SAFETY INVESTIGATION OF AN UNBONDED FLOOR SYSTEM WITH RESTRAINT CRACKS[1]

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CONTENTS

G.1 OVERVIEW

This Technical Note covers the investigation of a post-tensioned floor slab that had developed extensive cracking due to the restraint of the supports to the free shortening of the slab. The investigation concluded that, in this instance, remedial measures such as epoxy injection of the cracks to restore the structural bond between the two faces of the crack could not compensate for the loss of strength from the cracking. More extensive repairs were necessary to restore the level of safety intended by the original design. The theoretical background to the procedure followed in this Technical Note is outlined in reference [TN455, 2015].

Pardis Parking Structure, the subject of the investigation, features one-way slab and beam construction. The single-span beams are 60 ft (18.288 m) long and are spaced 30 ft (9.140 m) apart. A 7- in (180 mm) thick post-tensioned slab spans between the beams. Figure G.1-1 shows a typical panel of the parking deck with its tendon layout and rebar. The lateral force resisting system of the structure is a system of shear walls between selected bays.

Shortly, after the tendons were stressed, extensive cracking was observed in a number of the panels of the parking deck. Most of the cracks originated from the wall along one side of the parking garage, ran essentially parallel to the beams along the length of the bay, and extended through the entire depth of the slab. Figure G.1-2 shows the crack formation at the top and bottom of one of the slab panels. All of the cracks were long; in some panels they extended the entire length of the bay, thus completely interrupting the transfer of precompression through the slab.

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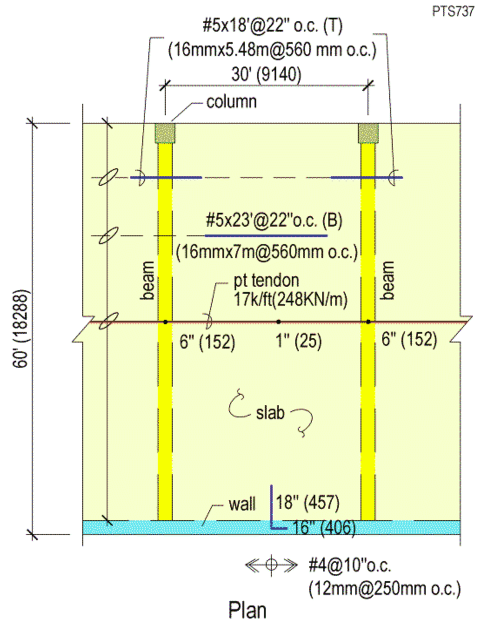


FIGURE G.1-1 Plan; Reinforcement in Slab, show tendon spacing and column at the far end (enter the PT force).

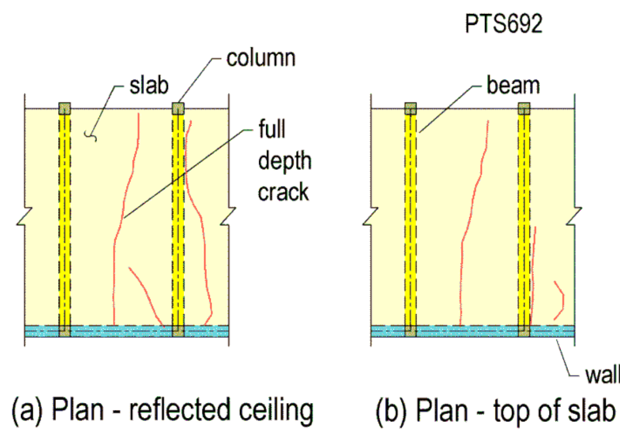


FIGURE G.1-2 Plan of One of the Slab Panels, Showing the Cracks at the Top and Bottom of the Slab

The material, geometry and details of the reinforcement in the slab are given in Appendix A.

G.2 IMPACT OF CRACK ON SLAB STRENGTH

Technical Note TN455 concluded that moment capacity is reduced at the location of a through crack caused by support restraint. The reduction in moment capacity is due to the loss in contribution from the post-tensioning tendons that cross the crack. The restraint from the supports diverts a fraction of the post-tensioning force, reducing the amount available to develop resistance at the cracked location. Figure G.2-1, taken from TN455, shows a post-tensioned member with a through crack and its tendon force at the service and strength limit states.

Part (c) of the Figure shows the left side of the member at the moment of collapse, when contact between the two faces of the crack is re-established as a result of the member's large displacement, and a compressive force is developed over the contact surface. From the equilibrium of part (c) of the figure, the compression force C is given by:

$C = (F - F_4)$ (Exp G.2-1)

Further, TN455 concludes that the force $(F - F_4)$ is equal to the friction force (P) between the strand and its sheathing for the tendon on the side of the crack that develops a smaller friction force. The moment capacity M will be:

$M = Cz$ (Exp G.2-2)

where z is the lever arm between the tension and compression forces.[3]

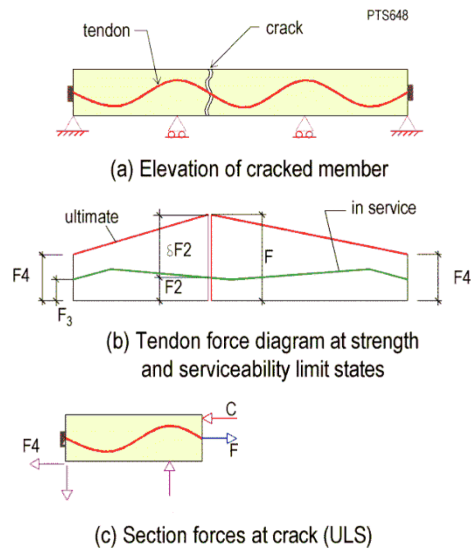


FIGURE G.2-1 Member with Unbonded Tendon; Tendon Force Diagram at the Service and Ultimate Limit States

To calculate the maximum moment that can develop at the crack location, the investigation assumes that before failure the slab will undergo a large displacement as the tendons elongate; at failure, the tendons elongate to the point of rupture. As an upper value for the moment that can develop at the crack location, the tendon force is assumed to reach its rupture value at the crack location.[4] The friction force P along the tendon length is calculated for the shorter length of tendon.

The steps to follow in the investigation are summarized below:

[3] The design moment capacity includes the contribution of non-prestressed reinforcement across the section, and is based on the code-specified strength and/or material factors. These are accounted for in the numerical calculation that follows.

[4] Neither ACI 318 nor EC2 allow the moment capacity of a section to be based on an unbonded tendon reaching its rupture force. The assumption made herein is to obtain an upper value for the capacity in this investigation.

Step 1. Verify that the crack under investigation appears at both the top and bottom of the slab: The precompression across the crack will only be zero if the crack extends through the full depth of the member. In addition, the crack must be fairly long to have a significant impact on the load capacity of the entire panel. As a first estimate, assume that full-depth (through) cracks less than one-quarter of the span in the direction of the crack are not of primary concern.

Step 2. Determine the location of anchors of the tendons that cross the crack: The distance from the tendon anchorages to the crack must be determined. Anchorages can be dead (fixed) end, stressing end, or intermediate stressing anchorages at a construction joint. The stressing and dead end anchorages can be several spans away from the panel under investigation.

Step 3. Select the shorter tendon segment on the side of the crack: The crack essentially divides the tendon length into two parts. For this investigation select the shorter part, under the assumption that conditions on both sides of the crack are similar, and thus the friction that develops over the shorter tendon length is less than what develops over the longer length.

Step 4. Assume that at the strength limit state, the tendon has reached its maximum stress (f_{pu}) at the crack location: Hence, the force F in each tendon at the location of crack will be:

$$F = A_{ps}\ f_{pu} \qquad \text{(Exp G.2-3)}$$

where A_{ps} is the tendon cross-sectional area.

The local increase in force to F at the crack location will cause the tendon on either side of the crack to stretch. The stretching develops a new distribution of friction force between the tendon and it sheathing as shown by the line labeled "ultimate" in part (b) of Fig. G.2-1

Step 5. Calculate the tendon force available to resist an applied moment: Consider the tensile force available to resist a moment at the crack location to be the difference between the force at the crack location ($A_{ps} f_{pu}$), shown as F in Fig. G.2-1(b) and the force in the tendon at the end anchorage[5], shown as F_4 in the G.2-1(b).

Step 6. Calculate the moment capacity at the crack location: Use the force in tendon determined in the previous step to calculate the moment capacity of the section at the crack location.

Step 7. Recalculate the load capacity of the cracked panel using the reduced moment at location of crack: Compare the calculated load capacity with the specified load on the slab. If the load capacity is less, devise a remedial measure.

G.3 COMPUTATION OF SLAB CAPACITY AND SAFETY FACTOR

[5] The background to this assumption and its validity is given in reference [TN455, 2015]

Step 1

The cracks were verified to extend through the depth of the crack. The crack length was well in excess of a quarter of the span length in the direction of the crack.

Steps 2 and 3

The shortest distance of the crack to an anchor is 120 ft (36.58 m). The tendon profile for this distance is given in Fig. G.3-1

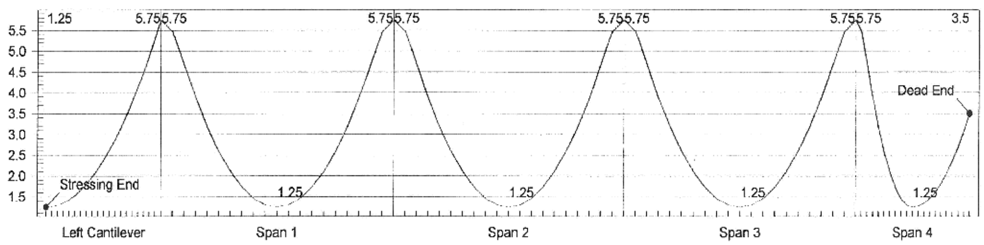


FIGURE G.3-1 Tendon Profile from Location of Crack (shown on the left) to the Tendon Anchorage at the Far End (shown on right). Dimensions are in inches, measured from the bottom of the slab

Step 4 - Calculate the loss in tendon force at the ultimate limit state

Assume that the tendon at the crack reaches its maximum stress (270 ksi; 1860 MPa). Using the friction parameters of the structure, the tendon profile given in Fig. G.3-1, and the friction formula given below, the distribution of stress along the tendon is calculated (see Appendix B for details of calculation).

$$P_x = P_j e^{-(\mu\alpha + Kx)} \qquad \text{(Exp G.3-1)}$$

Where,

P_x = stress in the tendon at a distance x from the point of application of force to the tendon;

P_j = stress in the tendon at the point of application of force;

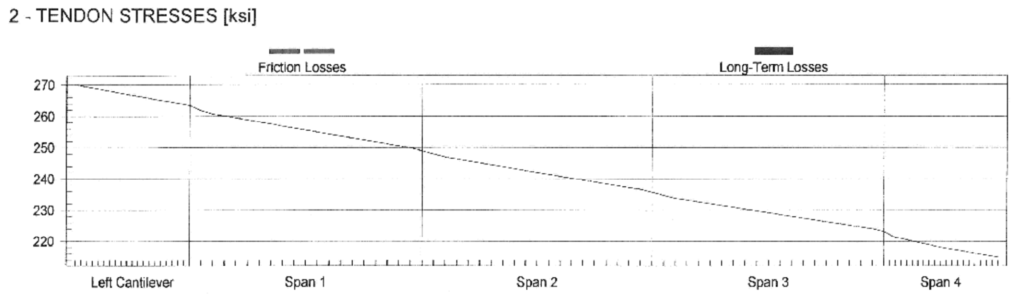
μ = coefficient of angular friction (/radian);

α = total angle change of the strand in radians from the stressing point to distance x;

x = distance from the stressing point; and

K = coefficient of wobble friction (/ft; /m).

In this case, P_j is the stress in the tendon at the crack location, which has increased due to the local elongation of the strand and P_x is the force at the anchorage. P_j corresponds *to F* in Fig. G.2-1; P_x corresponds to F_4 . The distribution of stress in the tendon along its length from the analysis is reported in Appendix B and shown in Fig. G.3-2.



G.3-2 Distribution of Stress in Tendon from the Location of the Crack (shown on left; f_{pu} = 270 ksi; 1860 MPa) to the Tendon Anchorage (shown on right F_4=215 ksi; 55 MPa)

From the stress diagram of Figure G.3-2, the change in tendon stress due to friction along the tendon between the crack being investigated and the tendon end is:

$F - F_4 = 270 - 215 = 55$ ksi (379.71 MPa)

where the stress at the tendon anchorage is 215 ksi.

Step 5 – calculate the effective tendon force
The force T in each strand available to resist the applied moment at the crack location is:

$T = A_{ps}(F - F_4) = 0.153 \times 55 = 8.43$ k (37.50 kN)

Step 6 – Calculate the moment capacity at the crack location
The design capacity of the section at the crack location depends on the number, location, and the available force of each tendon as determined in the previous step. In addition, the amount, location and orientation of the non-prestressed reinforcement crossing the crack surface must be accounted for.

The moment capacities of the section along the crack at mid-span and over the support are calculated in Appendix C for the reinforcement layout shown in Fig. G.1-1, accounting for the presence of non-prestressed reinforcement as well as the tendons. The design moment capacity at mid-span and over the supports is determined to be:

At midspan: Moment capacity per ft ϕM_n = 6.53 k-ft (8.85 kNm)

At supports: Moment capacity per ft ϕM_n = 6.53 k-ft (8.85 kNm)

Step 7 – Calculate the moment capacity of the panel
The load capacity of the panel is estimating, using a representative strip for a collapse mechanism as shown in Fig. G.3-3. For the collapse mechanism shown, the virtual work equation is:

$$2M_{\text{support}}\theta + 2M_{span}\theta = W_c L\delta / 2 \qquad \text{(Exp G.3-2)}$$

where,

L = Span length of representative strip between hinge lines at supports;
$M_{support}$ = moment capacity at support per unit length;
M_{span} = moment capacity at midspan per unit length;
W_c = load capacity per unit area of slab surface;
δ = magnitude of virtual displacement; and
θ = virtual angle of displaced failure strip.

From the geometry of the deformed strip:

$\delta = \theta L / 2$
$L = 30 - 2 = 28$ ft (8.53 m)

Substituting in G.3-2
$2 \times 6.53\theta + 2 \times 6.53\theta = 28W_c \times 28\theta / 4$
$W_c = 0.133$ k/sf (6.37 kN/m^2)

Calculate the design (demand) load W_u
Selfweight = (7/12) $\times$ 0.150 = 0.088 k/ ft^2 (4.21 kN/m^2)
Superimposed dead load = 0.010 k/ ft^2 (0.49 kN/m^2)
Total dead load = 0.098 k/ ft^2 (4.69 kN/m^2)

Specified live load = 0.040 k/ ft^2 (1.92 kN/m^2)
Factored demand load:
W_u = 1.2Dead + 1.6 Live = $1.2 \times 0.098 + 1.6 \times 0.040$ = 0.181 k/ ft^2 (8.67 kN/m^2)

Capacity/Demand Ratio:
W_c / W_u = 0.133/0.181 = 0.73 < 1 NG

Since the capacity is less than the specified load demand, the slab is inadequate. The slab has to be reinforced to raise its load carrying capacity to the design-specified value. It is important to note that restoring the bond between the faces of the crack by epoxy injection or other means will not remedy the shortfall in the required strength.

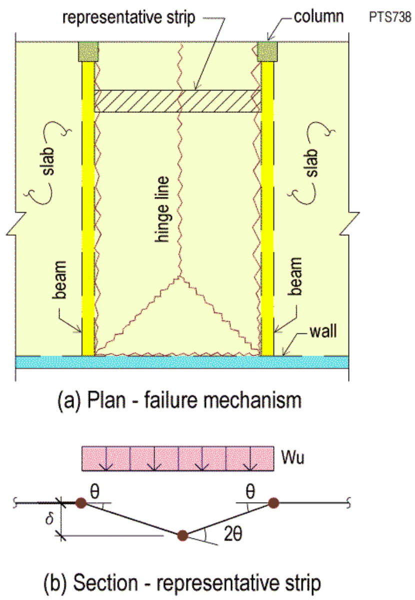


FIGURE G.3-3 Assumed Collapse Mechanism

APPENDIX A
GEOMETRY; MATERIAL AND REINFORCEMENT OF TYPICAL BAY

This appendix lists the material properties, geometry, and reinforcement of the panel being investigated.

Slab thickness	= 7” (178 mm)
Span in short direction	= 30’- 0"(9.14 m)
Span in long direction	= 60’-0” (18.28 m)
Beam size normal to slab span	= 24” wide, 32” deep (610x813 mm)
Beam span	= 60’-0” (18.28 m)

Concrete cylinder strength = 5000 psi [6] (34.5 MPa)

Non-prestressed reinforcement:
Top rebar over the supports in direction of slab span: #5 x 18’-0” @ 22” o.c. (16 mm x 5.48 m @ 560 mm o.c.)
Bottom rebar at midspan: #5 x 23’-0“ @ 22” o.c. (16 mm x 7.00 m @ 560 mm o.c)
Top angle rebar along the end wall: #4 x1’-6” @ 10” o.c. (12 mm x 460 mm @ 250 mm o.c.)
Cover to non-prestressed reinforcement:

Top rebar cover	= 1” (25 mm)
Bottom rebar cover	= 1” (25 mm)

Post-tensioning:
Unbonded system; 7-wire strand; f_{pu} = 270 ksi (1860 MPa)

[6] Actual strength is probably somewhat higher than the specified value due to the age of the concrete

Strand area	= 0.152 in^2 (98 mm^2)
Specified prestressing on plan[7]	= 17 k/ft (248 kN/m)
Effective force per strand specified	= 25.5 k (113.43 kN)
Coefficient of angular friction	= 0.07
Coefficient of wobble friction	= 0.0014 rad/ft (0.0046 rad/m)
Cover to tendon	= 0.75" (19 mm)

Tendon shape reversed parabola, with inflection points at one-tenth of span. Tendon CGS is as shown on plan (Fig. G.1-1).

APPENDIX B

FRICTION CALCULATION AND FORCE IN STRAND AT SLAB FAILURE

The friction calculation for the short side of the tendon is carried out using commercially available software [8]. Details of the input to the software and the report of the computations are given below.

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 ------------------------------------------------------------------------------
 |                             ADAPT CORPORATION                              |
 |         1733 Woodside Road, Suite 220, Redwood City, CA 94061 USA          |
 |                             www.adaptsoft.com                              |
 ------------------------------------------------------------------------------
 P R O J E C T   T I T L E :
 PARDIS PARKING STRUCTURE
 S P E C I F I C   T I T L E :
 Length between the crack (gridlines 12-13) and gridline 17
 F R I C T I O N  &  E L O N G A T I O N  C A L C U L A T I O N S :
 INPUT PARAMETERS :
 Coefficient of angular friction (meu)................       .07000 /radian
 Coefficient of wobble friction (K)...................       .00140 rad/ft
 Ultimate strength of strand .........  .............       270.00    ksi
 Ratio of jacking stress to strand's ultimate strength       1.00
 Anchor set ..........................................       .00    inch
 Cross-sectional area of strand ......................       .153   inch^2
 Total Number of Strands per Tendon...................        1
 Modulus of elasticity of strand .....................        28000.00    ksi
 STRESSING ...........................................        AT LEFT END
LEGEND :
 P ....... = Tendon profile type defined as: 1=reversed parabola;
             2=partial/regular parabola; 3=harped; 4=general
 X1/L etc  = horizontal distances to control points in geometry of the
             tendon divided by span length
 Stresses tabulated are after anchor set but before long-term losses.
 TENDON ID,  GEOMETRY AND STRESS PROFILE  (grid12)
 LENGTH    < TENDON HEIGHT in.>  Horizontal ratios   <-  STRESS (ksi)  -->
 SPAN  ft    P  start  center  right   X1/L X2/L X3/L     start   center  right
 -1----2-----3----4------5------6-------7----8----9--------10------11------12-
 CAN  15.00  1    1.25          5.75                .00     270.00         263.49
  1   30.00  1    5.75   1.25   5.75    .08  .50  .08     263.49  255.59 248.94
  2   30.00  1    5.75   1.25   5.75    .08  .50  .08     248.94  242.03 235.74
  3   30.00  1    5.75   1.25   5.75    .08  .50  .08     235.74  229.20 223.23
```

[7] This is based on 200 psi (1.37MPa). This corresponds to approximately a bundle of 2 strands at 3'-0"(914 mm) o.c,
[8] FELT; www.adaptsoft.com

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 4   15.00  1    5.75   1.25   3.50    .08  .50  .00     223.23  218.02 214.93
--------------------------------------
    120.00 ft (total length of tendon)
SUMMARY :
Average initial stress (after release)...................      242.38  ksi
Long term stress losses .................................         .00  ksi
Final average stress ....................................      242.38  ksi
Final average force in tendon ...........................       37.08  k
Anchor set influence from left pull  (270.00ksi;1.000) ..         .00  ft
Elongation at left pull before anchor set ...............       12.47  inch
Elongation at left pull after anchor set ................       12.47  inch
Total elongation after anchor set .......................       12.47  inch
Ratio of total elongation to
      tendon length after anchor set ....................          0.  inch/ft
Jacking force ...........................................       41.31  k
```

APPENDIX C

MOMENT CAPACITIES

For expediency, the design moment capacity at the hinge locations of the strip shown in Fig. G.3-3 is determined using the approximate procedure outlined below. If the result of the approximate method is close to the required strength of the slab, a more detailed analysis should be done. The design capacity is calculated for a 12 inch (205 mm) width of the representative strip, based on the reinforcement and the available force in the strands that cross the unit strip.

At mid-span
There are 2 strands spaced 30” (762 mm) apart.
Available prestressing force per ft: 2x.8.43/2.5 = 6.74 k/ft (29.98 kN/ft)
Rebar area per ft from reinforcement shown on plan (Fig. G.1-1) = 0.17 in^2 (110 mm^2)

Distance of rebar centroid to compression fiber *dr*:
dr = 7-1-0.625/2 = 5.69 in (145 mm)

Distance of tendon centroid to compression fiber *dp*:
dp = 7-1.25 = 5.75 in (146 mm)

Assume the distance from rebar and tendon forces to the center of compression is 0.9 times the distance to the extreme compression fiber.

Design moment capacity φM_n

φM_n = 0.9[$6.74\times 0.9\times 5.75$ + $60\times 0.17\times 0.90\times 5.69$]/12 = 6.53 k-ft (8.85 kNm)

Hinge at support
The tendon force at the hinge over the support is slightly less than at mid-span but for the first try, it will be assumed that it is the same as at mid-span. Since the rebar and tendon eccentricity at the support are the same as at mid-span, following

the assumption of equal tendon forces, the design capacity over the support will be essentially the same as that of midspan. Hence:

φM_n = 6.53 k-ft (8.85 kNm)

R.3 REFERENCES

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ADAPT TN455, Aalami, B. (2015), "*Crack Evaluation of Post-Tensioned Members*, " ADAPT Corp. www.adpatsoft.com; www.PT-Structures, pp. 12, 2015

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R.4 NOTATIONS

C = compression developed in section at ultimate limit state;
F = Total force in tendon at ultimate limit state;
F_3 = Force at support restraint; also force in tendon at location of through cracks under service condition;
F_4 = Force in tendon at support restraint under ultimate limit condition;
M = demand moment at location of through crack;
T = amount of tendon force available to resist the demand moment;
δF_2 = local increase in tendon force at ultimate limit state;
M = moment capacity; and
W = load per unit surface of slab.