STRUCTURAL ADEQUACY OF RUBBLIZED PCC PAVEMENT

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INTRODUCTION

There are several Portland Cement Concrete (PCC) rehabilitation techniques. Some of these techniques are commonly used in Indiana. These are: overlay, crack-and seat with overlay, break-and-seat with overlay, and total reconstruction. With the exception of the most expensive technique in terms of initial cost (i.e. total reconstruction), these options have exhibited various shortcomings in the past. Rubblizing is a relatively new technique that is used for rehabilitation of PCC pavements. In this technique, the concrete PCC slab is reduced in-place to approximately aggregate base material size.

In 1991, the Indiana Department of Transportation (INDOT) awarded a rubblizing contract for a portion of US-41 in Benton County in preparation for overlay. The INDOT Division of Research evaluated the pavement prior to rubblizing and again after rubblizing & overlay in order to estimate the AASHTO "layer coefficient (*1*)" of the rubblized concrete slabs.

The AASHTO pavement design procedure requires estimating the layer coefficient. However, currently there is little existing data upon which to make such an estimate. At the time of the US-41 contract award, only 19 pavement sections in 9 states had been rubblized. The layer coefficient for the rubblized PCC layer for most of these designs had been extrapolated from AASHTO recommendations for crushed aggregate base.

As a result of the analysis of the 19 existing sections, layer coefficient values in the practical range of 0.23 to 0.31 was recommended by PCS/Law (*2*) in a report dated June 1991. This range is useful to the extent that it demonstrates that the layer coefficient of a rubblized concrete pavement can be approximately twice that of a crushed aggregate base (layer coefficient = 0.14).

OBJECTIVES

This study is limited to an analysis of the relative strengths of the US-41 pavement prior to, and after rubblizing. The study includes a pre-analysis of the existing pavement to ensure section uniformity, and a post-construction analysis designed to yield estimates for the layer coefficient of the in-situ rubblized pavement.

COMPARISON METHODS

Crack and Seat

Traditionally, crack and seat is used to eliminate reflection cracking in the Hot Mix Asphalt (HMA) overlay. The major objective of the crack and seat technique is to reduce accumulation of horizontal slab movement to a level that it can be accommodated by the HMA overlay. Ideally the crack and seat technique should produce pieces of the slab 0.3-1.0 meters (1-3 feet) in length to achieve this objective.

Cracking is intended to produce tight cracks that permit load transfer with little loss of structural value. On the other hand, seating of the cracked concrete slabs is intended to re-establish support of the foundation for the PCC slab. Crack and seat has been used successfully for rehabilitation of Jointed Plain Concrete Pavements (JPCP) that are generally structurally sound, but have functional distresses in the form of roughness, patching, spalling, etc. When this technique is used, consideration should be given to any loss of slab integrity.

Break and Seat

The objective of break and seat is similar to that of crack-and seat. However, break and seat is used with Jointed Reinforced Concrete Pavements (JRCP). This technique requires that the bond between the concrete and reinforcement steel be destroyed. This lack of bond minimally reduces the differential movements at working joints and cracks (*2*).

Because of the bond disruption, the amount of energy required to reduce the slab size is greater than that required for crack and seat. The reduction of structural capacity due to disruption of the reinforcing steel/concrete bond should be considered during overlay design. If pavement deterioration is such that little slab integrity can be preserved after breaking, rubblization or reconstruction should be considered as alternative procedures.

Rubblizing and Overlay

The objective of rubblization is to eliminate reflection cracking in the HMA overlay by the total destruction of the existing slab action. Rubblization is applicable when there is minimal slab integrity and structural capacity of the original JRCP. It has also been used successfully for rehabilitation of other PCC pavement types. Typically, the slab is reduced to small pieces and diminished to “a high-strength granular base (*2*).” Restoration of structural capacity is accomplished with an HMA overlay.

SITE DESCRIPTION

The rubblization project was on a portion of US-41 in Benton County, Indiana, extending from the northern limits of the city of Boswell to approximately 22 km (14 miles) north of the Benton/Newton County line. The northbound lanes were to be rubblized, with the exception of sections adjacent to various structures such as bridges, culverts etc. Approximately 14 km (9 miles) of the southbound lanes were also rubblized. The rehabilitation work was divided into two contracts. The northern contract included approximately 9 km (6 miles) of the northbound lanes and all of the 14-km (9 miles) of the south-bound lanes between US-24 and the US-24 & US-52 intersection. The southern contract included approximately all of the remaining 14 km (9 miles) of the northbound lanes between SR-352 and the US-41 & US-52 intersection.

Project activities were divided into rubblization, patching and HMA overlay. Within the limits of the rubblizing contract, the north-bound pavement consisted of 14 km (9 miles) of 200 mm (8 inches) thick CRCP on 150 mm (6 inches) of uniform subbase, and 9 km (5.7 miles) of 255 mm (9 inches) thick JRCP slabs on a variable 125-200 mm (5-8 inches) thick subbase. The southbound pavement consisted of 14-km (9 miles) of 255 mm (9 inches) thick JRCP slabs on a variable 125-200 mm (5-8 inches) thick subbase. Table 1 gives the construction activities on these pavement sections.

Generally, the existing concrete pavement was in poor to very poor condition. Significant faulting and slab cracking were exhibited by most of the joints in the JRCP. Punchouts were common in the CRCP. There was visual evidence of pumping. Much of pavement had been undersealed with bituminous material. The INDOT - District reported that more than one application of undersealing had been undertaken on these sections of pavement. INDOT uses bituminous undersealing for loss-of-support of concrete and HMA overlaid concrete pavement structures. US-41 Highway provides a convenient route south from Chicago, Illinois to

Evansville, Indiana with connections to both I-70 and I-74. Therefore, the truck traffic is heavy (approximately 3.9 million ESALs per year).

PRE-CONSTRUCTION TESTING

In preparation for the rubblizing contract, the INDOT - Division of Research tested the entire length of both northbound and southbound lanes using a Dynatest Falling Weight Deflectometer (FWD). This information was used to select two test sections for the study, each 1.6 km (1 mile) in length. The sections identified were (stations are based on the mile/foot):

Section 1: From station 409+00 to 460+00 (northbound driving lane) between Boswell and the US-52 junction.

Section 2: From station 515+00 to 525+00 and from station 210+00 to 250+00 north of the US-52 junction.

PRE-CONSTRUCTION ANALYSIS

In performing the analysis to determine the moduli of the concrete and subgrade, it was necessary to employ an algorithm appropriate to the construction. There are several software solutions available, such as MODULUS, ELSBACK, WESBACK, MODCOMP, etc. However, while these are generally applicable to multi-layer elastic systems, they are better suited to the analysis of flexible pavement systems. Hall (*3*) reported an improved system for the analysis of rigid pavement systems that has since been incorporated in AASHTO (*4*) recommendations for analysis of rigid pavement systems. This analysis procedure was applied to the data from this project.

The recorded pavement deflection basins are reduced to a normalized parameter, AREA2, by dividing the area of the deflection basin between the axis of load and a radial offset of 914 mm (36 inches) by the deflection at 0 mm offset. Only the deflections measured at 0, 305, 610 and 914 mm (0, 12, 24 and 36 inches) offsets are used. Thus:

$$AREA2 = 6\left(\frac{d(0) + 2d(12) + 2d(24) + d(36)}{d(0)}\right) \qquad (1)$$

This parameter has been analytically identified (*5*) as having a relationship to the radius of relative stiffness (l_k or l_e) of the pavement system. The radius of relative stiffness is calculated as:

$$l_k = \left(\frac{E_{pcc} D_{pcc}^3}{12(1-\mu^2{}_{pcc})k}\right)^{1/4}$$

$$l_e = \left(\frac{E_{pcc} D_{pcc}^3 \left(1-\mu^2{}_s\right)}{6\left(1-\mu^2{}_{pcc}\right)E_s}\right)^{1/3} \qquad (2)$$

where,

E_{pcc} = The elastic modulus of the PCC slab,

E_s = Elastic modulus of the supporting medium,
μ_{pcc} = The Poisson's ratio of the concrete slab,
μ_s = The Poisson's ratio of the supporting medium,
D_{pcc} = The thickness of the slab (inches), and
K = The modulus of the subgrade reaction.

The difference between l_k and l_e relates to the subgrade support models for a dense liquid (l_k) and the elastic solid (l_e) respectively. Traditional rigid pavement design is based on the dense liquid model. However, highway designers (as opposed to airfield designers) rarely measure the modulus of subgrade reaction, k, taking instead an estimate based on correlation with the California Bearing Ratio (CBR). The elastic solid model has become more widely applied since the advent of FWD backcalculation technology. With the FWD estimates of the subgrade, in-situ elastic response are easily determined. The elastic solid model was used for the US-41 analysis. Hall (*3*) provides a relationship between the radius of relative stiffness, l_e, and AREA2 .

$$l_e = \left(\frac{\ln\left(\dfrac{36 - AREA2}{4521.676}\right)}{-3.654} \right)^{\frac{1}{0.187}} \tag{3}$$

Using this estimate of the radius of relative stiffness, the elastic modulus of the subgrade, E_s, may be estimated using Losberg's Deflection Equation (*6*).

$$E_s = \left[\frac{2p(1-\mu_s^2)}{d_0 l_e}\right]\left[.019245 + .0272\left(\frac{a}{l_e}\right) + 0.0199\left(\frac{a}{l_e}\right)^2 \ln\left(\frac{a}{l_e}\right)\right] \tag{4}$$

Applying this result to Equation 2, an estimate of the elastic modulus of the PCC slab may be obtained.

$$E_s = \frac{E_c D_c^3 (1-\mu_s^2)}{6 l_e^3 (1-\mu_s^2)} \tag{5}$$

When these transformations are applied to the FWD data obtained from sections 1 and 2 on US-41, the overall uniformity of the sections is confirmed. The results are shown in Table 2. From these results, it is clear that the elastic moduli of the concrete slabs tested are not significantly different. Also, the elastic moduli are close to the normal design value of 27,579 MPa (4,000,000 psi). Section 1 and 2 are more than 1.6 km (1 mile) apart. The variation in the foundation moduli (which includes the subbase layer) is also reasonable.

POST-RUBBLIZING ANALYSIS

After completing the patching, rubblizing, and overlay, FWD tests were repeated at the same locations tested before rubblization. In this instance, since the pavement approximates a flexible pavement structure, the previous analysis was no longer appropriate. Two new analyses were applied. The first, the BOUSDEF (*7,8*) software (Oregon State University), was used to determine the extent, if any, of subgrade nonlinear response. BOUSDEF is the only program available to the INDOT - Division of Research that can make this determination. It is known that many subgrade soils exhibit a non-linear elastic response that is different for cohesive and non-cohesive soils. The non-linear models are:

$$E = K_1 \Theta^{k_2}$$
$$E = K_3 \sigma_d^{k_4} \qquad (6)$$

Where,

Θ = the first stress invariant,
σ_d = the deviator stress, and
k_i = experimental constants.

The BOUSDEF analysis revealed that the effect of non-linearity is marginal. Consequently, linear elastic response was accepted for the subgrade. Having determined that the subgrade response was essentially linear, the same data was analyzed using MODULUS (*9*,*10*,*11*). This program is capable of backcalculating layer moduli and detecting the presence of any real, or effective, lower rigid boundary. A limiting lower boundary conditions can affect the predicted response of a pavement analyzed as a layered elastic problem.

The average results of the MODULUS backcalculations are shown in Table 3. There is a reasonable degree of uniformity. The difference in the modulus of the AC overlays in section l and 2 may be partially accounted for by the difference in pavement temperatures. The two sections were tested on successive days, and a 7°C (12°F) difference was measured in-situ. Variability of computed subgrade and rubblized layer moduli is small.

It should be noted that the subgrade moduli before and after rubblization are different. This may be a reflection of the different modes by which rigid and flexible pavements transmit load to the subgrade. Rigid pavements transfer the load in an entirely different manner from flexible pavements. This difference may also be explained by the two different analysis approaches that were used to compute the moduli.

LAYER COEFFICIENTS

The layer coefficient concept is derived from the AASHO Road Test in which the structural capacity of the flexible pavement is represented by a single parameter, the structural number (SN). The structural number is determined by SN= $a_1t_1 + a_2t_2 + a_3t_3$ and assuming a drainage coefficient of one for each layer. Examination of the structural number concept in terms of an analogy to the Odemark Hypothesis (*12*) of equivalent thickness shows that:

$$h_e = h_1 \sqrt[3]{\frac{E_1}{E_{ref}}} + h_2 \sqrt[3]{\frac{E_2}{E_{ref}}} + h_3 \sqrt[3]{\frac{E_3}{E_{ref}}} \qquad (7)$$

where,

h_e = the equivalent thickness,
h_1, h_2, h_3 = the thickness of layer 1, 2, and 3 respectively,
E_{ref} = the reference modulus (75,644 MPa), and
E_1, E_2, E_3 = the elastic modulus of 1st, 2nd and 3rd layers respectively.

By analogy, the layer coefficient of the second layer, a_2, would be evaluated as (*2*)

$$a_2 = \sqrt[3]{\frac{E_1}{E_{ref}}} = 0.0045\sqrt[3]{E_2} \qquad (8)$$

Where, in order to provide results equivalent to the layer coefficients derived at the AASHO Road Test, E_{ref} would have to be equal to 75,664 MPa (10,974,000 psi). Equation 8 has been applied to the data from sections 1 and 2 and the results are reported in Tables 4 and 5. Table 6 provides a summary of the derived layer coefficients for the rubblized layers from the two sections.

Figure 1 through Figure 8 illustrates the *Cumulative Frequency Distribution* of backcalculated layer moduli and computed layer coefficients (a_2) for both sections. The *Cumulative Frequency Distribution* shows that the data are almost normally distributed. These results are remarkably consistent. The rubblized layer coefficient, a_2, has a mean value of 0.25. However, the distributions of the derived layer do differ. The layer coefficient for section 2 is less variable than that observed in section 1. In comparison, the PCS/Law (*2*) report gives a mean value of $a_2 = 0.33$ with a standard deviation of 0.15. Also, the US-41 results ($a_2 = 0.25$) correspond to a reliability of 94 percent, where-as only six percent of the PCS/Law values are less than 0.25.

It should be borne in mind that the PCS/Law recommendations are based upon results from a number of states, and therefore reflect differences in material specifications and construction practices. They must therefore be expected to demonstrate a higher degree of variability than values obtained from a single state agency, and more specifically from a single stretch of highway. For example, the observed variabilities in Indiana (overall coefficient of variability or COV = 8 percent) are significantly less than those reported by PCS/Law (COV = 13 percent). The apparent difference in average layer coefficient may be attributed to a number of sources. Sources include different concrete strength specifications, different disruptive responses to the rubblizing equipment (i.e., the size distribution of the resulting rubblized concrete), and differences in rubblizing equipment (frequency of applied vibration, speed of travel, etc.).

DISCUSSION

Rubblizing concrete pavement reduces heavily distressed PCC slabs to a uniform granular structure, thereby providing a structurally valid base/subbase layer in a flexible pavement structure. Rubblizing destroys the slab action of rigid pavements and this loss of structure must be accounted for in the HMA overlay design thickness. The results of one project in Indiana shows an AASHTO Layer Coefficient of 0.25. No adjustment should be made to this value for reliability. Reliability is dealt with separately in the AASHTO design process. The layer coefficient (a_2) determined in this study represents a value two standard deviations less than that reported by PCS/Law (*2*).

Currently, INDOT uses a layer coefficient of 0.20 for rubblized PCC Pavements. Based on the results of this study, the layer coefficients can be set within two standard deviations of the mean (.25). This value of layer coefficient (0.22) represents a reasonable value that is recommended for rubblized PCC pavements with similar conditions.

Cores taken from the rubblized sections after overlay indicate that the thickness of the rubblized concrete slab is, within normal variability ranges, the same as the thickness of the unrubblized slabs at least for design purposes. If INDOT continues to use the rubblization technique in pavement rehabilitation, a further study encompassing slabs of different thicknesses on different subgrades should be undertaken for confirmation of the values that are reported herein. Rubblizing equipment effect should also be considered as a factor in the proposed study.

CONCLUSIONS AND RECOMMENDATIONS

Results of the Indiana study allow the following conclusions to be made:

1. Rubblizing appears to provide a uniform, stable/high-strength granular base/subbase for asphalt concrete overlays.

2. To ensure structural adequacy, a layer coefficient of 0.22 appears to be reasonable for rubblized PCC pavements with similar conditions. This layer coefficient is set within two standard deviation of the mean.

3. Rubblization appears to be a valid option for INDOT to use in the rehabilitation of PCC pavements.

4. If INDOT continues to use rubblization in the rehabilitation of PCC pavements, further studies are recommend. These studies should encompass slabs of different thicknesses on different subgrades. In addition, the effect of the rubblizing equipment should also be examined.

DISCLAIMER

The contents of this paper reflect the views of the authors, who are solely responsible for the facts and the accuracy of the data presented herein. The contents of this paper do not necessarily reflect the official views or the policies of the Indiana Department of Transportation or the Federal Highway Administration at the time of publication. This paper does not constitute a standard, specification or regulation.

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TABLE 1 Construction Activities.

Location	Activity	Length (km)
South-Bound (N)	Rubblizing	13.65
	Patch & Overlay	0.78
	Bridge	0.14
	TOTAL	14.57
North-Bound (S)	Rubblizing	9.00
	Patch & Overlay	3.63
	Remove & Replace	0.06
	Bridge	0.05
	TOTAL	12.75
North-Bound (N)	Rubblizing	7.36
	Patch & Overlay	1.78
	Bridge	0.14
	TOTAL	9.12

TABLE 2 Pre-Rubblization Analysis Summary.

	Concrete Modulus (MPa)		Subgrade Modulus (MPa)	
	Mean	Std. Dev.	Mean	Std. Dev.
Section 1	26,794	5,395	159	23
Section 2	25,478	3,329	195	31

TABLE 3 Summary of Moduli from Rubblized Pavements.

	AC Modulus (MPa)		Rubblized layer (MPa)		Subgrade Modulus (MPa)	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Section 1	8190	1101	1253	376	128	20
Section 2	5967	807	1152	195	159	26

TABLE 4 Summary of Layer Coefficients - US-41 Section #1

	E1	E2	E3	a2
Mean	1187.83	181.763	18.5135	0.25288
Standard Error	22.3679	7.64942	0.39925	0.00356
Median	1186	170.05	17.95	0.25
Mode	1169	#N/A	18	0.27
Standard Deviation	161.297	55.1608	2.87907	0.02569
Sample Variance	26016.9	3042.71	8.28903	0.00066
Kurtosis	-0.79124	-0.00995	0.02502	-0.66695
Skewness	-0.07563	0.61169	0.74557	0.13514
Range	653	232.3	11.7	0.1
Minimum	883	95	13.6	0.21
Maximum	1536	327.3	25.3	0.31
Sum	61767	9451.7	962.7	13.15
Count	52	52	52	52
Confidence Level(95.0%)	44.9054	15.3568	0.80154	0.00715

TABLE 5 Summary of Layer Coefficients - US-41 Section #2

	E1	E2	E3	a2
Mean	865.480769	167.117308	23.0827	0.24654
Standard Error	16.3932051	3.96689517	0.52778	0.00188
Median	848	163.7	22.2	0.25
Mode	931	148.7	18.7	0.24
Standard Deviation	118.213083	28.6056879	3.80586	0.01356
Sample Variance	13974.333	818.285381	14.4846	0.00018
Kurtosis	-0.1073082	-0.04742418	1.72738	0.09629
Skewness	0.42829157	0.5937137	1.31421	0.37243
Range	531	117.6	15.8	0.06
Minimum	646	117.7	18	0.22
Maximum	1177	235.3	33.8	0.28
Sum	45005	8690.1	1200.3	12.82
Count	52	52	52	52
Confidence Level(95.0%)	32.9107071	7.96386824	1.05956	0.00377

TABLE 6 Layer Coefficient Summary.

	Layer Coefficient	
	Mean	Std. Dev.
Section 1	0.25	0.03
Section 2	0.25	0.01

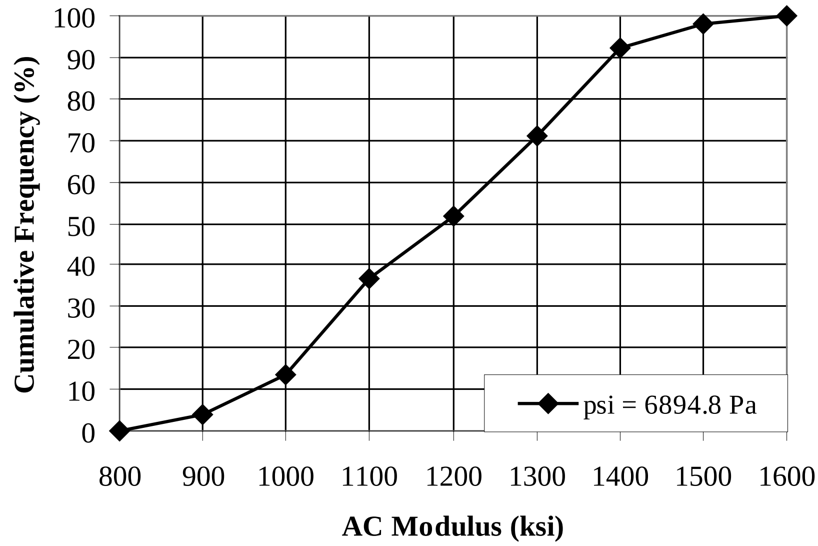

FIGURE 1 US-41 Section 1 - AC Modulus

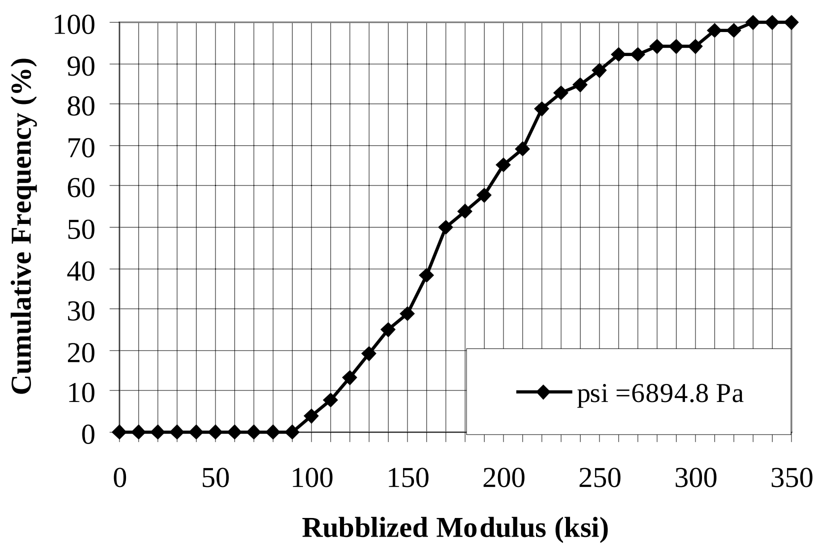

FIGURE 2 US-41 Section 1 - Rubblized PCC Modulus

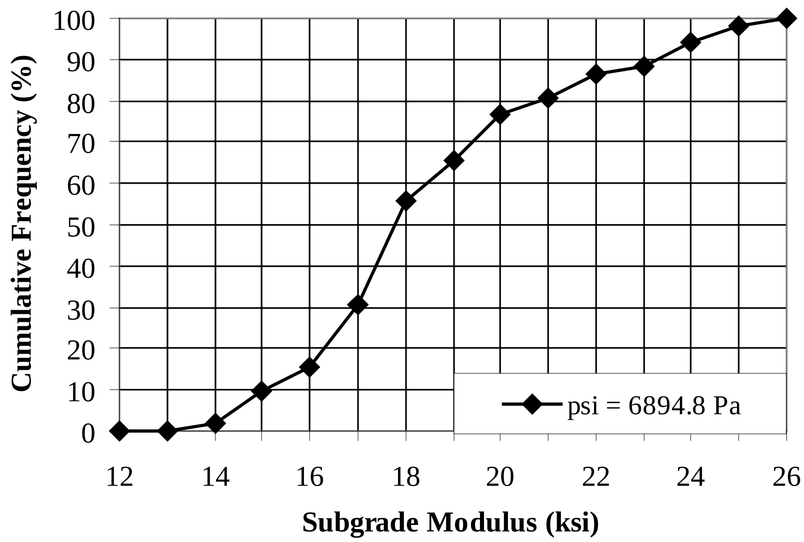

FIGURE 3 US-41 Section 1 - Subgrade Modulus

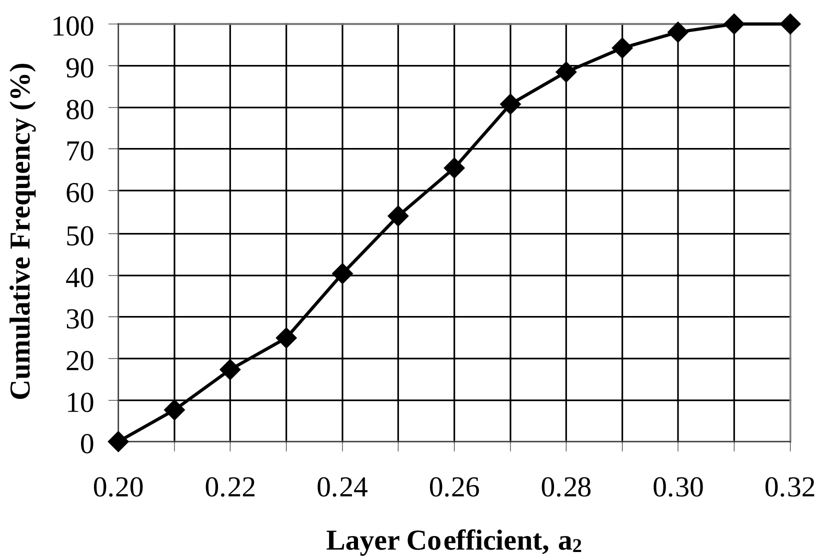

FIGURE 4 US-41 Section 1 - Layer Coefficient, a_2

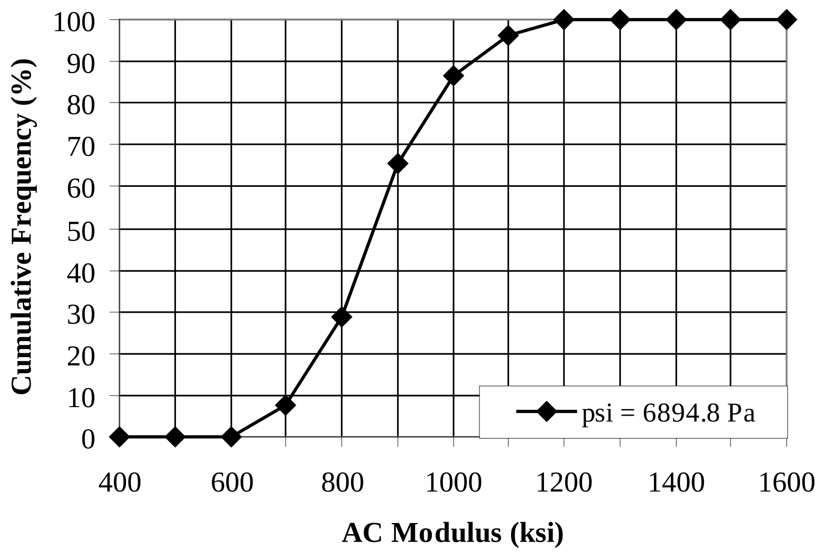

FIGURE 5 US-41 Section 2 - AC Modulus

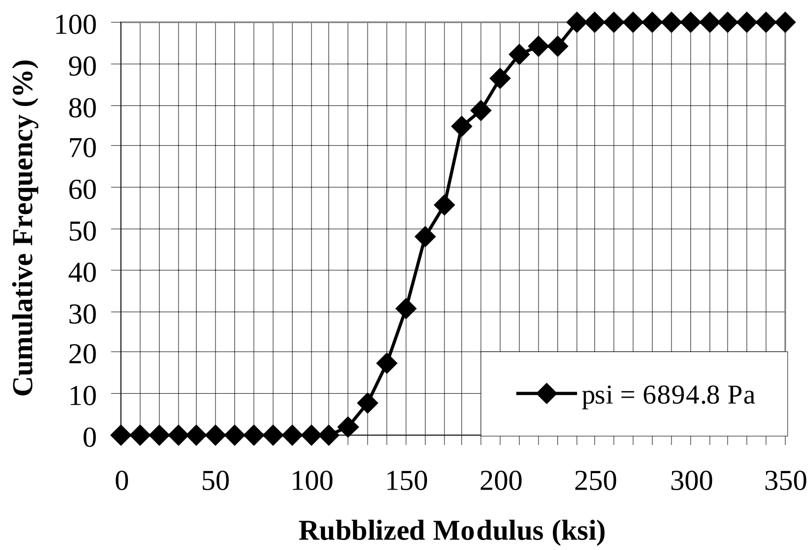

FIGURE 6 US-41 Section 2 - Rubblized PCC Modulus

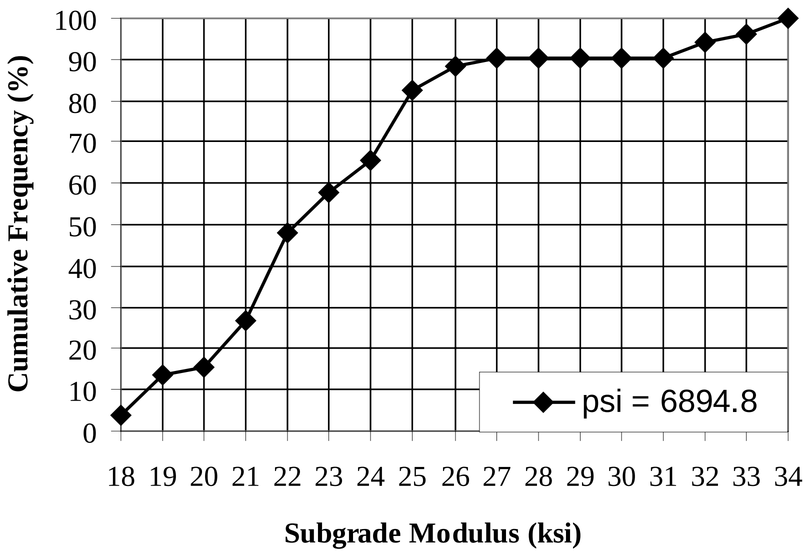

FIGURE 7 US-41 Section 2 - Subgrade Modulus

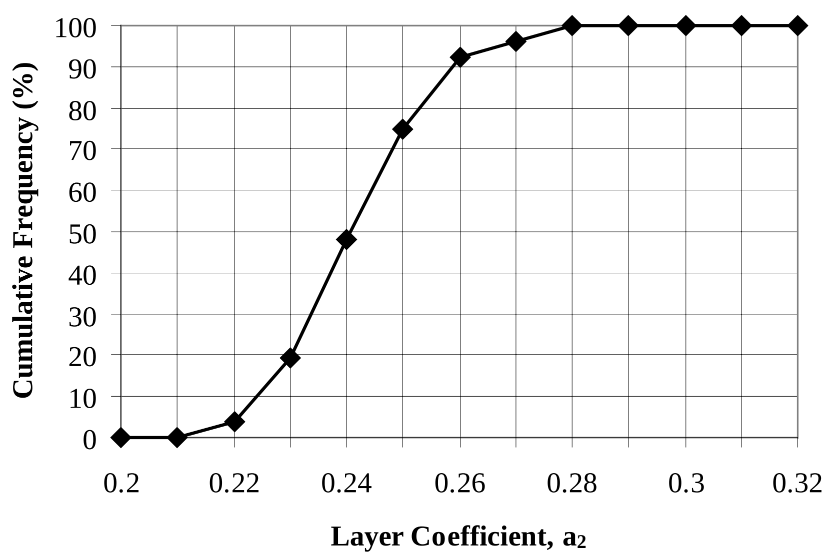

FIGURE 8 US-41 Section 2 - Layer Coefficient, a_2