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**STRUCTURAL DESIGN OF PORTLAND CEMENT  
CONCRETE OVERLAYS FOR PAVEMENTS**


**MBTC FR-1052**

**Kevin D. Hall and Nataraj Banihatti**

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## FINAL REPORT

MBTC-1052

### Structural Design of Portland Cement Concrete Overlays for Pavements

Kevin D. Hall and Nataraj Banihatti  
Dept. of Civil Engineering  
University of Arkansas

#### ABSTRACT

The most common method used to rehabilitate existing portland cement concrete (PCC) pavements is to place an overlay consisting of asphalt concrete. However, problems with premature rutting of the asphalt overlay and early appearance of reflective cracks in the asphalt overlay have made *overlays* consisting of portland cement concrete a viable alternative rehabilitation method. Most PCC overlay failures can be attributed to causes other than improper overlay thickness, suggesting that existing design procedures such as the AASHTO procedure provide sufficient overlay thickness to satisfy design requirements. In this study, major factors affecting overlay *performance* were identified; guidelines for considering those factors in design are presented. In addition, user-friendly computer spreadsheets were developed to aid designers in completing AASHTO-based thickness design for unbonded and bonded PCC overlays.

## INTRODUCTION

Portland cement concrete (PCC) pavements constitute a relatively large percentage of pavements that are designed to carry high volumes of heavy traffic. When designed, constructed and maintained properly, PCC pavements can be expected to provide long service lives. Many factors, however, contribute to the accelerated deterioration of PCC pavements, including construction deficiencies, design loading in excess of that forecasted, material problems, and unanticipated changes in traffic patterns. It is not surprising, therefore, that rehabilitation of PCC pavements is a topic receiving much attention. Of the major rehabilitation approaches - resurfacing, recycling, restoration and reconstruction - resurfacing (or overlays) is one of the most commonly performed methods of restoring rideability and improving structural capacity **(1)**.

The most frequently constructed type of overlay is made of hot-mix asphalt concrete (HMAC). An HMAC overlay can be placed fairly rapidly, at a very competitive cost, and with little shut down of the facility. However, there are two major problems associated with HMAC overlays: reflection cracking and rutting. These problems contribute to a shorter service life than is desired in many cases for a rehabilitation strategy on high-volume, heavily loaded pavements. Also, a relatively thick HMAC overlay is required to improve the structural capacity of the pavement **(1)**. Hence, resurfacing with portland cement concrete is gaining popularity.

There have been significant improvements in the area of pavement and overlay design procedures. What was once a more-or-less purely empirical

method now involves a significant amount of mechanistic procedures. Some of the overlay thickness design procedures used successfully in the recent years are those developed by McCullough et al., Trebig et al., the U.S. Army Corps of Engineers, and the Minnesota DOT **(2)**. Even with numerous options available, many highway agencies use no formal design procedure, but rely on engineering judgment and experience for PCC overlay designs of both rigid and flexible underlying pavements. A few agencies use the AASHTO design procedure, which is a mechanistic-empirical approach.

## **PROJECT OBJECTIVE**

The original overall objective of the project was to develop a rational, consistent method for designing PCC overlays for pavements. The specific objectives were as follows.

1. Identify the major factors involved in PCC overlay design; determine the method(s) and extent to which these factors are considered in current design procedures, with particular emphasis on the AASHTO procedure.
2. Conduct an in-depth evaluation of current AASHTO PCC overlay design procedures with respect to roadway and environmental conditions in Arkansas and the surrounding region.
3. Develop supplemental or companion procedures to current AASHTO procedures that are consistent with sound pavement design principles.
4. Develop a computer program to design PCC overlays.

A review of the existing literature on PCC overlays suggested that most overlay failures could be attributed to causes other than improper overlay

thickness. In light of this, the objectives of the project had to be slightly revised. Instead of developing an entirely new design procedure, major factors that affect overlay performance were identified. Guidelines were given on the AASHTO procedure and user-friendly computer spreadsheets were developed to design bonded and unbonded PCC overlays.

## **PCC OVERLAYS**

Use of portland cement concrete to resurface existing pavements can be traced back to as early as 1913 **(2)**. With increasing axle loads and traffic volumes, PCC resurfacing is becoming even more popular as a rehabilitation alternative with many highway agencies. While there were 375 catalogued projects in 1982, the number increased to more than 700 in 1993 **(2)**. This has been made possible due to the progress in technology, improvements in design procedures, construction guidelines and specifications for all types of PCC overlays.

Concrete resurfacing is used primarily to improve the structural capacity of the existing pavement or to enhance ride quality (functional enhancement) **(1)**. One of the evolving uses of the structural resurfacing involves stage construction, in which future bonded resurfacing may be planned at the time of original pavement design **(1)**. "Whitetopping", or PCC resurfacing of existing asphalt pavements, is also becoming very popular.

### **Types of PCC Overlays**

Depending on the type of interface used, concrete overlays can be classified as bonded, unbonded, or partially-bonded overlays. Depending on the

presence and type of reinforcement, they can further be classified as jointed plain concrete overlays, jointed reinforced concrete overlays, continuously reinforced concrete overlays or fibrous concrete overlays.

Bonded overlay means that special efforts are employed to enhance bonding between the existing pavement and the overlay. Unbonded means that specific actions are taken to ensure that there is no bond between the concrete layers. Partially bonded means that bonding is not particularly addressed, and as the name itself suggests, some areas may in fact be bonded **(1)**.

### ***Bonded Overlays***

Bonded concrete overlays provide two improvements to an existing pavement: increased structural capacity and a new riding surface. Bonded resurfacing is usually thin and hence depends on the existing pavement for structural capacity **(3)**. This means that the existing pavement should be free from distresses if good performance is to be realized.

Bonding a resurfacing to the underlying pavement to achieve monolithic behavior of the two layers is a very efficient means of structural enhancement. A 25 mm (1 in) bonded concrete overlay has approximately the same structural benefit in reducing the stresses as 62.5 mm (2.5 in) of asphalt concrete **(4)**. When the effective slab thickness is increased by a bonded overlay, vertical deflection and subgrade stresses are decreased significantly. When a 75 mm (3 in) bonded overlay is applied to a 225 mm (9 in) concrete pavement, the deflection under a 550 kPa (80 psi) corner wheel load decreases by 31 percent (assuming 90 percent load transfer efficiency at the transverse joint). This



reduction in deflection will likely result in reduced pumping, faulting, and loss of support **(5)**.

A bonded PCC overlay when properly constructed, holds the promise of an extended service life, increased structural capacity, and lower life cycle costs, compared with other overlay techniques. Although the initial cost of a bonded PCC overlay may be higher than those of an HMAC overlay, the benefits of longer life and reduced maintenance costs suggest that bonded overlays can be a viable resurfacing alternative **(1)**.

Bonded concrete overlays must be matched by type to the existing concrete slabs. That is, jointed concrete overlays must be used only on jointed concrete pavements, and continuously reinforced overlays can be used only on existing continuously reinforced concrete slabs. Furthermore, for bonded overlays the existing slabs must be distress-free, since most distress in the existing slab will ultimately reflect through the overlay **(6)**.

Early studies showed that bonding between the two layers is principally a mechanical process that depends primarily on the soundness and cleanliness of the underlying pavement **(7)**. However, later work **(8)** recognized a degree of chemical bonding between the overlay and the underlying pavement. Felt **(7)** suggested that "a slight degree of roughness is desirable, but an extremely rough surface is not required". When properly constructed, the bond strength often exceeds the strength of even the strongest layer, so that bond test specimens fail in one of the layers rather than at the interface **(10)**.

Important design and construction considerations for bonded concrete overlays are:

1. existing slab cracking
2. pre-overlay repair
3. surface preparation
4. overlay thickness
5. sawing of the joints
6. curing of the overlay concrete

Following are the advantages and disadvantages of the bonded type of overlays.

Advantages:

1. Thin overlays can be used. Though 50-125 mm (2-5 in) thick overlays are typical, overlays as thin as 25 mm (1 in) have been successfully used on sound existing concrete pavements **(6)**.
2. Thin overlays mean lower costs and fewer problems in maintaining minimum overhead clearances and matching existing facilities, which is particularly advantageous in urban areas.
3. Because of the smaller amount of concrete used with overlay, higher-quality concrete can be used without significant adverse costs.

Disadvantages:

1. These overlays can be used only on sound, distress-free pavements.
2. Proper preparation of the existing surface is most critical to achieve bond.
3. These overlays must be matched by type to the existing concrete slabs. That is, JRCP overlays can be used only on existing JRCP and so on.

4. Some minor adjustments may be necessary in the concrete mixture to achieve a dense, durable surface
5. The joints in the overlay must be matched to the joints in the existing slab by both location and type **(6)**.

### ***Unbonded Overlays***

PCC unbonded overlays are designed with an interlayer between the new overlay and the existing slab to isolate the overlay from distress in the underlying pavement and, thereby, eliminate or reduce reflective cracking **(11, 12)**. This type of overlay has been used effectively over both concrete and bituminous pavements **(12,13,14)**. Particular economic and performance advantage is gained when used on existing pavements that have become significantly deteriorated.

Unbonded overlays are intended for use on existing pavements in which distress is too extensive and too severe to be effectively eliminated before overlaying **(6)**. The bond breaker layer often is composed of HMAC covered with membrane curing compound to impede bonding. With a few special considerations, the resurfacing may then be constructed as if the underlying pavement were a conventional subbase layer **(1)**. Fully unbonded PCC overlays behave eventually as slabs supported by a firm subgrade **(6)**. However, due to the very stiff nature of the existing pavement, thermal curling stresses in unbonded pavement can cause cracks if the joints are not closely spaced. Following are the advantages and disadvantages of the unbonded type of overlays.

### Advantages:

1. A big advantage of this type of overlay is that it is not necessary to match the joints between the existing pavements and overlays or even to clean or seal these joints **(6)**.
2. Surface preparation is not as critical as in bonded type of resurfacing. However, structural distresses cannot be ignored and uniform support should be ensured.
3. No special construction techniques are needed for construction.

### Disadvantages

1. The major disadvantage of unbonded PCC overlays is the greater thickness required, potentially resulting in higher costs and greater clearance problems. Minimum thickness for unbonded overlays is 150 mm (6 in); typical thickness is likely 175 mm to 200 mm (7 to 8 in), depending on the traffic and the condition of the existing pavement **(6)**. The thickness of the overlay may not be economically feasible for most projects.
2. When relatively thin unbonded overlays are going to be constructed, it is extremely important that the existing pavement be properly prepared (undersealed, broken slabs replaced, patched, etc.) to ensure good performance.

### ***Partially Bonded Overlays***

If the issue of bonding between the resurfacing and the underlying pavement is of little importance, such as on thick airfield pavement, the partially bonded approach may be employed **(1)**. Grout or special additives are not

required to promote bond when partially bonded overlays are used. These overlays are sometimes referred to as direct overlays **(12)**, implying that little or no surface preparation is done. The only requirements for partially bonded overlays are that the surface be free of loose materials and that the existing concrete surface be sound. Because no particular attention is paid to cleaning or grinding the base pavement, various degrees of bonding may occur, but will have little bearing on the performance of the resurfacing. Recent literature considers partially bonded overlays to be special cases of the unbonded type, because the evidence shows that the performance is similar **(16)**. Partially bonded overlays should also be used only on reasonably sound existing pavements, since most cracks in the existing slab will reflect through the overlay within a short period of time.

Ideally, the minimum thickness for partially bonded overlays is 150 mm (6 in), although 125 mm (5 in) overlays have been used successfully. Unless joints are closely spaced, however, significant cracking between joints can be expected when thin partially bonded overlays are used. It should be noted that partially bonded overlay is not considered a usual alternative for highway pavement **(2)**. Furthermore, recent airfield pavement related literature makes little reference to the partially bonded type of overlay. For design purposes, only the bonded, unbonded and whitetopping types of overlays are considered **(2)**.

## **DESIGN OF PCC OVERLAYS**

Overlays are constructed to correct two deficiencies namely, functional deficiency and structural deficiency. In general, structural deficiency will override

functional deficiency because a thicker resurfacing is almost always required - that is, a resurfacing thick enough to satisfy structural requirements should be more than thick enough to correct any functional deficiency **(2)**.

The following are some of the basic requirements governing the design of PCC overlays **(17)**.

1. Thickness must be sufficient for the anticipated service conditions.
2. Joints (longitudinal and transverse) and cracks must have the capacity to transfer applied loads without loss of surface smoothness. The joint and crack system should minimize the migration of moisture between it and the underlying pavement.
3. Reinforcement must have adequate cover for the exposure conditions and should be of such size and spacing that all cracks are held tight.
4. The maximum size aggregate must be compatible with the resurfacing thickness and spacing of steel.
5. Sound, durable aggregate must be used; air entrainment must also be used if freezing and thawing or the use of de-icing salts might occur.
6. Shoulders should be of concrete, tied to the resurfacing, or another material stabilized for the full depth of the resurfacing to minimize infiltration of shoulder material between the underlying pavement and the resurfacing.

An important consideration in resurfacing design is the condition of the existing pavement on which the resurfacing is proposed. Barenberg **(6)** in 1981 put condition evaluation of the existing pavement in perspective as one of the most important resurfacing considerations:

*Evaluating the true condition of the existing pavement is one of the most critical factors in selecting the best overlay option. This evaluation should reflect how the existing pavement will affect the behavior and performance of the overlaid pavement. Such an evaluation should be based on structural or behavioral considerations rather than serviceability considerations.*

It should also be noted that PCC resurfacing shares at least one design requirement with on-grade PCC pavements: they require uniform support conditions if satisfactory performance is to be realized. Nearly all the documented cases of premature overlay failure can be traced to some violation of this single requirement **(2)**.

### **Functional Design**

Since a functional resurfacing needs to be only thick enough to restore the ride quality or repair surface defects, it may in fact be relatively thin. Typically, the capability of paving machines, the sizes of the aggregate particles, and geometric considerations (overpass elevations, guardrail heights, grades, etc.) will dictate how thick such resurfacing must be **(2)**. On the other hand, reinforced sections may need to be a minimum of 75 to 100 mm (3 to 4 in) thick to accommodate the reinforcing steel with sufficient cover to impede earlier corrosion.

### **Structural Design**

In general, structural deficiency will override functional deficiency because a thicker resurfacing is almost always required - that is, a resurfacing thick enough to satisfy structural requirements should be more than thick enough to correct any functional deficiency. While there are numerous approaches to

resurfacing thickness design, all are conceptually similar and involve the determination of: (1) the structural capacity required to carry the prevailing and projected traffic for the design life of the resurfacing; (2) the structural capacity of the existing pavement; and (3) the difference between (1) and (2).

In the AASHTO terminology, the structural capacity for a PCC pavement is the slab thickness (D). The structural capacity of a pavement decreases with time and accumulated traffic and by the time an overlay is considered, the effective structural capacity of the existing PCC pavement becomes  $D_{\text{eff}}$ . The difference between the structural capacity required to carry the future traffic (e.g.  $D_f$ ) and effective structural capacity ( $D_{\text{eff}}$ ) will be the structural capacity of the overlay ( $D_{\text{OL}}$ ) which is shown by the following general equation.

$$D_{\text{OL}} = D_f - D_{\text{eff}}$$

The design of overlays begins with an evaluation of the existing pavement to determine thickness, type of load transfer, and type of shoulder. Next, the projected 18-kip equivalent single axle loads (ESALs) in the design lane for the design period are determined.

A condition survey is used to determine the types and severity of distress present. Non-destructive deflection testing is done to evaluate the effective k-value (subgrade support), slab elastic modulus, and joint load transfer. If a bonded overlay is planned, it is recommended that the modulus of rupture of the existing slab be determined by testing pavement cores.

The overlay thickness is determined as follows **(18)**:

**Bonded overlays:**  $D_{\text{ol}} = D_f - D_{\text{eff}}$



**Unbonded overlays:**  $D_{oi} = \sqrt{D_f^2 - D_{eff}^2}$

- where  $D_{oi}$  = thickness of the overlay (in)  
 $D_f$  = Slab thickness required to carry the future traffic (in)  
 $D_{eff}$  = Effective slab thickness of the existing pavement (in)

Effective slab thickness of the existing pavement  $D_{eff}$  is determined from condition survey and is dependent on the amount of distress such as durability problems, unrepaired transverse joints and cracks, fatigue cracks, punch-outs, etc.

Slab thickness ( $D_f$ ) to carry the future traffic is determined from the following AASHTO rigid pavement design equation **(18)**.

$$\log_{10} W_{18} = Z_R * S_o + 7.35 * \log_{10}(D + 1) - 0.06 + \frac{\log_{10} \left[ \frac{\Delta PSI}{4.5 - 1.5} \right]}{1 + \frac{1.624 * 10^7}{(D + 1)^{8.46}}}$$

$$+ (4.22 - 0.32P_t) * \log_{10} \left[ \frac{S'_c * C_d [D^{0.75} - 1.132]}{215.63 * J \left[ D^{0.75} - \frac{18.42}{(E_c / k)^{0.25}} \right]} \right]$$

- where,
- $W_{18}$  = 18 kip ESALs in the design period
  - $Z_R$  = Standard Normal Deviate (function of reliability R)
  - $S_o$  = Standard Deviation
  - $D$  = Slab thickness to carry future traffic ( $D_f$ )
  - $\Delta PSI$  = Loss in Serviceability ( $P_{initial} - P_{terminal}$ )
  - $P_t$  = Terminal Serviceability
  - $S'_c$  = PCC Modulus of Rupture (psi)
  - $C_d$  = Coefficient of Drainage
  - $J$  = Load Transfer Coefficient
  - $E_c$  = PCC Elastic Modulus (psi)
  - $k$  = Effective Static k-value (pci)

It should be noted that the method for obtaining design inputs such as properties of concrete depends on the type of overlay to be constructed. For an unbonded overlay, PCC properties are representative of the overlay concrete whereas for bonded overlays, the properties are representative of the existing pavement concrete.

### **Selection of Overlay Type:**

Selection of the proper type of overlay is very important to ensure good performance. For example, Bonded overlays depend entirely on the existing pavement for structural capacity **(3)**, implying that they should never be constructed on a pavement suffering from structural damages. Also, bonded overlays should not be constructed on pavements suffering from durability problems because D-cracking will reflect through the overlay.

On the other hand, there is not much restriction for constructing unbonded overlays. In fact, unbonded overlays are most cost-effective when the existing pavement is badly deteriorated because of reduced need for pre-overlay repair. However, unbonded overlays are not intended to bridge localized areas of non-uniform support. Hence, areas of non-uniform support should be identified and proper repairs should be done to ensure uniform support for the overlay. Figure 3 illustrates the selection process for the overlay based on existing pavement condition and pre-overlay repairs. However, the final selection of the overlay type also depends on other factors such as availability of equipment, economics, agency experience, etc.

### **UNBONDED OVERLAYS**

## **Design Inputs: Determination of Slab Thickness for Future Traffic ( $D_f$ )**

Since the AASHTO overlay design procedure is basically a structural deficiency approach, the required slab thickness for the future traffic will have a direct impact on the overlay thickness obtained. The inputs that affect  $D_f$ , in order of significance, are as follows **(19)**:

1. PCC modulus of rupture and elastic modulus ( $S_c$  and  $E_{pcc}$ )
2. Reliability level (  $R$  )
3. Drainage Coefficient ( $C_d$ )
4. Load Transfer Coefficient ( $J$ )
5. Overall Standard Deviation ( $S_o$ )
6. Design ESALs ( $W_{18}$ )
7. k-value

### ***PCC Modulus of Rupture and Elastic Modulus***

The PCC modulus of rupture and elastic modulus to determine  $D_f$  for unbonded overlay of an existing PCC pavement are representative of the new PCC overlay to be placed and not those of the existing slab. Elastic modulus of overlay concrete can be obtained by the following correlation recommended by American Concrete Institute for normal weight portland cement concrete:

$$E_c = 57,000 (f'_c)^{0.5}$$

where  $E_c$  = PCC Elastic Modulus (in psi)

$f'_c$  = PCC Compressive strength (in psi) as determined using  
AASHTO T 22, T 140 or ASTM C 39 (see appendix)

The modulus of rupture required by the AASHTO design procedure is the mean value determined after 28 days using third-point loading (AASHTO T97,

ASTM C 78). Because of the treatment of reliability in the AASHTO design procedure, the use of the normal construction specification for modulus of rupture is not recommended to be used as input, since it represents a value below which only a small percent of the distribution may lie. If it is desirable to use the construction specification, then some adjustment should be applied, based on the standard deviation of modulus of rupture and the percent (PS) of the strength distribution that normally falls below the specification **(18)**:

$$S'_c \text{ (mean)} = S_c + z (SD_s)$$

where  $S'_c$  = estimated mean value for PCC modulus of rupture (psi)

$S_c$  = construction specification on concrete modulus of rupture (psi)

$SD_s$  = estimated standard deviation of concrete modulus of rupture (psi)

$z$  = standard normal variate:

= 0.841 for PS = 20 percent\*

= 1.037 for PS = 15 percent

= 1.282 for PS = 10 percent

= 1.645 for PS = 5 percent

= 2.327 for PS = 1 percent

\*Note: Permissible number of specimens, expressed as a percentage, that may have strengths less than the specification value.

### ***Reliability***

The reliability of a pavement design-performance process is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period.

Reliability is a means of incorporating some degree of certainty into the design process to ensure that the various design alternatives will last the analysis period. A detailed discussion of reliability is beyond the scope of this

project. For more information, chapter 4 of the 1993 AASHTO Guide may be consulted. Table 1 gives the suggested levels of reliability for various facilities.

Functional Classification	Recommended Level Of Reliability	
	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterial	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

**Table 1. AASHTO Reliability Levels for Pavement Design (18).**

For a given level of reliability (  $R$  ), the reliability factor is a function of overall standard deviation ( $S_o$ ) that accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for ESALs. The AASHTO Guide (18) states:

“It is important to note that by treating design uncertainty as a separate factor, the designer should no longer use “conservative” estimates for all the other design input requirements. Rather than conservative values, the designer should use his best estimate of the mean or average value for each input value. The selected level of reliability and overall standard deviation will account for the combined effect of the variation of all the design variables”.

### ***Drainage Coefficient (Cd)***

In the AASHTO design procedure for rigid pavement, drainage coefficient has a significant effect on the resulting thickness. Because drainage condition influences slab support and therefore overall stress condition in the slab,  $C_d$  was introduced into the portion of the AASHTO rigid pavement performance (design) equation that considers the slab’s strength, stress and support condition (20). As a matter of fact,  $C_d$  has the same relative impact on rigid pavement

performance as both concrete modulus of rupture ( $S_c$ ) and the load transfer coefficient ( $J$ ). A 20 percent increase in  $C_d$  would have the same effect as a 20 percent increase in  $S_c$ , or 20 percent increase in  $1/J$ .

The selection of drainage coefficient, which ranges from 0.4 to 1.4, is based on the quality of drainage and percent of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation. The percent time during which the pavement structure is saturated depends on the average yearly rainfall and the prevailing drainage conditions.

Table 2 provides the recommended  $C_d$  values.

Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation				
Quality of Drainage	Less Than 1%	1-5%	5-25%	Greater than 25%
Excellent	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

**Table 2. AASHTO Drainage Coefficient for Pavement Design (18).**

While one can spend a significant amount of time trying to come up with accurate  $k$ -values, a small change in the value of  $C_d$  is equivalent to a big change in  $k$ -value as illustrated in Table 3 (20).

Quality of Drainage	Selected $C_d$ Value	Corresponding $k$ -value (pci)
Excellent	1.2	942
Good	1.1	501
Fair	1.0	200
Poor	0.9	44
Very Poor	0.8	1

**Table 3. Relative Effect of  $C_d$  and  $k$ -value for Rigid Pavement Design**

Unfortunately, selecting a proper value for  $C_d$  has been a point of concern for pavement engineers. One of the problems with the methodology is that there are no well-defined procedures for translating the results of various drainage design procedures into the rather subjective input (coefficient) used in the AASHTO design procedure **(20)**.

As a basis for comparison, the value of  $C_d$  for conditions at the AASHTO Road Test was 1.0; however, pavements at AASHTO Road Test had very poor subdrainage. Indeed, the pavements didn't have a subdrainage facility at all. In light of this, a  $C_d$  value of even 1.0 may be too high and a value of 0.8 to 0.9 may be appropriate for the AASHTO pavements.

As far as modern pavements with better subdrainage facilities are concerned, a value of 1.0 to 1.15 may be assigned. However, this should be based on the actual drainage conditions of the pavement.

### ***Joint Load Transfer Coefficient (J)***

The joint load transfer coefficient relates to the ability of a joint to transfer shear load and this coefficient has a significant effect on the resulting thickness of the slab. Though the load transfer coefficient *appears* to be related to joint faulting in a pavement, it has nothing to do with faulting. Darter, et. al. **(21)** observe:

It is very important to remember that the J-factor is an adjustment for slab stresses that cause corner breaks, and has absolutely nothing to do with joint faulting. No joint faulting existed at the Road Test. One cannot design a reduction or an increase in joint faulting by changing the J-factor. This has been a point of major confusion among pavement engineers for years.

It should be noted that increasing the thickness of slab cannot prevent faulting **(21)**. Experience has shown that installing dowel bars is the most effective way of providing load transfer across the joints, and thus reduces faulting.

For designing an unbonded PCC overlay of an existing concrete pavement, the J factor should be representative of the overlay and NOT the existing pavement. If a bonded PCC overlay or an AC overlay is being constructed on top of an existing PCC pavement, then the J factor should be based on the existing pavement. The load transfer coefficient is obtained from Table 4, based on the type of shoulder.

Asphalt Concrete Shoulder		
Load Transfer Devices →	YES	NO
Pavement Type		
1. JPCP or JRCP	3.2	3.8 - 4.4
2. CRCP	2.9-3.2	N/A
Tied PCC Shoulder		
Load Transfer Devices →	YES	NO
Pavement Type		
1. JPCP or JRCP	2.5 - 3.1	3.6 - 4.2
2. CRCP	2.3 - 2.9	N/A

**Table 4. Selection of AASHTO Load Transfer Coefficient (J-Factor) (18).**

Tyner et. al **(22)** recommend that only tied concrete shoulders should be used when concrete overlays are constructed. Since AHTD specifications call for tied PCC shoulders and use of dowel bars, the J-factor may be assigned a value of 2.8 for JPCP or JRCP overlays and 2.6 for CRCP overlay.



As a general guideline, the dowel diameter should be equal to 1/8<sup>th</sup> of the slab thickness in inches. The dowel spacing and length are normally 300 mm (12 in) and 450 mm (18 in) respectively.

***Effective k-value (Modulus of Subgrade Reaction)***

**Definition:**

*The modulus of subgrade reaction is the stress (in lb/in<sup>2</sup>) that will cause one inch of deflection in the underlying soil. (Units: psi/in, or pci)*

The above definition indicates that stiffer the subgrade, higher the k-value. Soils such as clay will have a lower k-value compared to cement treated or asphalt treated bases. Research has shown that the value of k depends on certain soil characteristics such as density, moisture, soil texture and other factors that influence the strength of the soils. The k-value of a particular soil will also vary with size of the loaded area and the amount of deflection. The modulus of subgrade reaction is directly proportional to the loaded area and inversely proportional to the deflection.

Modulus of subgrade reaction is obtained by conventional plate bearing tests, correlation with soil properties or other soil tests and also by backcalculation from deflection testing on concrete pavements. In overlay design, it is almost always obtained by deflection testing using the following back-calculation equations **(25)**:

$$AREA = 6 * \left[ 1 + 2 \left( \frac{d_{12}}{d_0} \right) + 2 \left( \frac{d_{24}}{d_0} \right) + \left( \frac{d_{36}}{d_0} \right) \right]$$

$$l_k = \left[ \frac{\ln\left(\frac{36 - AREA}{1812.279133}\right)}{-2.559340} \right]^{4.387009}$$

$$k = \left( \frac{P}{8d_0 l_k^2} \right) \left\{ 1 + \left( \frac{1}{2\pi} \right) \left[ \ln\left( \frac{a}{2l_k} \right) + \gamma - 1.25 \right] \left( \frac{a}{l_k} \right)^2 \right\}$$

where  $d_{0, 12, 24, 36}$  = deflections @ 0 in, 12 in, 24 in, 36 in. (inches)

$l_k$  = radius of relative stiffness

$k$  = Modulus of subgrade reaction ( effective dynamic k, pci)

$P$  = FWD load, pounds

$a$  = load radius, in. (usually 5.6 in)

$\gamma$  = Euler's constant, 0.57721

Deflection data should be collected on the outer wheel path along the project at an interval sufficient to adequately assess the conditions. Intervals of 30 m (100 ft) to 300 m (1000 ft) are typical. A load magnitude of 4100 kg (9000 lb) or more is recommended. The k-value should be obtained for each slab tested.

The k-value backcalculated from NDT data is a dynamic k-value whereas the required input to the AASHTO design equation is a static k-value. In an analysis of AASHTO Road Test Data, dynamic repeated-load k-values were found to exceed static values by a factor of 1.77 on the average **(23)**. Research work by Foxworthy involving seven Air Force Base pavements indicated that dynamic k-values exceeded static k values by a factor of 2.3 on the average. Reducing backcalculated k-values by 2 has been found to produce reasonable values for static k-values **(25)**. Hence it is recommended in the overlay design procedures

that backcalculated k-values be divided by 2 to obtain static k-values for determining  $D_f$ .

If a single overlay thickness is being designed for a uniform section a mean k-value may be used. However, experience has shown that k-value for edge conditions is two to four times larger than that for center conditions for the same site. Uzan et. al. (26) suggest that one should use the values obtained only for the same conditions that prevailed in their derivations.

However, it should be noted that k-value can change substantially and have only a small effect on overlay thickness. Darter et. al (25) concluded in a recent study that even 50% error in estimating k-value can cause only a 5% error in new rigid pavement slab thickness ( $D_f$ ). The error will be even smaller in terms of overlay thickness ( $D_{OL}$ ). Hence, using an average value for k should not lead to serious errors.

Also, a previous research at the University of Arkansas showed that freezing of the subgrade is not a problem in the lower two-thirds of the Arkansas (27). In addition, Arkansas doesn't have a well-defined "rainy" season. In light of this, it is not recommended that the k-value should be adjusted for seasonal effects.

Though k-value doesn't affect the overlay thickness significantly, it does play a major role in how unbonded overlays perform. A slab built on top of a stiff base (high k-value) as in the case of an unbonded PCC overlay, can be subjected to very high curling and warping stresses. The current AASHTO design procedure doesn't address this problem in detail.

### ***Effect of k on Curling and Warping Stresses***

Curling and warping are associated with temperature and moisture differentials in the slab. During daytime, the top surface of the slab becomes hotter than the bottom and the slab tends to bend upward, resulting in a void below the middle portion of the slab. On the other hand, a negative temperature differential occurs at night and results in the corners and edges displacing upward, creating the potential for a void near the edges of the slab. When this happens, traffic load near the corners or joints can induce very high stress on the surface of the slab and lead to corner breaks **(21)**.

Warping of a slab takes place due to moisture gradient (top drier than the bottom) and this occurs seasonally. Data from Illinois showed that substantial drying occurs only at the top surface to a depth of less than 50 mm (2 in). The rest of the slab remains at 80 percent saturation or higher **(28)**. However, in dryer, less humid climates, greater drying and upward warping of the slab may occur.

Yoder and Witzak explain the effects of k-value and slab thickness on curling and warping stresses as follows **(29)**:

Curling and warping stresses increase as subgrade stiffness (k) increases since for very stiff subgrades (those with high k values) the subgrade does not yield. For softer subgrades (clays for example) the subgrade will yield as the slab warps and the subgrade will assume the general contour of the pavement. For this case the pavement is supported uniformly over its entire length and stresses are reduced. For the extreme case, wherein a slab is placed directly upon another slab, such as in overlays, curling and warping stresses combined with traffic loading may be so high that the slab will crack. One method of combating this is to make the overlay slab quite thick.

Experience has shown that *thin* concrete overlays built over existing concrete slabs may crack badly. The material from above paragraph explains a contributing factor in this, although curling/warping stresses are not the only factors operating in this situation. Because of the above, thin concrete overlays should be bonded to the existing pavement.

Regarding thermal curling stresses, Voigt, et. al. (5) concluded:

Thermal curling stresses are critical in unbonded concrete overlays because the temperature gradient through the overlay becomes so large during many days and nights of the year, and because of the very stiff support from the existing slab. At these times, curling may cause the overlay slab to lift from the underlying slabs and create voids between slabs, which (when combined with traffic load stresses and stiff foundation of the underlying slab) can cause transverse cracking. It is highly recommended that the overlay joint spacing be kept short. Maintaining a [slab length (in)/radius of relative stiffness in.] ratio between 6 and 7 should ensure that cracking will not develop in the overlay. If, however, longer slabs are used, reinforcement must be included to keep the cracks tight.

Darter et. al have concluded (21) as follows about the effect of subgrade and base stiffness:

The effect of subgrade and base stiffness on slab stresses is very different when a temperature differential exists through the slab. When no temperature gradient exists through a slab, increased subgrade k-value or base modulus value will always show a reduced tensile stress in the slab under loading and thus design will require a thinner slab. However, very stiff foundations may actually increase combined load and temperature curl stresses resulting in thicker slab requirements. Greatly increased base and subgrade stiffness may not always be beneficial. Under these conditions it may be necessary to shorten joint spacing to avoid premature transverse cracks in the slab.

The above discussion shows the importance of proper joint spacing to prevent transverse cracks and corner breaks in unbonded overlays.

Recently, researchers in Germany and Chile have identified a permanent form of slab curling which is caused by a temperature differential during construction (30,31,32,33). Permanent upward corner and edge curling may

occur if a high positive (top warmer than bottom) temperature differential exists through the slab as it hardens. This occurs especially on sunny days and unless proper measures are taken to keep the top of the slab cool. If the slab solidifies with a large positive thermal gradient (~30 F) during construction, the corners and edges will be permanently curled upward for any lower temperature gradient and such curling would create a serious loss of support under the corners and edges leading to corner breaks. This suggests that proper measures are essential to keep the top of the slab cool and thus avoid construction curling.

***Summary: Design Inputs for  $D_r$ , Unbonded Overlays***

- k-value can be back-calculated by deflection testing
- Obtain deflection data at 100 ft. to 1000 ft. intervals in the outer wheel path
- Use an FWD load of 9000 lbs. or greater
- If a single overlay thickness is being designed, compute the mean k-value for use in design
- Adjusting k-value for seasonal effects is not recommended
- The k-value doesn't need to be estimated with great accuracy. An error of 50% or less will not affect the resulting overlay thickness significantly
- The stiff nature of the existing pavement can cause very high curling and warping stresses in the overlay. This can be prevented by proper joint spacing or increasing the overlay thickness.

**Effective Slab Thickness of The Existing Pavement ( $D_{eff}$ )**

The effective slab thickness or  $D_{eff}$  is needed to determine the thickness of the new overlay.  $D_{eff}$  is dependent on the amount of distress present. The distresses to be considered include:

- deteriorated joints
- deteriorated cracks
- deteriorated punchouts
- expansion joints, exceptionally wide joints (>1 in) or full-depth, full-lane width AC patches.

Since unbonded concrete overlays are resistant to reflective cracking caused by surface problems in the existing slab such as durability cracking and fatigue cracking, these distresses are not considered in determining  $D_{\text{eff}}$ .

The effective thickness of the slab is computed from the following equation:

$$D_{\text{eff}} = F_{\text{jcu}} * D$$

where  $D$  = existing PCC slab thickness, in. (10 in maximum, even if existing  $D > 10$ in.)

$F_{\text{jcu}}$  = joints and cracks adjustment factor for unbonded concrete overlays

The  $F_{\text{jcu}}$  factor adjusts for the extra loss of PSI caused by deteriorated reflection cracks or punchouts in the overlay that result from any unrepaired joints, cracks and other discontinuities in the existing slab prior to overlay. Very little such loss in PSI has been observed for JPCP or JRCP unbonded overlays. Hence this factor ranges from 0.90 to 1.00, which means that even if there were significant number of deteriorated joints and cracks, the effective slab thickness of the existing pavement will not be less than 0.9\*(slab thickness). In light of this, it may not be necessary to conduct detailed distress surveys if an unbonded overlay is going to be constructed. Instead, information should be obtained on major distresses which contribute to non-uniform support such as moving slabs,

punchouts, AC patches, voids beneath slabs, and load transfer. Information about pumping is also essential to assess the quality of drainage.

If the existing pavement is very badly deteriorated, a thicker separation layer ( $\geq 50$  mm [2in]) should be used. Also, it should be noted that unbonded overlays are **not** intended to bridge localized areas of non-uniform support. Consequently, all tipping or rocking slabs should be stabilized by slab jacking or sealed by using heavy rollers to provide uniform support for the overlay **(6)**. For very badly deteriorated pavements, in addition to placing a thicker separation layer, it may be necessary to break and seat the slabs to ensure uniform support.

***Determination of Joints and Cracks Adjustment Factor ( $F_{jcu}$ ):***

The following information is needed to determine  $F_{jcu}$  to adjust overlay thickness for the extra loss in PSI from deteriorated reflection cracks that are not repaired:

- Number of unrepaired deteriorated joints/mile
- Number of unrepaired deteriorated cracks/mile
- Number of expansion joints, exceptionally wide joints ( $> 25$  mm [1 in]) or full depth, full lane width AC patches/mile

The total number of unrepaired deteriorated joints/cracks and other discontinuities per mile prior to overlay is used to determine the  $F_{jcu}$  from a figure given in the AASHTO Guide **(18)**. As an alternative to extensive full-depth repair for an unbonded overlay to be placed on a badly deteriorated pavement, a thicker AC interlayer ( $\geq 50$  mm [2 in]) should eliminate any reflection cracking problem, in which case  $F_{jcu} = 1.0$ .



### ***Determination of Existing Slab Thickness:***

This is an important parameter in overlay design. While historic information may be available, the extreme importance and sensitivity of this variable calls for the use of destructive testing to verify the available historic information. A limited amount of coring at randomly selected locations may be used to verify the historic information **(18)**. These cores can be used for determining the PCC modulus of rupture and PCC Elastic modulus.

### ***Summary: Design Inputs for $D_{eff}$ , Unbonded Overlays***

- Distress survey for unbonded overlays is not as critical as in bonded overlays.
- Obtain information about very badly deteriorated areas, non-uniform support, load transfer, and drainage conditions.
- A thicker separation layer means less pre-overlay repair.
- Consider a thicker separation layer if existing pavement has many joints and cracks with poor load transfer.
- $F_{jcu}=1.0$ , if using a thicker separation layer.
- If the existing pavement has deteriorated due to poor drainage, proper measures should be taken to improve drainage.
- Punchouts in existing CRCP should be full-depth repaired.

## **BONDED OVERLAYS**

### **Design Inputs for the Determination of $D_r$**

#### ***Elastic Modulus of Concrete***

In the case of bonded overlays, the elastic modulus (E) and modulus of rupture ( $S_c$ ) of the existing pavement concrete will be used to design the

thickness of the overlay. Since in this case, the overlay will be bonded to the existing pavement, the performance life of the overlay depends on the fatigue life of the existing slab. Among the rigid pavement design inputs, E and  $S_c$  have the most significant effect on the resulting thickness.

Elastic Modulus can be obtained by (1) backcalculation using FWD data or (2) by estimation from indirect tensile strength. The following equations are used in backcalculation **(25)**:

$$AREA = 6 * \left[ 1 + 2 \left( \frac{d_{12}}{d_0} \right) + 2 \left( \frac{d_{24}}{d_0} \right) + \left( \frac{d_{36}}{d_0} \right) \right]$$

$$l_k = \left[ \frac{\ln \left( \frac{36 - AREA}{1812.279133} \right)}{-2.559340} \right]^{4.387009}$$

$$l_k = \sqrt[4]{\frac{E_{pcc} D_{pcc}^3}{12(1 - \mu_{pcc}^2)k}}$$

- where  $E_{pcc}$  = Elastic Modulus of concrete (psi)  
 $D_{pcc}$  = Thickness of the existing pavement (inches)  
 $l_k$  = radius of relative stiffness  
 $d_n$  = corresponding deflection in inches.  
 $k$  = Modulus of Subgrade Reaction (pci)  
 $\mu_{pcc}$  = PCC Poisson's ratio (0.15)

For deflection testing, an FWD load of 4100 kg (9000 lb). or more should be used. Slab deflections should be obtained on the outer wheel path at an interval sufficient to adequately assess conditions. Intervals of 30 m to 300 m (100 to 1000 ft) are typical. ASTM D 4694 and D 4695 provide additional guidance on deflection testing (see appendix).

AREA will typically range from 29 to 32 for sound concrete. Typical slab E values range from 20.7 to 55.2 MPa (3 to 8 million psi). Older pavements will have a higher elastic modulus than newer ones. If a slab E value is obtained that is out of this range, an error may exist in the assumed slab thickness, the deflection basin may have been measured over a crack, or the PCC may be significantly deteriorated.

If a single overlay thickness is being designed for a uniform section, a mean E value of the slabs can be used in design. Any E-value that appears to be significantly out of line with the rest of the data should be discarded.

### ***PCC Modulus of Rupture***

Like elastic modulus, PCC modulus of rupture is an important parameter that needs to be determined as accurately as possible since it has a significant effect on the slab thickness. It can be estimated from indirect tensile strength or from backcalculated E. However, it is highly recommended that this be estimated from indirect tensile strength.

### **Estimation from Backcalculated E:**

A correlation between elastic modulus and modulus of rupture was developed by Foxworthy (24). This correlation can be used for quick determination of modulus of rupture using backcalculated E- values.

$$S'_c = 43.5 \left( \frac{E}{10^6} \right) + 488.5 \quad (R^2 = 0.71)$$

where  $S'_c$  = third-point modulus of rupture, psi

E = backcalculated PCC slab modulus, psi

### Indirect Tensile Strength Test:

Cut several 6-inch diameter cores at mid slab and test in indirect tension (ASTM C 496). Compute the indirect tensile strength (psi) of the cores. Estimate the modulus of rupture using the following equation **(34)**.

$$S'_c = 210 + 1.02 IT$$

where

$S'_c$  = modulus of rupture, psi

IT = indirect tensile strength of 6-inch diameter cores, psi.

### Effect of the Magnitude of FWD Load on Backcalculated E :

One of the objectives of this project was to determine the effects of FWD load on the backcalculated E values. In an earlier research work at Illinois, Foxworthy et. al., concluded that consistently higher and often unrealistic E values are backcalculated for low FWD load levels **(24)**.

Foxworthy's work involved airfield pavements that are usually thicker than highway pavements. However, modern highway pavements are often built on strong subbases such as lean concrete base that may affect the backcalculated E-values depending upon the magnitude of the FWD load.

Data Acquisition. A significant amount of FWD data was needed for this task. Since the AHTD's Falling Weight Deflectometer was unavailable due to some equipment problems, FWD data from a SHRP Long Term Pavement Performance study was used. Table 5 shows the details of pavement sections on which FWD data was obtained. Data was obtained on outer wheel path and also in the mid-lane region.

Test Section ID	Pavement Thickness (Flexural Strength)	Base
01	8 in. (550 psi)	6 in. DGAB
02	8 in. (900 psi)	6 in. DGAB
04	11 in. (900 psi)	6 in. DGAB
05	8 in. (550 psi)	6 in. LCB
06	8 in. (900 psi)	6 in. LCB

DGAB: Dense Graded Aggregate Base

LCB: Lean Concrete Base

Loads used : 9000 lbs, 13000 lbs and 17500 lbs.

**Table 5. Pavement Sections used in FWD Backcalculation Analysis**

Analysis of the data showed some interesting results. Back-calculated E-values for pavements built on aggregate bases didn't differ much. In some cases, such as Section 050201C1 midlane, slightly higher e-values were back-calculated for higher FWD load range, which contradicts with Foxworthy's findings. However, for section 050206C1, which is built on Lean Concrete Base, the E-values back-calculated for lower FWD loads exceeded those for higher FWD load by an average of 70,000 psi which agrees with Foxworthy's findings.

Back-calculation Procedures for Multi-layered PCC Pavements. The above mentioned backcalculation procedures are meant for slabs on grade or slabs built on bases that are not very stiff. In cases of slabs built on stiff bases such as lean concrete base, extremely high elastic moduli will be back-calculated for slabs unless the effect of stiff base is considered in back-calculation. One way to consider this is to use the total thickness i.e. thickness of the slab + thickness of the base in the back-calculation equations.

A different method has been presented by Ioannides and Khazanovich for back-calculation elastic moduli for three layered concrete pavements (35). A brief account of the procedure is given below.

- The elastic modulus of the slab is calculated using just the slab thickness.. This will be known as the effective E-value or E<sub>eff</sub>. Now, depending on the type of subbase, a modular ratio β has to assumed:

$$\beta = \frac{E_{Base}}{E_{Slab}}$$

- After assuming a proper value for β, the Elastic Moduli of the two layers are given by the following formulas:

$$E_{Slab} = \left[ \frac{h_1^3}{h_1^3 + \beta h_2^3} \right] E_{eff}$$

$$E_{base} = \beta(E_{Slab})$$

Since the Modular Ratio β has to be assumed, the accuracy of the E-values depend upon the accuracy of β. Hence, when dealing with slabs built on stiff bases, it is better to determine the E-values by testing cores rather than by back-calculation. When back-calculation methods are used, proper engineering judgment should be used and unrealistic E-values should not be used blindly.

Effect of Temperature on Backcalculated E-values. As far as the effect of temperature on E-values is concerned, Foxworthy et. al., concluded that only temperature extremes substantially influence the back-calculated dynamic E values. Temperature fluctuations between 4 and 32 C (40 and 90 F) are relatively insignificant, producing little variation in addition to that which is already inherent in the equipment and pavement materials. However, the overwhelming temperature effect occurs at the joint, where load transfer plays an important role in the pavement response to load.

### ***Load Transfer Coefficient J***

For bonded JPCP and JRCP overlays, the J-factor should be determined by testing the joint load transfer efficiency of the existing pavement. Joint load transfer should be determined on the outer wheel path by using a Falling Weight Deflectometer. The procedure involves loading one side of the joint and measuring the deflections on either side.

The AASHTO guide suggests that the load plate be placed on one side of the joint with the edge of the plate touching the joint. The deflections should be measured at the center of the plate and at 12 inches from the center. Since the load plate is about 5.9 inches in diameter, this configuration enables to take deflection readings at about 6 in. on either side of the joint.

The AHTD practice is to place the load plate at a distance of 10.5 in. to 11.5 in. on the leave-side of the joint and measure the deflections on either side of the joint using sensors #2 (8 in. from load plate) and #3 (12 in. from load plate). Since these sensors are only 4 inches apart, deflections can be measured very close to the joint - at 2 in. on each side. **However, it is very important that the sensors must be located in such a way that the joint is equidistant from the two sensors.**

The deflection load transfer can be computed from the following equation:

$$\Delta LT = 100 * \left( \frac{\Delta_{ul}}{\Delta_l} \right) * B$$

where  $\Delta LT$  = deflection load transfer, %

$\Delta_{ul}$  = deflection of the unloaded side, inches

$\Delta_l$  = deflection of the loaded side, inches

B = slab bending correction factor.

The slab bending correction factor, B, is necessary because the deflections measured by the two sensors would not be equal even if measured in the interior of the slab. This is due to the bending of the slab. An appropriate value for the correction factor may be determined from the ratio of  $d_8$  to  $d_{12}$  for typical center slab deflection basin measurements as shown in the equation below.

$$B = \frac{d_8}{d_{12}}$$

( $d_8$  and  $d_{12}$  measured at the interior)

Analysis of SHRP FWD data has shown that for sensors placed 4 inches apart, the B factor ranges from 1.04 to 1.05. Hence, it is reasonable to assume the value of B as 1.045.

If a single overlay thickness is being designed for a uniform section, compute the mean deflection load transfer value of the joints tested in the uniform section. Table 6 gives the value of J for JPCP and JRCP depending on the mean load transfer efficiency.

Percent Load Transfer	J
>70	3.2
50 - 70	3.5
<50	4.0

Table 6. AASHTO Load Transfer Coefficients (J-Factor) (18).

If the overlay construction includes a tied PCC shoulder, a lower J factor may be appropriate.



For CRCP, a J-value of 2.2 to 2.6 may be used for overlay design, assuming that working cracks and punchouts are repaired with continuously reinforced PCC.

**Effect of Temperature on Load Transfer Efficiency:**

Research has shown that temperature can affect the load transfer efficiency to a great extent. Foxworthy et. al., mention that in rigid pavements, temperature changes influence load transfer efficiency more than any other characteristic of the system. This temperature effect is composed of both curling effects and expansion and contraction effects. However only the combined effect is considered important in joint load transfer efficiency **(24)**.

Research has shown that for transverse dummy groove joints and longitudinal key joints, the load transfer efficiency approaches 100 percent as the temperatures increase and a minimum value of 20 to 25 percent as the temperature decreases **(25)**. Hence, it is very important that load transfer tests be done only when the ambient temperature is less than 27 C (80 F).

**Effective k-value**

The effective static k-value may be obtained by backcalculation using deflection data. The following equations are used in back-calculation **(25)**:

$$AREA = 6 * \left[ 1 + 2 \left( \frac{d_{12}}{d_0} \right) + 2 \left( \frac{d_{24}}{d_0} \right) + \left( \frac{d_{36}}{d_0} \right) \right]$$

$$l_k = \left[ \frac{\ln \left( \frac{36 - AREA}{1812.279133} \right)}{-2.559340} \right]^{4.387009}$$

$$k = \left( \frac{P}{8d_0 l_k^2} \right) \left\{ 1 + \left( \frac{1}{2\pi} \right) \left[ \ln \left( \frac{a}{2l_k} \right) + \gamma - 1.25 \right] \left( \frac{a}{l_k} \right)^2 \right\}$$

where

- $d_{0, 12, 24, 36}$  = deflections @ 0 in, 12 in, 24 in, 36 in. (inches)
- $l_k$  = radius of relative stiffness
- $k$  = Modulus of subgrade reaction ( effective dynamic k, pci)
- $P$  = FWD load, pounds
- $a$  = load radius, in. (usually 5.6 in)
- $\gamma$  = Euler's constant, 0.57721

Deflection data should be collected on the outer wheel path along the project at an interval sufficient to adequately assess the conditions. Intervals of 30 m to 300 m (100 ft. to 1000 ft) are typical. A load magnitude of 9000 lbs. or more is recommended. The k-value should be obtained for each slab tested.

The k-value backcalculated from NDT data is a dynamic k-value whereas the required input to the AASHTO design equation is a static k-value. In an analysis of AASHTO Road Test Data, dynamic repeated-load k-values were found to exceed static values by a factor of 1.77 on the average **(23)**. Research work by Foxworthy involving seven Air Force Base pavements indicated that dynamic k-values exceeded static k values by a factor of 2.3 on the average. Reducing backcalculated k-values by 2 has been found to produce reasonable values for static k-values **(25)**. Hence it is recommended in the overlay design procedures that backcalculated k-values be divided by 2 to obtain static k-values for determining  $D_f$ .

If a single overlay thickness is being designed for a uniform section a mean k-value may be used. However, experience has shown that k-value for edge conditions is two to four times larger than that for center conditions for the

same site. Uzan et. al., **(26)** suggest that one should use the values obtained only for the same conditions that prevailed in their derivations.

However, it should be noted that k-value can change substantially and have only a small effect on overlay thickness. Darter concluded in a recent study that even 50% error in estimating k-value can cause only a 5% error in new rigid pavement slab thickness ( $D_f$ ) **(21)**. The error will be even smaller in terms of overlay thickness ( $D_{OL}$ ). Hence, using an average value for k should not lead to serious errors.

The AASHTO guide provides procedures to adjust the k-value for seasonal effects and loss of support. However, seasonal adjustment is inconsistent with lowest springtime gross k-value used in the AASHTO model **(21)**. Also, an earlier research project at the University of Arkansas showed that freezing of the subgrade is not a problem in the lower two-thirds of the Arkansas **(27)**. Also, Arkansas doesn't have a defined rainy season. In light of this, it is not recommended that the k-value should be adjusted for seasonal effects.

As far as adjusting the k-value for loss of support is concerned, an investigation by Darter revealed that substantial loss of support is already built into the model from AASHO Road Test and no further adjustment is needed **(21)**. Additional reduction of k-value for loss of support may lead to overdesign. Hence engineering judgment should be used when using the loss of support criteria. If the pavement doesn't have a strong non-erodible base it may be necessary to use loss of support when constructing unbonded overlays. However, as stated earlier, k-value can doesn't affect the thickness significantly.

Unlike in unbonded overlays, curling stresses are not severe in a bonded overlay due to the monolithic action of the overlay and the existing pavement. Actually, if full bonding is successfully achieved, slab curling will be reduced due to increased thickness of the resulting slab.

**Summary: Design Inputs for  $D_r$ , Bonded Overlays**

- k-value can be back-calculated by deflection testing
- Obtain deflection data at 100 ft. to 1000 ft. intervals in the outer wheel path
- Use an FWD load of 9000 lbs. or greater
- If a single overlay thickness is being designed, compute the mean k-value for use in design
- Adjusting k-value for seasonal effects and loss of support is not recommended
- The k-value doesn't need to be estimated with great accuracy. An error of 50% or less will not affect the resulting overlay thickness significantly

**Effective Slab Thickness of The Existing Pavement ( $D_{eff}$ )**

The effective thickness of the existing slab ( $D_{eff}$ ) depends upon the amount of durability cracking, fatigue cracking and unrepaired joints and cracks.

$D_{eff}$  is computed from the following equation (18):

$$D_{eff} = F_{jc} * F_{dur} * F_{fat} * D$$

Where  $F_{jc}$  = Joints and cracks adjustment factor

$F_{dur}$  = Durability adjustment factor

$F_{fat}$  = Fatigue damage adjustment factor

Proper values are assigned to the various factors after conducting a condition survey. *It should be noted, however, that a bonded overlay is not a feasible option if the amount of distress is severe.*

### ***Joints and Cracks Adjustment Factor ( $F_{jc}$ )***

Since unrepaired joints and cracks in the existing slab will reflect through a bonded concrete overlay, it is recommended that all deteriorated joints and cracks and any other major discontinuities be full-depth repaired with doweled or tied PCC repairs prior to the overlay, so that  $F_{jc}=1.00$ .

If it is not possible to repair all deteriorated areas, then the  $F_{jc}$  factor is determined as follows depending on the presence of “D” cracking:

#### ***Pavements With No “D” Cracking Or Reactive Aggregate Distress:***

For existing pavements with no “D” cracking or reactive aggregate distress (i.e. alkali-silica reaction), obtain the following information:

- Number of unrepaired deteriorated joints/mile
- Number of unrepaired deteriorated cracks/mile
- Number of unrepaired punchouts/mile
- Number of expansion joints, exceptionally wide joints (> 25 mm [1 in]) and full depth, full-lane-width AC patches/mile

Tight cracks held together by reinforcement in JRCP or CRCP should not be included. However, if the crack is spalled and faulted, the crack should be considered as working. Surface spalling of cracks in CRCP is not an indication that the crack is working.

The total number of unrepaired deteriorated joints, cracks, punchouts, and other discontinuities per mile is used to determine the  $F_{jc}$  factor from a figure in the AASHTO Guide **(18)**.

#### ***Pavements with “D” Cracking or Reactive Aggregate Deterioration:***

If the pavement is suffering from D cracking or reactive aggregate deterioration,  $F_{jc}$  should be determined by considering only those deteriorated joints and cracks that are not caused by durability problems. Distresses related to durability problems are considered separately under  $F_{dur}$ . If all the deteriorated joints and cracks are spalling due to “D” cracking or reactive aggregate, then  $F_{jc}$  should be assigned a value of 1.0. This will avoid “double-adjusting” the  $F_{jc}$  and  $F_{dur}$  factors.

***Durability Adjustment Factor ( $F_{dur}$ ):***

Durability cracking or “D” cracking is caused by the use of non-durable material and/or climatic conditions which results in disintegration of concrete. This type of cracking is progressive in nature and will gradually cover increasingly large areas until nearly complete deterioration might result (29). Hence bonded overlays should not be constructed on pavements suffering from severe durability problems.

A distress survey should be conducted to obtain information about durability problems and  $F_{dur}$  is determined as follows:

- No sign of PCC durability problems: 1.0
- Durability cracking exists, but no spalling 0.96-0.99
- Cracking and spalling exist (*bonded overlay NOT recommended*) 0.80-0.95

***Fatigue Damage Adjustment Factor ( $F_{fat}$ ):***

$F_{fat}$  depends on the amount of transverse cracking (JPCP, JRCP) or punchouts (CRCP) that is caused mainly by repeated loading and is determined as shown in Table 7. *Transverse cracks and punchouts caused mainly by*

*durability problems (D-cracking or reactive aggregates ) should not be included under fatigue damage.*

Amount of Distress	$F_{fat}$
Few transverse cracks/punchouts exist JPCP: < 5 percent of slabs are cracked JRCP: <25 percent working cracks per mile CRCP: < 4 punchouts per mile	0.97 - 1.0
A significant number of transverse cracks/punchouts exist JPCP: 5 - 15 percent slabs are cracked JRCP: 25 - 75 working cracks per mile CRCP: 4 - 12 punchouts per mile	0.94 - 0.96
A large number of transverse cracks/punchouts exist JPCP: >15 percent slabs are cracked JRCP: >75 working cracks per mile CRCP: > 12 punchouts per mile	0.90 - 0.93

**Table 7. Determination of  $F_{fat}$  Factor for Bonded Overlays (18).**

#### **Determination of Existing Slab Thickness:**

This is an important parameter in overlay design. While historic information may be available, the extreme importance and sensitivity of this variable calls for the use of destructive testing to verify the available historic information. A limited amount of coring at randomly selected locations may be used to verify the historic information (18). These cores can also be used for determining the PCC modulus of rupture and PCC Elastic modulus.

## CONCLUSIONS

As stated earlier, most overlay failures can be attributed to causes other than proper overlay thickness. Hence it is necessary to know and understand the general causes of overlay failures and the steps that should be taken to avoid them. The listing that follows provides general conclusions for both Bonded and Unbonded overlays, resulting from the studies performed under this project.

- Nearly all the documented cases of premature concrete overlay failure are due to lack of uniform support conditions. **(2)**
- All tipping and rocking slabs must be stabilized by slab jacking or sealed by using heavy rollers to provide uniform support for the overlay. **(6)**
- If the existing pavement is suffering from extreme distress, a thicker bond breaker ( $\geq 50$  mm [2in]) should be used. **(18)**
- In most cases, minimum thickness for unbonded overlays will be 175 to 200 mm (7 to 8 in). **(6)**
- An unbonded overlay is a good rehabilitation candidate for severely D-cracked pavements. **(3)**
- Proper selection of the interlayer material is critical to the performance of the unbonded overlay. **(3)**
- Due to stiff support from the existing pavement, curling stresses are very high in unbonded overlays.
- Short joint spacing or continuously reinforced design will alleviate high curling tensile stresses in the overlay caused by curling action. **(3)**
- For non-reinforced unbonded overlay, a joint spacing (in feet) should not exceed 1.75 times the overlay thickness (in inches). **(3)**



- For reinforced unbonded overlays, a joint spacing of 10 m (30 ft) will result in improved performance. **(3)**
- In unbonded overlays, deliberate mismatching of the overlay joints has been shown to reduce pumping action and thus, extend the service life of overlay. It is recommended that the joints be placed at least 1 m (3 ft) from existing transverse cracks or working cracks. **(5)**
- Transverse and longitudinal joints must be sawed as soon as possible to relieve initial stresses.
- Multiple cracking in the CRC overlay over the existing joints will occur if moving slabs are not stabilized. **(22)**
- If a relatively thin unbonded overlay is going to be built, the existing pavement must be properly prepared (undersealed, broken slabs replaced, patched, etc.). **(22)**
- Longitudinal cracking in unbonded overlays can be attributed mainly due to late sawing or improper saw depth of the longitudinal centerline joint.

## COMPUTER PROGRAM

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As seen in Chapter 2, the AASHTO design procedure involves complex mathematical equations and cumbersome procedures to determine the thickness of rigid pavement. The 1993 AASHTO Guide for Design of Pavement Structures contains a nomograph for designing the thickness of rigid pavement. However, nomographs have certain limitations- they are prone to errors and are time consuming. Hence, user friendly spreadsheets were to easily design bonded or unbonded overlays. The paragraphs that follow briefly describe the operation of the spreadsheets.

### **Procedure:**

The worksheets have been developed in Excel™ Version 7.0 for WINDOWS™ '95. The user just needs to open the required file (Bonded or Unbonded) in Excel and enter the right data in the right location. A basic knowledge of WINDOWS and Excel will be quite helpful. However, the worksheets do not require the designer to do anything other than entering data and using the mouse.

### **Opening the Worksheet:**

*(Before using the program, it is recommended that a back-up of the worksheets be made on a separate disk and store it aside)*

Open the required Worksheet as you would any spreadsheet file. You can do this three ways:

1. By pressing "Ctrl+O"
2. By clicking the "Open" Icon
3. By using the menu on the top

Once the worksheet is opened, you can adjust the Zoom to suit your convenience. This can be done by clicking the Zoom icon in the tool bar.

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*Note: EXCEL™ and WINDOWS™ are registered trademarks of the Microsoft Corporation.*

**Entering Data:**

Once the worksheet is opened, the user can click the "Instructions " button on the top right hand corner which will give some information on where to enter the data, how to move to different sheets, etc. The worksheets are protected to prevent accidental deletion of formulas. The user can enter the data only in certain cells which have been highlighted yellow.

Once the data is entered in a certain cell, pressing the "Tab" key moves the cursor forward in sequence to the next data cell. If the user wants to go back, pressing "Shift+Tab" will move the cursor backward. Of course, the user can also use the arrow keys to move to different cells on the work sheet. However, using the Tab key will move the cursor only to those cells which accept the data.

**Calculation:**

In Excel, calculations can be done either automatically or manually. "Automatic" option means that calculations are done automatically each time the content of any cell is changed. "Manual" option means that the user tells the computer when to do the calculation. In other words, the user can enter all of the data and tell the computer in the end to perform the calculation. The "Manual calculation" feature prevents the computer from performing unnecessary calculations during the intermediate steps.

The user can chose between the two options by clicking the corresponding buttons at the top. When the user has chosen the "Manual Calculation" option, the computer will not perform any calculations until the user clicks the "Calculate" button or presses the F9 key. Hence it is very important to remember this whenever the user changes data.

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