AN ELEVEN-YEAR EVALUATION OF ARIZONA'S PRESTRESSED PAVEMENT

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INTRODUCTION

While prestressing is a familiar concept in the construction of concrete structures and is frequently used in highway bridge construction, the concept has found very limited application in highway construction in this country. Nevertheless, prestressed or post-tensioned concrete pavements are an intriguing solution to many of the joint-related performance problems experienced by jointed concrete pavements (JCP), such as joint faulting and joint spalling.

The first field construction of a prestressed pavement was at the Orly Airport, Paris, in 1946. Since then, numerous applications of this technology have been made at European airports. The Europeans also took the lead on the application of the prestressed concept to highway pavements, constructing over 30 of these pavements during the late 1940's, and on into the 1950's and 1960's. A Federal Highway Administration (FHWA) review of prestressed concrete pavements in 1976 showed that six airport projects in the U.S. were constructed during the 1950's using this technology (1). Additionally, four roadway demonstration projects were built during the early 1970's; three highway pavements and an access road at Dulles Airport.

Prestressing is actually a catch-all term to describe several different methods of construction. These include post-stressed, pretensioned, and post-tensioned slabs. Common to these methods is the inducement of a compressive stress in the slab by mechanical means. The continual compressive stress applied to the pavement permits the slab to withstand higher flexural stresses than a typical concrete slab of similar thickness and conventional design. This stress has been induced longitudinally, longitudinally and transversely, and diagonally. Pretensioning and post-tensioning involve the use of wires, strands, or bars; poststressing is accomplished without strands, by applying a compressive force at the ends of the cast slabs. It is common practice to refer to all of these procedures generically as "prestressing," even though the methods involved in their construction are not at all similar.

As part of a number of experimental projects built on S.R. 360 (Superstition Freeway) in Phoenix, Arizona, a post-tensioned PCC pavement was designed and constructed in 1977 in both the eastbound and westbound lanes. Within the experimental prestressed section, several secondary factors were evaluated, including single vs. double sheets of polyethylene between the base and slab, and slab lengths varying from 207 feet to 500 feet. In 1988, the Arizona Department of Transportation (ADOT) contracted for an evaluation of the design and rehabilitation of concrete pavements in the Phoenix area. In March and April 1988, a project team evaluated and tested the prestressed pavement. A field survey was conducted at night under lane closure, as this was the only time that ADOT permits closing of the highway. This survey included a visual distress summary, including mapping of all distresses, and a roughness survey with a Mays Meter. Deflections were taken with a Falling Weight Deflectometer (FWD) at intervals of every 100 ft in the wheel path and center slab. Deflections were measured across the anchor slabs at the slab corner and in the wheel path. In all, over 6000 ft of this pavement was surveyed in the eastbound direction. The results of that evaluation, and general observations concerning prestressed pavements, are presented here.

THEORY

Both the design and expected performance of prestressed pavements are quite different from either "conventional" jointed concrete pavements, or even continuously reinforced concrete pavements (CRCP), which prestressed pavements most closely resemble. The assumptions and general approaches for design and expected performance are discussed below.

PRESTRESSED SLAB DESIGN

General procedures for the calculation of stresses in the elastic range for a prestressed pavement follow the principle that the combination of concrete flexural strength and applied compressive stress are greater than the induced stresses. These applied stresses include temperature and moisture gradient stresses, friction stresses between the base and the slab, and the stresses caused by an applied load. These inputs are summarized in the following equation (2, 3):

$$f_t + f_p > f_{c+w} + f_f + f_l \qquad (1)$$

where:

f_t = flexural strength of the concrete
f_p = applied prestressed compression
f_{c+w} = curling and warping induced stresses
f_f = frictional restraint between the slab and base
f_l = stress due to the applied load

Design in the elastic range assumes that no cracking of the slab will occur.

Thickness design has been proposed as that amount of concrete sufficient to provide adequate coverage for the prestressing steel. Recent work by the American Concrete Institute (ACI) suggests that the thickness of the prestressed pavement should be slightly greater than 0.65 of the thickness of an alternative plain concrete pavement for highways and that the edge deflection under load should not exceed 0.03 in under heavy traffic (2). The slab length is determined by the calculated prestress at the mid-length of the slab. Friberg recommended a minimum mid-slab prestress of 50 psi (1); ACI recommends a minimum of 100 psi (3). Different researchers have proposed solutions to each of the inputs to the general design equation shown above based on both a purely theoretical approach and a combination of theoretical and practical approaches. These are discussed in greater detail in the FHWA review (1).

PRESTRESSED PAVEMENT PERFORMANCE

The elimination of transverse joints removes a source of many of the distresses typically found on JCP, including faulting, spalling, and sealant deterioration. With the elimination of these distresses, secondary distresses, such as pumping or corner breaks, do not occur. The construction of a pavement without either longitudinal joints or the joint at the lane-shoulder interface effectively removes the most significant source of moisture entering the pavement system and thus greatly reduces the potential for moisture-related damage to the pavement. Transverse cracks which do form in prestressed pavements are expected to be held tightly together by the compressive stress applied by the post-tensioning strands. The serviceability of prestressed pavements is expected to be higher, as there are no joints to deteriorate and cause roughness and fewer of the common causes of other distresses.

In theory, therefore, there are strong performance-based reasons to build prestressed pavements. Their design promises a smooth, distress-free ride compared to more conventional pavement designs. They are also more resistant to flexural stress, as long as they do not lose their applied stress.

ADOT's PRESTRESSED SLAB DESIGN

The design of the prestressed project on Superstition Freeway followed the general procedures described in the FHWA review (4). Thickness design was based on similar projects which preceded it and on recommendations of a consultant and FHWA. A slab thickness of six inches was selected. A minimum support value of 300 pci was recommended for the base, based on minimum deflection requirements. Actual plate bearing tests on the subgrade showed a support value of 100 pci. The subgrade was a silty-clay, classified as an AASHTO A-6. Based on these results and using PCA inputs, a 4-in cement-treated base (CTB) was recommended. This was replaced in construction by a 4-in layer of lean concrete, at the contractor's request.

A slab length of 400 ft was selected and then verified using calculations. The prestressing steel used was 1/2-in diameter seven-wire strands with a yield strength of 270 ksi. The steel was placed one-half inch below mid-depth and at two-foot centers. The strands were coated with grease to inhibit corrosion and protected from the concrete by a thin extruded jacket. Two sheets of polyethylene were placed between the concrete slab and the lean concrete base to reduce frictional drag. The sheets were 6 mils thick and had a lab-tested coefficient of friction of 0.20. Sand was broomed onto the base as needed to provide a level surface prior to placement of the polyethylene. A cross-section of the pavement design is shown in Figure 1.

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 entire width of the pavement was prestressed. Texture was applied longitudinally by a nylon bristle broom. Longitudinal joint tie bars, consisting of 42-in lengths of No. 4 rebar, were placed mechanically on 5-ft centers at both the pavement centerline and at the lane-shoulder joint. The lane-shoulder joint and the longitudinal joint were both formed with a plastic ribbon placed during the paving operation. Keyways were placed on the outer edges of the concrete. On the outer lane, a concrete curb and gutter was placed adjacent to the shoulder. The inner shoulder was originally placed as asphalt concrete; in 1984 this was removed and a third lane and inner shoulder of CRCP were constructed along the prestressed sections.

The post-tensioning was applied at gap slabs located at the ends of each prestressed slab. A space of 8 ft was left between each prestressed slab to accommodate the end hardware and to provide space for application of the post-tensioning stress. Load transfer between the prestressed slab and the gap slab was provided by 1.25-in diameter dowel bars, 18 in long, placed on 12-in centers. Both longitudinal and transverse reinforcing steel were placed mid-depth in the 10-in thick slabs. Joint openings of 1.5 in were provided at either end of the slab to allow for movement of the large slabs. These were later filled with neoprene sealants. Figure 2 shows the design of the gap slabs. Excellent details of the construction procedure are presented by Morris and Emery (4, 5).

VARIATIONS IN ADOT'S DESIGN

While the design slab length was nominally 400 ft, several slabs were paved at different lengths to evaluate their performance. There were two 500-ft slabs placed, and four slabs ranging in length from 207 ft to 307 ft were constructed adjacent to fixed structures. Two slabs were placed on a single layer of polyethylene rather than a double layer. In addition, two other areas are identified where there is a single layer of polyethylene due to construction difficulties.

GENERAL CONDITIONS

The climatic conditions in the Phoenix area are worthy of mention because they tend to the extreme more than in other locations. The same can be said of the traffic to which the pavements in the urban area are subjected. Because traffic and environment can have a great effect on pavement performance, these deserve separate consideration.

ENVIRONMENT

This pavement is located in Phoenix, in the south-central part of Arizona. This area is classified as a dry, non-freeze environmental zone, has a Thornthwaite Moisture Index of -47, and a Corps of Engineers Freezing Index of 0. The average annual rainfall is eight inches. The highest average daily maximum temperature is 105° F and the lowest average daily minimum temperature is 36° F. These parameters describe a hot, arid environment, with a deficit of free moisture and an absence of freezing conditions. Pavements built in this area are subject to a large temperature variation over a 24-hour period, but very little moisture.

TRAFFIC

The design traffic estimated by ADOT was a one-way average daily traffic (ADT) of 26,000 vehicles, of which 1% were heavy truck traffic. The twenty-year design traffic was 191 million vehicles, or approximately 2.3 million 18-kip equivalent single axle loads (ESAL). Traffic data collected several months after the pavement opened showed a one-way ADT of 62,000, and 2% heavy trucks. This had risen to a one-way ADT of 80,000 and 3.5% trucks by 1984, when the third lane was added. The 1986 ADT was 98,000 and a 1987 estimate of ADT is almost 109,000 vehicles.

An approximate cumulative ESAL calculation was made using loadometer data reported in W-4 tables, and typical lane distributions. This estimate must be considered rough because there are no loadometer stations in the urban area and truck factors are computed from rural traffic. Truck factors, which are a computed average ESAL/truck classification, will vary from location to location. Also, there are no reliable truck counts currently available. ADOT is in the process of collecting Weigh-in-Motion data on this pavement which, when obtained, will provide a much more accurate indication of the truck factors. This data also will provide an estimate of the ADT, percent trucks, lane distribution, and directional distribution for comparison purposes.

As of 1988, ADOT was predicting a 1.9% growth in their ADT. If this rate is a simple (non-compounded) growth rate, projecting through the original twenty-year design life from 1988 on, by 1997 this pavement will have been subjected to approximately 650 million vehicles, or approximately 15 million ESAL's. This is over six times the original design total. The cumulative ESAL calculations, shown in Table 1, are even more revealing. Taking into account heavy truck loadings, they show that, to date, this pavement has been subjected to a very large volume of heavy loads, far in excess of the design cumulative ESAL. A rough approximation is that by 1984 the section had already exceeded its design ESAL. By the end of 1988, four years later, the outer lane has been subjected to almost twice the design ESAL's. This is only an estimate, as the actual truck factors are not available.

SUMMARY OF PREVIOUS EVALUATIONS OF ARIZONA'S PRESTRESSED PAVEMENT

The performance of this experimental section was formally evaluated three times and informally evaluated two other times. The first evaluation was performed during and shortly after the construction of the pavement. The parameters examined in that evaluation include: slab movements due to both temperature variations and the stressing operation; joint width variations; slab curling; and slab cracking (4, 5). The second evaluation was performed nine years after the pavement was constructed. In that evaluation, the surface condition/joint performance, skid condition, and pavement roughness were studied (6). The most recent evaluation was conducted eleven years after the pavement was constructed.

INITIAL PERFORMANCE

Temperature measurements were taken with thermistors embedded in various positions and depths in the slabs. The results indicate that the slab temperature was fairly constant in the morning (measurements were taken at 7:00 am) and that the temperature at the top of the slab was significantly higher than that at the bottom during the afternoon (measurements were taken at 3:00 pm). The maximum temperature difference observed was 12° F.

The slab length was measured prior to the application of the jacking force, after each prestressing stage, 28 days after construction, and periodically thereafter. A typical slab shortened 0.29 in upon initial prestressing, 0.76 in upon final prestressing, and 0.94 in two months after construction. The average change in length two months after construction was 0.70 in, with a low of 0.29 in and a high of 1.2 in . The change in length after prestressing is due to the application of the prestressing force. The long-term change in length may be due to shrinkage, creep, and frictional resistance between the slab and base course.

Joint widths at the gap slabs were measured at the same time as the slab lengths were measured. The joint width was designed for a maximum movement of two inches. The joint width data collected does not exceed two inches; however, measurements were not taken during periods of cool temperature.

Slab curling was monitored after construction. Two slabs were instrumented at the slab corners. The air and concrete temperatures and curling data were gathered for 32 hours. As expected, the slab corners curled downward as the surface temperature increased.

Crack surveys were performed after placement of the slabs, after each application of the jacking force, approximately one week following the final prestressing, and two months after construction. The transverse cracks that were noted were concentrated in the middle of the slabs. The crack openings ranged from 1/64-in to 1/4-in. Nearly all of the initial cracks closed upon the final prestressing.

NINE-YEAR PERFORMANCE

Both the eastbound and westbound directions were evaluated during the nine-year study. As measured by the Mu-Meter and Mays Ride Meter, there was a difference in performance between the two directions, with the westbound lanes performing better than the eastbound lanes. This is attributed to the fact that the eastbound direction was constructed first and the information learned from its construction was applied to the construction of the westbound lanes. The results from the eastbound lane follow, since it is felt that more can be learned from the pavement showing more distress.

Joint Distress

In the visual pavement survey, it was noted that the most obvious problem was at the joints between the prestressed pavements and the gap slabs. Ninety percent of the joints in the eastbound direction were patched the full width of the pavement. It was concluded that the combination of impact loading at the joints and poor consolidation of the concrete around the joint hardware caused the joint failures.

Longitudinal Cracking

During the summer of 1985 one prestressing strand anchor failed. The bearing plate was removed and an asphalt patch was installed. A longitudinal crack was observed near the broken strand. The longitudinal joint between the shoulder and outer lane was formed by a plastic insert placed during construction. It was observed that this crack had widened considerably near the broken strand.

A section of the pavement was removed and replaced immediately after construction due to strand failure. The continuity of the other strands was maintained for the repair. The repair was reported to be performing well; however, longitudinal cracks were beginning to appear in the original pavement on either side of the repair.

Other Pavement Distresses

Several slabs and gap slabs were reported to have experienced corner breaks. This was attributed to the high curling stresses in the slabs, the restraint of the load transfer devices, and the infiltration of incompressible material into the joint. Also, many of the transverse joint patches were failing at the time of the survey.

Many surface defects were observed, including pop outs, spalling of the cracks, poor concrete consolidation, depressions caused by paving and texturing operations, and map cracking.

Roughness

A roughness index on the experimental section was measured immediately after construction using the Mays Ride Meter. The initial roughness of the section was 50% to 80% higher than the conventional concrete pavements constructed in Arizona.

It was reported that surface roughness increased for three years after the initial construction and then stabilized with time.

Surface Friction

A Mu-Meter was used to measure the surface frictional resistance. As expected, the initial friction number was high. The friction number decreased with time and stabilized to a constant value of 45.

11-YEAR PERFORMANCE OF ARIZONA'S PRESTRESSED PAVEMENT

In the spring of 1988, deflection testing was performed with a Falling Weight Deflectometer (FWD) on the outer lane of the prestressed section for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. A visual pavement survey of the both lanes was conducted in conjunction with the NDT. The Roughness Index (RI) was measured in both lanes using the Mays Roughness Meter. Also, a panel of users rated the pavement's rideability (PSR). The survey was conducted solely in the eastbound lanes.

It is difficult to draw conclusions regarding performance differences between the various slab lengths due to the number of slabs constructed of each length. Fifteen of the prestressed slabs in the eastbound direction were constructed 402 feet long. One slab in the eastbound direction was constructed 502 feet long, and two slabs (eastbound), were constructed 207 and 309 feet long. These odd-length slabs were placed at the end of the experimental project and straddled a bridge. Due to the small sample size (only one slab of each size) and the placement of these slabs adjacent to structures, the data collected from these odd-length slabs may, in some cases, be misleading.

The results of the testing and evaluation of the prestressed section will be discussed and the results will also be compared and contrasted to the conventional concrete pavements built on the Superstition Freeway during a similar time period.

STRUCTURAL EVALUATION

Nondestructive testing (NDT) using the Falling Weight Deflectometer (FWD) was performed on the sections of the prestressed pavements in the eastbound direction. The testing was performed in the outer lane only. The test pattern was established as follows: every 100-feet in the wheel path, every 100-feet in the center of the slab (four-feet from any crack or joint), and at every approach and leave corner of the gap slabs.

Layer Strength Characteristics

The layer strength characteristics were determined through finite element backcalculation procedures using the center slab deflections. Table 2a shows the backcalculated E for the surface and the composite k-value on top of the base course. As expected, no discernible difference can be seen in the backcalculated materials properties for the different slab lengths. The composite k-value is typical of lean concrete base (LCB) built on an A-6 subgrade; however, it is slightly lower than the design support value of 300 pci.

The surface modulus values and the support values for the base course for conventional concrete pavements are summarized in Table 2b. These conventional pavements were constructed on the Superstition Freeway between 1981 and 1985 and consist of a 9 in concrete surface over a 4 in LCB. The modulus of elasticity of the prestressed surface is about the same as the surface modulus of conventional jointed concrete pavements. As expected, the k-value on top of the lean concrete base/A-6 subgrade combination is equivalent to that of the LCB's constructed in conventional pavements.

Load Transfer Efficiency

The use of 1.25-in diameter stainless steel dowels was mandated in order to provide for load transfer from the prestressed slabs to the gap slabs. It was felt that these would maintain alignment and reduce slab end deflections. The load transfer efficiencies across the joints between the prestressed and gap slabs were calculated. Table 2 shows the average load transfer efficiency for all sections. Since dowels are expected to provide load transfer efficiencies of greater than 70%, the load transfer between the gap and prestressed slabs is somewhat low, especially in the longer slabs.

The shorter slab length (207 feet and 307 feet) sections appeared to perform better in terms of load transfer efficiency across the transverse joints. The higher load transfer efficiencies in the shorter slabs may be due to the amount of movement and the development of tensile stresses in these slabs relative to the longer slabs. In order for the slab to move, it must overcome the frictional resistance force between the bottom of the slab and the top of the lean concrete base course. As the slab tries to move against this frictional resistance force, a tensile stress develops. The longer slabs, due to increased surface area between the surface/base interface, may be able to overcome this resistance force, therefore sustaining higher tensile stress at the joints. The higher tensile stress at the joint of the longer-jointed sections may cause more rapid deterioration of the load transfer system. Thus, the shorter-jointed sections experience higher load transfer efficiencies.

Even though the prestressed sections had dowel bars between the prestressed slabs and the gap slabs, the load transfer efficiencies calculated for the joints in the prestressed sections were similar to the load transfer efficiencies calculated for nondoweled conventional concrete pavements. As explained above, this may be attributed to the tremendous movement/tensile stresses developed in these long slabs. The joint width of 1.5 in may also help to reduce the load transfer efficiency.

Erodibility/Loss of Support

An erodiblity analysis was performed using the void detection procedures outlined in NCHRP 1-21 (7). Table 2 shows the results of this analysis. All of the prestressed sections exhibit a substantial amount of loss of support/erosion beneath the slab corners with no significant difference seen between the various slab lengths. Fifty to sixty percent of the corners exhibited loss of support.

The loss of support experienced by the prestressed sections is similar to that found on the conventional JPCP slabs on LCB constructed on the Superstition Freeway in the same time period. The loss of support at the slab corners in this arid, moisture-free environment is thought to occur as a result of thermal curling movements at the slab corners resulting in the migration of fines.

DETERIORATION OF THE PRESTRESSED PAVEMENT

Results of the visual distress survey in terms of the major pavement performance indicators are summarized in Table 3 for the outer lane and Table 4 for the second lane. These tables distinguish between the performance of the gap slabs and the performance of the prestressed slabs because most of the distresses occurred at the gap slabs. The relative performance of the outer lane of these pavements are discussed in the following sections.

It is important to note that the number of slabs surveyed of the standard slab size (402-foot) is significantly greater than the other slab sizes. The limited number of the other slab sizes surveyed prevents a valid comparison being made between the various slab lengths in some instances.

Gap Slab and Joint Performance

There has been considerable deterioration of the gap slabs and their associated joints. Spalling and corner breaks have occurred at the joints, resulting in extensive joint patching. Nearly every joint face surveyed (30 total) had some patching. Several types of patches were used, including bituminous materials, epoxy, and cementitious materials. These patches are found on both the prestressed and gap slab side of the joint, and across the full width of the pavement.

Due to the lack of data on the shorter- and longer-jointed sections, it is difficult to draw valid conclusions regarding performance differences with respect to slab length. However, conclusions can be drawn with respect to the general performance of the gap slabs and joints. The gap slabs and joints have performed poorly. All of the neoprene joint sealant was in poor condition and all of the joints were filled with incompressibles. In many cases, the joint sealant was entirely missing from the joint. The deterioration of the gap slab and joints has occurred because of poor consolidation of the concrete around the joints, in combination with the large stresses at the joints. The infiltration of incompressibles into the joints adds to this deterioration by not allowing the joints to open and close properly. Freedom of joint movement is especially important in long-jointed pavements.

Although containing far fewer joints, the amount and severity of joint deterioration in the prestressed sections is far greater than that found in conventional, short-jointed JPCP constructed during the same time period. Table 5 illustrates the results of a distress survey for a conventional concrete pavement.

Transverse Cracking

The amount of transverse cracking, reported in terms of number of transverse cracks (all severity levels) per mile experienced by the various sections, is fairly high for the 307-ft, 402-ft, and 502-ft sections. The 207-foot slab experienced no transverse cracking. Again, due to the lack of data on the odd-length sections it is not possible to draw valid conclusions from the data.

Problems encountered during the paving operation resulted in many starts and stops of the paver. This caused the creation of shallow undulations, several inches wide, extending transversely across the pavement. Many of the transverse cracks were induced by the undulations left in the pavement. These are noted as "paver stops" and are summarized in Tables 3 and 4. If the pavement stresses increase to the point were the pavement will crack, the cracking will probably occur at these undulations because the pavement is thinner beneath them.

The prestressed sections showed significantly more transverse cracking than conventional concrete pavements. Transverse cracking is expected in long-jointed pavements; the cracks should be held tight by the prestressing steel. The transverse cracks which developed in this pavement are due to both the problems encountered during the paving operation and the tensile stresses that develop in long-jointed pavements. They were all rated as low-severity cracks. No transverse cracking was observed in the other (short-jointed) pavements surveyed on the Superstition Freeway.

Longitudinal Cracking

The prestressed sections experienced substantial amounts of longitudinal cracking. Both low and medium severity longitudinal cracks were noted. The longitudinal cracks appear to have developed over the prestressing strands. Several of the cracks continued for 100 feet or more. The cause of these longitudinal cracks may be the increased stress around the prestressing strands or the improper placement of the strands. The prestressing strands and surrounding concrete are under high tensile stresses. If the tensile stresses in the concrete become too great, a crack may propagate upward. Also, the placement of the strands is critical. If the strands are too close to the surface or are at an improper angle, a crack may develop. It must be remembered that this is a very thin concrete pavement, and the longitudinal cracks that have formed may be a fatigue response of the concrete.

This pavement was built with post-tensioning strands in the longitudinal direction only. These will only keep tightly together cracks that are formed in the transverse direction. Longitudinal cracks, no matter how they were formed, are much more likely to increase in severity than transverse cracks.

The amount of longitudinal cracking is significantly higher than that experienced by conventional concrete pavements. The jointed plain concrete pavements surveyed experienced very little longitudinal cracking. The longitudinal cracking that was seen in the jointed plain concrete pavements was most likely due to late sawing of the longitudinal joint.

Roughness/Serviceability

The paving problems and the patching of the gap slabs and the joints are most likely the cause of the high roughness and low serviceability ratings of the prestressed sections. The undulations left by the frequent stopping of the paving operation caused a great deal of roughness. Also, the contractor was not required to grind after construction because it was felt that the pavement was too thin. Thus, surface roughness, which would normally be removed by grinding immediately after construction for acceptance, remains. The largest contributor to roughness is most likely the gap slabs/joints.

The prestressed sections had much higher roughness measurements than the conventional concrete pavements surveyed on the Superstition Freeway. However, there has been some grinding on these sections.

Other Pavement Distress

The lane/shoulder joint experienced low- to medium-severity spalling. This joint was formed using a plastic insert. Joints formed with inserts typically experience some degree of spalling due to the nature of their formation.

Other surface distresses were observed, including crazing, pop-outs, poor concrete consolidation, and surface finishing problems. The conventional concrete pavements surveyed experienced similar surface distresses.

COMPARISON OF OUTER LANE PERFORMANCE TO INNER LANE PERFORMANCE

Although being subjected to fewer traffic loadings than the outer lane, the performance of the inner lane sections provide an interesting comparison. Table 4 summarizes prestressed inner lane performance for the various slab sizes. The gap slab performance was poor in both lanes. The 402-foot slabs, which comprise 80% of the prestressed section surveyed, showed little difference in patching and spalling between the inner and outer lanes. This shows that the distresses are due more to a materials/construction problem than to load-related causes.

No discernible difference was observed in the amount of transverse cracking between the inner and outer lanes. Since the transverse cracking was most likely induced by the undulations which resulted from frequent stops in the paving operation, little difference was observed between the two lanes.

The amount of longitudinal cracking was much greater in the outer lane than in the inner lane. Increased traffic loadings in the outer lane may have caused a crack to propagate above a strand more quickly in that lane than in the inner lane.

LIFE-CYCLE COSTS

Prestressed pavements are substantially thinner than conventional concrete pavements of any other design. The savings in concrete, combined with the savings in joint costs, should more than offset the costs of the tendons, gap slabs and post-tensioning. Unfortunately, very little cost data is available. As shown in Table 6, the initial construction cost was very high by comparison to conventional designs. However, if more prestressed pavements were built, the initial cost of this type of construction would probably decrease, as contractors became more familiar with the design.

Several other costs must be considered to develop life-cycle costs. These include maintenance costs, rehabilitation costs, and user costs. The maintenance and rehabilitation costs are not available and user costs are difficult to quantify. It is important to note, however, that the costs to the user, in terms of discomfort and vehicle maintenance, are high for the prestressed section because of increased roughness.

The life expectancy of this type of pavement is not easily predictable. Very little performance information has been collected on prestressed pavements and none on pavements subjected to similar traffic and environment. As noted before, this pavement has already experienced its design life traffic and projections indicate that in 20 years it would experience over 650 million vehicles and perhaps 15 million ESAL's.

MAINTENANCE AND REHABILITATION

These pavements have posed a maintenance problem from the time of their initial construction until the present. During the initial application of the post-tensioning on one of the slabs, an area of poorly consolidated concrete was damaged and needed to be replaced, along with some of the joint hardware. A similar problem occurred over a larger area several months after the pavement was opened to traffic. It required the closing of the road for two weeks and replacement of a full-width section of pavement 16 ft long. The repair necessitated in 1985 is discussed earlier.

The major distress requiring maintenance that has been noted at the four-, six-, and nine-year evaluation, are failures at the joints. These failures involve the concrete in the prestressed slab immediately adjacent to the gap slab, the steel facing plate, and the sealant material. This continues to be the major maintenance problem, as evidenced both by the field survey and comments from the Area Maintenance Engineer. Their performance was documented in a preceding sections. At the time of the survey during April 1988, there was only one gap slab in the entire eastbound direction which did not exhibit any distress at all. The latest repair had been placed as recently as the morning before the survey.

It should be noted that if it were not for the unique design of this pavement, it would most certainly have been subjected to grinding. It is only the fact that the slab is very thin that keeps it from being ground. As discussed in the previous section, surface roughness is a predominant distress on the entire project.

CONCLUSIONS

While the traffic calculations presented earlier are a rough estimate, it is clear that the pavement has already far exceeded its 20-year design life. This supports the original design assumptions and suggests that theoretical design procedures concerning prestressed pavements can be applied in the field with success.

Performance theory suggests that this type of design would offer a distinct advantage over conventional jointed pavements. With fewer joints, there should be far fewer joint-related distresses and hence it was anticipated that a smoother ride would result. In reality, so many problems arose during the construction of the pavement that a large amount of roughness was actually built into the pavement. Depressions which were set into the concrete when the bristle broom used for surface texturing rested on the plastic mix numbered on the order of 100/mile. Some of these were deep enough to have initiated transverse cracking. It would have been very informative to have collected data from the pavement in the westbound direction. This other section may have provided a more reasonable basis for comparison with the performance of other designs in the area, as it was constructed with the benefit of knowledge gained from the construction of the eastbound lanes.

While there are far fewer joints on this prestressed pavement, most of the distresses requiring maintenance and contributing to the overall low rideability of this pavement occurred at the joint gap slabs. This is a result of both a poor design of the slab and a reported failure to obtain adequate concrete consolidation at the end of each slab. Consequently, almost all of the joints are patched, resealed, or are filled with incompressibles and spalled, waiting to be repaired.

There were not enough slabs of the different lengths to make a statistically sound comparison among them. One longer slab and two shorter slabs were included in the survey and any projections based on their performance would be misleading. From a cost standpoint, it would make sense to construct the slabs as long as possible, since the savings are obtained from reducing the number of joints, and performance is better on the slabs than at the joints. However, the shorter slabs will have less movement, which may result in less cracking and better joint performance. The comparison of single sheets of polyethylene to double sheets as a bondbreaker is not possible because of the small sample size. The construction reports recommended the use of a single sheet of polyethylene because no adverse effects were observed during construction or shortly after. Also, the use of a single sheet eliminated the construction difficulties experienced with double sheets (5, 6).

Since the design of this pavement, researchers have proposed several alternative approaches to the application of the post-tensioning stress and the design of the gap slab (8, 9). It is quite possible that an improved paving operation and an improved gap slab design would have eliminated many of the performance-related distresses associated with the gap slabs on this pavement.

This pavement was placed at a total cost of \$13.31/yd^2. With the cost of the lean concrete base added, the total comes to \$15.97/yd^2. Table 6 compared these costs to those of conventional pavement designs on the Superstition Freeway. As this is an experimental project, the costs are certainly higher than a regular concrete paving job. These costs would most likely come down if more were built.

These slabs are constructed substantially thinner than conventional concrete pavements and the savings in material costs should be reflected in the overall price. Most of the maintenance work has not been done on a contract basis and thus cost records are not kept. These pavements are not competitive on a life cycle cost basis unless they last significantly longer than traditional designs or their design is altered to correct the problems encountered with the gap slabs and joints. Other pavements constructed at the same time have not required any maintenance or rehabilitation to date.

The effects of the environment should be considered in assessing the performance of these pavements. It is commonly accepted that in the absence of excess moisture year round and without a freeze-thaw problem, pavements that perform well in Phoenix would not perform as well where these are a problem. Corrosion of the tendons in the slab could prove to be fatal to a prestressed pavement, whereas the same problem in JPCP or CRCP would not be as serious. Typical conventional pavements in this climate perform much better than predicted. This is shown in Figure 3, where expected performance was compared to actual performance for 53 in-service pavements in the dry non-freeze climatic region (10).

Prestressed pavements are not an obsolete new technology. Recent jobs have been completed or are planned at O'Hare Airport in Chicago and along a section of U.S. 220 in Bedford County, Pennsylvania (12). Clearly there is confidence in the paving community that future applications do exist for this technology.

Unfortunately, portions of the prestressed pavement in Phoenix are slated for removal in the near future. A victim of the rapid growth of the Phoenix area, this section is located where a new alignment is planned to be built. Since it has been subjected to far more loadings than any of the other three prestressed pavements in the United States, it would have been of enormous benefit to the pavement community to have the opportunity to continue to monitor the performance of this pavement over the coming years. Two three-slab sections in either direction will be left intact, however, and an opportunity will be provided to conduct forensic studies on the pavement as it is removed.

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DISCLAIMER

Table 1. ESAL Estimation for Arizona's Prestressed Pavement Section, S.R. 360.

YEAR	2-WAY ADT	% TRUCKS	LANE DIST.	TRUCK FACTOR	OUTER LANE ESAL's	OUTER LANE CUMULATIVE ESAL's
1977	62,000	0.020	0.72	0.974	158,700	158,700
1978	64,600	0.022	0.71	1.240	229,386	388,086
1979	67,100	0.024	0.70	1.247	259,750	647,836
1980	69,700	0.026	0.70	1.045	245,648	893,485
1981	72,300	0.029	0.70	1.045	276,046	1,169,531
1982	74,900	0.031	0.69	1.402	405,957	1,575,488
1983	77,400	0.033	0.57	1.343	355,755	1,931,243
1984	80,000	0.035	0.57	1.226	357,097	2,288,340
1985	89,000	0.037	0.56	1.226	413,719	2,702,058
1986	98,000	0.039	0.55	1.250	483,230	3,185,289
1987	107,000	0.041	0.54	1.275	556,610	3,741,899
1988	116,000	0.044	0.53	1.300	635,955	4,377,854

Table 2a. Backcalculated Layer Properties, Load Transfer Efficiencies, and Void Detection for the Prestressed Pavement Sections.

SLAB LENGTH, ft	E_{pcc}, psi	COMPOSITE K-VALUE, pci	LOAD TRANSFER EFFICIENCY, %[1]	SLAB CORNERS WITH VOIDS, %
207	5.81×10^6	252	68.1	50
309	5.71×10^6	266	52.3	50
402	5.45×10^6	241	40.9	58
502	5.71×10^6	258	40.2	50

Table 2b. Backcalculated Layer Properties, Load Transfer Efficiencies, and Void Detection for Conventional PCC (9" PCC/4" LCB) Constructed between 1981 & 1985.

SLAB LENGTH, ft	E_{pcc}, psi	COMPOSITE K-VALUE, pci	LOAD TRANSFER EFFICIENCY, %	SLAB CORNERS WITH VOIDS, %
13-15-17-15	5.49×10^6	309	50.8	37
13-15-17-15	5.93×10^6	251	42.3	60
13-15-17-15	6.73×10^6	218	27.4	90

[1] Load transfer efficiency is defined as the deflection of the unloaded slab/deflection of the loaded slab.

Table 3. Results of the Distress Survey for the Outer Lane of the Prestressed Pavement.

Performance Indicator	SLAB LENGTH, ft 207	309	402	502
Gap Slab/Joint				
% Joint Face Area Patched	41.7	12.5	27.4	0.0
Patching (Sq ft/Joint Face)	5.0	0.75	3.0	0.0
% Patch & Joint Face Spall	3.3	0.0	1.9	8.3
Prestressed Pavement				
Transverse Cracks/mile	0	34	31	21
Longitudinal Cracks, ft/mile	1530	938	617	2341
Paver Stops/mile	179	273	99	217
Patching, ft^2/mile	0	0	0	0
Longitudinal Joint Spall (ft/mile)	0	0	0	0
Pumping	NONE	NONE	NONE	NONE
Roughness Index, in/mi	136	122	100	122
PSR	3.9	3.9	3.5	4.0
Length of Survey, ft	210	309	4934	512

Table 4. Results of the Distress Survey for the Inner Lane of the Prestressed Pavement.

Performance Indicator	SLAB LENGTH, ft 207	309	402	502
Gap Slab/Joint				
% Joint Face Area Patched	0	0	29.3	0
Patching (Sq ft/Joint Face)	0	0	3.4	0
% Patch & Joint Face Spall	0	0	0.3	0
Prestressed Pavement				
Transverse Cracks/mile	0	34	24	25
Longitudinal Cracks, ft/mile	0	0	43	0
Paver Stops/mile	255	290	99	236
Patching, ft^2/mile	0	0	5.6	0
Longitudinal Joint Spall, ft/mile	0	0	0	0
Pumping	NONE	NONE	NONE	NONE
Roughness Index, in/mi	131	119	94	119
PSR	4.0	3.9	4.1	3.9
Length of Survey, ft	210	309	4934	829

Table 5. Results of the Distress Survey for a Conventional Concrete Pavement (9" Concrete Surface/4" LCB).

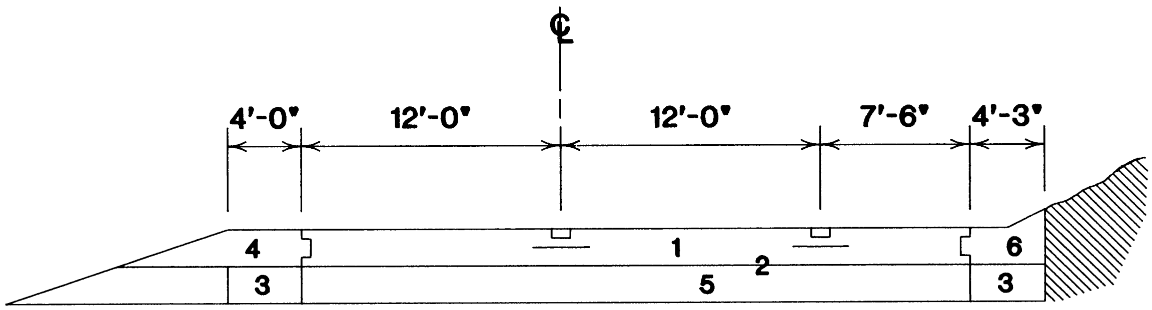
VARIABLE	OUTER LANE	INNER LANE
Transverse Cracks/mile	0	0
Longitudinal Cracks (ft/mile)	0	0
Patching (Ft^2/mile)	0	0
Transverse Joint Spall, %	16.7	12.5
Longitudinal Joint Spall (ft/mile)	0	0
Pumping	NONE	NONE
Roughness Index, in/mi	62	70
PSR	4.4	4.5

Table 6. Initial Construction Costs for the Prestressed Pavement and Other Conventional Designs in the Phoenix Area.

YEAR CONSTRUCTED	DESIGN	COST, $\$/yd^2$
1977	6 inch Prestressed 4 inch LCB	$15.97
Early 1970's	9 inch PCC 4 inch CTB	$9.00 - $10.00
Early 1970's	11 or 13 inch PCC on grade	$9.00 - $10.00

NOTE: The costs for the early 1970's pavements were estimated by ADOT personnal.

PRESTRESSED CONCRETE PAVEMENT



1. 6" P.C.C.P. w/16-1/2" Ø 270 ksi STRANDS
2. SAND LEVELING COURSE + DOUBLE LAYER H.S. 6 MILL POLYETHYLENE
3. 4" CEMENTED TREATED BASE COURSE
4. 6" ASPHALTIC CONCRETE
5. 4" LEAN CONCRETE BASE
6. SPECIAL CURB AND GUTTER

Figure 1. Prestressed Concrete Pavement Cross-Section (5).

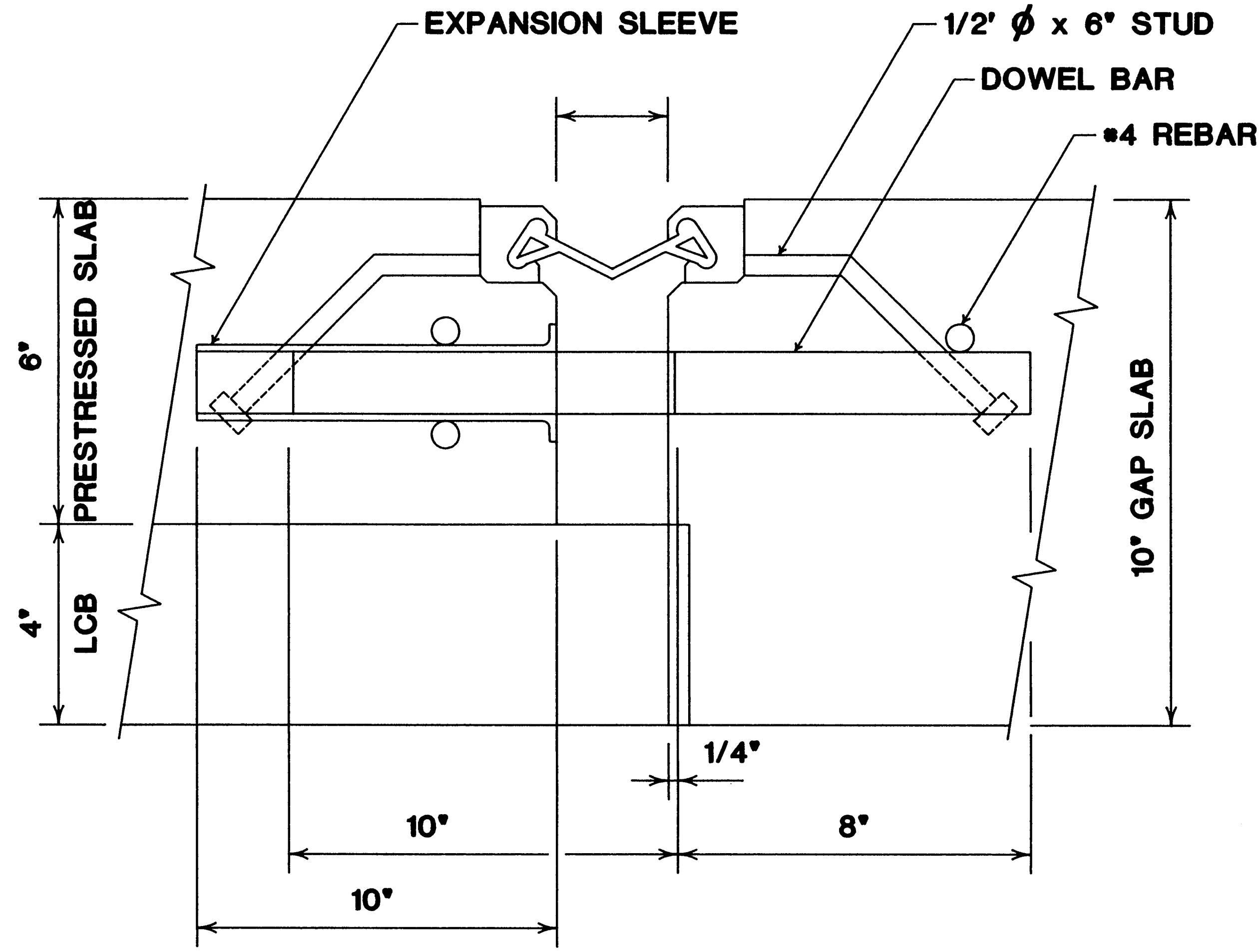


Figure 2. Cross-Section of the Joints at the Gap Slabs (4).

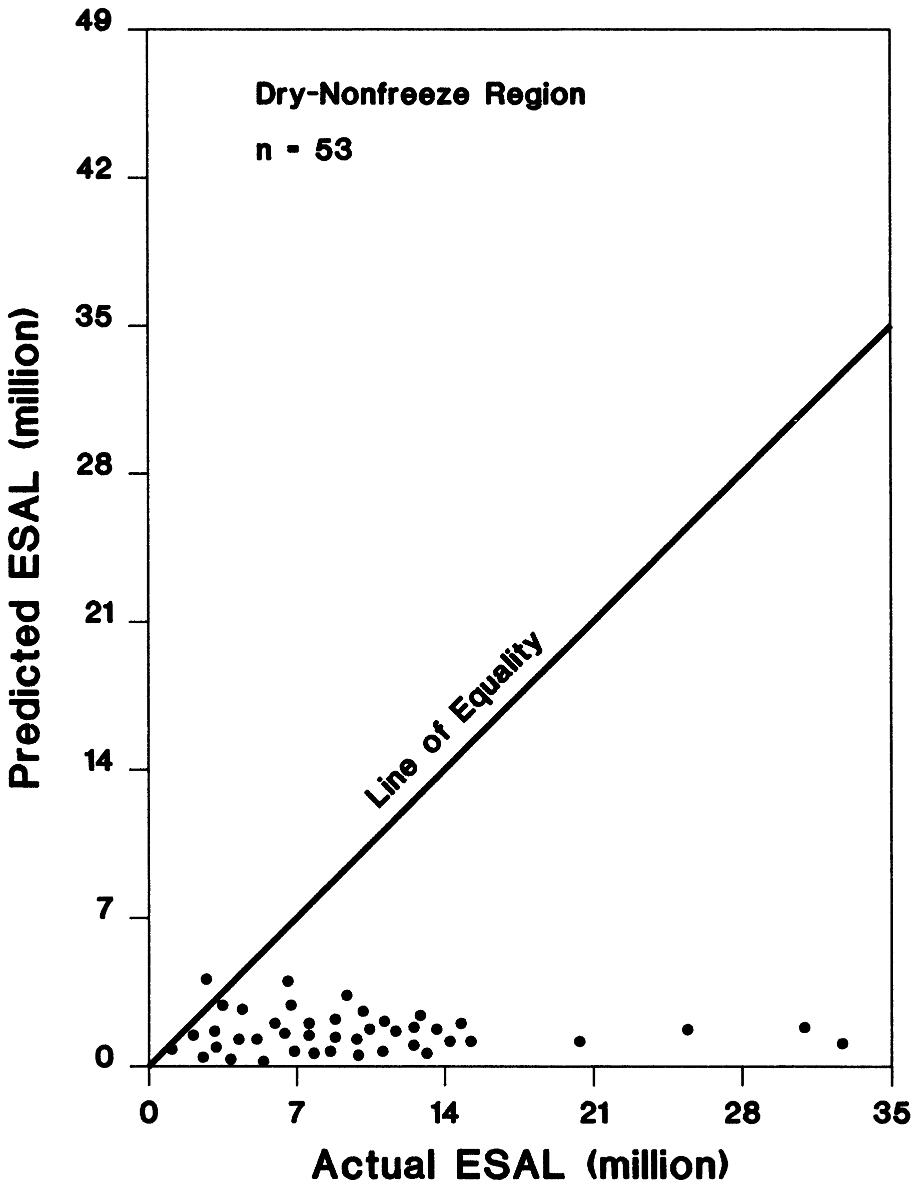


Figure 3. Predicted ESAL Versus Actual ESAL for JPCP (12).