 0.002411 \times 60/5.8 = 0.02494$

% redistribution $= 20[1 - (0.08037 + (5.87/6.1) \times 0.02494)/0.36 \times 0.76]$
$= 12.371\%$

(iii) At center span

$\rho = 2.20/(342 \times 5.87) = 0.00110$
$\omega = 0.00110 \times 60/5.80 = 0.01138$

% redistribution = 20[1 - (0.08037 + (5.87/6.1) x 0.01138)/0.36 x 0.76]
= 17.592 %

3.4.5 Demand Envelope

Using the maximum percentages for redistribution obtained in sections 3.4.4, the permissible upper and lower bound of redistributed moments at the critical sections is computed next.

(i) At left: 13.487% redistribution
Upper limit = 1.13487 x 458.37 = 520.19 k-ft
Lower limit = (1-0.13487) x 458.37 = 396.55 k-ft

(ii) At center: 17.592 %
Upper limit = 1.17592 x 412.06 = 484.55 k-ft
Lower limit = (1-0.17592) x 412.06 = 339.57 k-ft

(iii) At right: 12.371 %
Upper limit = 1.12371 x 580.59 = 652.41 k-ft
Lower limit = (1-0.12371) x 580.59 = 508.77 k-ft

The demand envelope is constructed in Fig. 3.4-1.

3.4.6 Compare the Available Capacity with the Demand Envelope

The available capacities at the critical locations are entered in Fig. 3.4.2. The capacities fall all within the demand envelope, hence the computation continues further.

3.4.7 Adjusted Demand Profile

Refer to Fig. 3.4-2.

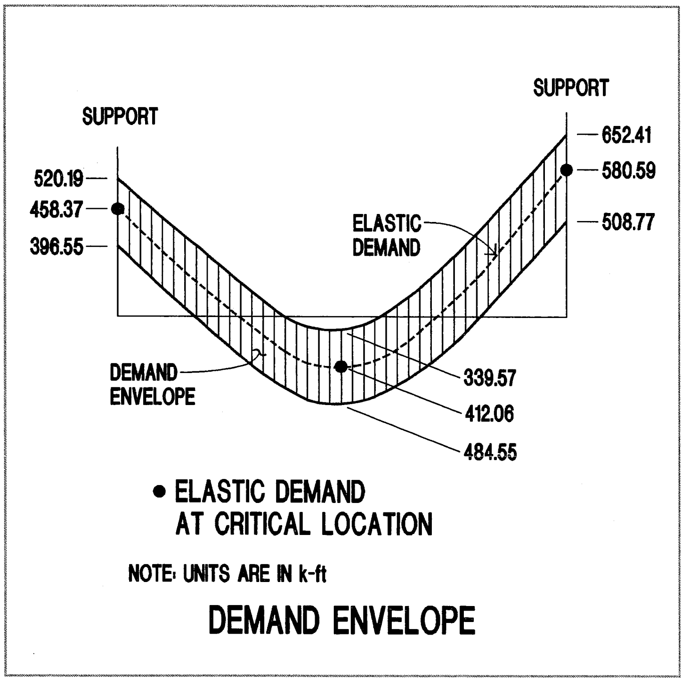


FIGURE 3.4-1

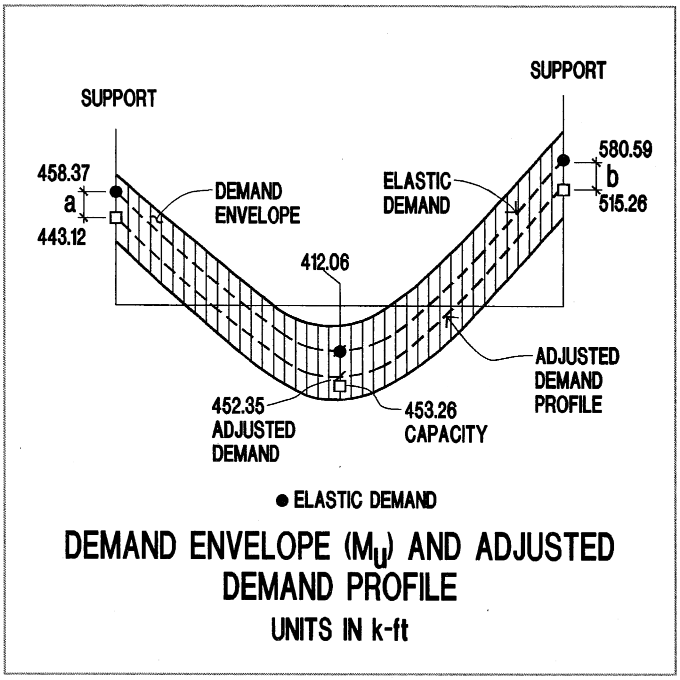


FIGURE 3.4-2

Adjustment at left support:
a = 458.37 - 443.12 = 15.25 k-ft

Adjustment at right support:
b = 580.59 - 515.26 = 65.33 k-ft

Adjustment to be made for midspan:
c = 0.5(15.25 + 65.33) = 40.29 k-ft

Adjusted demand at midspan:
412.06 + 40.29 = 452.35 k-ft

3.4.8 Check for Midspan Capacity

Adjusted midspan demand = 452.35 k-ft (Fig. 3.4-2)
Available midspan capacity = 453.26 k-ft

(Available capacity) > (Adjusted demand) OK

Hence, the span analyzed meets the strength requirement of the code.

4 - Notation:

a = depth of compression zone
b = width of cross section
A_{ps} = area of prestressed reinforcement
A_s = area of nonprestressed reinforcement
d_p = distance of compression fiber to center of prestressing force
d_r = distance of compression fiber to center of nonprestressed force
f_c = compressive strength of concrete at a given time
f'_c = compressive strength of concrete at 28 days
f_{ps} = stress in prestressing at nominal strength
f_{pu} = ultimate stress of prestressing strand

f_{se} = stress in prestressing strand after stress losses
f_{si} = initial stress in prestressing strand
f_{ten} = tension stress in nonprestressed reinforcement
K = wobble coefficient of friction
L = span length
M_d = dead load moment
M_{hp} = hyperstatic (secondary moment) due to prestressing
M_ℓ = live load moment
M_n = nominal strength of section in bending
M_u = factored demand moment
β_1 = coefficient for depth of compression block
ϕ = strength reduction factor
μ = angular coefficient of friction
ρ = reinforcement ratio
ρ_p = ratio of prestressed reinforcement
ω = reinforcement coefficient for nonprestressed tension reinforcement
ω_p = reinforcement coefficient for prestressing
ω' = reinforcement ratio for nonprestressed compression reinforcement

5 - REFERENCES

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READERS TECHNICAL COMMENTS

Comments on:

Wedge Forces on Post-Tensioning Strand Anchors
Chacos, G.; PTI-Technical Notes, Issue 2; September 1993

By: P. Dawson
Taywood Engineering Ltd
Southall, Middlesex, UK

The Technical Note article rightly draws attention to the high forces which can occur in the anchorage when the wedges are clean, new and well lubricated. Under these conditions, there is also a correspondingly high compressive load on the strand. If the wedges are poorly designed, this can lead to a strand failure at the nose of the wedge grips.

I have known cases of failures at loads as low as 85% of the characteristic strength of the strand, even lower if the strand beyond the grips is not wholly in line with the axis of the wedges.

For unbonded tendon systems, where the ultimate load efficiency of the anchorage is vital, it is very important to ensure that the detailed design of the wedges is such that it grips the strand at the back of the anchorage (where the tensile load in the strand is least), and is relieved at the nose of the wedges (where the tensile load in the strand is highest).

A consequence of this is that the forces trying to burst the anchor piece are not at its mid-thickness, but nearer its outer surface. This is where the anchor piece is thinnest (refer to Fig.1 of Technical Notes, Issue 2), and is therefore least able to provide the force required. The detailed design of an efficient wedge anchor is therefore more complex than might at first be expected.

I know that these issues were well understood, on both sides of the Atlantic, 30 years ago, in the pioneering pre-computer days of prestressing, but I suspect that they may get increasingly overlooked as engineers loose their feel for forces and materials and increasingly expect the computer to solve everything for them.

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