An Evaluation of Granular Overlays in Washington State — A Summary

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16. ABSTRACT

Granular overlays have been used by the Washington State Department of Transportation (WSDOT) for about 30 years. Since the mid-1980s and along with the full implementation of the WSDOT Pavement Management System (WSPMS), WSDOT has been interested in examining the performance of granular overlays. It is felt by WSDOT that the performance of this rehabilitation treatment is better than one might reasonably expect. Further, past practice in Washington state occasionally required that the preexisting surfacing (often several bituminous surface treatment (BST) layers) be scarified prior to placement of the crushed rock layer. This summary report will overview the study and conclusions.

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Research Project 8719, Task 31
Granular Overlay (Cushion Course)

AN EVALUATION OF GRANULAR OVERLAYS IN WASHINGTON STATE — A SUMMARY

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DISCLAIMER

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SUMMARY AND CONCLUSIONS

Although granular overlays have been used in a variety of countries around the world, their behavior has not been well documented. This study examined granular overlays from several perspectives and drew initial conclusions about how to design and build the overlays.

The study reviewed some of the advantages and limitations of granular overlays. They were found to be useful in reducing reflection cracking, insulating the old pavement surface against extremes in temperature, and improving the road geometry. However, the added thickness of the granular overlay can preclude its use in areas where the road geometry is limited.

In designing granular overlays, it is important to protect the crushed rock layer. Several South African studies have shown a reduction in stress-stiffening in inverted pavements when the crushed rock layer becomes saturated. In light of this, the authors of this report suggest that the old pavement surface be sealed and that the new surface layer be made as impermeable as possible to maximize the performance of the granular overlay.

This study also reviewed the affect of the rock quality and compaction of the crushed rock layer. In all of the South African studies, the inverted pavements with crushed rock that was more densely compacted and contained particles with a higher percentage of fractured faces was more resistant to deformation and to the reduction of the stress-stiffening caused by moisture infiltration. One study found that by tightening the specifications for the crushed rock, engineers could obtain densities in the crushed rock layer in excess of 100 percent of modified AASHTO. Therefore, by using a crushed rock with a dense gradation and a high percentage of fractured faces and by compacting it the maximum possible amount, engineers can produce a granular overlay with a better durability. The costs and benefits of tightening these specifications were not determined.

The suitability of BST and AC overlays as surfacings for granular overlays were examined, and BSTs were chosen as the more advantageous. BSTs have the advantage of being more "flexible." BSTs are also less expensive and easier to construct; however, they do not last as long as AC surfaces (although performance data from Districts 2 and 6 suggest that the differences are not large).

The thickness of the crushed rock layer was examined and was found to have a maximum recommended value of about 6 in. (150 mm). This recommendation came primarily from a South African study that found that thicknesses beyond 6 in. (150 mm) did not significantly increase the pavement's resistance to deformation. Additionally, in a paper on the use of granular overlays in Zimbabwe, problems with compacting crushed rock layers thicker than 6.0 in. were cited as the reason that thicker layers are not used there. That author also mentioned that thinner layers were difficult to compact, but WSDOT, which regularly builds crushed rock layers as thin as 3.0 in. (75 mm), has not had that problem.

Sibal studied the comparison of the number of 18 kip (80 kN) ESALs that both AC and granular overlay pavements can withstand. He examined both rutting and fatigue failure and calculated the thickness of crushed rock in a granular overlay that would provide the same life as an AC overlay. These equivalency factors ranged from 1.6 to 5.8, with most around 2.0. Mitchell also found that the AC equivalency factor was 2.0 for the confined crushed rock layer.

The survival lives of BST, AC, and granular overlays were examined. Although this study revealed that most WSDOT pavements were badly distressed at the time that they were overlaid with granular overlays, their useable life was still at least as long as those that did not receive the granular overlay (but some form of resurfacing). The difference was largest when a BST surface was placed over the crushed rock layer. In the case of AC surfaced pavements, the increase caused by the crushed rock layer was not significant. The additional useable life offered by the granular overlay was small in part because of the type of roads that were overlaid with the different techniques. Granular overlays were generally only used on roads that had significant cracking. On SR17 between Mileposts 120 and 128, a BST lasted less than 3 years. After the BST had failed, a granular overlay was applied, and this overlay had already lasted 6 years at the time this report was prepared with only slight deterioration. The researchers concluded that if granular overlays had been applied to roads in conditions comparable to roads that had received the BST and AC resurfacings, the useable life of granular overlays probably would have been longer. To reinforce this, the performances of not only SR17 but also SR21A, SR21B, SR231A, and SR231B were reviewed. The review suggested that granular overlays outperform plain BSTs by factors ranging from a low of two to a high of ten times better (as measured in years to comparable PCR levels). Further, the initial performance of granular overlays was even better when compared to plain BSTs (15 to 30 times better), which suggested that the quick failures of BSTs placed on pavements in poor condition was precluded by use of the granular overlays.

Eight pavements with granular overlays were tested with a FWD

The results of the direct analysis of the data showed that the pavements with the granular overlays were generally stiffer than those without the overlays. An exception to this was SR231A, where the section with the granular overlay was approximately equal to the section without it. A possible reason for this discrepancy was that the PCR of the section without the granular overlay was much higher when the overlay was laid than the section with the granular overlay. Therefore, the pavement under the granular overlay was probably weaker than that under the section without the overlay.

When the results of the tests were used in a backcalculation program, most of the test sections could not be analyzed. The reasons for these problems included inaccurate layer thicknesses, non-level or non-uniform pavement layers or subgrade, and the complexity of the pavement structure including unknown depths to a rigid (rock) layer. The two sections that were successfully backcalculated were in good condition and had crushed rock layer moduli around 80 ksi (550 MPa) under a 9.0 kip (40 kN) load. To more successfully calculate the moduli for these roads, the true layer thicknesses would have to be determined.

In general, granular overlays are an effective method of rehabilitating pavements where the thickness of the pavement structure needs to be increased or if there are problems with reflection cracking.

Granular overlays can also be a less expensive alternative to AC overlays in situations where the traffic loadings are appropriate. This is especially true for rural areas, where haul distances are long, because AC is more difficult to transport than crushed rock (or the need for portable AC mixing plants is eliminated).

The design of a granular overlay should concentrate on making the crushed rock layer as dense as possible and protecting it from moisture infiltration. A properly designed and constructed confined crushed rock layer will provide comparable performance as an AC layer that is half as thick (Equivalency Factor = 2.0).

A limitation on the use of granular overlays that was not studied was the maximum traffic or ESAL count. Although the South African studies commonly cited 18 kip (80 kN) ESAL counts of over 20 million for inverted pavements, WSDOT experience has shown that this is much higher than can be expected for its granular overlays as they are currently designed and constructed. Although an attempt was made to study the ESAL count on the test roads that were evaluated during this study, the ESAL count made was not considered to be accurate. The recommended maximum ADT for BSTs of 2,000 to 5,000 with 15 percent trucks is the best estimate of the maximum traffic at this time (May 1991).

INTRODUCTION

This summary report overviews information contained in Research Report WA-RD 226.1, "An Evaluation of Granular Overlays in Washington State."

The granular overlay system (hereafter referred to as "granular overlay") is an alternative type of overlay for rehabilitating mostly low volume, rural roads. The overlay consists of a layer of densely compacted, crushed rock overlain by a generally thin surface layer. Figures 1 and 2 show typical granular and asphalt concrete (AC) overlays.

Granular overlays have been used throughout the world as a pavement rehabilitation treatment. The reasons for their use fall into four primary categories:

1. to reduce reflective cracking from a preexisting pavement structure,
2. to add extra pavement structure thickness to combat frost related effects,
3. to improve the cross-slope, road profile (and ride in general), and
4. to strengthen the pavement structure.

This last category will be the primary focus of this report.

Granular overlays have been used by the Washington State Department of Transportation (WSDOT) for about 30 years. Since the mid-1980s, and along with the full implementation of the WSDOT Pavement Management System (WSPMS), WSDOT has been interested in examining the performance of granular overlays. One reason is that WSDOT engineers have found that the performance of this rehabilitation treatment has been better than could be reasonably expected. Further, past practice in Washington state occasionally has required that the preexisting surfacing (often several bituminous surface treatment (BST) layers) be scarified before placement of the crushed rock layer.

One view of why the granular overlays have worked well structurally is that they take advantage of the stress stiffening behavior of granular materials. When a crushed rock layer is subjected to a confining pressure, it stiffens because of the friction between

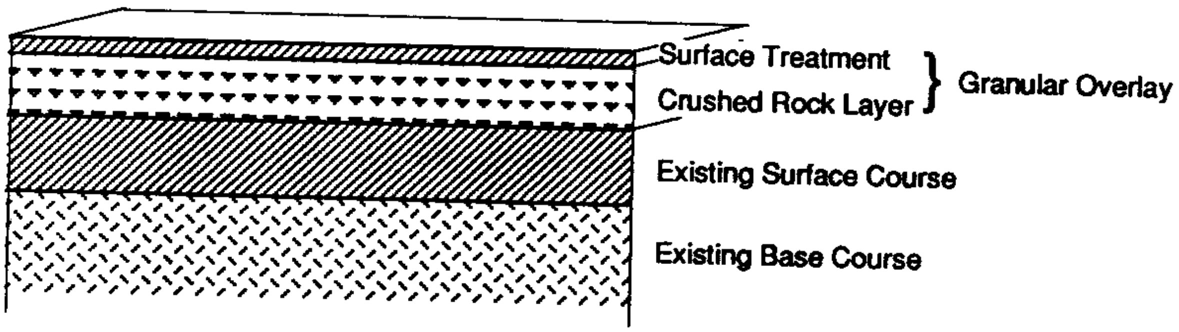


Figure 1. Typical Granular Overlaid Pavement

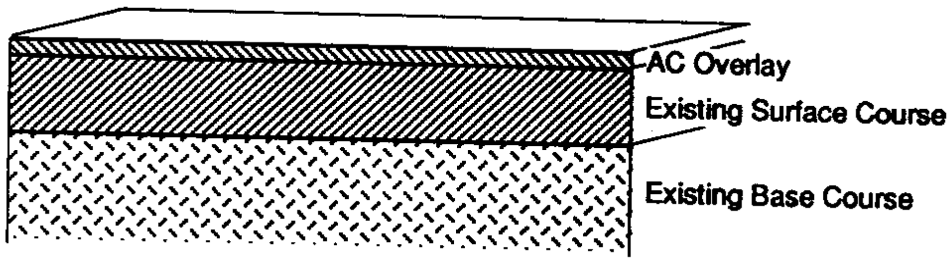


Figure 2. Typical AC Overlaid Pavement

the grains. Since the old pavement surface and the new surfacing confine the crushed rock layer in a granular overlay, traffic loads can provide high confining stresses which, in effect, increase the stiffness of the crushed rock layer.

As the use of the granular overlay increased in Washington state, WSDOT realized that to improve and continue to use granular overlays, it needed to better understand how they worked, where they were appropriate, and how best to design and build them.

In cooperation with WSDOT, two initial studies were undertaken at the University of Washington. (1, 2) The results of these graduate student studies were encouraging. They led WSDOT and the associated Washington State Transportation Center (TRAC) at the University of Washington to enter into an agreement with the Federal Highway Administration to prepare a report overviewing this topic, hence the main report (WA-RD 226.1) and this summary.

METHODOLOGY

This study examined granular overlays in three ways. First, previous research on the behavior of confined crushed rock layers was reviewed. These studies provided information concerning the stiffnesses that have been found in crushed rock layers, the actions that can be taken to improve the crushed rock layer, and the problems that have been encountered in working with confined crushed rock layers. Next, the useable life of the granular overlay was compared with that of other types of pavement resurfacing, including asphalt concrete (AC) overlays and BST. Finally, the granular overlays were field tested to determine their properties and to measure the effect of different designs on their performance.

The rest of this report is organized to cover these topics:

- Literature review
- Survival and performance statistics
- NDT testing and evaluation, and
- Conclusions.

TERMINOLOGY AND LITERATURE REVIEW

TERMINOLOGY

The two types of resurfacings generally used to seal and/or strengthen flexible pavements are BST and AC overlays. Because BST and AC surfacings will be mentioned frequently, a clear statement of those terms follows.

Bituminous Surface Treatments. A BST is composed of alternating layers of asphalt emulsion or cut-back asphalt and rock. The asphalt binder is sprayed onto the road, and the stone is spread across and compacted into it. These layers are repeated as often as required. WSDOT generally uses two- or three-layer BSTs. Since the BST is a thin layer and somewhat flexible, it does not add much strength to the pavement. Instead, it serves to seal the road and to add a new wearing surface. Additionally, BSTs are relatively inexpensive to construct ($1.00 to $2.00/sy). Because the construction technique does not require a nearby hot mix plant and the construction costs are low, this type of resurfacing is frequently used on low volume rural roads.

Asphalt Concrete Overlays. An AC overlay consists of a mixture of asphalt cement and stone. The ingredients are heated, then mixed together in a batch plant and trucked to the project. The AC is then placed on the road and compacted.

Once the AC has cooled, it provides a relatively stiff, new wearing surface to the road. Although this technique is effective in strengthening the pavement system, it is also substantially more expensive than a BST surfacing (by about a factor of 3).

GRANULAR OVERLAYS

The behavior of granular overlays depends upon the condition of the crushed rock layer. Both the surface and the old pavement serve to protect this layer and to confine it. The crushed rock layer can provide much of the "strength" of the overlay. When crushed rock is used as a base course it generally has a modulus of elasticity of about 15 to 30 ksi

(100 to 200 MPa). (3) When it is subjected to a confining pressure of 125 psi (0.9 MPa), its modulus of elasticity can exceed 100 ksi (690 MPa) or more. (4)

When a granular material is subjected to a confining pressure, interparticle friction causes the material to behave in a stiffer manner. The degree of this stress sensitivity depends on the roughness of the particles, the percentage of fine particles, and the moisture content of the material. In general, the stress stiffening will follow Equation A:

$$E = K_1 \theta^{K_2} \qquad \text{(Equation A)}$$

where
E = modulus of elasticity (psi),
K_1, K_2 = constants,
θ = $\sigma_1 + \sigma_2 + \sigma_3$ = bulk stress, and
$\sigma_1, \sigma_2, \sigma_3$ = principal stresses.

A study by the University of Washington and WSDOT found that the crushed rock WSDOT uses (crushed surfacing top and base course) has the "typical" values for the constants shown in Table 1. (5) The WSDOT laboratory tests were conducted at bulk stresses ranging from 4 to 28 psi.

In a traditional pavement system, the confining stresses on the crushed rock base depend on a number of factors, including the stiffness of the subgrade. Since the granular overlay is sandwiched between two "stiff" pavement layers, it will be subjected to higher confining pressures.

Table 1. "Typical" Values for the Constants K_1 and K_2 Based on a Laboratory Analysis (5) Compared with AASHTO Values (3)

Constants	Mean (WSDOT)	Standard Deviation (WSDOT)	AASHTO Range
K_1	8500	2300	2000 - 4000
K_2	0.375	0.067	0.5 - 0.7

Equivalency Factors

The stiffness of a granular overlay is provided largely by the crushed rock layer (assuming that the surfacing is relatively thin). One method for comparing granular overlays and AC overlays is to determine the thickness of a granular overlay that would provide the same "life" as a thickness of AC overlay. This is the technique used by Sibal (2) and Deoja (1) in their studies of granular overlays.

Sibal used two elastic layer programs to model the behaviors of AC and granular overlays: ELSYM5 and EVERSTR. (2) The ELSYM5 program treats all layers as linearly elastic. EVERSTR treats the granular layers as nonlinear. Sibal determined the thicknesses of the crushed rock layer in a granular overlay that would provide the same pavement performance as different thicknesses of AC. In both the granular and AC overlay analyses, he varied the moduli of the subgrade and the granular overlay crushed rock layer.

Sibal considered three modes of failure: fatigue cracking of the surface, fatigue cracking of the preexisting pavement surface layer, and rutting. He determined which of the three modes of failure was critical for each model and used the corresponding number as the number of loads that would cause failure for the pavement. Finally, he compared the number of loads that would cause failure for each of the models to determine the equivalent thicknesses of granular overlays and AC overlays. The 1.0 in. of AC on top of the crushed rock layer was not calculated in the equivalency factor. For example, if a 4-in. AC overlay was to be converted to a granular overlay with an <u>equivalency factor of 1.70,</u> then the conversion would be as follows:

4-in. AC = 1.0-in. AC + 3.0 in. of AC

= 1.0-in. AC + 3.0 x 1.70 of crushed rock

= 1.0-in. AC + 5.1 in. of crushed rock (or 6.1 in. total thickness)

Sibal's analyses are illustrated in Figure 3. His results suggest equivalency factors of about 2.0 for the "stiffer" crushed rock moduli.

The Crushed Rock Layer in Inverted Pavements

A series of South African studies investigated the effects of different parameters on the behavior of the crushed rock layer in inverted pavements. These studies provided verification that the modulus of the crushed rock layer can be quite high and offered insight into improved designs for this layer.

The South African system of classifying granular pavement materials is shown in Figure 4. The material generally used in the crushed rock layer was the G1 material, although G2 was used on occasion.

In most cases, multidepth deflectometers (MDD) were placed in the pavement, and the pavement was loaded with a heavy-vehicle simulator (HVS). This system offered the advantage of directly measuring the response of the pavement layers to loading. The HVS applied 4.5- to 22.5-kip (20 to 100 kN) dual-wheel loads (or single axle loads ranging from 9,000 to 45,000 lbs (40 to 200 kN)). Measurements included surface deflections, MDD deflections, road surface profile, temperature, precipitation, and depth to the water table. The testing was conducted on each road for two to six months. (6)

An early study compared the effects of the different loadings on two "traditional," thin pavements and two inverted pavements. (4) The test sections were all in the Transvaal Province (see map Figure 5) and had pavement profiles as shown in Figures 6 and 7. The major differences between the light and inverted pavements were the use of higher quality crushed rock layers beneath the surfacing and cement stabilized subbases below the crushed rock layer.

In every case, following HVS testing, the base course that was supported by the cemented subbase had significantly higher bulk stresses and higher moduli of elasticity than did the light pavements. Most notable were the differences in quality among the base courses in the four roads. In both of the light pavements the base consisted of natural gravels (classified G5), whereas the inverted pavement base consisted of graded crushed

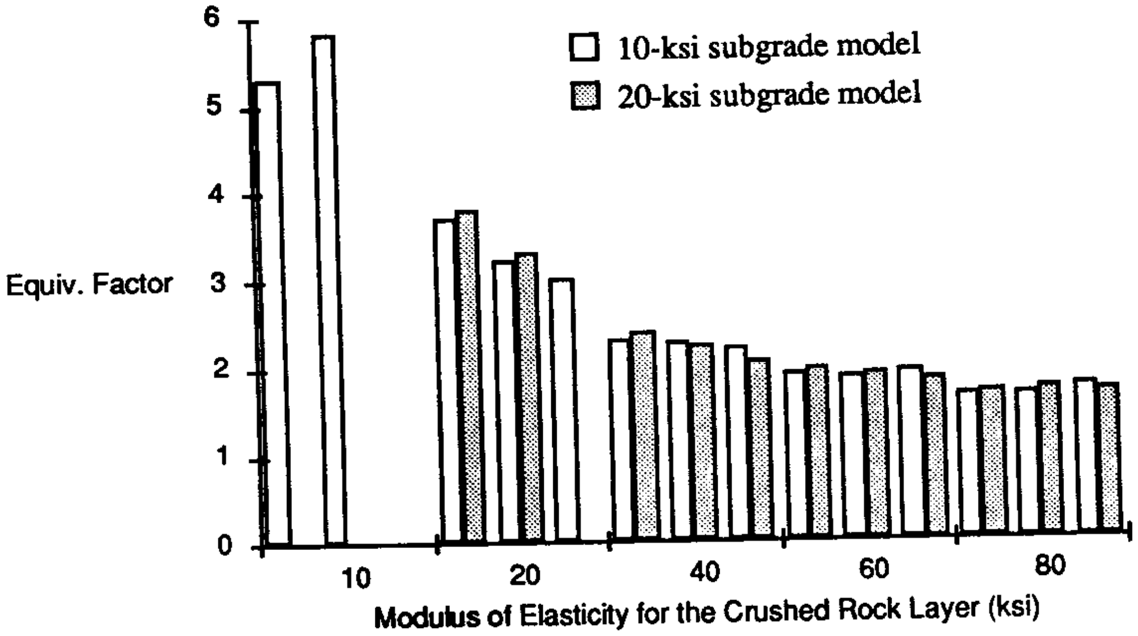


Figure 3. The Equivalency Factors versus the Modulus of Elasticity for the Crushed Rock Layer Based on Sibal's Calculations (2)

Symbol	Code	Material	Abbreviated Specifications
	G1	Graded crushed stone	Dense-graded unweathered crushed stone. Max. size 1.5 in. (37.5 mm) G1A: 86 - 88% of apparent density. G1B: 98% modified AASHTO (density lower than G1A)
	G2	Graded crushed stone	Dense - graded stone and soil binder. Max. size 1.5 in. (37.5 mm), min. 98% modified AASHTO
	G4	Natural gravel	CBR ≥ 80, PI ≤ 6
	G5	Natural gravel	CBR ≥ 45, PI ≤ 10 to 15, depending on grading; Max. size 2.5 in. (63 mm)
	G6	Natural gravel	CRB ≥ 25, Max. size ≤ 2/3 layer thickness
	G7	Gravel - soil	CRB ≥ 15, Max. size ≤ 2/3 layer thickness
	G8	Gravel - soil	CBR ≥ 10 at in situ density
	G9	Gravel - soil	CBR ≥ 7 at in situ density
	C3	Cemented natural gravel	Unconfined Compressive Strength 220 - 440 psi (1.5 - 3.0 MPa) at 100% mod. AASHTO; Max. size 2.5 in. (63 mm)
	C4	Cemented natural gravel	Unconfined Compressive Strength 110 - 220 psi (0.75 - 1.5 MPa) at 100% mod. AASHTO; Max. size 2.5 in. (63 mm)
	C5	Treated natural gravel	Modified mainly for Atterberg limits

Figure 4. South African Pavement Material Classifications (4, 6)

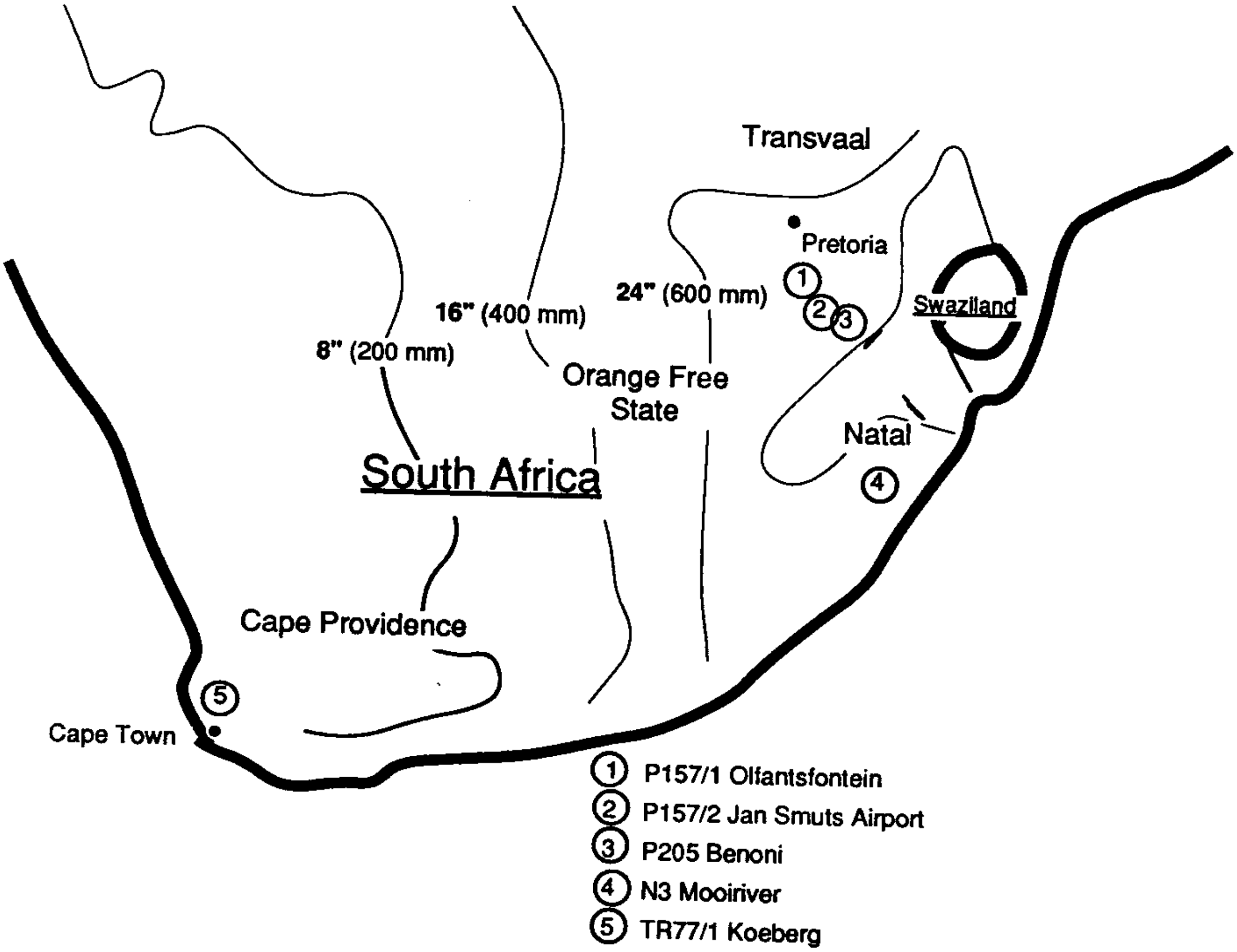


Figure 5. Map of South Africa Showing the Average Annual Precipitation (7) and Location of Five Test Sites (6)

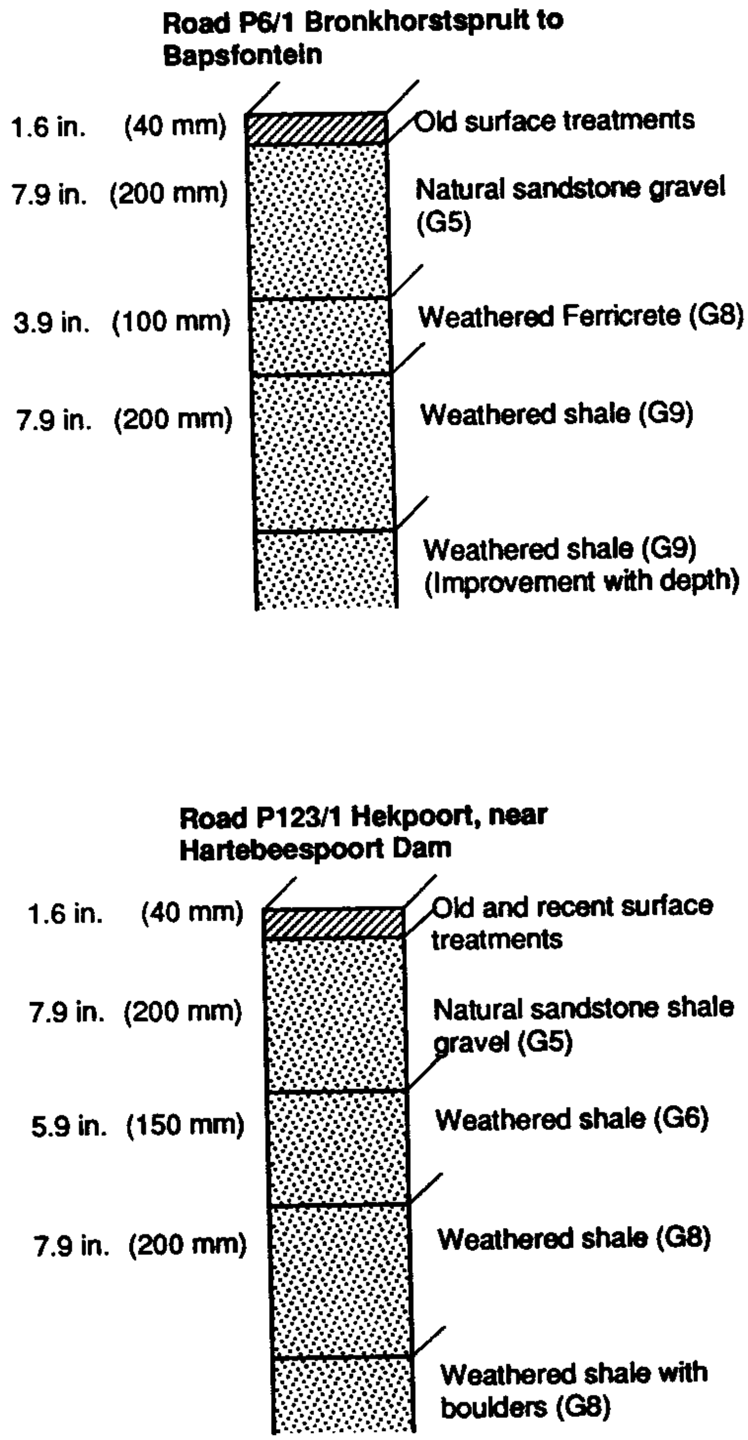
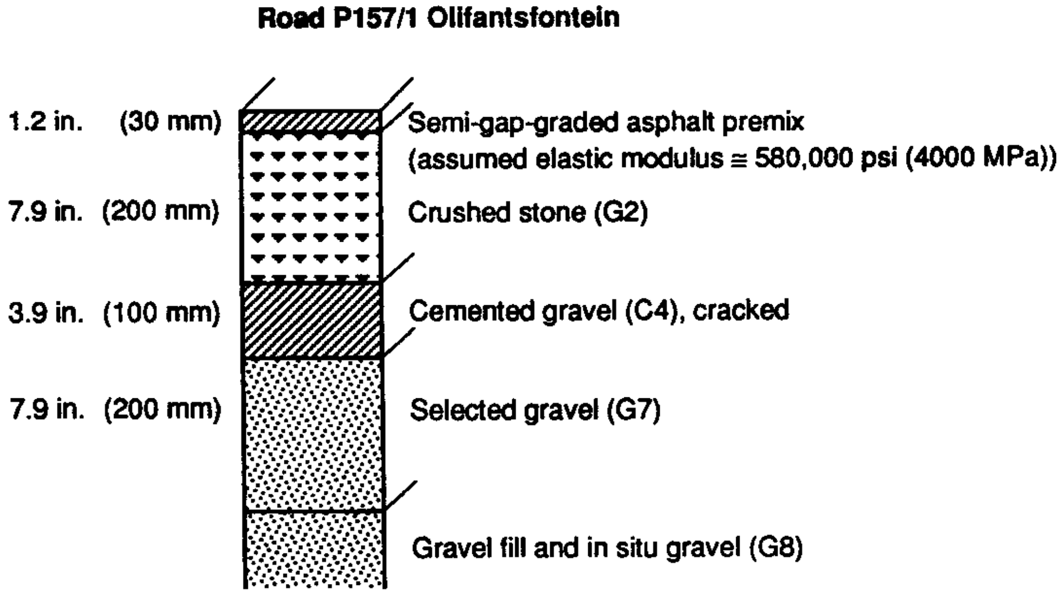


Figure 6. The Light Pavements, Road P 6/1 and Road P 123/1, Tested by the Heavy Vehicle Simulator (4)



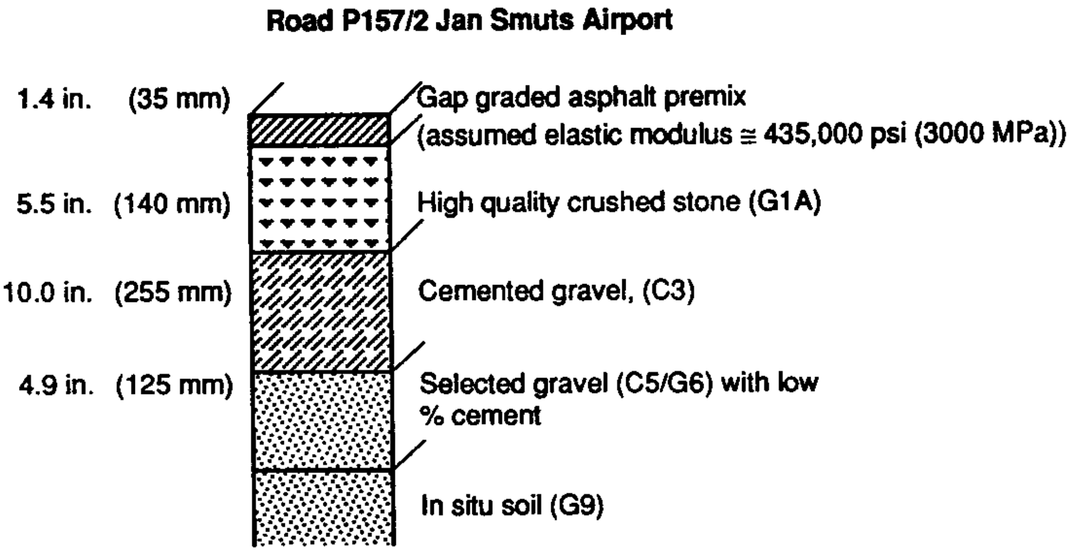


Figure 7. Inverted Pavements, Road P157/1 and P157/2, Tested with the Heavy Vehicle Simulator (4)

stone (classified G1 and G2). This difference accounts for much of the difference in the moduli between the light and inverted pavement bases. The moduli were not at all similar for the light pavements (Roads P6/1 and P123/1) or Road P157/2. The moduli were somewhat similar for Road P157/1.

Gradation of Granular Material. As Horak discovered, gradation specifications are very important for achieving the high densities required for optimal performance of confined crushed rock layers. (8) Horak published a paper that dealt with the effects of tightening the grading specifications beyond those normally required for the G1 (South African) base. (9) Although they mentioned the importance of the strength, durability, shape, and Atterberg limits of the aggregate, the report focused on changes to the specifications to produce a better compacted base.

For convenience, the gradation bands for the South African G1 material, as well as somewhat similar gradations from AASHTO M147 (Gradings A and B) and WSDOT (Crushed Surfacing Top Course and Base Course), are listed in Table 2. The gradation band for the G1 material was obtained directly from a figure in Horak. (9) It appears that the most similar gradations to the G1 are AASHTO Grading B and the WSDOT Crushed Surfacing Base Course. (Note that the majority of granular overlays constructed by WSDOT to date have used the Crushed Surfacing Top Course grading.)

The base course Horak worked with was compacted to 99 to 103 percent of modified AASHTO and had a gradation that fell within the specifications for a G1 base, but with the additional requirements listed below. (9) They found that the greater effort required to set up the crusher and obtain the correct gradation was more than offset by the increased ease in compacting the material to a higher density. These additional requirements were as follows:

1. 40 to 45 percent had to pass No. 4 (4.75 mm) sieve
2. 7 to 9 percent had to pass No. 200 (0.075 mm) sieve

Table 2. Gradation Bands for Various Crushed Rock Specifications

Sieve Designation		Percent Passing				
		South African G1*	AASHTO M147-65		WSDOT 9-03.9(3) Crushed Surfacing	
Standard	mm		Grading A	Grading B	Top Course	Base Course
2 in.	50	100	100	100		
1 1/4 in.	32	93 - 97				100
1 in.	25	82 - 92		75 - 95		
3/4 in.	19	72 - 85				
5/8 in.	15.9	64 - 78			100	50 - 80
3/8 in.	9.5	50 - 67	30 - 65	40 - 75		
1/4 in.	6.35	40 - 57			55 - 75	30 - 50
No. 4	4.75	35 - 52	25 - 55	30 - 60		
No. 10	2.00	23 - 39	15 - 40	20 - 45		
No. 30	0.600	14 - 26				
No. 40	0.425	11 - 24	8 - 20	15 - 30	8 - 24	3 - 18
No. 200	0.075	5 - 12	2 - 8	5 - 20	10 max	7.5 max.

* South African G1 grading taken from plotted gradation band (7)

3. A coarse sand ratio of 30 to 50 percent. Coarse sand ratio is defined as the ratio of the fraction that passes the No. 10 (2.00 mm) sieve divided by the fraction that passes the No. 40 (0.425 mm) sieve.

Construction of the Crushed Rock Layer. As was previously stated, the crushed rock layer is stiffer and more durable if it is well constructed. The compaction and integrity of the crushed rock layer is very important. The material for the crushed rock layer must be durable. As was mentioned by Horak, the easiest way to obtain the highest compaction is to use an optimum gradation. (8) Interviews during 1990 with WSDOT project engineers on granular overlay construction projects indicated that the moisture content is also important. (10, 11, 12) Because the crushed rock is spread in a thin layer, the moisture from the rock tends to evaporate rapidly. Therefore, the surface must be sprayed with water to maintain the optimal moisture content.

A significant problem that WSDOT has encountered in the construction of the granular overlay is traffic. When there is no possible diversion, the traffic has to pass over the crushed rock layer during construction. This frequently causes washboarding. Therefore, the granular surface must be rebladed immediately before the surface layer is placed. WSDOT currently requires that its crushed surfacing have at least one fractured face. (13) Washboarding might be reduced if this requirement were raised.

Improvement in BST Construction

A recent WSDOT study examined the effects that construction practices have on the problems associated with BSTs. (14) The main problems that were investigated included flushing of excess asphalt, windshield damage due to loose rock, and aggregate loss due to poor embedment. Through a review of the use of BSTs in other western states and an examination of BST construction projects, a series of design and construction guidelines were developed.

The guidelines stressed different methods for assuring that the chips were 50 to 70 percent embedded in the asphalt binder. These techniques included using the minimum number of chips and amount of emulsion possible, requiring that the chips be applied within one minute after the asphalt emulsion has been applied, and using choke stones and fog seals. These guidelines led to a significant reduction in the problems of aggregate loss and windshield damage.

The study also recommended several guidelines for the proper choice of roads to be overlaid with BSTs. The report suggested that the BST be applied only to roads that were not considered a high traffic risk, i.e., roads with average daily traffic (ADT) counts in excess of 5,000. If a BST surface was used on granular overlays, these same limitations were applicable. (WSDOT mostly requires BSTs on routes with ADTs of 2,000 or less and discourages the use of BSTs on routes with ADTs of 5,000 or more.)

Existing Pavement

Although granular overlays are good at reducing the rate of reflective cracking, they are also sensitive to moisture infiltration through breaks in the existing pavement surface. Infiltration of moisture lowers the stress-sensitivity of the crushed rock layer.

To circumvent the problem of an inconsistent base for the overlay, the highway department in Zimbabwe rips and spreads the existing base and shoulder, then stabilizes with two percent cement or lime (if required) and recompacts it. In this manner, the granular overlay is assured of having a solid base. (15) Normally, all that is required is to patch any holes and seal any cracks in the existing pavement surface.

Other Advantages and Limitations

Insulation. An advantage of the granular overlay is that it protects the existing pavement from daily extremes in temperature. This is important during hot summer months and in tropical countries where pavement surface temperatures can approach and exceed 160°F (70°C). (15) Because the stiffness of the pavement is directly proportional to the temperature, the hotter the pavement is, the less stiff it is. Reduced stiffness increases the potential for rutting and leads to early pavement failure. However, the crushed rock layer is unaffected, thus providing a relatively stiff layer.

Increased Frost Resistance. Because frost heave and thaw weakening problems are often a result of subgrade freezing, if the subgrade is insulated from the cold, then these problems are reduced. Several states, including Alaska, Iowa, Oregon and Washington, use frost protection in their pavement thickness design calculations. (16) Most of these states design the pavement thickness to be at least 50 percent of the total expected frost depth. In this manner, the subgrade is at least partially insulated against frost.

In Washington state, designing the overall pavement depth to be equal to 50 percent of the total frost depth has worked well for controlling all but the most severe frost problems. (17) Unfortunately, many of the state's low volume rural roads were built in the

1930s to 1950s before this design procedure was adopted. These roads frequently consist of only a thin BST over 6 to 9 in. (150 to 225 mm) of base course when the frost design thicknesses are 15 to 24 in. (380 to 610 mm). Rehabilitating these roads typically means adding a granular overlay with a minimum of 4.2 in. (107 mm) of crushed rock.

To prevent the crushed rock layer from contributing to frost heave, the amount of material that passes the No. 200 sieve (0.075 mm) may have to be limited. Researchers have observed that the fines content of soils is an important indicator of frost-susceptible soils. (18) The different pavement agencies surveyed specified that the maximum percentage of material that passed the No. 200 sieve be 5 to 15 percent. The lower range of this specification is lower than the range suggested by Horak for obtaining the maximum compaction. (9)

Change in Road Geometry. By adding 3 to 6 in. (75 to 150 mm) to the overall pavement structure, the granular overlay can alter the road geometry. It can be used to increase drainage, to improve the road profile, and to level off inconsistencies in the pavement. This additional height makes it unusable in areas where the road geometry is restricted by curb height or other considerations.

Resistance to Shear. Little information could be found concerning the ability of confined crushed rock layers to resist shear. However, this is a potential problem. Although granular materials are very strong in compression, they have little resistance to tension. When subjected to a high confining pressure, the crushed rock particles can distort, be crushed, shift, roll, or slide. The amount of movement caused by any of these actions is directly proportional to the confining pressure. If the shear stress becomes sufficiently large, the combined movement from these actions will result in a shear failure. (19) No calculations were found to predict the effective shear stress at failure.

Design of Granular Overlays

There are two basic techniques for designing granular overlays. One is based on mechanistic calculations. The studies done by Sibal (2) and Deoja (1) are such examples. Another technique is based on practical construction considerations.

Mechanistic Design. Mechanistic design means designing the appropriate thickness of crushed rock to provide the necessary strength to ensure the desired life for the overlay. Designing an overlay with the mechanistic approach is similar to designing an AC overlay. Because much research has been done to determine the appropriate thicknesses of AC for different road conditions, one approach to designing granular overlays is to design an AC overlay and determine the equivalent thickness for a granular overlay. These equivalency factors were investigated by Sibal. (2)

One Thickness Approach. The second design approach is to use the maximum thickness of granular material that can be easily compacted in one lift. This suggestion was offered in a report by Maree, in which the authors found that a crushed rock layer 12.6 in. (320 mm) thick did not perform significantly better than one 5.5 in. (140 mm) thick (both crushed rock layers were on cement stabilized subbases). (6) Maree concluded that the crushed rock layer need not exceed the maximum thickness that can be placed and compacted in one lift (6 in. (150 mm)).

A study by Otte and Monismith produced similar conclusions. (20) They used a layer elastic program, PSAD2A, to model the behavior of several inverted pavement structures. The computations were made for 9.0 kip (40 kN) dual wheels, 13.0 in. (330 mm) apart. The pavement that was simulated was an inverted pavement with a 1.4-in. (35-mm) BST surface. The thicknesses of the crushed rock layer and the cement stabilized base and subbase were varied over a wide range of thicknesses.

The authors found that the primary stresses in an inverted pavement were on the surface course and cement stabilized layers. They found that because of stress-stiffening, as the thickness of the crushed rock layer increased from 5 to 20 in. (125 to 500 mm), the equivalent elastic modulus of the granular base declined about 30 percent. Otte and Monismith recommended the following. (20)

1. The bituminous surfacing for "inverted" pavement designs should not exceed 1.2 to 1.4 in. (30 to 35 mm).
2. For typical highway traffic loads, the granular layer should have a thickness of about 5.0 to 6.0 in. (125 to 150 mm).
3. The cement stabilized layers supporting the granular layer should be
   a. two layers, each 6 in. (150 mm) thick, if the subgrade has a CBR of 15 or better, or
   b. one layer, 6 in. (150 mm) thick, for light traffic (rural).

The one thickness approach is also used in Zimbabwe. (15) The highway department in Zimbabwe has found that the practical range for the construction of the crushed rock layer in an overlay is 5 to 6 in. (120 to 150 mm). Thinner layers tend to shear under a roller. If a thicker layer is needed, then the road probably needs to be reconstructed.

In Washington state, granular overlays are generally built with thicknesses of 3 to 6 in. (most with a 4.2-in. (107-mm) thickness).

Costs

Some typical project specific costs for granular overlays with both AC and BST surfaces are shown in Table 3 and Figure 8. In general, granular overlays surfaced with AC are about twice as expensive as those surfaced with a BST (about $7.00/sy for AC versus $3.50/sy for BST).

Table 3. Summary of Total Construction Cost of Granular Overlays Based on WSDOT Projects

Route No.	Year Constructed	Length		Surface Course			Crushed Rock Thickness		Total Costs	
					Thickness					
		miles	km	Type	in.	mm	in.	mm	$/sy	$/m^2
281	1985	11.34	18.25	AC	2.4	60	3.0	75	7.54	8.97
261	1987	8.20	13.19	AC	2.4	60	3.0	75	5.59	6.65
231	1985	8.64	13.90	BST	0.7	18	4.2	105	3.25	3.87
24	1986	13.62	21.91	BST	0.7	18	3.6	90	2.52	3.00
25	1987	14.60	23.49	BST	0.7	18	6.0	150	3.50	4.16
155	1989	12.96	20.85	BST	1.1	28	3.6	90	3.07	3.65
20	1989	7.95	12.79	BST	1.1	28	4.2	105	4.08	4.49

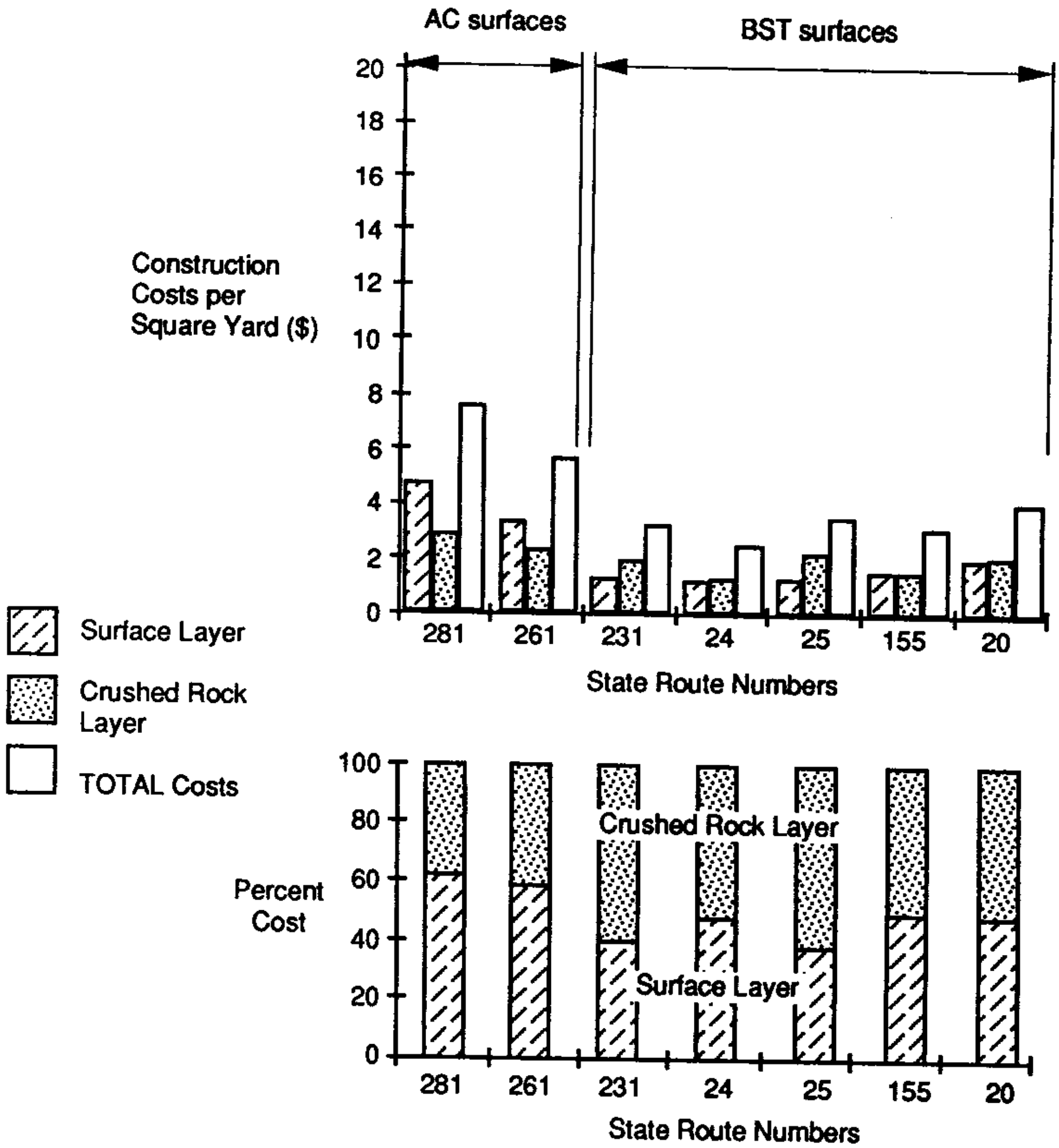


Figure 8. Comparison of the Cost of the Different Layers in a Granular Overlay Based on WSDOT Cost Estimates

SURVIVAL AND PERFORMANCE STATISTICS

INTRODUCTION

The survival life of a pavement is the amount of time between a pavement's construction and its resurfacing. The performance period is the amount of time from its initial construction to the time it reaches a minimally acceptable level. Both survival lives and performance periods provide valuable estimates of pavement "life." In this portion of the report, the survival lives and performance periods of AC overlays, BST resurfacings, and granular overlays are estimated and compared.

This part of the study considered only the ages of the different types of surfaces. Other factors, such as traffic loadings, weather conditions, and soil support, affect the amount of time that a pavement lasts, but these were not considered. Since the study was conducted within a small geographic area, the weather and subgrade effects were assumed to be constant for all roads. The effects of higher traffic loadings were expected to be somewhat cancelled by the thickness of the resurfacing.

An important point to be noted was the difference in characteristics of the roads overlaid by the different techniques. In general, roads with AC surfaces had higher traffic counts than roads with BST surfaces. Additionally, granular overlays were often used as a method for repairing badly distressed roads. Most roads that received a granular overlay had suffered from either thermal cracking or roughness problems and therefore had required special treatment.

In comparing the survival times and performance periods, it is important to remember that the sets of data did not come from the same time period. Since the performance life equations were only calculated for the present road surface, they represented only resurfacings built since the late 1970s. The data for calculating the survival lives, on the other hand, were equally spread from the 1980s to the 1960s, with some dating as far back as the 1940s. Because of the problems previously mentioned with the "old" survival life data, the old information tended to increase the average survival life slightly.

Source of Data

The source of data for this analysis was the WSDOT Pavement Management System.(21) The WSPMS contains records of work done on the roads and the pavement condition analyses. Data from the WSPMS were spot checked against as-built plans and pavement conditions and were found to be accurate.

The pavement condition ratings (PCR) were collected from surveys that WSDOT had conducted at least every other year since 1969. (22) The rating was on a scale with a maximum of 100 and an open-ended bottom. These ratings roughly corresponded to the AASHTO Present Serviceability Index (PSI), with a PCR of 100 corresponding to a PSI of 5.0 and a PCR of 40 corresponding to a PSI of 3.0. The road analyzed was divided into 1-mile (1.6-km) sections, and each section was analyzed separately. The ratings for the sections were then averaged together to give an overall rating for the road.

WSDOT has developed a program entitled Management Information Data Access Linkage (MIDAL) to manipulate the information in the WSPMS. This system takes the PCRs for a road section and attempts to develop a pavement performance prediction equation. If there are enough PCRs, the program calculates a regression curve for the data (PCR vs. age). If the root mean square error (RMSE) of this curve is high, indicating a poor data fit, or if the equation does not follow the expected trend, then the program modifies the equation to better represent the expected trend. If the pavement surface is too new to have enough points to define a curve, then the program adapts the standard (default) pavement equations to fit the data. Only equations that were based entirely on the actual PCR data and on road segments that were more than 5 years old were used in this analysis (i.e., no default equations were used). Of the over 600 road sections examined, only 67 had useable equations.

In Figure 9, the regression equation for a road with a granular overlay built in 1983 is plotted. The predicted performance equation for this road section is as follows:

$$PCR = 98.4 - 0.682\ (Age)^{2.00} \qquad \text{(Equation B)}$$

where PCR = Pavement Condition Rating

Age = Time (years) since granular overlay placed.

For illustration purposes, the predicted performance periods for the above performance equation and PCR levels are shown in Table 4.

The estimation of survival lives and performance periods was restricted to WSDOT Districts 2 and 6 (Figure 10). These two districts are generally rural areas where the topography ranges from mountainous to rolling hills. The average annual precipitation is 16.7 in. (424 mm) in Spokane (23), but the area has severe frost in the winter. These are also the two districts that contain the majority of roads with granular overlays.

The WSPMS was searched to locate all roads containing BST, AC, and granular resurfacings. First, the actual survival time for the different layers of pavement surfaces was calculated by subtracting consecutive resurfacing construction years from the previous years. Next, PCR data on the most recent resurfacing were examined to determine whether the regression equation in the model represented a "true" regression equation. Finally, the survival lives and the performance periods were compared for each type of resurfacing and among the types of resurfacings.

Table 4. Performance Periods for SR21A Milepost 44.73 to 46.95

Final PCR	Initial Year	Final Year	Performance Period
40	1983	1992	9 years
20	1983	1994	11 years
0	1983	1995	12 years

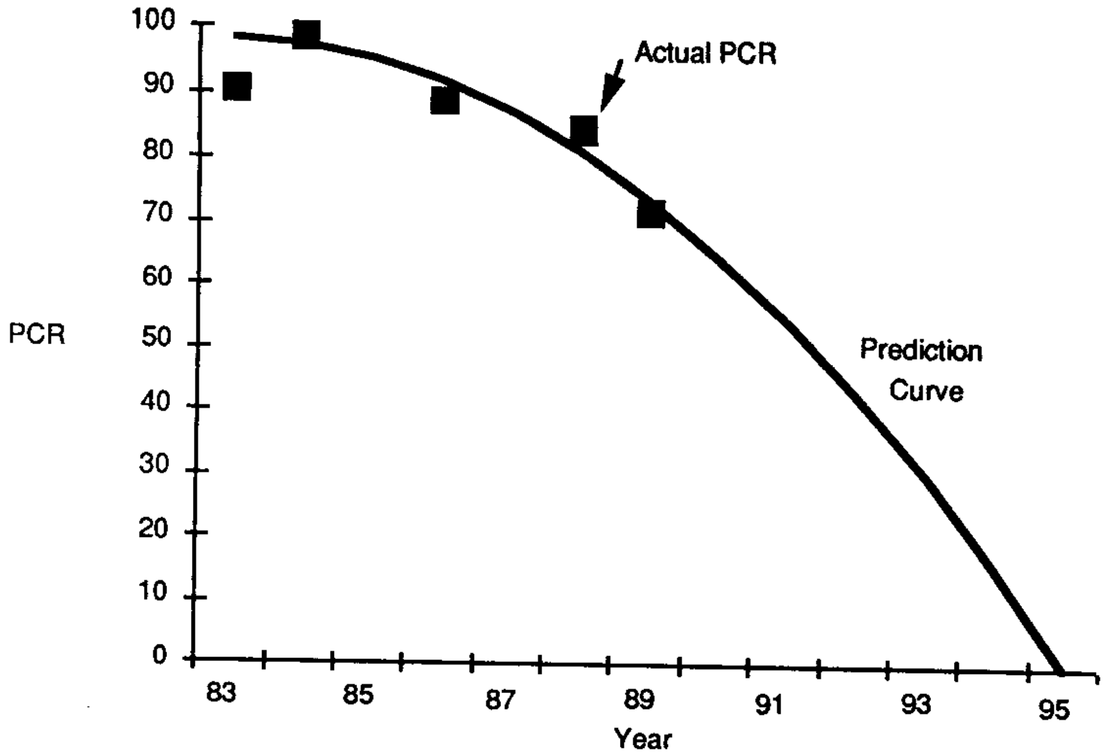


Figure 9. Plot of Regression Equation for SR21A Milepost 44.73 to 46.95

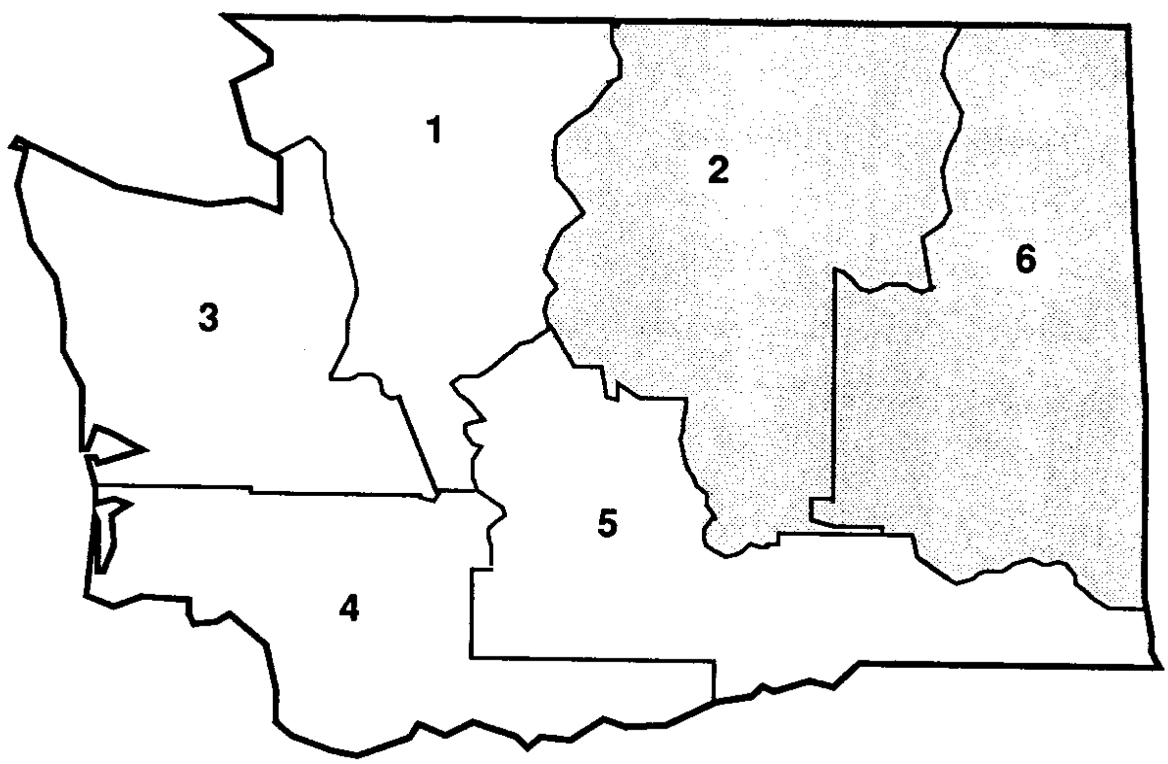


Figure 10. Map of the Granular Overlay Study Area and WSDOT District Boundaries

BITUMINOUS SURFACE TREATMENTS

The actual survival time for BST resurfacings is shown in Figure 11. The figure is similar to a skewed, normal distribution, with the exception of the peaks at 5, 7, 10 and 13 years. The long tail to the right was the result of special cases. All of the survival lives over 26 years were from resurfacings in the 1930s.

Next, the PCR equations were examined to determine the performance periods. Although over 200 road sections were analyzed, only 21 had useable regression equations. These results are summarized in Table 5.

The differences between the actual survival time and predicted performance periods have already been discussed. If the historical differences are ignored and the other effects are assumed to balance out, then a comparison of the values for the actual survival times and the performance periods reveals that WSDOT resurfaces BST roads when their PCR is between 20 and 0.

ASPHALT CONCRETE OVERLAYS

The actual survival times for AC overlays are shown in Figure 12. The figure shows a basically normal distribution. The statistical summary of these data is shown in Table 6.

The survival times based upon AC overlays with thicknesses of less than 1.2 in. (30 mm) were separated out. According to the 1988 WSDOT specifications, only AC overlays thicker than 1.2 in (30 mm) are subject to compaction control. It is evident from Figure 12 that the values from the thin overlays represented the majority of the data in the lower range of survival times.

There were also a significant number of survival times of 5 years and less. If only the survival times of AC overlays thicker than 1.2 in. (30 mm) and lasting more than 5 years are considered, the average survival life increases to 11.5 years.

The predicted performance periods are summarized in Table 7.

Table 5. Basic Statistics for the Performance Periods of BSTs
(Based on 21 Data Points)

PCR Level	Median	Mean	Std. Dev
PCR 40	6.1	7.0	2.5
PCR 20	7.4	8.2	2.4
PCR 0	8.7	9.3	2.4

Table 6. Basic Statistics for the Survival Times of AC Overlays in Districts 2 and 6

AC Thicknesses	Median	Mean	Std. Dev	Count
Overall	10	9.4	4.2	328
< 1.2 in. (30 mm)	8	8.7	2.8	91
> 1.2 in. (30 mm)	10	9.7	4.5	237

Table 7. Statistical Values for the Performance Periods for AC Overlays in Districts 2 and 6. Statistics are Based on 29 Data Points.

	Median	Mean	Std. Dev
PCR 40	10.6	10.2	1.67
PCR 20	11.4	11.3	1.69
PCR 0	12.2	12.1	1.81

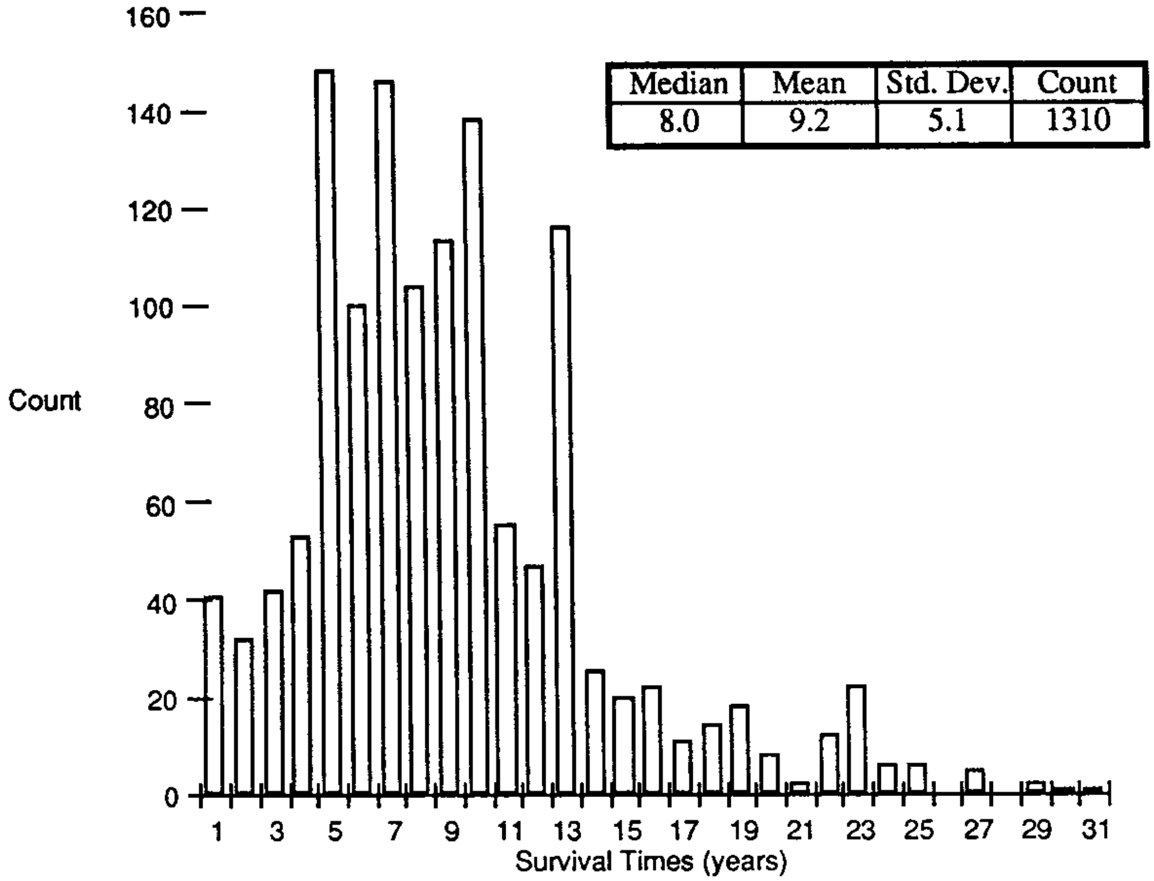
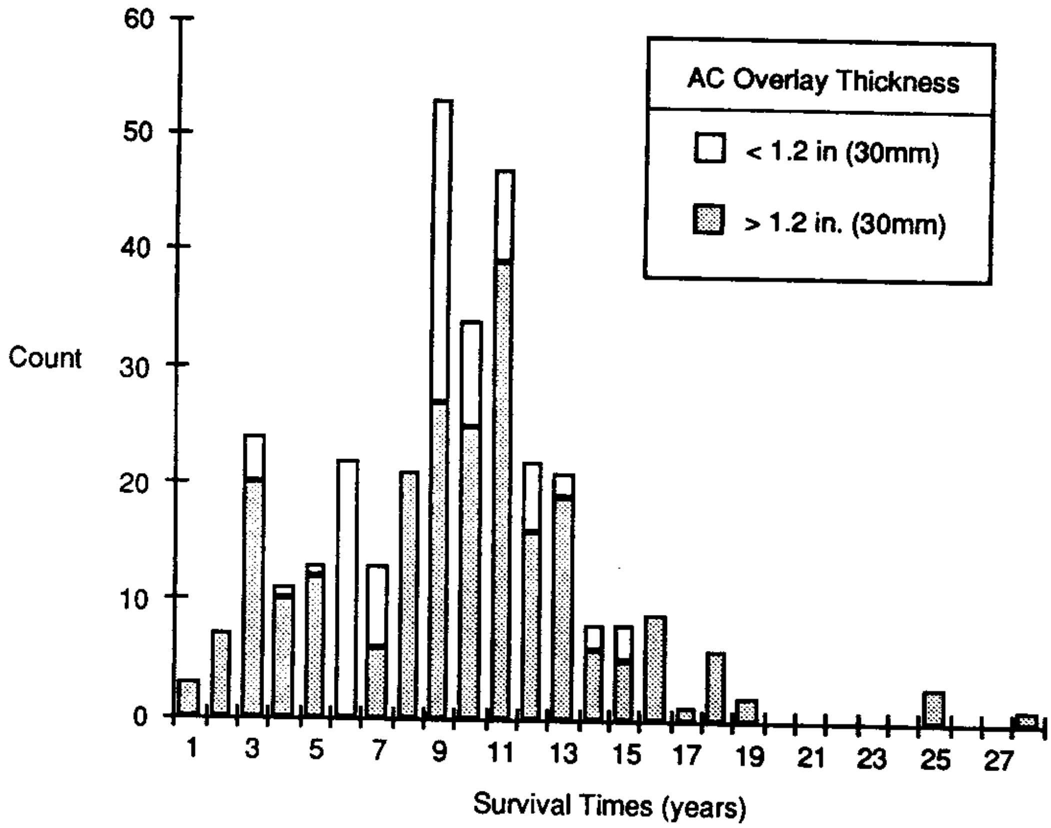


Figure 11. Survival Times for BSTs in Districts 2 and 6



Thickness	Median	Mean	Std. Dev.	Count
<1.2"	8	8.7	2.8	91
>1.2"	10	9.7	4.5	237

Figure 12. Survival Times for AC Overlays in Districts 2 and 6

GRANULAR OVERLAY

Unlike the other two resurfacings (or overlays) that were examined, there are not a large number of granular overlays on the WSDOT route system. Additionally, because granular overlays had only been used with greater frequency since the mid-1980s, few data were available concerning survival times. Therefore, the authors did not have enough data with which to analyze the actual survival times for granular overlays. This left the examination of the performance periods. To increase the number of available data, all roads with granular overlays in eastern Washington were examined, rather than just those in Districts 2 and 6. The data are summarized in Table 8.

PERFORMANCE SUMMARY

Although both methods used for calculating the useable life of the resurfacings have uncertainties, the two methods together provide reasonable estimates of useable life. The average times are shown in Table 9.

Table 8. Basic Statistics for the Performance Periods of Granular Overlays Based on 17 Data Points.

	All Surfaces			Only BST Surfaces	Only AC Surfaces
PCR Level	Median	Mean	Std. Dev	Mean	Mean
PCR 40	8.7	9.2	1.8	7.5	9.5
PCR 20	10.7	10.8	2.1	8.3	11.3
PCR 0	12.2	12.2	2.6	11.0	11.8

Table 9. Survival Times and Predicted Performance Periods for the Resurfacings.

Type of Resurfacing	Actual Survival Life	Predicted Performance Period		
		PCR 40	PCR 20	PCR 0
BST Only	8	6.1	7.4	8.7
GO w/BST surface	-	7.5	8.3	11.0
AC Only (> 1.2 in. (30 mm)	10	10.6	11.4	12.2
GO w/AC surface*	-	9.5	11.3	11.8

*GO = Granular Overlay

A comparison of BST resurfacings to granular overlays with BST surfaces shows that the difference in predicted performance is relatively small (7 percent) at a PCR of 40 but increases (18 percent) at a PCR of 0 (the granular overlays in both cases last longer than a simple BST). However, for AC, the granular overlays surfaced with AC do not last as long as a conventional AC overlay (7 percent less at a PCR of 40 and 2 percent less at a PCR of 0). Again, it is interesting that the granular layer is of extra benefit if the pavement structure has deteriorated significantly (say to a PCR of 0).

Although these comparisons do not show that the granular overlay performs significantly better than the resurfacings without the crushed rock layer, this result is likely due to the current use of the granular overlay. As previously stated, granular overlays are generally used to repair a pavement structure with significant distresses. If the granular overlays had been used on pavements in better condition, these comparisons would have likely produced more favorable results concerning the granular overlay.

NDT TESTING AND EVALUATION

To assess the actual performance of roads with granular overlays, over 50 centerline miles (80 km) of roads were tested with a Dynatest Falling Weight Deflectometer (FWD) Model 8000. These roads were located throughout WSDOT Districts 2 and 6 and had a variety of structures, ages, and conditions. The tests were designed to provide evidence about the comparative performance and stiffness of granular overlays.

SELECTION OF TEST SECTIONS

All of the selected roads were in rural eastern Washington. Traffic on these roads is mostly local, with occasional long haul trucks. The topography of eastern Washington ranges from mountainous to rolling hills, and most of the land is either dry land wheat farms or scrub fields. The bedrock consists of generally horizontal layers of fractured basalt.

A sample of roads with enough different characteristics to be representative of all the other roads in these two districts was sought. Characteristics that were considered included age, pavement structure, traffic flow and road location. The roads that were selected are shown in Figure 13 and are further summarized in Table 10. Where more than one section was tested on a specific road, each section was designated by a letter.

The ESAL count for each road was estimated by multiplying the average number of trucks per year for each road, as listed in the WSPMS by 1.03, the average number of ESALs/truck for Washington state. (17) These ESAL estimates were considered to be only approximate at best.

ANALYSIS OF THE SUBGRADE

A most important parameter to understand in a pavement system is the subgrade. Since the deflections at a distance of several feet from the center of the load are little affected by the surface layers, these deflections provide a means of estimating the moduli of the subgrade. Equation C is one such equation (24):

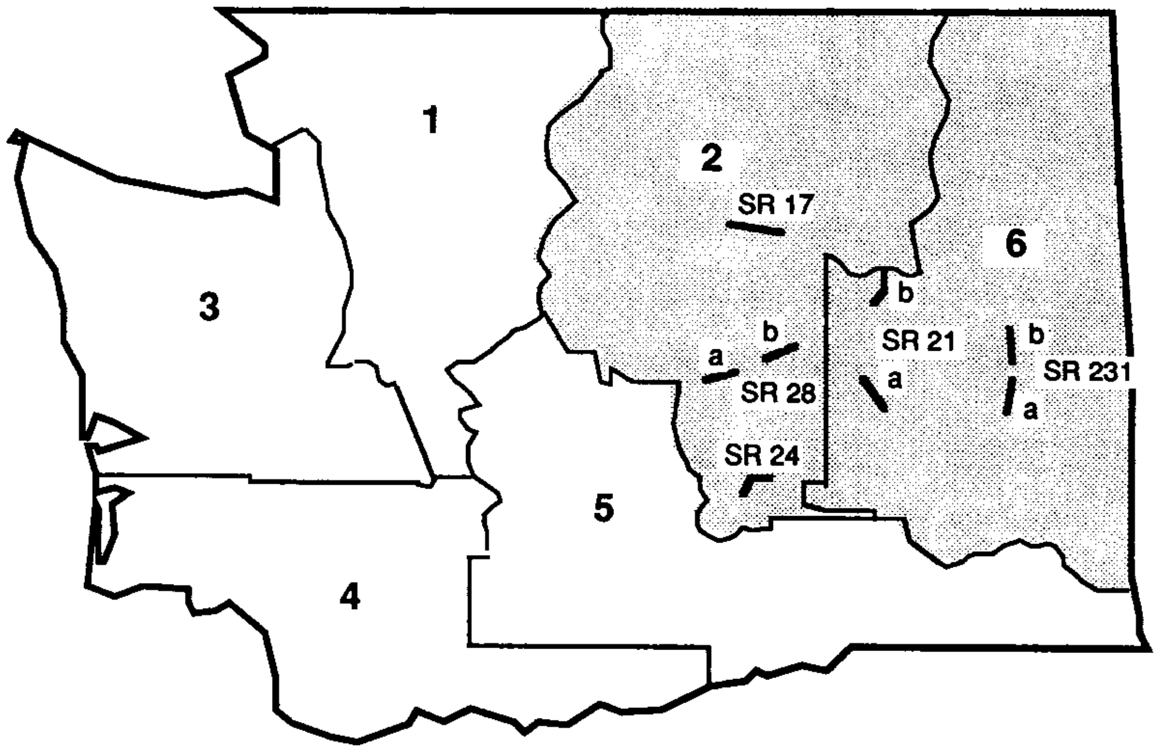


Figure 13. Sections of Roads Tested during the Granular Overlay Study

Table 10. Road Sections Tested with the FWD

Location	Surfacing Type	Granular Thickness		Age at testing	Test Length		Average Daily	Annual 18kip
		in.	mm	(years)	miles	km	Traffic[1]	(80 kN) ESAL[2]
District 2								
SR17	BST	2.4-12[3]	60-300	6	8.0	12.8	960	107,000
SR 24	BST	3.6	90	4	11.0	17.6	920	26,000
SR 28A	AC	3.0	75	6	5.0	8.0	4,700	180,000
SR 28B	BST	3.6	90	6	5.8	9.3	640	29,000
District 6								
SR21A	AC	4.2	107	11	5.0	8.0	330	28,000
SR21B	BST	3.0	75	10	5.0	8.0	280	13,000
SR231A	BST/AC	4.2	107	11	5.0	8.0	180	5,700
SR231B	BST/AC	4.8	122	8	5.0	8.0	230	6,400

[1]WSPMS, 1990
[2]Calculation is based on WSDOT's estimate of truck count (MIDAL, 1990)
[3]Thickness varies

Table 11. Calculated Subgrade Moduli for the Test Sections

Route Number	Direction of Travel	Subgrade Modulus of Elasticity Average (psi)	Standard Dev. (psi)
District 2			
SR17	North	16,200	4,300
	South	16,500	5,200
SR24	North	15,800	2,000
	South	15,700	2,300
SR 28A w/o GO	East	25,000	4,000
w/GO	East	24,400	4,900
w/o GO	West	22,200	3,600
w/GO	West	22,000	3,500
SR28B	East	22,300	6,300
	West	22,500	5,800
District 6			
SR21A	North	14,300	4,200
	South	15,100	7,500
SR21B	North	11,000	4,000
	South	13,000	7,200
SR231A w/o GO	North	18,100	5,200
w/ GO	North	20,300	24,500
w/o GO	South	13,100	4,300
w/ GO	South	15,800	12,500
SR231B	North	14,400	10,100
	South	12,100	3,400

1 psi = 6.89 kPa

$$E_S = -111 + 0.00577(P/D_4) \quad \text{(Equation C)}$$

where: E_S = Modulus of elasticity of the subgrade (psi)

P = Applied load (lbs), and

D_4 = Deflection 4.0 ft (1.22 m) from applied load.

The results of this calculation on the roads tested are shown in Table 11. The moduli for District 2 were generally higher than those of District 6. The effect of this will be seen in the subsequent analyses.

ANALYSIS OF D_0

One method for analyzing deflection basins is to examine the measurements taken directly under the load, D_0. This measurement is important because it is the only deflection

Table 12 Summary of D_0 Values Normalized to a 9000 lb (40.0 kN) Load and 77° F (25° C)
(Temperatures were normalized using WSDOT temperature correction procedure.)

Route Number	Direction of Travel	D_0(mils) Average ($\bar{x}$)	Standard Deviation	$\bar{x}$ + 2SD
District 2				
SR17	North	26.6	5.1	36.8
	South	23.2	4.3	31.9
SR24	North	16.7	2.1	20.9
	South	19.2	3.0	25.2
SR 28A w/o GO	East	14.0	2.2	18.5
w/o GO	West	15.3	2.7	20.8
w/ GO	East	12.1	1.9	15.8
w/ GO	West	13.1	2.1	17.3
SR28B	East	15.7	3.5	22.7
	West	18.0	4.5	27.0
District 6				
SR21A	North	26.4	4.1	34.6
	South	26.4	5.1	36.5
SR21B	North	48.6	12.3	73.2
	South	38.9	11.4	61.8
SR231A w/o GO	North	25.1	4.4	33.9
w/o GO	South	30.5	6.9	44.4
w/ GO	North	33.6	7.6	48.9
w/ GO	South	31.2	9.2	49.7
SR231B	North	29.8	5.8	35.6
	South	32.7	5.4	43.8

1 mil = 25.4 μm

that results from stresses on all of the pavement layers. Therefore, it is a reflection of the stiffness of both the pavement system and the subgrade. By comparing the magnitude of D_0 among different pavement sections, one can determine the relative stiffness of the different pavements. Table 12 summarizes the D_0 readings for the pavements tested in this study.

To compare these results, typical pavements were modeled with the linear elastic layer program, ELSYM5. The AC thicknesses were chosen to represent thin, medium, and thick pavements. The elastic moduli for the subgrade were varied to represent common moduli from the test sections. The layer properties are listed in Table 13. The D_0s for these sections, based upon a linear elastic layer analysis, ELSYM5, are listed in Table 14.

Table 13. Layer Properties in the Typical Pavement Sections

Material	Thickness		Elastic Modulus		Poisson's Ratio
	(mm)	(in)	(P_a)	(ksi)	
Asphalt Concrete	76, 127, 178	3.0, 5.0 and 7.0	3,445	500	0.35
Crushed Stone Base	152	6.0	172	25	0.40
Subgrade	semi-infinite	semi-infinite	103 and 152	15 and 22	0.45

Table 14. D_0 Values for the Typical Pavement Sections

Thickness AC		Subgrade Elastic Modulus		D_0	
(mm)	(in)	P_a	(psi)	mm	(mils)
76	3.0	103	15000	.67	26.3
127	5.0	103	15000	.47	18.7
178	7.0	103	15000	.37	14.5
76	3.0	151	22000	.67	21.3
127	5.0	151	22000	.47	15.0
178	7.0	151	22000	.37	11.7

The values for D_0 were higher in District 6 than in District 2 (again, refer to Table 12). As shown in Table 11, the subgrades under the roads in District 6 were less stiff than those in District 2. This reduced stiffness accounts for a large part of the difference.

For each road, one lane had higher deflections than the other. The subgrades were approximately equal and the pavement structures were assumed to be equal. One possible explanation for the difference is that one of the lanes had received higher traffic loadings than the other and therefore had deteriorated more.

For SR28A, the section of road with the granular overlay had a lower D_0 than the section without the granular overlay. Although the AC was slightly thicker (the section with the granular overlay had a total of 5.8 in. of AC versus 4.0 in. for the section without the granular overlay), the granular layer was probably also serving to increase the stiffness of the pavement.

The section of SR231A without the granular overlay had a lower D_0 than did the section with the granular overlay. The difference was likely due to the poor condition of the road that was overlaid with the granular overlay and the lower subgrade elastic moduli. In the last PCR survey taken before the road was resurfaced, the PCR for the section without the granular overlay was twice as high as the section with the granular overlay (a PCR of 96 versus 41 in 1977). Therefore, the granular overlay was probably applied to remedy a problem in the pavement from Milepost 2.7 to 7.3. The overlay for Milepost 0.0 to 2.7 might have been applied for other reasons.

ANALYSIS OF THE AREA PARAMETER

Although D_0 is a good indicator of the relative, overall stiffness of the pavement, it is strongly influenced by the stiffness of the subgrade. To reduce the effect of the subgrade, the Area Parameter uses the measurements taken by several of the sensors, thus providing a measurement of the shape of the deflection basin. The equation upon which the Area Parameter analysis is based is as follows:

$$\text{Area Parameter} = (6/D_0)(1 + 2D_1 + 2D_2 + D_3) \qquad \text{(Equation D)}$$

where D_n = the deflection at a distance of 'n' feet from the load.

By "normalizing" the deflections at some distance form the load plate with D_0, the Area Parameter indicates the relative stiffness of the pavement structure, more or less eliminating the influence of the subgrade. In the extreme case, where there is no pavement structure (e.g.,the testing is conducted directly on the subgrade), the ratios of

$$\frac{D_1}{D_0}, \frac{D_2}{D_0}, \frac{D_3}{D_0}$$

will result in ratios of 0.26, 0.125 and 0.083 (for a one layer pavement system). Inserting these into Equation D, the Area Parameter is

$$\text{Area Parameter} = 6(1 + 2(0.26) + 2(0.125) + 0.083) = 11.1$$

Therefore, 11.1 is the minimum possible value for the Area Parameter.

Conversely, if the pavement is very rigid, the values of D_1 through D_3 are very close to D_0. In the case of a pavement that is absolutely rigid, all of the deflections are equal to 1.0.

$$\frac{D_1}{D_0} = \frac{D_2}{D_0} = \frac{D_3}{D_0} = 1.0$$

Therefore, the Area Parameter is

$$\text{Area Parameter} = 6(1 + 2(1.0) + 2(1.0) + 1.0) = 36.0$$

Thus, the maximum value for the Area Parameter is 36.0.

The average Area Parameters for the test sections are given in Table 15. The Area Parameters for the "typical" pavement sections are given in Table 16.

The Area parameters for the test sections in District 2 ranged from 15.9 to 21.2 and had a mean value of 17.5. This was between the values for the Area Parameter of the 3.0- and 5.0-in. AC pavement over the 22-ksi subgrade. The Area Parameters for District 6 varied from 17.0 to 18.8 and averaged between the values of the Area Parameter for the 3.0- and 5.0-in. pavement over the 15-ksi subgrade.

In each road, one lane had a higher Area Parameter than the other. As was discussed for D_0, this was likely the result of higher traffic in one lane.

In SR28A, the section of road with the granular overlay had a higher Area Parameter than did the section without the granular overlay. This was not true for SR231A. The reasons for this were the same as the reasons stated for the difference in D_0.

BACKCALCULATION OF MODULI

Because of the complexities of the pavement system and the inaccuracies of the layer thickness data, most of the back calculation analyses (by EVERCALC 3.0) of the test sections contained unacceptably large convergence errors. Out of the eight test sections

Table 15. Comparison of the Area Parameters for the Tested Roads
(All deflections were normalized to a 9.0 kip (40.0 kN) load and 77° F (25° C).)

Route Number	Direction of Travel	Area Parameters	
		Average	Standard Deviation
District 2			
SR17	North	15.9	1.1
	South	17.2	1.6
SR24	North	21.2	1.0
	South	19.5	1.1
SR 28A w/o GO	East	18.0	1.3
w/o GO	West	18.3	0.5
w/GO	East	19.5	1.5
w/GO	West	19.7	1.6
SR28B	East	18.2	1.7
	West	16.8	1.3
Overall Mean		18.4	
District 6			
SR21A	North	18.2	1.2
	South	18.2	1.6
SR21B	North	15.3	1.4
	South	16.1	1.8
SR231A w/o GO	North	18.8	1.9
w/o GO	South	18.2	1.9
w/ GO	North	17.3	1.8
w/ GO	South	18.3	2.7
SR 231B	North	17.2	1.7
	South	17.0	1.2
Overall Mean		17.5	

Table 16. Area Parameters for the Typical Pavement Sections

Thickness AC (in.)	Subgrade Elastic Modulus (psi)	Area Parameter
3	15000	16.5
5	15000	20.2
7	15000	22.7
3	22000	15.1
5	22000	18.6
7	22000	21.0

analyzed, only two had a significant number of tests with reasonable results. Reasonable results were defined as results that had RMS errors of less than 1.5 percent and moduli within an acceptable range of the expected moduli for the material. The acceptable range of moduli for the crushed rock layer was 20 to 130 ksi (138 to 896 MPa). The lower limit of 20 ksi (138 MPa) was the typical stiffness of an unconfined crushed rock layer. Test sections that had fewer than 10 percent of the results within the acceptable range were considered unacceptable.

The best results came from the analysis of SR28A. This section had an AC surface course and included a mile section of road without a granular layer. The test section was located in an open, level area with no visible rock. The road was well elevated and there was no signs of water.

Initially, the section of road without the overlay was analyzed. For the purpose of this analysis the pavement was divided into three layers: the surface, base course, and subgrade. The results of this analysis are shown in Table 17.

The results of this analysis for the surface and subgrade were consistent, in that the standard deviations were low and the correspondence between the lanes was good. Additionally, the mean values for the subgrade from this analysis were close to the values of 25 ksi (172 MPa) and 22 ksi (152 MPa). The only result that did vary widely was K1 for the granular layer. This was the case in every analysis. On the basis of this analysis, it was assumed that the base course and the subgrade had very close moduli and could be considered one layer. Also, since the pavement layer in this model was the same as the old pavement layer for the pavement with the granular overlay, the old pavement should have had a modulus of elasticity of no lower than 300 ksi (2067 MPa).

Then the granular overlay section was analyzed. This was modeled as a four-layer section, with the granular overlay as two layers, the old pavement layers as a third, and the base and subgrade combined into a fourth. The first analysis was conducted without any fixed layers. This yielded widely varying results. Next, the old pavement layer was

Table 17. EVERCALC Results for SR28A without the Granular Overlay
(Included are only the results with RMS errors of less than 1.5 percent — total of 17 deflection locations.)

Direction of Travel	AC	Base Course				Subgrade			
	E adj. (ksi)	E (ksi)	θ (psi)	K_1	K_2	E (ksi)	θ (psi)	K_1	K_2
North									
Mean	312	39.1	14.6	1.9E4	0.35	22.7	6.10	2.6E4	0.08
S D	92.3	15.4	1.7	1.8E4	0.14	3.5	0.58	4.5E3	0.03
South									
Mean	323	22.2	14.0	5.9E3	0.52	21.5	5.58	2.4E4	0.07
S D	45	5.3	0.52	1.6E3	0.10	1.8	0.21	2.3E3	0.02

Table 18. EVERCALC Results for SR28A with the Granular Overlay
(Included are only the results with RMS errors of less than 1.5 percent and reasonable values for the moduli, as defined earlier.)

Direction of Travel	AC	Crushed Rock Layer				Old AC	Count
	E adj.	E (ksi)	θ (psi)	K_1	K_2	E (ksi)	(total)
North							
Mean	959	75.9	104	4.8E3	0.74	500	14
S D	457	40.0	13.4	9.4E3	0.15		(49)
South							
Mean	817	74.2	94.0	3.7E3	0.76	500	12
S D	435	53.0	16.0	4.8E3	0.12		(48)

fixed. The modulus of this layer was fixed at 500 ksi (3445 MPa). A value slightly higher than 300 ksi (2067 MPa) was chosen to reflect the protection that would be offered by the granular overlay. The results of this analysis are shown in Table 18.

Although these results represented only 25 percent of the sections, the results that had RMS errors of less than 2.5 percent and represented 75 percent of the data were quite similar. Additional analyses varied the modulus of elasticity of the old pavement from 300 ksi (2067 MPa) to 800 ksi (5512 MPa), but this only changed the moduli for the granular overlay layers by approximately 20 percent.

Table 19. EVERCALC Results for SR17 with the Granular Overlay (Included are only the results with RMS errors less than 1.5 percent and reasonable values for the moduli.)

Direction of Travel	BST	Crushed Rock Layer				Old BST	Count
	E fixed (ksi)	E (ksi)	θ (psi)	K_1	K_2	E (ksi)	(total)
East							
Mean	500	85.1	51.0	2.4 E4	0.12	113	19
SD		19.0	34.2	3.4 E4	0.86	177	(81)
West							
Mean	500	82.3	49.0	5.4 E4	0.22	101.8	13
SD		27.4	33.9	8.2 E4	0.38	124	83

Table 20. Typical Elastic Moduli and Bulk Stresses from Inverted Pavements (16)

Road	Thickness (in.)	Load (ksi)	Elastic Modulus (ksi)	θ (psi)
P157/1	7.0	9.0	29.0	48.9
		15.7	43.5	58.4
P157/2	5.5	9.0	48.9	53.7
		15.7	75.4	89.6

A second road that produced good results was SR17. The results of this analysis are shown in Table 19. The values for the modulus of elasticity of the granular layer corresponded closely with those of SR28A (Table 18). One difference in the data was that the bulk stress for SR17 was approximately half of that of SR28A. SR17 had a BST surface while SR28A had an AC surface. Since the BST surface on SR17 was thinner and less stiff than the AC surface on SR28A, it is reasonable that the BST surface would exert less confining pressure than the AC and this is why the confining pressure was different.

On the basis of the analyses of SR28A and SR17, the modulus of elasticity for the crushed rock layer of the granular overlay was estimated to be approximately 80 ksi (551.2 MPa) under a 9.0-kip load. The South African studies showed elastic moduli ranging from 29.0 to 75.4 (and higher), as shown in Table 20. One of the reasons for the higher modulus of elasticity was that the bulk stress in the crushed rock layer in the granular overlay was higher than in the inverted pavement.

CONCLUSIONS

The following conclusions are based on the entire study of granular overlays.

1. Granular overlays are effective at reducing reflection cracking, insulating the old pavement surface against extremes in temperature, and improving the road geometry.
2. To protect the granular layer, the old pavement surface should be sealed, and the new surface layer should be made as impermeable as possible.
3. BSTs are more appropriate surfacings for granular overlays than are AC overlays.
4. By using a crushed rock with a dense gradation and a high percentage of fractured faces and by compacting it the maximum possible amount, engineers will be able to produce a granular overlay with better performance.
5. The crushed rock layer should have a maximum recommended thickness of 6 in. (150 mm) (based on structural considerations only) and a minimum value of 3.0 in. (75 mm).
6. The AC equivalency factor for the confined crushed rock layer that is properly constructed and well protected is 2.0.
7. When designed with an equivalency factor of 2.0, granular overlays are slightly less expensive than AC overlays.
8. The "typical" survival lives for different overlays are shown in Table 21.
9. The "typical" moduli for confined crushed rock layers are shown in Table 22.
10. The recommended maximum ADT for BSTs of 5,000 with 15 percent trucks is the best estimate of the maximum traffic count that roads with

granular overlays can withstand as they are currently designed and constructed.

11. Consideration should be given by WSDOT to using Crushed Surfacing Base Course (maximum aggregate size = 1-1/4 inches) for the crushed rock portion of the granular overlay on some projects and evaluating its performance. The gradation is similar (but not the same) to the South African G1 material specification.

12. WSDOT should consider building an "inverted" pavement section on an appropriate rehabilitation project in the "dryer" part of the state Districts 2,5 or 6). The existing section could be scarified, treated with cement (CTB), crushed rock placed over the CTB plus surfacing. In this manner, the section could be evaluated for performance. Granted, this is not a "classic" granular overlay but it is an extension of the concept and, in theory, accommodates modest to high ESAL levels.

Table 21. Survival Times and Predicted Performance Periods for the Overlays

Type of Resurfacing	Actual Survival Life	Predicted Performance Period		
		PCR 40	PCR 20	PCR 0
BST Only	9.2	7.0	8.2	9.3
G O w/BST surface	—	7.5	8.3	11.0
AC Only	9.7	10.2	11.3	12.1
G O w/AC surface	—	9.5	11.3	11.8

Table 22. Moduli Calculated for Confined Crushed Rock Layers in Granular Overlays (WSDOT) and Inverted Pavements (South Africa)

Road	Crushed Rock Thickness (in.)	Load (kips)	Modulus of Elasticity (ksi)	Bulk Stress (psi)
WSDOT				
SR28A	3.0	9.0	75	99
SR17	2.4-12	9.0	84	50
South African				
P157/1	7.0	9.0	29	48.9
		15.7	43.5	58.4
P157/2	5.5	9.0	48.9	53.7
		15.7	75.4	89.6

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