Estimating Voids in a Double Chip Seal

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In the design of double chip seals, perhaps the most important factor to be computed is the amount of bituminous material required to fill the voids between the aggregate to an optimum depth. A design method developed by the National Institute for Transport and Road Research was evaluated by Texas Transportation Institute (TTI). This method includes a simple test procedure for measuring the void content and effective layer thickness of the stone layers. It also provides for a way of estimating the loss of voids in the seal over its expected life due to embedment in the underlying surface and wear and degradation of the stone. This method also considers the fact that voids within the aggregate layers vary nonlinearly with depth. This design approach is quite different from anything currently used in the United States. TTI evaluated this method by using chip seal aggregates graded to Texas State Department of Highways and Public Transportation specifications. Two double chip seal test roads built in Texas according to the design methods discussed are performing well.

A double chip seal is a bituminous surface that results from two successive alternating applications of bituminous binder and cover aggregate to an existing paved surface. In the design of double chip seals, perhaps the most important factor to be computed is the amount of bituminous material required to fill the voids between the aggregate to an optimum depth. This simple and logical principle was first stated by Hanson (*1*) in his study of the performance and design of single surface treatments. Since there is a direct relationship between the void space and the amount of bituminous material needed, it is essential to have a good indication of the actual void content in a layer of aggregate with shoulder-to-shoulder contact to execute an effective design.

VOIDS IN A STONE LAYER

Voids as used in Existing Methods of Designing Seals

Hanson (*1*) found that a single layer of one-size cover aggregate in a loose spread condition is oriented in random directions. In this state, the volume of voids between the aggregate particles is approximately 50 percent. He observed that after some rolling and traffic compaction, the aggregate particles tend to become oriented in a position so that they lie on their flattest side with their least dimension normal to the road surface. Under these conditions, Hanson reported that the voids between the aggregate were approximately 20 percent. This void space of 20 percent is independent of the size of the one-size cover aggregate. It is thought by some investigators that the volume of voids in the chip seal aggregate is only related to the position or orientation of the aggregate and not by the size or type of the aggregate.

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The Country Roads Board of Victoria, Australia, and McLeod (*2,3*), whose methods of designing chip seals are based principally on Hanson's work, indirectly consider the shape of the aggregate by varying the amount of bituminous material needed to fill the aggregate voids to an optimum amount according to the type of aggregate to be used.

Several engineers take into consideration the shape of the aggregate by determining the volume of the voids to be filled by first placing the aggregate in a large cylinder. Kearby (*4*) and later Benson and Gallaway (*5*) computed the percent voids from the loose unit weight of the aggregate. In these cases, it is assumed that the aggregate in the one-stone-thick layer on the road surface will have the same arrangement and voids as it will have in the cylinder.

All of the chip seal design methods presently being used in the United States assume that the volume of voids in a single layer of stone varies linearly with depth. No design method considers the fact that voids within the aggregate layer vary nonlinearly with depth.

Saner and Herrin (*6*) were the first engineers to conclude from their research study on voids in one-size surface treatment aggregates that the linear relation assumed in the chip seal design methods was not true and that a curvilinear relationship exists. Their study revealed that although the curvilinear relationship varies for different aggregate sizes, it has the same basic shape. They also concluded that aggregate samples of different shape have significant differences in percent voids and that a suitable shape factor needs to be developed for design purposes to relate the volume of voids to the shape of the aggregate.

Marais (*7,8*) was the first engineer to incorporate the variation of the void volume with depth within a single layer of stone into a design method. His proposed method differs from any previous design method in that it analyzes the factors that affect a change in void volume in a single layer of stone with shoulder-to-shoulder contact between particles to determine the rate of binder application.

Change In Void Volume

A certain amount of empty space is present in a double layer of stone. A portion of these voids is lost during the life of the seal because of the effect of traffic on (a) the embedment of the aggregate at the bottom of the seal layer and (b) the wear and degradation of the aggregate at the top of the seal layer (*9*). Also, a certain portion of the voids must be left unfilled with binder to ensure good skid resistance.

The void volume that must therefore be filled with binder is the balance of this void volume that remains after the estimated amount of loss that results from embedment and wear, and the amount required for good skid resistance, have been subtracted.

It is clear that a better knowledge of the actual void content in a stone layer is essential to execute an effective design. This is an area in which problems have been experienced in the past because most design methods assume a fixed equation for the void content in relation to the average least dimension (ALD) of the aggregate or a fixed value for the void content regardless of the ALD value.

For a proper design procedure, the following factors have to be considered.

Embedment

For single or double seals, the embedment of the layer of stone in contact with the road surface is of particular importance in the subsequent performance of the seal coat or surface treatment. The embedment is independent of the thickness of the binder film and refers to the gradual immersion of the stone into the underlying road surface due to traffic compaction (*7,8,10*).

It is believed that insufficient attention to embedment results in the majority of chip seal failures in practice. Some embedment is necessary to ensure that the seal is well bonded with the existing road surface. However, excessive embedment can result in premature bleeding of the chip seal. Researchers (*2,11*) have recognized to a limited extent that embedment of the surfacing stone is desirable, but they have not quantified the amount of embedment that is likely to occur in practice and have merely left embedment depth to the judgment of the designer.

Embedment is by far the most important factor to be considered in the reduction of the volume of voids that takes place in a single layer of stone in shoulder-to-shoulder contact (*7*), and, therefore, it requires special consideration. Careful measurements have shown that embedment does occur and that the amount of embedment is dependent on the intensity of traffic and the hardness of the underlying surface (*7,12*). Research by Potter and Church (*12*) revealed that traffic has a larger effect on embedment than does the hardness of the underlying surface (except for PCC surface). Potter and Church also showed that the reduction in the effective voids due to embedment is marked after only 3 months of service.

The following question arises: How long does embedment continue to increase? It seems likely that the bulk of the embedment will have occurred in the first 12 months under normal traffic (*12*). In areas subjected to freezing, the time of the wear in which the seal is completed could have a bearing on the rate of embedment. The accurate measurement of the embedment of the stone into the underlying surface is a serious practical problem. Studies (*7,12*) have been undertaken to assess the amount of embedment under known traffic. It seems to be that the embedment is a gradual process considered to have reached equilibrium condition after 3 years. Even so, the time is affected by the amount of traffic and by the temperature of the road surface (when reseals are considered). A higher rate of embedment occurs under high road surface temperature.

Wear and Degradation

Wear of the aggregate in a double chip seal occurs due to the action and the intensity of traffic. Observations have shown that the wear of the aggregate takes place at the topmost (exposed) face of the stone layer and is more noticeable with weak than with strong aggregates. Studies by Marais (*7*) revealed that after 5 years of service the stone changed to a more spherical shape. For even longer service life, heavy traffic can reduce stones to flat particles, increasing the aggregate Flakiness Index. The wearing of the stone in a single seal coat or surface treatment reduces the available voids to be filled with asphalt.

Degradation of the stone takes place mainly during the construction phase, particularly when steel-wheel rollers are used (*7*). Owing to this fact, steel-wheel rollers are not recommended, and the pneumatic type roller is preferred. The net effect of degradation is that it changes the grading of the stone, producing smaller-sized particles. This change in the grading of the cover aggregate decreases the available voids either by filling the existing voids (acting as a wedge between larger stones) or through the reduction of the overall size of the original particles (lowering the ALD).

Skid Resistance

The skid resistance of highway pavements, particularly when wet, is a serious problem of increasing concern to highway engineers and researchers. As traffic speeds and traffic densities continue to rise, the frequency of skidding accidents increases at an alarming rate each year. Therefore, maintaining pavement friction is a high priority in the continuing campaign to reduce traffic accidents.

The term, "skid resistance," as commonly used, refers to the characteristics of pavement surfaces that inhibit skidding; that is, the sliding of a tire or a vehicle in an uncontrolled manner.

The texture in a double chip seal coat surface is significantly influenced by the aggregate size of the top stone layer. Texture generates resistance to sliding by the hysteresis effects in the tread rubber and facilitates the expulsion of water from the tire-pavement interface. Hysteresis reflects the energy loss that occurs as the rubber is alternately compressed and expanded (the lost energy appears as heat). Thus, as the tire slides over the irregularities of the textured surface, resistance develops even if the surface is perfectly lubricated.

Surface texture is beneficial to the generation of friction, but its most important function is to provide channels by which the water can escape from under the tire, enabling the tread rubber to make contact with the pavement. Providing and maintaining a skid-resistant surface is an important factor in the performance of any highway, and a primary purpose for applying any type of chip seal is to improve the skid resistance characteristics of an existing asphalt concrete pavement. In the design of both single and double chip seals, the macrotexture of the aggregate is taken into account to ensure good skid resistance. Most of the existing design methods (*2,11,13*) indirectly consider the texture of the aggregate by selecting the aggregate size. The most recent design methods (*8,9*) take into account a portion of the aggregate surface texture depth (void space not to be filled with binder) to ensure satisfactory skid resistance properties in wet weather and to prevent hydroplaning.

Engineers in general agree that an increase in the quantity of binder is required to allow for the existing road surface texture. Some adjustments to the cold binder volume calculation is then necessary to allow for the existing texture of the road to be sealed.

PREDICTION OF VOIDS

Measuring Voids in a Double Stone Layer

In an attempt to measure more accurately the actual void content of a single layer of stone in shoulder-to-shoulder contact, the National Institute for Transport and Road Research (NITRR) devised a very simple test known as the Modified Tray Test (*9,14–16*). The Modified Tray Test was developed to determine the true layer void content and the effective layer thickness (ELT) of a single layer of stone. This test was further extended to measure the voids in a double layer of stone.

The test equipment essentially consists of a circular tray and a shoulder piece that fits snugly on top of the tray. The shoulder piece has the same internal diameter as the tray and is fitted to a loose-fitting cloth membrane. The purpose of the membrane is to prevent the "density sand" from flowing into the voids between the stone. The test is performed by packing the stone in the tray in a single layer with the least dimension vertical. The stone should be packed shoulder to shoulder (Figure 1). The shoulder with the membrane is then placed on top of the tray, and the membrane is smoothed without disturbing the stone (Figure 2). This entire mass is determined.

The space above the stone is then filled with "density sand" in one smooth pour (Figure 3). The tray should be overfilled and the excess sand scraped off with a straight edge. This mass is then determined. The aggregate sample used in the tray is then poured into a plastic measuring cylinder, and the average volume is read off in milliliters. This quantity is used to determine spread rate of the aggregate.

To determine the internal volume of the tray, the same procedure should be performed but without the stone. The

FIGURE 1 Packing of stone in modified tray.

FIGURE 2 Placement of shoulder with cloth membrane on modified tray.

FIGURE 3 Pouring density sand into modified tray.

shoulder should be placed on top of the empty tray, smoothing out the membrane against the bottom and sides of the tray. The tray should then be filled with the density sand and this mass determined. Because this step is performed with the membrane in place, the volume of the membrane is accounted for in the test procedure.

The void space occupied by the stone plus voids in the layer is determined as follows:

$$V_1 = \frac{M_1 - M_2}{\mathrm{BDS}} \qquad (1)$$

where

V_1 = space occupied by stone plus voids in layer (ml),
M_1 = mass of sand in tray (without stone) (g),
M_2 = mass of sand in tray (with stone) (g), and
BDS = bulk density of the sand (g/ml).

The ELT is calculated as follows:

$$\text{ELT} = \frac{(V_1 \times 10)}{\text{area of tray in cm}^2} \tag{2}$$

The actual void content is calculated as follows:

$$\text{void content} = \frac{(V_1 - V_{\text{stone}})}{V_1 \times 100} \tag{3}$$

where

$$V_{\text{stone}} = \text{space occupied by stone (ml)} = \frac{\text{mass of stone}}{\text{relative density of stone}} \tag{4}$$

To measure voids in a double layer of stone, the same test can be performed on the two layers of stone separately. A relationship for the ELT of the double seal and the sum of the ELTs of the bottom and top layers was developed by the NITRR and was verified by TTI by using Texas chip seal aggregate. This is presented in Figure 4.

Estimating Embedment into Underlying Surface

The Ball Penetration Test (*19*) measures the penetration resistance of a road surface such that an estimate can be obtained of the future embedment of the stone into the underlying surface.

The equipment required to perform the Ball Penetration Test consists of a circular tripod stand and cross bar, a 19-mm steel ball, a depth gauge, a surface thermometer, and a standard Marshall compaction hammer (Figure 5). The steel ball bearing is forced to penetrate the old road surface to be sealed under a standard effort (one blow of a Marshall hammer), and the depth of penetration is measured. The road surface temperature at the time of the test is recorded, and the penetration value is converted to a standard temperature for that location. A possible relationship between this penetration value, estimated traffic, and the predicted embedment of the stone into the underlying surface is presented in Figure 6. This relationship was developed by Marais (*7*). Data from a small-scale experiment, where embedment depths were measured after 5 years, were used to determine the position of the traffic lines. The slope of these lines was fixed in an arbitrary manner by the following reasoning: (a) As the road surface becomes softer, a higher embedment takes place and (b) as the traffic intensity increases and the road surface becomes

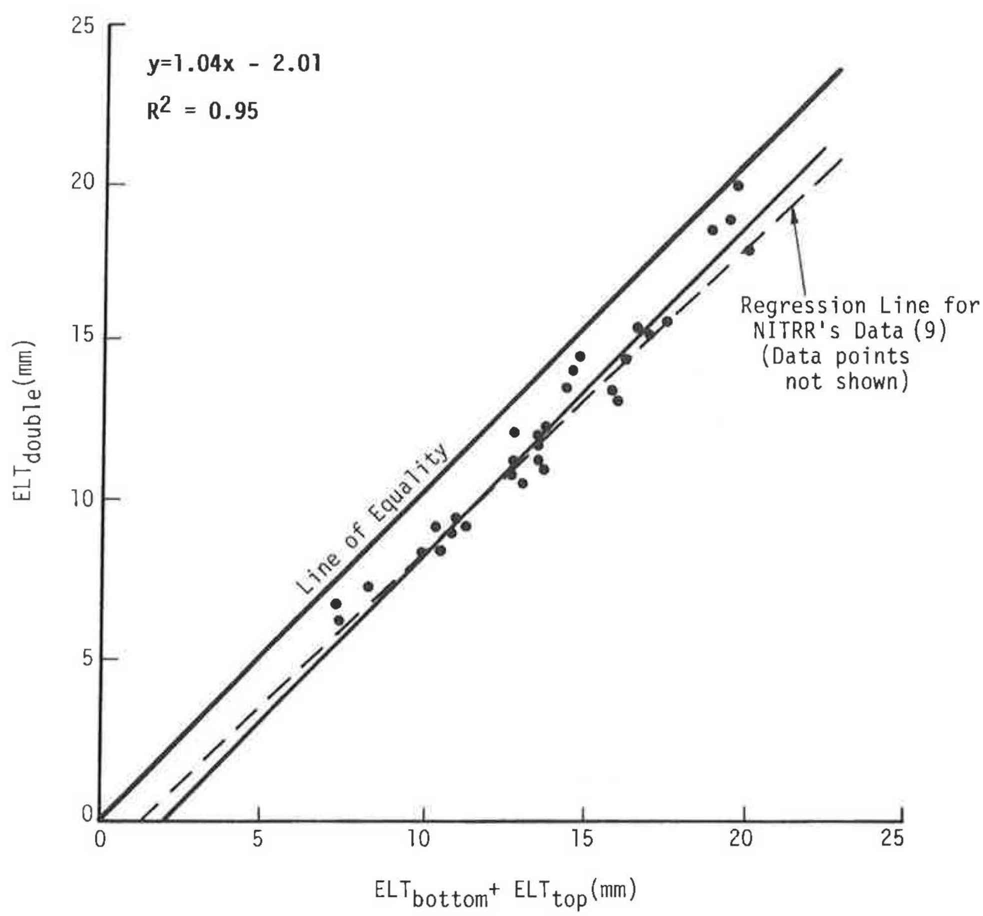

FIGURE 4 The sum of the ELTs of the bottom and top stone layers versus the ELT of the double stone layer.

FIGURE 5 Equipment required for Ball Penetration Test.

softer, it is probable that the embedment increases at a slightly faster rate.

Predicting Degradation and Wear

Wear and degradation of the stone seems to be directly related to the strength of the aggregate. Degradation is thought to take place mainly during the construction process (rolling), and wear is due to the effect of traffic. Results obtained from a small-scale road experiment (*17*) have shown that after 5 years of service the physical dimensions of the aggregates used changed significantly. The original ALD and the Flakiness Index of the aggregate used in the test road were reduced.

On the basis of this study, values for the total degradation and wear that is thought to take place under various traffic intensities over the expected life of a seal (10 years) were estimated based on the Los Angeles Abrasion Value and are given in Table 1 (*7,8*). These values are estimates and require field verification.

Allowance for Texture Depth

The texture depth of the double chip seal surfacing plays a very important role in providing resistance to skidding under wet conditions. Gallaway et al. (*18*) recommend that where longitudinal grades do not exceed 3 percent and drainage is not over three 12-ft lanes, the texture depth should not fall below 1.0 mm.

Allowance for Surface Texture of Existing Surface

Engineers, in general, agree that the quantity of binder to be applied is affected by the texture of the existing surface. This surface hunger is higher on surfaces with coarse textures than those with smooth textures. Therefore, an adjustment for the quantity of additional binder required to allow for the road surface hunger should be taken into account in the design process. A suggested relationship between the texture depth, traffic intensity, and the additional quantity of cold binder required to properly satisfy the surface hunger of the existing road surface is given in Figure 7 (*7*).

By using the Sand Patch Test, the texture depth can be calculated by dividing a given volume of material (sand) by the area it covers. By knowing the texture depth it can then be converted to a quantity of binder expressed in liters/square meter by simply multiplying the texture depth in millimeters by its unit (i.e., 0.64 mm of texture depth = 0.64 l/m^2) or by multiplying by 4.6875 to get gallons/square yard providing that the texture depth is in inches.

Allowance for bleeding surfaces is not explicitly accounted for by this procedure. However, a reduction in the binder content to account indirectly for bleeding surfaces has been taken into consideration when the Ball Penetration embedment value was calculated. Possible binder absorption is also not accounted for because it is doubtful that absorption would take place given the viscosity of the binder at the time of spraying.

Minimum Quantity of Binder

There is a minimum quantity of binder film thickness that is required depending on the size of the stone given the viscosity of the binder at the time of spraying. However, a reduction in the binder content to account indirectly for bleeding surfaces has been taken into consideration. There is a minimum quantity of binder film thickness, determined by the size of the stone, that is required to retain the cover aggregate effectively, withstanding the combined effects of traffic and weather. Laboratory studies in South Africa have revealed that this required binder thickness is the quantity that will occupy just 50 percent of the total voids in a single layer of stone. According to the theoretical void distribution for single seals presented in Figure 8, the binder would cover the aggregate halfway if 50 percent of the voids were filled. The NITRR recommends that to cover the top layer of aggregate halfway in a double seal, 65 percent of the voids should be filled with binder.

A field study on double chip seals by Texas Transportation Institute (TTI), using this design method, revealed that filling 65 percent of the voids with binder was excessive. Traditionally, in Texas the chip seal aggregate should be covered with binder approximately one-third of its depth rather than halfway (immediately after construction). Therefore, TTI used this design method, filling 55 percent of the voids with binder and achieved satisfactory field performance on two following test roads.

FIELD STUDY

The following three test roads were constructed to evaluate the design procedure. Therefore, certain preconstruction field data measurements were obtained that were used in the design procedure.

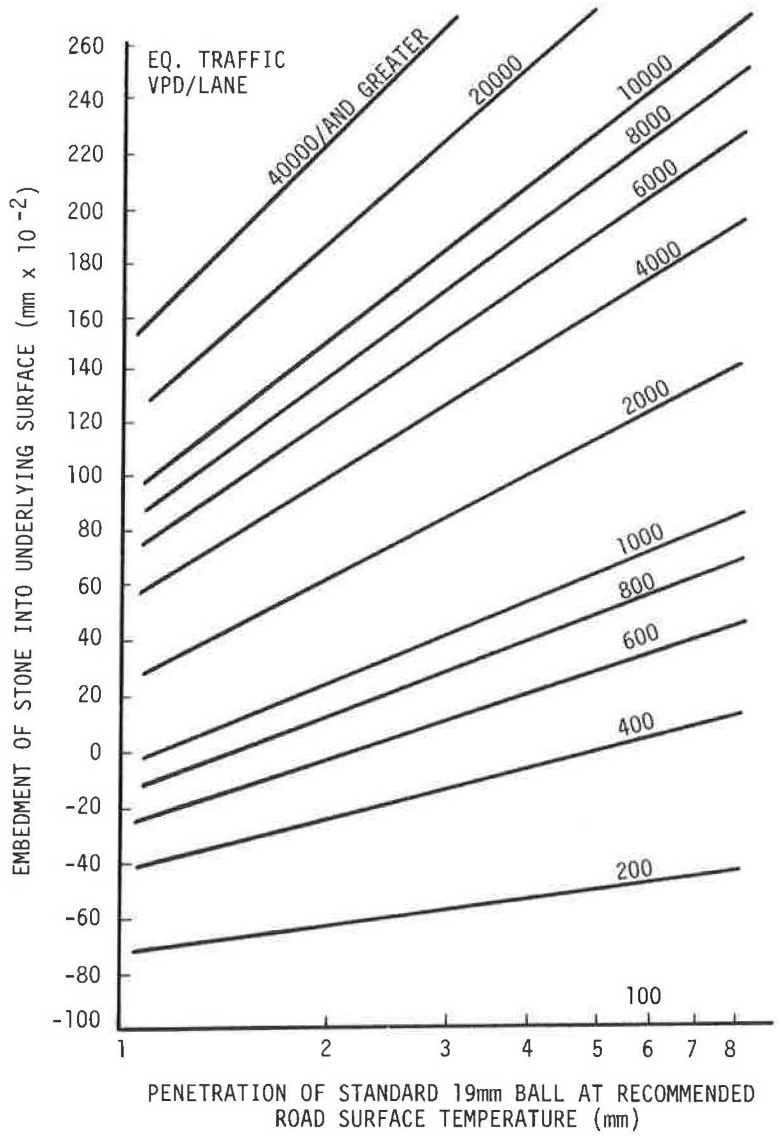

FIGURE 6 Relationship between penetration of standard ball, traffic, and embedment of stone into underlying surface of road (*7*).

TABLE 1 ESTIMATED DEGRADATION AND WEAR UNDER CONSTRUCTION ROLLING AND TRAFFIC (10-YEAR LIFE) (*7*)

Los Angeles Abrasion Value % Loss	Degradation and wear of stone m at (mm x 10^{-2})									
	Equivalent traffic (vpd/lane)									
	>4,000	4,000	3,000	2,000	1,000	800	600	400	200	100
34 - 27	100	92	86	78	66	66	58	52	44	37
26 - 22	90	86	80	72	60	58	54	48	40	34
21 - 15	80	78	74	68	56	54	50	46	38	32
14 - 10	75	72	68	62	52	48	46	42	36	30
9 - 4	70	68	62	56	48	46	42	38	32	28

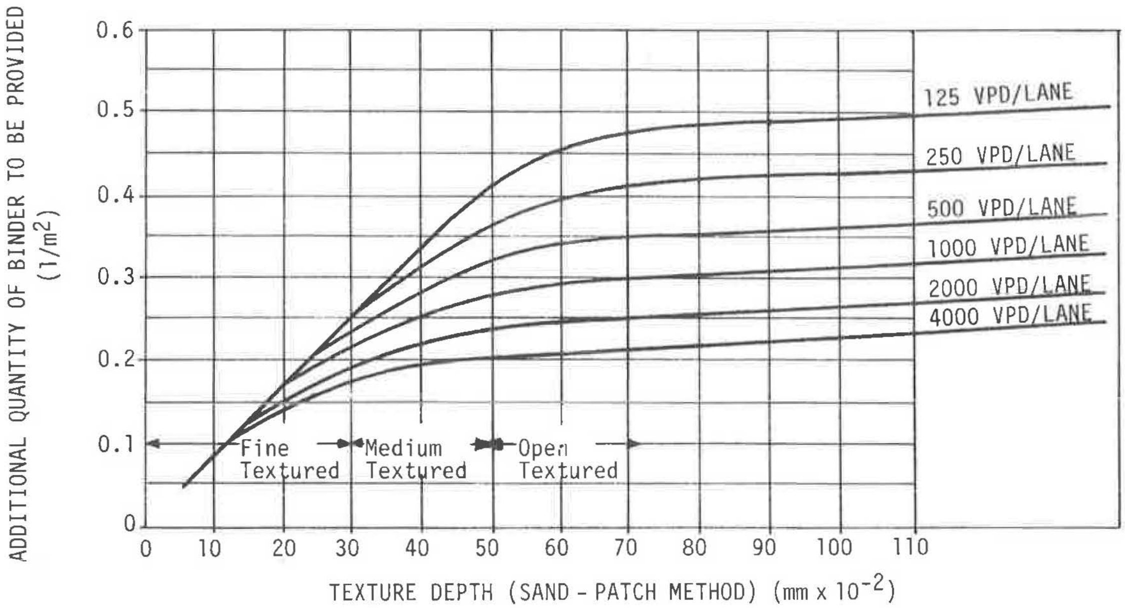

FIGURE 7 **Additional quantity of binder required to allow for texture depth of existing surface (*7*).**

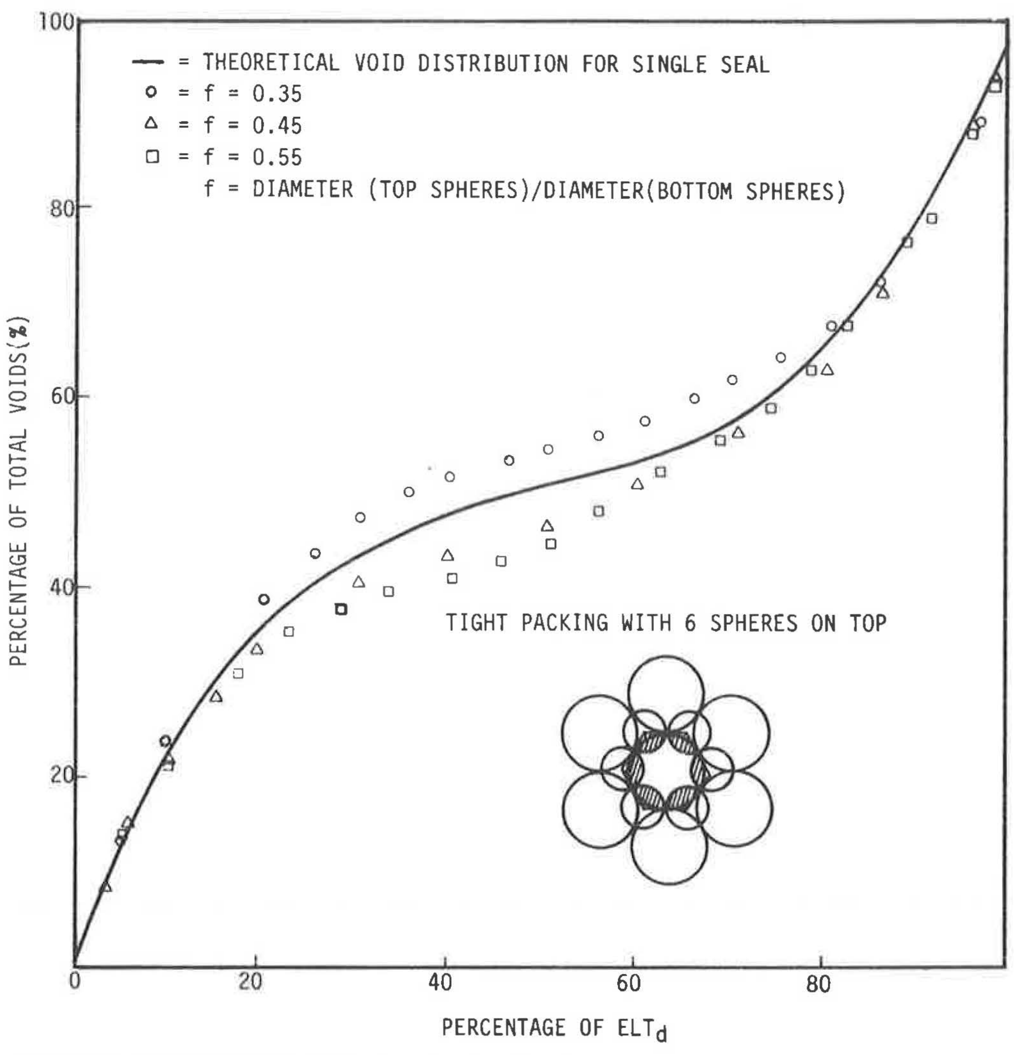

FIGURE 8 **Theoretical void distribution in a double seal.**

The Sand Patch Test was performed on the existing surfaces to determine the surface texture. The Ball Penetration Test was performed on the existing surface to estimate the future embedment of the stone into the underlying surface.

Eastland Test Road

Objectives

The objectives in construction of the Eastland Test Road were to (a) test the design procedure at predicting asphalt and aggregate quantities, (b) evaluate the use of different combinations of aggregate grades in the field, and (c) evaluate the field performance of double seals.

Test Road Construction

The Eastland Test Road is located on the north feeder road of Interstate 20 near Eastland, Texas, and was constructed in 1987. The average daily traffic for this section is approximately 1,000 vehicles per day.

Materials Materials used for the construction of the Eastland Test Road consisted of a Grade 3, Grade 4, and Grade 5 lightweight aggregate from Ranger, Texas, and the binder was an emulsion (HFRS-2) from Texas Emulsions. Grades 3, 4, and 5 refer to an aggregate gradation used in Texas for chip seal aggregates. The specification for these aggregates is as follows:

	Percent by Weight Retained		
Sieve Size	Grade 3	Grade 4	Grade 5
¾ in.	0	—	—
⅝ in.	0–2	0	—
½ in.	20–35	0–2	0
⅜ in.	85–100	20–35	0–5
¼ in.	95–100	—	—
No. 4	—	95–100	40–85
No. 10	99–100	99–100	98–100
No. 20	—	—	99–100

Preconstruction Prior to construction of the test road, an evaluation of the existing pavement was performed along with some field tests. The existing road surface was asphalt concrete in relatively good condition with minimal cracking. The laboratory and field data used to calculate design application quantities for this test road are as follows:

1. Surface texture = 0.89 mm.
2. Corrected Ball Penetration Value = 2.7 mm.
3. ADT = 2300 equivalent light vehicles/day/lane.
4. Note: 1 truck = 25 cars.
5. Los Angeles Abrasion Value = 25 percent.
6. ELT = 7.87 mm for Grade 3, 7.00 mm for Grade 4, and 4.59 mm for Grade 5.
7. Voids in aggregate layer = 50.6 percent for Grade 3, 56.1 percent for Grade 4, and 55.4 percent for Grade 5.

Test Section Layout The test road consisted of four different sections constructed in the westbound lane, and each section was approximately 1,000 ft in length. The first section was a Grade 4 aggregate on top of a Grade 3. The second section was a Grade 5 on top of a Grade 3. The third section was a Grade 5 on top of a Grade 4, and the fourth section was a single Grade 4 seal. The single Grade 4 seal was constructed based on the standard design procedure normally used by this particular district.

Construction The test road was built in August 1987. Traffic was diverted to the eastbound lane until all four test sections were completed. Pneumatic rollers were used on each aggregate layer. The aggregate and binder quantities designed by using the Modified Tray Test and those quantities actually used in the field are shown in Table 2.

Performance of Eastland Test Road

Others (*9*) have recommended that for multiple seals each successive layer should have a stone size approximately half the size of the preceding layer. This was found to be a good recommendation based on the construction of this test road. The Grade 4 on 3 and the Grade 5 on 4 both appeared to be good combinations. However, the Grade 5 on 3 caused some problems during the construction process. Because the Grade 5 is much smaller than the Grade 3, all of the Grade 5 stones collect in the big voids of the Grade 3, leaving an exposed film of binder on the surface of the Grade 3. This causes problems during the rolling process and immediately after traffic has been allowed onto the surface. Because there is an exposed film of asphalt, the asphalt collects on the tires of the rollers and vehicles, and then the tires begin to pick up the stones.

On the basis of visual observations immediately after construction, the design binder quantities for the double seals may have been excessive. Therefore, modifications were made to some of the design parameters to represent more closely what happens in the field. No changes regarding the test procedure were made.

On the basis of a field evaluation of the test sections 1 year after construction, bleeding was observed in the wheel paths. This confirmed the previous conclusion that the binder quantities were excessive.

Paige Test Road

Objectives

The primary objective in construction of the Paige Test Road was to make a final evaluation of the design procedure after the modifications were made. A secondary objective was to evaluate the performance of a double seal on a road with a relatively high traffic volume. Another objective was to observe the construction process used by this particular district that builds double chip seals routinely and with great success.

Test Road Construction

The Paige Test Road was constructed the week of June 27, 1988. It is located between Paige and McDade, Texas, on

TABLE 2 DESIGN AND ACTUAL APPLICATION QUANTITIES FOR THE EASTLAND TEST ROAD

Section	Layer	Aggregate Spread Rate, sy/cy		Residual Binder Application Rate, gsy	
		Design	Actual	Design	Actual
Grade 4 on 3	Grade 4	140	135	0.44	0.45
	Grade 3	126	120	0.30	0.30
Grade 5 on 3	Grade 5	223	210	0.37	0.35
	Grade 3	126	120	0.24	0.29
Grade 5 on 4	Grade 5	223	210	0.35	0.31
	Grade 4	140	135	0.24	0.22
Grade 4*	Grade 4	140	135	0.35	0.35

* The Grade 4 single seal was built according to District 23's standard design procedure.

U.S. Highway 290. Average daily traffic was approximately 7,400 vehicles per day.

Materials The aggregate for construction of the first or bottom layer of the double seal was a Grade 3 limestone from Texas Crushed Stone in Georgetown. The top layer was constructed of a Grade 4 synthetic lightweight from TXI-Streetman. The binder was a polymer-modified emulsion: HFRS-2p from Gulf States.

Preconstruction Prior to construction of the test road, an evaluation of the existing pavement was performed along with some field tests. The existing road surface was a Grade 3 limestone chip seal. The pavement was in relatively good condition with slight to moderate bleeding in the wheel paths. Samples of the aggregate were brought back to the laboratory to perform the Modified Tray Test and calculate design quantities. Laboratory and field data used to calculate design application quantities for this test road are as follows:

1. Surface texture = 0.58 mm.
2. Corrected Ball Penetration Value = 1.05 mm.
3. ADT = 7100 equivalent light vehicles/lane/day.
4. Los Angeles Abrasion Value = 24 percent.
5. ELT = 8.25 mm for Grade 3 and 7.48 mm for Grade 4 aggregate.
6. Voids in aggregate layer = 58.9 percent for Grade 3 and 51.6 percent for Grade 4.

Construction The first layer of binder and aggregate was placed and then rolled with a lightweight steelwheeled roller. The first layer of the seal was placed during the morning, and the second layer was placed in the afternoon. The second layer was then rolled with a medium pneumatic roller followed by a small pneumatic roller. Traffic was not allowed on the first seal layer. The design and actual binder and aggregate quantities applied are shown in Table 3.

Performance

Immediately after construction, the pavement surface appeared to be in good condition, and the application quantities appeared to be the correct amount. One month after construction, the surface was still in good condition and performing as would be expected.

TABLE 3 DESIGN AND ACTUAL APPLICATION QUANTITIES FOR THE PAIGE TEST ROAD

Layer	Aggregate Spread Rate, sy/cy		Residual Binder Application Rate, gsy	
	Design	Actual	Design	Actual
Grade 3	95	92	0.27	0.28
Grade 4	123	120	0.40	0.38

Circleville Test Road

Objectives

The objective in construction of the Circleville Test Road was essentially the same as for the Paige Test Road: to make a final evaluation of the design procedure by comparing the design quantities calculated with the field performance.

Construction of Test Road

The Circleville Test Road was constructed the week of July 6, 1988. It is located between Circleville and Georgetown, Texas, on State Highway 29. The pavement is a two-lane roadway, and the test section is located in the eastbound lane. Average daily traffic is approximately 2,000 vehicles per day.

Materials The aggregate used for construction of the first or bottom layer of the double seal was a Grade 3 limestone from Texas Crushed Stone. The top layer was constructed of a Grade 4 from Delta Materials. The binder was HFRS-2p from Gulf States.

Preconstruction Prior to construction of the test road, an evaluation of the existing pavement was performed along with some field tests. The existing road surface was a seal coat built with a sandstone aggregate. There was slight to moderate bleeding in the wheel paths but no signs of cracking. Samples of the aggregate were brought back to the laboratory to perform the Modified Tray Test and to calculate design quantities. Laboratory and field data used to calculate design application quantities for this test road are listed as follows:

1. Surface texture = 0.60 mm.
2. Corrected Ball Penetration Value = 1.95 mm.
3. ADT = 2500 equivalent light vehicles/lane/day.
4. Los Angeles Abrasion Value = 18 percent.
5. ELT = 8.37 mm for Grade 3 and 6.99 mm for Grade 4 aggregate.
6. Voids in aggregate layer = 52.8 percent for Grade 3 and 56.8 percent for Grade 4.

Construction The first layer of binder and aggregate was placed and then rolled with a medium followed by a small pneumatic roller. The surface was then blade broomed and rolled with a lightweight steel-wheeled roller. The second layer was then rolled with the pneumatic rollers only. Since State Highway 29 is a two-lane roadway, traffic could not be kept off the newly constructed surfaces for the desired length of time. To mimimize rock turn up, pilot trucks were used to lead the traffic back and forth at a low speed. This alleviated but did not eliminate the problem. Another problem was encountered when it appeared that the bond did not occur as quickly as expected between the emulsion and the Grade 4 stone. This also caused damage by traffic. The actual binder and aggregate quantities applied are shown in Table 4.

Performance

Immediately after construction, the pavement surface was in relatively good condition, except for the problems noted previously. One month later, there was virtually no change in the road surface characteristics.

DESIGN PROCEDURE MODIFICATIONS

Two parameters in the design procedure were altered as a result of a laboratory study (*20*) and the performance of the Eastland Test Road:

1. The final surface texture required for adequate skid resistance.
2. The minimum quantity of voids that must be filled to prevent initial stone loss.

The design procedure as developed by the NITRR (*15*) requires a texture depth of the final surface of 0.64 mm. Gallaway et al. (*18*) recommends that the texture depth not fall below 1.0 mm. Therefore, the required texture depth of the final surface was changed from 0.64 to 1.0 mm.

The NITRR design procedure recommended that immediately after construction the aggregate be covered at least halfway with binder to prevent initial stone loss due to whip off. In the case of a double seal, 65 percent of the voids would have to be filled with binder to ensure that the top layer of aggregate is covered at least halfway. Figure 8 (*9*) shows a theoretical distribution of the voids in a double seal.

On the basis of recommendations by Epps et al. (*11*) and on field reports by experienced engineers in the Texas Highway Department, the aggregate should be covered with binder approximately 30 percent (rather than 50 percent as recommended by the NITRR) immediately after construction

TABLE 4 DESIGN AND ACTUAL APPLICATION QUANTITIES FOR THE CIRCLEVILLE TEST ROAD

Layer	Aggregate Spread Rate, sy/cy		Residual Binder Application Rate, gsy	
	Design	Actual	Design	Actual
Grade 3	90	85	0.29	0.28
Grade 4	120	116	0.38	0.36

to prevent initial stone loss. Suggested depths at which the aggregate should be covered with binder are as follows (*11*).

- Immediately after construction, 30 ± 10 percent.
- Start of cool weather (first year), 35 ± 10 percent.
- Start of cold weather (first year), 40 ± 10 percent.
- After 2 years of service, 70 ± 10 percent.

On the basis of recommendations by Epps et al. (*11*) and on field experience, the quantity of voids that has to be filled with binder to ensure that the top layer of aggregate is adequately covered to prevent initial stone loss was decreased from 65 to 55 percent.

CONCLUSIONS

The key to executing an effective design for double chip seals is in the ability to measure the available void space in the stone layers that can be filled with binder. A design method developed by the NITRR (*9*), including a simple test procedure for measuring the void content and effective thickness of stone layers, was evaluated by the TTI. This innovative design method also provides for a way of predicting the loss of voids in the chip seal over its expected life. Double chip seal test roads built by TTI with the cooperation of the Texas State Department of Highways and Public Transportation by using the procedures described in this paper are performing well.

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