A Study of *In Situ* Pavement Material Properties

Determined from FWD Testing

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Vermont Agency of Transportation

Pavement Design Committee

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EXECUTIVE SUMMARY

The Vermont Agency of Transportation (Agency) developed pavement design procedures patterned after the American Association of State Highway and Transportation Officials (AASHTO) pavement design model described in the AASHTO 1993 Pavement Design Guide (Guide). While the Guide provides one of the most widely used empirical design models for flexible pavement design, a factor complicating its utility is the use of an abstract quality, the structural number (SN), to quantify the strength of the total pavement structure. A consequence of the SN is the need for structural layer coefficients (a_i) to characterize the component materials of the pavement structure. The Agency found it difficult to quantify these layer coefficients because they are difficult to assess directly, and consequently, found it equally difficult to calibrate the AASHTO model to Vermont conditions.

However, the Agency has developed and tested a method for determining layer coefficients using a falling weight deflectometer (FWD), and the resulting layer coefficients are representative of the *in situ* behavior of the pavement materials. This method is based on a model provided in the Guide for assessing the effective SN of a pavement structure. The Agency found layer coefficients determined for unbound subbases to be reasonable, while layer coefficients estimated for ACC materials were generally 25-35% higher than AASHTO's implied maximum of 0.44. However, a statistical analysis indicates considerable support for the predictive qualities of FWD derived layer coefficients to approximate layer coefficients simulated from the *in situ* conditions expected to prevail in the final pavement structure.

OBJECTIVE

Ever since the Vermont Agency of Transportation (Agency) adopted the American Association of State Highway and Transportation Officials (AASHTO) pavement design method in 1993, one of the most vexing problems facing Agency pavement designers has been the calibration of the AASHTO pavement design procedure for Vermont conditions. Key to this calibration is the determination of the layer coefficients necessary for characterizing Vermont pavement materials. It has been well established by others that there is no direct method for quantifying the layer coefficient for a particular material. The AASHTO Pavement Design Guide (Guide) does provide relationships for determining layer coefficients for several pavement materials (1), however, these relationships were unique to the materials used to build the AASHO Road Test in the 1950s. The Guide cautions against using the relationships provided to characterize local materials. The Guide also suggests that each design organization determine relationships unique to the materials they use to build pavement structures (1). Unfortunately, the Guide stops short of recommending any procedure that may be used to determine layer coefficients, or how to develop models for predicting layer coefficients from some other material property. The Agency's Pavement Design Committee (Committee) undertook a serious investigation into determining layer coefficients for Vermont pavement materials. The Committee evaluated as much research as was available on the topic before proposing the multi-year investigation summarized in this report.

BACKGROUND

There have been several investigations reported using a falling weight deflectometer (FWD) to characterize the structural properties of pavement materials. Zhou, et al. (2), Hossain, et al. (3), and Janoo (4) all used an FWD in one way or another to determine material properties for the constituent pavement materials, some conducting FWD testing on top of each material as the structure was being constructed. However, the Committee did not consider the methods described for determining layer coefficients, utilizing the AASHTO modulus/coefficient relationships provided in the Guide, desirable.

While the layer coefficient relationships provided in the Guide are convenient and tempting to use once a resilient modulus has been established, their use is not necessarily appropriate. The Guide gives no specific direction, but it does emphasize the importance for designers to calibrate various components of the design model to local conditions and experience before implementing the AASHTO procedure. Layer coefficients are certainly no exception to this caveat. Layer coefficients themselves are believed to be a function of material thickness, underlying material support, and stress state. Further, the modulus/coefficient relationships provided in the AASHTO Guide were developed for AASHO Road Test materials as they were constructed at the Road Test site in 1958. The usage of these relationships for materials considerably different from those used at the Road Test is unsubstantiated and can be misleading. Ideally, AASHTO should have provided a procedure for designers to develop their own layer coefficient relationships for the materials with which they commonly build pavement structures.

The AASHTO approach to flexible pavement performance quantifies the pavement structure as a structural number (SN) and further divides the pavement structure into three constituent parts: surface, base, and subbase. Although it is not very clear what conditions or stress states constitute or distinguish the surface, base, or subbase from each other, the interplay among the three pavement components and how they work in concert as a single structure is illustrated by Equation 1,

$$SN = a_1 \times D_1 + a_2 \times D_2 + a_3 \times D_3 \qquad (1)$$

where a_i represents the layer coefficient and D_i is the thickness of the material.

Accordingly, layer coefficients for a particular material can be thought to represent the SN-contribution per unit thickness of that material to the total SN of the pavement structure.

Ideally, what is needed is a way to measure the SN provided by a particular material as a component of a final pavement structure. This method should be relatively easy to perform so that a variety of conditions may be surveyed.

AASHTO METHOD

It was not until the publication of the 1993 edition of the AASHTO Guide that a procedure was provided by AASHTO for determining the in-place SN of a pavement structure using FWD deflection data. This procedure is described in Appendix L of the 1993 Guide and provides a method for determining the "effective structural number," designated as SN_{eff}. However, Ioannides expressed concern about the development of this method, particularly the introduction of mechanistic properties into the statistical/empirical AASHTO model (5). Regardless, the Committee considered the possibility of deriving layer coefficients from SN_{eff} determinations to be worth investigating, particularly since other researchers' efforts with this model have given this method tacit legitimacy. Specifically, if FWD testing were performed on the top surface of each component material in a manner similar to that described by Zhou, et al., and Janoo, the SN_{eff} may be characterized for individual components of a pavement structure. It would follow that layer coefficients should result from dividing the SN_{eff}-contribution for each material by the thickness of that material. The veracity of these resulting layer coefficients should then be supported by a comparison with the layer coefficients that would be expected for the final pavement structure under *in situ* conditions.

DEVELOPMENT OF EXPERIMENTAL MODEL

The Committee decided to evaluate the SN_{eff} method described in the Guide on several years of seasonal FWD data initially collected to support the Strategic Highway Research Program (SHRP). Ultimately, over five years of data, collected at eight different locations throughout the state and representing close to 30,000 deflection basins, provided a comprehensive assessment of the variation in SN for Vermont pavement structures due to annual seasonal variability. It was observed after spring thaw, a somewhat elusive phenomenon to capture, the SN_{eff} remained fairly stable between days 100 and 300 and exhibited a coefficient of variation under 10%. This stable time period corresponds very well with the typical April 15 to November 1 construction timeframe established for Agency construction projects. A summary of these findings for five of the eight sites is illustrated in Figure 1, with SN_{eff} values plotted against the Julian day of the year (1-365).

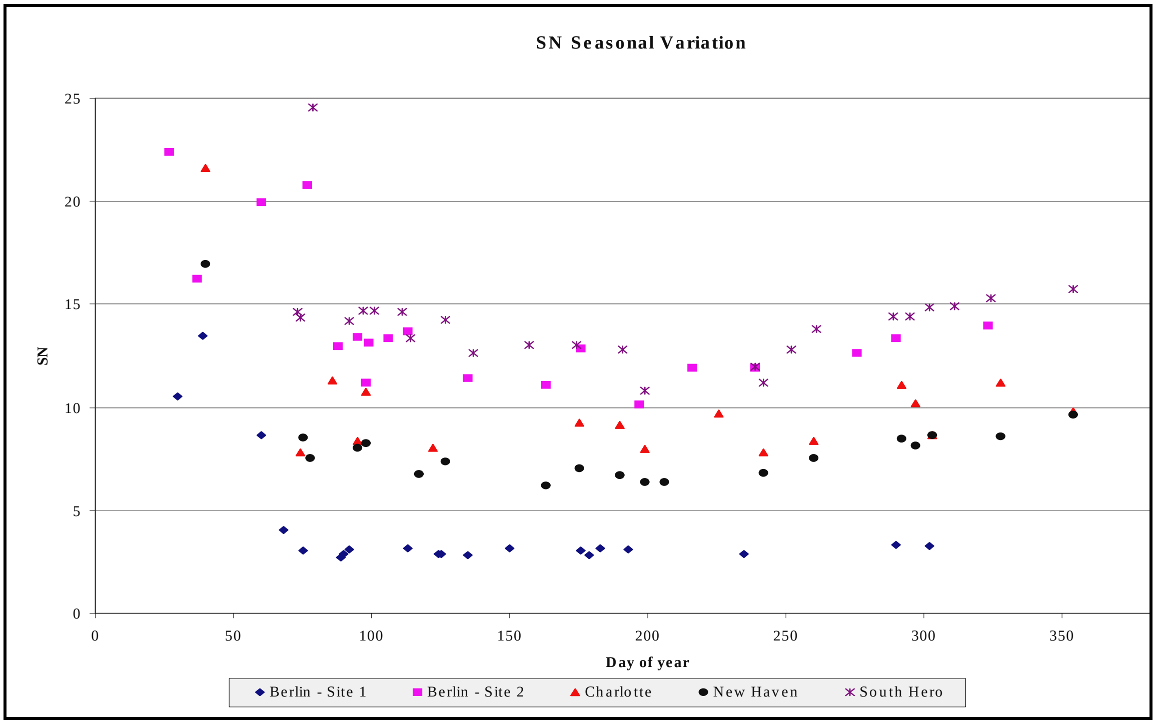

Figure 1 – Seasonal Variation in SN

The foregoing findings led the Committee to form several assumptions:

- *if FWD testing were restricted to the May through October timeframe, fairly stable, essentially unchanging, effective SNs may be expected at a given location,*
- *barring any extreme fluctuations in temperature or moisture conditions, the SN contribution of any component material, hence the layer coefficient, should also remain fairly stable during the May through October timeframe*, and
- *the SN contribution of any pavement structure component is independent of the stress states produced from the range of loads (6,000 to 16,000 pounds) applied to the surface.*

The first two assumptions seemed rather obvious from observation of the data presented in Figure 1. The third assumption was a result of evaluating the daily results and recognizing that all seasonal locations were tested using the SHRP FWD protocol, which targets four different loads: 6,000, 9,000, 12,000, and 16,000 pounds. Upon a detailed observation, the effective SNs derived from the SHRP protocol loading range were surprisingly consistent for a given testing day and the coefficient of variation on the range of effective SNs characteristic for any given day was typically about 1%. Put another way, 95% of the effective SNs for a particular location and developed during a given day of testing were within less than 1% of the average SN for that day.

The consistency in the SN was unexpected and truly remarkable. Considering the impulse load more than doubles during the FWD test, the stress-dependency of the modulus for the unbound materials, and the visco-elasticity of the asphalt stabilized materials, it seemed highly unlikely that the interplay among the various material stiffnesses would exactly compensate to provide a constant SN to such a precise degree. It seemed more plausible that each constituent SN associated with the surface, base, and subbase, should remain relatively constant on its own.

If the foregoing is true, this last assumption supports the notion that the SN_{eff} established for a particular material may remain reasonably stable from its placement to its service in the final structure <u>if</u>:

1. All construction and FWD testing activities take place during May through October,
2. No extreme temperature or moisture fluctuations occur prior to FWD testing, and
3. FWD target loads for the base and subbase materials are within the magnitude of stresses likely for the final structure under normal loadings and do not induce shear failure in the unbound materials.

While strongly implied from the analysis of the seasonal data, the Committee nonetheless attempted to analytically corroborate the second assumption of a stable SN contribution from any component material. Unfortunately, this analysis of the SN_{eff} method described in the Guide proved beyond a simple algebraic manipulation of the SN_{eff} model. A more practicable solution considered was to perform a simulation of the expected behavior of typical Vermont subbase materials using an elastic layer simulation (ELS).

Two conditions were simulated with the ELS to evaluate the behavior of a pavement structure subjected to an FWD test. Of particular interest in this simulation is the behavior of the granular subbase material. Two different stages of the pavement construction were examined. The first condition simulated the FWD test on the stress-dependent granular subbase resting on a stress-dependent fine-grained subgrade. The second condition simulated the FWD test of a constant-modulus surface material on stress-dependent granular base, subbase, and fine-grained subgrade materials. The material properties and performance of the <u>subbase</u> were compared as illustrated in Figure 2.

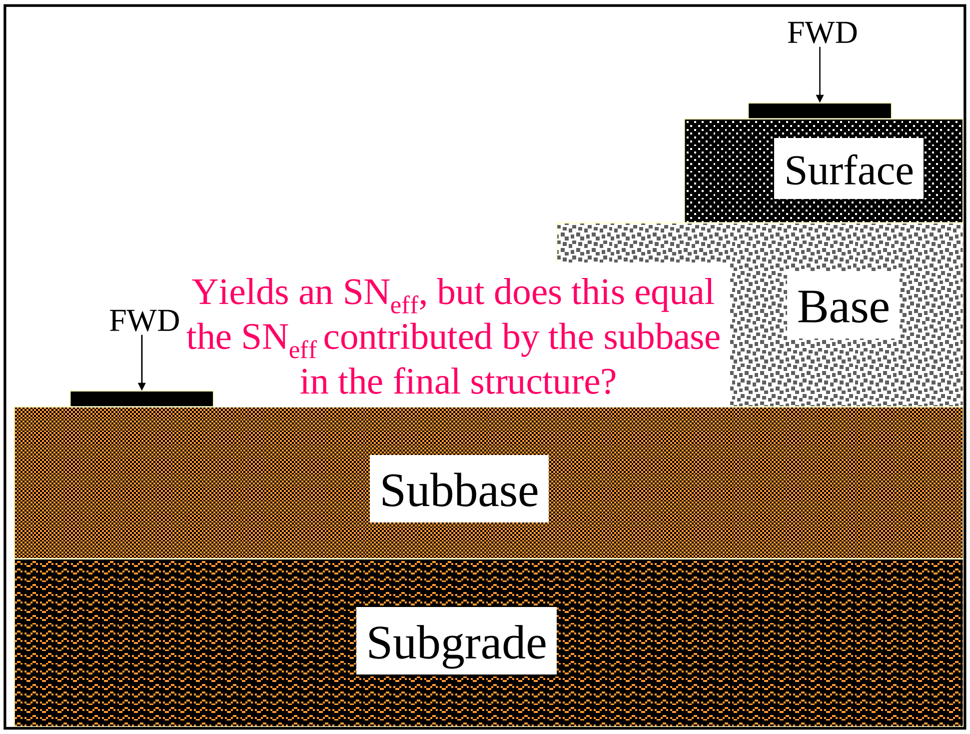

Figure 2 – Granular Subbase Behavior

Resilient moduli for these stress-dependent materials in both simulations were determined using a simple K-theta model as illustrated in Equation 2,

$$M_R = k_1 \times \Theta^{k_2} \tag{2}$$

where: M_R is the resilient modulus,

k_1 and k_2 are material-specific regression constants, and

Θ is the stress state of the material.

Under the initial conditions, FWD tests were simulated on the surface of each component material. This was a straightforward analysis from which deflections, loading plate pressures, and subgrade properties were readily available. However, when simulating the final condition, the loading plate pressures for the base and subbase were a function of an "effective" plate radius, which is subject to conjecture. While most soils engineers may agree on a Boussinesque stress-distribution for a point load, a typical pavement structure does not behave the same as an equivalent, relatively homogeneous, soil mass. A different approach is necessary to model the stress-distribution occurring beneath a circular load on a relatively stiff upper layer into a less stiff (by an order of magnitude) unbound aggregate. Noureldin and Al Dhalaan (6) proposed a stress-distribution under a circular load of radius "a" that gradually transitions from the circular area under the loading plate to a circular area with a radius corresponding to the depth from the surface within a depth of twice the loading plate radius of the surface. Noureldin and Al Dhalaan's proposal provided a definitive "effective plate radius" for determining effective structural numbers for base and subbase materials in the final structure simulation.

Subbase layer coefficients determined from the simulation results of the initial condition described above were generally within 5% of the layer coefficients determined for the subbase performing in the final condition and are illustrated in Figure 3. The Committee interpreted the results of this pavement simulation to validate the assumption that the SN for any component of a pavement structure may remain stable enough for the design of flexible pavement structures. Without finding any research to contradict the findings of the simulation, the Committee decided to sponsor a pilot study to determine real world layer coefficients from FWD testing.

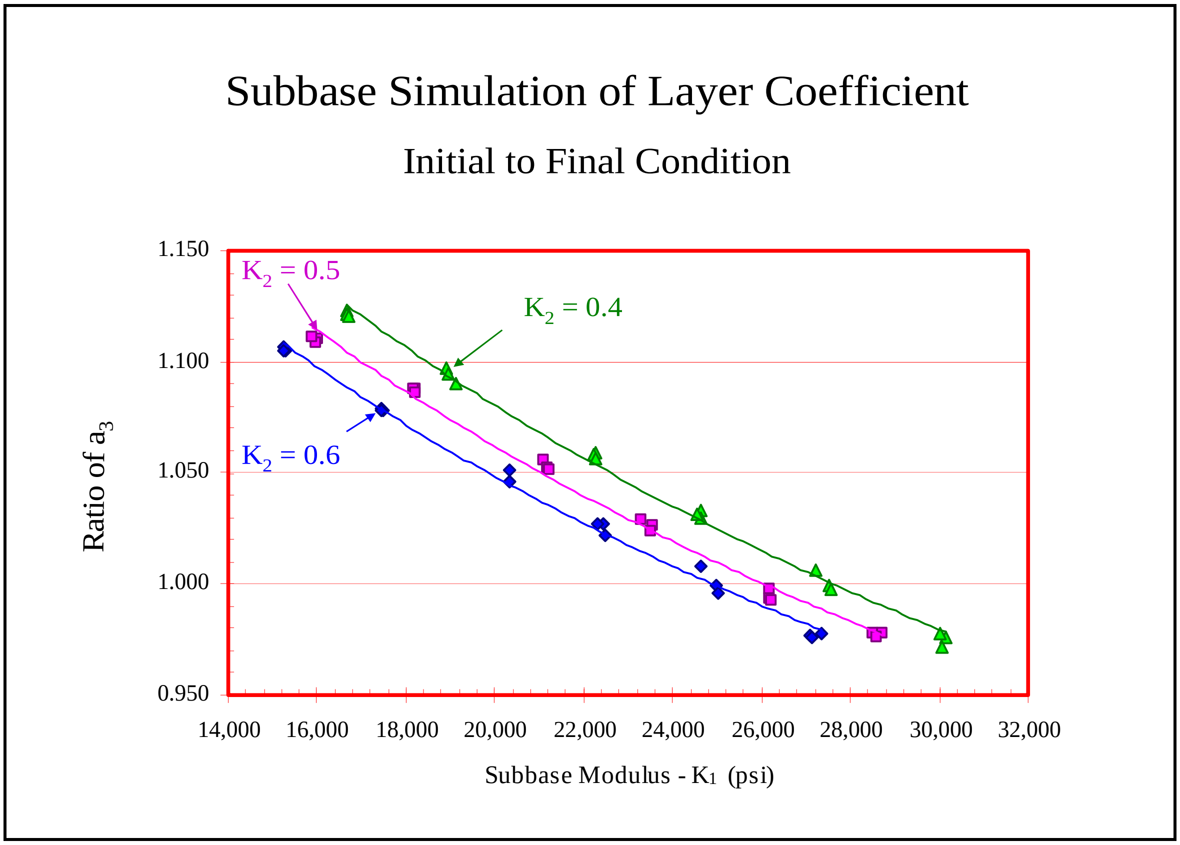

Figure 3 – Subbase Simulation of Layer Coefficient

In summary, the layer coefficient determination model consisted of the following steps:

1) Assume the SN for any material is a fixed property and remains constant throughout the construction operation, after it has reached its design condition,

2) Collect FWD deflection data on the top surface of each pavement material, during the construction season of April 15 through November 1,

3) Use backcalculation software to determine the subgrade M_R at the centerline of the load for each

FWD test,

4) Correct any deflections taken directly on the pavement, or asphalt cement concrete (ACC), to 68° F,

5) Determine the SN_{eff} appropriate for each successive build-up of pavement material, and

6) Determine each layer coefficient for each material by taking the difference in the SN_{eff} determined directly on top and directly below the material layer, and dividing by the material thickness. Note: The SN_{eff} on top of the subgrade is defined as zero.

PILOT PROJECT TO TEST EXPERIMENTAL MODEL

The next step was to identify a pilot project and collect real data representative of materials used for the construction of pavement structures in Vermont.

Since analysis of the seasonal data would seem to indicate the drop weight used has little effect on the SN_{eff} finally determined for any given pavement structure, this study focused on the deflection basins generated by a single target weight for each material. The target drop weights applied on the surface of each material were consistent with the effects that would be expected from a 100-psi tire pressure applied at the surface of the finished structure. However, limitations with the Agency's FWD equipment made achieving plate pressures below 10 psi difficult. This only presented a concern with the sand subbase, which should have been tested using a pressure in the range of two to three psi. But, testing the sand subbase at 10 psi yielded no evidence of shear failure due to overstressing and backcalculation results exhibited root-mean-square (RMS) variations from the FWD-measured deflection basins of less than 25%. The Committee considered this compromise to be satisfactory for a sand subbase.

The layer coefficients for the pilot project were 0.074, 0.163, and 0.639 for the sand borrow subbase, dense-graded crushed stone (DGCS), and ACC, respectively. These findings were encouraging since the layer coefficients established for the unbound materials were within the ranges established by AASHTO for these materials.

The layer coefficient for the ACC was not discounted outright. Although 0.639 is almost 50% higher than the 0.44 upper limit established by AASHTO for ACC surface course, two other indicators of layer coefficients for ACC, a Marshall stability of 2,730 lbf. and a resilient modulus of 580,000 psi, were also beyond the upper AASHTO limits of 2,100 lbf. and 450,000 psi respectively.

The findings from the data analysis of the pilot project were encouraging. Consequently, the Committee considered the experimental model developed thus far to be a success. The Committee endorsed further collection of FWD data, using the experimental model developed with the pilot project, at several more projects to determine if the method developed was capable of providing satisfactory estimates of material properties and that these properties are representative of in-service performance. In all, nearly 50 test sites were evaluated for this next phase of the research.

DATA ANALYSIS

FWD Results

Backcalculations were performed on all deflection basins to determine the resilient modulus of the subgrade, a necessary input for the SN_{eff} calculations. Two independent applications were used: ELMOD 4.0 and EVERCALC 5.0. These two applications perform similar functions, using different algorithms. Both attempt to achieve convergence between the FWD measured deflection basin and a calculated deflection basin based on the backcalculated layer moduli.

A "manual" backcalculation method, a spreadsheet employing the method of equivalent thickness developed by Odemark and described by Ullidtz (7), was used to spot check a random sample of ELMOD and EVERCALC output, to ensure reliability of the backcalculation results.

In order to control the quality of the backcalculation findings, goodness-of-fit thresholds were established for deflection basins taken on the sand, DGCS, and ACC surfaces of 25, 10, and 2% RMS, respectively. That is, if a backcalculation for a sand deflection basin could not produce a solution with an RMS less than 25%, that site was removed from further consideration in this study. Similarly, if either the DGCS or ACC backcalculation failed to meet the appropriate RMS threshold, the entire site was considered compromised and removed from the study. Figure 4 illustrates how the SN_{eff} progresses as FWD testing is conducted on each successive pavement material.

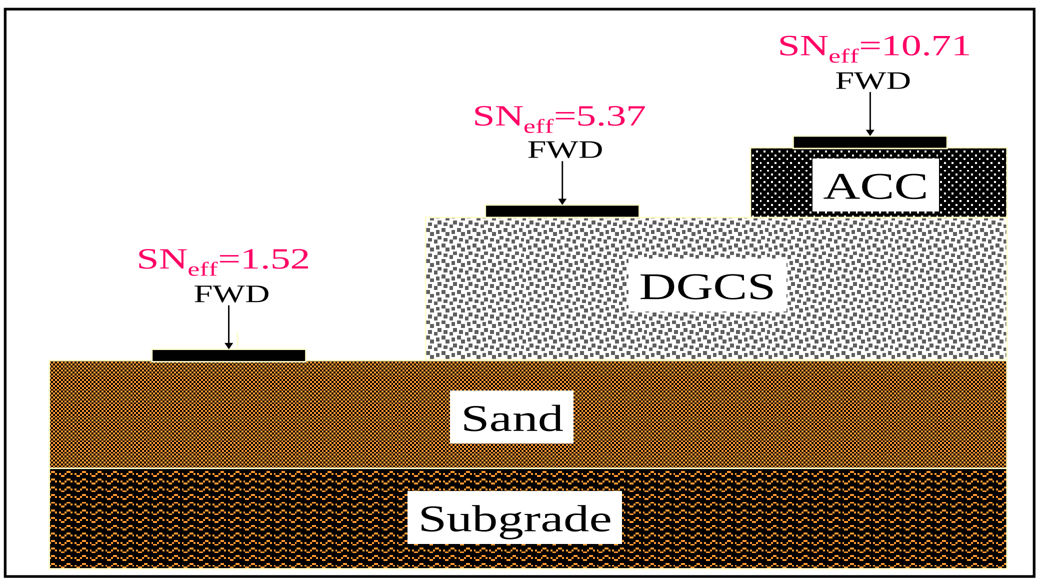

Figure 4 – FWD Testing Progression

Layer Coefficients

The estimation of layer coefficients (a_i) uses the SN_{eff} contributed by each pavement material. Figure 5 illustrates as the SN_{eff} is established for each material interface, the change in SN_{eff} for any two adjacent material interfaces represents the SN contribution for the material bounded by these adjacent interfaces. The resulting layer coefficient is the SN contribution for any particular material divided by the thickness of that material layer. But, if the thickness has not been accurately assessed, this will have a corresponding adverse effect on the layer coefficient.

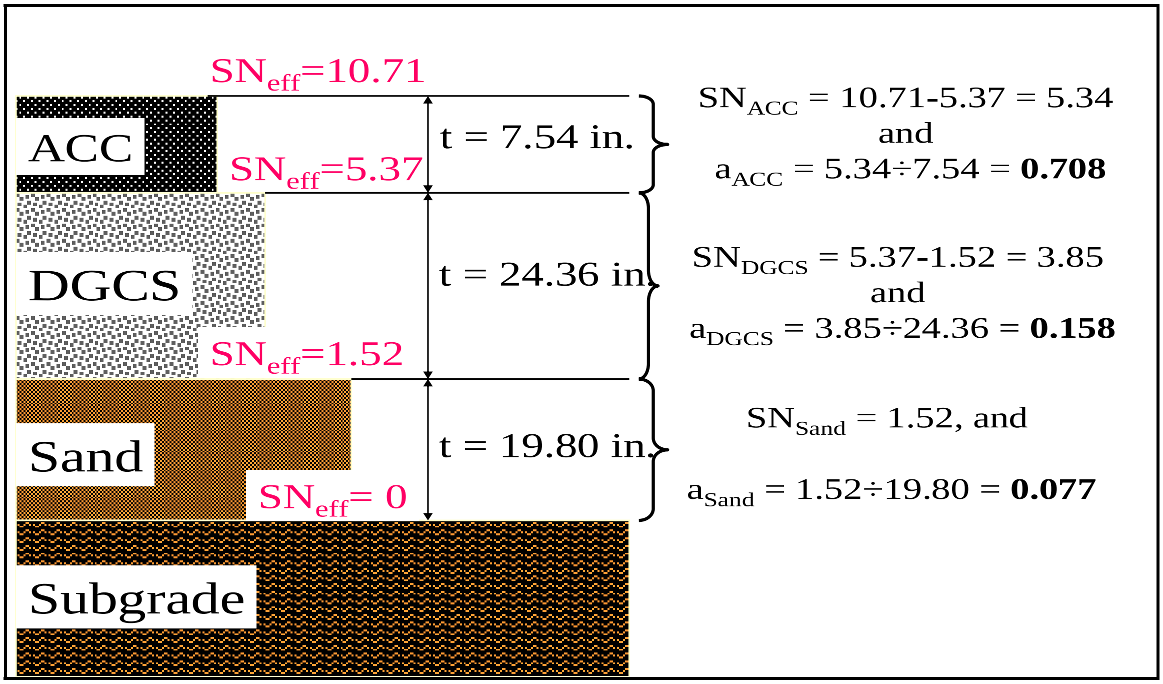

Figure 5 – Determination of Layer Coefficients from FWD Testing

The development of layer coefficients using the procedure just outlined is relatively easy and applicable to the materials in question. The issue of whether layer coefficients developed in this manner are characteristic of material performance of the final (in-place) structure and appropriate for design must be supported.

Final Structure Simulation

When evaluating the suitability of layer coefficients for use as design parameters, the only pertinent standard should be their prediction of layer coefficient behavior in the final structure. Ideally, a fully instrumented pavement structure, with a full array of stress and strain sensors to monitor the behavior of each material interface, would provide the necessary data to make this comparison. However, based on past experience, the Committee considered subsurface instrumentation too unreliable.

Instead, an ELS was conducted to simulate the response of the final structure. The simulation was carried out on a model of the final structure using the actual layer thickness and backcalculated resilient modulus for each material.

The results from the ELS were used, along with an "effective plate radius" calculated below the surface as proposed by Noureldin and Al Dhalaan, to simulate the behavior of the final structure under an

FWD load and to estimate the SN_{eff} at each material interface. Once the SN_{eff} was determined at the surface of each material, the layer coefficients were calculated as illustrated previously in Figure 5.

Comparative Analysis of Layer coefficients

In all, six experimental projects amounting to almost 50 test sites were evaluated to establish the significance between calculating layer coefficients from FWD testing and how well they represent the *in situ* conditions estimated by simulation of the final structure. Although cursory observation revealed satisfactory agreement between centerline deflections measured with the FWD and deflections predicted by the ELS, a more detailed statistical analysis of the layer coefficients developed from FWD test data and the ELS was done to provide a more objective means of establishing that no significant difference existed between the results of the two methods. If no statistically significant difference is found, then any distinction observed may be attributable to normal variation in the material properties – i.e., the materials do not exhibit linear elastic, isotropic, and homogeneous properties – and normal error in data acquisition. Also, if both methods yield similar results, it would further substantiate the assumption that the SN contributed by a pavement material is a fixed property and, more importantly, layer coefficients determined via FWD tests are suitable for design of the final structure.

The statistical analysis was carried out using a paired *t* Test, which assumes the difference between pairs of data to average zero. Ordinarily, a low p-value (a statistical metric that quantifies the rarity of an occurrence) resulting from a paired *t* Test indicates little relationship between the two data sets being compared. For this research, a high p-value (>0.05) suggests a statistically significant correlation exists between the paired data sets. Thus, the p-values determined by this analysis indicate the layer coefficients determined via FWD tests are suitable for design of the final structure as indicated in Table 1.

Table 1 lists both a "Project specific" p-value (that is, for those results determined at each project location) and a "Research cumulative" p-value (representing the results of an analysis carried out as the results of each successive project are added to the cumulative database). These results indicate a significant level of agreement, or correlation between, the two data sets suggesting no statistically significant difference exists between the two methods: FWD- and ELS-derived layer coefficient determinations. Thus, it may be concluded, with a high degree of certainty, that FWD-derived layer coefficients are sufficiently accurate to predict *in situ* behavior to be useful pavement design parameters.

Table 1 – p-values from Paired *t*-Testing of FWD Computed and Simulated Layer Coefficients		
	p-value (at 95% level of confidence)	
	Project specific	Research cumulative
Vergennes-Ferrisburgh	0.29	0.29
Montpelier State Highway	0.48	0.34
Bolton-South Burlington	0.10	0.28
Burlington	0.09	0.13
Colchester	0.09	0.33
Addison	0.09	0.32

DISCUSSION OF RESULTS

As the results of the statistical analysis supported the Committee's assumption that layer coefficients determined from FWD testing are sufficiently representative of *in situ* conditions (exhibited via simulation of the final structure) an evaluation of the results determined up to this point was warranted.

A summary of the findings for the first six projects studied in this investigation are presented in Table 2, listing the layer coefficients and resilient moduli so far determined and the number of test locations for which all quality control criteria were met.

Table 2 – Summary of Material Properties for First Six Projects								
	Sand	DGCS	ACC I	ACC II	ACC III	ACC IS	ACC IIS	ACC IIIS
a_i	0.073	0.152	0.386	0.687	0.855	0.839	0.588	0.495
M_r (psi)	18,900	41,900	397,000	343,000	360,000	321,000	153,000	346,000
N	47	47	30	30	30	15	17	17

Of particular interest are the layer coefficients determined for the unbound materials. The sand subbase value of 0.073, while on the low side of AASHTO's unbound subbase scale, may be due in part to the fact that this material is much deeper than the unbound subbase materials were at the Road Test. Since the sand is placed so deeply in Vermont pavement structures, where it would experience lower stress states than Road Test unbound subbases, it may be performing like a fine-grained material and may explain why the resilient modulus of 18,900 psi falls on the high side of the AASHTO scale, in relation to the layer coefficient. The value of 0.152 for the DGCS falls on the higher end of the range established by AASHTO for an unbound base material. This higher layer coefficient is consistent with the higher resilient modulus of 41,900 psi determined for DGCS and also conforms to the behavior one would expect of a stress-stiffening coarse-graded granular material. By comparison, laboratory testing of these materials has established estimates of the resilient modulus to be 25-35% of the backcalculated resilient modulus for sand (8) and 30-

45% of the backcalculated resilient modulus for DGCS (9).

Probably the most conspicuous eccentricity with the results established thus far in this effort is the unusually high layer coefficients established for the ACC materials. Although there appears to be nothing fundamentally wrong with the layer coefficients determined for the ACC materials – i.e., the same procedure was used to derive reasonable unbound layer coefficients and the elastic layer simulation would seem to indicate an accurate prediction of in-place behavior – their use with the AASHTO design model presented some concerns. Most obviously, any layer coefficients over 0.50 represent a range of conditions as of yet unsubstantiated for the empirically derived AASTHO model. Also, the ACC layer coefficients developed under this investigation were established for materials that were designed using much lower layer coefficients (0.32-0.39) with the AASHTO model. And finally, if the layer coefficients presented here (>0.50) are used for an AASHTO design under typical Vermont traffic loading, almost no base material (DGCS) is called for because all of the strength (SN) is provided by a few inches of ACC. The Committee considered several mechanisms likely to generate layer coefficients outside the traditional range established by AASHTO.

First, environmental conditions in Vermont necessitate thick pavement structures to mitigate the effects of frost penetration. These substantial structures are likely far beyond anything studied at the Road Test.

Second, the two primary components of the Agency's ACC materials are likely to be different from, if not an improvement upon, those materials from which the AASHTO relationships have been derived. Vermont is fortunate to have readily available, high-quality, and affordable aggregates. The Agency has also traditionally used stiff asphalt cements and high compactive efforts in an attempt to minimize distresses associated with Vermont's extreme temperature fluctuations.

Third, the ELSs were conducted using the elastic moduli determined from backcalculations of the FWD deflection basins taken on the surface of the finished pavement structure. Even though many of the ACC moduli were consistently in excess of the 450,000-psi upper limit published by AASHTO, the layer coefficients determined via ELS still corroborated the layer coefficients determined from the FWD deflection data.

And finally, the FWD measures *in situ* behavior. It is not unreasonable to contend that laboratory-supported AASHTO modulus/coefficient relationships may not accurately predict *in situ* behavior for any material, whether unbound or asphalt stabilized. Indeed, Figure 6 illustrates how the ACC layer coefficients exhibit an inflationary effect as the ACC is supported by an increasingly substantial "subgrade." Interestingly enough, when analyzed using the top of the unbound portion of the structure as the subgrade, the ACC layer coefficients thus determined cluster within the more traditional range of 0.20-0.44 established for ACC materials used in the AASHTO model. This interplay between ACC layer coefficients and its support structure may be analogous to the synergism of a concrete bridge deck supported by steel girders. Neither is adequate to the task in isolation, but when acting in unison, they achieve an effect of which each is individually incapable.

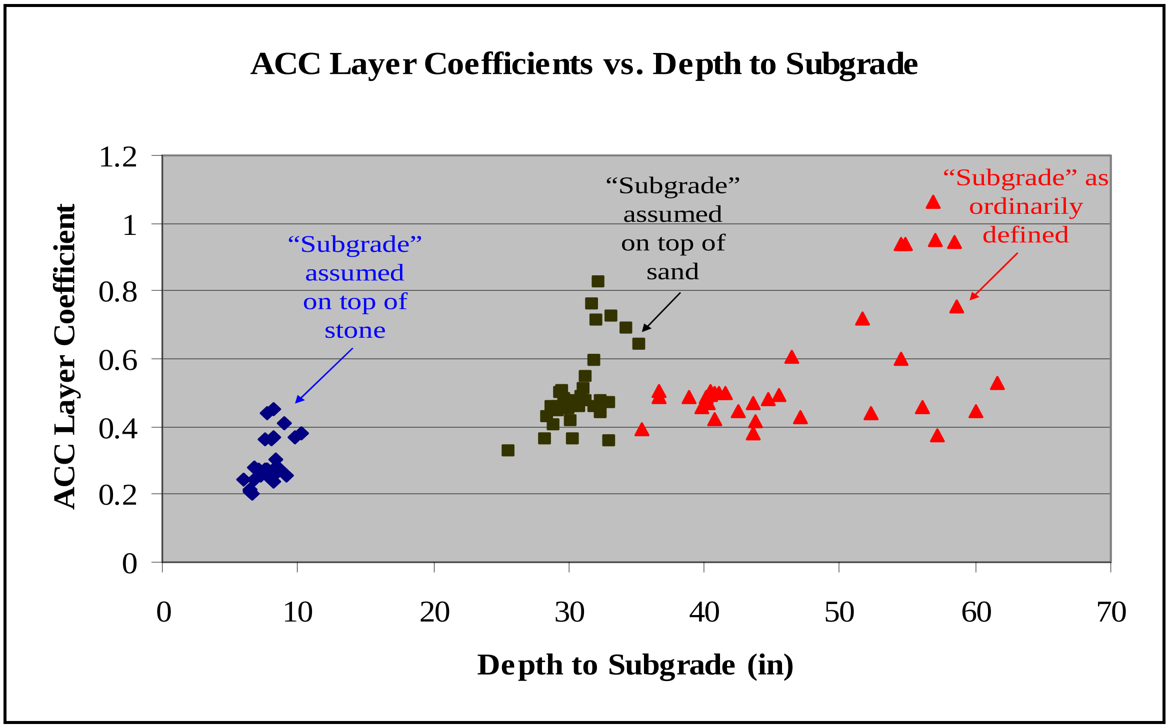

Figure 6 – ACC Layer Coefficient vs. Depth to Subgrade

Valid resilient moduli for the various types of ACC (I, II, III, etc.) materials used by Agency designers may have to be determined indirectly, since backcalculation limitations cannot distinguish such subtleties within the FWD loading plate radius of the testing surface. Marshall stabilities were considered useful for estimating the resilient moduli of the ACC materials, assuming there exists a correlation between Marshall stabilities and resilient moduli (a notion implied by AASHTO). The Marshall stabilities may give an indication of the relative proportions of the individual resilient moduli compared to the resilient modulus backcalculated for the total ACC thickness. Another possibility may be an indirect tension test (like ASTM D4123), which establishes the resilient modulus for ACC samples. For this investigation, Marshall stabilities were used, when available, as a proxy to isolate the resilient moduli for different ACC types.

At this time, it is uncertain why there exist such marked disparities between the layer coefficients determined for Marshall and Superpave materials. The Committee debated this issue extensively and finally conceded that Marshall and Superpave mixes are two different materials and layer coefficients may simply be one more manifestation of these differences. The Committee endorsed further study to bolster or refute some of these concerns with the ACC properties.

Ten additional projects were identified for further study to allow for additional data collection and to improve the predictive capabilities of the subsequent estimates. Another benefit to further study was the potential for investigation into additional materials. Two of the additional projects used gravel for subbase instead of the DGCS usually required on the State system. One Interstate project provided for an investigation into the performance of the original DGCS ("old stone"), which had been in service for nearly 40 years. Also novel to the Interstate project was an experimental material to provide for better drainage: an asphalt-treated permeable base (ATPB).

Table 3 summarizes the properties established from all 16 projects investigated.

Table 3 – Summary of Material Properties

Material	Layer Coefficient				Resilient Modulus (psi)			
	N	Standard deviation	Average	95% Pre.	N	Standard deviation	Average.	95% Pre.
Sand	139	0.013	0.078	2.9%	139	10,200	19,100	9.0%
Gravel	21	0.033	0.134	11.1%	21	12,500	29,600	19.2%
Old stone	21	0.021	0.102	9.2%	19	12,100	26,200	22.2%
DGCS	164	0.032	0.137	3.6%	164	16,800	29,700	8.7%
ATPB	21	0.067	0.398	7.7%	21	64,700	110,500	26.6%
ACC I	75	0.190	0.483	9.1%	76	169,800	357,600	10.8%
ACC II	62	0.284	0.630	11.5%	62	188,600	347,500	13.8%
ACC III	76	0.517	0.844	14.0%	76	200,500	304,500	15.0%
ACC IS	83	0.256	0.536	10.4%	21	85,300	191,200	20.2%
ACC IIS	102	0.184	0.504	7.2%	40	44,100	140,600	10.0%
ACC IIIS	93	0.170	0.533	6.6%	65	213,100	322,500	16.4%
ACC IVS	35	0.223	0.570	13.4%	35	49,700	92,400	18.5%

In addition to the number of data points (N), the standard deviation, and the average, Table 3 includes the level of precision on the average at the 95% level of confidence. Put another way, the level of precision ensures that if one were to use the average value for design, it would be reasonable to assume that the value provided under conditions of actual performance would be within the precision indicated 95% of the time.

CONCLUSIONS

The AASHTO guide describes a procedure for determining the effective SN provided by a pavement structure from FWD deflection data. While Ioannides presented compelling justification for questioning the theoretical purity of this concept, the success of its practical application as investigated by this research is difficult to ignore.

When FWD testing is conducted during the April 15 through November 1 construction season, and no drastic temperature and moisture fluctuations occur, the SN_{eff} and resulting layer coefficient associated with a particular component of a pavement structure appear to remain <u>reasonably</u> stable, even after additional material is placed.

The stress distribution described by Noureldin and Al Dhalaan appears to provide a reasonably accurate portrayal of the effective plate radius that develops below the surface of a pavement structure for an applied circular load, without which the simulated layer coefficients would have been difficult to determine.

It is paramount to accurately and precisely determine the thickness of each material being evaluated. Depending upon the material, any error in the thickness assessment can have a corresponding error in the layer coefficient determination, e.g., a 25% thickness error may lead to a 25% error in the layer coefficient determination. While this magnitude of error is not desirable in any of the materials, it can certainly have alarming consequences with the stiffer and thinner ACC materials.

The layer coefficients determined for the unbound materials appear reasonable, while the ACC layer coefficients are outside the range typical for the AASHTO procedure. However, there does appear to be substantiation for these higher ACC layer coefficients from other material properties, namely the Marshall stabilities and backcalculated resilient moduli. Further, all the layer coefficients determined by the method developed under this investigation are reasonably accurate estimates of the *in situ* behavior simulated by elastic layer theory. Indeed, such high correlation between these two different procedures would be highly unlikely, considering the variables that lead to their development.

Whether by serendipity or by design, the development of the AASHTO effective SN procedure provides designers with a very powerful tool for the determination of layer coefficients.

RECOMMENDATIONS

Considering the emphasis that will be placed upon mechanistic design in the next version of the AASHTO Guide, it is likely that the effort described in this report represents both the solution to the Agency’s quest to calibrate the AASHTO pavement design model to Vermont materials and the conclusion to that effort as well. Table 4 summarizes the Committee’s recommendations for the layer coefficients to be used in conjunction with the current AASHTO pavement design model. The Committee considered the 85th-percentile for ACC layer coefficients to ensure reasonableness of designs provided by the model.

Any follow up research should focus on supplementing the database for the mechanistic properties thus far established. Work should continue on the resilient modulus for all unbound materials and the “dynamic modulus” for ACC materials, which is a new property identified in the upcoming AASHTO pavement design guide for ACC materials.

Table 4 – Recommended Material Properties for Design Using the AASHTO Model								
Material	Layer Coefficient				Resilient Modulus (psi)			
	N	Standard deviation	Average	**Rec**.	N	Standard deviation	Average	**Rec**.
Sand	139	0.013	0.078	**0.078**	139	10,200	19,100	**19,100**
Gravel	21	0.033	0.134	**0.134**	21	12,500	29,600	**29,600**
Old stone	21	0.021	0.102	**0.102**	19	12,100	26,200	**26,200**
DGCS	164	0.032	0.137	**0.137**	164	16,800	29,700	**29,700**
ATPB	21	0.067	0.398	**0.331**	21	64,700	110,500	**110,500**
ACC I	75	0.190	0.483	**0.293***	76	169,800	357,600	**357,600**
ACC II	62	0.284	0.630	**0.346**	62	188,600	347,500	**347,500**
ACC III	76	0.517	0.844	**0.327**	76	200,500	304,500	**304,500**
ACC IS	83	0.256	0.536	**0.280***	21	85,300	191,200	**191,200**
ACC IIS	102	0.184	0.504	**0.320**	40	44,100	140,600	**140,600**
ACC IIIS	93	0.170	0.533	**0.363**	65	213,100	322,500	**322,500**
ACC IVS	35	0.223	0.570	**0.347**	35	49,700	92,400	**

***** If an ATPB is used, the layer coefficient for the base course (either ACC I or ACC IS) should be increased to at least the 0.331 used for the ATPB.

****** At this time, there is no recommendation for the ACC IVS resilient modulus.

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