Report No. KS-02-5
FINAL REPORT

CROSS TENSIONED CONCRETE PAVEMENT

Muhammet Vefa Akpinar, Ph.D., P.E.
Jeffrey Hancock, I. E.
Mustaque Hossain, Ph.D., P.E.

Kansas State University
Manhattan, Kansas

OCTOBER 2003

KANSAS DEPARTMENT OF TRANSPORTATION

Division of Operations
Bureau of Materials and Research

1 Report No. KS-02-5	**2 Government Accession No.**	**3 Recipient Catalog No.**
4 Title and Subtitle CROSS TENSIONED CONCRETE PAVEMENT		**5 Report Date** October 2003 **6 Performing Organization Code**
7 Author(s) Muhammet Vefa Akpinar, Ph.D., P.E., Jeffrey Hancock, I.E., and Mustaque Hossain, Ph.D., P.E.		**8 Performing Organization Report No.**
9 Performing Organization Name and Address Kansas State University Department of Civil Engineering; 2118 Fiedler Hall Manhattan, Kansas 66506-5000		**10 Work Unit No. (TRAIS)** **11 Contract or Grant No.** C1257
12 Sponsoring Agency Name and Address Kansas Department of Transportation Bureau of Materials and Research, Research Unit 2300 Southwest Van Buren Street Topeka, Kansas 66611-1195		**13 Type of Report and Period Covered** Final Report December 2000 - March 2002 **14 Sponsoring Agency Code** RE-0267-01
15 Supplementary Notes For more information write to address in block 9.		

16 Abstract

Joints are the weakest parts of the Portland Cement Concrete Pavement (PCCP). The deterioration of PCCP often happens due to intrusion of water into the pavement system as well as due to inferior performance of the transverse joints. The infiltration of surface run-off commonly occurs at the transverse joints and cracks. This problem could be solved by eliminating transverse joints and constricting the cracking capability of the pavement by applying an external force to the pavement in the form of post-tensioning. The post-tension strands can be arranged diagonally resulting in Cross Tensioned Concrete Pavement (CTCP). Finite element analysis results show that the maximum tensile stress at the crossing of the strands near the pavement edge is much lower than the recommended allowable stress. The tensile stress between the opposing bearing plates is also reasonable. The proposed design also resulted in maximum compressive stresses and displacements as per design expectation. Thus, the proposed CTCP design appears to be a feasible design solution for longer lasting concrete pavement.

16 Key Words Crack, Joint, CTCP, Cross Tensioned Concrete Pavement, PCCP, Portland Cement Concrete Pavement, Post-Tension, Stress, Transverse Joint	**17 Distribution Statement** No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

18 Security Classification (of this report) Unclassified	**19 Security Classification (of this page)** Unclassified	**20 No. of pages** 32	**21 Price**

Form DOT F 1700.7 (8-72)

CROSS TENSIONED CONCRETE PAVEMENT

Final Report

Prepared by

Muhammet Vefa Akpinar, Ph.D., P.E.
Kansas State University

and

Jeffrey Hancock, I.E.
Kansas State University

and

Mustaque Hossain, Ph.D., P.E.
Kansas State University

A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

KANSAS STATE UNIVERSITY
MANHATTAN, KANSAS

October 2003

NOTICE

DISCLAIMER

ABSTRACT

Joints are the weakest parts of the Portland Cement Concrete Pavement (PCCP). The deterioration of PCCP often happens due to intrusion of water into the pavement system as well as due to inferior performance of the transverse joints. The infiltration of surface run-off commonly occurs at the transverse joints and cracks. This problem could be solved by eliminating transverse joints and constricting the cracking capability of the pavement by applying an external force to the pavement in the form of post-tensioning. The post-tension strands can be arranged diagonally resulting in Cross Tensioned Concrete Pavement (CTCP). Finite element analysis results show that the maximum tensile stress at the crossing of the strands near the pavement edge is much lower than the recommended allowable stress. The tensile stress between the opposing bearing plates is also reasonable. The proposed design also resulted in maximum compressive stresses and displacements as per design expectation. Thus, the proposed CTCP design appears to be a feasible design solution for longer lasting concrete pavement.

ACKNOWLEDGEMENTS

The financial support for this study was provided by the Kansas Department of Transportation (KDOT). The authors wish to thank Mr. Andrew J. Gisi, P.E., Project Monitor of KDOT for his interest, cooperation and advice during this study. The study may have not been possible without the valuable work and supervision of Mr. Zhon Wu, former graduate student in the Civil Engineering Department of Kansas State University. The authors would like to acknowledge his contribution with deep gratitude.

TABLE OF CONTENTS

LIST OF FIGURES

1.1 Introduction

Joints are the weakest parts of the Portland Cement Concrete Pavement (PCCP). The deterioration of PCCP is most often caused by intrusion of water into the pavement system as well as due to inferior performance of the joints. Part of the run-off usually infiltrates through the joints and cracks in concrete pavement. During construction process and routine maintenance, the transverse joints and cracks are typically sealed with a flexible joint sealant. The sealant keeps most of the water out, however, some water does get into the pavement. Eventually the pavement may deteriorate due to pumping and subsequently, would develop faulting.

Eliminating transverse joints and cracks is one solution to this common problem of PCCP, and this has been demonstrated in the Continuously Reinforced Concrete Pavements (CRCP). Pre-stressing PCCP could also eliminate joints and cracks. By applying an external force in the form of post-tensioning, theoretically all transverse joints and cracks in the pavement can be eliminated. In practice, concrete cracks, but the high stresses induced by post tensioning should hold the cracks very tight making those impervious to water. Extended pavement life should also be expected as a result of pre-stressing PCCP. Pre-stressed PCCP's should have better ride quality too due to absence of transverse joints. Pre-stressing also should increase the load carrying capacity of the pavement and lower the maintenance costs.

1.2 Objective

One alternative to the conventional concrete pavement is Cross Tensioned Concrete Pavement (CTCP). The objective of this report is to present the fundamental design concepts for CTCP development. As a result of introducing cross pattern post tension stresses in CTCP, there are many unknown stress effects. Typical stresses developed in CTCP are also analyzed and discussed here.

1.3 Historical Background

Post-tensioning of PCCP is not a new concept. The first construction of a post-tensioned pavement was at the Orly Airport, Paris, in 1946. The Europeans have long since advocated the use of post-tensioning in airport pavements. Europe has also taken the lead in the application of post-tensioning to highway pavements. During the 40's, 50's, and 60's, the Europeans constructed over thirty Post-Tensioned Concrete Pavements (PTCP). During the same time six airport pavements were constructed in the United States. No highway pavements were post-tensioned. In the early 70's, three PTCP's and a prestressed access road at Dulles International Airport were constructed (1).

Since the early 70's, post-tensioning has become almost obsolete in highway design in the United States. However, as part of a number of experimental projects in 1977 the Arizona Department of Transportation (ADOT) constructed its first PTCP (2). The ADOT PTCP will be highlighted in this paper for comparison with the CTCP concept proposed.

1.4 Fundamental Design Concepts

Conventional PCCP design uses the modulus of rupture as the strength parameter of concrete and does not directly take advantage of the high compressive strength of concrete. Post-tensioned concrete pavement (PTCP) increases the flexural stress range by introducing compressive stresses in the concrete slab. The resulting pavement has several advantages: (a) absence of cracks, (b) reduced cost through a reduction in slab thickness, and (c) a large increase in load carrying capacity.

The fundamental formula for the design of pre-stressed pavements is (7):

$$f_t + f_p \geq f_{\Delta t} + f_F + f_L$$

where:

f_t = allowable concrete flexural stress;

f_p = compressive stress in concrete due to post-tensioning stresses;

f^{a}_{t} = curling stress due to temperature difference between top and bottom of slab;

f_F = stress due to subgrade friction; and

f_L = stress due to traffic load.

The allowable concrete flexural stress may be taken as high as 80 to 100 percent of the modulus of rupture of concrete. Introducing a compressive stress in the concrete pavement changes the failure mechanism from a bottom tension crack to a top circular crack. The failure load is at least twice the load that would produce the first bottom crack. This allows the designer to choose a factor of safety between 1 and 1.25 for f_t (7).

Stresses induced in PTCP should not exceed the compressive strength of the concrete. During construction stresses must be applied before the concrete has reached its maximum compressive strength. Construction stressing resists the formation of shrinkage cracks. However, tensioning to design may cause a compressive failure of the concrete. Therefore, tensioning of post-tension strands should be executed in stages (1).

Allowable stresses in the post-tensioning strands, f_{se}, after all losses including creep should not exceed 80 percent of the ultimate strength, f_s', of the post tensioning steel (7). A typical post-tensioning mono-strand will be a one-half inch diameter strand, made with six wires twisted around one wire. The ultimate strength of a typical post-tensioning strand is 1,862 MPa (270 ksi) (1). Thus the design stress in each strand should not exceed 1,490 MPa (216 ksi).

Curling stresses in PTCP happens due to temperature differential between the top and the bottom of the slab. Concrete slabs curl when one face is warmer or cooler than the other. The warmer face of the concrete tends to expand while the cooler face contracts. Curling stresses in PTCP can have adverse effects on the stresses within the post-tensioning strands. Shortening of the slab caused by curling will cause some relaxation in the post-tensioning strands. Consequently, unlike typical PCCP design, it is essential to design for curling in PTCP (1). The curling stresses in concrete can be calculated from (7):

$$f_{\Delta t} = \pm \frac{\alpha E_c \Delta t}{2(1 - \nu)}$$

where,

α = coefficient of thermal expansion (2.74 x 10^{-6} mm/mm/°C or 6 x 10^{-6} in/in/°F);

Δt = temperature gradient (usually 3°F/in.);

E_c = Modulus of elasticity of concrete (in psi or MPa) ; and

ν = Poisson's ratio for concrete (0.15).

Friction between the slab and the subgrade can give a false indication of compression in PTCP. Consider, for example, that a slab is pinned to the sub-grade at two different points between the ends of a post-tensioning strand. When the strand is tensioned a compressive force is exerted into the concrete slab. However, the region between the two pins experiences no compressive force. For this reason the frictional stress between the slab and the subgrade must be considered in the design of PTCP.

The coefficient of subgrade friction is substantial with a value of 0.2 to 1.5 for slabs resting on a sand or granular subbase. For design purposes, the coefficient of subgrade friction can be taken as 0.5 to 0.8. Most historical designs of PTCP incorporate two layers of 6 mil polyethylene sheeting which have a coefficient of friction of less than 0.2 (7). In the Arizona PTCP project, almost no differences in friction between the slab and subgrade on either one or two layers of polyethylene sheeting were found (5). In either case, it is recommended that the designer use a conservative value of 0.5 for the coefficient of friction in design (7).

Friction between the subgrade and the slab produces a tensile stress that is most significant in the middle portion of the slab. For a slab that is less than 213 meters (700-ft) in length, the frictional stress between the subgrade and the slab can be estimated as (7):

$$\max f_F = \frac{c\gamma L}{2x144}$$

where:

c = coefficient of subgrade friction;

L = total longitudinal length between each in line post-tension bearing plate; and

γ = unit weight of concrete.

All conventional PCCP designs consider traffic load as the major cause of stresses in pavement. For PTCP, traffic load consideration is no different. Traffic loads induce a tensile stress at the bottom of the pavement. The stress is calculated by the Westergaard's formula. The edge loading of PCCP is always the controlling criteria. The stresses in the pavement induced by repeated traffic loads can lead to the elastic deformation of the slab to the point where the maximum moment beneath the loaded area exceeds the sum of the flexural strength of the concrete and the induced prestressing force. At that point a crack forms a hinge under the load, and repeated load applications will cause a moment in the slab some distance away from the loaded area. If the load repetition continues tensile cracks can form at the top of the pavement.

When loading is increased beyond this point, the load will eventually punch through the slab. For this reason it is extremely important to consider traffic load in PTCP (7).

Now the Westergaard's formula for edge loading of PCCP is given by (4):

$$f_{L:} = \frac{0.803\text{P}}{h^2}\left[4\log\left(\frac{l}{a}\right) + 0.666\left(\frac{a}{l}\right) - 0.034\right]$$

where:

f_L = maximum edge stress under load;

l = radius of relative stiffness = $\left[\frac{Eh^3}{12(1-v^2)k}\right]^{0.25}$

E = modulus of elasticity of concrete (assume 27,579 MPa or 4,000,000 psi);

k = modulus of sub-grade reaction (assume 27 kPa/mm or 100 psi/in);

h= slab thickness (in or mm);

P = concentrated load, lbs.; and

a = contact radius = $\sqrt{\frac{P}{tirepressurex \prod}}$

For a typical maximum load of 98 kN (22,000 lbs) per axle on dual wheels, P can be estimated as 24.5 kN (5,500 lbs) and the tire pressure can be taken as 690 kPa (100 psi).

Figure 1 shows the distribution of stresses in the fundamental design equation. The tensile strength of the concrete and the compression of the slab by post-tensioning hardware must be able to overcome the combined stresses of traffic, curling, and subgrade friciton .Typical jointed PCCP will deflect at the joints under repeated loads. A large part of the deflection in PCCP's is most often caused by deformation of the subgrade. However, PTCP loads are distributed over a continuous rigid slab. Due to PTCP's interaction with the loading and the subgrade, it behaves much like a spread foundation, where large loads are distributed over wider area. Also, voids in the subgrade can be spanned due to an increase in the flexural strength of the post-tensioned concrete.

1.5 ADOT Experience with PTCP (Longitudinal Post Tensioning)

1.5.1 Design

In 1977, a PTCP was constructed as part of a number of experimental test sections on the Superstition Freeway in Mesa, Arizona. The longitudinally PTCP slabs varied in length from 207 to 500 ft. The design of the Arizona PTCP was based on experience on similar projects and recommendations from a consultant and the FHWA (3). A slab length of 400-ft was selected as typical and verified by calculations. The prestressing steel used was a seven wire, 0.5-inch nominal diameter strand with yield strength of 270 ksi. The strand was coated with grease to inhibit corrosion and protected from the concrete with a thin plastic jacket. Two sheets of 6 mil polyethylene sheeting were typically placed between the slab and the subbase. The subbase was a 4-in layer of lean concrete. The pavement was placed 31.5-ft wide and consisted of two 12-ft traffic lanes and a 7.5-ft wide outside shoulder. The post-tensioning was applied at gap slabs located at the ends of each prestressed slab as shown in Figure 2. A space of 8-ft was left between each of the continuous post tensioned slabs to accommodate the end hardware and to provide a working area to apply the tensioning force (5).

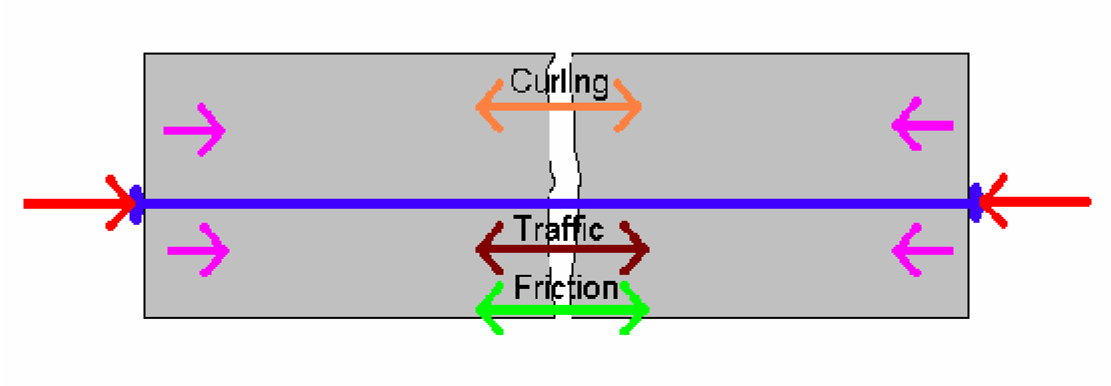

FIGURE 1: Stresses in PTCP

FIGURE 2: Gap Slab Construction

1.5.2 Initial Performance

Slab lengths shrunk to a varying degree upon application of the staged post-tension force. The typical slab shortened by 0.29-in upon initial stressing, 0.76-in upon final stressing, and 0.94-in two months after construction. The application of the post-tension force was responsible for the initial and final stressing stage slab shortening. Long term change in slab length was due to a combination of factors that include shrinkage, creep, and frictional resistance between the slab and the subbase (5).

Crack surveys were done after placement of the slabs, after initial stressing, once a week following the final stressing, and two months after construction. Transverse cracks were noted after placement of the slabs. The cracks varied in width from 1/64-in to ¼-in however, nearly all the cracks closed upon final stressing (5).

1.5.3 Nine Year Performance

In 1985, an evaluation of the Arizona PTCP was completed by ADOT. The most obvious problem found during visual survey was at the joints between the post-tensioned slabs and the gap slabs. Several slabs and gap slabs were reported to have corner breaks attributed to the high curling stresses, restraint of the load transfer devices in the gap slabs, and the infiltration of incompressible materials into the joints between the gap slabs and the post-tensioned slabs (6). The as-constructed roughness of the PTCP was reported to be 50 percent to 80 percent higher than the conventional PCCP's in Arizona. For the first 3 years after construction roughness continued to increase and then stabilized with time (6).

1.5.4 Eleven-Year Performance

In 1988, the performance of the PTCP was reevaluated. The survey included mapping of distresses, roughness survey with a Mays Meter, Falling Weight Deflectometer (FWD) tests, and general observations (2). As mentioned earlier, most of the deterioration of the Arizona PTCP occurred at or in the vicinity of the gap slabs. Both spalling and corner breaks were common at the joints between the gap slabs the post- tensioned slabs. The PTCP had far few joints however, the severity of joint deterioration was worse than that found in conventional PCCP's (2).

Transverse cracks did occur in larger than 300-ft sections of post-tensioned slabs. The transverse cracks were due to the paving operation. Problems encountered during paving resulted in many starts and stops of the paving operation. This caused transverse undulations in the pavement. All cracks however were reported as low severity cracks (2).

Many longitudinal cracks in the low to medium severity ranges were encountered. This was attributed to the high tensile stresses around the post-tensioning strands and improper placement of the strands. The amount and severity of longitudinal cracking was far worse than that of transverse cracking. The Arizona PTCP was post-tensioned in the longitudinal direction only, keeping transverse cracks to minimum. Longitudinal cracks were far more likely to occur at a greater severity (2). The Arizona PTCP had high roughness and low serviceability ratings. The undulations left by the paver and patching of gap slabs had a great deal to due with the increase in roughness values. The largest contributor to the roughness was the gap slabs and joints (2).

1.6 CTCP Design Considerations

Cross-tensioning PCCP may give the designer the option to construct an unjointed, uncracked pavement of any length. The procedure for designing such a pavement is similar to PTCP which is post-tensioned in the longitudinal direction. The only difference is the interpretation of the frictional losses between the concrete slab and the subgrade. In longitudinal post-tensioning design, the designer considers a finite section of pavement resulting in gap slabs between post-tensioned slabs. The length of the pavement between the gap slabs is the effective slab length that is subjected to the frictional force between the PCCP slab and the subgrade. In CTCP, the designer post-tensions the pavement in both diagonal directions, as shown in the plan view Figure 3. Because of cross- tensioning, the concrete slab cannot expand in any direction. This results in unjointed and continuous pavement. The slab length would no longer be limited by current shrinkage crack concerns.

The appropriate angle, at which the cross tensioning should be applied, should be less than 45 degrees to the longitudinal direction of the pavement. This allows the majority of the stressing force in the strands to act against the weak plane of the concrete, the transverse plane. The less than 45-degree specification also resists cracking the in the longitudinal direction, as did the Arizona PTCP.

Strand location in the slab is most important. Historically the strands of post-tensioning steel have been place 12.5 mm (0.5 inches) below the mid-depth of the slab. At this depth, the strands gain tension under heavy loads through elongation. Lowering the strands to 12.5 mm (0.5 inches) below mid-depth causes the slab to have an induced negative moment. When a heavy load traverses the pavement, the induced negative moment is canceled by a positive moment. This allows a much thinner slab than a conventional concrete pavement.

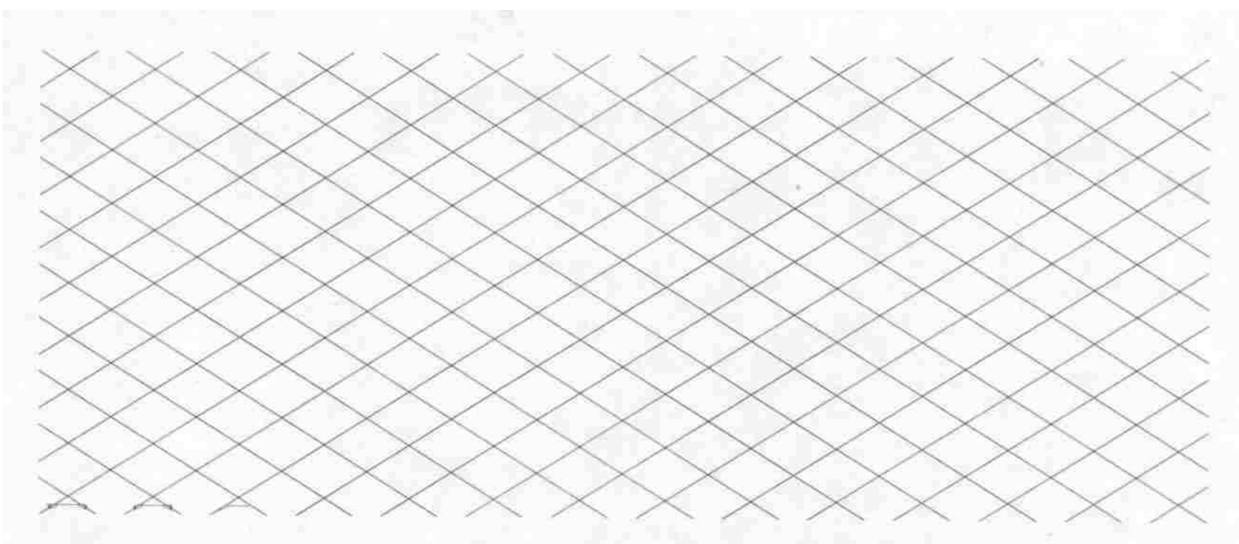

FIGURE 3: Plan View of Diagonal (Cross) Tensioning in CTCP

Strand placement at the edge of the pavement is also very significant. It is important that when the strands are stressed in different directions on the same side of the pavement very near to each other, they be placed so that compression happens in between the strands. This requires that the strands be crossed very close to the pavement edge. For example, assume that a pavement is stressed with the 45-degree post-tensioned strands. Consider now that the strands are laid out in such a way that two strands, perpendicular to each other, are very close, but not crossing at the pavement edge. This will cause a tensile stress in the concrete between the two strand ends. If the tensile stress is too great, the concrete may develop a transverse crack.

Now consider the same two strands as crossing very near the pavement edge. The only stress that will exist between the two strands in that case is a compressive stress. It is well known that concrete cannot support excessive tensile forces. The tensile strength of concrete is highly variable and is usually only considered to be 10 to 15 percent of the compressive strength.

The thickness of the concrete slab used in PTCP is less than conventional concrete pavement. Pavements as thin as 89 mm (3.5 inches) have been developed for use with longitudinally post-tensioned pavements. However, problems associated with coverage and uniform paving make this an unfavorable choice on most highway pavements. ACI (*1*) recommends that a pavement thickness at least 65% of the thickness of an alternative plain concrete pavement be used for PTCP's. This allows for appropriate coverage and variations in construction. For most cases, a pavement thickness of 150 mm (6 inches) is sufficient for coverage and construction tolerances.

Special attention needs to be given to the stressing operation. The strands cannot be stressed to design until the concrete has gained sufficient strength. Arizona stressed its longitudinally post-tensioned PCCP in three stages (*5*). Since cracking occurs from volume change, thermal gradient, and subgrade restraint, it is imperative that the first stage tensioning be applied to the cables at the earliest practical time, usually within 24 hours. Some small cracks may form before initial jacking; however, they should close upon application of the jacking force. The second application of stressing should be done within 24 hours of the previous operation. Lastly, the final jacking should be done when the concrete strength has reached at least 21 MPa (3,000 psi), independent of time. It is imperative not to jack the strands beyond the strength of the concrete. Jacking forces should be determined considering the size and thickness of the bearing plate end anchors and minimum concrete strength necessary to withstand the applied force. It may be necessary to increase the end anchor bearing plate size to distribute the high bearing stresses at the slab edges.

1.7 Proposed CTCP Design

If we apply the fundamental equation for the design of post-tensioned pavements to the cross-tensioned pavements, the design is similar to those pavements stressed in the longitudinal direction. As an example, the design of a 152 mm (6-inch), 7.32 m (24 ft.) wide concrete pavement is illustrated here. An angle of post-tensioning of 30 degrees is assumed. Solving for the fundamental factors, we get the following stresses:

$f_{\Delta t}$ = 1.76 MPa (255 psi)

f_t = 80% of the Modulus of Rupture = 3.03 MPa (440 psi)

With: P = 2,497 kg (5,500 lbs.), k = 2767990 kg/m^3 (100 pci), l = 744 mm (29.3 in.), tire pressure = 690 kPa (100 psi):

f_L = 2.90 MPa (422 psi)

With c = 0.5:

F = 83 kPa (12.08 psi)

Now,

f_p = 1.76 + 0.083 +2.90 - 3.03 = 1.71 MPa (249 psi)

The value of f_p is the minimum amount of compressive stress required in the concrete to overcome the effects of warping, friction, and traffic loads. Therefore, the stress force required per ft. width of pavement is:

1.71 MPa (250 psi) x 0.1524 m (6 in.) depth = 260.6 kN/m (18,000 lbs./ft)

Across the entire pavement, the total force required is:

260.6 kN/m (18,000 lbs./ft) x 7.32 m (24 ft) = 1907.6 kN (432,000 lbs).

Since the compressive force is going to be applied at an angle of 30 degrees, we need to resolve the compressive force required into components.

For a 30 degree skew:

1907.6 kN /Cos (30) = 2202.7 kN (498,831 lbs.)

In addition, the force will be applied from two different sides of the pavement. Therefore, we can split the force required into two halves. This would allow the strands from both sides of the pavement to contribute to the total force required in the concrete.

Splitting the force yields:

2202.7 kN/2 = 1101.35 kN (249,416 lbs.)

The strands applying the force will be tensioned to the fullest capacity however losses do occur due to relaxation of the strands. ACI recommends that the effective stress in the strand not exceed 80 percent of the ultimate stress (ACI 1988). Effective stress is the stress in the strand after all losses have occurred. The effective stress is computed as:

80% x (1860 MPa ultimate strength) = 1488 MPa (215 ksi)

Using 12.5 mm (0.5 inch) diameter strands with an area equal to 98.7 sq. mm (0.153 sq. in.), the effective force in each strand can be computed as:

$$9.87x10^{-05}\,m^2 x1488.2MPa = 146.9kN(32{,}895lbs)$$

The minimum number of strands needed across the transverse plane in the pavement can be computed as:

$$\frac{1101.35 compressive kN req}{146.9 kN per strand} = 7.5 \cong 8 strands from each side$$

The spacing of the strands across the transverse section will vary. Using geometry and a minimum of 16 strands per transverse section, it can be determined that a spacing of 1.53-m (5-ft) between the strands tensioned in the same direction will be required along the pavement edge.

1.8 Finite Element Analysis of Stresses Near the Pavement Edge

As mentioned earlier, stresses at the pavement edge are very critical to the performance of CTCP. One basic model of the stresses in the PTCP edge has been developed using the ANSYS finite element (FE) software. Finite element (FE) analyses provide a significant basis for the development of mechanistic design of pavements. Available finite element programs are powerful tools for stress-strain analysis in pavement structures. Therefore, it is expected that they can be used to investigate the stresses and strains in CTCP.

To investigate the stress at the edge of CTCP, a three-dimensional finite element model was built using ANSYS 5.5 FE software. The software enabled the modification of the geometry of the FE model grid and selection of several element types. These options allowed performing more specific analyses, such as, changes in geometry, boundary conditions, input parameters, material properties density, elastic modulus, and friction coefficient. SOLID45 element was used to capture behavior of the CTCP and subbase layers in three dimensions. CONTAC174 and

TARGE170 elements were used to model the interface friction between the CTCP slab and subbase layers.

1.8.1 Meshing, Model Dimensions and Mechanical Properties

Automatic mesh generator option that can perform structured and unstructured mesh generation was used to mesh the CTCP slab and subbase layers. Mesh dimensions were kept small enough to allow detailed analysis of the edges. The mesh was made finer around the region with the strands. However, as the mesh was made finer, the number of elements increased resulting in increased memory and computational time requirements. The software was limited to 32,000 nodes. Thus, some modifications (smaller dimension) were required in the modeling process.

A model that contains more strands was desirable. Several FE models with different dimensions were tried. Mesh problems occurred with models containing higher numbers of strands. Less accurate results were expected with coarser mesh structures. The loaded CTCP edges had non-smooth stress distribution as shown in Figure 4. Due to memory and convergence problems, the best possible model structure was a slab 3.1m (120 inch) long (x direction), 3.8 m (150 inch) wide (z direction), and a typical 300 mm (6 inches) thick (y direction). Below the CTCP slab, a 300 mm (6 in) thick subbase layer was assumed. The elasticity moduli of the CTCP slab and the subbase were 27,579 MPa (4,000,000 psi) and 690 MPa (100,000 psi), respectively. A typical value of Poison's ratio (0.15) was assigned to both layers. The weights of the layers were also included in the model. Figures 5 and 6 show the layer dimensions and the element mesh used to model the layers, respectively.

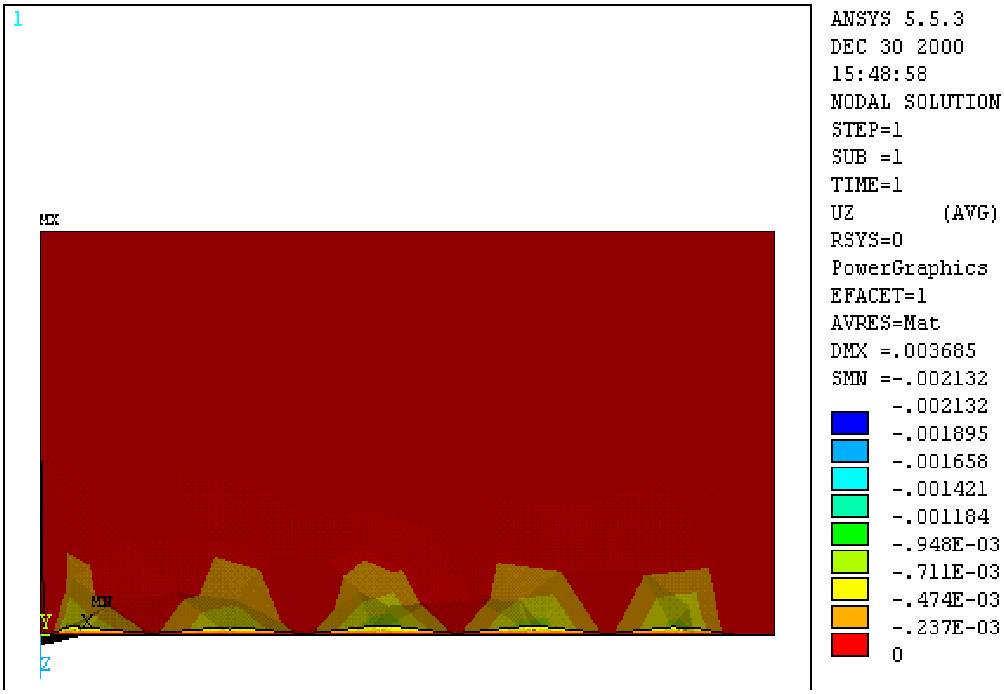

FIGURE 4: Non-Smooth Mesh Plan View with Five Strands

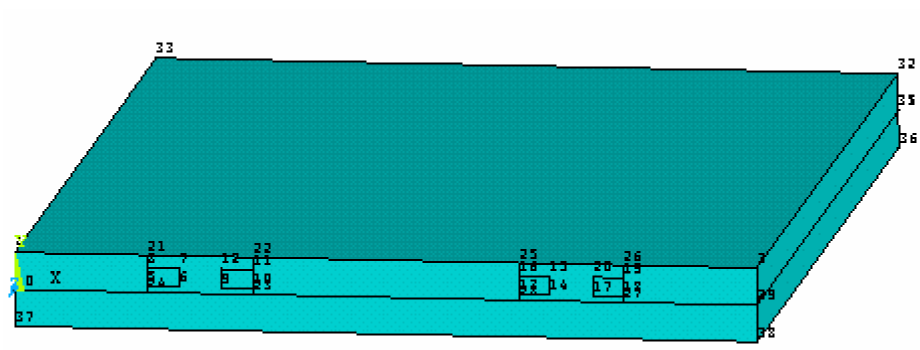

FIGURE 5: View of the 3-D Model with Four Strands, CTCP and Subbase Layers

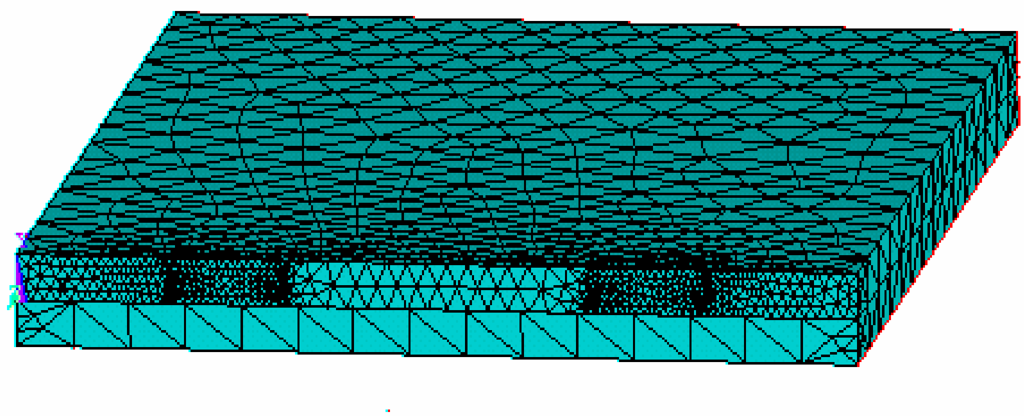

FIGURE 6: Three Dimensional Mesh of the Structure

The friction behavior between the CTCP slab and the subbase layer was modeled with the contact elements. Contact problems are highly non-linear and require significant computer resources to solve. This was another reason for using small structural dimensions. The temperature and tangential velocity effects on the friction were neglected. Classical Coulomb friction was considered.

The contact problems were categorized into two general classes in the software: rigid-to-flexible and flexible-to-flexible. In the FE analysis the rigid-to-flexible contact option was used since one of the contacting surfaces, CTCP, was assumed as rigid because it had a much higher stiffness relative to the deformable body, subbase, it is in contact. The software supported three contact models: node-to-node, node-to-surface, and surface-to-surface. Each type of model uses a different set of contact elements. In this analysis, surface-to-surface option was used. The surface-to-surface elements have several advantages over the node-to-surface elements. These elements:

1. Support lower and higher order elements on the surface;
2. Support large deformations;
3. Provide better contact results needed for normal pressure and friction stresses;
4. Have no restrictions on the shape of the target surface; and
5. Require fewer contact elements than the node-to-surface elements.

Contact elements were generated using the contact pair option in order to include surface-to-surface and rigid-to-flexible options. Contact pair consists of target surface and contact surface. In this research, the TARGE170 and CONTA174 elements were used to represent target surface and contact surface, respectively. Elastic Coulomb friction was allowed, where sliding was along the target base. The coefficient of friction, μ, was chosen as 0.5.

1.8.2 Loading and Boundary Conditions

Two loading positions were considered. The CTCP edges were subjected to 119 kN (26,800 lbs) force acting +30 and −30 degree angles with the z direction. The strands along the CTCP edge was primarily been set at 1.5 m (5 feet) between +30 strands and 1.5 m (5 feet) between −30 strands. The strands were also set so that the −30 and +30 strands were opposing each other in compression with spacing of 0.3 m (1 foot). The bottom of the subbase was constrained in the x, y, z directions. Symmetry option was used at the centerline of the 7.6 m wide CTCP pavement (3.8 m width was modeled in this study).

1.8.3 Finite Element Analysis Results

Figure 7 shows the compressive and tensile stress distributions in the z direction. The stress distributions at the loading surface and at the center line of the CTCP are of particular interest in this study.

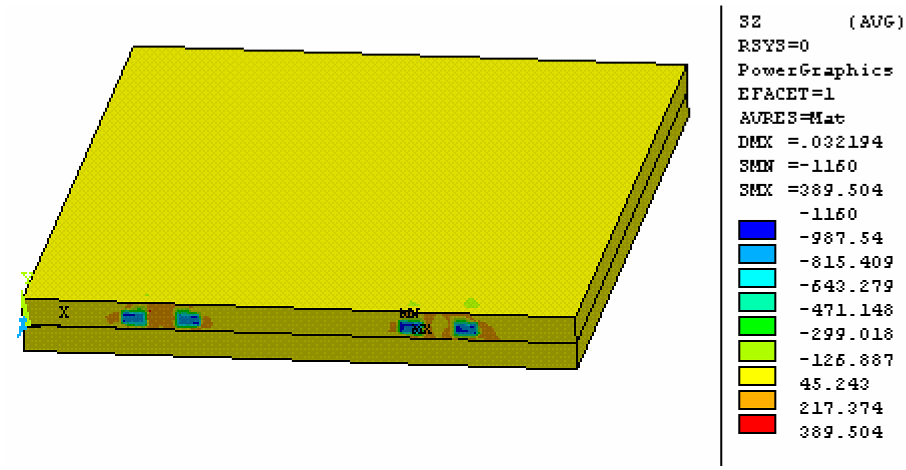

FIGURE 7: Stress Distribution in the z-Direction (toward the centerline)

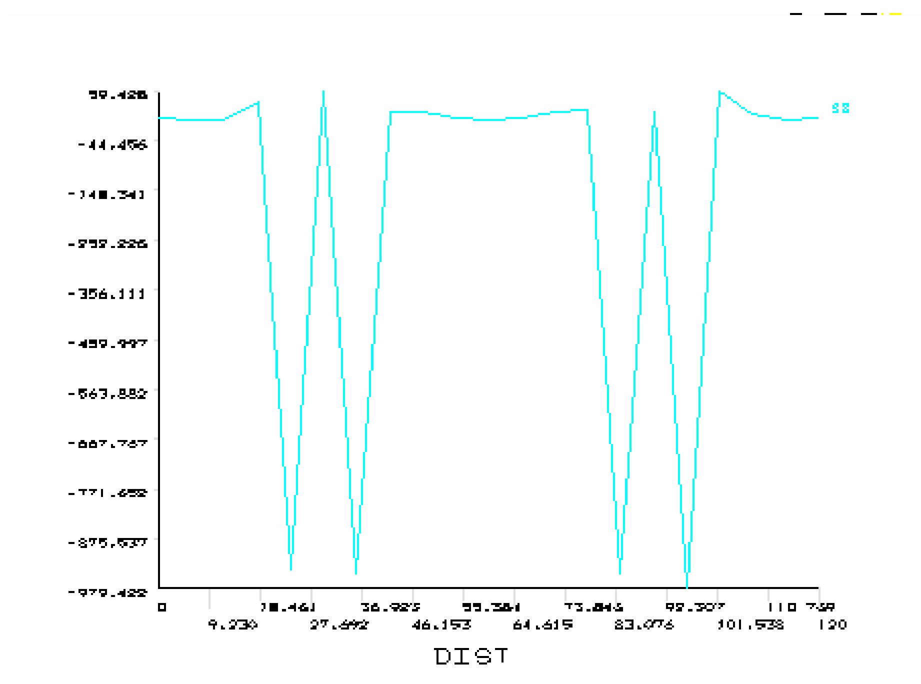

FIGURE 8: Stress Distribution on the Loading Edge in the z Direction

Figure 8 shows the stress distribution in the z direction (toward centerline) through the loading edge. The "-" and "+" symbols refer to compressive and tensile stresses, respectively. Stress distribution at the loading surface revealed that the maximum tensile stress occurred between two opposing strands. The stress values were as high as 400 kPa (55 psi) which is much less than 80% of the concrete modulus of rupture, 3 MPa (440 psi) suggested by ACI. The lowest tensile stress, 120 kPa (17 psi), occurred at the middle of the loading edge. As expected, the maximum compression stress of 6.2 Mpa (approximately 900 psi) occurred at center of the strands on the loading edge. The compressive stress at center line of the CTCP slab varied from 500 kPa to 600 kPa (80 psi to 90 psi).

Figures 9 and 10 show the horizontal displacements in the z direction toward the centerline. The "-" symbols refer to the displacements in the direction of the centerline. The maximum displacement occurred at the strand locations and was 0.04 mm (0.0015 inch). As expected, no displacements occurred at the centerline.

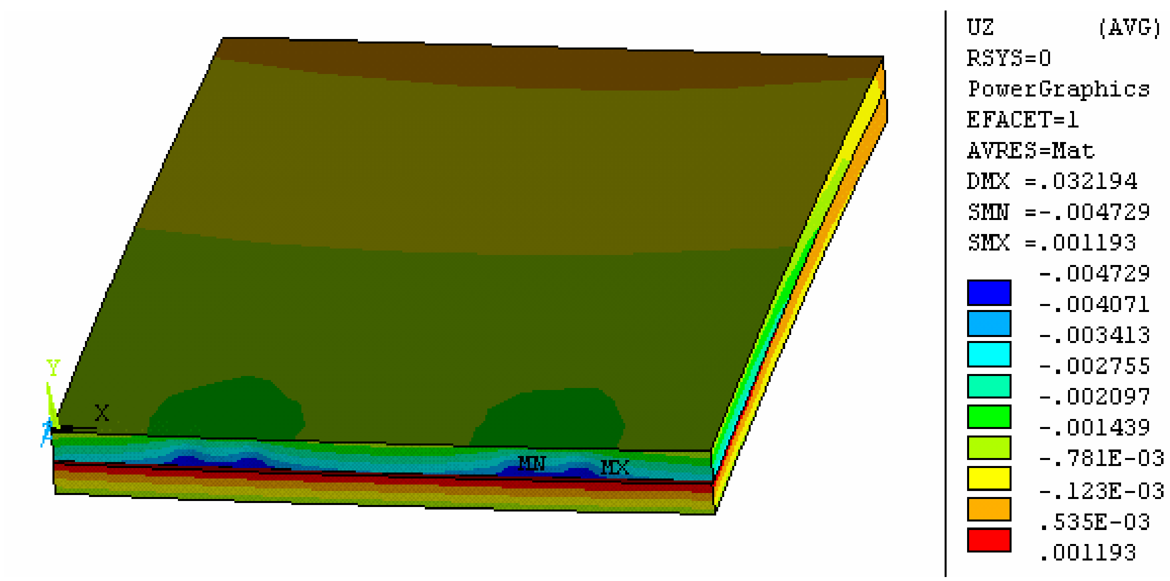

FIGURE 9: Displacements on the Loading Edge and the Centerline of the CTCP slab in the z direction

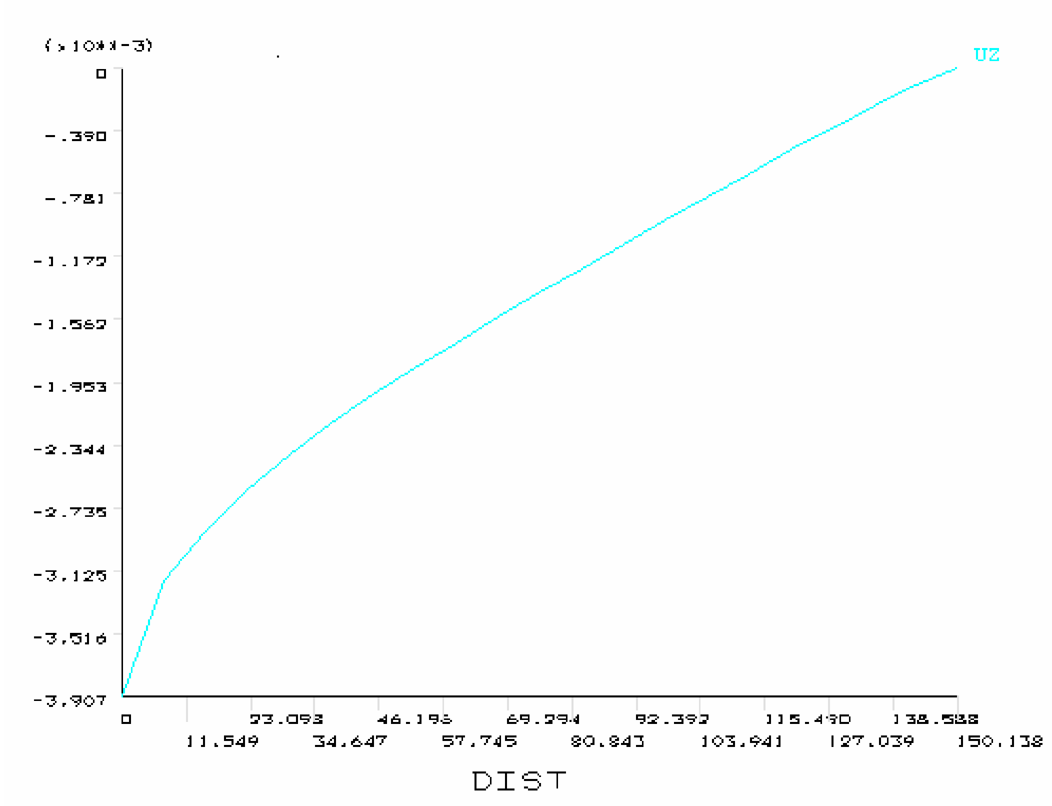

FIGURE 10: Displacements in the z Direction Toward the Centerline of the CTCP Slab

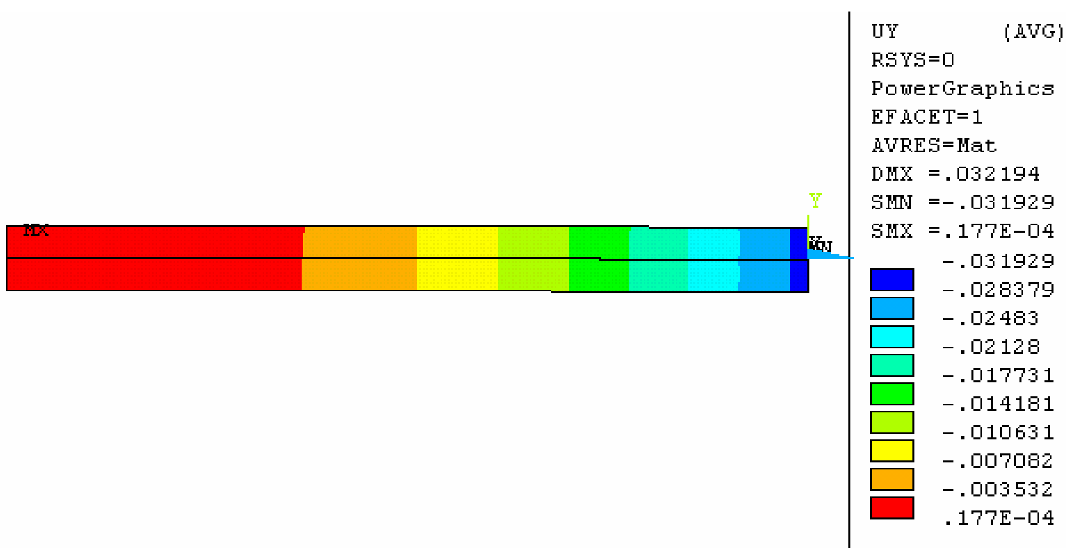

FIGURE 11: Curling of the CTCP Slab Due to Compressive Load

Figure 11 shows the vertical deflection in the y direction which would indicate upward or downward curling. It can be noticed that the CTCP is curling down as per the design expectation due to the eccentricity of load application. The maximum curling was at the loaded edge and was 0.075 mm (0.003 inches).

1.9 Conclusions and Future Research

The fundamental design and some advanced analyses for the Cross Tensioned Concrete Pavements (CTCP) are outlined. Finite element analyses were done using ANSYS to investigate the stresses in CTCP. The results show that the maximum tensile stress at the crossing of the strands near the pavement edge is much lower the recommended allowable stress. The tensile stress between the opposing bearing plates is also reasonable. The proposed design also resulted in maximum compressive stresses and displacements as per design expectation. Thus, the proposed CTCP design appears to be a feasible design solution for longer lasting concrete pavement. However, constructability of this innovative design using current paving technology needs to be investigated.

REFERENCES

1. Darter, M. I., Mueller, A. L., Peshkin, D. G. and Smith, K. D. An Eleven-Year Evaluation of Arizona's Pre-stressed Pavement. *Proceedings of the 4th International International Conference on Concrete Pavement Design and Rehabilitation,* Purdue University, West Lafayette, IN, April 18-20, pp. 561-575.
2. Hanna, A.N., Nussbaum, P.J., Arriyavat, P., Teseng, J.J.and Friberg, B.F. *Technological Review of Pre-stressed Pavements* Interim Research Report, FHWA-RD-77-8, 1976.
3. Huang, Yang H. *Pavement Analysis and Design*, Englewood Cliffs, New Jersey, 1993
4. Morris, Gene R. *Full-Depth, Full Width Designs and Pre-stressed Concrete Pave the Way for Arizona's Highways*, Civil Engineering, 1978.
5. Powers, R.L. and Zaniewski, J.P. Nine-Year Performance Evaluation of Arizona's Prestressed Concrete Pavement. Paper presented at the Annual Meeting of the Transportation Research Board, National Research Council, Washington D.C., January 1987.
6. Sargious, M. and Wang, S.K. *Economical Design of Pre-stressed Concrete Pavements.* Journal of The Prestressed Concrete Institute, Vol. 16, No., July/August 1971, pp. 64-79.
7. The American Concrete Institute (ACI). *Recommendations for Designing Pre-stressed Concrete Pavements.* Committee 325 Report, ACI Structural Journal, July-August 1988. pp. 451-470.