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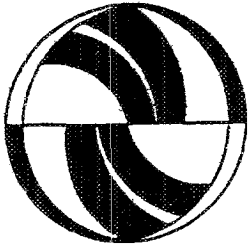
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**An Economic Model of Pavement Rutting in  
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# **A Economic Model of Pavement Rutting in Asphalt Concrete Mixes**

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## **ABSTRACT**

Pavement deterioration models are an important input for the efficient management of pavement systems, the allocation of cost responsibilities to various vehicle classes for their use of the highway system, and the design of pavement structures. This paper is concerned with the development of an empirical rutting progression model using an experimental data set from WesTrack. The data used in this paper consist of an unbalanced panel data set with 860 observations. The salient features of the model specification are: 1) three properties of the mix are sufficient to model the performance of the asphalt concrete pavement accurately, 2) the model captures the effects of high air temperatures at WesTrack, and 3) the model predicts rut depths by adding predicted values of the increment of rut depth for each time period, which is particularly advantageous in a pavement management context. The three mix properties are a gradation index, which is obtained from the aggregate gradation, the voids filled with asphalt obtained for the construction mix in the Superpave gyratory compactor, and the initial in-place air voids. The specified model is non-linear in the variables and the parameters, and is estimated using a random effects specification to account for unobserved heterogeneity. The estimation results and prediction tests show that the model replicates the observed pavement behavior at WesTrack well.

## 1. Introduction

The accurate prediction of rutting development on asphalt concrete pavement is an essential input for the efficient management of pavement systems. In addition, pavement rutting progression models can be used to study the effects of different loading levels, and thus in allocating cost responsibilities to various vehicle classes for their use of the highway system. Further, such models can be used for evaluating different strategies for design, maintenance and rehabilitation (Archilla and Madanat 1999b). Finally, they can also provide directions in the proportioning of aggregate, asphalt and air in the asphalt concrete mix.

This paper is concerned with the development of an empirical rutting progression model using experimental data. The data set used in this paper is taken from the WesTrack road test, which consists of a specially built track in the state of Nevada approximately 100 km southeast of Reno. The track contains 26 test sections of hot-mix asphalt (HMA) construction.

Archilla and Madanat (1999a) have pointed out the attributes of experimental data, as opposed to field data collected from condition surveys of in-service pavements. In the context of the WesTrack road test, the most relevant attributes of experimental data are:

*Advantages:* The main factors affecting rutting, such as axle loads and asphalt mix properties, are carefully controlled, therefore the researcher can capture their effects on rutting progression. This is hardly possible using field data alone. Field data generally involve a distribution of loads whose measurement is not very accurate. Further, in field data, the control of the constructed asphalt concrete properties is of lower quality.

*Disadvantage:* The main disadvantage is that experimental data may not represent the true deterioration mechanism of in-service pavements. For example, traffic wander may not be adequately replicated in accelerated pavement loading tests.

The salient features of the model specification presented in this paper are:

- 1) three properties of the mix are sufficient to model the performance of the asphalt concrete mix accurately,
- 2) the model captures the effects of environmental factors at the WesTrack test site, and
- 3) the model predicts rut depths by adding predicted values of the increment of rut depth for each time period; this is particularly advantageous in a pavement management context where the engineer is interested in predicting changes in rut depth.

To estimate the model parameters, an unbalanced panel data set with 860 observations from WesTrack is used. An unbalanced panel data set consists of observations for different pavement units through time, where the numbers of observations for each pavement section are not necessarily the same. The model is nonlinear in the parameters and the variables.

The paper is organized as follows. Section 2 briefly describes the test at WesTrack, the source of the data set used for model development. Section 3 analyzes some variables that are usually identified in the literature as affecting the rutting performance of asphalt concrete pavements. Section 4 introduces a gradation index that is used in the model specification, which is presented in Section 5. Section 6 presents the results of the parameter estimation of that model and Section 7 concludes the paper.

## **2 The WesTrack Project**

WesTrack consists of a specially built track in the state of Nevada. The oval track consists of two tangent sections connected by two spiral curves. The 26 test sections of hot-mix asphalt (HMA) concrete were constructed on the straight tangents between the curves. The emphasis of this test was on studying the effects of deviations about target values in materials and construction factors (WesTrack 1996). Thus, asphalt contents, air void contents and aggregate gradations were systematically varied among the sections to represent typical construction variations (Epps et al 1998). The 26 sections have the same structural design, which consisted of 150 mm (6 in) of HMA, 300 mm (12 in.) of crushed aggregate base course and 450 mm (18 in.) of engineered fill.

The experimental design consists of three asphalt content levels, three aggregate gradation levels, and three air-void content levels. The original test sections included both fine and coarse graded Superpave mixtures developed from a locally available crushed gravel and a non-modified PG64-22 binder as graded in accordance with the Superpave binder classification system (AASHTO 1996).

The climate at the test site is typical of high desert climates in the intermountain western portion of the United States. A few summer air temperatures approach 40 C (104 F) and some winter air temperatures may fall to -18 C. Annual precipitation, in the form of both rain and snow, is approximately 100 mm (4 in.). Frost penetration into the fill and subgrade material is unlikely at this site (Epps et al 1998).

In the WesTrack project, routine cross-section profiles were obtained approximately every two weeks. These profiles were measured over time with three different devices: the "Dipstick", an Arizona Department of Transportation (ADOT) transverse profile device, and a project developed laser device (Hand 1998). From the cross section profiles, rut depth measurements were obtained in accordance with the LTPP protocol (FHWA 1998).

Traffic loading is provided primarily by an autonomous vehicle technology. Four tractor/triple-trailer combinations are utilized for pavement loading, as shown in Figure 1. The loads are also shown in that figure. Tire pressures of 655 to 689 kPa (95 to 100 psi) are used. The target vehicle speed for the test is 64.4 kph (40 mph).

The transverse distribution of the loading applied at WesTrack approaches that of normal traffic. However, as pointed out by Hand (1998), the distribution is achieved by manually moving guidance antennas transversally on the front bumpers of the trucks. Since the antennas are not continually moved, they are set at specific positions for several days. For some sections, clear reductions in rut depths (as high as 7 mm) between monitoring sessions were noted. These reductions can be connected to changes in the vehicle wander that flatten the peaks of the previously created ruts, thus reducing their depths. Such effects seem to be transient because the rut depths usually return quickly to the rutting levels before the changes in antenna location.



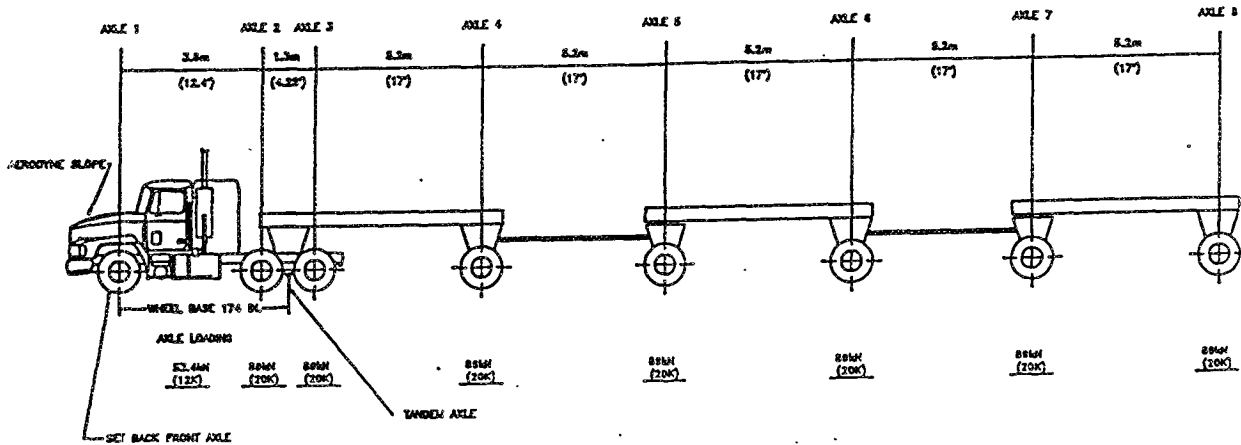
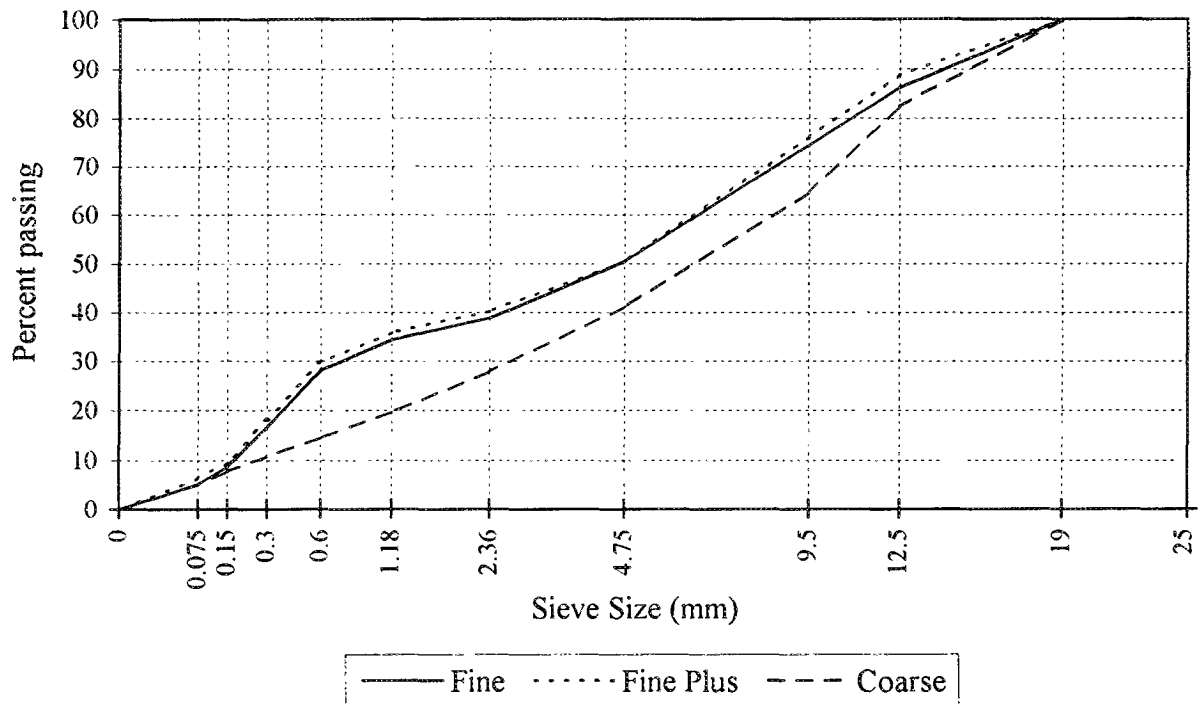


Figure 1: Truck configuration used in WesTrack.

### 3. Variables that Affect Mixture Performance

In this section, we describe the observed behavior of the different pavement sections (different asphalt concrete mixes) at WesTrack and we relate it to the literature on factors that affect the mix rutting performance. The results of the analysis in this section provide a basis for the model specification of Section 5.

Aggregate Gradation: The experimental design included three aggregate gradation levels. The mixes for these three gradation levels were termed fine, fine plus, and coarse. Figure 2 illustrates the target gradations on a 0.45 power chart (an explanation of this chart is given in Section 4). The fine and fine plus gradations are very similar and so was their behavior. Contrary to what the designers had anticipated the fine graded mixtures provided the best performance followed by the fine plus graded mixtures. The coarse graded mixtures had the worst performance. After approximately 267,000 vehicle passages, all 8 of the coarse mixes had failed, 3 of the 9 fine plus mixes had failed, and none of the fine mixes had failed. The fine mixes continued to perform satisfactorily until the end of the test.



**Figure 2:** Target gradation for fine, fine plus, and coarse WesTrack mixes.

Anderson and Bahia (1997) found similar results with laboratory experiments. They determined the final shear strain in *repeated shear at constant height* tests. The highest shear strains were found for the coarse mixes indicating that they are less resistant to rutting. They reported that these trends were inconsistent with their prior expectations.

Asphalt content: For good rutting performance the quality and the amount of asphalt in the mix is important. Since only one type of asphalt was used in WesTrack, we concentrate on the latter variable. Asphalt content is the percent by mass of asphalt binder in the total mixture that includes asphalt binder and aggregate (AASHTO 1997). The role of the asphalt binder is to adhere the aggregate skeleton and provide sufficient flexibility for durability (Anderson and Bahia, 1997). For a given stability range, it is usually accepted that the rate of rutting will be higher for high asphalt contents than for low ones. For example, Brosseau et al (1993) concluded from laboratory experiments that there exists a critical level of filling of the voids by the binder beyond which the material becomes unstable. These observations are consistent with

the pavement behavior at WesTrack. Figure 3 illustrates the rut depths for the 26 mixes vs. asphalt content after 130,300 vehicle passages. With only two exceptions, there is a clear increase in rut depths when the asphalt content increases above 5.5%. The two exceptions are fine mixes. This mix category does not seem to be affected by asphalt content within the range of asphalt contents used in the experiment.

Air voids: Given the aggregate gradation and asphalt content, the *initial* percent air voids is a function of construction compaction. Air voids is the total volume of the small pockets of air between the coated aggregate particles throughout a compacted paving mixture, expressed as percent of the bulk volume of the compacted paving mixture (AASHTO 1997). Results from laboratory experiments indicate that for air voids below 3% the stability (rutting resistance) of many mixes will decrease substantially. For example, Brown (1989) concluded that the major causes of rutting are excessive asphalt content and low air voids in the asphalt mixtures.

Figure 4 shows the rut depth after 130,300 vehicle passages vs. construction air voids. As can be observed in that figure, section 18, which has a high asphalt content (6.22 %) and very low air voids (2.4 %), was the mix that presented the lowest rutting at that point in time. Section 12 also had what would generally be considered as unacceptably low in place air voids (2.7 %), yet that mix performance was satisfactory.

A downward trend of rutting with in-place air voids for coarse and fine plus mixtures can be observed, whereas for fine mixtures the trend is slightly upward. The scatter observed for coarse and fine-plus mixtures indicates that in-place air voids by itself cannot explain asphalt concrete mix behavior. In addition, some researchers do not consider pavements with air voids above 8%, which is the case for many of the WesTrack sections, to be properly compacted (Discussion by Mr. R. Davis in Epps et al. 1998). Nevertheless, such high values do occur in practice.

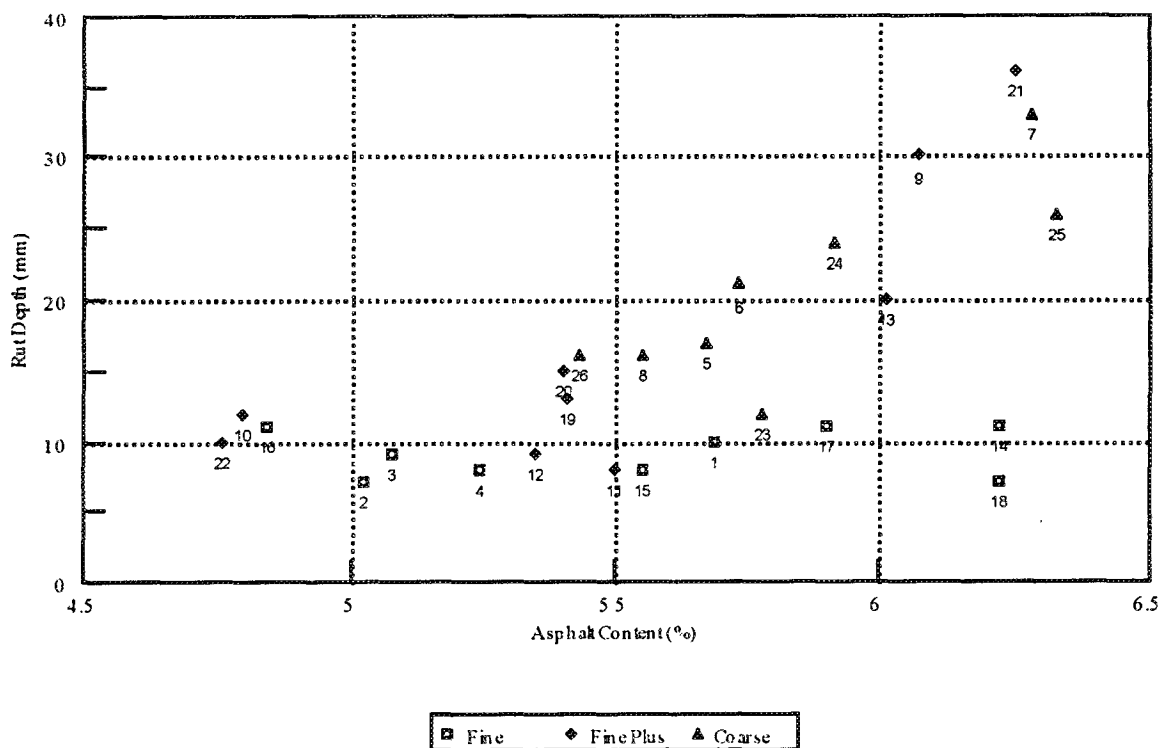


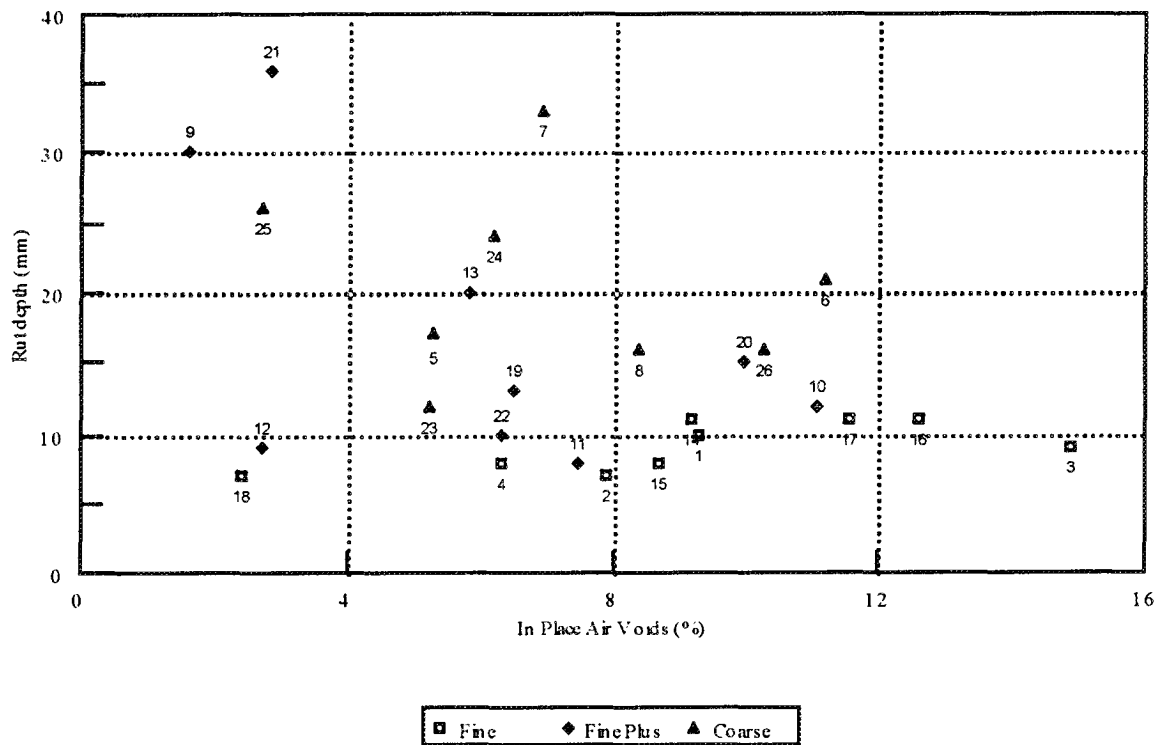
Figure 3: Observed rutting vs. asphalt content after 130,300 vehicle passages.

Voids Filled with Asphalt: Voids filled with asphalt (*VFA*) is the percentage of the voids in the mineral aggregate filled with asphalt binder (AASHTO 1997). This variable is interrelated the air voids content (*AV*), and the voids in the mineral aggregate (*VMA*)<sup>1</sup>. In fact, with knowledge of the *AV* and *VMA*, one can obtain the *VFA* as  $(VMA - AV) / VMA * 100$ . *VFA* is also interrelated to the effective asphalt content (*AC*) (which accounts for the volume of asphalt binder absorbed into the aggregate), because, for a given compaction energy, as the effective asphalt content increases the air voids decrease. Since *AV* and *VMA* are in part, a function of the compaction energy so is the *VFA*.

For WesTrack, neither the *VMA* nor the *VFA* are known for the as constructed mixes and therefore their effect can not be evaluated. However, Superpave Gyrotory Compactor (SGC) tests

<sup>1</sup> *VMA* is the volume of intergranular void space between the aggregate particles of a compacted paving mixture that includes the air voids and the effective asphalt content, expressed as a percentage of the total volume of the specimen (reference to AASHTO 1997).

(AASHTO 1997) were performed for quality assurance (QA). These tests were performed on materials sampled from transport trucks at the hot plants (Hand 1998). For these samples, all four variables were computed for the top and bottom lift of the HMA. Here, we use the data for the top lift because that is where most of the rutting occurred.



**Figure 4:** Observed rutting vs. in-place air voids after 130,300 vehicle passages.

Figure 5 shows the observed rut depths after 130,300 vehicle passages vs. *VFA* obtained in the SGC (after the samples were subjected to 96 gyrations, the design number of gyrations for the design traffic in WesTrack). Two very clear trends (shown with two hand drawn lines) can be observed from this figure. For the fine and fine plus mixes, rut depth remains approximately constant up to *VFA* values of about 85 %. After that, rut depth increases significantly with *VFA*. A similar trend is noted for the coarse mixes but the *VFA* value at which the rutting increases is somewhere between 60 and 70 %. Further, the rutting values for the fine and fine plus mixes at any *VFA* level are lower than the ones observed for the coarse mixes.

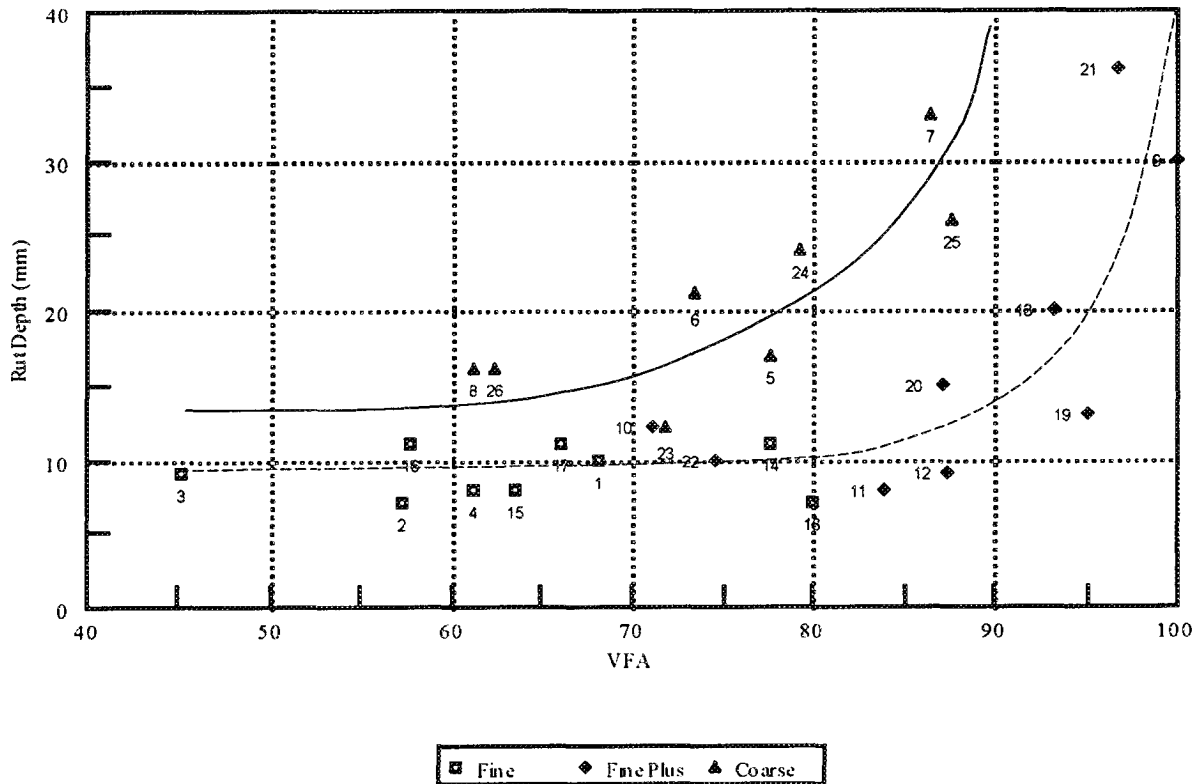


Figure 5: Observed rutting vs. in-place air voids after 130,300 vehicle passages.

Aschenbrener and MacKean (1994) found similar trends for Hveem stability (an indicator of the resistance to rutting of a mix) with *VFA* for fine, medium, and coarse gradations. They pointed out that *VFA* of less than 75 percent to 80 percent appeared to be necessary to avoid having a mix whose stability is sensitive to asphalt content or *VFA* (i.e, a stability that varies by a substantial amount with a small variation in *VFA*). The major difference from their study was that the curves for stability as a function of *VFA* for the fine and coarse mixes crossed for high *VFAs*. Thus, their fine mixes performed worse than their coarse mixes for high *VFAs*. As shown in Figure 5, this was not the case for WesTrack mixes. These differences in response of coarse and fine mixes may be attributed to differences in the definitions of what is meant by “coarse” and “fine” mixes, to other unobserved factors such as aggregate surface texture, or simply to different responses between a laboratory experiment and an actual pavement.

#### 4 Gradation Index

As we mentioned in section 3, the aggregate gradation had a substantial effect on the rutting performance observed in WesTrack. Engineers have long been aware that gradation of the aggregate is one of the factors that must be carefully considered in mix design, but there is still disagreement as to what gradations are the most satisfactory. Despite considerable research during several decades, there is no acceptable aggregate gradation index that indicates the rutting potential of asphalt mixes.

In this research, we found that the sum of square deviations from the maximum density line in the 0.45 power chart provided a good ordering of the gradations according to the observed rutting performance. Before introducing this index, however, an explanation of the 0.45 power chart, maximum density lines, and some associated issues is in order.

A 0.45 power plot of an HMA's aggregate gradation consists of the sieve sizes raised to the 0.45 power plotted on the  $x$  axis and the percent passing each sieve size plotted on an arithmetic  $y$  axis. This chart was developed by Goode and Lufsey (1962). All straight lines plotted in the chart from the lower left corner, zero percent passing a zero theoretical sieve size, upward and toward the right to any specific maximum size, represent "*maximum density gradations*". These lines are usually called "*maximum density lines*". It must be noted however that since the 0.45 power is an empirically derived value, the maximum density lines drawn as described above have only *approximate* maximum densities. The chart was created as a tool for determining proper adjustments in gradations to provide greater or lesser void in the mineral aggregate (*VMA*) in compacted mixtures.

Mixes prepared with aggregates of maximum density gradation have a minimum volume of space between the aggregate particles. These mixes usually have good stability (rutting resistance) because of aggregate interlock but their *VMA* can be insufficient to accommodate the proper quantity of asphalt to make a durable pavement (i.e., a pavement with long fatigue life). Since mix design is a compromise between rutting performance and fatigue performance, the

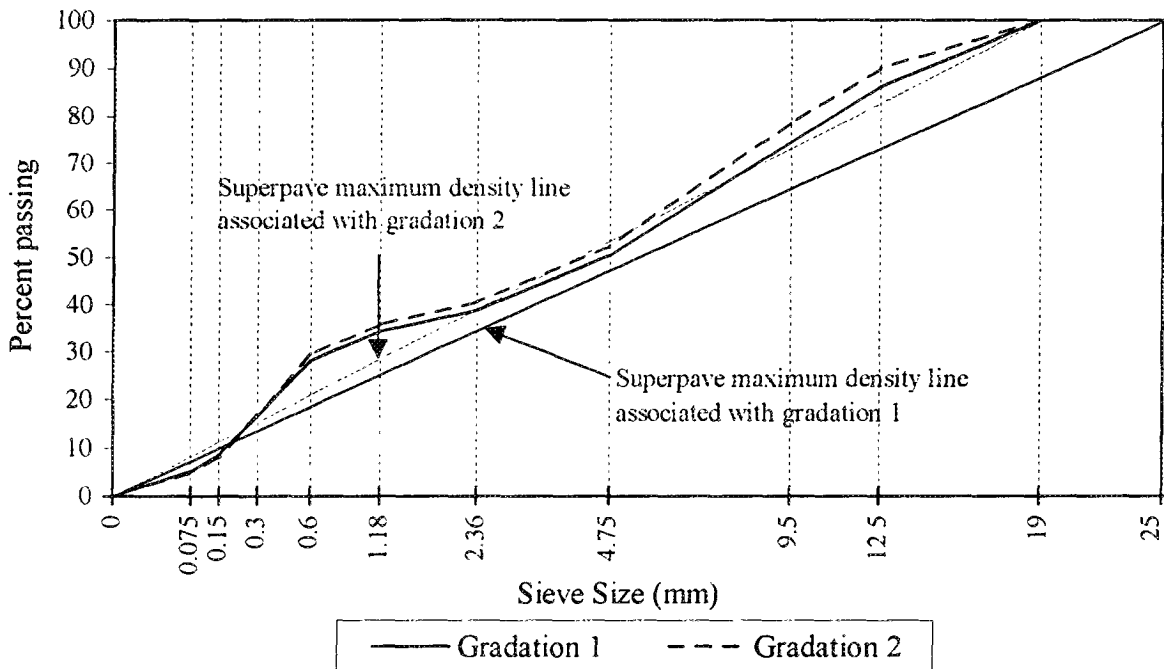
design gradations do not usually follow the maximum density line. However, there is evidence to suggest that the farther away is a mix from the maximum density line, the more susceptible it is to rutting (Goode and Lufsey 1962).

Notice that the definition of the maximum density line given above does not provide guidelines for choosing the maximum aggregate size from which the line should be drawn for a given gradation. This definition, however, is of paramount importance since it determines the slope of the line and the deviations of the given gradation from it. Unfortunately, there is no consensus as to how to define that maximum aggregate size.

In the Superpave mix design system (Harrigan et al 1994), the maximum aggregate size is one size larger than the *nominal* maximum aggregate size. In turn, the nominal maximum aggregate size is one size larger than the first sieve that retains more than 10 percent of aggregate. This definition is based on the results of Huber and Shuler (1992), who found that *VMA* correlated best with the sum of absolute distances from the maximum density line when using the above definitions. One of the data sets used by Huber and Shuler was the same one used by Goode and Lufsey (1962). Surprisingly, the first six gradations, which were used by Goode and Lufsey to develop the rationale for the 0.45 chart, were excluded from Huber and Shuler's regressions with this data set. Three of the points corresponding to these six gradations plot very far from their regression line. This may explain some existing problems with the definition of maximum aggregate size in Superpave. As illustrated in Figure 6, two gradations may be very similar but the maximum density lines associated with them (and consequently the deviations from the maximum density lines) may be significantly different. The maximum density line for gradation 1 is particularly troublesome. There is no reason why the maximum aggregate size (25 mm in the figure) should be so much greater than the actual maximum (somewhere between 12.5 and 19 mm). In fact, that gradation is representative of most of the gradations in WesTrack. None of the gradations from Goode and Lufsey's study used by Huber and Shuler's had this problem.

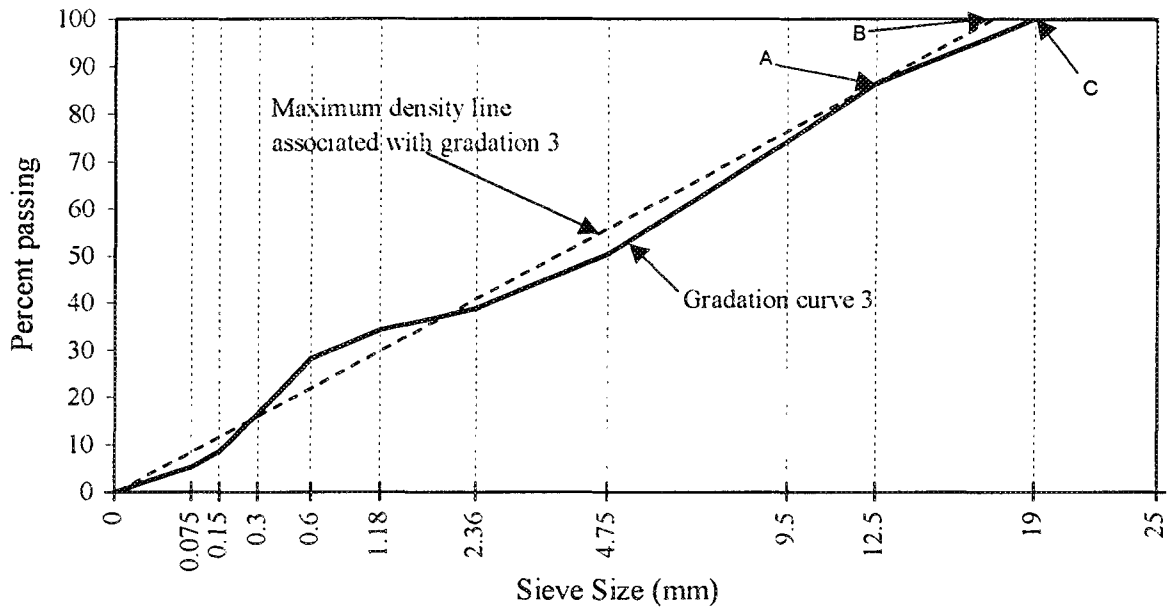
Clearly, a better definition of the maximum aggregate size associated with a specific gradation is needed. Although other researchers have examined different ways to define the maximum density line (e.g., Aschenbrener and MacKean 1994), no definite conclusion has been reached.



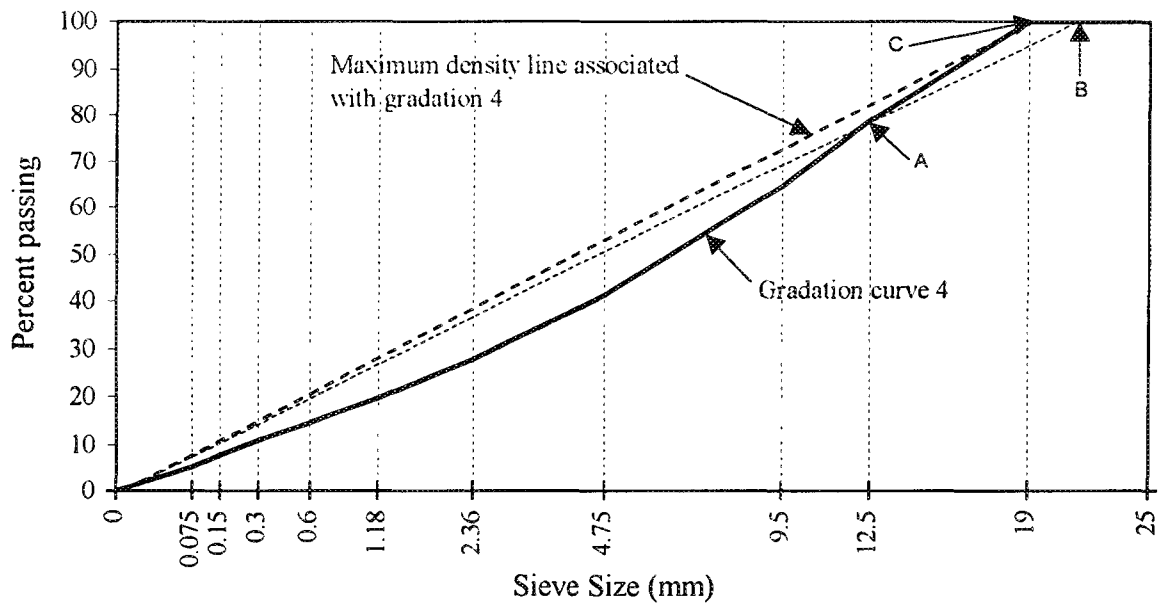


**Figure 6:** Problems with the Superpave definition of Maximum Aggregate Size.

In light of the above, we used the following definition of the maximum density line which is more in accordance with the method used by the developers of the chart (Goode and Lufsey 1962). First, identify point A as the point on the gradation curve corresponding to the sieve one size smaller than the smallest sieve with 100 % passing. Then, draw a straight line from the origin to point A and extend it until it intersects the horizontal line corresponding to 100% passing (point B). If point B is to the left of the point corresponding to the smallest sieve with 100 % passing (point C), use the straight line from the origin to point B as the maximum density line. Otherwise, draw the maximum density line from the origin to point C. This definition of the maximum density line identifies maximum aggregate sizes that are always relatively close to the actual maximum aggregate size. In addition, for the cases in which the problem described above is not present, this line is always somewhat to the left of the maximum density line as defined in Superpave. This is also convenient since in Goode and Lufsey's study it was found that the maximum stability was found for gradations a little to the left of the gradation with minimum *VMA*. The definition is illustrated for two hypothetical gradations 3 and 4 in Figures 7 and 8 respectively.



**Figure 7:** Illustration of the new maximum aggregate size definition for plotting maximum density lines for a hypothetical gradation curve 3.



**Figure 8:** Illustration of the new maximum aggregate size definition for plotting maximum density lines for a hypothetical gradation curve 4.

Based on the idea that gradations that are farther away from their corresponding maximum density lines have less aggregate interlock and thus are less rutting resistant, we defined a gradation index that is a measure of closeness to the maximum density line (as defined above). The gradation index (*GI*) is defined as 1/100 times the sum of the square of the differences in percent passing between the actual gradation and the maximum density line corresponding to that gradation. The summation is over the Superpave standard nest of sieves. These sieves are: 37.5, 25, 19, 12.5, 9.5, 4.75, 2.36, 1.18, 0.60, 0.30, 0.15, and 0.075 mm. Table 1 shows the gradation index computed for the 26 WesTrack original mixes. The gradation index seems to give a good characterization of the aggregate susceptibility to rutting. The index is generally smaller for the fine and fine-plus gradations and greater for the coarse gradations. This corresponded closely, other things being equal, with the way the sections rutted.

**Table 1:** Computed gradation indexes.

Fine		Fine-Plus		Coarse	
Section	GI	Section	GI	Section	GI
1	1.313	9	1.283	5	5.003
2	1.366	10	1.274	6	5.490
3	1.299	11	1.173	7	4.470
4	1.535	12	1.194	8	2.640
14	1.361	13	1.215	23	3.779
15	1.267	19	1.303	24	4.544
16	1.391	20	1.252	25	4.021
17	2.390	21	2.051	26	5.280
18	1.244	22	1.163		

## 5. Model specification

This section presents the rationale for the model specification. Many of the concepts used are taken from the specification of a rutting progression model developed by Archilla and Madanat (1999a) using a data set from the AASHO Road Test. Based on evidence from the material laboratory testing literature, and from the concave trend of deformation with respect to the

number of load applications appearing in most empirical and accelerated test studies the following equation is used as the starting point for the model specification:

$$RD_{it} = \beta_{i10} + a'_i (1 - e^{-b_i N_{it}}) \quad (1)$$

where

- $RD_{it}$  = rut depth for section  $i$  at time  $t$  (mm);
- $N_{it}$  = a variable representing the cumulative number of load repetitions applied to pavement section  $i$  up to time period  $t$  (a more complete definition is given later);
- $a'_i$  and  $b_i$  = functions of the characteristics of pavement  $i$  such as aggregate gradations and
- $\beta_{i10}$  = rut depth immediately after construction for pavement section  $i$ .

This equation produces a concave shape as the ones observed in laboratory studies. The parameter  $b_i = b$  is assumed constant for all sections while  $a'_i$  is assumed to vary with pavement characteristics. Since, in WesTrack, almost 100% of the rutting is due to permanent deformation of the asphalt concrete layer (Hand 1998),  $a'_i$  is assumed to be a function of the mix characteristics only. Clearly, the predictions of our model will be limited to rutting originating in the asphalt concrete layer.

In Archilla and Madanat's paper,  $N_{it}$  was defined as follows:

$$N_{it} = \sum_{s=i}^t \Delta V_{is} \left[ \left( \frac{FL_i}{80} \right)^{\alpha_1} + R_i \left\{ \left( \frac{AL1_i}{80} \right)^{\alpha_1} + \left( \frac{AL2_i}{\alpha_3 80} \right)^{\alpha_2} \right\} \right] \quad (2)$$

where

- $\Delta V_{is}$  = number of vehicle passes on section  $i$  during period  $s$ ,
- $FL_i$  = load in front axle of truck used in section  $i$  (kN);
- $AL1_i$  = load in *single* load axle(s) (rear axle(s)) of truck used in section  $i$  (kN);
- $AL2_i$  = load in *tandem* load axle(s) (rear axle(s)) of truck used in section  $i$  (kN);
- $R_i$  = number of load axles in truck used in section  $i$  ( $R_i=1$  or  $2$ ); and

$\alpha_j$  = estimated parameters (j=1,2,3). These parameters determine the equivalencies between axle loads.

This definition of  $N_{it}$  uses the concept of axle load equivalencies, which is well-accepted in pavement engineering. In equation (2) all the single loads have been standardized to an equivalent 80 kN (18,000 lbs) *single* axle load, which is the standard practice in pavement engineering. Tandem axles have been standardized by  $\alpha_3 \cdot 80$  kN, which is the standard *tandem* axle producing the same rutting as a *single* 80 kN axle. Further, different load equivalence coefficients are assumed for single ( $\alpha_1$ ) and tandem axles ( $\alpha_2$ ).

The fact that most of the rutting in WesTrack occurred in the upper portion of the asphalt concrete layer makes some aspects of equation (2) questionable. The state of stresses at a point on a pavement structure caused by a load at the pavement surface depends upon the applied pressure and the total load. For example, Figure 9 illustrates the effect of these two factors on the variation of the vertical stress with depth using the theory of elasticity on a semi-infinite medium and the assumption that the tire contact pressure is uniformly distributed over the pavement surface. For a given load, different tire contact pressures produce different stress distributions near the pavement surface but the stresses deeper in the pavement are virtually identical. On the other hand, for a constant tire contact pressure, the stresses caused by different load magnitudes immediately below the pavement surface are identical. However, a higher total load increases the vertical stresses below the surface.

As noted before, in equation (2) different equivalence coefficients are assumed for single and tandem axles. For WesTrack this assumption is unrealistic. Most of the rutting occurs near the surface, where the pressure bulbs of the axles in a tandem axle configuration do not overlap. Therefore, a tandem axle acts in effect as two single axles each carrying half the load of the tandem axle. Thus, the loading in WesTrack is specified as follows:

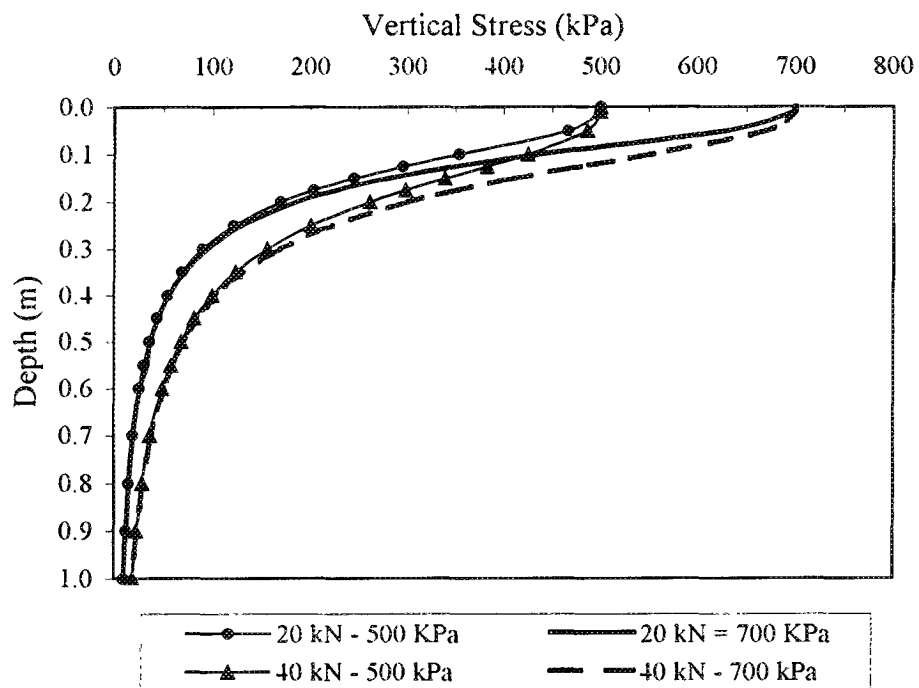
$$N_{it} = \sum_{s=1}^I \Delta V_{is} \left[ \left( \frac{FL}{80} \right)^\delta + 5 \left( \frac{AL1}{80} \right)^\delta + 2 \left( \frac{AL2}{2 \cdot 80} \right)^\delta \right] = \sum_{s=1}^I \Delta V_{is} \left[ \left( \frac{53.4}{80} \right)^\delta + 7 \left( \frac{89}{80} \right)^\delta \right] \quad (3)$$

where

- $\Delta V_{is}$  = number of vehicle passes on section  $i$  during period  $s$ ,
- $FL$  = load in front axle of the truck (53.4 kN);
- $AL1$  = load in *single* load axle(s) (89 kN);
- $AL2$  = load in *tandem* load axle (2 x 89 = 178 kN);
- $\delta$  = parameter determining the equivalencies between axle loads.

Since the loading is identical for all the sections in WesTrack, the coefficient  $\delta$  cannot be statistically identified. This does not present a problem for the model estimation since  $N_{it}$  can simply be expressed as:

$$N_{it} = \text{constant} \sum_{s=1}^t \Delta V_{is}$$



**Figure 9:** Effect of wheel load and tire inflation pressure on the vertical stress distribution assuming a semi-infinite elastic material.

Assuming a given value for the constant (and consequently a value for  $\delta$ ) affects the magnitude of the coefficients multiplying  $N_{it}$  but not the predicted values of rutting. In other words if the constant is multiplied by a factor  $\kappa$ , the estimate of the parameter that multiplies  $N_{it}$  is divided by  $\kappa$  but the rutting predictions are unaffected. The value assumed in our model was transferred from a revised AASHO model, which is described in detail in Archilla and Madanat (1999b). The revised model considers rutting as the sum of two components. The first component captures rutting in the underlying layers. The second component accounts for rutting in the asphalt concrete layer. The value of  $\delta$  in (4) is assumed the same as the one obtained by Archilla and Madanat for rutting in the asphalt concrete layers at the AASHO Road Test, which was 0.39. This value is significantly different from the commonly assumed value of 4. However, as we observed before, near the pavement surface, the effect of the tire inflation pressure may dominate the effect of the load. The total load still has an effect, but it is much less pronounced than the effect caused deeper in the pavement structure. The value of 0.39 is consistent with these observations. It must be noted that the actual value of  $\delta$  for WesTrack could be different because the tire inflation pressure used in WesTrack (704 kPa) is different from the one used in the AASHO Road Test.<sup>2</sup>

In WesTrack, high air temperatures played a very important role in the development of rutting. Since the high air temperatures alter the materials' properties, one could try to incorporate its effect in  $a'_i$ . The problem is that equation (1) is not suitable for this kind of adjustment. The reason is that one would like to obtain a monotonic increasing function with traffic. If during hot periods,  $a'_i$  increases, then it is possible that the function decreases afterwards when there are cooler temperatures.

Since it was desirable to keep that functional form, a first order Taylor series approximation was used. This gave:

$$RD_{it} \approx RD_{i,t-1} + a'_i b \Delta N_{it} e^{bN_{it}} = RD_{i,t-1} + a'_i \Delta N_{it} e^{bN_{it}} \quad (4)$$

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<sup>2</sup> The tire inflation pressures at the AASHO Road Test were 169 kPa for loop 2 lane 1, 316 kPa for loop 2 lane 2, 528 kPa for loops 3 to 5 and 563 kPa for loop 6.

where:

$$\Delta N_{i,t} = N_{i,t} - N_{i,t-1} = \Delta V_{it} \left( \left( \frac{53.4}{80} \right)^\delta + 7 \left( \frac{89}{80} \right)^\delta \right) \quad (5)$$

and

$$\alpha_i = \alpha', b$$

With this new formulation,  $\alpha_i$  can be assumed to vary when the environmental conditions change, so equation (4) can be modified as follows:

$$RD_{it} = RD_{i,t-1} + \alpha_{it} \Delta N_{it} e^{bN_{it}} \quad (6)$$

where the subscript  $t$  is added to  $\alpha_i$  to indicate that it varies over time. Substituting successively the values of  $RD_{i,t-1}$ ,  $RD_{i,t-2}$ , etc, the following equation is obtained

$$RD_{it} \approx \beta_{i10} + \sum_{s=1}^t \alpha_{is} \Delta N_{is} e^{bN_{is}} \quad (7)$$

To complete the model specification; the specification of  $\alpha_{it}$  must be finalized. As we pointed out in section 3, for a given gradation, rut depth remains approximately constant up to certain *VFA* values. After that, rut depth increases significantly with *VFA*. The gradation seems to influence the rut depth values for low *VFAs*, the threshold at which higher *VFAs* cause increases in rutting and the rate of increment with higher *VFAs*. Based on these observations, we hypothesized that  $\alpha_{it}$  has two additive components  $\alpha_{1it}$  and  $\alpha_{2it}$ , one that is a function of gradation only and another which is a function of gradation, *VFA* and temperature. Since the asphalt is very temperature susceptible, the effects of temperature are considered along with *VFA*.

When there is adequate compaction of the asphalt concrete mix and a minimum amount of asphalt to hold the particles together, there seems to be an intrinsic resistance of the aggregate structure that is independent of the amount of asphalt in the mix. As mentioned before, evidence in the literature suggests that aggregates with gradations closer to the maximum density line (as defined in this paper) provide more rutting resistance. Thus, gradations with smaller *GIs* (closer



to the maximum density line) will deform less than gradations with greater  $GI$ s. In order to express this dependency on the gradation index we used a linear function of the  $GI$ :

$$\alpha_{1it} = \beta_1 + \beta_2 \cdot GI_t \quad (8)$$

As mentioned before, when the  $VFA$  in the mix exceeds a certain threshold (that also seems to depend on gradation), rutting increases significantly. In addition, these increments occur mostly during periods with high temperatures.

In order to model this complex interaction between gradation,  $VFA$ , and air temperature, the following form was originally specified for  $\alpha_{2it}$ .

$$\alpha_{2it} = (\beta_3 + \beta_4 \cdot GI_t) \left( \frac{VFA_t}{100} \right)^{(\beta_5 + \beta_6 \cdot GI_t)} \cdot f(MeanMaxT_t) \quad (9)$$

where  $MeanMaxT_t$  is the mean maximum temperature during period  $t$  and  $f(\cdot)$  is a function of

$MeanMaxT_t$ . The first part of the equation,  $(\beta_3 + \beta_4 \cdot GI_t) \left( \frac{VFA_t}{100} \right)^{(\beta_5 + \beta_6 \cdot GI_t)}$ , captures the sharp

increments of rut depth with  $VFA$  observed in Figure 5. In the discussion of that figure in section 3, we noted that there seems to be a  $VFA$  threshold above which the rut depth increases dramatically and that the threshold depends on the gradation. Those two properties are accounted for with the above specification.

The multiplicative function  $f(\cdot)$  in the second part of equation (9) has a displaced logistic form shown in curve a of Figure 10. This function can be written as:

$$f(MeanMaxT_t) = \left( 1 + \frac{\alpha_1}{1 + \exp(\alpha_2 + \alpha_3 MeanMaxT_t)} \right) \quad (10)$$

The displacement, which is equal to one, is intended to capture what happens at very low temperatures. The logistic part of the factor is intended to capture the increment of rut depth that occurs with higher air temperatures. As shown in that figure, we do not expect temperature to play a role for low air temperatures but as the air temperature increases, the asphalt in the mix

will lose viscosity and the mix will rut more easily. However, we do not expect the behavior of the mix to change much either once the asphalt has become too soft. This is represented by the constant values for high temperatures.

Although equations (9) and (10) give a valid specification, two problems arose during the estimation phase. The first was that  $\alpha_1$  was several orders of magnitude larger than 1 and  $\alpha_2$  and  $\alpha_3$  were several orders of magnitude smaller than 1. This is simply an indication that all the rutting related to *VFA* and air temperature occurs only at high air temperatures. Therefore, the first 1 in equation (10) is dropped from the specification.

The second problem was that the combinations of  $\alpha_2$  and  $\alpha_3$  produced a curve like **b** in Figure 10. Although we did not specify a threshold type relation, the model indicated that such a relation exists<sup>3</sup>. The threshold for the mean maximum air temperature we obtained was 28.6°C. Since, there are two parameters to define one threshold, the model specification presented a problem of perfect collinearity.

In light of the above two problems, the specification for  $a_{it}$  was simplified as follows:

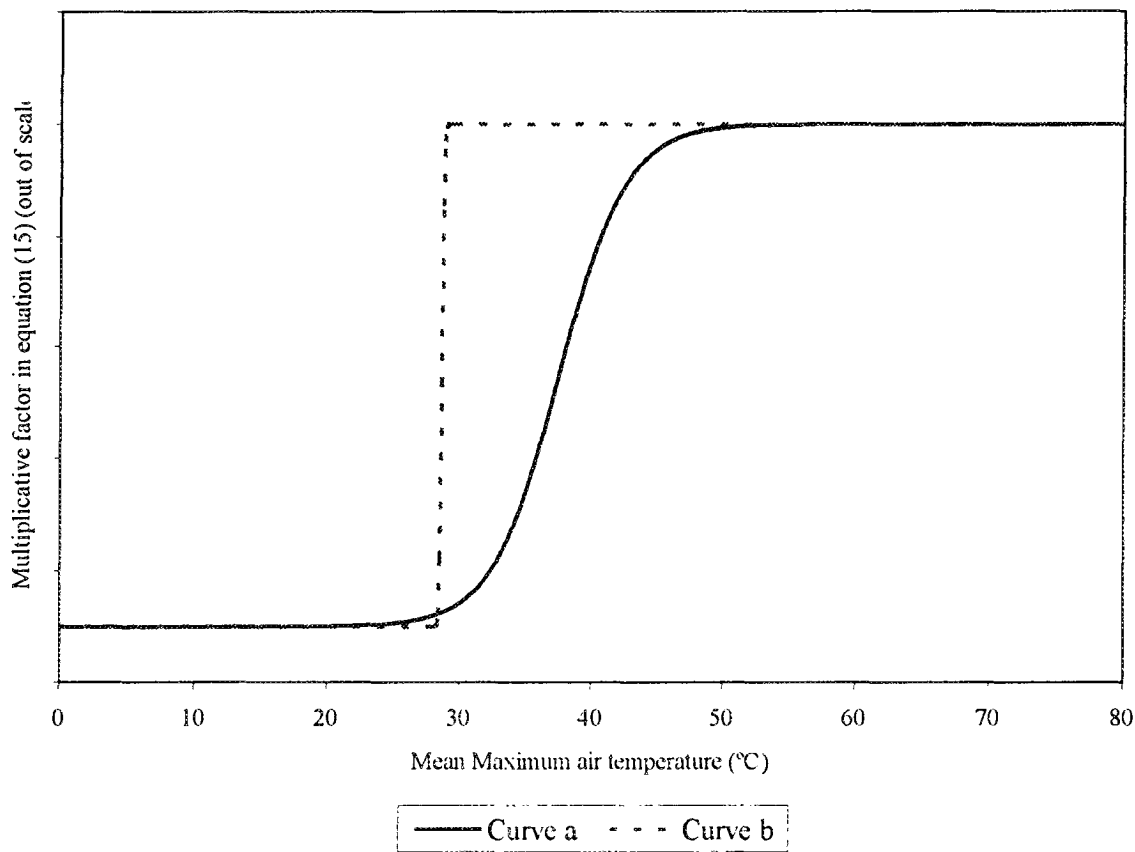
$$a_{it} = (\beta_1 + \beta_2 GI_t) + (\beta_3 + \beta_4 \cdot GI_t) \left( \frac{VFA_t}{100} \right)^{(\beta_5 + \beta_6 \cdot GI_t)} \times TempDum_t \quad (11)$$

where *TempDum<sub>t</sub>* is equal to one if the *MeanMaxT<sub>t</sub>* > 28.6°C and 0 otherwise. Thus, the second term in equation (11) only plays a role for high air temperatures.

The model as currently specified produced very good results. Nevertheless, we added one more additive term to  $a_{it}$  to account for the effects of initial in place air voids because many engineers consider this variable as a main determinant of rutting. In addition, this factor gives an indication of the effects of the levels of compaction on the mix rutting performance.

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<sup>3</sup> A relation like the one depicted by curve a in Figure 9 is likely to exist, but it is probable that the data set is not rich enough to identify it.



**Figure 10:** Functional forms for equation (10).

Indirectly, the model already considers the effect of air voids near the end of the pavement life. This is because the specification of the model already includes a term for  $VFA$  as obtained in the SGC for 96 gyrations and  $VFA$  can be expressed as a function of  $AV$  (or effective  $AC$ ) and  $VMA$ . The 96 gyrations are intended to produce the same level of compaction observed after the pavement has been subjected to the design traffic. The term including  $VFA$  in equation (11) accounts for the higher rut depth values usually observed for very low air voids.

However, for higher air voids (say above 4 %) we expect that a pavement with higher initial air voids will tend to compact more under traffic than a pavement with low initial air voids. We also expect this to be a transient effect that is more pronounced immediately after construction. This is consistent with the usually observed reductions in air voids over time.

In order to include this effect of high initial air voids in the model, we define a compactable voids variable ( $CV$ ). The compactable voids are initially considered equal to the initial air voids and they are reduced during each time period as a function of the applied traffic, until  $CV$  becomes zero. Whenever  $CV$  is greater than zero, the mix will be less rutting resistant (implying greater  $a_{it}$ 's). Thus, the new specification for  $a_{it}$  is

$$a_{it} = (\beta_1 + \beta_2 GI_t) + (\beta_3 + \beta_4 \cdot GI_t) \left( \frac{VFA_t}{100} \right)^{(\beta_5 + \beta_6 \cdot GI_t)} \cdot TempDum_t + \beta_8 CV_{it} \quad (12)$$

where

$$CV_{it} = AV_t \exp(\beta_9 N_{it}) \quad (13)$$

The lower limit of 0 % for the compactable voids (obtained as  $N_{it} \rightarrow \infty$  with a negative value for  $\beta_9$ ), is questionable. Perhaps a value near 2 % is more appropriate. Nevertheless, the predicted values for rutting are not very sensitive to low values of  $CV$  and thus we may ignore this difference<sup>4</sup>.

In summary, our model specification is given by

$$RD_{it} \approx \beta_{i10} + \sum_{s=1}^t a_{is} e^{\beta_7 N_{is}} \Delta N_{is} \quad (14)$$

with  $a_{is}$  given by equation (12)<sup>5</sup>.

## 6. Model estimation results

Equation (14) is the conditional expectation function of rut depth for section  $i$  at time  $t$ ,  $E(RD_{it}|X_{it},\beta)$ . This function gives expected rut depth conditional on the set of regressors  $X_{it} =$

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<sup>4</sup> Recall that in equation (8) as  $N_{it} \rightarrow \infty$ ,  $\exp(b N_{it}) \rightarrow 0$  for a negative  $b$ , and therefore a 10 or 20% change in  $a_{it}$  is negligible.

$(1, \Delta V_{it}, \dots, \Delta V_{it}, GI, VFA, AV, TempDum_t)'$  and on the vector of parameters  $\beta = (\beta_1, \dots, \beta_{10}, \beta_{10,S})'$ . The model can be expressed as the following set of regression equations:

$$RD_{it} = E(RD_{it} | X_{it}, \beta) + \varepsilon_{it} \quad i = 1, \dots, S, \quad t = 1, \dots, T_i \quad (15)$$

where  $S$  is the number of sections,  $T_i$  is the number of observations for section  $i$  and  $\varepsilon_{it}$  is the error term which is assumed to have mean 0 and constant variance  $\sigma_\varepsilon^2$ . As can be seen from equation (14) this model is nonlinear in the variables and the parameters. Moreover, the vector  $X_{it}$  contains the whole history of loading through the  $\Delta I$ 's. All these factors make the estimation of the model fairly complex.

When a data set consists of observations for different pavement units through time, several methods of pooling the data can be used. Such data sets are known as panel data sets. One could estimate separate cross-section regressions (each using observations for different pavement sections at the same point in time) or separate time-series regressions (each with observations for a single pavement section over time). However, if the model parameters are constant over time and over cross-sectional units, more efficient parameter estimates (i.e., estimates with lower variance) can be obtained if all the data are combined and a single regression is run. This is the case if all observations are the result of a single underlying deterioration process.

The simplest technique is to combine all cross-section data and time series data and perform ordinary least-squares regression on the entire data set. In the present context, this would mean to perform a regression using equation (15) with  $E(RD_{it} | X_{it}, \beta)$  given by equation (14) and assuming that  $\beta_{i10} = \beta_{10}$  is the same for all  $i$ . The problem with this procedure is that despite the reasonableness of the assumption that all the observations are the result of a single underlying process, some unobserved heterogeneity (representing unobserved and persistent pavement-specific factors) is still expected among different pavement sections. An example of unobserved heterogeneity is the initial cross-section profile, which directly influences the intercept term in

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<sup>5</sup>  $\beta_7$  replaces  $b$  in equation (8).

our model. This is an example of the kind of unobserved heterogeneity that we account for in our model.

The advantage of a panel data set over a cross sectional data set is that it allows the researcher greater flexibility in modeling differences in behavior across individual units (Greene 1997). The two most widely used frameworks for modeling unobserved heterogeneity are called fixed and random effects respectively. Both approaches assume that the unobserved heterogeneity can be captured through the constant term. In the *fixed effects* approach, the individual effect ( $\beta_{i10}$ ) is taken to be constant over time and specific to the individual pavement section  $i$ . This approach always produces consistent results, as the number of sections  $S$  approaches infinity, for the vector  $\beta_I = (\beta_1, \dots, \beta_9)'$ .<sup>6</sup> That is, it produces consistent results for the vector of parameters excluding the intercepts. The problem with this approach is that it is costly in terms of the number of degrees of freedom lost, because a different intercept term is required for each pavement section.

An alternative approach is the random effects specification. Since the inclusion of different constant terms ( $\beta_{i10}$ ) represents a lack of knowledge about the model, it is natural to view the section specific constant terms as randomly distributed across pavement sections. Specifically, it is assumed that  $\beta_{i10} = \beta_{10} + u_i$ , where  $u_i$  is a random disturbance characterizing the  $i^{\text{th}}$  section and is constant through time with mean  $E(u_i) = 0$  and constant variance equal to  $\sigma_u^2$ . With these assumptions the random effects specification is:

$$RD_{it} = \beta_{10} + \sum_{s=1}^t \alpha_{is} e^{\beta_1 N_{is}} \Delta N_{is} + u_i + \varepsilon_{it} \quad (16)$$

This approach is appropriate if it is believed that the sampled cross sectional units are drawn from a large population (Greene 1997). However it yields consistent parameter estimates only if the regressors are uncorrelated with the individual effects  $u_i$ . This can be tested using a Hausman specification test (Greene 1997).

Both approaches are used to estimate the model parameters in this paper, but since they yield very similar parameter estimates, we only present the results using the random effects approach. The estimation approach for linear models can be found, for example, in Greene (1997). The estimation of our model parameters is more complicated since our model is nonlinear in the variables and the parameters, and the panel is unbalanced (that is, there are different numbers of observations for different pavement sections). Therefore special routines had to be programmed for estimation of the model. The details of the estimation approach are given in Archilla (1999).

Table 2 shows the estimation results. During estimation,  $\beta_1$  and  $\beta_3$  were constrained to be greater than or equal to zero since there is no reason to expect a reduction in rut depth for any value of  $GI$ , and particularly when  $GI$  equals zero. As can be observed, the constraint  $\beta_3 \geq 0$  is binding. It can also be observed that only seven of the other nine parameters are statistically significant at a 5 % significance level. However, the joint hypothesis, that  $\beta_8$ , and  $\beta_9$ , are jointly equal to zero produces a  $\chi^2$  statistic of 8.02. Since the 5 percent critical value from the chi-squared distribution with 2 degrees of freedom is 5.99, the hypothesis that  $\beta_8$  and  $\beta_9$  are jointly equal to zero is rejected.

The interpretation of  $\beta_1$ ,  $\beta_2$ ,  $\beta_4$ ,  $\beta_5$  and  $\beta_6$  is done with the help of Figure 11. The figure shows the value of  $a_{it}$  as given by equation (11) for different values of  $GI$  as a function of  $VFA$  when the mean maximum temperature is above 28.6 C. When  $VFA$  is low or when the mean maximum temperature is below 28.6C, only  $\beta_1$  and  $\beta_2$  play a role in  $a_{it}$ . The positive signs of the coefficients  $\beta_1$  and  $\beta_2$ , indicate that independently of temperature, an asphalt concrete pavement will always suffer some rutting that will be more severe for higher gradation indexes. This is represented in the figure by the different constant values of  $a_i$  for different values of  $GI$  and low  $VFA$ 's.

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<sup>6</sup> An estimator  $\hat{\theta}$  of a parameter  $\theta$  is a consistent estimator of  $\theta$  if and only if  $\hat{\theta}$  converges in probability to  $\theta$  (Greene 1997). Convergence in probability implies that the values that the variable may take that are not close to  $\theta$  become increasingly unlikely as the sample size increases.

The combination of  $\beta_4$ ,  $\beta_5$  and  $\beta_6$  causes  $a_{it}$  to increase rapidly with  $VFA$  after a certain value of  $VFA$ . The  $VFA$  at which the sudden increase in  $a_{it}$  occurs is higher for low  $GI$ 's than for high  $GI$ 's.

$\beta_7$  is statistically significant from zero at a 5 % level and it has the correct sign. That  $\beta_7$  is negative indicates that the material hardens over time. In other words, the same loading increment will produce a smaller increment in rut depth for more trafficked pavements.

**Table 2:** Model estimation results.

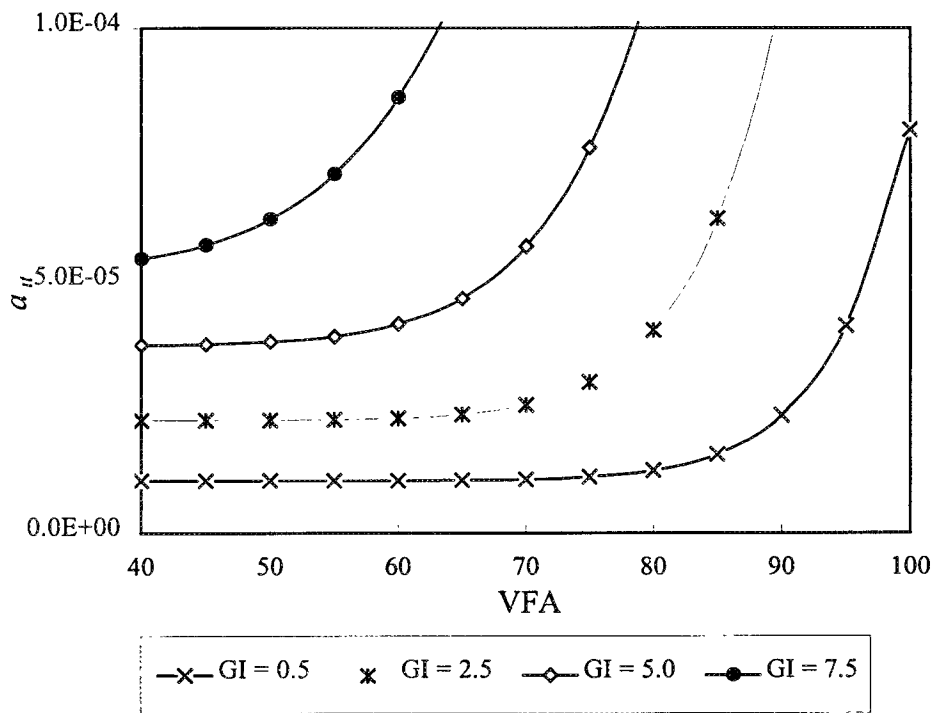
Parameter	Parameter Estimate	Asymptotic t-statistic
$\beta_1$	7.44e-6	3.00
$\beta_2$	5.97e-6	4.21
$\beta_3$	0.00e+0	0.00
$\beta_4$	1.39e-4	10.81
$\beta_5$	1.66e+1	21.77
$\beta_6$	-1.32e-0	-4.23
$\beta_7$	-2.50e-6	-11.38
$\beta_8$	1.61e-6	2.54
$\beta_9$	-2.25e-6	-1.21
$\beta_{10}$	3.40e-1	1.44
$\sigma_e^2 = 3.01$		$\sigma_v^2 = 8.16$
Number of observations = 860		

Finally,  $\beta_8$  is positive, which as described in the previous section, has the expected sign. It should be noted that this result does not indicate that to reduce rutting one should try to obtain an air voids value as small as possible. Recall that  $VFA$  can be linked to air voids, and as shown in Figure 11, high  $VFA$  values (and consequently low air voids) will lead at some point to substantial rutting increments. Thus, the estimate of  $\beta_8$  together with estimates of  $\beta_4$ ,  $\beta_5$  and  $\beta_6$  indicate that there is an optimum air voids at which rutting is minimized. This result is consistent with traditional asphalt mix design. It should be noted that the optimum is dependent on whether the asphalt concrete mix is subjected to high air temperatures and, because of the hardening of the mix, on when those high air temperatures occur.



The estimates of  $\sigma_u^2$ , 8.16 and of  $\sigma_\varepsilon^2$ , 3.01 indicate that the individual effects produce more than 50 % of the variance. This shows that the size of the unobserved heterogeneity is significant.

Finally, the estimated standard error of the regression ( $=\sqrt{\sigma_u^2 + \sigma_\varepsilon^2}$ ), 3.34 mm, is within the accuracy with which rut depth can be measured. The result is even better in a pavement management context where the random effects are less important since previous observations of rut depth are used to predict the future observations. In this case the estimate of  $\sigma_\varepsilon$ , which is only 1.73 mm, is more relevant.



**Figure 11:**  $a_{it}$  as given by (12) for different values of  $GI$  as a function of  $VFA$  when the mean maximum temperature is above 28.6C.

Figure 12 shows a comparison of the predicted rut depths and the observed rut depths for eight of the 26 sections in the estimation sample. The three mix categories, fine, fine-plus, and coarse are

represented in the figure. It can be observed that in general the pavement behavior is replicated well. Only section 19 shows a significant overestimation of the rut depths. Actually, for this section the model performed the worst. Only two other sections presented similar problems but of lesser magnitude. Sections 20 and 22 are two good examples of the intercepts' heterogeneity. The under-prediction for these two sections results from the under-prediction of the intercept.

The above results indicate that the model assumptions seem to be generally valid. However, it should be noticed that the residuals do increase somewhat with time (particularly for sections 17 and 19), which may lead to some estimation inefficiency.

## **7. Conclusions and Recommendations**

The goal of this paper was to develop a model of pavement rutting from the WesTrack Road Test. A non-linear model in the variables and the parameters was specified and estimated. The model specification uses concepts that are familiar to pavement engineers such as load equivalencies but it also presents some unique features. Three properties of the mix are sufficient to model the performance of the asphalt concrete pavement accurately. The three mix properties are a gradation index, which is obtained from the aggregate gradation, the voids filled with asphalt obtained for the construction mix in the Superpave gyratory compactor, and the initial in-place air voids.

The model also captures the effects of high air temperatures at WesTrack. The effect of high temperature depends on how far the aggregate gradation of the mix is from its corresponding maximum density line. In addition, the model estimation results indicate that there is a value for air voids that optimizes rutting performance. This result is consistent with traditional asphalt concrete mix design.

Finally, the model predicts rut depths by adding predicted values of the increment of rut depth for each time period. This is particularly advantageous in a pavement management context where

the current rut depth value is known and the interest is in predicting the rut depth value for the next time period (and hence only the change in rut depth).

The gradation index defined in this paper seems to be a good indicator of the resistance to rutting provided by the aggregate structure, at least with the WesTrack data set. However, since only 26 sections were used in the model estimation, these results should be taken with caution. Results in the literature indicate that some gap graded aggregates that depart from the maximum density line can still produce mixtures with good rutting resistance. Ideally, it would be better to use a mechanical test to measure the resistance of the aggregate structure to rutting. One such measure could be, for example, the slope of the densification curve from the Superpave Gyrotory Compactor. The slope is calculated as the change in the percentage of maximum specific gravity as a function of the log the number of gyrations. This slope is currently considered by some researchers as an indicator of the resistance to compaction (Anderson and Bahia 1997). However, as currently performed, the results of this test can be severely influenced by the amount of asphalt in the mix. This would not be the case, however, if the test were performed at a *VFA* value of say 50%. As shown in Figure 11, even the gradations that depart considerably from the maximum density line (that is, gradations with high *GP*'s) are not affected by *VFA* values this low.

The model fits were good. Both fixed effects and random effects specifications were used to account for unobserved heterogeneity. The results showed that the size of the unobserved heterogeneity was significant.

Despite the significant estimation results in this paper, the model has the following limitations. First, our model is limited to the materials used at the WesTrack Test, and for those materials, to the range of values of the independent variables used in the test. For example, the model may be overly optimistic for *GP*'s lower than 1.2. Second, the model can not account for pavement rutting due to the deformations in the underlying layers. Finally, the experimental data may not represent the true deterioration mechanism of in-service pavements because of differences in factors such as traffic wander, traffic speed, and material aging.

For the development of better pavement rutting models, it is necessary to overcome the above limitations. A promising approach is the use of joint estimation from different data sources (Ben-Akiva and Morikawa 1990). The objective of joint estimation is to yield a more reliable model of pavement rutting than those produced with either data source alone. This is the subject of ongoing research by the authors.

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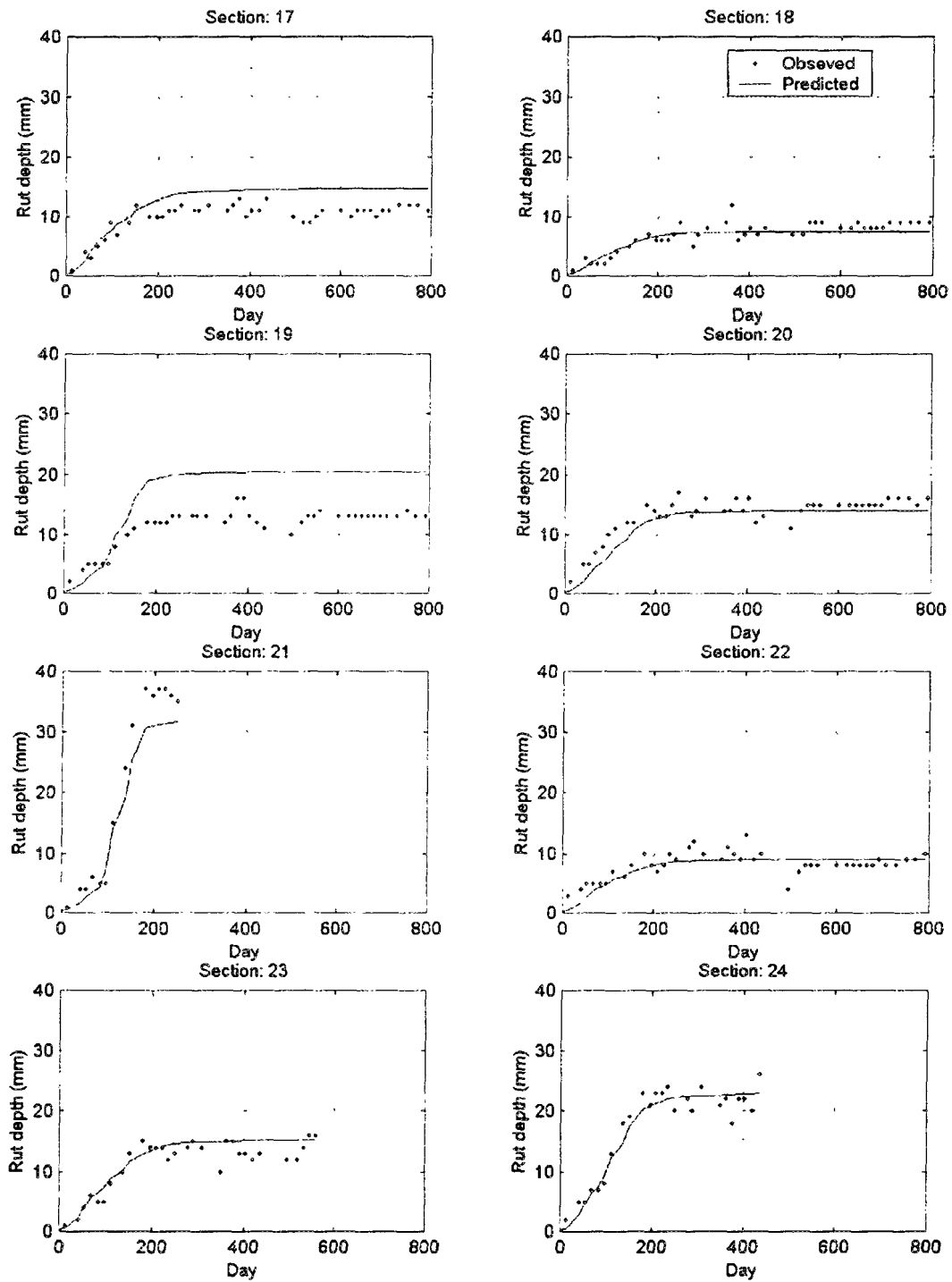


Figure 12: Observed and predicted rut depth vs. time for eight sections.

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